REPORT

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

Submitted to:

Dillon Consulting Ltd.
130 Dufferin Avenue
London, Ontario
N6A 5R2

Submitted by:

Golder Associates Ltd.
1931 Robertson Road Ottawa, Ontario, K2H 5B7 Canada

Latitude 45.114025
Longitude: -74.544989

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Table of Contents

PART A

Foundation Investigation Report
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with the replacement of the existing Fraser Road Bridge (Site No. 31-230) over Highway 401, which is located in the United Counties of Stormont, Dundas and Glengarry (SDG), Ontario (GWP 4248-15-00, WP 4290-15-01), as part of the Mega 10 Project (Purchase Order No. 4017-E-0019).

The purpose of this foundation investigation was to assess the subsurface conditions in the area of the proposed bridge and associated approach embankment areas and to provide information for the preliminary design of the new replacement bridge. The foundation investigation included drilling boreholes and installing one monitoring well, as well as carrying out in-situ testing (including packer testing, piezocone penetration tests, and geophysical shear wave velocity testing) and laboratory testing on selected soil and rock core samples.

The terms of reference for the original scope of work are outlined in the MTO's Work Item Order Form for Assignment 4, dated July 13, 2018. The terms of reference for the additional work are outlined in the MTO's Work Order Form for Assignment 5, dated August 30, 2018.

The work was carried out in accordance with Golder's Quality Control Plan dated April 2018.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 General

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 (see Key Plan in Drawing 1).

The new replacement bridge will be designed to accommodate the future widening of Highway 401 traffic. It is understood that both the existing alignment and an alignment shift of up to 12 m on both sides of the existing structure are being considered for the new bridge.

It is also understood that a grade change is required to accommodate the increased superstructure depth and to address the deficient vertical clearance, which is currently planned to be approximately 1 m.

A previous investigation was carried out in 1965 for the design of the original/existing bridge. The results of that investigation are contained in the following report:

- Report on "*Soil Conditions and Foundations, Proposed Fraser Road Underpass, Highway 401, Glengarry County, Ontario, WP 107-59 (Geocres 31G00-142)*", by H.Q. Golder & Associates, dated January 1966.

2.2 Regional Geological Conditions

As delineated in *The Physiography of Southern Ontario*[1], 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 have a significant role on the foundation design.

2.3 Existing Structure

The existing bridge currently carries two lanes of traffic of Fraser Road over the four-lane and median-divided Highway 401.

The bridge 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 and about 9 mm in thickness) and the piers founded on 12 BP 53 piles. The existing structure is aligned approximately northwest to southeast and is about 89.6 m long and 10.4 m wide. It is understood that the structure was built in 1968.

The natural ground surface is relatively flat at about Elevation 49 m north and south of Highway 401.

The existing bridge embankments are approximately 8 m in height above the natural ground level (i.e., the top of the abutments is at about elevation 57 m). The embankment side slopes are oriented at about 2 horizontal to 1 vertical (2H:1V). For stability reasons, the embankment fill was provided with front and side berms, about 4 m in height (i.e., the crest of the berms is at about elevation 53 m) and about 16 to 18 m in length. The front berms have forward slopes at 1.5H:1V, immediately adjacent to the existing piers, and the side berms have sides sloped at 2H:1V. Based on visual observation at the time of the site investigation, the existing embankment side slopes appear to be performing satisfactorily.

The existing embankment loading over the deep sensitive and compressible clay deposit has led to very large settlements of the approach embankments since the original construction. Based on the available documentation from MTO (Geocres numbered 31G00-192), settlement readings on the approach embankments within a few years following construction measured up to about 0.3 m of settlement at that time, which necessitated restoration of the approach pavement. The bridge itself has not settled as the structure is founded on deep foundations supported on bedrock.

Selected site photographs taken by Golder personnel showing the existing structure and surrounding area are included in Appendix D.

[1] 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.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the proposed underpass bridge was carried out between September 4 and 19, 2018. During that time, five boreholes (numbered 18-1101, 18-1102, 18-1103, 18-1103A, and 18-1103B) and two piezocone penetration tests (CPT) (numbered CPT 18-1101 and CPT 18-1103) were advanced at locations shown on Drawing 1.

The boreholes were advanced using 108 mm inside diameter (I.D.) continuous-flight hollow-stem augers on a truck or track mounted drill rig, supplied and operated by George Downing Estate Drilling Ltd. Of Hawkesbury, Ontario.

- Boreholes 18-1101, 18-1102, and 18-1103 were advanced at about the proposed foundation locations for the north abutment, central pier, and south abutment, respectively. These boreholes were advanced to depths of about 13.1 to 15.3 m below the existing ground surface in the overburden. Upon encountering split spoon or auger refusal, the boreholes were advanced into the bedrock to final depths of about 17.8 to 21.0 m (i.e., Elevation 31.9 to 32.8 m) using rotary diamond drilling techniques while retrieving HQ3 and NQ3 sized bedrock core. In addition, two relatively undisturbed 73 mm diameter thin-walled Shelby tube samples of the clay were retrieved from Boreholes 18-1101 and 18-1103 each, using a fixed piston sampler.
- Boreholes 18-1103A and 18-1103B were advanced immediately adjacent to 18-1103 to depths of about 11.3 m and 7.8 m (i.e., Elevation 41.8 and 45.1 m) for the installation of a monitoring well and to retrieve two relatively undisturbed 73 mm diameter thin-walled Shelby tube samples of the clay using a fixed piston sampler.

Soil samples in the boreholes were generally obtained at vertical intervals of about 0.60 and 0.76 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. In-situ vane testing, using an MTO “N”-size vane, was carried out to measure the undrained shear strength of the cohesive soils encountered at the site.

Packer testing of the bedrock was carried out in Borehole 18-1103 using a single pneumatic packer to estimate the hydraulic conductivity of the bedrock unit (as part of the additional scope of work). The details of the packer testing of the bedrock is discussed further in Section 4.9 below.

A water truck was on site to supply the drill rigs with water for advancing the casing in the overburden, coring the bedrock, and for carrying out the packer testing of the bedrock. Traffic control was provided for the duration of the field work for the centre pier in accordance with the Ontario Traffic Manual, Book 7, Temporary Conditions.

A PVC casing was installed and grouted into each of Boreholes 18-1101 and 18-1103 following completion of drilling to allow for subsequent Vertical Seismic Profiling (VSP) geophysical testing for seismic site characterization.

One monitoring well was installed in Borehole 18-1103A to monitor the groundwater level at the site. The well consists of 50 mm inside diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The water level in the monitoring well was measured on September 18, 2018.

An in-situ rising head slug test was carried out in the monitoring well sealed into Borehole 18-1103A on September 18, 2018 (as part of the additional scope of work). The details of the rising head test is discussed further in Section 4.9 below. The monitoring well may be useful in the future should additional investigations be required during the detailed design and is to be decommissioned at a later time.

The field investigation program also included two CPT's (numbered CPT 18-1101 and CPT 18-1103). The CPT's were carried out using portable CPT equipment supplied and operated by ConeTec Investigations Ltd. of Richmond Hill, Ontario. The CPT equipment was advanced using a track-mounted drill rig, supplied and operated by George Downing Estate Drilling Ltd. of Hawkesbury, Ontario.

- CPT 18-1101 and CPT 18-1103 were advanced immediately adjacent to Boreholes 18-1101 and 18-1103 within the proposed north and south approach embankments, respectively. In each CPT hole, the existing berm fill was augered through and piezocone was pushed starting from approximately the top of the silty sand layer (between about 3.6 and 3.8 m depths), through the inside of the hollow-stem augers, and using the loading head of the drill rig to advance the piezocone at a rate of about 2 cm per second. The tip resistance, shaft friction, and pore water pressure were measured at approximately 0.025 m depth intervals. The CPT holes were advanced until encountering practical refusal to piezocone advancement at depths of about 10.6 and 8.9 m below the existing ground surface at the locations of CPT 18-1101 and CPT 18-1103, respectively.

The boreholes and CPT holes were backfilled with bentonite pellets, mixed with native soils in the overburden and bentonite pellets in the bedrock, except as indicated previously for the boreholes with installations. The site conditions, with exception of the monitoring wells and VSP installations, were restored following completion of work.

The field work was supervised by a member of Golder's technical and engineering staff, who located the boreholes, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and bedrock samples.

The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratories in Ottawa and Mississauga 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 at the Golder Ottawa laboratory. Four consolidation tests were performed on selected Shelby tube samples from Boreholes 18-1101 and 18-1103B. Unconfined compression strength (UCS) testing was performed on three bedrock samples at Golder's Mississauga laboratory. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

Following drilling, the borehole and CPT hole locations were surveyed by Golder personnel using a Trimble R8 GPS unit. The borehole and CPT hole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole/CPT Depth (m)
18-1101	North Approach Embankment	4997670.5	222557.1	52.8	20.9
18-1102	Central Pier, Highway 401 Median	4997639.0	222573.6	50.6	17.8
18-1103	South Approach Embankment	4997614.4	222587.4	53.0	21.0
18-1103A		4997612.2	222588.7	53.1	11.3
18-1103B		4997611.2	222587.4	52.9	7.8
CPT 18-1101	North Approach Embankment	4997668.9	222554.5	52.8	10.6
CPT 18-1103	South Approach Embankment	4997610.3	222586.8	52.9	8.9

In addition to the borehole investigation, VSP testing was carried out within the PVC casings, grouted in place in Boreholes 18-1101 and 18-1103, on September 18, 2018 by Golder's geophysics personnel. Compression and shear wave seismic sources at about 2 m from the boreholes were used. The seismic source for compression wave test consisted of a 9.9 kg sledge hammer vertically impacted on a metal plate. The seismic source for the shear wave test consisted of a 2.4 m long, 150 mm by 150 mm wooden beam, secured by the weight of a vehicle and horizontally struck with a 9.9 kg sledge hammer on alternate ends of the beam to induce polarized shear waves.

4.0 SITE STRATIGRAPHY

4.1 General

As part of the subsurface investigation at this site, five boreholes and two CPT holes were advanced within the limits of the proposed bridge replacement. The borehole and CPT hole locations from the current and previous investigations at the site are shown on Drawing 1. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 1 are inferred from observations of drilling progress and from non-continuous 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.

The subsurface conditions encountered in the boreholes advanced during the current investigation are shown on the Record of Borehole in Appendix A. The results of the water content, Atterberg limit testing, grain size distribution and UCS testing obtained from Golder's laboratories are indicated on the Record of Borehole sheets.

The results of the laboratory testing carried out for the current investigation, including grain size distribution graphs, plasticity charts, oedometer consolidation and UCS testing results obtained at Golder laboratories, are presented on the Figures B1 to B13 in Appendix B. Photos of the bedrock core from the current investigation are presented on Figures B14 to B16 in Appendix B.

The Record of Borehole sheets and laboratory testing results from the previous investigations at the site are provided for reference in Appendix C. The CPT results including profiles of the tip resistance (q_t), sleeve friction (f_s), porewater pressure (u_2) during pushing and the corrected tip resistance (q_t) and sleeve friction (f_t) are presented on the ConeTec Investigation Ltd. Report in Appendix E. The VSP test results and report are presented in Appendix F and include the calculated shear wave velocity profile measured from the field testing and a graphical representation of the shear wave velocity profile with depth.

In general, the subsurface conditions at the site consist of surficial fill and/or topsoil, overlying a discontinuous layer of silty sand to sandy silt, underlain by compressible clay. The clay is in turn underlain by deposits of glacial till and/or sand and gravel, over limestone bedrock.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. In the following discussion, emphasis is placed on the subsurface conditions from the boreholes advanced during the current investigation, which are in general agreement with the Geocres information. The Geocres information is referenced herein only in regard to the clay parameters in Section 4.5 and the bedrock surface elevation in Section 4.8.

4.2 Fill

Borehole 18-1102 was advanced through the median left shoulder of the Highway 401 eastbound, adjacent to the existing central pier. The remaining boreholes were advanced through the existing side berm, immediately adjacent to the approach embankments.

At the granular shoulder, the fill is about 0.6 m thick and consists of gravelly sand to sandy gravel, underlain by gravelly sandy silt. At the side berm, the fill extends from ground surface to about 3.8 to 4.6 m depths (i.e., Elevation 48.3 and 49.0 m) and generally consists of silty sand, with varying amounts of gravel, and some cobbles and boulders.

SPT ‘N’ values obtained within the fill generally range from about 12 to 53 blows per 0.3 m of penetration, indicating a compact to very dense state of compactness.

The measured water contents of the fill ranges from approximately 6 to 18 percent. Grain size distribution testing was carried out on two samples of the fill, the results of which are provided on Figure B1. It should be noted that the samples were retrieved using a 50 mm diameter sampler and therefore the results do not reflect the larger gravel, cobble and boulder content of the fill.

4.3 Topsoil

Surficial topsoil fill exists at ground surface in all boreholes, with exception of 18-1102 (which was encountered within the granular shoulder of highway), and measures about 100 to 200 mm in thickness.

A buried layer of topsoil was encountered beneath the fill in Boreholes 18-1101 and 18-1102 at depths of about 3.8 and 0.6 m (i.e., Elevation 49.0 and 50.0 m) respectively. The thickness of the buried topsoil measures about 0.5 and 0.8 m at the borehole locations.

4.4 Silty Sand to Sandy Silt

In Boreholes 18-1101 and 18-1102, a discontinuous layer of silty sand to sandy silt was encountered below the topsoil at depths of about 4.3 and 1.4 m (i.e., Elevation 48.5 and 49.2 m), respectively. The silty and sandy layer encountered is about 0.4 m and 0.9 m thick.

One SPT ‘N’ value of 9 per 0.3 m of penetration was measured in the sandy silt layer, indicating a loose state of compactness

The measured water content on one sample of sandy silt measured approximately 28 percent. Grain size distribution testing was carried out on the same sample and the results are provided on Figure B2.

4.5 Clay

The surficial materials are underlain by a deposit of sensitive clay. The clay deposit was fully penetrated in all boreholes, except in 18-1103B, and extends to depths of about 7.0 to 9.8 m (i.e., Elevation 43.0 to 44.9 m), with thicknesses varying from about 3.6 to 5.1 m. In the previous boreholes, the clay thickness was recorded to vary from about 3.4 to 7.0 m (i.e., Elevation 41.4 to 44.9 m).

The upper portion of the clay has been weathered to form a grey brown crust. The thickness of the crust ranges from about 1.1 to 1.8 m and extends to depths of about 3.4 to 6.1 m (i.e., Elevation 46.6 and 47.3 m). The weathering was also noted in the previous boreholes and was indicated to extend to depths ranging from about 1.8 to 3.7 m (i.e., Elevation 46.7 and 47.5 m).

Standard penetration tests carried out within the weathered crust measured ‘N’ values ranging from ‘Weight of Hammer’ to about 6 blows per 0.3 m of penetration. The weathered silty clay is considered to have a stiff to very stiff consistency.

The results of Atterberg limit testing carried out on one sample of the weathered clay is shown on Figure B3 and indicates a plasticity index value of 57 percent and liquid limit value of 83 percent, reflecting a clay of high plasticity. The measured water content of the weathered clay ranges from approximately 52 to 78 percent.

Standard penetration tests carried out within the unweathered portion of the deposit (below the crust) measured ‘N’ values ranging from ‘Weight of Hammer’ to about 3 blows per 0.3 m of penetration. In situ shear vane testing carried out within this deposit measured undrained shear strengths ranging from about 23 to 56 kPa, indicating a soft to stiff consistency, however the deposit was generally found to be in firm consistency. The measured in-situ remoulded strengths in the clayey deposit ranged from about 3 to 10 kPa, with sensitivity varying from about 4 to 11, but more generally from 4 to 7, indicating a soil of medium sensitivity to sensitive (CFEM, 2006).

The results of Atterberg limit testing carried out on six samples of the unweathered clay are shown on Figure B4 and measured plasticity index values ranging from about 44 to 56 percent and liquid limit values ranging from about 66 to 82 percent, respectively, indicating a high plasticity clay. The measured water contents of the unweathered portion of the deposit were between about 54 to 94 percent.

Oedometer consolidation testing (including both incremental and long-term loading) was carried out on four samples of clay, the results of which are provided on Figures B5 to B8. The load increments for the consolidation testing was selected based on the measured undrained strength and anticipated preconsolidation pressure of the soil samples.

The available consolidation test results are summarized in the table below and indicate that the clay is normally consolidated to slightly overconsolidated, with preconsolidation pressures ranging from about 110 to 180 kPa and overconsolidation ratio from 1.0 to 1.3.

Borehole/ Sample Number	Type of Test	Sample Depth/ Elevation (m)	Unit Weight (kN/m^3)	σ_P' (kP)	σ_{VO}' (kP)	σ_P' - σ_{vo}' (kPa)	Cc	Cr	e_o	OCR
18-1101 / 9	IL	6.6 / 46.2	14.9	110	110	-	2.29	0.026	2.42	1.0
18-1101 / 10	IL	7.9 / 44.9	15.2	140	120	20	2.20	0.016	2.19	1.2
18-1101 / 10	LT	7.9 / 44.9	15.1	-	120	-	NA	0.017	2.28	-
18-1103B / 1	IL	7.6 / 45.4	15.6	180	135	45	1.60	0.019	2.02	1.3

Notes:

σ_P' Apparent preconsolidation pressure
σ_{VO}' Computed existing vertical effective stress
C_c Compression index
C_r Recompression index
e_o Initial void ratio
OCR Overconsolidation ratio
IL Incremental loading oedometer consolidation test
LT Long-term oedometer consolidation test

A summary of engineering properties for the clay deposit is provided on Figure B13, which includes the parameters calculated/measured within the clay during both the current and past Geocres investigations.

4.5.1 CPT Results

The undrained shear strength profile of the clay has been evaluated based on the results of the piezocone testing program, using the following equation:

$$Su = (q_t - \sigma_{VO}) / N_K$$

Where:

Su = Calculated undrained shear strength (kPa);

q_t = Measured net tip resistance (kPa);

σ_{VO} = Calculated total vertical stress (kPa); and,

N_K = Correlation factor, selected by ConeTec.

The undrained shear strength profiles for the clay determined from the results of the piezocone testing, as described above, are summarized in Appendix E.

Based on the estimates from the CPT results, the undrained shear strength of the clay decreases steadily with depth from 70 kPa at the top of the weathered crust, generally reaching about 31 to 36 kPa at the bottom of the crust. The CPT test results also indicate undrained shear strengths ranging from about 42 to 27 kPa over the depth of unweathered clay.

The CPT results have also been interpreted and calibrated against the laboratory consolidation test results to provide a profile of the preconsolidation pressure with elevation, as shown on Figure B13 in Appendix B. The method used to process the data is suggested by Demers and Leroueil (2002) for Champlain Sea clay, with:

$\sigma_P' = (q_t - \sigma_{VO}) / N_{\sigma t}$ Where:

σ_P' = Calculated preconsolidation pressure (kPa);

q_t = Measured net tip resistance (kPa);

σ_{VO} = Calculated total vertical stress (kPa); and,

$N_{\sigma t}$ = Correlation factor, selected as 3.7 based on Bjerrum (1975) correlation.

As can be seen on Figure B13, similar preconsolidation pressures were recorded for both CPT holes at the north and south approach embankments. The results from the CPT indicate that the preconsolidation pressure of the clay decreases steadily with depth from about 260 kPa at the top of the weathered portion of the deposit, generally reaching about 110 kPa at Elevation 47 m. Below that elevation, the preconsolidation pressure of the unweathered clay consistently increases with the existing overburden effective stress, since the clay is effectively normally consolidated over most of its thickness due to the existing embankment loading (which generally exceeded the preconsolidation pressure prior to construction of the embankments).

4.6 Silt and Sand, Silty Sand, Sand and Gravel to Sandy Gravel (Till)

A deposit of glacial till was encountered directly beneath the clay in the current boreholes, except in Borehole 18-1103B, where the clay was not fully penetrated. The till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silt and sand, silty sand, sandy gravel to sand and gravel.

The till was fully penetrated at Boreholes 18-1101 to 18-1103, inclusive, and is about 3.9 to 6.9 m in thickness, extending to depths of about 13.1 to 15.3 m (i.e., Elevation 37.5 to 39.1 m). The till was not fully penetrated at Borehole 18-1103A but was proven to a depth of at least about 11.3 m (i.e., Elevation 41.8 m).

Standard penetration test ‘N’ values of 3 to 118 blows per 0.3 m of penetration were measured in the glacial till, indicating a very loose to very dense state of compactness, generally increasing with depth. The higher ‘N’ values could reflect the presence of cobbles and boulders, rather than the state of compactness of the soil matrix. More generally, the till was found to be compact to dense.

The measured water contents of samples of till were between about 7 and 14 percent. Grain size distribution testing was carried out on four samples of the till, the results of which are provided on Figures B9 and B10. It should be noted the samples were retrieved using a 50 mm diameter sampler and therefore the results do not reflect the larger gravel, cobble and boulder content of the deposit.

4.7 Sand and Gravel

Deposits of sand and gravel were encountered within the glacial till layer in Boreholes 18-1102 and 18-1103, at depths of about 8.4 and 10.7 m (i.e., Elevations 42.2 and 42.3 m). The sand and gravel deposit is about 0.6 m in thickness.

Standard penetration test ‘N’ values of 13 and 47 blows per 0.3 m of penetration were measured in the deposit, indicating a compact to dense state of compactness.

The measured water contents of two samples of sand and gravel were between about 10 and 11 percent. Grain size distribution testing was carried out on two samples of the sand and gravel, the results of which are provided on Figure B11.

4.8 Bedrock

Bedrock was encountered beneath the till deposits in Boreholes 18-1101 to 18-1103, inclusive, at depths ranging from about 13.1 to 15.3 m (i.e., Elevations 39.1 to 37.5 m). The bedrock was cored to depths of between about 4.7 and 7.2 m below the bedrock surface using HQ3 or NQ3 sized drill bits and rods. Photos of the bedrock core are shown on Figures B14 to B16 in Appendix B.

The following table summarizes the bedrock surface or refusal depths and elevations as encountered at the borehole locations during the current and previous Geocres investigations at the site. Only the previous bedrock surface information where bedrock was proven by coring is included.

Borehole Number	Borehole Location with respect to Bridge Structure	Existing Ground Surface Elevation (m)	Depth to Bedrock/Refusal (m)	Bedrock Surface/Refusal Elevation (m)
18-1101	North Abutment	52.8	13.7	39.1
18-1102	Central Pier, Highway 401 Median	50.6	13.1	37.5
18-1103	South Abutment	53.0	15.3	37.7
BH 1	South Abutment	49.0	11.4	37.6
BH 2	South Approach Embankment	50.6	12.4	38.2
BH 3	Central Pier, Highway 401 Median	49.6	13.1	36.5
BH 4	North Approach Embankment	50.6	10.8	39.8
BH 5	North Abutment	49.1	11.2	37.9

The bedrock encountered in the boreholes consist of fresh, thinly bedded, dark grey to black, fine grained limestone with occasional shale interbeds. The Rock Quality Designation (RQD) values measured on recovered bedrock core samples during the current investigation ranged widely from about 10 to 100 percent, however generally over 50 percent, indicating fair to excellent quality rock.

The results of unconfined compressive strength testing carried out on three bedrock core samples ranged from about 22 to 37 MPa, as shown on Figure B12 and the results of UCS testing on selected samples of the bedrock are provided in Appendix B. The results of the UCS testing indicate a weak to medium strong rock.

A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.9 Groundwater Conditions

A monitoring well was installed within glacial till in Borehole 18-1103A. The static water level measured in the monitoring well is noted in the following table:

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
18-1103A	53.1	6.1	47.0	Sept. 18, 2018

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt and is expected to be higher during the spring and periods of precipitation.

Packer testing of the bedrock was carried out in Borehole 18-1103, including both falling head and constant head packer tests, on September 11 and 12, 2018. The downhole testing equipment consisted of a single pneumatic packer lowered through the drilling casing using AQ-sized rods. The packer was subsequently inflated with nitrogen gas to isolate the test interval.

The falling head testing was carried out by quickly adding a known volume of water to the test interval, through the rods that extend from the tested interval to the surface, and then monitoring the subsequent decrease in water level in the rods over time. The falling head tests were performed open to atmospheric pressure. For the constant head test, the Lugeon methodology was followed by pumping water into the borehole at a number of increasing and decreasing constant pressure values and recording the resulting flow rate (at each pressure value) into the interval. For both types of tests, a pressure transducer and datalogger was placed within the test interval to monitor and record the real-time pressure responses during testing.

The data obtained from the falling head tests were analyzed using Hvorslev's (1951) method to estimate the hydraulic conductivity of the rock mass interval. The results of the constant head test were analyzed using the Thiem equation (Thiem, 1906) in accordance with the Lugeon method.

Testing was completed on two intervals in the bedrock at Borehole 18-1103. Both a falling head test and a constant head test were completed on the first interval, located from approximately 17.1 to 21.0 m below ground surface. After field observations of a mud seam (and loss of drill water) at approximately 18.2 m depth, a second interval was hydraulically tested below the mud seam. Only a falling head test was completed on the second interval, at approximately 18.6 to 21.0 m below ground surface.

The results from the packer testing of bedrock are summarized in the following table:

Strata of Test Interval	Type of Test	Test Interval Below Ground Surface (m)	Estimated Hydraulic Conductivity (cm/s)
Bedrock	Falling Head	17.1 – 21.0	2 x 10^{-3}
	Constant Head	17.1 – 21.0	1 x 10^{-3}
	Falling Head	18.6 – 21.0	2 x 10^{-4}

An in-situ rising head slug test was carried out in the monitoring well sealed into Borehole 18-1103A on September 18, 2018. The screened interval of the monitoring well was installed within the gravelly silty sand unit (glacial till). The rising head test involved quickly removing a known volume of water from the monitoring well and monitoring the subsequent increase in water level in the monitoring well over time.

The data obtained from the rising head test were analyzed using the Hvorslev (1951) method to estimate the hydraulic conductivity of the glacial till. A summary of the hydraulic conductivity slug test result is presented in the following table:

Strata of Test Interval	Test Interval Below Ground Surface (m)	Estimated Hydraulic Conductivity (cm/s)
Glacial Till	9.2 – 10.7	7 x 10^{-3}

4.10 Results of Chemical Analysis

Three soil samples, one from each of Boreholes 18-1101 to 18-1103, were submitted to Eurofins Environment Testing for chemical analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The results of the testing are provided in Appendix G and are summarized in the table below.

Borehole No.	Sample Depth (m)	Sample Type	Chloride (%)	pH	Electrical Conductivity (mS/cm)	Resistivity (ohm-cm)	Sulphate (µg/g)	Sulphide (µg/g)
18-1101 / 5	3.1 – 3.7	Fill	0.087	8.11	0.54	1850	360	3.1
18-1102 / 5	3.1 – 3.7	Clay	0.049	8.13	1.00	1000	30	<0.2
18-1103 / 11	7.6 – 8.2	Clay	0.006	8.04	0.46	2220	240	<0.2

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Christine Ko, P.Eng., and reviewed by Mr. Bill Cavers, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Fin Heffernan, P.Eng., the MTO Foundations Designated Contact for this assignment, conducted an independent quality review of this report.

Golder Associates Ltd.

Christine Ko, P.Eng.
Geotechnical Engineer

William Cavers, P.Eng.
Associate, Senior Foundations Engineer

Fin Heffernan, P.Eng.
MTO Foundations Designated Contact

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PART B

Foundation Design Report
Preliminary Design of Replacement of Fraser Road Underpass
Highway 401, Site 31-230
United Counties of Stormont, Dundas and Glengarry, Ontario
GWP 4248-15-00, WP 4290-15-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing Fraser Road Underpass Bridge (Site No. 31-230) over Highway 401 in the United Counties of Stormont, Dundas and Glengarry (SDG), Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes and piezocone tests advanced during the current subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the foundations for the replacement structure. It is understood that the bridge is to be designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (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.

Where comments are made on construction, they are provided to highlight those aspects that could affect the preliminary or detailed design of the project, and for which special provisions may be required in the contract documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing bridge is shown in plan on Drawing 1 and consists of a two-lane, four-span prestressed precast concrete structures, with the abutments founded on pipe (tube) piles (0.6 m outside diameter and about 9 mm in wall thickness) and the piers founded on 12 BP 53 piles. The existing structure is aligned approximately 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 (i.e., the top of the abutments is at about elevation 57 m). The embankment side slopes are oriented at about 2 horizontal to 1 vertical (2H:1V). For stability reasons, the embankments were provided with front and side berms, about 4 m in height (i.e., the crest of the berms is at about elevation 53 m) and about 16 to 18 m in length. The front berms have forward slopes at about 1.5:1V, immediately adjacent to the existing piers, and the side berms have side slopes at about 2H:1V.

The existing embankment loading over the thick sensitive and compressible clay deposit has resulted in large settlements of the embankments since the original construction in 1968. Based on available MTO documentation (Geocres reports numbered 31G00-142 31G00-192), significant settlement of the approach fills in the order of 0.6 m were predicted, with the majority of the settlement anticipated to occur in the first 2 to 3 years. Settlement readings taken on the approach embankment pavement a few years following the construction measured up to about 0.3 m of settlement at that time, which necessitated restoration of approach pavement in 1971.

It is understood that the existing structure is to be replaced by a new bridge to accommodate the future widening of Highway 401.

Various structure replacement alternatives are being considered as part of the preliminary design for this project. It is understood that both the existing alignment as well as an alignment shift of up to 12 m on both sides of the existing structure are being considered for the new bridge. It is also understood that a grade change is required to accommodate the deeper superstructure and to address the deficient vertical clearance such that the proposed pavement grades at the new structure will be approximately 1 m higher than the existing pavement grades.

Based on the discussions and information provided by Dillon, the alternatives currently being considered can be summarized as the following for foundation considerations:

- **Shorter span on the existing alignment**: The new bridge will be a two-span structure located on the same alignment as the existing bridge, with the new abutments between the existing piers and existing abutments (i.e., about 33 m from the central pier);
- **Longer span on the existing alignment**: The new bridge will be a four-span structure and the current overall structure length will be maintained, with the new abutments at approximately the same location as the existing abutments (i.e., about 45 m from the central pier). Reuse of the existing foundation elements is also being considered for this alternative; and,
- **New structure on a new alignment**: The new bridge will be located along a new alignment up to 12 m away from (either east or west of) the existing bridge. The approach embankments are expected to be founded within the footprint of the existing side berms (which are about 18 m in length). Both shorter and longer spans are considered for this alternative.

As previously noted, in order to address the current deficient vertical clearance, the proposed pavement grades at the new structure will need to be increased by approximately 1 m higher from the existing pavement grades. Therefore, no profile grade increase is not considered a feasible option.

For the purposes of the geotechnical investigations, and based on initial discussion with MTO, the boreholes completed as part of this investigation were positioned assuming the preferred alternative is to replace the existing bridge on the same existing alignment and to minimize the overall structure length, while accommodating the future widening of Highway 401.

6.2 Seismic Design

6.2.1 Site Seismicity and Importance Category

The site falls within the Western Québec Seismic Zone (WQSZ) according to the Geological Survey of Canada (GSC). The WQSZ constitutes a large area that extends from Montréal to Témiscaming. Within the WQSZ, recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the WQSZ includes the 1935 Témiscaming event, which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2 and the 1944 Cornwall-Massena event which had a magnitude of 5.6. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

The CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the GSC. The GSC has developed a new set of seismic hazard maps (referred to as the 5th generation seismic hazard maps) that were made available for public use in December 2015.

It is understood that the proposed bridge structure has an importance category of *other* bridge in accordance with Section 4.4.2 of the CHBDC.

6.2.2 Seismic Site Classification

Analysis of VSP geophysical testing was carried out at two locations, immediately west of the existing north and south embankments respectively, to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site. The shear wave velocities measured are presented in a technical memorandum (see Appendix F) and indicate that the average shear wave velocity in the upper 30 m of the subsurface stratigraphy is 404 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.

6.2.3 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 bridge (latitude 45.114 and longitude -74.544), the following are the reference Site Class C peak seismic hazard values based on the 5th generation seismic hazard maps published by the GSC. Since this site is assigned a Site Class of C as noted above, the seismic hazard values given in the table below can be used for preliminary design purposes.

Seismic Hazard Values for Reference Ground Condition Site Class C

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.379
PGV (m/s)	0.258
S (0.2) (g)	0.596
S (0.5) (g)	0.313
S (1.0) (g)	0.150
S (2.0) (g)	0.069
S (5.0) (g)	0.018
S (10.0) (g)	0.006

Fundamental period of the structure is expected to be greater than 0.5 s, which, in consideration of its *other* importance category and the site-specific seismic hazard values given above, would indicate that the bridge structure falls in Seismic Performance Category 2 in accordance with Table 4.10 of the CHBDC. Based on this Seismic Performance Category and the *regular* geometry of the bridge, the structure would be designed using a “force-based approach” as defined in the CHBDC.

6.2.4 Liquefaction Potential

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface settlements) and under undrained conditions generate excess pore pressures. The excess pore pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

Where the calculated shear stress is greater than the shear resistance, liquefaction of the soil with an associated significant strength loss is predicted to occur. This methodology considers that the soil behaves as a “sand-like” material and is applicable to assessment of liquefaction of cohesionless soils.

Post-seismic strength loss may also occur as a result of similar but different cyclic mechanisms. Cohesionless soils are also susceptible to cyclic mobility which, in contrast to liquefaction, can still occur when the static shear stress is less than the shear resistance of the soil. The deformations associated with cyclic mobility failure develop incrementally during the earthquake event. Further, soils that are predominantly fine-grained typically do not respond with liquefaction or cyclic mobility, but they can experience strength reduction as a result of prolonged shaking knows as cyclic softening.

The liquefaction potential at the site was initially assessed using the approach outlined in the CHBDC (based on work by Idriss and Boulanger, 2008), which is appropriate for granular soil deposits that will behave as a “sand-like” material and involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil. The results of these liquefaction analyses indicated a potential for liquefaction within discrete portions of the upper portion of the glacial till at the site.

Further interpretation of the results of the CPTs put down at the site (which penetrated through the upper portion of the glacial till) suggests that the deposit will generally exhibit a more “clay-like” behaviour:

- Relatively high measured and interpreted fines content within the upper portion of the glacial till (between about 25 and 45 percent, see Figures B9 and B10 in Appendix B) suggest clay-like behaviour; and,
- The Soil Behaviour Type Index (Ic) for much of the upper portion of the glacial till is above the accepted boundary of clay-like behaviour (Ic = 2.6).

In addition, the previous liquefaction analyses did not explicitly consider the aging of this glacial deposit. Table C4.4 in the Commentary to the CHBDC suggests that glacial till deposits greater than 500 years old generally have a “very low” liquefaction potential.

Based on the above, the glacial till is not expected to behave as a sand-like material and is considered to have a low potential for flow liquefaction during the design seismic event.

The factor of safety against cyclic softening of the clay deposit at the site was also assessed based on the guidance provided in Idriss and Boulanger (2008), in which the CRR for clay-like soil is calculated based on the undrained shear strength and approximate OCR of the soil. The CRR is equated with the CSR (for reference stress equal to 65 percent of peak shear stress) to calculate the factor of safety against cyclic softening that would be expected to result in greater than 3 percent shear strain. Based on the results of the analyses, the clay at this site is considered to have a low potential for cyclic softening.

The results of the analyses described above indicate that the soils at this site may be considered as non-liquefiable and not susceptible to cyclic softening for preliminary design. However, this should be confirmed during detailed design based on the preferred alignment.

6.3 Bridge Foundations

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

A summary of the advantages and disadvantages associated with each deep foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, constructability and relative costs is provided in Table 1 following the text of this report.

- **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.
- **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 to steel H-piles in terms of high geotechnical resistances and minimal settlements. This option may also permit the use of integral abutments. 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.
- **Caissons:** Caissons deriving their support from bearing within the limestone bedrock are also feasible for this site. Caissons would require the use of temporary or permanent liners to mitigate the potential risks of ground loss from potential flowing clay or water-bearing cohesionless layers below the clay during construction. In addition, the caissons could be socketed at least nominally into the bedrock to permit cleaning of the caisson bases, and such sockets could be advanced by rock coring and/or chisel drilling into the weak to medium strong limestone bedrock. This foundation option is considered feasible at the pier.
- **Reuse of existing piles:** The existing structure is supported by abutments founded on concrete-filled pipe (tube) piles (0.6 m outside diameter and about 9 mm in thickness) and the piers are founded on steel H-piles (12 BP 53). Reusing the existing foundation elements may be considered if the new structure will be constructed on the same existing alignment and the current overall structure length will be maintained (i.e., the new abutments and central pier will be at approximately the same location as the existing foundations). However, additional testing would be required to provide recommendations for reusing the existing pier piles and for compatibility with the new piles (see Section 6.6).

Other foundation options such as Rock Socketed Steel Pipe (Tube) Piles, Micropiles and Continuous Flight Auger Piles (CFA) are typically considered when driven steel H-piles, driven steel pipe pies or caissons are not feasible, which is not the case at this site, and therefore have not been considered for the new structure.

Based on the above considerations, the preferred options from a geotechnical/foundations perspective is to support the abutments and central pier on steel H-piles driven to found on the bedrock for the proposed bridge replacement.

6.3.1 Feasibility of Integral Abutments

As outlined in MTO's report SO-96-01, 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 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 35º; and abutment wall heights less than 6 m without a retained soil system.

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

6.3.2 Downdrag Load (Negative Skin Friction)

The placement of granular embankment fill would raise the effective stress level in the clay deposit, leading to some consolidation of the deposit. As discussed previously, this condition would result in downdrag forces on driven piles or caissons supporting the abutments. Since there is no grade raise proposed at the central pier, no downdrag forces are anticipated on deep foundations at the pier location.

It is our understanding that downdrag loading was not included in the original design of the existing bridge, based on discussions with Golder personnel familiar with the original investigation for the existing structure. There is currently a separate MTO study on a different site to determine if downdrag forces from an original construction still acts on the existing piles. The study is currently ongoing and therefore we are not able to comment on the magnitude of the downdrag force on the existing piles, if any, at this time.

6.3.3 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the proposed underpass structure and foundation system may be classified as having low traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a "typical" consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in Sections 6.4 to 6.7 below.

6.4 Driven Steel H-Pile or Driven Steel Pipe (Tube) Pile Foundations

6.4.1 Founding Elevations and Pile Driving

The abutments and central pier for the replacement bridge may be supported on steel H-piles or steel pipe piles driven to found on the limestone bedrock. Based on the geotechnical investigations carried out at the site, the following pile tip elevations are recommended for design of piles:

Foundation Element	Borehole Number	Bedrock Surface / Pile Tip Elevation
North Abutment	18-1101, BH 5	37.9 - 39.1
Central pier	18-1102, BH 3	36.5 - 37.5
South Abutment	18-1103, BH 1	37.6 - 37.7

The pile caps should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario)*.

Based on the results of the field investigations, occasional cobbles and boulders are expected in the till deposits. Therefore, each pile should be reinforced at the tip with suitable driving points (such as Titus Standard 'H' Points for H-piles or Titus Open Cutting Shoe for pipe piles, or equivalent) to reduce the potential for damage to the piles during driving through soils that may contain boulders, in accordance with OPSS.PROV 903 *(Deep Foundations).* For steel pipe piles the driving shoes should be in accordance with OPSD 3001.100 Type II *(Steel Tube Pile Driving Shoe).*

If the new structure is to be constructed on the existing alignment, it is expected that the existing bridge will be removed prior to commencement of construction, and vibration monitoring of the existing bridge should not be required during pile installation. However, if a new alignment is considered and the existing bridge is to remain operational during construction, vibration monitoring will likely be required during foundation excavation and pile driving adjacent to the existing structure. Pre- and post-condition survey of the existing bridge is not considered necessary as the structure is being replaced.

For the shorter span alternative on the existing alignment, based on the currently proposed new abutment locations (i.e., the new abutments being about 33 m from the central pier), the piles for the new abutments are not expected to be in conflict with the vertical or battered piles supporting the existing structure. Therefore, the foundation elements at the existing piers (No. 1 and 3) and the existing abutments should be able to remain in place. The existing piles at the central pier (No. 2) may also remain in place, from a geotechnical prospective, provided that the new piles could be installed without interference with the existing pile group, based on the structural design. Otherwise, the existing piles at central pier may have to be removed prior to construction of the new pier foundations.

Similarly, for the longer span alternative on the existing alignment or a new structure on a new alignment, the existing foundation elements may remain in place from a geotechnical prospective, provided that the new piles could be driven without interference with the existing pile group. Consideration could also be given to reusing of the existing piles for this alternative. However, additional testing would be required to provide recommendations for reusing existing piles and for compatibility with the new piles, which is further discussed in Section 6.6.

6.4.2 Axial Geotechnical Resistance

Based on the measured uniaxial compressive strength of the rock at this site and the rock quality, for HP 310x110 piles, the axial factored ultimate geotechnical resistance (ULS) will be 3,200 kN. The axial factored ultimate geotechnical resistance for 324 mm diameter pipe piles will be 2,800 kN. The factored ULS 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. The serviceability geotechnical resistance (SLS) does not apply to piles founded on the bedrock at this site, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Pile installation should be in accordance with OPSS.PROV 903 *(Deep Foundations).* The drawings should incorporate the appropriate note stating that the piles should be equipped with suitable driving points and should be driven to bedrock. For piles driven to refusal on bedrock, and as described in OPSS.PROV 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

The pile termination or set criteria for H-piles will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known.

6.4.3 Downdrag Load (Negative Skin Friction)

Since there is no grade raise proposed at the central pier, no downdrag forces are anticipated on piles supporting the pier.

The placement of earth fill for the new embankments, over the existing berms and embankments, however, will raise the effective stress level in the clay deposit which underlies the site. This increase in stress will lead to elevated settlement of the underlying clay deposit, as well as in any soil above the clay layers, and corresponding downdrag loads on the piles at the abutments, which will in turn reduce the available capacity of the piles.

The magnitude of settlements needed to cause negative skin friction is small (i.e., about 10 mm or more). Such small relative movements occur easily as a result of the large stiffness difference between the pile and the clay soil. Therefore, the magnitude of downdrag loads is independent of that fact whether the clay has previously experienced any excessive settlements as a result of increase in stresses (caused by factors such as applied loads or groundwater dewatering).

The downdrag loads (or negative skin friction) will need to be taken into account during the design of the piles supporting the bridge abutments.

The downdrag loads could vary depending on the selected embankment fill material, on the sequence of construction, and on the underside of pile cap elevation.

As discussed further in Section 6.7 below, the clay deposit below the existing berms and roadway embankments are most likely now normally consolidated (i.e., any additional load will result in overstressing of the clay and significant settlements). Since the maximum height of the new embankment fill at the new abutments will be about 9 m, the resulting downward movement of the clay around the piles, as well as in any earth fill above the clay layers will induce downdrag forces on the piles through negative skin friction. The magnitude of the downdrag forces is expected to be high due to the thickness of new and existing fills above the clay layer and the low undrained shear strength of the underlying clay deposit.

In addition, based on discussions with Dillon, grade raises of 0.5 to 1 m are currently being considered and therefore the associated downdrag values are estimated. Various methods have been used in calculating the magnitude of the downdrag force, including the Nordlund method, 1979 and β-Method in cohesionless, and α-Method in cohesive soil described in the Canadian Foundation Engineering Manual (CFEM), in order to refine the estimation.

Based on the results of the analyses, for a 1 m of grade raise and assuming an underside of the pile cap of about Elevation 56 m, the unfactored downdrag load acting on a single HP 310x110 pile, over the length of pile is estimated to be about 800 kN. For a 0.5 m of grade raise and assuming an underside of the pile cap of about Elevation 55.5 m, the unfactored downdrag load acting on a single HP 310x110 pile, over the length of pile is estimated to be about 750 kN. The unfactored downdrag load acting on a single 324 mm diameter steel pipe pile, if considered, would be slightly lower.

The estimated downdrag load is the same for both shorter and longer span alternatives. The downdrag forces are dependent on the undrained strength of the underlying clay soils and the compactness of the granular fill above the clay soils. The distribution of the downdrag forces is non-linear from underside of pile cap to the bottom of clay layers.

If a new alignment adjacent to the existing structure is considered and the existing bridge is to remain operational during construction, settlement of the existing embankments (due to the loading from the additional fill for the new embankments) will result in downdrag loads on the existing piles. The magnitude of the downdrag loads acting on the existing piles may be similar in magnitude to the downdrag loads indicated above for new piles.

If the settlements are mitigated as discussed further in Section 6.8, the downdrag loads at the abutments (for both new and existing piles) would be greatly reduced. If EPS lightweight fill is used to construct the new embankments (such that the net load increase is negligible), particularly in the area of influence of the abutments, no downdrag loads would be expected on the piles.

The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.11.4.10 of the CHBDC.

6.4.4 Lateral Geotechnical Resistance

To accommodate the movements associated with integral abutments, a sand-filled corrugated steel pipe (CSP), 0.6 m in diameter and 3 m in length, is typically provided extending below the underside of the pile cap.

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. Alternatively, the resistance to lateral loading can be derived from the soil in front of the piles, and it may be assumed that this resistance will be nearly the same for vertical and inclined piles as indicated in Section C6.11.2.2 of the Commentary to the CHBDC.

The SLS geotechnical response 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 equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).

For cohesionless soils:

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

Where: n_h is the constant of horizontal subgrade reaction, as given below;

z is the depth (m); and,

B is the pile diameter/width (m).

For cohesive soils:

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

Where: su is the undrained shear strength of the soil (kPa); and,

B is the pile diameter/width (m).

The following ranges for the values of n_h and s_u may be used in the structural analysis. The ranges in values reflect:

- The variability in the subsurface conditions and the soil properties;
- The approximate nature of the analysis;
- The non-linear nature of the soil behaviour (such that n_h is a function of deflection); and,
- The two extremes of the design; the requirement for flexibility in the case of integral abutments and the requirement for lateral resistance of horizontal loads.

Location	Elevation (m)	Soil Type	n_h (MN/m^3)	s_u (kPa)
North Abutment	49.0 – PCL[1]	Compact to Dense Silty Sand and Gravel (Fill)	6 to 15	-
	48.1 – 49.0	Loose to Compact Silty Sand	1 to 6	-
	46.7 – 48.1	Stiff to Very Stiff Weathered Clay Crust	-	50 kPa
	43.0 – 46.7	Firm Clay	-	30 to 40 kPa
	41.5 – 43.0	Loose to Compact Gravelly Silty Sand to Sand and Gravel (Till)	3 to 5	-
	39.1 – 41.5	Dense to Very Dense Sand and Gravel (Till)	8 to 15	-
	39.1	Bedrock	-	-
Central Pier	48.3 – PCL[1]	Loose Sandy Silt	1 to 3	-
	47.3 – 48.3	Stiff Weathered Clay Crust	-	50 kPa
	43.6 – 47.3	Soft to Firm Clay	-	25 to 30 kPa
	42.2 – 43.6	Compact Silt and Sand to Silty Sand (Till)	3 to 5	-
	41.6 – 42.2	Dense Sand and Gravel	6 to 11	-
	40.8 – 41.6	Very Dense Gravelly Silty Sand (Till)	8 to 15	-
	37.5 – 40.8	Loose to Dense Gravelly Silty Sand to Sand and Gravel (Till)	3 to 11	-
	37.5	Bedrock	-	-

Location	Elevation (m)	Soil Type	n_h (MN/m^3)	S_u (kPa)
South Abutment	48.4 – PCL[1]	Compact to Very Dense Silty Sand (Fill)	6 to 15	-
	46.9 – 48.4	Stiff to Very Stiff Weathered Clay Crust	-	50 kPa
	44.6 – 46.9	Firm to Stiff Clay	-	30 to 50 kPa
	42.3 – 44.6	Very Loose to Loose Compact Gravelly Silty Sand (Till)	1 to 3	-
	41.7 – 42.3	Compact Sand and Gravel	3 to 6	-
	37.7 – 41.7	Compact to Dense Sand and Gravel (Till)	4 to 11	-
	37.7	Bedrock	-	-

Note:[1] PCL = Pile Cap Level

The values of n_h and S_u provided above may be used for preliminary design but for detailed design, non-linear p-y curves should be used to model the soil-structure interaction and/or to refine the values above for a given range of anticipated lateral deflections.

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory outlined in Section C6.11.2.2.1 of the Commentary to the CHBDC.

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. Theses “group effects” can be incorporated into the design 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) for a range of pile spacings are provided in Section C6.11.3.4 of CHBDC.

As previously mentioned, the existing foundation elements may remain in place from a geotechnical prospective, provided that the new piles could be driven without interference with the existing pile group (i.e., without contacting the existing piles) based on the structural design. However, if the existing piles interfere with the new foundations and/or cannot be reused from a structural prospective, they may require removal prior to construction of the new foundations. It should be feasible to extract the concrete filled tube piles at the existing abutments and the 12 BP 53 piles at the existing piers, if required.

For calculation of the ULS resistances, a geotechnical resistance factor of 0.5 in accordance with the CHBDC is to be applied in calculating the horizontal resistance.

6.5 Caisson Foundations

6.5.1 Founding Elevations and Caisson Installation

The central pier may alternatively be supported by caisson foundations.

For design purposes, the following bedrock surface elevations should be considered:

Foundation Element	Borehole Number	Bedrock Surface Elevation (m)
Central Pier	18-1102, BH 3	36.5 - 37.5

The native marine (Champlain Sea) clay at this site is a sensitive soil. The disturbed clay could "flow" into the auger hole during drilled shaft installation if left unsupported. Furthermore, there are water-bearing cohesionless layers within the glacial till deposits. The use of a temporary or permanent liner or casing will therefore be required in order to advance the drilled shafts with minimal loss of ground. Casing installation through the bouldery glacial till deposits may be difficult. Churn drilling techniques may be required.

Additionally, it will be difficult to clean the bedrock surface, even with the use of liners, unless the liner is nominally socketed into the bedrock; once disturbed, the sensitive clay soils, as well as the sandy and gravelly material at depth could flow under the casings, at the interface with the bedrock (based on the hydraulic conductivity results). The casing should be extended so that it is "seated" a minimum of 300 mm into the bedrock.

Alternatively, the caisson excavations could be cleaned using methods such as airlifting prior to concreting, and tremie concreting techniques may be required for placing concrete. A minimum caisson diameter of 0.9 m is recommended, to facilitate inspection.

If caisson caps are to be included as part of the design, they should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

Similar to pile installation, vibration monitoring will not be required during caisson installation if the new bridge is to be constructed on the existing alignment. However, if a new alignment is considered and the existing bridge is to remain operational during construction, vibration monitoring will likely be required during foundation excavation and caisson installation adjacent to the existing structure. Pre- and post-condition survey of the existing bridge is not considered necessary as the structure is being replaced.

If the existing piles at the central pier cannot be reused and will interfere with the installation of the new foundations, they may have to be removed to allow for construction of caissons. It should be feasible to extract the existing 12 BP 53 piles at the central pier as have been successfully done in other previous projects.

6.5.2 Axial Geotechnical Resistance

End-bearing resistance may be considered in design, provided that the base of each caisson is thoroughly cleaned of any cuttings or other material. Based on the unconfined compressive strength results on the bedrock core samples at this site, the unfactored geotechnical end-bearing resistance at ULS can be taken as 10 MPa.

End bearing for the caisson relies solely on the quality of the rock surface at the base of the excavation. As such, it is imperative that the rock surface be adequately cleaned of loose soils, rock, and debris prior to construction of the caisson.

As noted above, it will be difficult to clean the bedrock surface, and preparation/cleaning of the bedrock surface for end-bearing may not be feasible. Caisson foundations could instead be designed for side-wall (shaft) shear rather than end-bearing and a factored geotechnical resistance at ULS of 1 MPa could be used. This ULS resistance considers the RQD values recorded for the bedrock and the vertical fracturing, as well as the compressive strength test results on the rock core.

SLS resistances do not apply to caissons end bearing or socketed in the bedrock, since the SLS resistance for 25 millimetres of settlement is greater than the factored axial geotechnical resistance at ULS.

6.5.3 Lateral Geotechnical Resistance

The resistance to lateral loading developed by the soil in front of the caissons, and the reductions due to group effects, may be determined as outlined in Section 6.4.4.

6.6 Reuse of Existing Piles

As previously noted, the existing structure is supported by abutments founded on concrete-filled pipe (tube) piles (0.6 m outside diameter and about 9 mm in thickness) and the piers are founded on steel H-piles (12 BP 53). Reusing the existing foundation elements may be considered if the new structure will be constructed on the same alignment and the current overall structure length will be maintained (i.e., the new abutments and central pier will be at approximately the same location as the existing foundations).

Based on discussions with Dillon, reuse of the existing abutment piles is feasible if a semi-integral abutment structure is being considered, although it is understood to not be a preferred option by MTO Structural. Reuse of the existing H-piles (12 BP 32) at the central pier location may be feasible if the existing piles are end-bearing. Based on the original drawings, the existing piles at the pier locations were driven to refusal (not to bedrock). Additional testing (as indicated below) would be required to provide recommendations for reusing the existing piles and for compatibility with the new piles.

Extraction of the existing piles to verify pile integrity and pile load testing to confirm the axial resistances of existing structure foundations should be considered during the detailed design stage, if reuse of the existing piles is planned.

6.7 Approach Embankments

As noted in Section 6.1, various structure replacement alternatives are being considered as part of the preliminary design for this project, which include shorter and longer spans on both the existing alignment and an alignment shift of up to 12 m. In addition, a grade change of approximately 1 m is anticipated above the existing pavement grades. It is assumed that the new approach embankments will have side slopes of 2H:1V. The stability of the approach embankments is discussed further in Section 6.9 below.

The existing embankments are currently provided with front and side berms about 4 m in height and 16 to 18 m in length. Based on the existing embankment geometry and the proposed grade change, the new embankment fill will be about 10 m wide at the crest with a total maximum height of about 9 m.

In general, the surficial soils at the location of the existing/proposed underpass alignment consist of a surficial layer of fill and/or topsoil, underlain by a thin layer silty sand and compressible clay deposit. The clay is in turn underlain by deposits of glacial till containing sand and gravel layers, over limestone bedrock.

6.7.1 Geocres Review of Existing Embankments

It is understood that the original bridge was constructed in 1968. Significant settlement of the approach fills in the order of 0.6 m were predicted during the original investigation (as indicated in the previous reports available from Geocres, numbered 31G00-142 and 31G00-192), with the majority of the settlement anticipated to occur in the first 2 to 3 years.

Based on the previous Geocres report numbered 31G00-192, settlement readings on the approach embankment pavement, carried out a few years following the construction, measured up to about 0.3 m of settlement at that time. Settlement of the approach fills due to consolidation of the underlying clay necessitated repair of the pavement structure at the approaches and the restoration of the pavement was carried out in 1971. No additional settlement records were available thereafter.

6.7.2 New Embankment Construction

The topsoil fills are compressible soils that are expected to experience settlement under increased load. It is recommended that all surficial topsoil fill, as well as any organic matter and softened/loosened soils present at surface within the footprint of the new embankment be stripped prior to placement of the new embankment fill. The topsoil and softened/loosed material should be stripped to expose the underlying undisturbed subgrade.

The buried topsoil encountered beneath the existing embankment and berms in Boreholes 18-1101 and 18-1102 would however not require removal and could remain in place, as it has been present beneath the existing embankment fills since the original construction in 1968.

The new embankment fill associated with the grade raise and bridge replacement should be placed and compacted in accordance with OPSS.PROV 206 (Earth Excavation and Grading) and OPSS.PROV 501 (Compacting). The use of EPS lightweight embankment fill is discussed further in Section 6.8.1.

The existing silty sand fill subgrade that will be exposed within the new embankment footprints will be susceptible to disturbance and degradation on exposure to water and construction traffic. Following the topsoil removal, travelling over the silty sand fill subgrade soils should be minimized to limit the disturbance.

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil *(OPSS 802 – Topsoil)* and seeding *(OPSS.PROV 804 – Seed and Cover)* or pegged sod *(OPSS.PROV 803 – Sodding)* is recommended as soon as practicable after construction of the embankments.

6.7.3 Approach Embankment Settlement

As noted previously, the existing embankment loading over the sensitive and compressible clay deposit has led to large settlements of the embankments since the bridge was originally constructed. Settlement of the existing embankments has likely have occurred over time since the original construction and the clay deposit below the existing roadway embankment are most likely now normally consolidated (i.e., any additional load will result in overstressing of the clay and significant settlements).

Based on the existing embankment geometry and proposed grade change, it is expected that the height of the new embankment fill will vary from about 1 m to up to about 5 m above the existing berms depending on the replacement alternative under consideration, as discussed further below.

- **Shorter span on the existing alignment**: For this alternative, the new abutments will be founded on the existing front berms, which are about 4 m in height (i.e., the crest of the berms is at about elevation 53 m). For a 1 m grade raise above the existing roadway embankments, the final grades will be at about elevation 58m at the new abutments. As such, the height of new embankment fill could be up to about 5 m above the existing berms (behind the new abutments) but reduced to about 1 m above the existing roadway embankments;
- **Longer span on the existing alignment**: For this alternative, the new abutments will be founded at the approximately the same locations as the existing abutments (i.e., the top of the existing abutments is at about elevation 57 m). For a 1 m grade raise above the existing roadway embankments, the final grades will be at about elevation 58 m at the new abutments. As such, the height of new embankment fill would be about 1 m above the existing embankments (behind the new abutments);
- **New structure on a new alignment**: The height of new embankment fill will depend on the exact location of the new alignment. It is understood that an alignment shift of 12 m is being considered and therefore the new bridge would be founded within the footprint of the existing berms (which are about 16 to 18 m in length). As such, the height of the new embankment fill would vary from about 1 m (above the existing roadway embankments) to up to about 5 m (above the existing berms).

If conventional earth fill or granular fill is used for the new embankments, additional settlement of the approach embankments will occur as a result of compression of the new embankment fill itself and the existing fill but, more significantly, due to consolidation of the clay deposit underlying the new approach embankments.

The potential settlement of the underlying clay deposit is much more significant than the potential compression of the fills. As previously discussed in Section 4.5, the preconsolidation pressure of the clay deposit was estimated based on the (incremental and long-term loading) oedometer consolidation testing as well as the interpretation of the CPT results. The results of the available in-situ and laboratory indicated that the clay below the existing berms is normally consolidated (which was evidenced by the significant settlements recorded shortly after the original bridge was constructed) and any additional load is expected to result in significant settlements.

In order to estimate the magnitude of settlement of the clay underlying the new approach embankments, settlement analyses were carried out using the commercially available Settle-3D software by Rocscience. These analyses were carried out using the interpreted preconsolidation pressure profile and consolidation parameters presented on the Summary of Engineering Properties, Figure B13.

Based on the results of the analysis, the calculated ultimate effective stress levels in the clay would exceed the deposit's preconsolidation pressure if it is built with granular embankment fill to full height. The consolidation settlements would therefore occur in the 'virgin' compression range and be significant in magnitude. Pore water would need to be expelled for these settlements to occur and therefore, due to the low hydraulic conductivity of the clay, considerable time could be needed for these settlements to complete.

For the existing alignment alternative, based on the indicated embankment heights, the assessed existing stress level and the interpreted preconsolidation pressure profile within the clay deposit, the calculated primary consolidation settlements as a result of the new construction are estimated to be in the order of 0.1 to 0.4 m (at the location of greatest settlement in the transverse direction), which is in *addition* to the settlements that have already occurred beneath the existing embankments and berms since the original construction.

The pore water within the clay deposit underlying the existing 4 m high berms is expected take a much longer time to be expelled following the loading of the new embankment fill, in comparison with the clay deposit underlying the existing 8 m high roadway embankment (i.e., it would take longer for the settlement underneath the front berms than the existing roadway embankments to complete).

As such, it is estimated that the primary consolidation settlements for the clay deposit beneath the existing berms (where up to about 5 m of grade raise is proposed) would be in the order of 0.2 m over a period of 20 years following construction of the new bridge (the likely approximate time until the first repaving, when the profile could be corrected), with about 0.1 m of the settlements occurring within the first year of construction. Over a 50-year time frame, the anticipated primary consolidation would be in the order of about 0.4 m.

For the clay deposit beneath the existing roadway embankments (where about 1 m of grade raise is proposed), it is estimated that the primary consolidation settlement would be in the order of 100 mm over a period of 20 years following the new construction, with about 25 mm of settlement occurring within the first year. Over a 50-year time frame, the anticipated primary consolidation would be in the order of about 0.2 m.

Since the excess pore water pressure is anticipated to take a long period of time to dissipate, these settlements are not expected to increase beyond the estimates given above due to secondary compression (i.e., creep) of the deposit over a period of 50 years following construction.

The results show the maximum settlements that are expected if granular embankment fill is used to construct the new embankment to full height. These settlements would also be entirely differential relative to the structure (which would be supported on deep foundations on bedrock) and differential in the transverse direction.

It should be noted that the above settlement values are estimated using the consolidation parameters interpreted based on the results of current boreholes put down on the existing 4 m high berms. The clay underneath the existing 8 m high embankments would have been overstressed and consolidated under a higher load and therefore the consolidation characteristics of the underlying clay may differ (i.e., the settlement estimates for clay beneath the roadway embankment could potentially be less than the estimated values). However, this would need to be confirmed by additional boreholes and testing advanced through the existing embankments during the detailed design.

The results of the settlement analyses (for a period of 20 years) are summarized in the table below.

Replacement Alternatives	Approximate Height of New Embankment Fill (m)	Estimated Settlement (mm)
Shorter Span on Existing Alignment	▪ 5 m above existing front berms ▪ 1 m above existing roadway embankments	▪ 0.2 m along existing berms ▪ 0.1 m along existing roadway
Longer Span on Existing Alignment	▪ 1 m above existing roadway embankments	▪ 0.1 m along roadway embankments
New Structure on New Alignment entirely within Existing Berm Footprint	▪ 5 m above existing side berms ▪ 1 m above existing roadway surface	▪ 0.2 m at/along existing berms ▪ 0.1 m along existing roadway ▪ Up to 0.1 m differential settlement in transverse direction

It should also be noted that if the new structure is founded on an alignment entirely outside of the existing berms (which is understood likely will not be the case), the height of new embankment fill could be up to about 9 m above the natural ground level. Based on the results from the previous investigation and available settlement records, the amount of settlements could potentially be greater than 0.6 m. In addition, if the new structure is to be constructed on a new alignment partially on the existing berms and partially on natural ground level (i.e., partially outside of the berm footprint), the differential settlements will be significant across the embankments. Therefore, full realignment (entirely outside of the footprint of the existing berms) and/or partial realignment (partially outside of the existing berm footprint) are not recommended.

The estimated settlements for the clay deposit beneath the existing berms (where up to about 5 m of grade raise is proposed) are considered to be excessive and would have a negative impact on the roadway performance. The estimated settlement values exceed the usual values accepted by MTO for the approaches to bridges for non-freeways, as shown in the following table:

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

The calculated settlements for the clay deposit beneath the existing roadway (where about 1 m of grade raise is proposed), which will be within about 20 m of the new abutments, are much lower in magnitudes but still higher than the values shown in the above table.

The differential settlement rate transversely across the top of the roadway surface also needs to be limited to 100H:1V for non-freeways. The new roadway will be about 10 m wide and therefore the design should limit the differential settlements to a maximum of 100 mm in the transverse direction. For the new alignment alternative, the differential settlement would be up to about 100 mm in the transverse direction.

These tolerable settlements are based on roadway performance criteria and are therefore applicable only to the life-span of the pavement; at each pavement rehabilitation, the roadway profile could be corrected and any differential settlement eliminated. That pavement life-span is typically taken to be 15 to 20 years.

These criteria are also only applicable to the situation were settlement-sensitive services/utilities are not present beneath the embankment. Where such services are present, the tolerable settlement over the full life-span of the utility needs to be considered, which is further discussed in Section 6.10.1.

6.8 Embankment Design Alternatives

As discussed in Section 6.7, the settlement magnitude of the clay deposit beneath the existing roadway embankments (where about 1 m of grade raise is proposed) is estimated to be in the range of 100 mm over a period of 20 years following the construction of the new bridge and 0.2 m over 50 years, with about 25 mm of settlement occurring within the first year of construction. Since this area will be within about 20 m of the new abutments, the magnitude of the settlements is higher than the usual values accepted by MTO.

The calculated settlements for the clay deposit beneath the existing berms (where up to about 5 m of grade raise is proposed) will be in the order of 0.2 m over a period of 20 years and in the order of 0.4 m over a period of 50 years following the new construction, with about 0.1 m of the settlements occurring within the first year, which is considered to be excessive.

Given the significant magnitude of the anticipated settlements, and their continuous/long-term nature, it is considered that periodic re-paving to correct for the settlement is not a feasible option for addressing/mitigating the settlement effects. Subexcavation of the clay would also not be feasible due to its thickness. In addition, as noted in Section 6.1, in order to address the current deficient vertical clearance, the proposed pavement grades at the new structure will need to be increased by approximately 1 m higher from the existing pavement grades. Therefore, no profile grade increase (additional fill) is as well not a feasible alternative.

The following feasible options may therefore be considered for mitigating the anticipated settlements:

1) Lightweight Fill: Lightweight fill materials such as expanded polystyrene (EPS) could be used for the embankment construction, reducing the stress increase on the compressible clay deposit and the long-term settlement magnitudes to acceptable levels.

2) Preloading/Surcharging together with Lightweight Fill: The new embankment areas could be preloaded and surcharged, in part, and allowed to settle in advance of the roadway being paved or put into service over the new approach embankments. Due to the sensitive nature of the clay and consolidation characteristics, the preload/surcharge height would have to be limited and the use of some EPS would still be required for this option to be feasible. Some EPS would also be required for slope stability of the embankment which is further discussed in Section 6.9 below. It is expected that the preload time (including surcharge if considered) could take a minimum of 1 to 2 years to complete. In addition, preloading/surcharging with wick drains is not considered a cost-effective option for this site due to the limited thickness of clay and relatively small area for installation.

3) Rigid Inclusions (RI): The installation of Rigid Inclusions (RI) is another alternative for mitigating settlements beneath the front berms and embankments. RI's constructed of ready-mix concrete or stone columns installed within the clay soil using specialty equipment would be suitable for this site. RI's could be installed in the clay deposit, up to original ground surface, to transfer the stress from the embankment loads down to the glacial till or bedrock. Due to the thickness and state of compactness of the existing berm fill, pre-drilling through the existing berm will likely be required prior to creating soil mixing columns. A Load Transfer Platform (LTP) created using granular material and geogrid, and/or concrete would be constructed above the RI's (i.e., beneath the new embankment) to transfer the embankment loads to the columns. The granular fill in the existing front berms could likely form part of the LTP.

4) Deep Soil Mixing (DSM): Deep soil mixing (DSM) is another alternative for mitigating settlements beneath embankments. DSM consists of in situ mechanical mixing of the native soil through a process that breaks down the soil without extraction while injecting a stabilizing agent in the mix at low pressure. Similar to RI's, predrilling through existing berm would be required prior to the installation.

5) Maintaining Existing Bridge Span: Constructing the new abutments at approximately the same location as the existing abutments on the existing alignment would eliminate the thicker additional fill to be placed above the existing berms, which would in turn limit the additional approach embankment settlements to a lesser magnitude.

The advantages, disadvantages, relative costs, and risks associated with the options are provided in Table 2 following the text of this report.

If Options 3 or 4 are selected in the detailed design, the design of the rigid inclusions or soil mixing columns should consider the potential for interference with underground utilities located within the specified ground improvement area or with the proposed pile configuration at the abutment locations. The construction of a suitable/stable working platform by the general contractor may be required for stability of the drill rig or other equipment used by the ground improvement contractor.

If Option 5 is considered, where the bridge span would be maintained (i.e., new abutments will be at approximately the same location as the existing abutments) on the same alignment, and assuming the proposed grade raise would be in the order of 1 m, the calculated primary consolidation settlements for the clay deposit beneath the new roadway embankments (behind the new abutments) are in the order of 100 mm over a period of 20 years following the new bridge construction, with about 25 mm of settlements anticipated to occur within the first year. The settlement values given above are higher than those usually accepted by MTO. Therefore, some preloading and/or EPS would still be required for this structure replacement alternative.

From a foundation prospective, Option 5 is preferred, as this replacement alternative will result in minimal amount of new embankment fill and therefore less settlement in comparison to the other alternatives. It is also understood that the cost difference between the various replacement alternatives is not significant.

6.8.1 Expanded Polystyrene Lightweight Fill Embankment Construction

The settlement analyses indicate that the clay is most likely now normally consolidated (i.e., the existing effective stress is at or near the pre-consolidation pressure of the deposit) and therefore it cannot take on any additional load without overstressing and causing significant settlements.

The total thickness of conventional embankment fill (including the pavement structure) needs to be limited if significant post-construction settlements are to be avoided. Given the required thickness of material needed for the pavement structure (about 1 m), the protective concrete slab over the EPS (discussed below), and a granular working/levelling pad for placement of the EPS, it is considered that the additional thickness of the new embankment (under the pavement structure) would need to consist of EPS for minimal settlements of the approach embankments to occur, as required in the area adjacent to the pile supported abutments. Some of the existing embankment material would therefore need to be removed in order to limit the stress increase on the underlying clay and hence prevent excessive settlements.

Based on the above, the thickness of EPS will need to be equal to the total increase in height of fill (i.e., the EPS will need to be 1 m in thickness for 1 m of grade raise or 5 m in thickness for the 5 m height of fill required at the berm locations).

The EPS will need to be covered with a concrete slab to protect it from being overstressed by the traffic loads; overstressing of the EPS could lead to rutting of the pavement surface. A concrete slab thickness of 125 mm with reinforcing that is typical for the protective slab.

A suitable lightweight fill type would be EPS22 in accordance with ASTM D6817-11, or equivalent.

The EPS is potentially soluble in hydrocarbons. To guard against dissolution of the EPS in the case of an accidental release and infiltration of fuel (such as could occur in the case of a collision), it is general practice to cover the outside surface of the EPS with 10 mil polyethylene sheeting.

The blocks beneath the side slopes can step up to match the 2H:1V side slope and, once covered with the polyethylene sheeting, can then be covered with soil.

An NSSP providing additional information on the EPS material and its placement as well as the concrete protective slab should be included in the contract documents.

6.8.2 Preloading/Surcharging together with Lightweight Fill

If time allows for a minimum of 1 to 2 years preload period, granular embankment fill may be placed as a preload (i.e., to allow for some of the primary settlement to occur in advance of the roadway being paved or put into service) over the new approach embankments. This option could lessen the thickness of EPS required for embankment construction.

As noted in Section 6.7.3, the calculated primary consolidation settlements for the clay deposit underlying the existing front berms (where up to about 5 m of grade raise is proposed) are in the order of 0.2 m over a period of 20 years following construction of the new bridge (the likely approximate time until the first repaving, when the profile could be corrected), with about half of that (i.e., 0.1 m) occurring within the first year. Therefore, some EPS would still be required for this option to be feasible.

Surcharging could be used to reduce the above settlements but the embankment has limited stability if fill heights greater than proposed are placed. Surcharging would also not reduce the settlement magnitudes immediately adjacent to the abutment to 25 mm or less. It is recommended surcharging, if considered, should not be greater than 1 m in height. The stability related to addition of 1 m surcharge is discussed further in Section 6.9 below.

A settlement monitoring program will need to be implemented to monitor the settlements prior to, during, and following the ‘preload/surcharge’ placement. Settlement monitoring would provide an indication that the settlements are occurring as anticipated and to determine if the granular fill heights have to be altered in consideration of the 1 m pavement structure thickness to be placed above the EPS. Additional guidelines for the settlement monitoring can be prepared for the detailed design if this alternative is adopted and should be included in the contract documents.

6.8.3 Rigid Inclusions (RI)

RI’s are used to transfer unacceptable embankment loads through compressible soils to stiffer soil or rock. This ground improvement method increases the load carrying capacity of the soil, reduces the compressibility (and therefore the settlement magnitudes) and helps prevent slope instability.

RI’s can consist of aggregate (with or without liners), cement-treated aggregate, grouted aggregate, or concrete columns. Aggregate columns without liners are not considered feasible for this site considering the low strength of the underlying clay deposit. The clay would likely offer little resistance/confinement during the installation of the aggregates and therefore there would be a high risk of column bulging in addition to the potential for shearing failures and remoulding of the clay structure.

RI’s using concrete or grouted columns or aggregates with liners would be the most feasible RI systems for the site, with a specifically designed LTP. The LTP, which transfers the load from the embankments to the rigid inclusions, is a key element of the design that distributes the loads to the columns. The system should be designed to satisfy MTO settlement and global stability criteria.

Due to the configuration of the proposed embankment with respect to the existing embankment, it is recommended that the RI's be installed within the existing front berms (i.e., from about Elevation 53 m) beneath the footprint of the new embankment. Due to the thickness and state of compactness of the existing berm fill, pre-drilling will likely be required prior to installing the RI's. The RI's would extend down to the bottom of the clay layer (to about Elevations 41 to 42 m) to reach a stiffer material. Additional slope stability analyses would be required to assess the stability of the existing and future embankments.

The LTP would also need to be designed by the specialty ground improvement contractor to limit the load that is directly transmitted to the compressible clay soils. The load transfer system could consist of several alternating layers of geogrid and engineered fill or could be a concrete layer at the top of the RI's.

Wick drains may be required, in addition to RI's, depending on the specialty contractor that is retained and their proprietary design. Wick drains or other structural reinforcement can be used to mitigate against seismic instability.

At the abutment locations, the RI pattern is typically modified to allow for pile installation after some settlement has occurred.

Some amount of EPS may still be required directly behind the abutments if the design cannot acceptably limit the settlements and resulting downdrag forces (i.e., if the settlements at the abutments will exceed 10 mm, then the downdrag forces indicated in the FIDR will need to be applied for design of the piles).

Field trials would be recommended prior to or at the same time as design of the rigid inclusions to establish the range of strength that can be achieved from the ground improvement. The CPT testing carried out during the current investigation will be useful to the specialty contractor for this option.

A settlement monitoring program is recommended to monitor the settlements following the installation of RI, if selected. Settlement monitoring would provide an indication whether the settlements are occurring as anticipated and an evaluation of the effectiveness of this ground improvement method.

6.8.4 Deep Soil Mixing (DSM)

Improvement of weak and compressible soils by DSM can be achieved by mixing the existing soils using either a slurry with binder (wet DSM) or a dry binder (dry DSM). Jetting of slurry can be also used to enhance mechanical mixing. Similar to RI's, DSM creates large columns of improved ground for embankment support. Due to the thickness and state of compactness of the existing berm fill, pre-drilling through the existing berm will likely be required prior to creating soil mixing columns.

Approximately 1.8 m diameter columns would be created with the DSM procedures, placed in a specific pattern. A track-mounted drill rig would be used to directly inject the binder into the column areas, add the stabilizing agent, and mixing.

The interaction of deep soil mixing columns with surrounding soils needs to be investigated to understand the possibility of settlement of the existing berms during remolding of the underlying clay and subsequent cement mixing.

The high plasticity clays with high shear strengths (i.e., the weathered clay crust at the site) may require pre-treatment for successful performance. Furthermore, areas with stiff soils and/or obstructions, such as the existing embankments and side slopes, may require pre-drilling ahead of the soil mixing process.

Some amount of EPS may still be required directly behind the abutments if the design cannot acceptably limit the settlements and resulting downdrag forces (i.e., if the settlements at the abutments will exceed 10 mm, then the downdrag forces indicated in the FIDR will need to be applied for design of the piles).

It is recommended that at least one pre-production test column be advanced prior to construction. The ground surface adjacent to the test column should be monitored for settlement during installation to assess the potential impacts during remoulding of the clay soil. In addition, the completed test column should be cored at 7 and 14 days to obtain samples for strength (UCS) testing of the in-situ soil-cement mix.

A settlement monitoring program is recommended to monitor the settlements following the installation of DSM, if selected. Settlement monitoring would provide an indication whether the settlements are occurring as anticipated and an evaluation of the effectiveness of this ground improvement method.

6.9 Global Stability

Static and seismic slope stability analyses of the proposed embankments (for the alternative of the shorter span on the existing alignment using conventional earth fills) were carried out using the commercially available SLOPE-W software (produced by Geo-Studio 2007), based on the soil parameters given in the following table.

Soil Stratum	Bulk Unit Weight (kN/m^3)	Shear Strength Parameters	
		Angle of Internal Friction (°)	Undrained Shear Strength (kPa)
Embankment Fill	21.5	35	-
Silty Sand	19.0	32	-
Weathered Crust	17.0	-	30 - 70
Grey Clay	15.5	-	30
Glacial Till	22.0	35	0

The unit weights of the weathered clay crust and the unweathered clay were inferred from the measured water content data for these deposits, as shown on Figure B13.

The mobilized/available undrained shear strength of the weathered crust and unweathered clay (C_u) was inferred from the results of the in-situ vane testing, CPT results, as well as from the results of the laboratory oedometer consolidation testing on samples obtained from the current boreholes put down on the existing 4 m high berms.

The analyses were carried out for undrained (i.e., short-term) conditions. Undrained conditions represent the critical condition experienced during and immediately following construction of the embankments. With time, the excess pore water pressures generated in the clay deposit as a result of the loading would dissipate and ‘drained’ conditions would exist, with a higher factor of safety against instability. A minimum factor of safety of 1.3 is considered acceptable against undrained deep-seated embankment instability.

The stability of the embankments was also evaluated under seismic loading conditions. The minimum factor of safety value that is typically required against instability during a seismic event is 1.1. A horizontal seismic coefficient of 0.19 was used for the analyses. This value is based on the peak horizontal ground acceleration for the site provided in Section 6.2.3 (with half that value, i.e., 0.5, being used, per standard practice), considering the potential amplification of the seismic ground motions that could occur through the clay deposit.

The results of the stability analyses indicate that, with appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the up to 9 m high new embankments (with vertical abutment walls) will have an acceptable factor of safety against deep-seated rotational instability for the undrained static condition, considering the clay has had over 50 years to consolidate and gain strength.

With the addition of 1 m surcharge, if considered, the factor of safety would be 1.2 under static condition, which may be considered acceptable for the construction stage (to be confirmed by MTO). The contractor will be responsible for the stability of the embankments during construction.

However, the results of the *seismic* slope stability analyses indicate that, even considering 10 percent strength gain during a seismic event, the up to 9 m high embankments will *not* have an acceptable factor of safety against deep-seated rotational instability (i.e., at least 1.1).

It should be noted that the slope stability analyses were carried out based on the interpreted strength parameters from the current boreholes put down on the existing 4 m high berms. The clay underneath the existing 8 m high embankments would have been overstressed and consolidated under a higher load and therefore the strength characteristics of the underlying clay may differ (i.e., the undrained shear strength parameters could potentially be better than those beneath the existing berms.). It should also be noted that the low factors of safety for the seismic loading condition are based on relatively conventional analyses, which is considered sufficient for preliminary design of the new structure. It is possible that more sophisticated analyses (based on the potential displacements) might indicate acceptable seismic performance, which could be considered during detailed design, if deemed necessary.

In addition, in the event of a seismic event, it is considered that sloughing of the slopes may take place, which may need to be repaired.

If Option 1 is considered, where the embankments are to be constructed (almost entirely) with lightweight fill material (such as EPS), to avoid excessive settlements, the weight of the embankment would be much less and there would be an adequate factor of safety against instability.

A sensitivity analysis was carried out to estimate the minimum amount of EPS required to achieve an acceptable factor of safety of 1.1 for seismic conditions. The results of the slope stability analyses are summarized in the table below and are graphically shown on Figures H1 through H6 in Appendix H.

EPS Thickness Below Roadway and Concrete Protection Slab (m)	Factor of Safety Under Seismic Conditions
None	0.9
1	1.0
2	1.1

Based on the results provided in the above table, for the shorter span alternative on the existing alignment, approach embankments with 2H:1V side slopes and a minimum 2 m of EPS should be stable under both static and seismic conditions.

As previously mentioned, the above analyses are for the shorter span alternative on the existing alignment using conventional earth fills. The undrained shear strength parameters of clay were interpreted based on the results of current boreholes put down on the existing 4 m high berms. The undrained shear strength parameters of the clay underneath the existing 8 m high embankments, which have been overstressed and consolidated under a higher load, would likely be the same or better than those beneath the existing berms. Therefore, for the longer span alternative on existing alignment, or new structure on a new alignment (constructed within the existing berm footprint), the approach embankments with 2H:1V side slopes and a minimum 2 m of EPS would also likely be stable under both static and seismic conditions.

However, if full realignment is considered (where the approach embankments are to be constructed at natural ground level and are entirely outside of the existing berm footprint) or partial realignment (where the approach embankments are to be constructed partially outside of the existing berm footprint), the new approach embankments will likely need to be provided with both front and side berms with similar height and lengths as for the existing embankments in order to achieve acceptable factors of safety for both static and seismic conditions.

If Options 3 or 4 are selected, the slope stability analysis would have to be reviewed/carried out by the specialty contractor based on the final embankment height and geometry.

6.10 Construction Considerations

The following sections identify construction considerations that may impact the future design and construction.

6.10.1 Existing Utilities

There is a buried Bell fibre optic utility located beneath the existing south embankment, approximately 12 m south of the south expansion joint. As discussed in Section 6.7.3, the primary consolidation settlements beneath the existing embankments, as a result of a 1 m of grade raise, is estimated to be in the order of 100 mm over a period of 20 years following the new construction and 0.2 m over 50 years, with about 25 mm of settlement occurring within the first year. If these magnitudes are not considered to be tolerable by the utility owner, the settlements will need to be mitigated by the use of lightweight fill or alternate settlement mitigation measures as previously discussed.

6.10.2 Existing Foundation Elements

The existing foundation elements may remain in place or be reused from a geotechnical prospective, provided that the new piles could be driven without interference with the existing pile group. However, the piles at the central pier may need to be removed to allow for caisson installation, if considered at that location. It should be feasible to extract the existing concrete fill tube piles at the abutments and 12 BP 53 piles at the piers, if required.

6.10.3 Open-Cut Excavations

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities.

Only minimal excavations are anticipated for this project for subgrade preparation, and the anticipated removal of the existing structure. Some limited excavation of the existing embankments fill may also be carried out for construction of new abutment foundations. Excavations will be made mostly through the existing fill.

No excavations are anticipated in the underlying silty sand and clay. The groundwater level is indicated to be at about Elevation 47.0 m. The soils at the site are generally classified as Type 3 soils according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these soils should be made with side slopes no steeper than 1H:1V.

6.10.4 Temporary Protection Systems

It is anticipated that temporary roadway protection will be required along the existing structure, if it is to remain operational, adjacent to the new alignment, to permit construction of the new abutments. For the pier footing construction, temporary excavation support may be required in the median, adjacent to the driving lanes of

Highway 401. It is considered that the temporary support system could consist of internally braced solider piles and lagging or steel sheet pilling.

The design of the shoring will be entirely the responsibility of the contractor. Where required, temporary protection systems shall be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection System*), and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities can tolerate this magnitude of deformation. Traffic loading should be included as a surcharge.

6.10.5 Vibration Monitoring

If the new structure is to be constructed on the existing alignment, it is expected that the existing bridge will be removed prior to commencement of construction, and vibration monitoring of the existing bridge should not be required during construction. However, if a new alignment is considered and the existing bridge is to remain operational during construction, vibration monitoring will likely be required during foundation excavation, pile driving adjacent to the existing structure, and/or caisson installation (including permanent/temporary liner installation).

6.10.6 Groundwater and Surface Water Control

The groundwater level at the site is measured at about Elevation 47 m. Only minimal excavations are anticipated for the construction of the new structure if supported on driven H-piles, which will likely involve minimal groundwater and surface water control. It should be possible to handle groundwater inflows by pumping from well filtered sumps established in the floor of the excavations. Surface water should be directed away from the excavations.

High water inflow could be expected if caissons are considered for the central pier. The caissons should be socketed into bedrock and tremie methods may be required for placing concrete. If dewatering is required, dewatering shall be carried out in accordance with OPSS 902 (*Excavating and Backfilling - Structures*).

6.11 Corrosion and Cement Type

Three soil samples, one from each of Boreholes 18-1101 to 18-1103, submitted to Eurofins Environment Testing for chemical analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The results of the testing are attached in Appendix G.

The results indicate a low potential for concrete degradation due to the presence of sulphates, and that concrete made with Type GU Portland cement should be acceptable for substructures. However, the results also indicate a high potential for corrosion of exposed ferrous metal.

6.12 Recommendations for Detailed Design

As a summary, additional foundations engineering investigations and design should be provided for the following foundation/structural aspects of the project during the detailed design stage:

- **Downdrag on Piles**: Various methods have been used in calculating the magnitude of the downdrag force, including the Nordlund method, 1979 and β-Method in cohesionless, and α-Method in cohesive soil, to refine the estimation of downdrag load on the new piles and those values have been presented in this report. However, based on discussions with Golder personnel familiar with the original investigation for the existing bridge, it is understood that downdrag loading was not included in the original design of the existing structure. There is currently a separate MTO study on a different site to determine if downdrag forces from an original construction still acts on the existing piles. Depending on the outcome of that study, a comment could potentially be made during the detailed design regarding the magnitude of the downdrag force on the existing piles, if any.

- **Reuse of Existing Piles**: Reusing the existing foundation elements may be considered if the new structure will be constructed on the same existing alignment and the current overall structure length will be maintained (i.e., the new abutments and central pier will be at approximately the same location as the existing foundations), although it would depend on the type of new structure (i.e., a semi-integral abutment structure will be required in order to reuse the existing abutment piles), as well as the nature and conditions of the existing piles (i.e., the existing piles at the central pier may be reused provided that they are end-bearing). Additional testing (including extraction and pile load testing) would be required to provide geotechnical recommendations for reusing the existing piles and for compatibility with the new piles. The feasibility of reusing the existing piles will also need to be confirmed from a structural perspective during the detailed design based on the results of the additional testing.
- **Site-specific Seismic Response Spectra Analyses**: The seismic hazard values provided in this report are based on the 5th generation seismic hazard maps published by GSC, which is considered sufficient for the preliminary design of the new structure. It should be noted that probabilistic ground motions were taken from the GSC online hazard calculator. Uncertainties incorporated into the seismic hazard model are those included in the 5th Generation Seismic Hazard Model as described by Halchuk et al. (2014). A site-specific seismic response spectra analyses could be considered during the detailed design, if deemed necessary from a structural prospective.
- **Approach Embankment Settlement**: The magnitude of settlements was estimated based on the interpreted consolidation parameters from the current boreholes put down on the existing 4 m high berms. The clay underneath the existing 8 m high embankments would have been overstressed and consolidated under a higher load and therefore the consolidation characteristics of the underlying clay may differ (i.e., the settlement estimates for clay beneath the roadway embankment could potentially be less than the estimated values). If the new structure is to be constructed on the existing alignment, additional boreholes should be advanced through the existing 8 m high embankments and additional testing should be carried out during the detailed design in order to refine the estimation of the magnitude of settlements.
- **Approach Embankment Stability**: The slope stability analyses were carried out based on the interpreted strength parameters from the current boreholes put down on the existing 4 m high berms. The clay underneath the existing 8 m high embankments would have been overstressed and consolidated under a higher load and therefore the strength characteristics of the underlying clay may differ. The results of stability analyses for the seismic loading condition are also based on relatively conventional analyses, which is considered sufficient for preliminary design of the new structure. It is possible that more sophisticated analyses (based on dynamic and damping soil properties and the potential displacements) might indicate acceptable seismic performance, which could be considered during the detailed design, if deemed necessary.
- **Foundation Boreholes**: Additional boreholes should also be advanced at each of the new abutments during the detailed design stage to determine the soil and bedrock conditions at the proposed foundation locations once the preferred alignment is selected.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Christine Ko, P.Eng., and reviewed by Mr. Bill Cavers, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Fin Heffernan, P.Eng., the Designated MTO Foundations Contact for this assignment, conducted an independent quality review of this report.

Golder Associates Ltd.

Christine Ko, P.Eng.
Geotechnical Engineer

William Cavers, P.Eng.
Associate, Senior Foundations Engineer

Fin Heffernan, P.Eng.
MTO Foundations Designated Contact

CRG/CK/WC/FJH/mvrd/sg

https://golderassociates.sharepoint.com/sites/25312g/deliverables/1100 - fraser road/05-final/1899802-1100-001-r-rev0-fraser road fidr-20190524.docx

Table 1: Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Constructability/Risks
Steel H-piles driven to bedrock	▪ Feasible for support of bridge replacement ▪ Preferred	▪ High geotechnical resistances and negligible settlement ▪ Allows for integral abutment construction	▪ Some potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in some piles "hanging up" in the glacial till/sand and gravel deposits and lower geotechnical resistances ▪ Pre-augering or additional piles may be required ▪ Temporary protection systems may be required at the central pier ▪ Negative skin friction (downdrag) loads must be considered in design	▪ Moderate	▪ Low risk of driven H-piles "hanging up" in glacial till or sand and gravel deposits

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Constructability/Risks
Steel pipe (tube) piles, driven to found in bedrock	■ Feasible, but not preferred	■ Higher geotechnical resistances and negligible settlement ■ Allows for semi-integral and potentially integral abutment configuration	■ Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in some piles “hanging up” in the glacial till/sand and gravel deposits and lower geotechnical resistances ■ Pre-augering or additional piles may be required ■ Temporary protection systems may be required at the central pier	■ Moderate	■ Slightly greater risk than for steel H-piles of pipe piles “hanging up” in glacial till or sand and gravel deposits
Caissons founded on or socketed into bedrock	■ Feasible ■ May be preferred for central pier	■ Abutment pile caps could be maintained higher than footings, reducing depth of excavation and potential for temporary protection system ■ High geotechnical resistances and negligible settlement ■ Allows for semi-integral abutment configuration	■ Permanent casings required to construct caissons ■ Possibility of encountering cobbles or boulders during augering ■ Coring or churn drilling may be required to form nominal socket in bedrock ■ High water inflow expected, tremie methods for placing concrete required	■ Moderate to High	■ Rock socketing would be required

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Constructability/Risks
Reusing Existing Piles	■ Feasible, but not preferred for abutments	■ Potential for minor cost saving	■ Semi-integral abutment structure required if the existing abutment piles are to be reused, which is not a preferred option by MTO Structural. ■ Additional testing would be required to provide recommendations for reusing the existing piles and for compatibility with the new piles. ■ Potential of no cost saving or more expensive as re-design of the new bridge may be required depending on the results of the additional testing.	■ Moderate to High	■ Moderate to high risk of need to re-design the new bridge depending on the results of the additional testing

Table 2: Comparison of Embankment Settlement Mitigation Alternatives

Embankment Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 1 Lightweight fill (EPS)	■ Limits post-construction maintenance ■ Eliminates or substantially reduces downdrag forces at the abutments ■ Minimal impact on schedule	■ Expensive	■ Moderate to High	■ Low risk option ■ Abutment piles will have to be carefully designed to resist seismic forces; a higher strength EPS may be needed behind the abutments
Option 2 Pre-loading/Surcharging with Lightweight fill (EPS)	■ Reduces the amount of EPS required	■ May delay paving required ■ May require postconstruction maintenance prior to end of pavement life cycle ■ EPS would still be required ■ Stability concerns limit surcharge height	■ Moderate to High	■ Some uncertainty about schedule, since cannot complete roadway construction until monitoring indicates sufficient settlement has occurred ■ Would lead to unacceptable settlement if not used in conjunction with EPS ■ Settlement monitoring recommended prior to final paving

Embankment Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 3 Rigid Inclusions (RI) (e.g., Concrete Columns)	■ Relatively rapid installation ■ Allows for greater bearing pressures and limited settlements	■ Mobilizing specialty subcontractor may have impact on schedule	■ Moderate to High	■ Some field testing ahead of production would be recommended ■ Design should consider risk of interference with piles advanced for abutment construction and impact on foundations or other utilities ■ May require predrilling through existing berms and roadway embankments where columns are required ■ Settlement monitoring recommended prior to final paving

Embankment Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 4 Deep Soil Mixing (DSM)	▪ Relatively rapid installation ▪ Allows for greater bearing pressures and limited settlements	▪ Mobilizing specialty subcontractor may have impact on schedule ▪ Sensitive to installation sequence and radial distance between the mixing columns.	▪ Moderate to High	▪ Some field testing ahead of production would be recommended ▪ Slightly higher risk option for high plasticity clay ▪ May require predrilling through existing berms and roadway embankments where columns are required ▪ Design should consider risk of interference with piles advanced for abutment construction and impact on foundations or other utilities ▪ Settlement monitoring recommended prior to final paving ▪ Unknown interaction of deep soil mixing columns with surrounding soils and possibility of existing embankment settlement during clay remolding and cement mixing procedure.
Option 5 Maintaining Current Overall Bridge Span on Existing Alignment	▪ Reduces the amount of EPS required	▪ Longer structure than required to meet geometric design requirements	▪ Moderate to High	▪ Low risk option

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

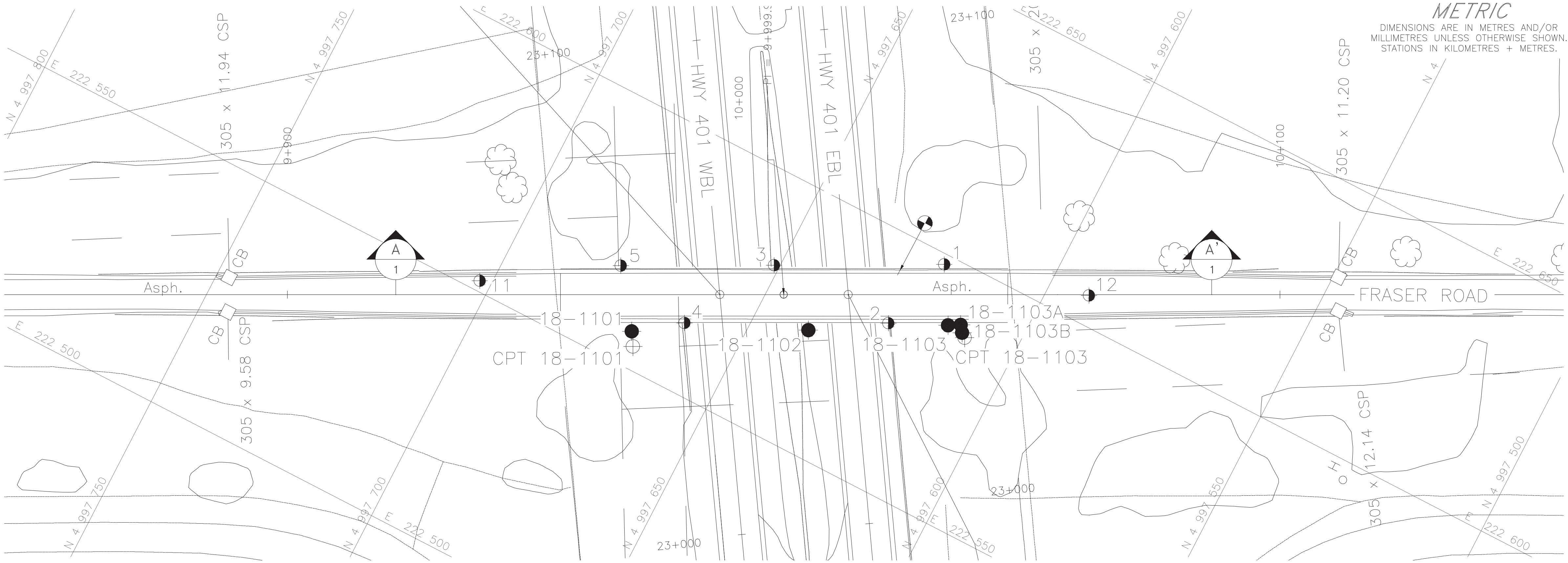
CONT No.
GWP No. 4248-15-00

FRASER ROAD UNDERPASS HIGHWAY 401

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

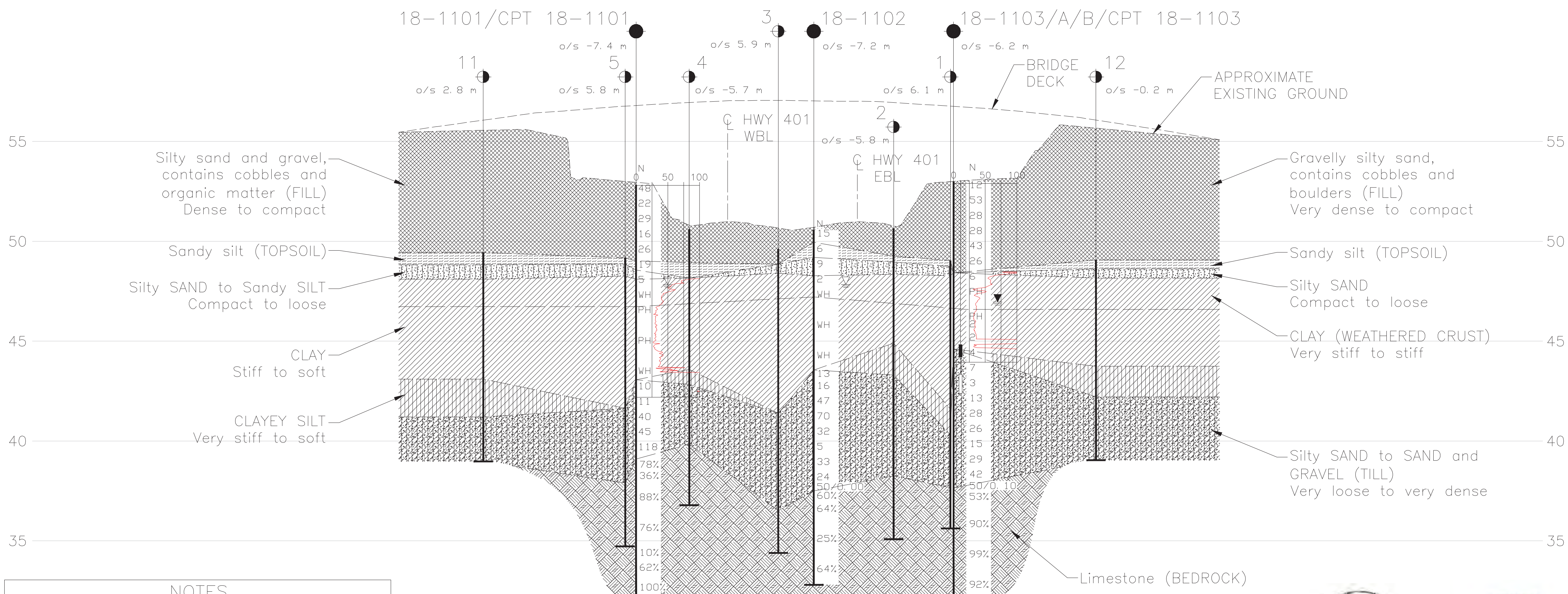
SHEET

GOLDER



PLAN
SCALE
10 0 10 20 m

KEY PLAN
SCALE
1 0 1 2 km



PROFILE A-A'
SCALE
10 0 10 20 m
2.5 0 2.5 5

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.

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.

F. J. HEFFERNAN
PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31G5-273

HWY. 401	PROJECT NO. 1899802-1100		DIST. EASTERN
SUBM'D. CK	CHKD. CK	DATE: 11/29/2018	SITE: 31-230
DRAWN: JM	CHKD. WC	APPD. FJH	DWG. 1

APPENDIX A

Record of Boreholes - Current Investigation

List of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Record of Boreholes 18-1101 to 18-1103, 18-1103A and 18-1103B

LIST OF SYMBOLS

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

I. GENERAL

π	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

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	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
γ'	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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	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
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

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

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

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

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 10	Trace	Trace sand
10 to 20	Some	Some sand
20 to 35	(ey) or (y)	Sandy
over 35	And	Sand and Gravel

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, 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 90º angle is horizontal.

Description and Notes

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

Abbreviations

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No 18-1101

SHEET 1 OF 4

METRIC

PROJECT 1899802-1100

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

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
52.8	GROUND SURFACE						
0.0 / 0.1	(SM) Silty sand, contains organic matter (rootlets) (TOPSOIL/FILL) Dark brown Moist	1	SS	48			
	(SM/GM) Silty sand and gravel, contains cobbles and organic matter (FILL) Dense to compact Brown Moist	2	SS	22		52	
		3	SS	29		51	32 36 26 6
		4	SS	16		50	
		5	SS	26			
49.0 / 3.8	(ML) Sandy silt, contains organic matter and silty sand seams (wood/rootlets) (TOPSOIL) Dark brown to black Moist	6	SS	19		49	
48.5 / 4.3	(SM) Silty SAND, fine, contains organic matter (rootlets/wood) Compact to loose Grey Moist						
48.1 / 4.7	(CH) CLAY, trace sand, highly fissured, contains silt and sand seams (WEATHERED CRUST) Very stiff to stiff Grey-brown Moist	7	SS	5	∇	48	
		8	SS	WH		47	
46.7 / 6.1	(CH) CLAY Firm Grey Moist	9	TP	PH			
						46	
		10	TP	PH		45	
						44	
		11	SS	WH			
43.0 / 9.8						43	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ kN/m³

Continued Next Page

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No 18-1101

METRIC

PROJECT 1899802-1100

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

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(SM) Gravelly Silty SAND (TILL) Loose to compact Grey Wet	12	SS	10			
42.1 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 13.7	Limestone (BEDROCK) Bedrock cored from depths 13.7 m to 20.9 m. For bedrock coring detail refer to Record of Drillhole 18-1101	1	RC	REC 79%			RQD = 78%
		2	RC	REC 81%			RQD = 36%
		3	RC	REC 100%		UCS = 32.9 MPa	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%

Elevation scale: 42, 41, 40, 39, 38, 37, 36, 35, 34, 33

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

Continued Next Page

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

RECORD OF BOREHOLE No 18-1101

SHEET 3 OF 4 **METRIC**

PROJECT 1899802-1100

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 (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
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%		32				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_IM\MTO\HWY401\FRASERROAD\02_DATA\GINT\1899802.GPJ GAL-GTA.GDT 19-5-23 ZS

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

RECORD OF DRILLHOLE: 18-1101

PROJECT: 1899802-1100
LOCATION: N 4997670.5 ;E 222557.1
INCLINATION: -90° AZIMUTH: ---
DRILLING DATE: September 4-5, 2018
DRILL RIG: CME 75
DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

NOTE: For abbreviations, symbols and descriptions refer to **LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY**

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	FEATURES
		BEDROCK SURFACE	39.10			
14–21	Rotary Drill, HQ3 Core	Nodular Limestone (BEDROCK), with fresh, thinly bedded, dark grey to black, fine grained, non-porous, weak shale Fresh Thinly bedded Medium grey Fine grained Non-porous Medium strong - Broken/lost core from 14.27 m to 14.45 m - Vertical joint from 14.61 m to 14.77 m - Broken core from 14.85 m to 14.92 m (with 40 mm clay seam) - Broken core from 15.68 m to 15.72 m (with 25 mm clay seam) - Broken core from 16.25 m to 16.26 m - Broken/lost core from 16.59 m to 16.74 m - Broken core from 17.35 m to 17.36 m - Vertical joint from 17.77 m to 17.92 m - Broken core from 18.14 m to 18.16 m - Broken/lost core from 18.24 m to 18.90 m - Broken/lost core from 18.90 m to 18.95 m (with 10 mm clay seam) - Broken core from 19.02 m to 19.06 m - Vertical joint from 19.06 m to 19.19 m - Broken core from 19.19 m to 19.21 m	13.73	1 2 3 4 5 6 7	100 0 0 0 0 0 0	UCS = 32.9 MPa (at ~15 m)
			31.95			
21		END OF DRILLHOLE	20.88			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: CK

GOLDER

Foundation Design

RECORD OF BOREHOLE No 18-1102

SHEET 1 OF 3 **METRIC**

PROJECT 1899802-1100

G.W.P. 4248-15-00

LOCATION N 4997639.0; E 222573.6 NAD MTM ZONE 8 (LAT. 45.113940; LONG. -74.545040)

ORIGINATED BY RI

DIST Eastern HWY 401

BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core

COMPILED BY ZS

DATUM Geodetic

DATE September 18-19, 2018

CHECKED BY CK

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m³

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

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
50.6	GROUND SURFACE							
0.0	(SW/GW) Gravelly sand to sandy gravel, angular (FILL)							
50.4 / 0.2	Compact, Grey, Moist	1	SS	15	25	30	(45)	
50.0 / 0.6	(ML) Gravelly sandy silt (FILL), Compact, Brown, Moist; (ML) sandy SILT, fine, contains organic matter and silty sand layers (rootlets/wood) (TOPSOIL), Dark brown, Moist	2	SS	6				
49.2 / 1.4	(ML) Sandy SILT, contains organic matter (rootlets/wood), Loose, Green-grey, Wet	3	SS	9	2	26	(72)	
48.3 / 2.3	(CH) CLAY, trace sand, highly fissured, contains thin to thick laminations of silty sand (WEATHERED CRUST), Stiff, Grey-brown, Moist	4	SS	2				
47.3 / 3.4	(CH) CLAY, trace sand, trace gravel, Firm to soft, Grey, Moist	5	SS	WH				
		6	SS	WH				
		7	SS	WH				
43.6 / 7.0	(ML/SM) SILT and SAND, some clay, trace gravel (TILL), Compact, Grey, Wet	8	SS	13	6	48	32	14
43.3 / 7.3	(SM) Gravelly Silty SAND (TILL), Compact, Grey, Wet	9	SS	16				
42.2 / 8.4	(SP/GP) SAND and GRAVEL, Dense, Grey, Wet	10	SS	47	49	40	(11)	
41.6 / 9.0	(SM) Gravelly Silty SAND, contains cobbles and boudlers (TILL), Very dense to loose, Grey, Wet	11	SS	70				

>96 (field vane, elev. approx. 43.6)

Continued Next Page

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No 18-1102

SHEET 2 OF 3

METRIC

PROJECT 1899802-1100

G.W.P. 4248-15-00

LOCATION N 4997639.0; E 222573.6 NAD MTM ZONE 8 (LAT. 45.113940; LONG. -74.545040)

ORIGINATED BY RI

DIST Eastern HWY 401

BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core

COMPILED BY ZS

DATUM Geodetic

DATE September 18-19, 2018

CHECKED BY CK

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(SM) Gravelly Silty SAND, contains cobbles and boudlers (TILL) Very dense to loose Grey Wet	12	SS	32		○		
					40			
		13	SS	5				
39.2 11.4	(SP/GP) SAND and GRAVEL, some silt (TILL) Compact to very dense Grey Wet				39			
		14	SS	33				40 40 15 5
		15	SS	24	38	○		
37.5		16	SS	50/0.00				
13.1	Limestone (BEDROCK) Bedrock cored from depths 13.1 m to 17.8 m For bedrock coring detail refer to Record of Drillhole 18-1102	1	RC	REC 100%				RQD = 60%
					37			
		2	RC	REC 87%				RQD = 64%
					36			
		3	RC	REC 56%	35			RQD = 25%
					34			
		4	RC	REC 81%			UCS = 37.0 MPa	RQD = 64%
32.8					33			
17.8	END OF BOREHOLE NOTES: 1. Water level in open borehole at a depth of 2.7 m below ground surface (Elev. 47.9), measured during drilling.							

Column headings also printed: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 25 50 75.

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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: 18-1102

PROJECT: 1899802-1100

LOCATION: N 4997639.0 ;E 222573.6

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 18-19, 2018

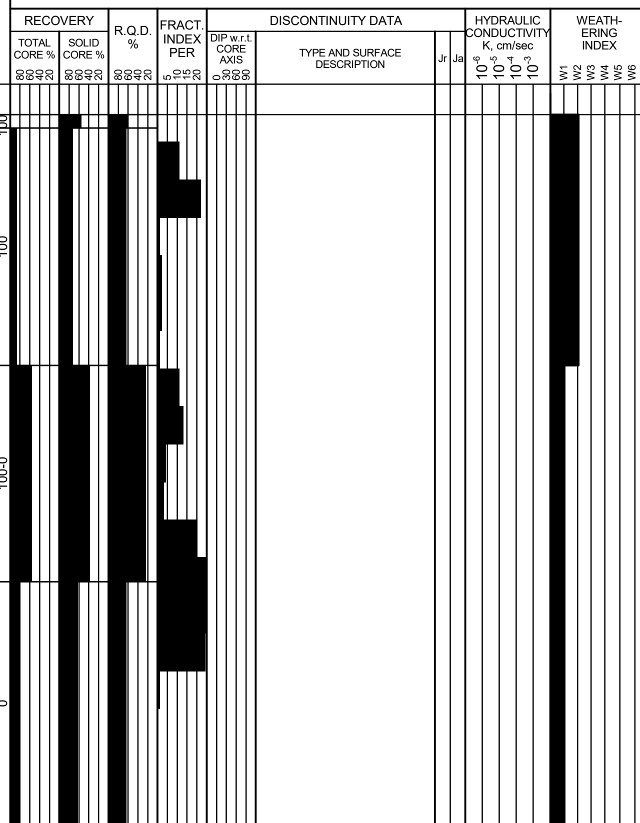
DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	FEATURES
		BEDROCK SURFACE	37.50			
	ROtary Drill / NQ Core	Limestone (BEDROCK), with slightly weathered to fresh, thinly bedded, dark grey to black, fine grained, non-porous to slightly porous shale Slightly weathered to fresh Thinly bedded Medium grey Fine grained Non-porous to slightly porous - Broken/lost core from 13.26 m to 13.31 m - Broken core from 13.36 m to 13.37 m - Broken core from 13.42 m to 13.43 m - Broken/lost core from 13.53 m to 13.75 m - Broken core from 14.90 m to 14.98 m - Broken/lost core from 15.08 m to 15.20 m - Broken core from 15.27 m to 15.30 m - Broken/lost core from 15.56 m to 16.73 m	13.07	1	100	
14				2	100	
15				3	100-0	
16						
17				4	0	UCS = 37.0 MPa
			32.80			
		END OF DRILLHOLE	17.77			
18						
19						
20						
21						
22						
23						



DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: CK

GOLDER

RECORD OF BOREHOLE No 18-1103

METRIC

PROJECT 1899802-1100

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
53.0	GROUND SURFACE				
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) Compact Brown to grey Moist				
48.4 / 4.6	(CH) CLAY, trace sand, highly fissured (WEATHERED CRUST) Very stiff to stiff Grey-brown Moist	7	SS	6	
		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	

ELEVATION SCALE: 52, 51, 50, 49, 48, 47, 46, 45, 44

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m³

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

Continued Next Page

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

RECORD OF BOREHOLE No 18-1103

METRIC

PROJECT 1899802-1100

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

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED: 20 40 60 80 100

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

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m³

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---					
	(SM) Gravelly Silty SAND (TILL) Loose to very loose Grey Wet	14	SS	3		
42.3 10.7	(SP/GP) SAND and GRAVEL Compact Grey Wet	15	SS	13	42	40 50 (10)
41.7 11.3	(GP) Sandy GRAVEL, some silt (TILL) Compact to dense Grey Wet	16	SS	28	41	
		17	SS	26		
		18	SS	15	40	55 28 12 5
		19	SS	29	39	
		20	SS	42	38	
37.7		21	SS	50/0.10		
15.3	Limestone (BEDROCK) Bedrock cored from depths 15.3 m to 21.0 m For bedrock coring detail refer to Record of Drillhole 18-1103	1	RC	REC 75%	37	RQD = 53%
						UCS = 21.5 MPa (unit weight column)
		2	RC	REC 100%	36	RQD = 90%
		3	RC	REC 100%	35, 34	RQD = 99%
		4	RC	REC 100%		RQD = 92%

Water content points (approx.): sample 15 ≈ 10%; sample 17 ≈ 8%; sample 19 ≈ 9%.

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No 18-1103

SHEET 3 OF 4 **METRIC**

PROJECT 1899802-1100
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

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
32.1	Limestone (BEDROCK) Bedrock cored from depths 15.3 m to 21.0 m For bedrock coring detail refer to Record of Drillhole 18-1103		4	RC	REC 100%						RQD = 92%
21.0	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.										

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

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

PROJECT: 1899802-1100

RECORD OF DRILLHOLE: 18-1103

LOCATION: N 4997614.4 ;E 222587.4

DRILLING DATE: September 10-11, 2018

DATUM: Geodetic

DRILL RIG: CME 75

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Downing Drilling

NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	FEATURES
		BEDROCK SURFACE	37.65			
16	Rotary Drill HQ3 Core	Nodular Limestone (BEDROCK), with fresh, thinly laminated to thinly bedded, dark grey to black, fine grained, non-porous, weak shale Fresh Thinly bedded Medium grey Fine grained Non-porous Medium strong	15.34	1	100	UCS = 21.5 MPa
17		- Broken/lost core from 15.34 m to 15.65 m - Broken core from 15.78 m to 15.82 m - Broken core from 15.90 m to 15.94 m - 15 mm clay seam at 16.38 m - Broken core from 16.44 m to 16.48 m - Vertical joint from 16.61 m to 16.73 m		2	100	
18–19		- 60 mm clay seam at 17.91 m		3	100-0	
20		- Broken core from 19.38 m to 19.51 m - Broken core from 20.03 m to 20.04 m	32.04	4	0	
21		END OF DRILLHOLE	20.95			

Column headings also include: SYMBOLIC LOG; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION, Jr, Ja); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WEATHERING INDEX (W1 W2 W3 W4 W5 W6).

Depth scale: 16, 17, 18, 19, 20, 21, 22, 23, 24, 25

DEPTH SCALE 1 : 50

GOLDER

LOGGED: RI

CHECKED: CK

RECORD OF BOREHOLE No 18-1103A

SHEET 1 OF 2 **METRIC**

PROJECT 1899802-1100
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

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
53.1	GROUND SURFACE				53
0.0	(SM) Silty sand, trace gravel, contains organic matter (rootlets) (TOPSOIL/FILL) Dark brown Moist				
0.2	(SM) Gravelly silty sand, contains cobbles and boulders (FILL) Very dense to compact Grey-brown Moist				52, 51, 50, 49
48.8 / 4.3	(SM) Silty sand, some gravel, contains organic matter (rootlets) (FILL) Compact Brown to grey Moist				
48.5 / 4.6	(CH) CLAY, trace sand, highly fissured (WEATHERED CRUST) Very stiff to stiff Grey-brown Moist				48
47.0 / 6.1	(CH) CLAY, contains sand layers and gravel Firm Grey Moist to wet	1	SS	3	47
		2	TP	PH	46, 45
44.9 / 8.2	(SM) Gravelly SILTY SAND, contains cobbles and boulders (TILL) Loose to compact Grey Wet	3	SS	7	44

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL; STRAT PLOT; GROUND WATER CONDITIONS.

Continued Next Page

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

GOLDER

RECORD OF BOREHOLE No 18-1103A

METRIC

PROJECT 1899802-1100

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

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	(SM) Gravelly SILTY SAND, contains cobbles and boulders (TILL) Loose to compact Grey Wet						43				
			4	SS	16		42				
41.8 11.3	END OF BOREHOLE 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.										

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

$○^{3\%}$ STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No 18-1103B

SHEET 1 OF 1 **METRIC**

PROJECT 1899802-1100

G.W.P. 4248-15-00

LOCATION N 4997611.2; E 222587.4 NAD MTM ZONE 8 (LAT. 45.113700; LONG. -74.544860)

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

ELEV / DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS
52.9	GROUND SURFACE				
0.0	(SM) Silty sand, trace gravel, contains organic matter (rootlets) (TOPSOIL/FILL) Dark brown Moist				
0.2	(SM) Gravelly silty sand, contains cobbles and boulders (FILL) Very dense to compact Grey-brown Moist				
48.6 / 4.3	(SM) Silty sand, some gravel, contains organic matter (rootlets) (FILL) Compact Brown to grey Moist				
48.3 / 4.6	(CH) CLAY, trace sand, highly fissured (WEATHERED CRUST) Very stiff to stiff Grey-brown Moist				
46.8 / 6.1	(CH) CLAY, contains silt layers and gravel Firm Grey Moist to wet	1	TP	PH	
		2	SS	50/0.15	
45.1 / 7.8	END OF BOREHOLE AUGER REFUSAL				

NOTES:

1. Soil stratigraphy from 0.0 to 6.1 m inferred from 18-1103.

2. Vane refusal was encountered at 6.7 m due possibly to presence of gravel.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL; GROUND WATER CONDITIONS; ELEVATION SCALE (52, 51, 50, 49, 48, 47, 46); STRAT PLOT.

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

APPENDIX B

Laboratory Test Results - Current Investigation

Figure B1 - Grain Size Distribution Test Results – Silty Sand and Gravel to Gravelly Sandy Silt (Fill)

Figure B2 - Grain Size Distribution Test Results – Sandy Silt

Figure B3 - Plasticity Chart – Clay (Weathered Crust)

Figure B4 - Plasticity Chart – Clay

Figures B5 to B8 - Consolidation Test Results

Figure B9 - Grain Size Distribution Test Results – Silt and Sand (Till)

Figure B10 - Grain Size Distribution Test Results – Sand and Gravel to Sandy Gravel (Till)

Figure B11 - Grain Size Distribution Test Results – Sand and Gravel

Figure B12 - Summary of Laboratory Compressive Strength Testing

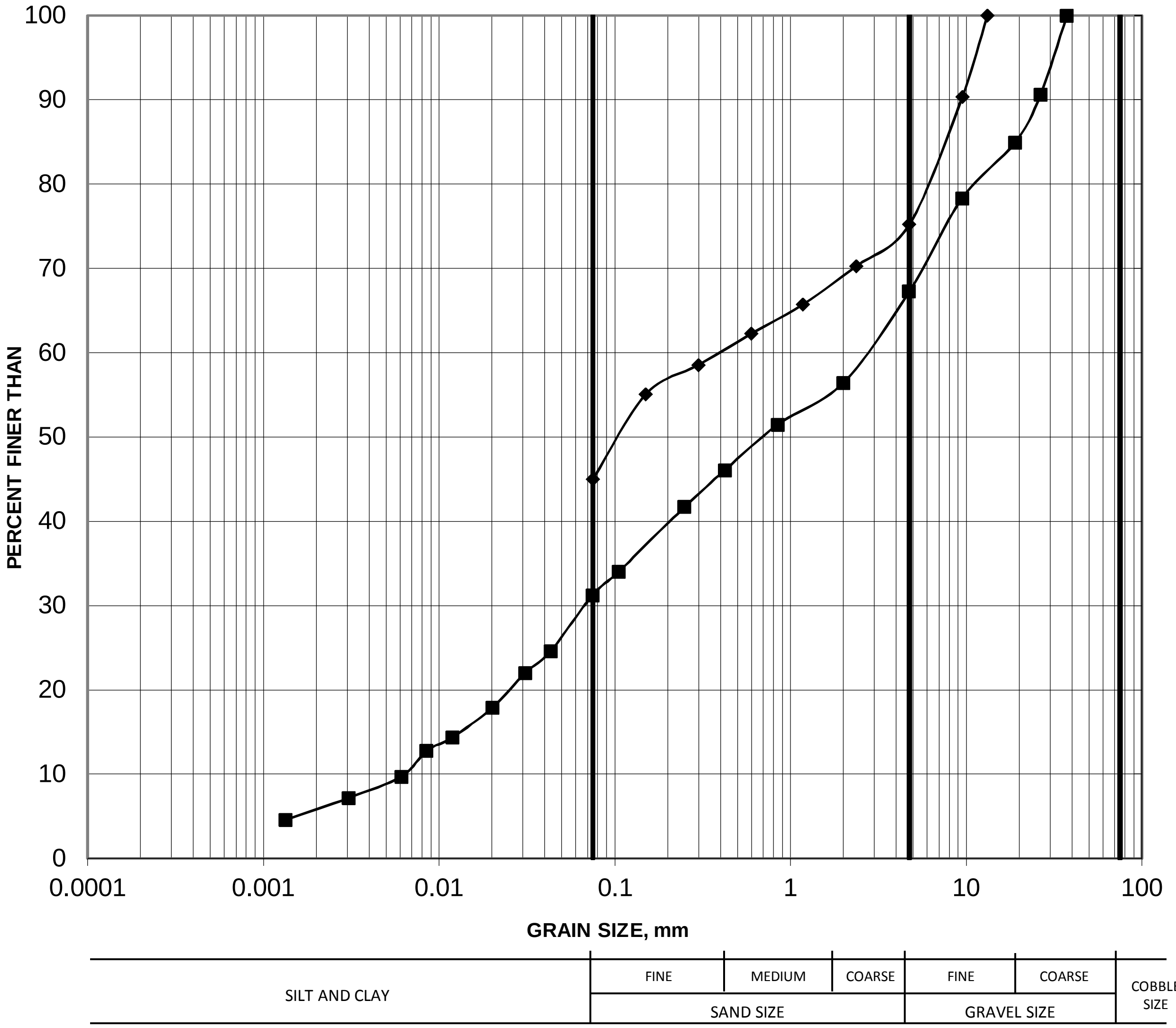
Figure B13 - Summary of Engineering Properties

Figures B14 to B16 - Bedrock Core Photographs

Unconfined Compression Test Results

GRAIN SIZE DISTRIBUTION	FIGURE B1

SILTY SAND AND GRAVEL TO GRAVELLY SANDY SILT (FILL)



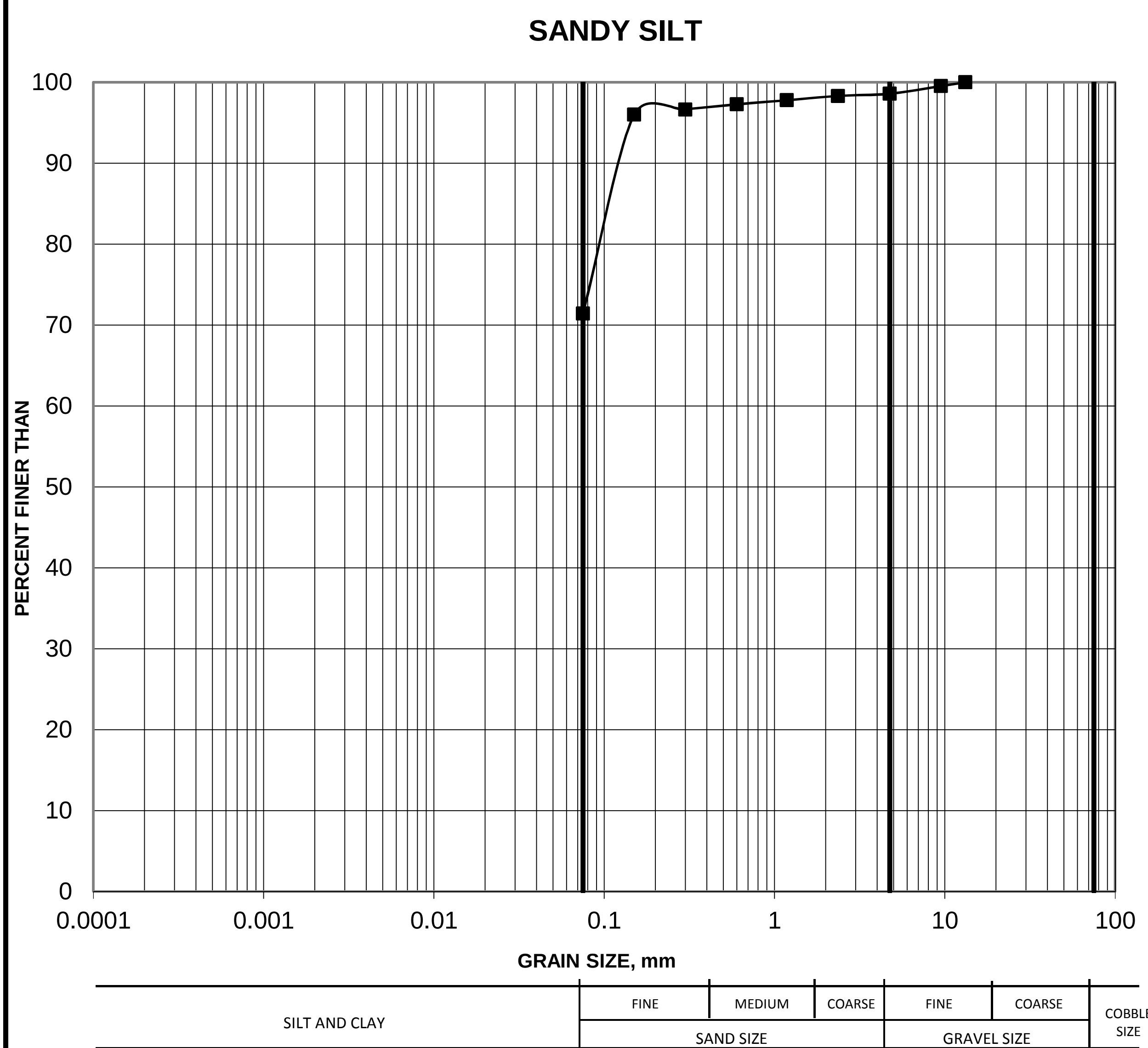
Borehole	Sample	Depth (m)
18-1101	3	1.52-2.13
18-1102	1B	0.23-0.61

Project: 1899802/1100 **Golder Associates**

Created by: MI
Checked by: AC

GRAIN SIZE DISTRIBUTION

FIGURE B2



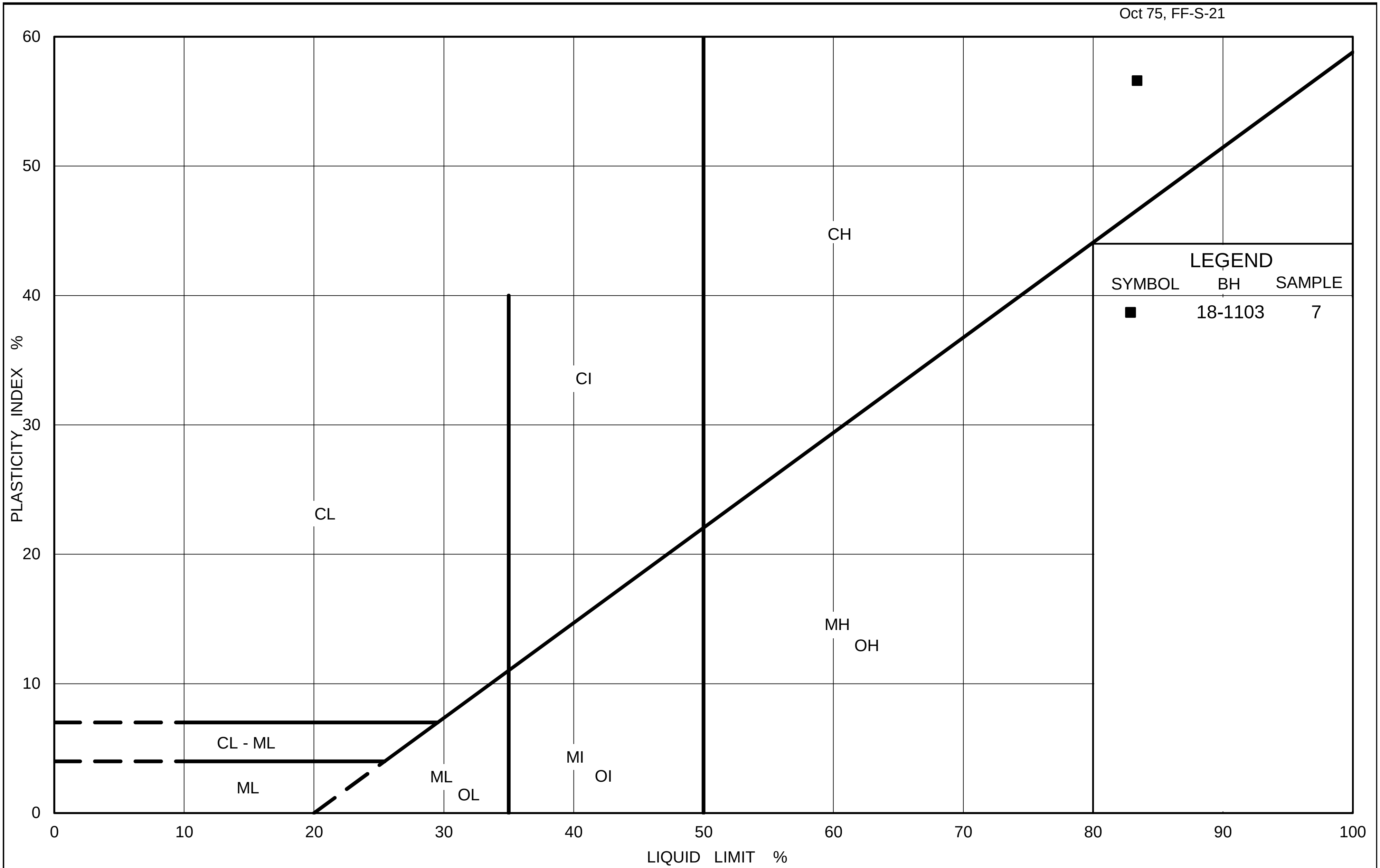
Borehole	Sample	Depth (m)
18-1102	3	1.52-2.13

Project: 1899802/1100

Golder Associates

Created by: MI

Checked by: AC



Ministry of Transportation

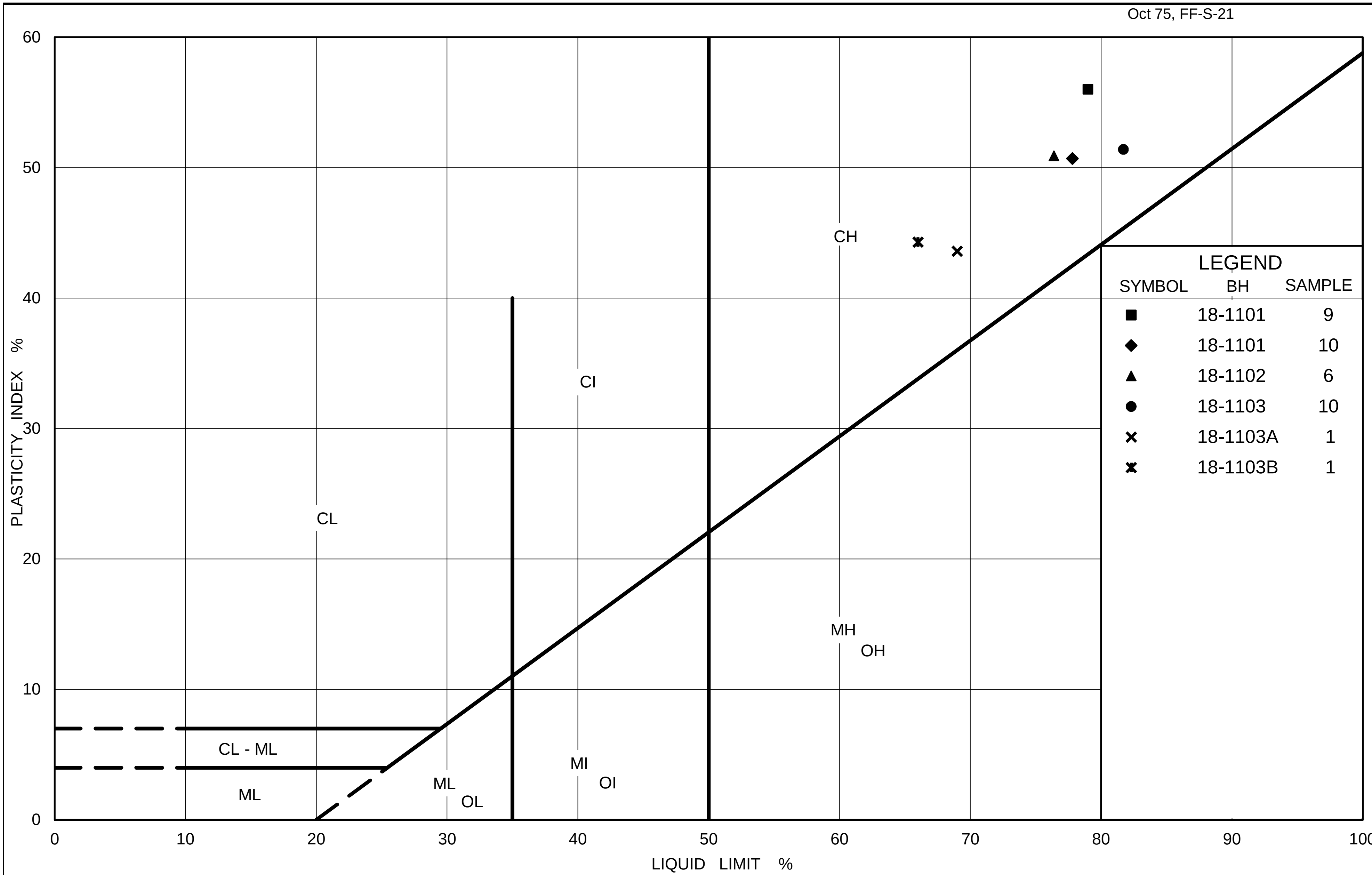
Ontario

PLASTICITY CHART
CLAY (WEATHERED CRUST)

FIG No. B3

Project No. 1899802/1100

Compiled By : MI Checked By : AC



SYMBOL	BH	SAMPLE
■	18-1101	9
◆	18-1101	10
▲	18-1102	6
●	18-1103	10
×	18-1103A	1
✖	18-1103B	1

Ministry of Transportation

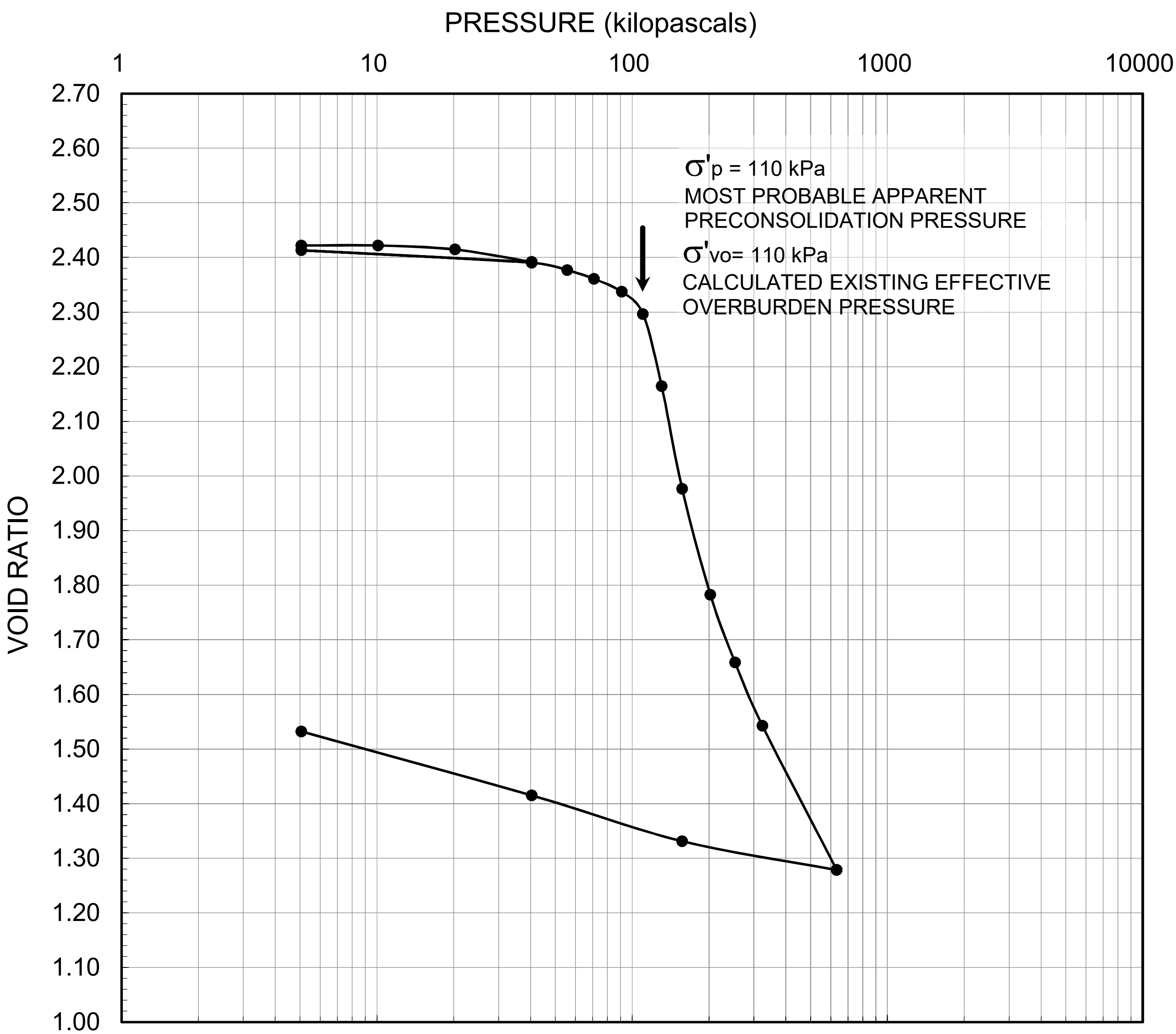
Ontario

PLASTICITY CHART
CLAY

FIG No. B4

Project No. 1899802/1100

Compiled By : MI Checked By : CM

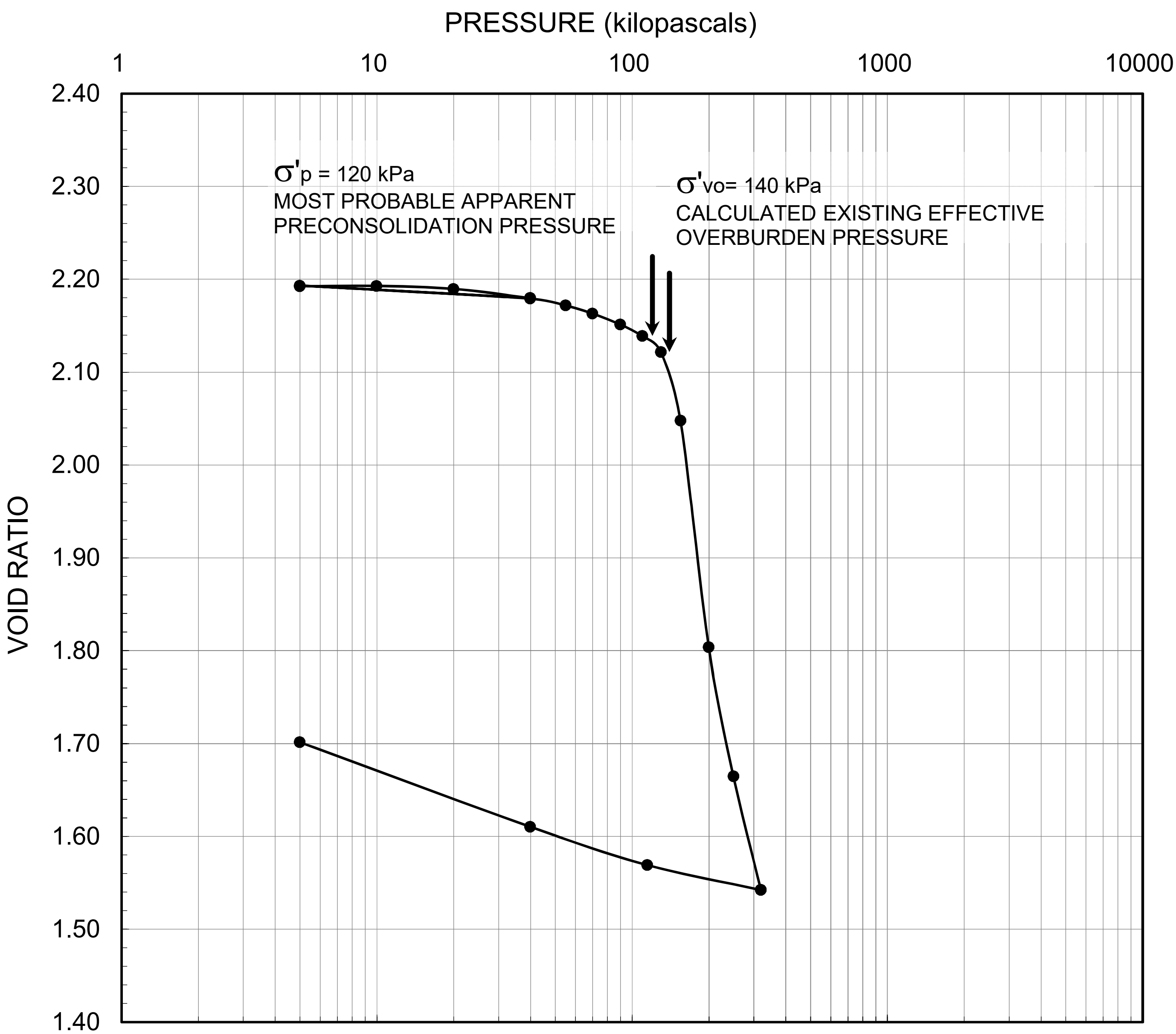


LEGEND

Borehole: 18-1101	w_i = 86%	S_o = 99%	g = 14.9 kN/m³
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	CONSOLIDATION TEST RESULTS
DATE	02-25-19		
CADD	N/A		
ENTERED	MI		
FILE No.	Consolidation summary	CHECK	CW
PROJECT No.	1899802/1100	REV.	0
REVIEW	CK	FIGURE	B5

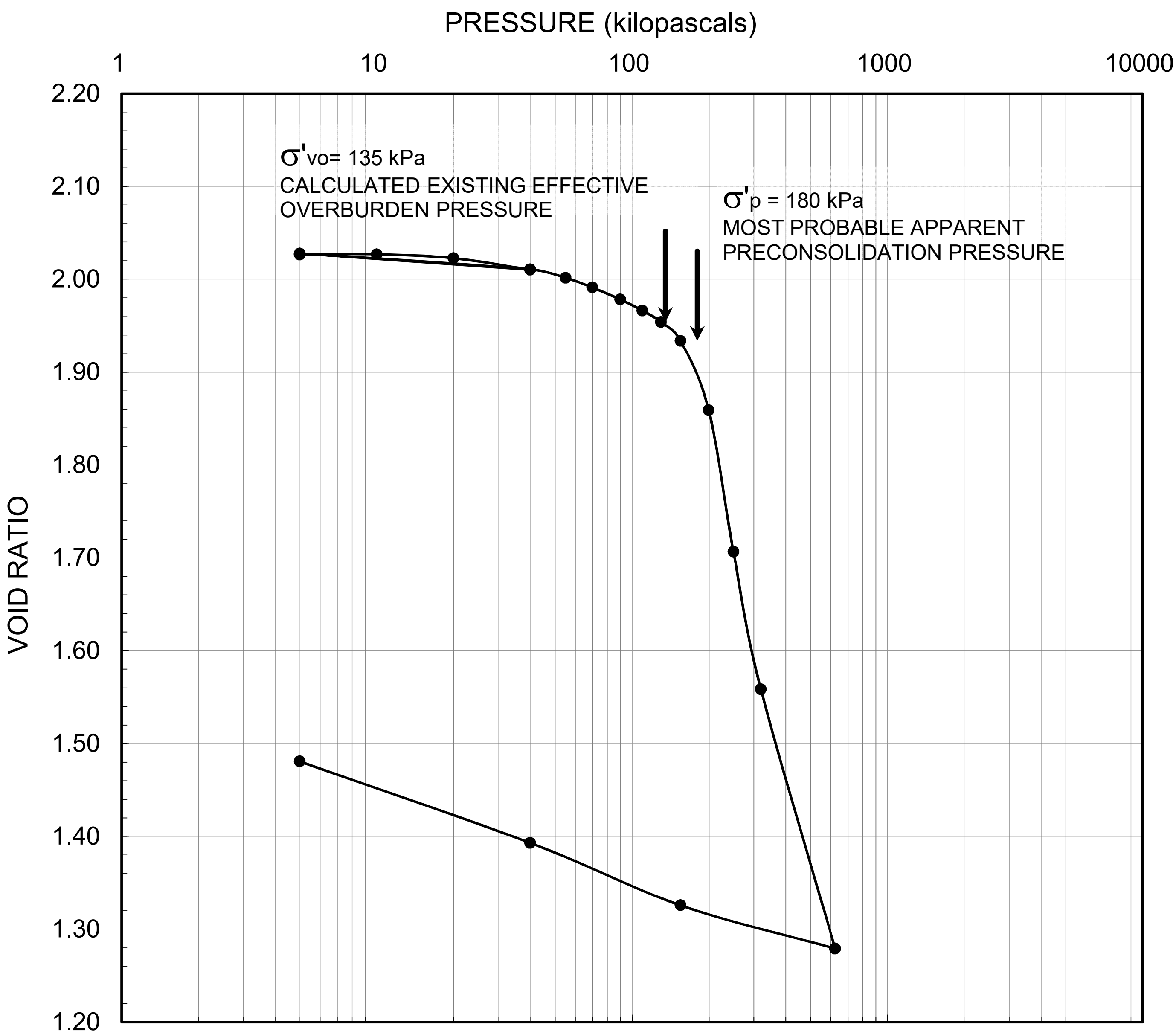


LEGEND

Borehole: 18-1101	w_i = 79%	S_o = 100%	g = 15.2 kN/m³
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	CONSOLIDATION TEST RESULTS
DATE	02-25-19		
CADD	N/A		
ENTERED	MI		
FILE No. Consolidation summary	CHECK CW		
PROJECT No. 1899802/1100 REV. 0	REVIEW CK	FIGURE	B6



LEGEND

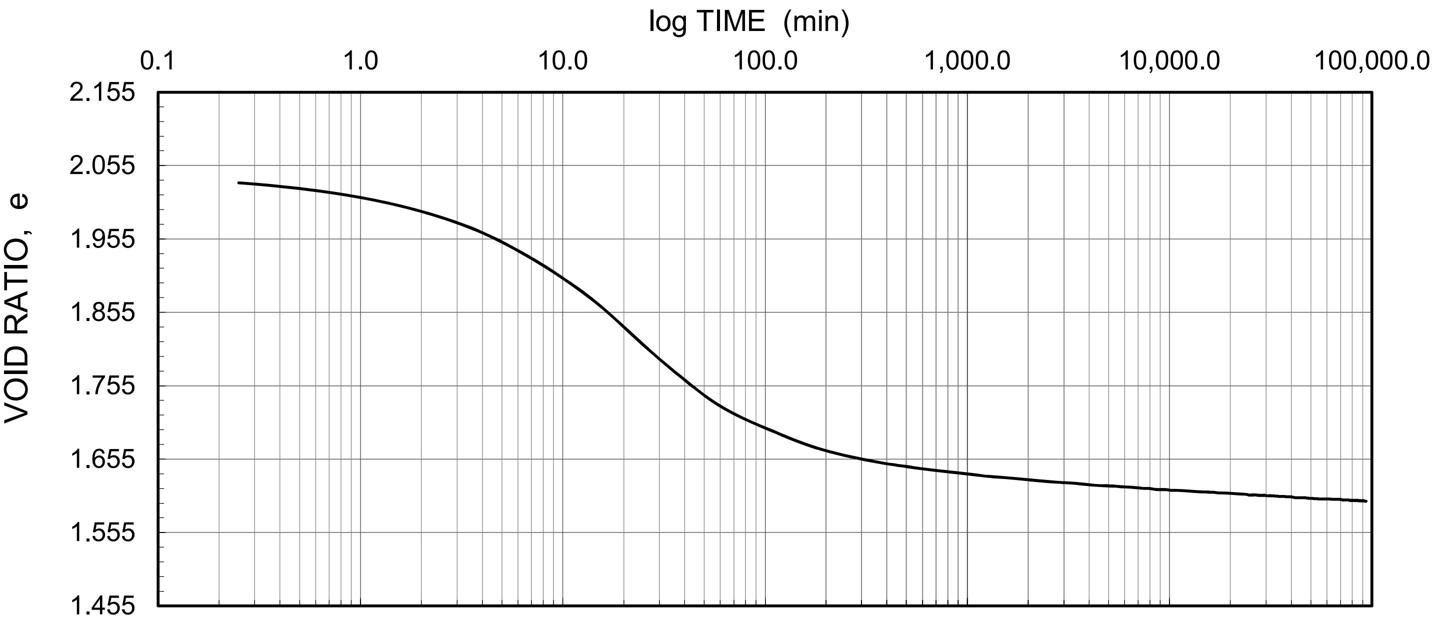
Borehole:	18-1103B	w_i = 72%	S_o = 100%	g = 15.6 kN/m³
Sample:	1	w_f = 54%	e_o = 2.02	G_s = 2.80
Depth (m):	7.6	w_l = 66%	C_c = 1.60	
Elevation (m):	45.4	w_p = 22%	C_r = 0.019	

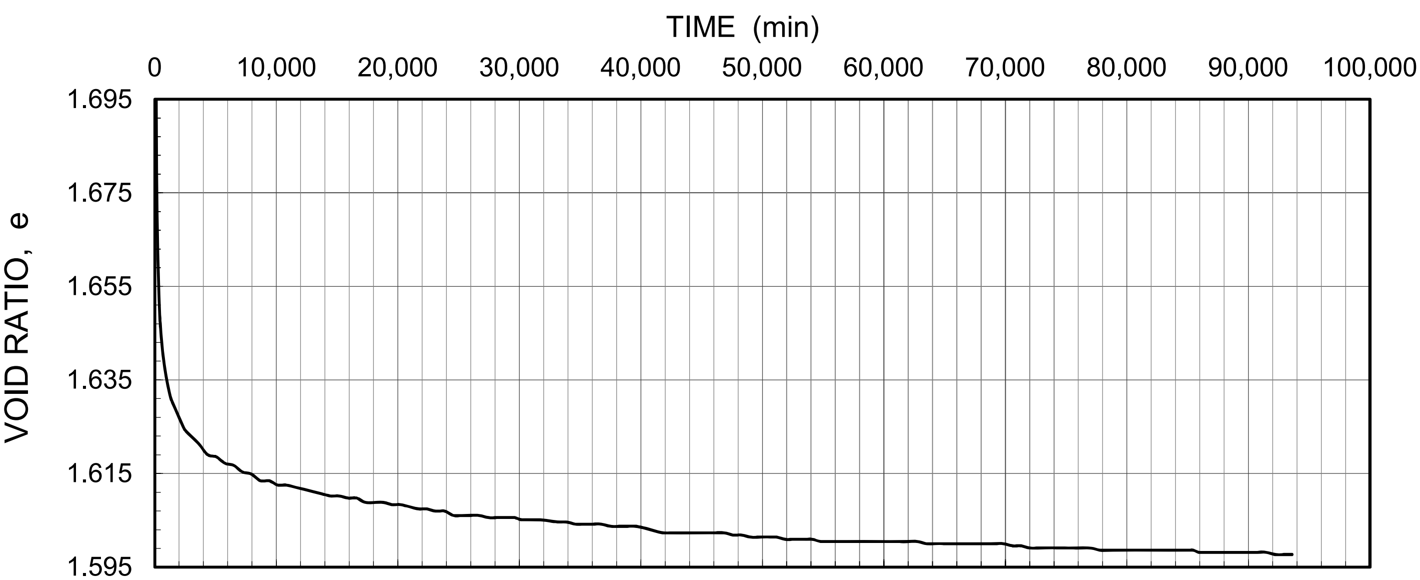
GOLDER

SCALE	AS SHOWN	TITLE	
DATE	02-25-19		
CADD	N/A		
ENTERED	MI		
FILE No.	Consolidation summary	CHECK	CW
PROJECT No.	1899802/1100	REV.	0
REVIEW	CK	FIGURE	B7

CONSOLIDATION TEST RESULTS

PRESSURE = 245 kPa





LEGEND

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

GOLDER

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

TITLE

SUMMARY OF SECONDARY COMPRESSION TEST

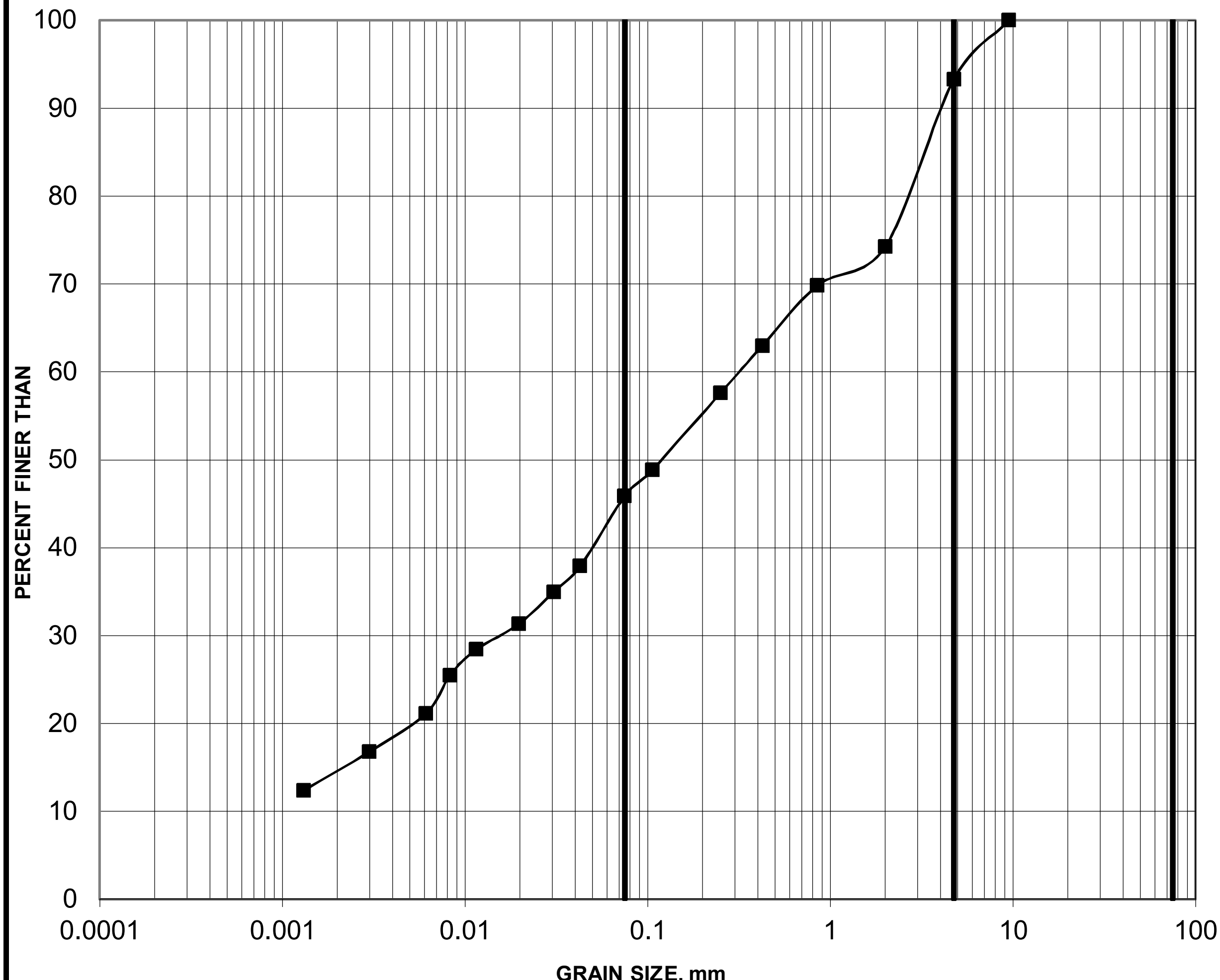
PROJECT No. 1899802/1100 REV. 0

FIGURE **B8**

GRAIN SIZE DISTRIBUTION

FIGURE B9

SILT AND SAND (TILL)



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

Borehole	Sample	Depth (m)
18-1102	8A	7.01-7.32

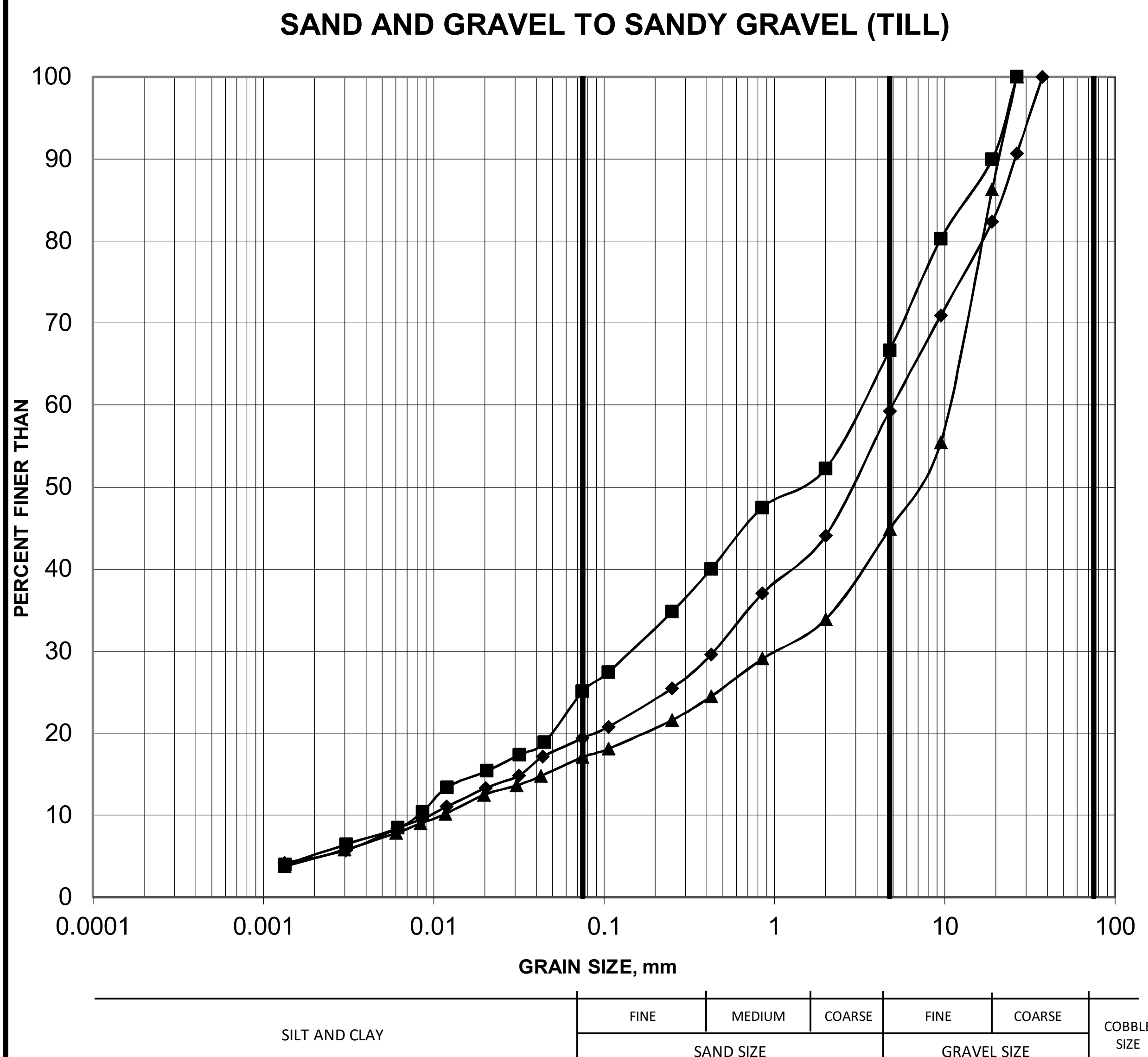
Project: 1899802/1100

Golder Associates

Created by: MI

Checked by: CW

GRAIN SIZE DISTRIBUTION	FIGURE B10



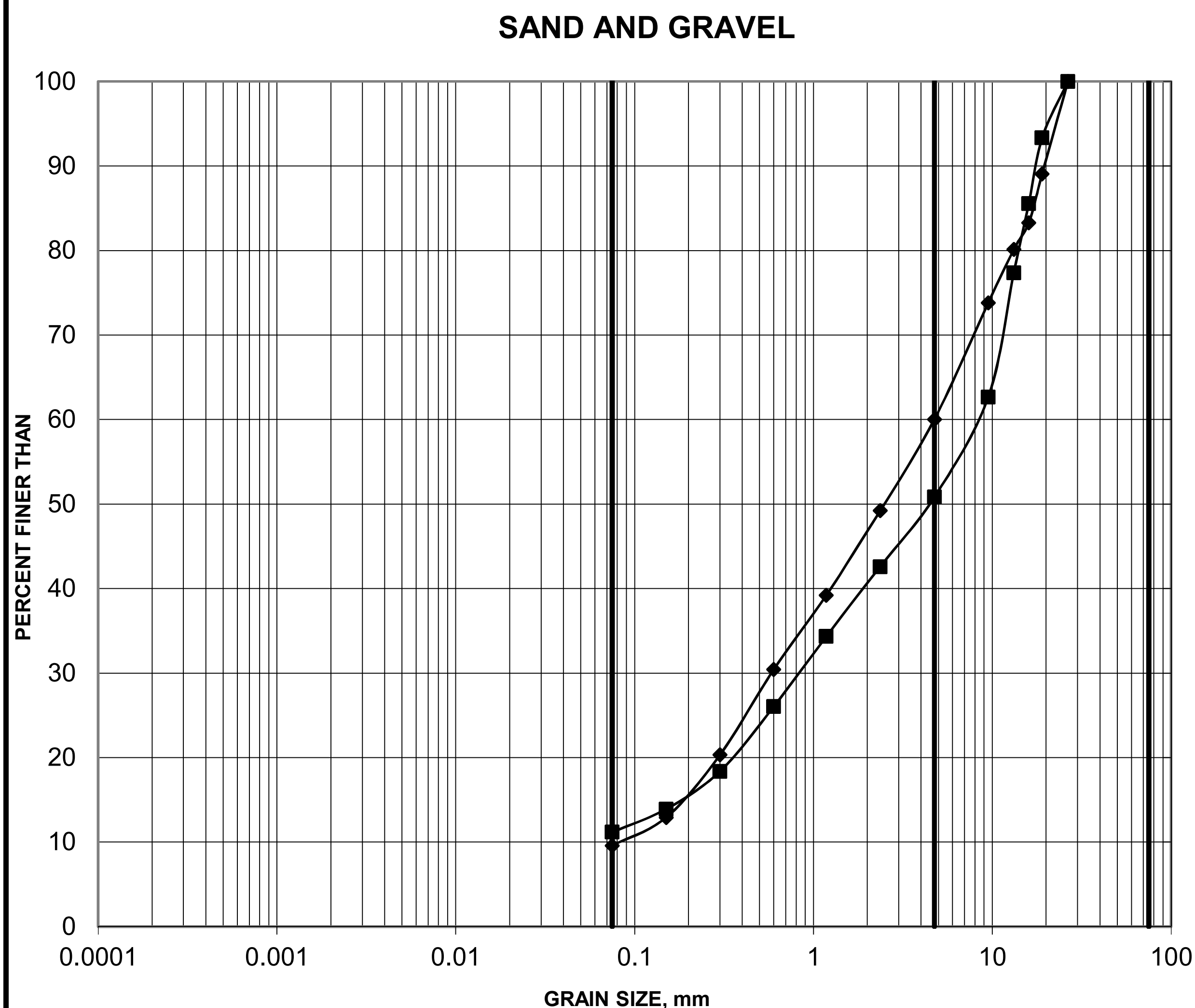
Borehole	Sample	Depth (m)
18-1101	13	10.67-11.28
18-1102	14	11.43-12.04
18-1103	18	12.95-13.56

Project: 1899802/1100

Golder Associates

Created by: MI
Checked by: CW

GRAIN SIZE DISTRIBUTION	FIGURE B11



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

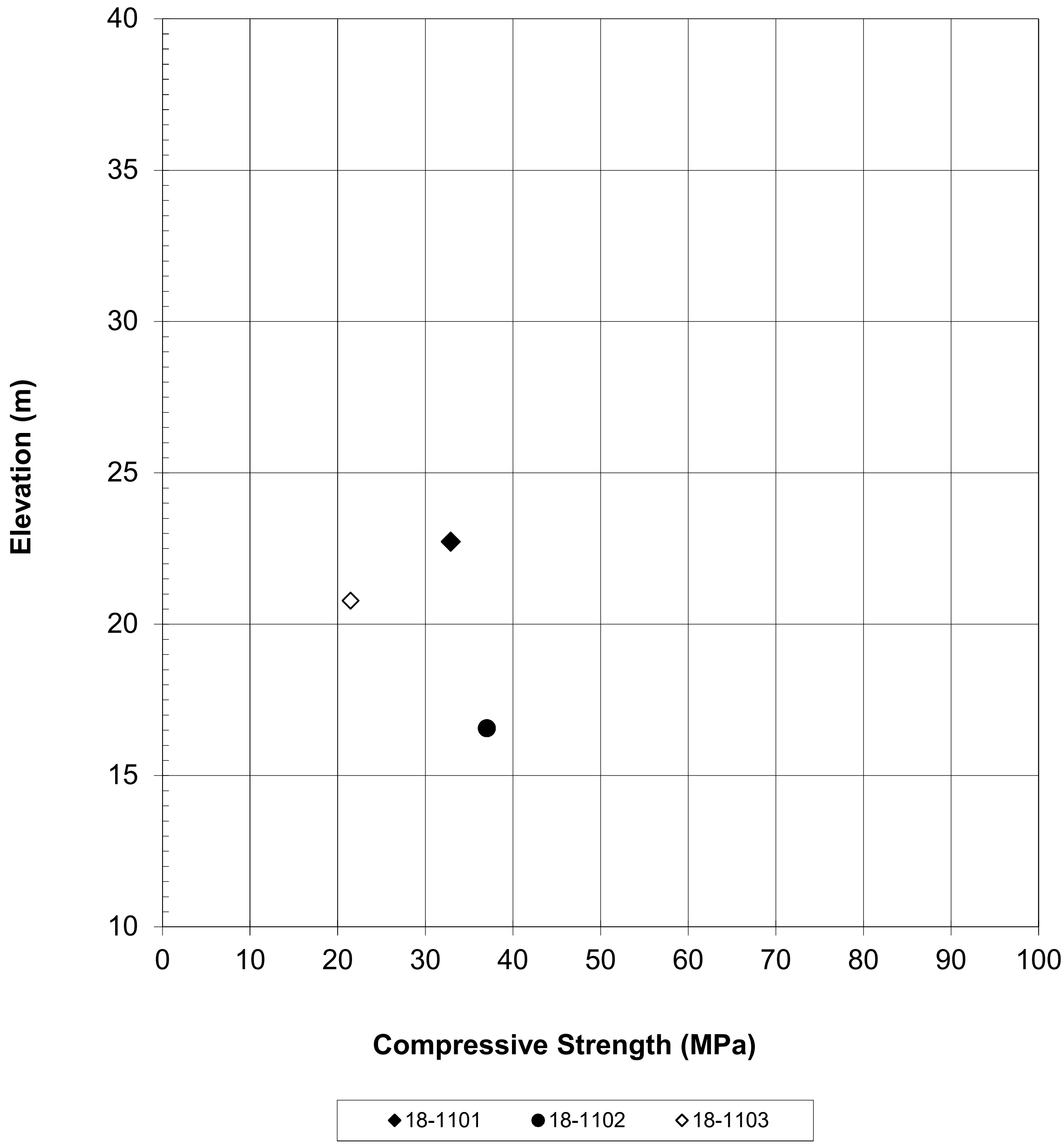
Borehole	Sample	Depth (m)
18-1102	10	8.38-8.99
18-1103	15	10.67-11.28

Project: 1899802/1100

Golder Associates

Created by: MI
Checked by: CW

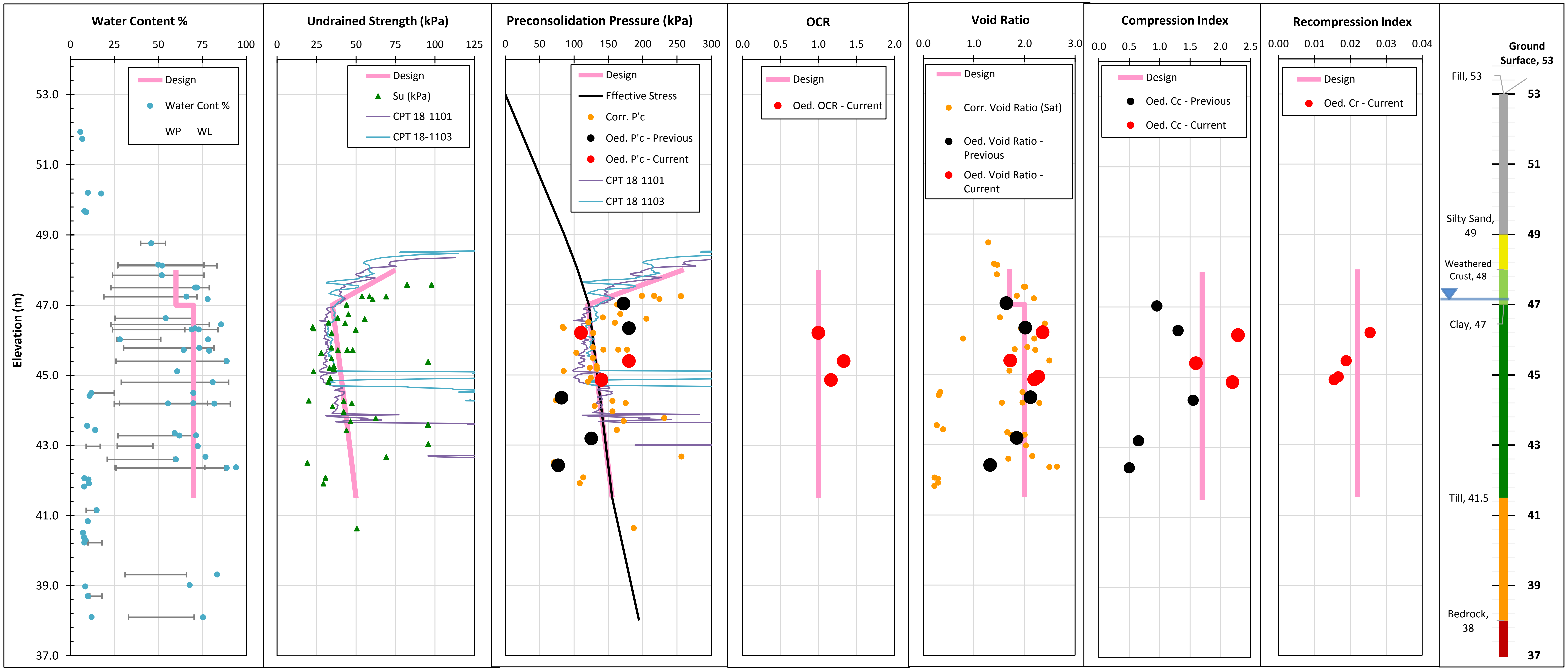
SUMMARY OF LABORATORY COMPRESSIVE STRENGTH UNCONFINED COMPRESSION TESTS	FIGURE B12



Project: 1899802 /1100

GOLDER

Created by: MI
Checked by: CW



GOLDER

Foundation Investigation and Preliminary Design

Replacement of Fraser Road Underpass at Highway 401
United Counties of Stormont, Dundas and Glengarry

Summary of Engineering Properties

Project No.:	1899802 - 1100
Date:	May-28-19
Drawn:	RK
Review:	CK

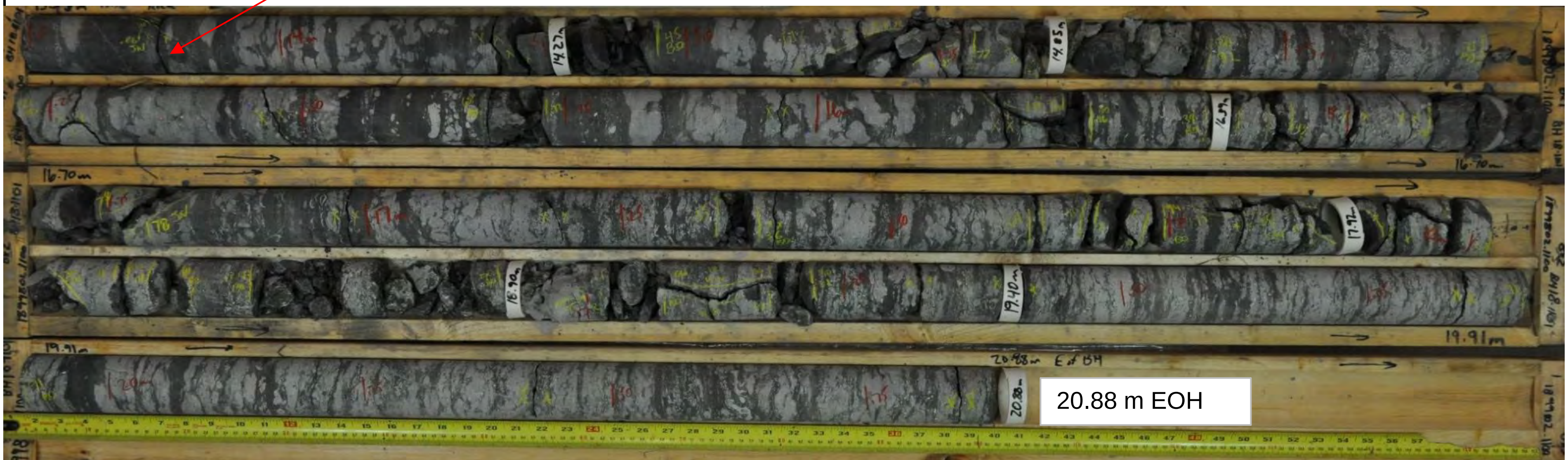
FIGURE B13

Path: https://golderassociates.sharepoint.com/sites/25312g/Deliverables/1100 - Fraser Road/05-Final/Appendices/App B - Current Lab/1899802-1100 - FIGURE B14-16 - Core Photos Plate.xlsx | File: 1899802-1100 - FIGURE B14-16 - Core Photos Plate.xlsx

BH 18-1101 (Dry)
Cored Length of 13.59 to 20.88 metres
Core Box 1 to 3 of 3

13.59 to 13.73 m Till

13.73 m Top of bedrock



CLIENT	
MINISTRY OF TRANSPORTATION ONTARIO (MTO)	
CONSULTANT	
DD/MM/YYYY	01/10/2018
PREPARED	KM
DESIGN	KM
REVIEW	CK
APPROVED	WC

GOLDER

PROJECT
FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN REPLACEMENT OF FRASER ROAD UNDERPASS AT HWY 401, UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON

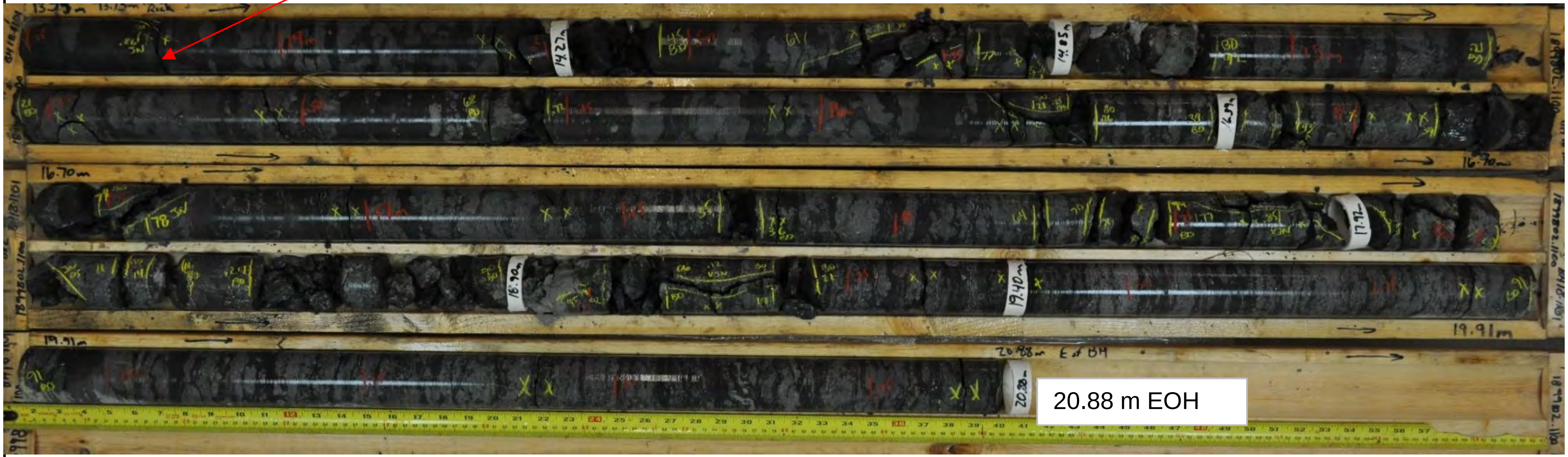
TITLE
BOREHOLE 18-1101 (DRY)
CORE PHOTOGRAPHS

PROJECT No.	PHASE	Rev.	FIGURE
1783774	1100	0	B14A

BH 18-1101 (Wet)
Cored Length of 13.59 to 20.88 metres
Core Box 1 to 3 of 3

13.59 to 13.73 m Till

13.73 m Top of bedrock



20.88 m EOH

CLIENT	MINISTRY OF TRANSPORTATION ONTARIO (MTO)		
PROJECT	FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN REPLACEMENT OF FRASER ROAD UNDERPASS AT HWY 401, UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON		
CONSULTANT	GOLDER		
DD/MM/YYYY	01/10/2018		
PREPARED	KM		
DESIGN	KM		
REVIEW	CK		
APPROVED	WC		

TITLE
BOREHOLE 18-1101 (WET)
CORE PHOTOGRAPHS

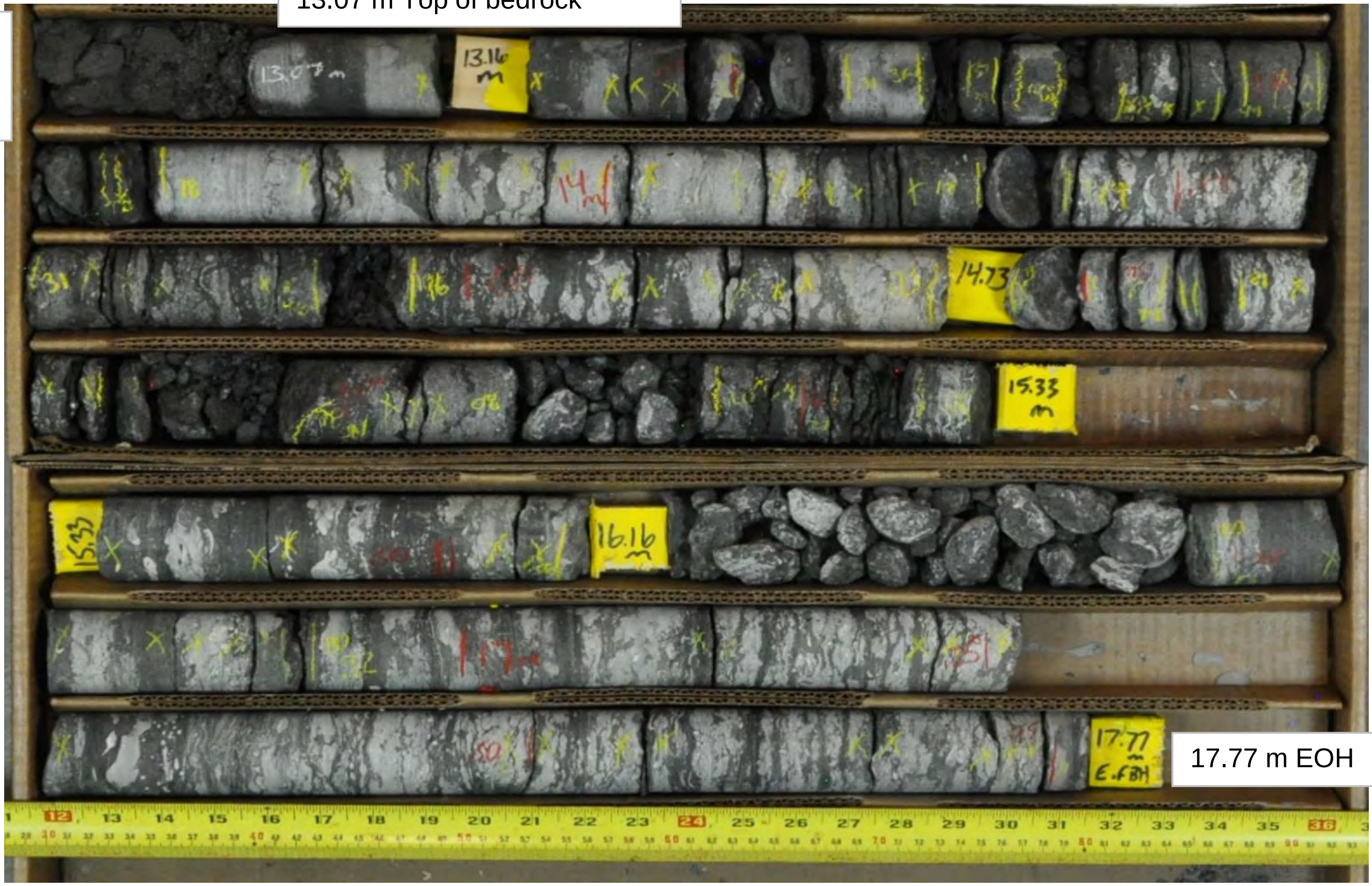
PROJECT No.	PHASE	Rev.	FIGURE
1783774	1100	0	B14B

Path: https://golderassociates.sharepoint.com/sites/25312g/Deliverables/1100 - Fraser Road/05-Final/Appendices/App B - Current Lab/1899802-1100 - FIGURE B14-16 - Core Photos Plate.xlsx | File: 1899802-1100 - FIGURE B14-16 - Core Photos Plate.xlsx

BH 18-1102 (Dry)
Cored Length of 13.01 to 17.77 metres
Core Box 1 to 2 of 2

13.07 m Top of bedrock

13.01 to 13.07 m Glacial Till



17.77 m EOH

CLIENT	PROJECT
MINISTRY OF TRANSPORTATION ONTARIO (MTO)	FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN REPLACEMENT OF FRASER ROAD UNDERPASS AT HWY 401, UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON

CONSULTANT: GOLDER

DD/MM/YYYY	01/10/2018
PREPARED	KM
DESIGN	KM
REVIEW	
APPROVED	

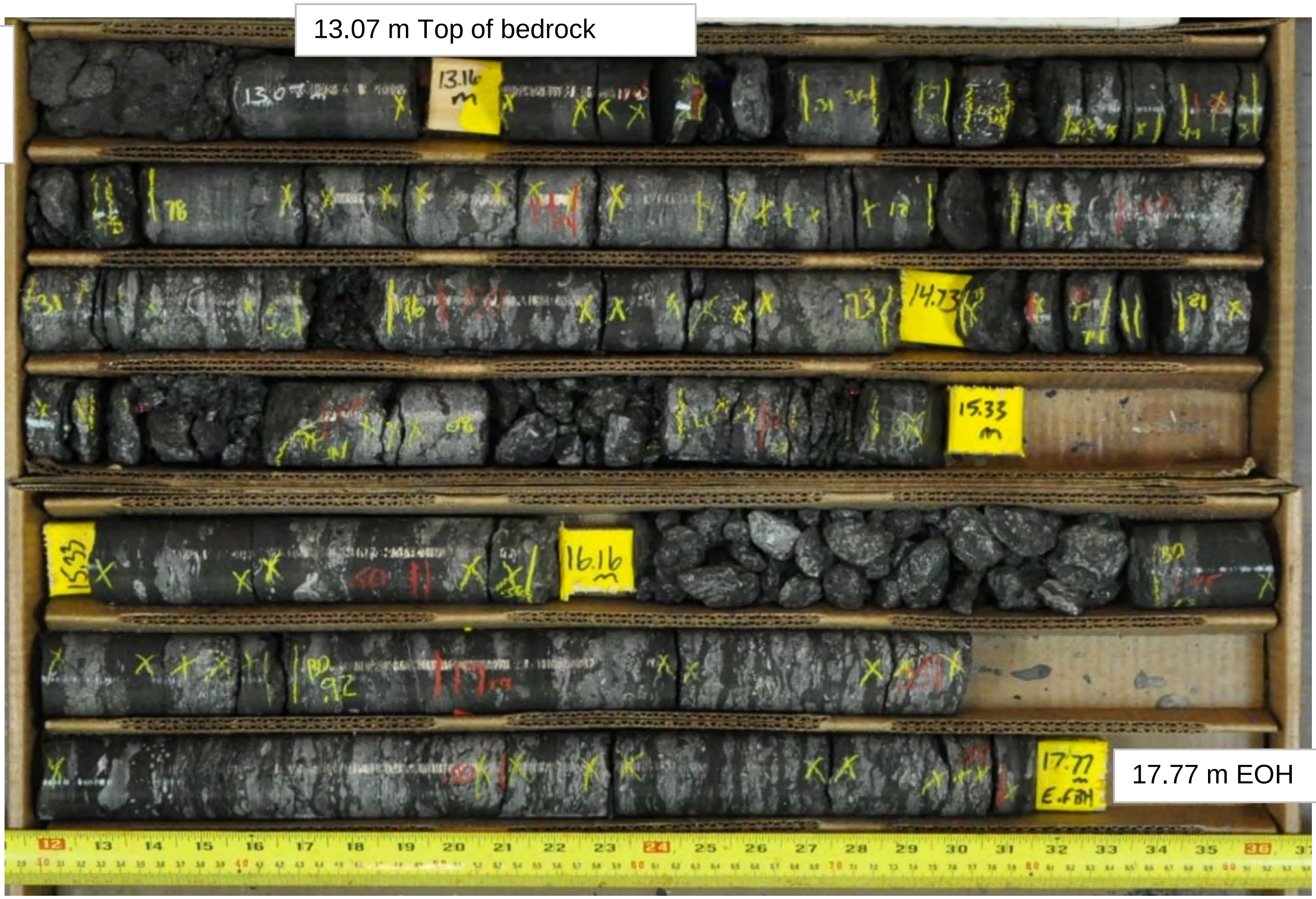
TITLE
BOREHOLE 18-1102 (DRY)
CORE PHOTOGRAPHS

PROJECT No.	PHASE	Rev.	FIGURE
1899802	1100	A	B15A

BH 18-1102 (Wet)
Cored Length of 13.01 to 17.77 metres
Core Box 1 to 2 of 2

13.01 to 13.07 m Glacial Till

13.07 m Top of bedrock

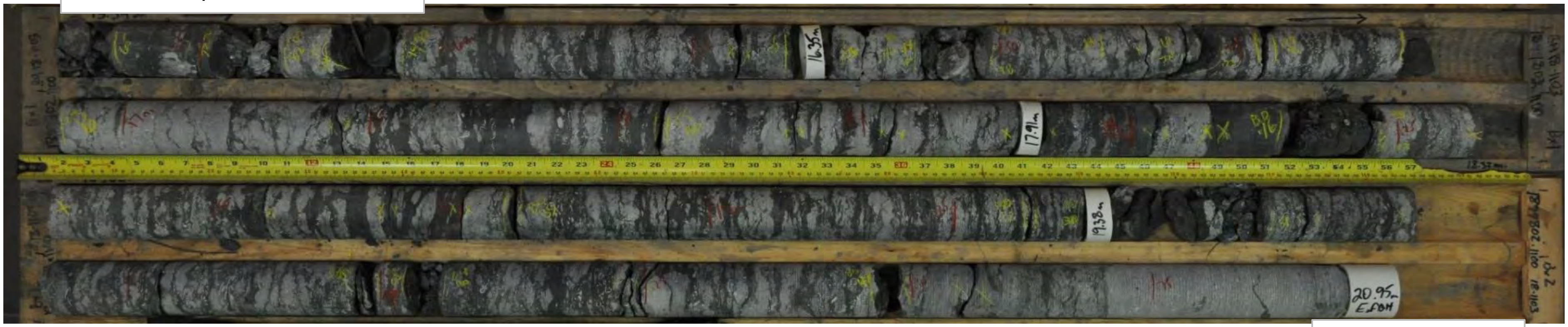


17.77 m EOH

CLIENT MINISTRY OF TRANSPORTATION ONTARIO (MTO)	PROJECT FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN REPLACEMENT OF FRASER ROAD UNDERPASS AT HWY 401, UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON
CONSULTANT GOLDER DD/MM/YYYY 01/10/2018 PREPARED KM DESIGN KM REVIEW APPROVED	TITLE **BOREHOLE 18-1102 (WET)** **CORE PHOTOGRAPHS** PROJECT No. 1899802 — PHASE 1100 — Rev. A — FIGURE B15B

BH 18-1103 (Dry)
Cored Length of 15.34 to 20.95 metres
Core Box 1 to 2 of 2

15.34 m Top of bedrock



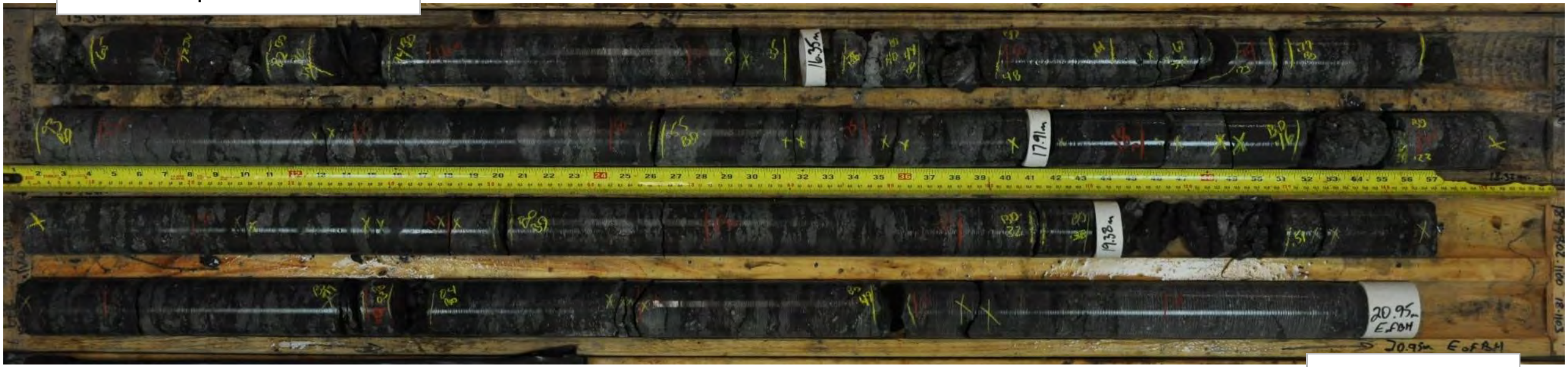
20.95 m EOH

CLIENT	MINISTRY OF TRANSPORTATION ONTARIO (MTO)
PROJECT	FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN REPLACEMENT OF FRASER ROAD UNDERPASS AT HWY 401, UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON
CONSULTANT	GOLDER
DD/MM/YYYY	01/10/2018
PREPARED	KM
DESIGN	KM
REVIEW	CK
APPROVED	WC
TITLE	**BOREHOLE 18-1103 (DRY) CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1783774	1100	0	B16A

BH 18-1103 (Wet)
Cored Length of 15.34 to 20.95 metres
Core Box 1 to 2 of 2

15.34 m Top of bedrock



20.95 m EOH

CLIENT MINISTRY OF TRANSPORTATION ONTARIO (MTO)	PROJECT FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN REPLACEMENT OF FRASER ROAD UNDERPASS AT HWY 401, UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON
CONSULTANT: GOLDER DD/MM/YYYY 01/10/2018 PREPARED KM DESIGN KM REVIEW CK APPROVED WC	TITLE **BOREHOLE 18-1103 (WET)** **CORE PHOTOGRAPHS** PROJECT No. 1783774 · PHASE 1100 · Rev. 0 · FIGURE B16B

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1899802 (1100)	SAMPLE NUMBER	-
PROJECT NAME	Dillon 4017-E-0019/0020 Eastern Reg	SAMPLE DEPTH, m	14.92-15.21
BOREHOLE NUMBER	18-1101	DATE:	11/10/2018

TEST CONDITIONS

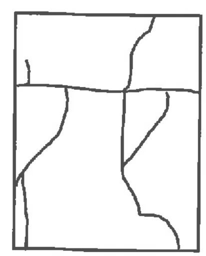
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.28

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.85	WATER CONTENT, (specimen) %	1.10
SAMPLE DIAMETER, cm	6.08	UNIT WEIGHT, kN/m^3	26.09
SAMPLE AREA, cm^2	29.03	DRY UNIT WT., kN/m^3	25.80
SAMPLE VOLUME, cm^3	402.03	SPECIFIC GRAVITY	-
WET WEIGHT, g	1069.81	VOID RATIO	-
DRY WEIGHT, g	1058.17		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	32.9

REMARKS:

Checked By: m

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE

BEFORE COMPRESSION

AFTER COMPRESSION

Date Oct.15, 2018
Project 1899802-1100

Golder Associates

Drawn Frank
Chkd.

GOLDER

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1899802 (1100)	SAMPLE NUMBER	-
PROJECT NAME	Dillon 4017-E-0019/0020 Eastern Reg	SAMPLE DEPTH, m	16.92-17.11
BOREHOLE NUMBER	18-1102	DATE:	11/10/2018

TEST CONDITIONS

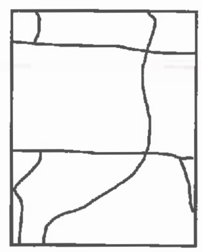
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.36

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.53	WATER CONTENT, (specimen) %	1.00
SAMPLE DIAMETER, cm	4.46	UNIT WEIGHT, kN/m^3	26.34
SAMPLE AREA, cm^2	15.64	DRY UNIT WT., kN/m^3	26.07
SAMPLE VOLUME, cm^3	164.61	SPECIFIC GRAVITY	-
WET WEIGHT, g	442.22	VOID RATIO	-
DRY WEIGHT, g	437.84		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	37.0

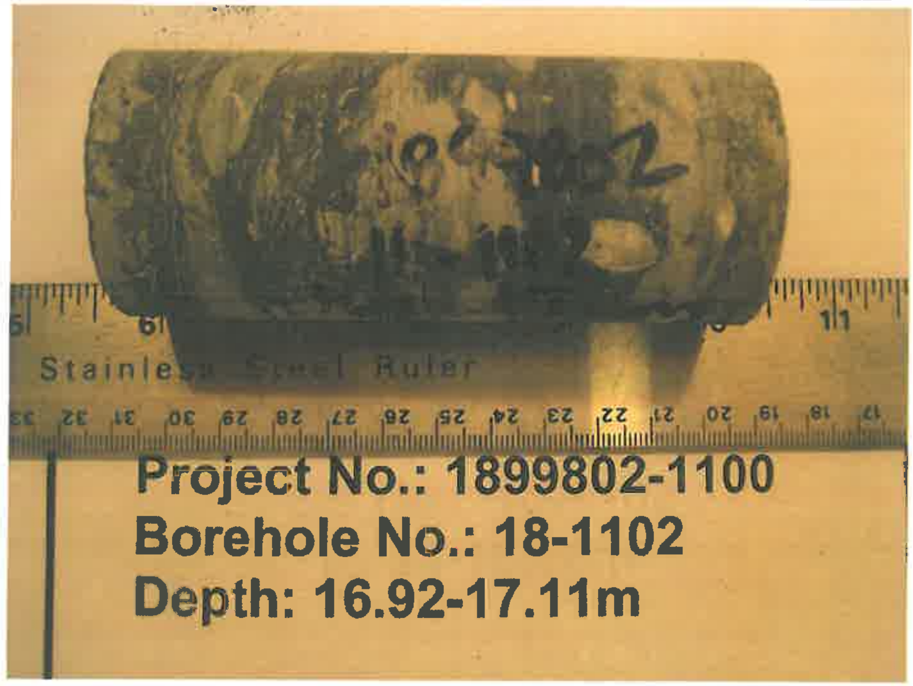
REMARKS:

Checked By: LM

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE



BEFORE COMPRESSION

AFTER COMPRESSION

Date Oct.15, 2018
Project 1899802-1100

Golder Associates

Drawn Frank
Chkd.

GOLDER

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

ASTM D7012

SAMPLE IDENTIFICATION

PROJECT NUMBER	1899802 (1100)	SAMPLE NUMBER	-
PROJECT NAME	Dillon 4017-E-0019/0020 Eastern Reg	SAMPLE DEPTH, m	15.94-16.28
BOREHOLE NUMBER	18-1103	DATE:	11/10/2018

TEST CONDITIONS

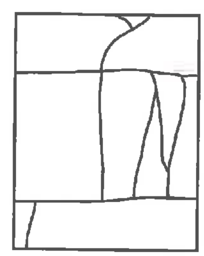
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.25

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.65	WATER CONTENT, (specimen) %	1.10
SAMPLE DIAMETER, cm	6.07	UNIT WEIGHT, kN/m^3	26.01
SAMPLE AREA, cm^2	28.96	DRY UNIT WT., kN/m^3	25.73
SAMPLE VOLUME, cm^3	395.18	SPECIFIC GRAVITY	-
WET WEIGHT, g	1048.49	VOID RATIO	-
DRY WEIGHT, g	1037.08		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	21.5

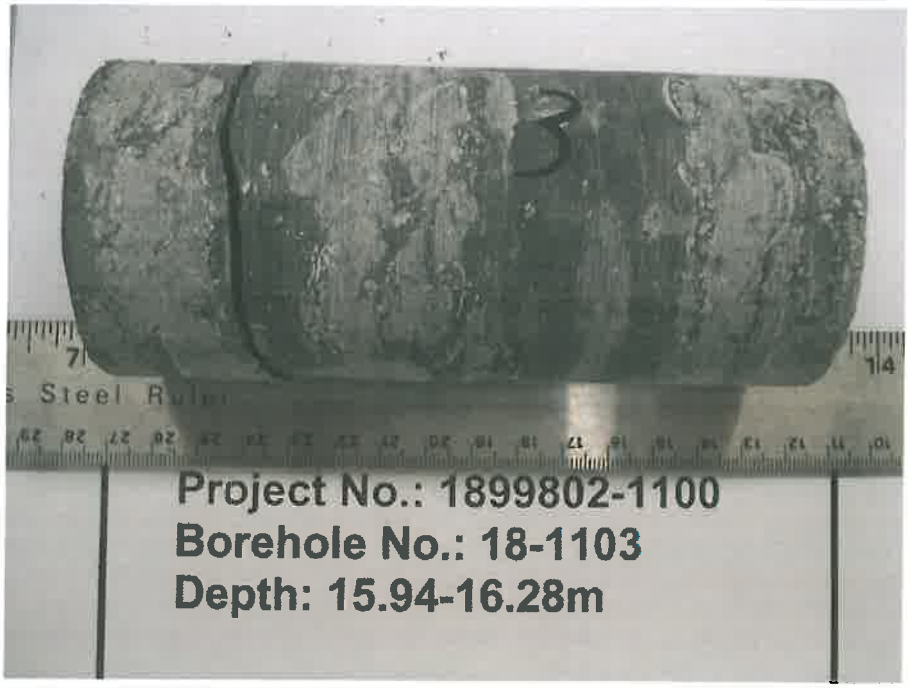
REMARKS:

Checked By: [signature]

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE



BEFORE COMPRESSION

AFTER COMPRESSION

Date Oct.15, 2018
Project 1899802-1100

Golder Associates

Drawn Frank
Chkd.

APPENDIX C

Borehole Records and Laboratory Test Results – Previous Investigation (Geocres No. 31G00-142)

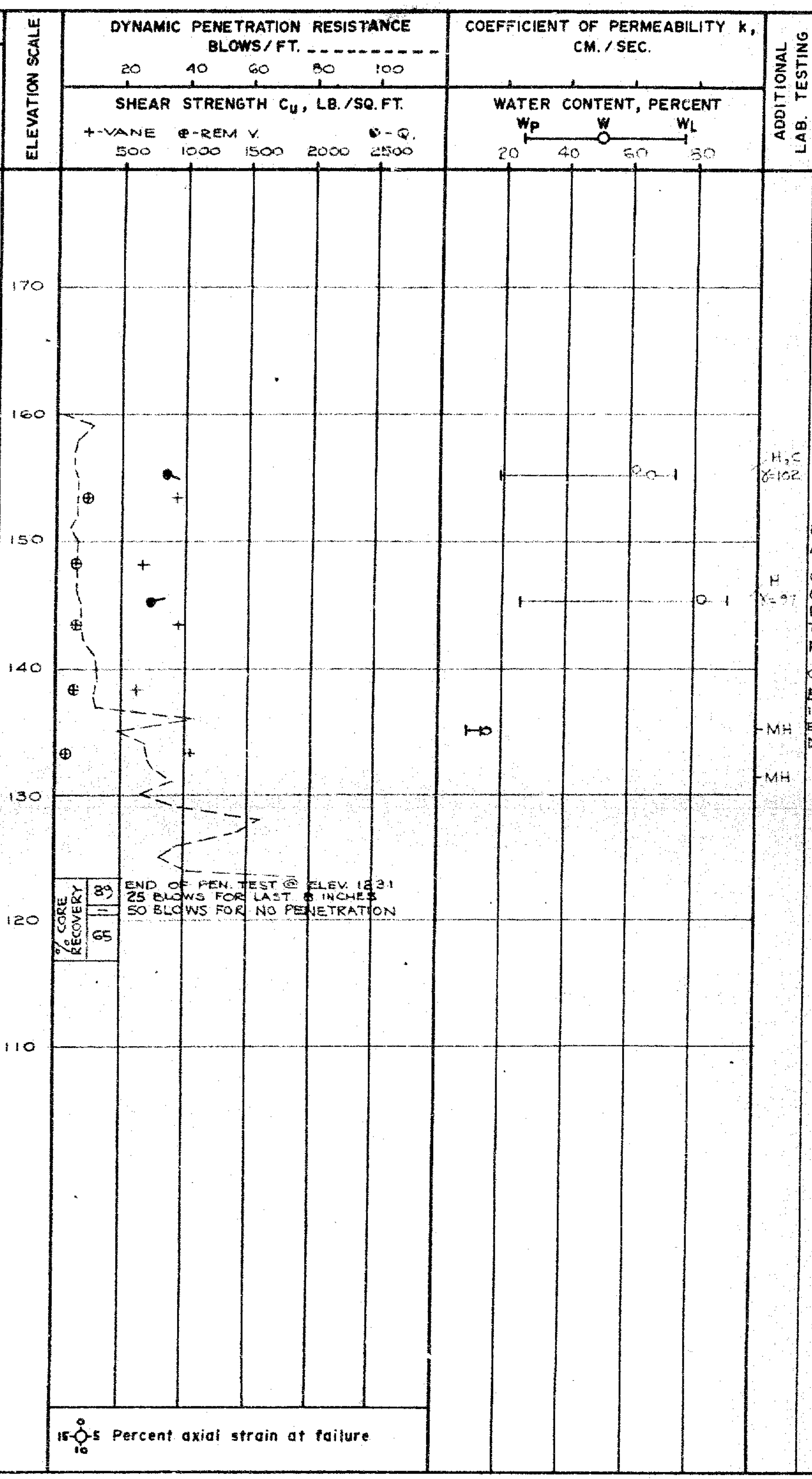
RECORD OF BOREHOLE 1

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

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	BLOWS / FT.
160.8	GROUND LEVEL (SOFT TO FIRM BLACK SILTY **TOPSOIL**)			
0.0		1	2" DO	8
1.0	COMPACT MOTTLED GREY AND BROWN **SANDY SILT**.			
2.5	FIRM MOTTLED GREY & BROWN (BECOMING GREY BELOW EL. 156) **SENSITIVE CLAY**, TRACE TO SOME SILT.	2	"	6
		3	2" TO	PM
		4	2" DO	12
		5	2" TO	PM
		6	2" DO	2
136.4 / 24.4	FIRM GREY **CLAYEY SILT** WITH SAND AND SOME GRAVEL.	7	2" TO	PM
132.2 / 28.6	COMPACT TO VERY DENSE GREY **SILTY SAND AND GRAVEL** WITH A TRACE OF CLAY **(SANDY TILL)**	8	2" DO	>100
		9	2" DO	12
123.5 / 37.3	FAIRLY SOUND GREY **LIMESTONE BEDROCK** WITH INTERBEDDED SHALE LAYERS.	10	NX RC	-
		11	"	-
		12	"	-
116.8 / 44.0	END OF HOLE			



PIEZOMETER OR STANDPIPE INSTALLATION

W.L. IN OPEN CASING AT ELEV. 162.8 DEC. 3, 1965.

DIAMOND CORING BIT BURNED AND CORE BARREL BROKE AT ELEV. 116.8.

CASING LEFT IN GROUND TO ALLOW DRILLING CONTRACTOR TO ATTEMPT TO RECOVER BIT.

CASING PULLED AND BENTONITE SEAL INSTALLED BETWEEN ELEV. 135 AND 141 DEC. 6, 1965.

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN RH

CHECKED JED

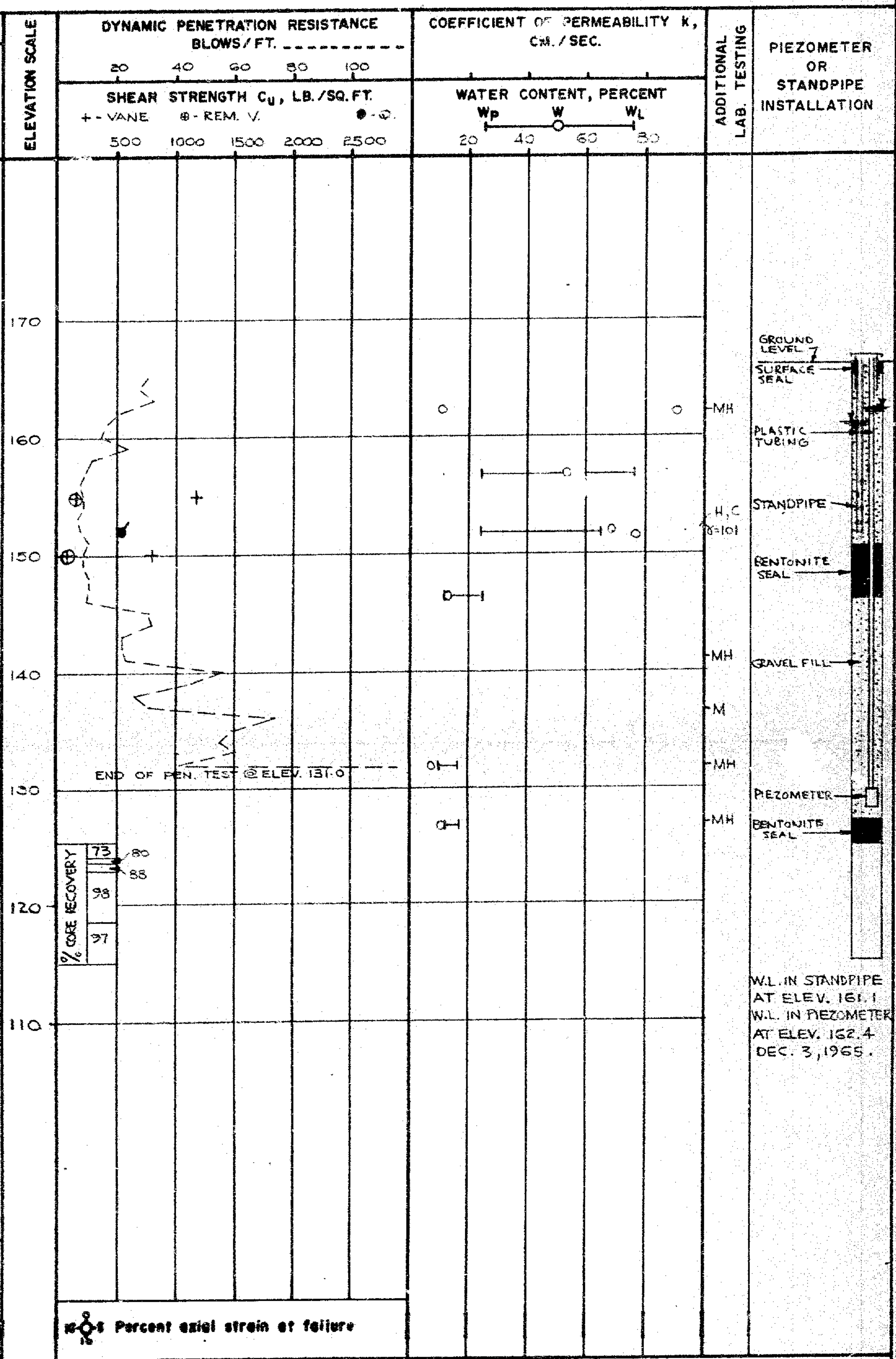
RECORD OF BOREHOLE 2

LOCATION See Figure 1 BORING DATE NOV. 13 - 19, 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

ELEVN. / DEPTH	SOIL PROFILE — DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.
166.0	GROUND LEVEL			
0.0 / 163.5	COMPACT TO DENSE CRUSHED STONE (ROADWAY FILL)	1	2" D.O.	14
2.5 / 161.5	LOOSE BROWN SILTY SAND & GRAVEL (ROADWAY FILL)	2	"	4
160.5	STIFF BLACK SILTY TOPSOIL			
5.5 / 158.5	COMPACT BROWN SANDY SILT TO SILTY SAND, TRACE GRAVEL			
7.5	STIFF TO FIRM BROWN (GREY BELOW ELEV. 154) SENSITIVE CLAY, TRACE TO SOME SILT.	3	"	6
		4	T.O.	Pm
147.2 / 18.8	FIRM TO STIFF GREY CLAYEY SILT WITH SAND, SOME GRAVEL.	5	D.O.	11
142.0 / 24.0	COMPACT TO VERY DENSE GREY SILTY SAND AND GRAVEL, TRACE TO SOME CLAY (SANDY TILL).	6	"	28
		7	"	85
		8	"	84
		9	"	44
125.4 / 40.6	FAIRLY SOUND GREY LIMESTONE BEDROCK WITH INTERBEDDED SHALE LAYERS	10	AX R.C.	—
		11	"	—
		12	"	—
		13	"	—
		14	"	—
115.0 / 51.0	END OF HOLE			



W.L. IN STANDPIPE AT ELEV. 161.1
W.L. IN PIEZOMETER AT ELEV. 162.4
DEC. 3, 1965.

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN RH

CHECKED TJD

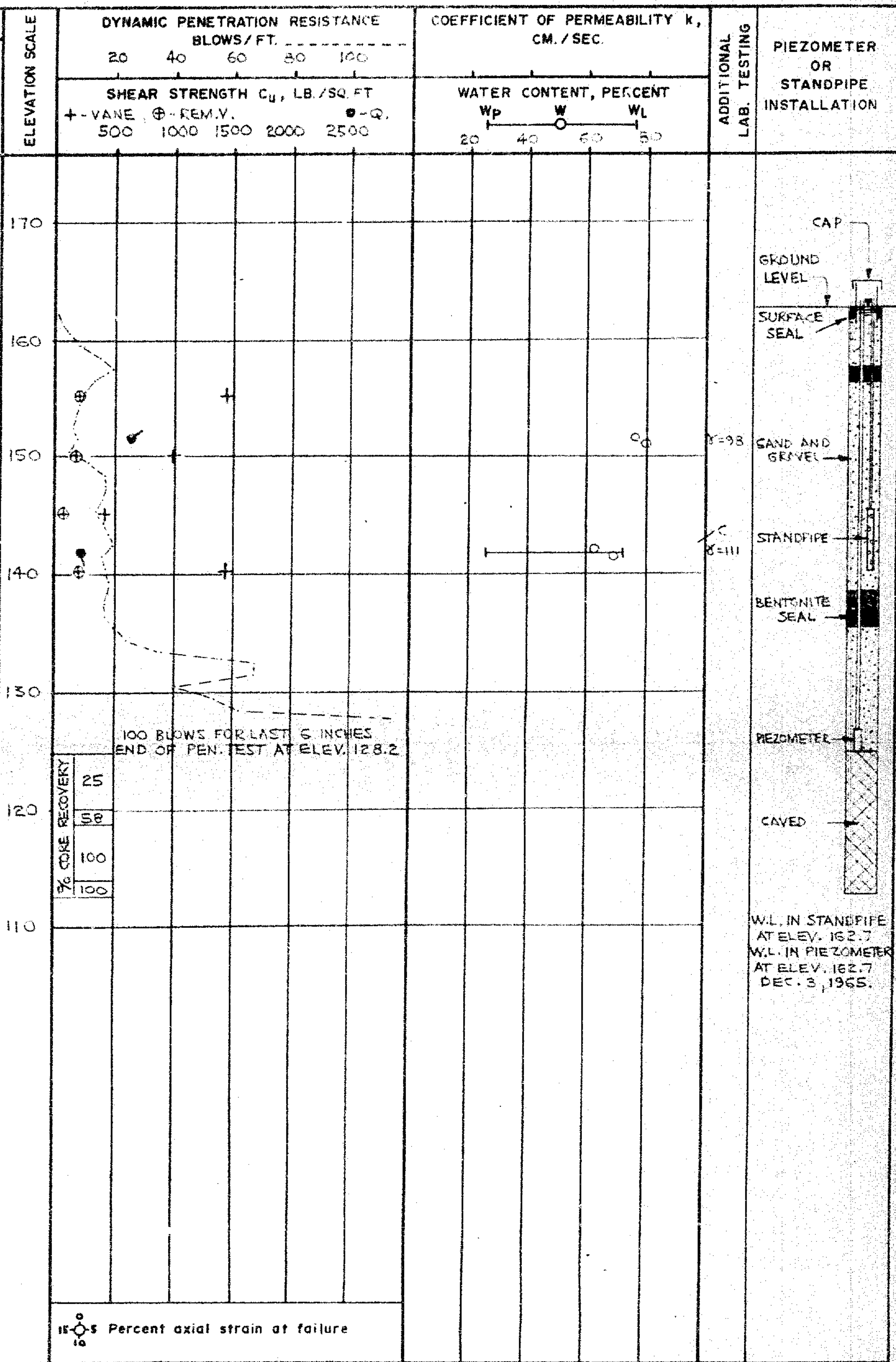
RECORD OF BOREHOLE 3

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

SOIL PROFILE			SAMPLES		
ELEVN. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.
162.7	GROUND LEVEL				
0.0	SOFT BROWN CLAYEY SILT **(FILL)**		1	2" D.O.	2
2.5	COMPACT GREY **SANDY SILT**		2	"	10
4.0	STIFF TO SOFT (GENERALLY FIRM) MOTTLED GREY AND BROWN (BECOMING GREY BELOW EL. 153) **SENSITIVE CLAY**, TRACE TO SOME SILT.		3	"	5
			4	T.O.	PM
			5	D.O.	1
			6	T.O.	PM
135.7			7	D.O.	4
27.0	DENSE TO VERY DENSE GREY **SILTY SAND** AND **GRAVEL**, WITH A TRACE OF CLAY. SOME COBBLES AND BOULDERS BELOW EL. 125 **(SANDY TILL)**		8	"	55
			9	"	100
119.7			10	AXT RC	–
43.0	FAIRLY SOUND GREY **LIMESTONE BEDROCK** WITH INTERBEDDED SHALE LAYERS.		11	"	–
			12	"	–
112.8			13	"	–
49.9	END OF HOLE				



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN MW

CHECKED [illegible]

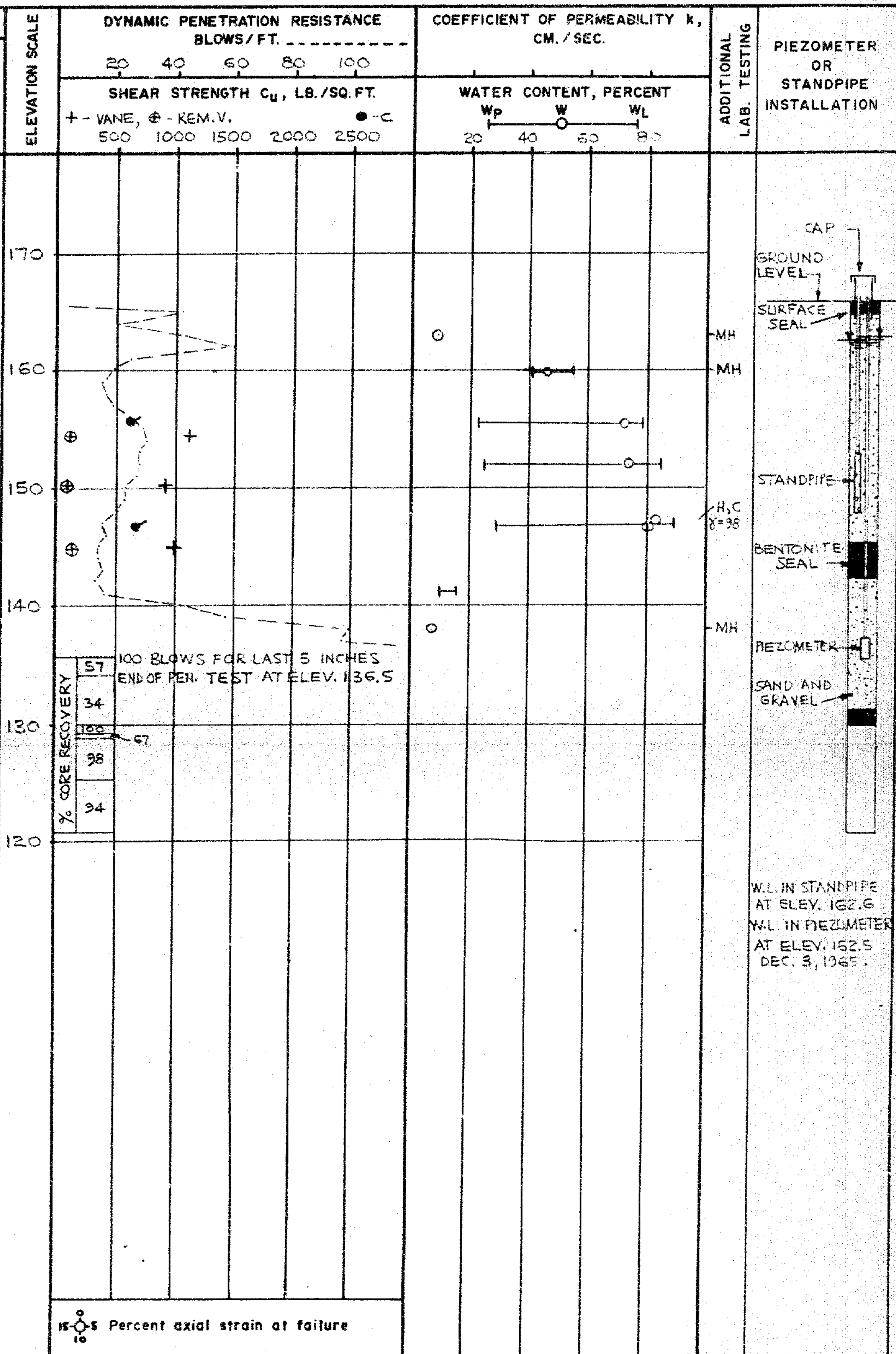
RECORD OF BOREHOLE 4

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

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FT.
165.9	GROUND LEVEL			
0.0	DENSE TO VERY DENSE BROWN SILTY SAND TO SAND AND GRAVEL (ROADWAY FILL)	1	2" D.O.	46
		2	"	39
160.4 / 5.5	VERY STIFF BLACK SILTY TOPSOIL	3	"	21
157.9 / 8.0	STIFF TO FIRM MOTTLED GREY AND BROWN (GREY BELOW EL. 156) SENSITIVE CLAY, TRACE TO SOME SILT	4	"	12
		5	T.O.	3
		6	D.O.	1
		7	T.O.	PM
142.9 / 23.0	VERY STIFF GREY CLAYEY SILT WITH SAND, SOME GRAVEL	8	D.O.	100
140.4 / 25.5	VERY DENSE GREY SILTY SAND AND GRAVEL WITH A TRACE OF CLAY, SOME COBBLES AND BOULDERS BELOW ABOUT EL. 136 (SANDY TILL)	9	"	26
		10	"	>100
		11	AXT RC	—
		12	"	—
130.6 / 35.3	FAIRLY SOUND GREY LIMESTONE BEDROCK WITH INTERBEDDED SHALE LAYERS		"	—
		15	"	—
		16	"	—
120.6 / 45.3	END OF HOLE			



VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN MW
CHECKED [illegible]

PROJECT No. [illegible]

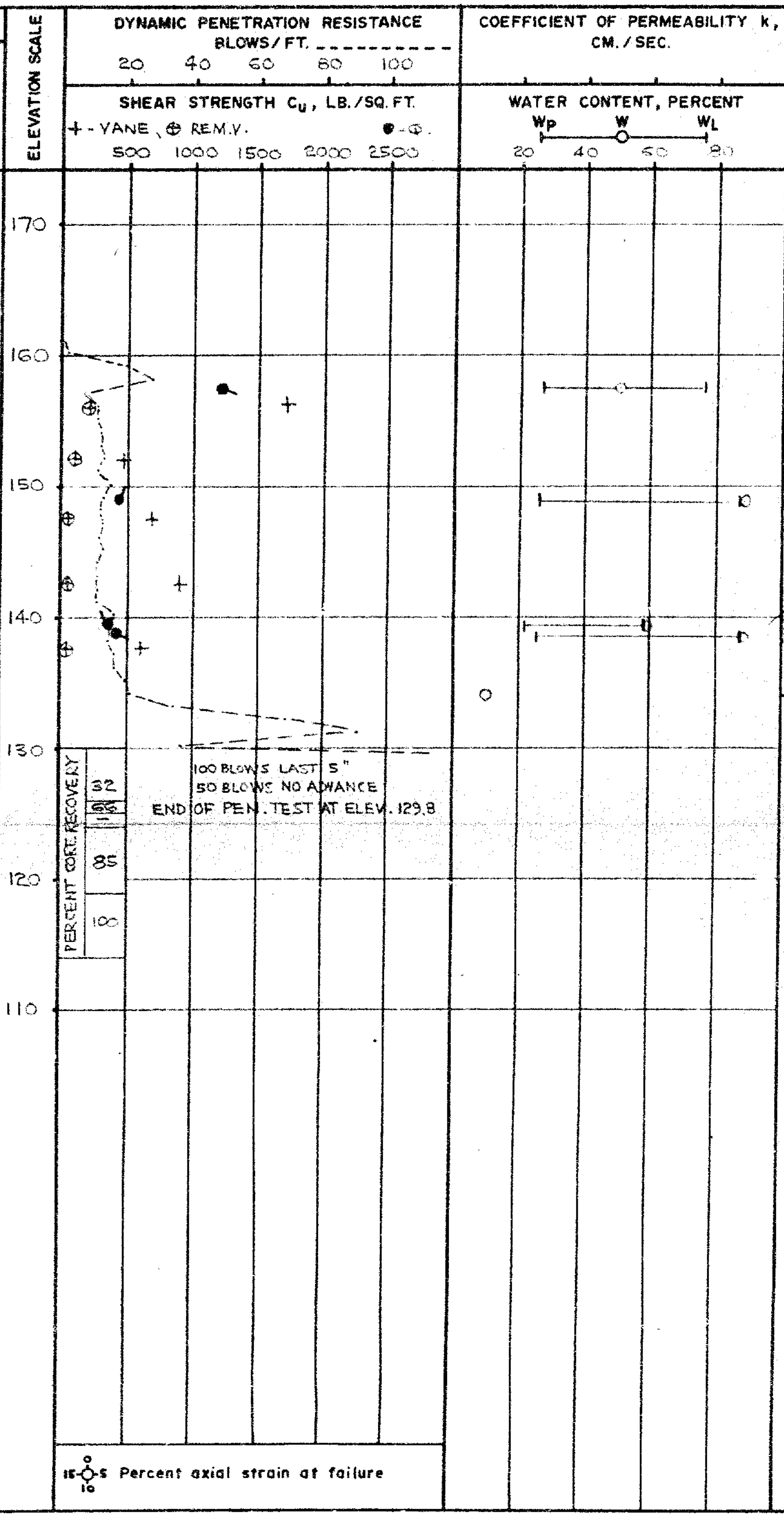
RECORD OF BOREHOLE 5

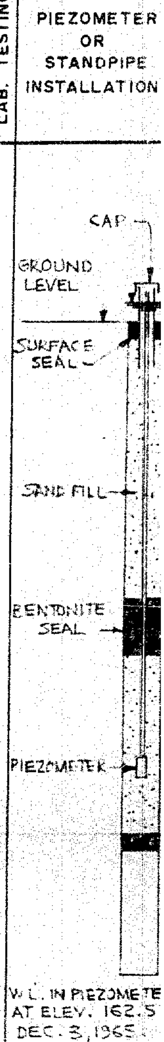
LOCATION See Figure 1 BORING DATE NOV. 25 - 29, 1965 DATUM GEODETIC

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

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.	ADDITIONAL LAB. TESTING
161.2	GROUND LEVEL				
0.0	SOFT TO FIRM BROWN SILTY TOPSOIL				
1.0	COMPACT TO DENSE GREY SANDY SILT WITH A TRACE OF CLAY	1	D.O.	20	
3.0	STIFF TO SOFT (GENERALLY FIRM) BROWN (BECOMING GREY BELOW EL. 155) SENSITIVE CLAY, TRACE TO SOME SILT	2	T.O.	PM	γ=107
		3	D.O.	WH	
		4	T.O.	PM	γ=96
		5	D.O.	WH	
		6	T.O.	PM	C γ=104 γ=98
136.5 / 24.7	COMPACT TO VERY DENSE GREY SILTY SAND AND GRAVEL WITH SOME COBBLES AND BOULDERS BELOW EL. 129 (SANDY TILL)	7	D.O.	21	MH
		8	"	56	
		9	AXT RC	–	
		10	"	–	
124.2 / 37.0		11	D.O.	64	
	FAIRLY SOUND GREY LIMESTONE BEDROCK WITH INTERBEDDED SHALE LAYERS.	12	AXT RC	–	
113.8 / 47.4	END OF HOLE	13	"	–	





VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN [illegible]
CHECKED [illegible]

PROJECT No. [illegible]

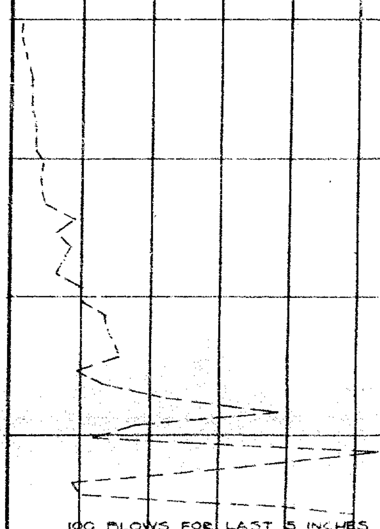
RECORD OF ~~BOREHOLE~~ PEN TEST 6

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

SOIL PROFILE: ELEVN. / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS / FT.	ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FT. (20, 40, 60, 80, 100) / SHEAR STRENGTH C_u, LB. / SQ. FT.	COEFFICIENT OF PERMEABILITY k, CM. / SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
						170				
160.7	GROUND LEVEL									
0.0 / 1.5	PROBABLY TOPSOIL					160				WATER FLOWED FREELY FROM OPEN CONE HOLE. HOLE PLUGGED BETWEEN ELEV. 138 AND 140.
	PROBABLY FIRM SENSITIVE CLAY					150				
140.0 / 20.7	PROBABLY FIRM CLAYEY SILT.					140				
134.7 / 26.0	PROBABLY COMPACT TO VERY DENSE SANDY TILL.					130				
124.3 / 36.4	END OF PEN TEST					120				



15-○-5 Percent axial strain at failure
10

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN RH

CHECKED [illegible]

PROJECT No. 65135

RECORD OF ~~BOREHOLE~~ PEN TEST 7

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

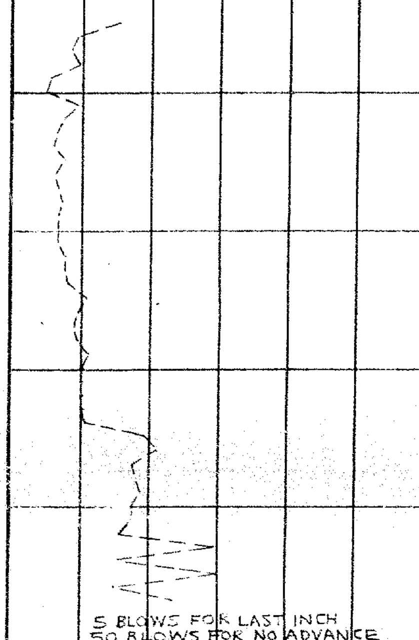
BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —

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

ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE
						170
166.0	GROUND LEVEL					
0.0	PROBABLY COMPACT ROADWAY FILL					
161.0 / 5.0	PROBABLY COMPACT SANDY SILT					160
157.9 / 8.1	PROBABLY STIFF TO FIRM SENSITIVE CLAY					150
146.0 / 20.0	PROBABLY FIRM TO STIFF CLAYEY SILT					140
136.0 / 30.0	PROBABLY COMPACT TO VERY DENSE SANDY TILL.					130
122.9 / 43.1	END OF PEN. TEST					120

DYNAMIC PENETRATION RESISTANCE BLOWS / FT. 20 40 60 80 100

SHEAR STRENGTH C_u, LB./SQ.FT.



COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

CONE HOLE PLUGGED BETWEEN ELEV. 141 AND 143.

15 5 10 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN RH

CHECKED [illegible]

PROJECT No. [illegible]

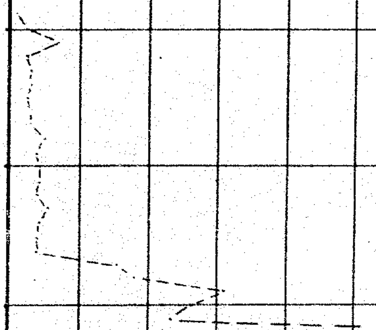
RECORD OF ~~BOREHOLE~~ PEN. TEST 8

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

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. (20 40 60 80 100) / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT (W_P W W_L)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.					
						170				
161.9	GROUND LEVEL									
0.0	PROBABLY SOFT CLAYEY SILT (FILL)									
1.9	PROBABLY COMPACT SANDY SILT.					160				
4.0	PROBABLY STIFF TO SOFT SENSITIVE CLAY									CONE HOLE PLUGGED BETWEEN ELEV. 143.9 AND 142.4.
						150				
143.9 / 18.0	PROBABLY DENSE TO VERY DENSE SANDY TILL.									
138.3 / 23.6	END OF PEN. TEST					140	100 BLOWS FOR LAST 7 INCHES			
						130				
						120				



15 ○ 5 (10) Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN [illegible]

CHECKED [illegible]

PROJECT NO. [illegible]

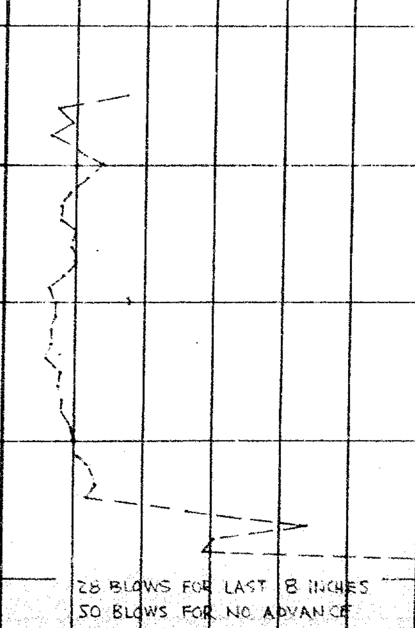
RECORD OF ~~BOREHOLE~~ PEN. TEST 9

LOCATION See Figure 1 BORING DATE NOV. 19, 1965 DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER ——

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

SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT. (20 40 60 80 100) / SHEAR STRENGTH C_u, LB./SQ.FT.	COEFFICIENT OF PERMEABILITY k, CM./SEC. / WATER CONTENT, PERCENT W_P W W_L	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.					
						170				
165.9	GROUND LEVEL									
0.0	PROBABLY DENSE ROADWAY FILL									
157.9 / 8.0	PROBABLY STIFF TO SOFT SENSITIVE CLAY					160				CONE HOLE PLUGGED BETWEEN ELEV. 137.9 AND 136.4.
						150				
139.9 / 26.0	PROBABLY VERY STIFF CLAYEY SILT					140				
135.9 / 30.0	PROBABLY VERY DENSE SANDY TILL									
131.2 / 34.7	END OF PEN. TEST					130	28 BLOWS FOR LAST 8 INCHES 50 BLOWS FOR NO ADVANCE			
						120				



15 10 5 Percent axial strain at failure

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED [illegible]

RECORD OF ~~BOREHOLE~~ PEN. TEST 10

LOCATION See Figure 1 | BORING DATE NOV. 25, 1965 | DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST | BOREHOLE DIAMETER —

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

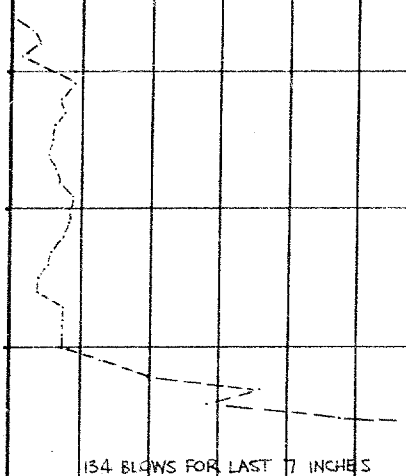
ELEVN. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
						170		
163.7	GROUND LEVEL							
0.0	PROBABLY TOPSOIL							
1.0	PROBABLY COMPACT TO DENSE SANDY SILT.					160		
158.7 / 5.0	PROBABLY STIFF TO SOFT SENSITIVE CLAY					150		CONE HOLE PLUGGED BETWEEN ELEV. 148.7 AND 147.2.
140.0 / 23.7	PROBABLY COMPACT TO VERY DENSE SANDY TILL.					140		
134.1 / 29.6	END OF PEN. TEST					130		
						120		

DYNAMIC PENETRATION RESISTANCE BLOWS/FT. - - - - - - - 20 40 60 80 100

SHEAR STRENGTH C_u, LB./SQ. FT.

COEFFICIENT OF PERMEABILITY k, CM./SEC.

WATER CONTENT, PERCENT W_P W W_L



15-○-5 Percent axial strain at failure (10)

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN MW

CHECKED J.E.D.

PROJECT No. [illegible]

RECORD OF BOREHOLE 11

LOCATION See Figure 1 BORING DATE NOV. 30-DEC. 1, 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

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	BLOWS / FT.
162.1	GROUND LEVEL			
0.0	SOFT TO FIRM BLACK SILTY **TOPSOIL**	1	2" D.O.	8
2.0	VERY STIFF GREY **CLAYEY SILT**	2	"	20
157.7 / 4.4	STIFF TO SOFT (GENERALLY FIRM) MOTTLED GREY AND BROWN (BECOMING GREY BELOW EL. 154.5) **SENSITIVE CLAY**, TRACE TO SOME SILT.	3	T.O.	PM
		4	D.O.	1
		5	T.O.	PM
141.2 / 20.8	SOFT TO FIRM GREY **CLAYEY SILT** WITH SAND, SOME GRAVEL.	6	D.O.	2
		7	"	15
135.1 / 27.0	COMPACT TO DENSE GREY **SILTY SAND & GRAVEL** WITH A TRACE TO SOME CLAY **(SANDY TILL)**	8	"	34
		9	"	>100
127.8 / 34.3	END OF HOLE			

ELEVATION SCALE

DYNAMIC PENETRATION RESISTANCE BLOWS / FT. (20, 40, 60, 80, 100)

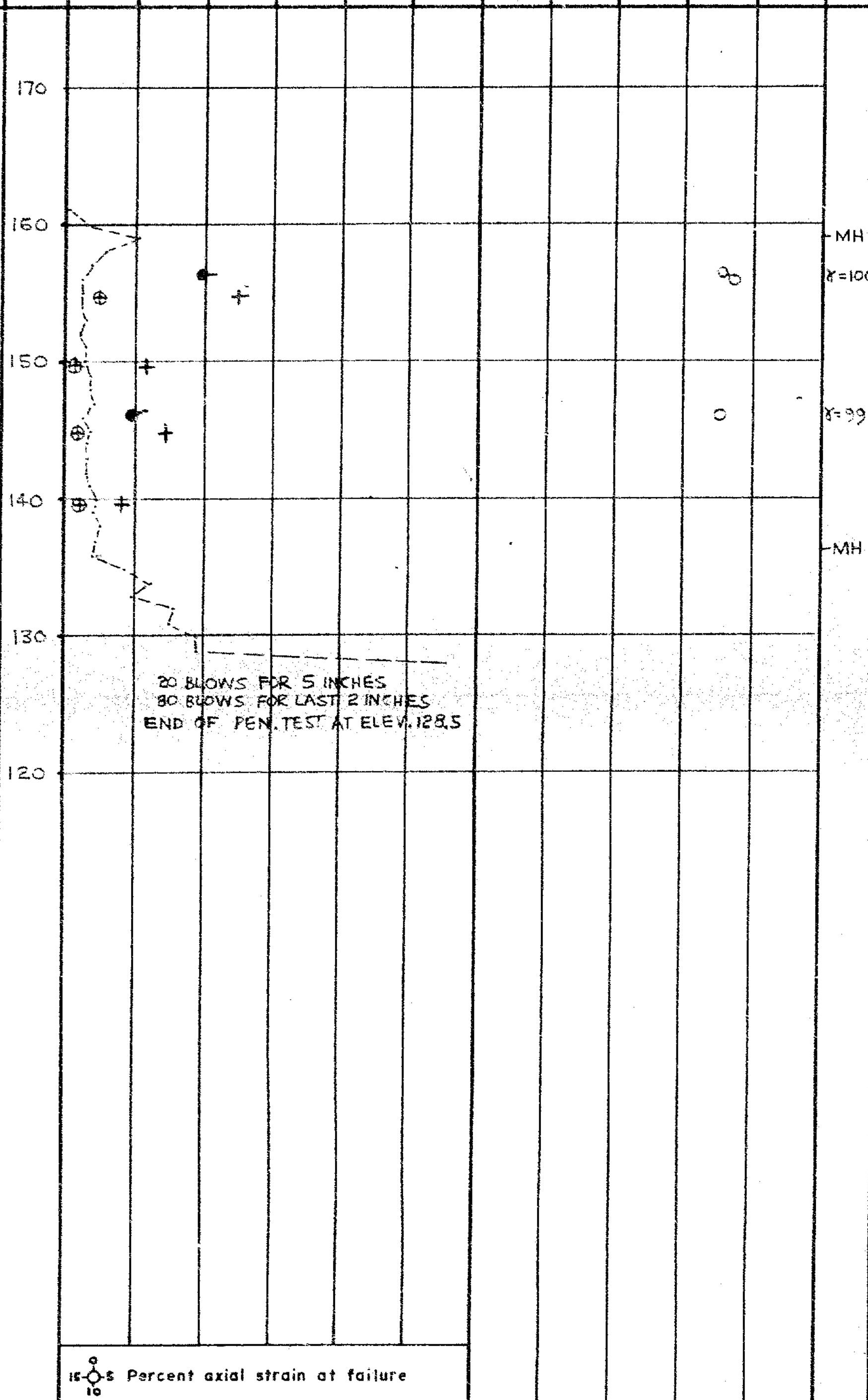
SHEAR STRENGTH c_u, LB. / SQ. FT. + - VANE, ⊕ - REM. V., ● - Q (500, 1000, 1500, 2000, 2500)

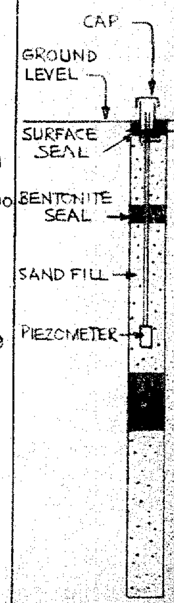
COEFFICIENT OF PERMEABILITY k, CM. / SEC.

WATER CONTENT, PERCENT W_P — W — W_L (20, 40, 60, 80)

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION





W.L. IN PIEZOMETER AT ELEV. 161.4 DEC. 3, 1965.

VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN M.W.

CHECKED [illegible]

PROJECT No. [illegible]

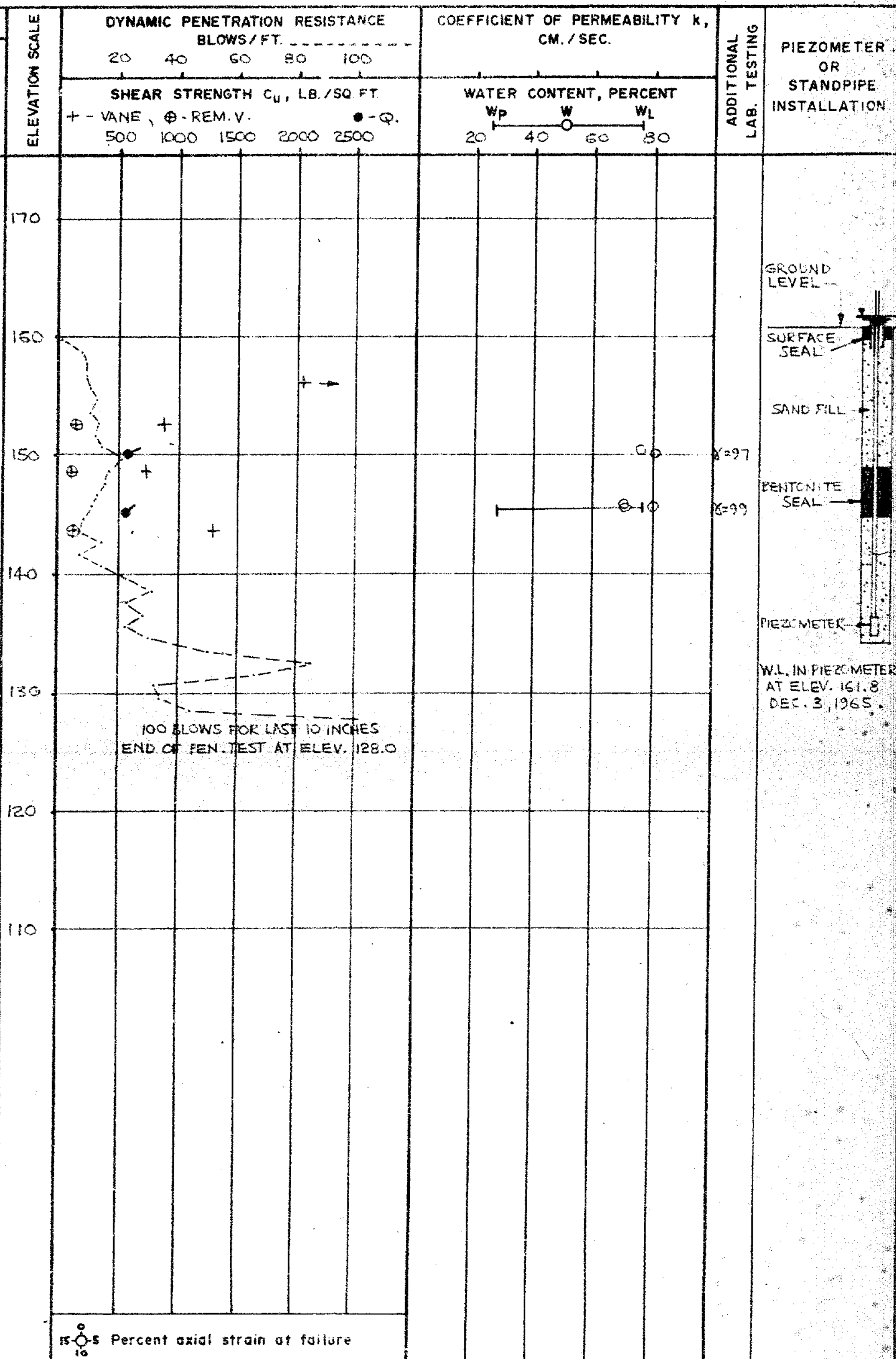
RECORD OF BOREHOLE 12

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

ELEVN. / DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FT.
160.9	GROUND LEVEL			
0.0	SOFT BLACK SILTY **TOPSOIL**	1	2" D.O.	5
1.5	COMPACT GREY **SANDY SILT**	2	"	14
3.0	VERY STIFF TO FIRM MOTTLED GREY AND BROWN (GREY BELOW EL. 155) **SENSITIVE CLAY**, TRACE TO SOME SILT (1 FOOT SAND LAYER AT EL. 155)	3	"	3
		4	T.O.	PM
143.4		5	"	PM
17.5	STIFF TO VERY STIFF GREY **CLAYEY SILT** WITH SAND, GRAVEL.	6	D.O.	11
138.4 / 22.5	DENSE TO VERY DENSE GREY **SILTY SAND & GRAVEL** WITH TRACE CLAY **(SANDY TILL)**	7	"	49
134.4 / 26.5	END OF HOLE PROBABLY DENSE SILTY SAND & GRAVEL (TILL)			
128.0 / 32.9	END OF PEN. TEST.			



VERTICAL SCALE 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

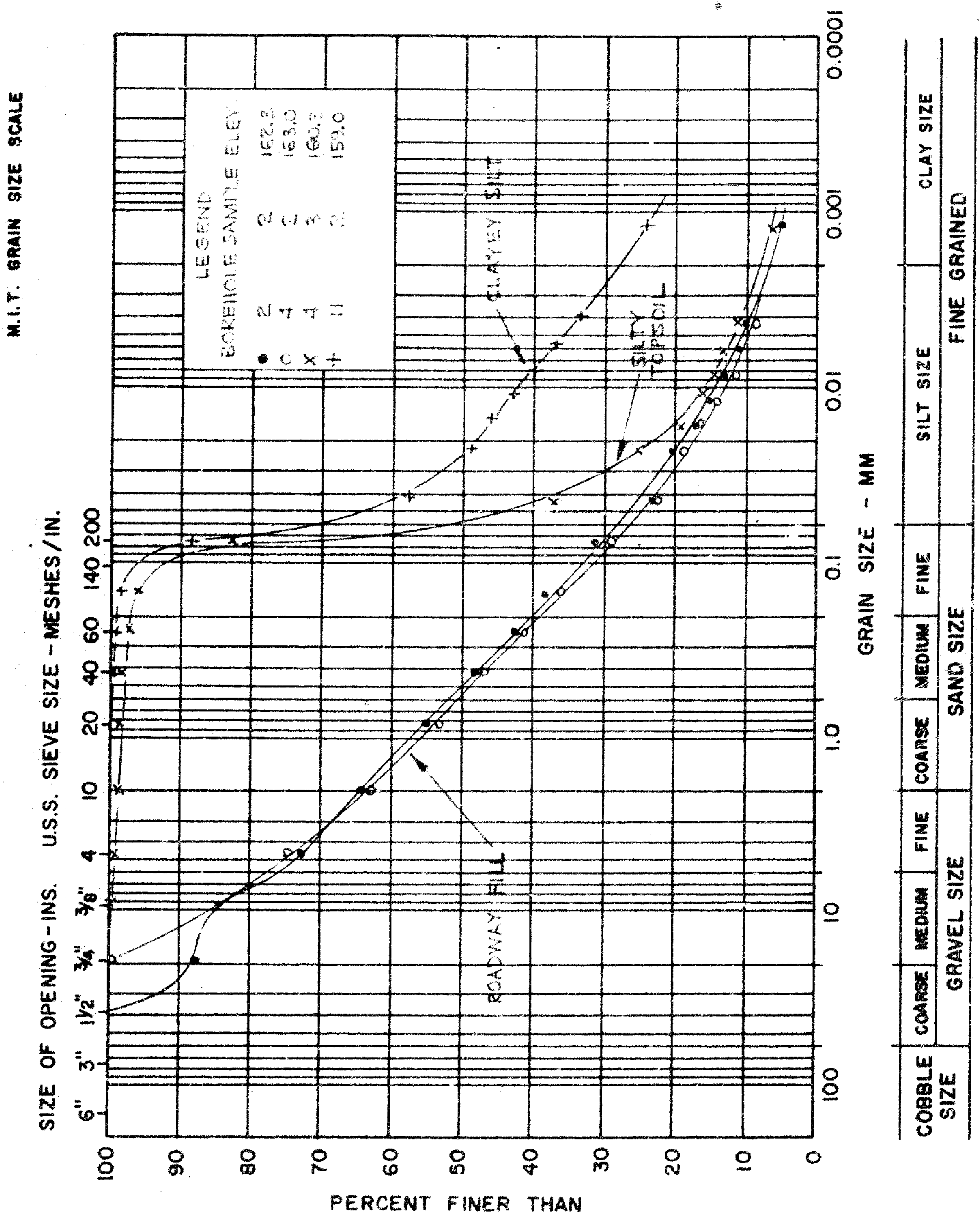
DRAWN M.W. CHECKED J.S.D.

PROJECT No. 65135

GRAIN SIZE DISTRIBUTION

SURFICIAL DEPOSITS

FIGURE 2

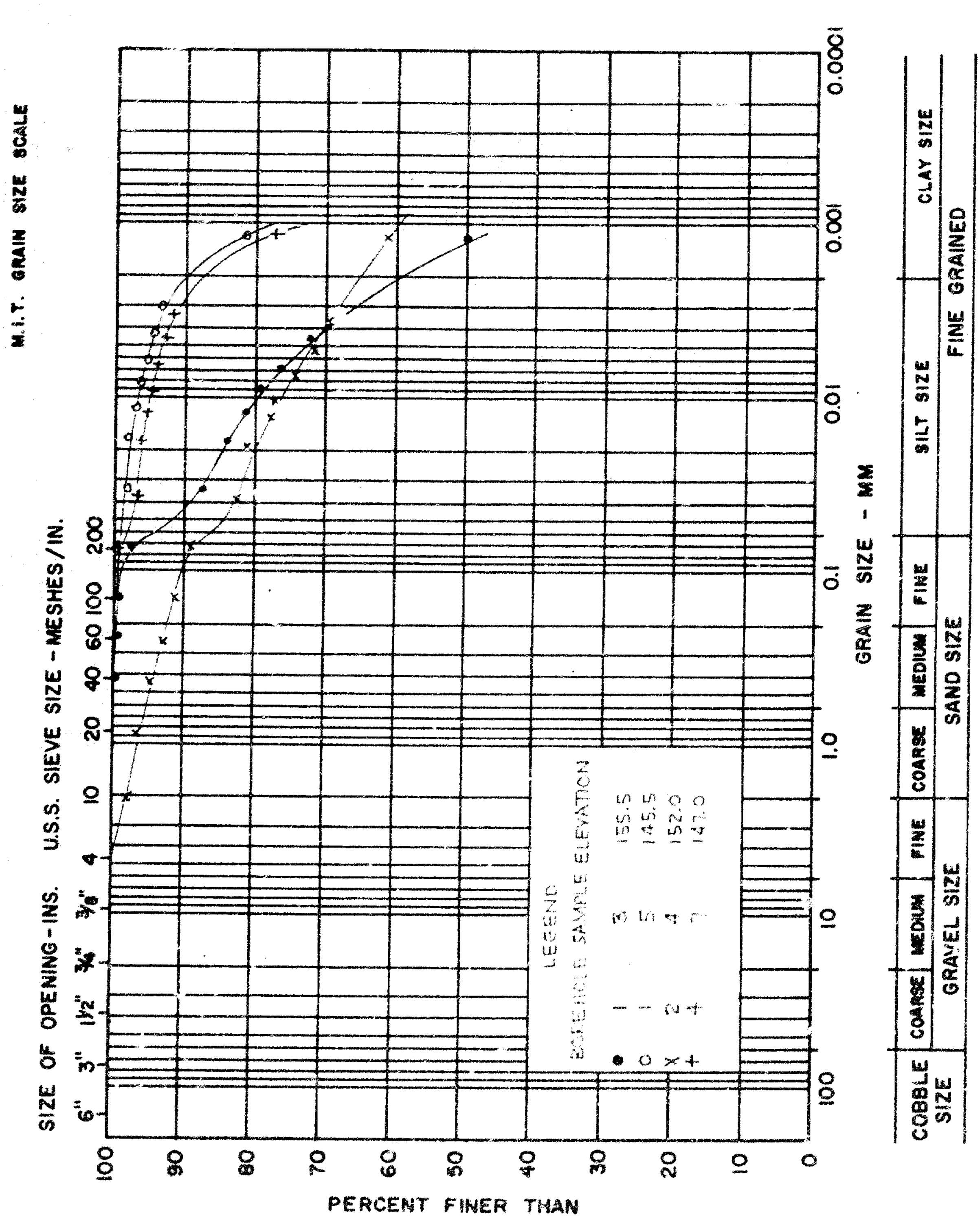


GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

SENSITIVE CLAY STRATUM

FIGURE 3



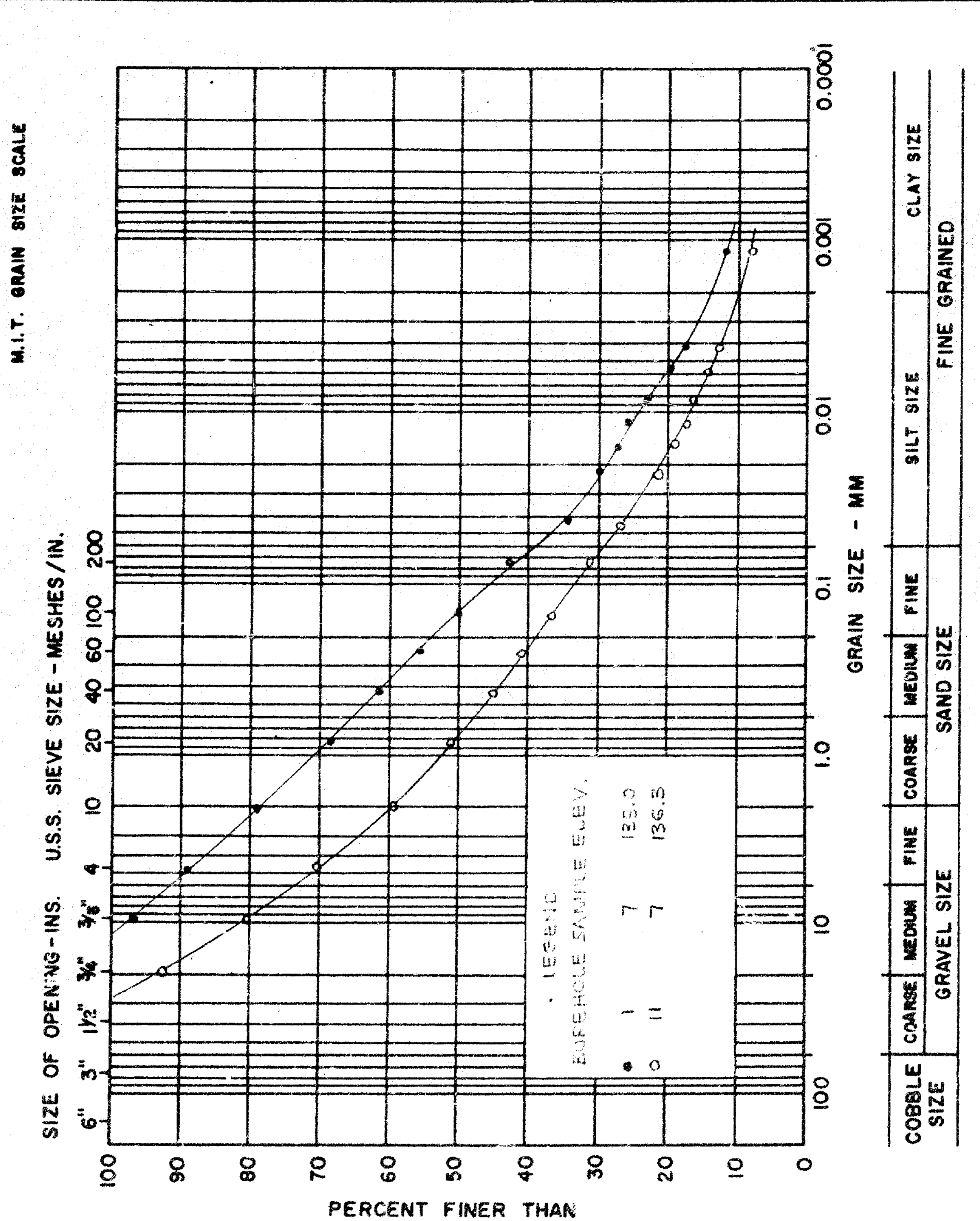
GOLDER & ASSOCIATES

PROJECT No. 35135

GRAIN SIZE DISTRIBUTION

CLAYEY SILT WITH SAND STRATUM

FIGURE 4



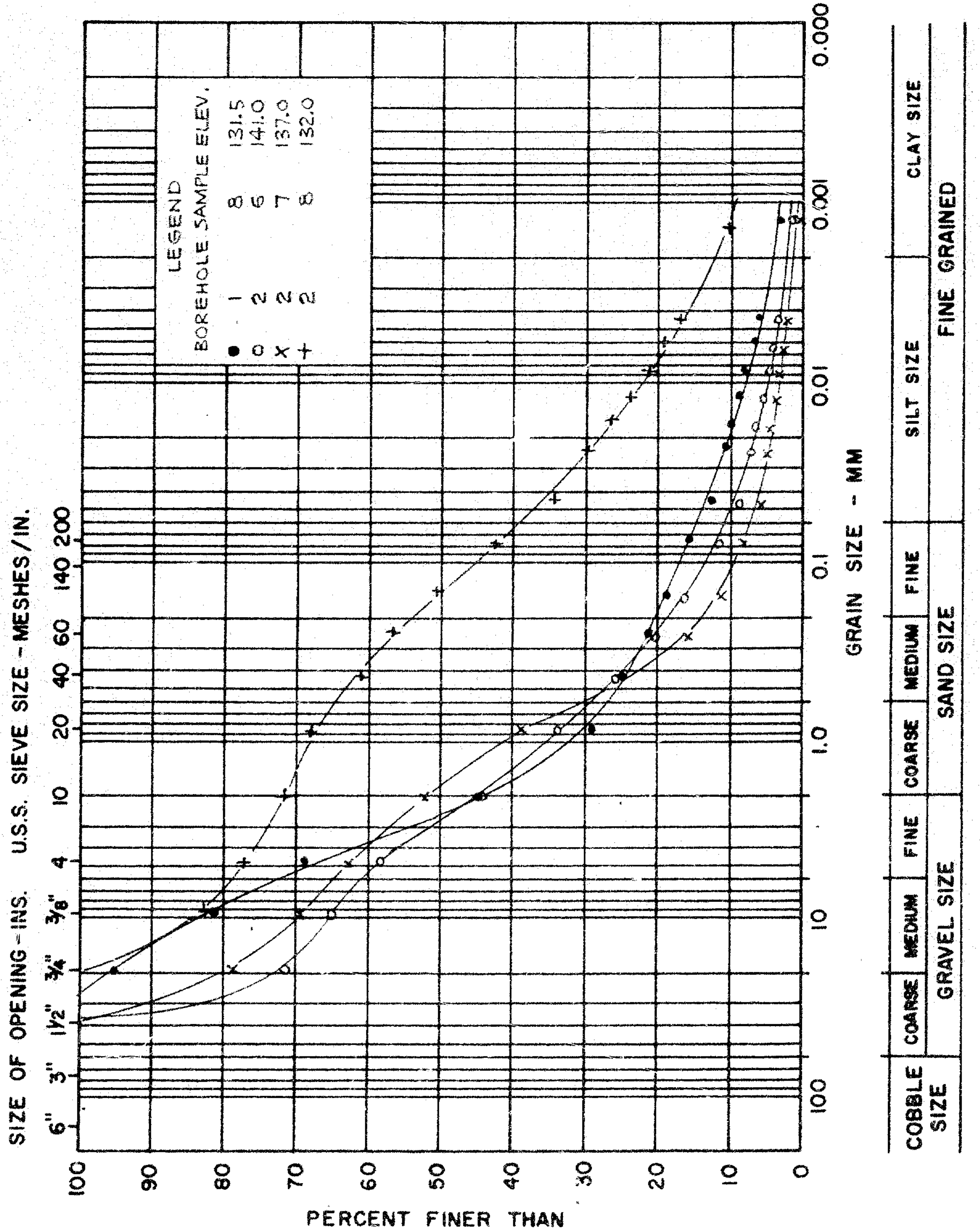
GOLDER & ASSOCIATES

PROJECT No. 65135

GRAIN SIZE DISTRIBUTION

TILL STRATUM

FIGURE 5



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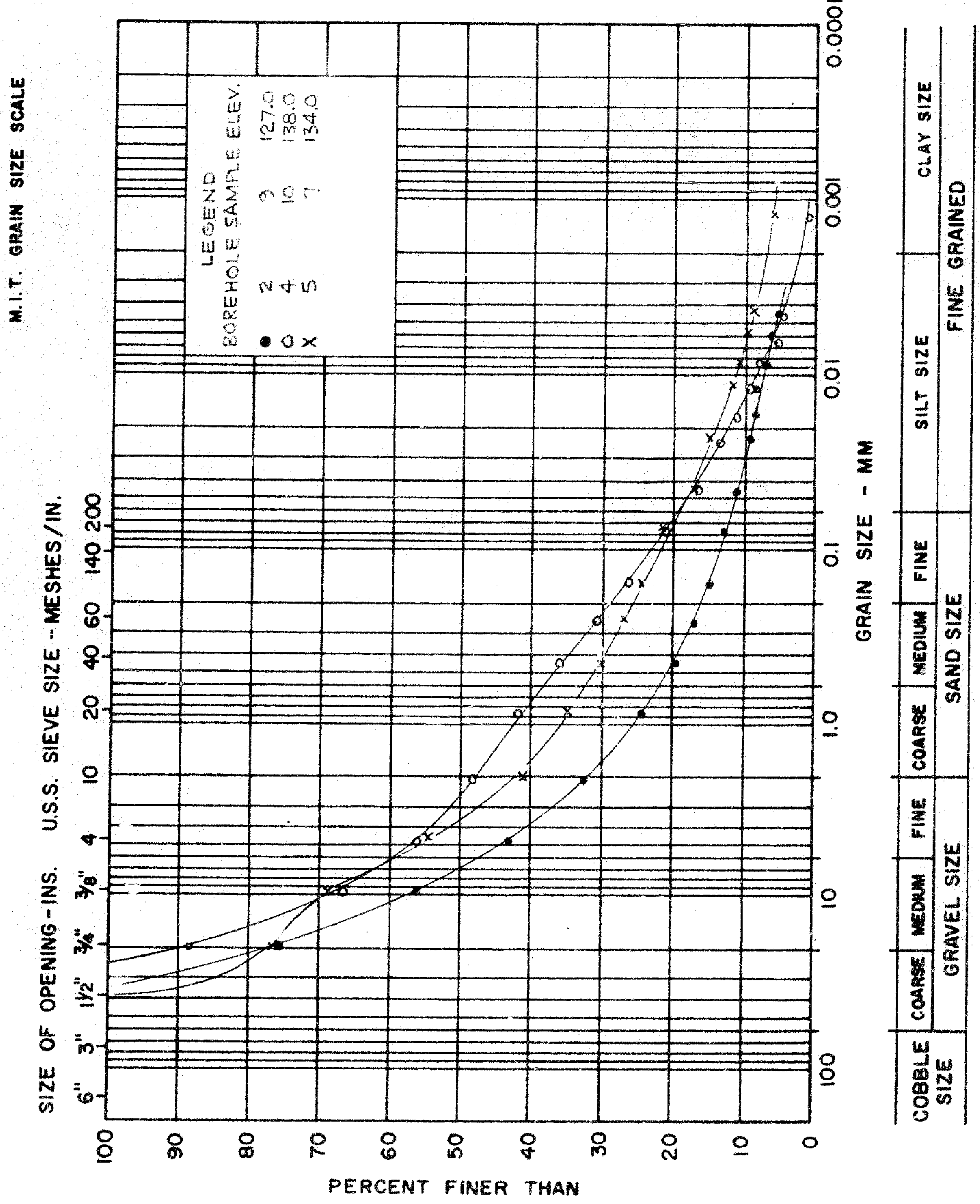
PROJECT No. 65135

GRAIN SIZE DISTRIBUTION

TILL STRATUM

FIGURE 6

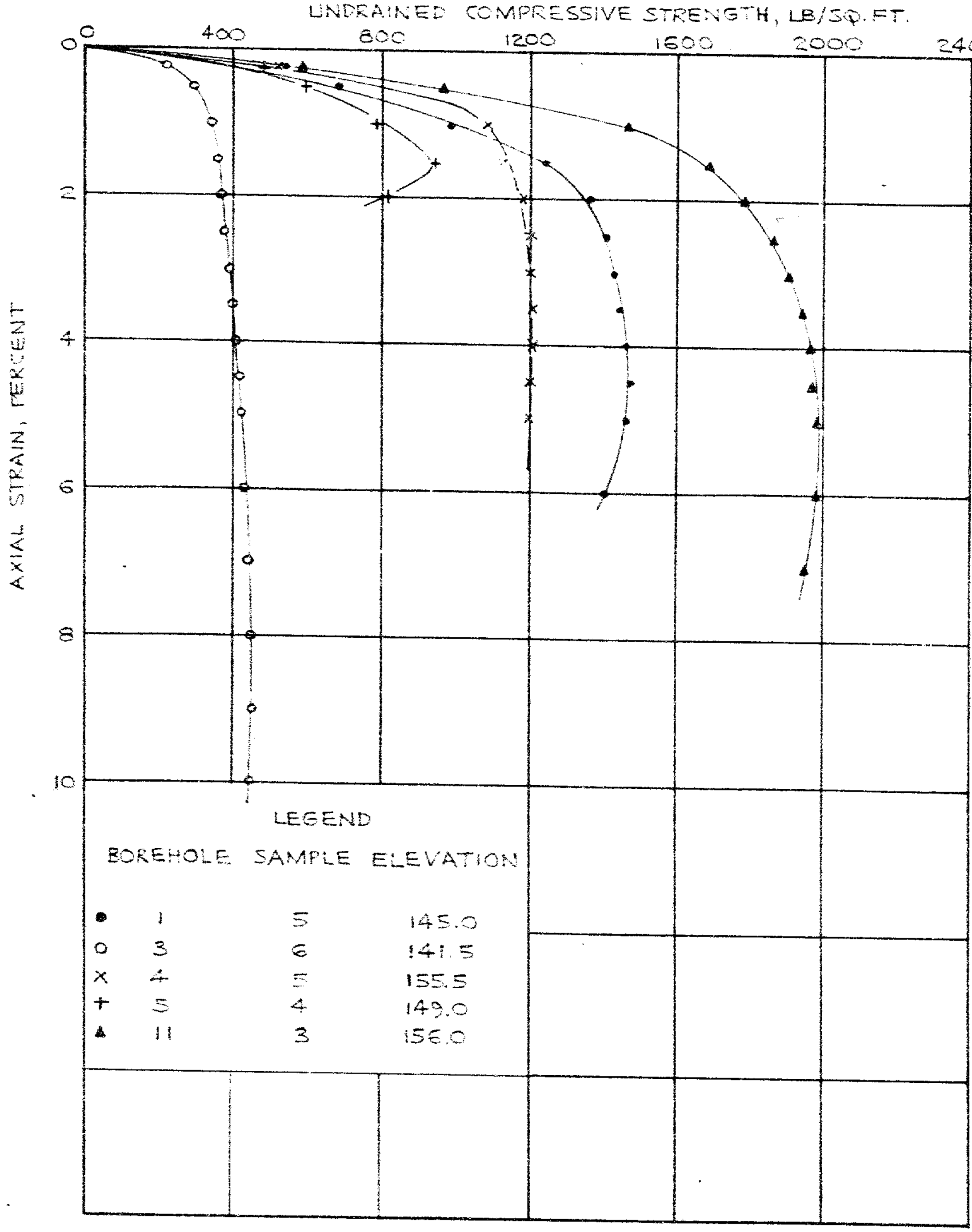


GOLDER & ASSOCIATES

PROJECT No.

UNDRAINED TRIAXIAL COMPRESSION TESTS
TYPICAL STRESS- STRAIN CURVES
SENSITIVE CLAY STRATUM

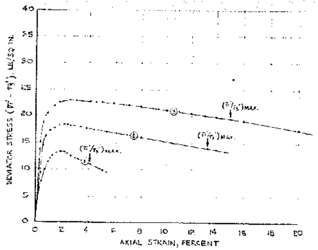
FIGURE 8

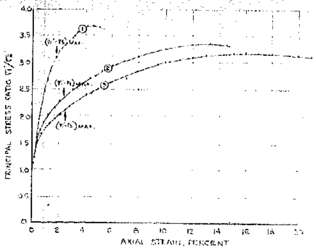


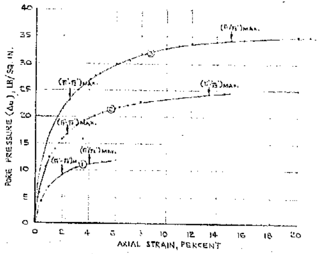
GOLDER & ASSOCIATES

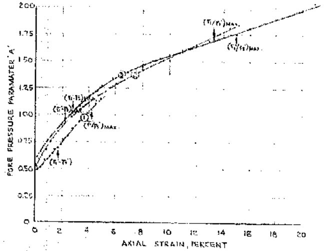
CONSOLIDATED UNDRAINED TRIAXIAL TESTS WITH PORE PRESSURE MEASUREMENTS
SENSITIVE CLAY STRATUM

FIGURE 5





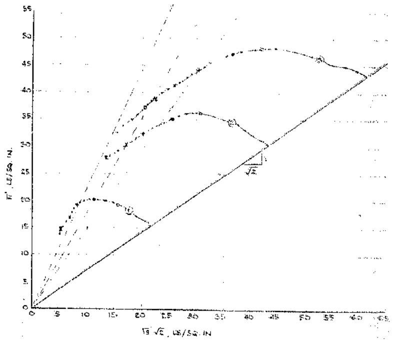




NOTE: PORE PRESSURE PARAMETER 'B' WAS FOUND TO BE BETWEEN ABOUT 0.98 AND 1.0.

LEGEND

TEST	HOLE	SAMPLE	ELEVATION	LIQUID LIMIT	PLASTIC LIMIT
1	2	4	151.6	65.6	24.9
2	1	5	144.7	85.9	25.5
3	4	"	146.4	86.8	28.5



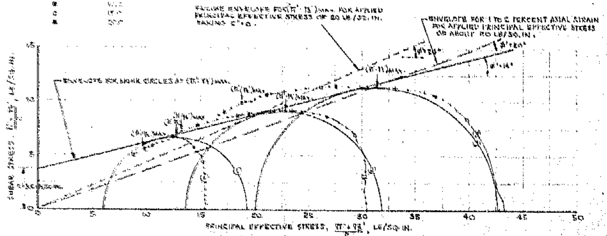
RENDULIC PLOT

TEST	CELL PRESSURE LB/SQ. IN.	AVERAGE RATE OF STRAIN TO ($\bar{\sigma}_1'$ - $\bar{\sigma}_3'$)max. %/HR.	W_i %	W_f %	'B' *
1	10.4	0.6	76	73	1.00
2	30.5	0.5	81	61	0.99
3	45.5	1.0	81	52	0.98

* PORE PRESSURE PARAMETERS, REF. SKEMPTON, A.W. (1954) "THE PORE PRESSURE COEFFICIENTS A AND B" GEOTECHNIQUE VOL. IV, 1954.

LEGEND

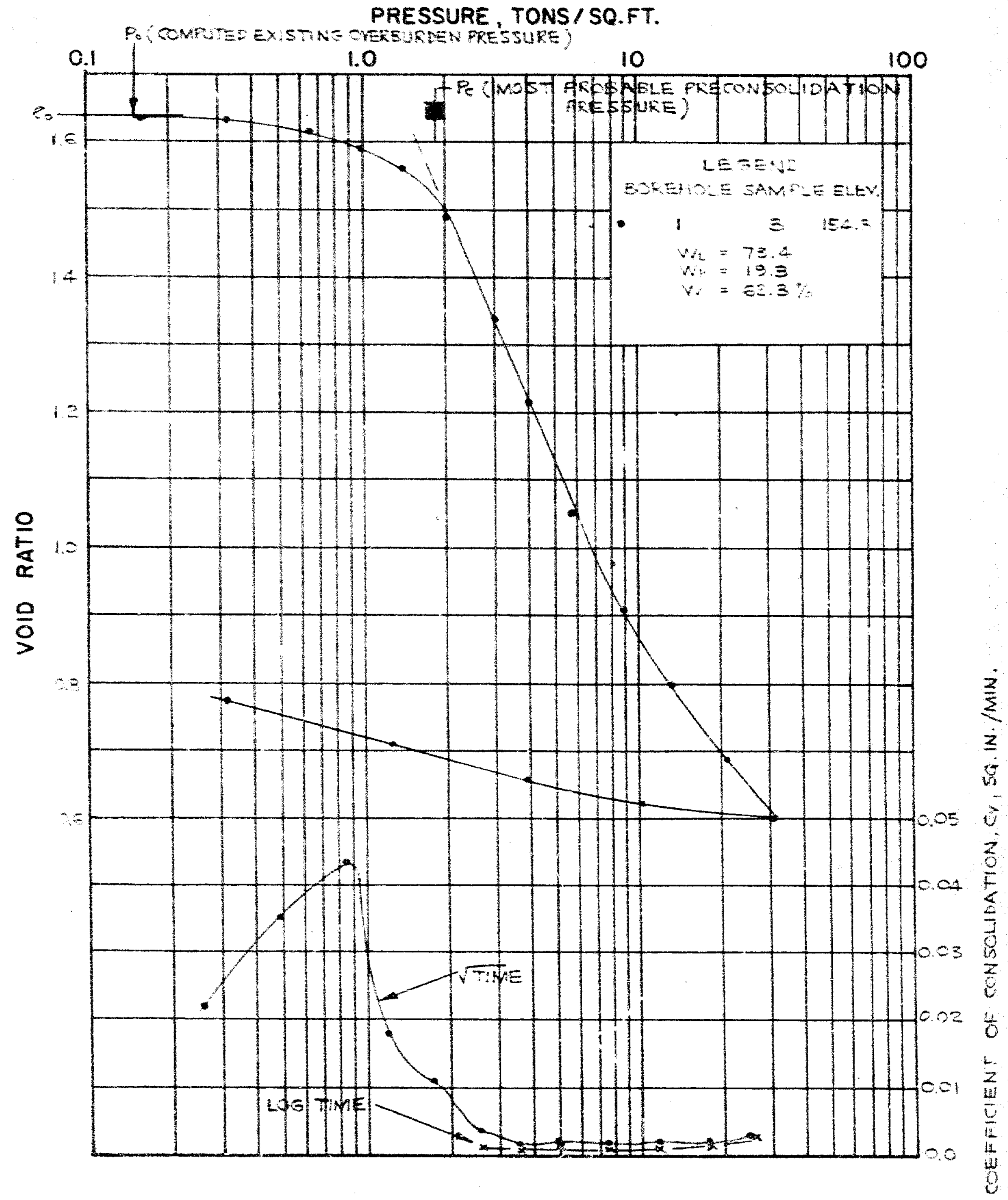
SYMBOL PERCENT AXIAL STRAIN



MOHR CIRCLE PLOT

GOLDER & ASSOCIATES

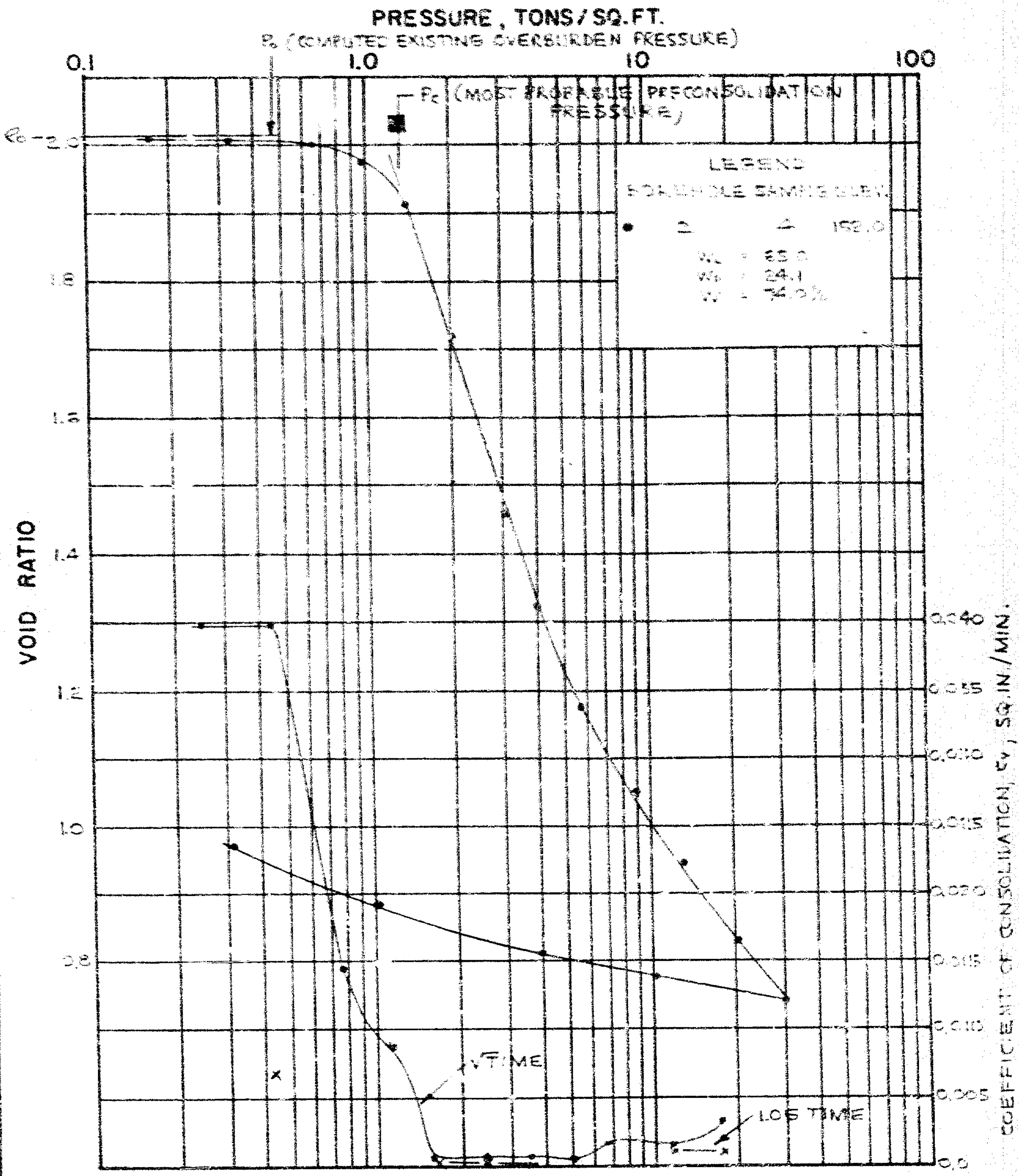
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST	FIGURE 10



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

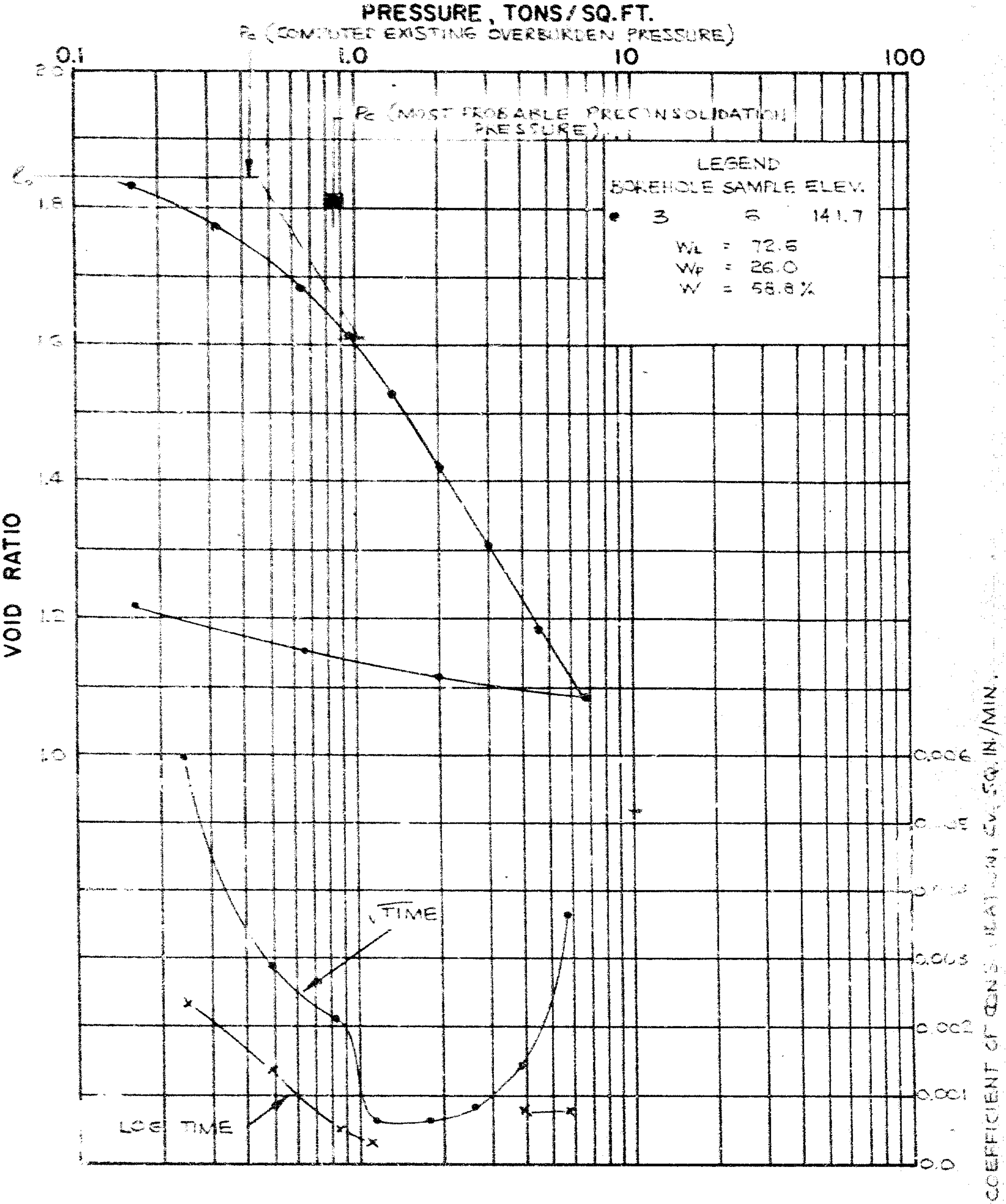
FIGURE 11



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 12

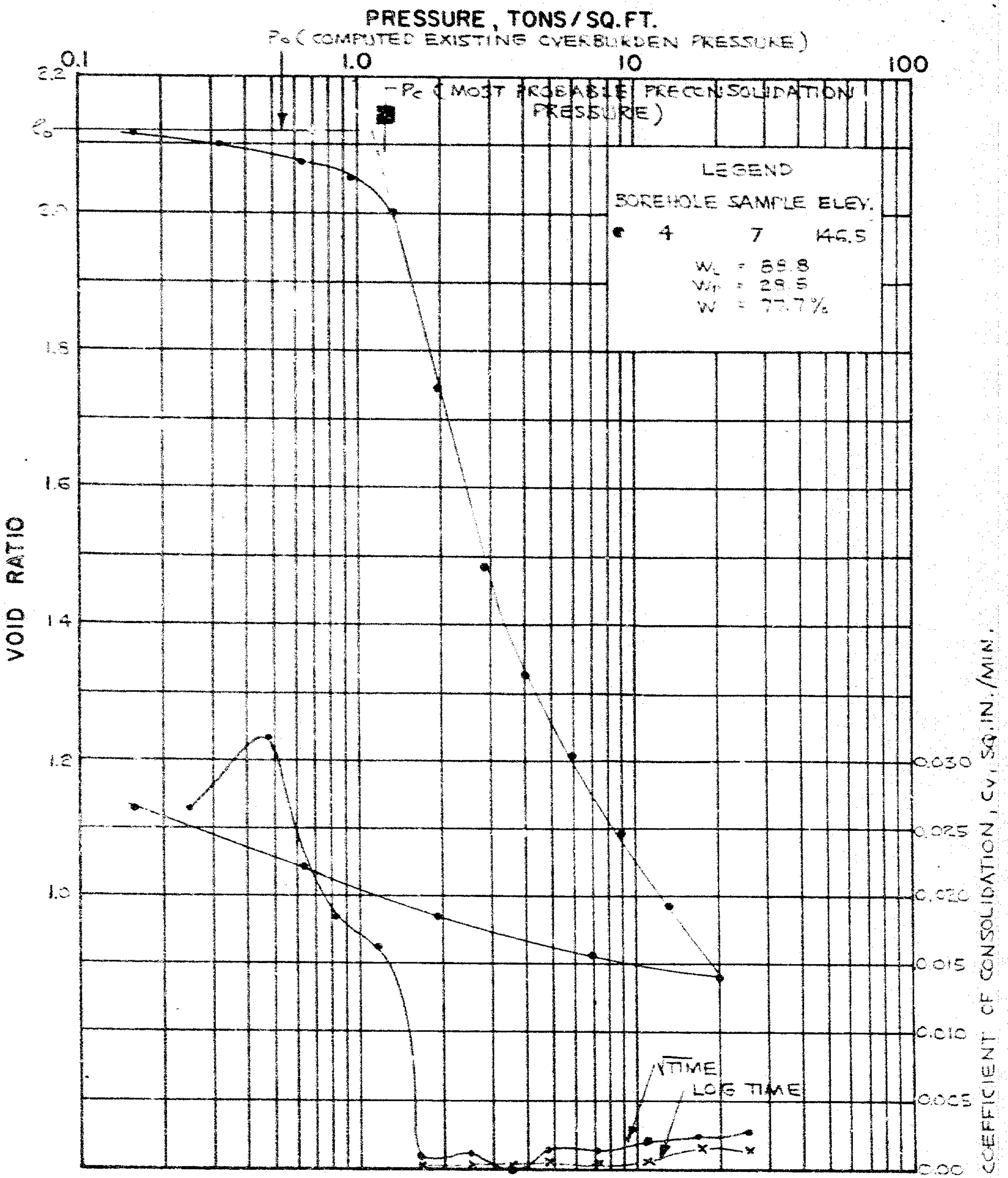


GOLDER & ASSOCIATES

PROJECT NO. ... 65/35

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 13

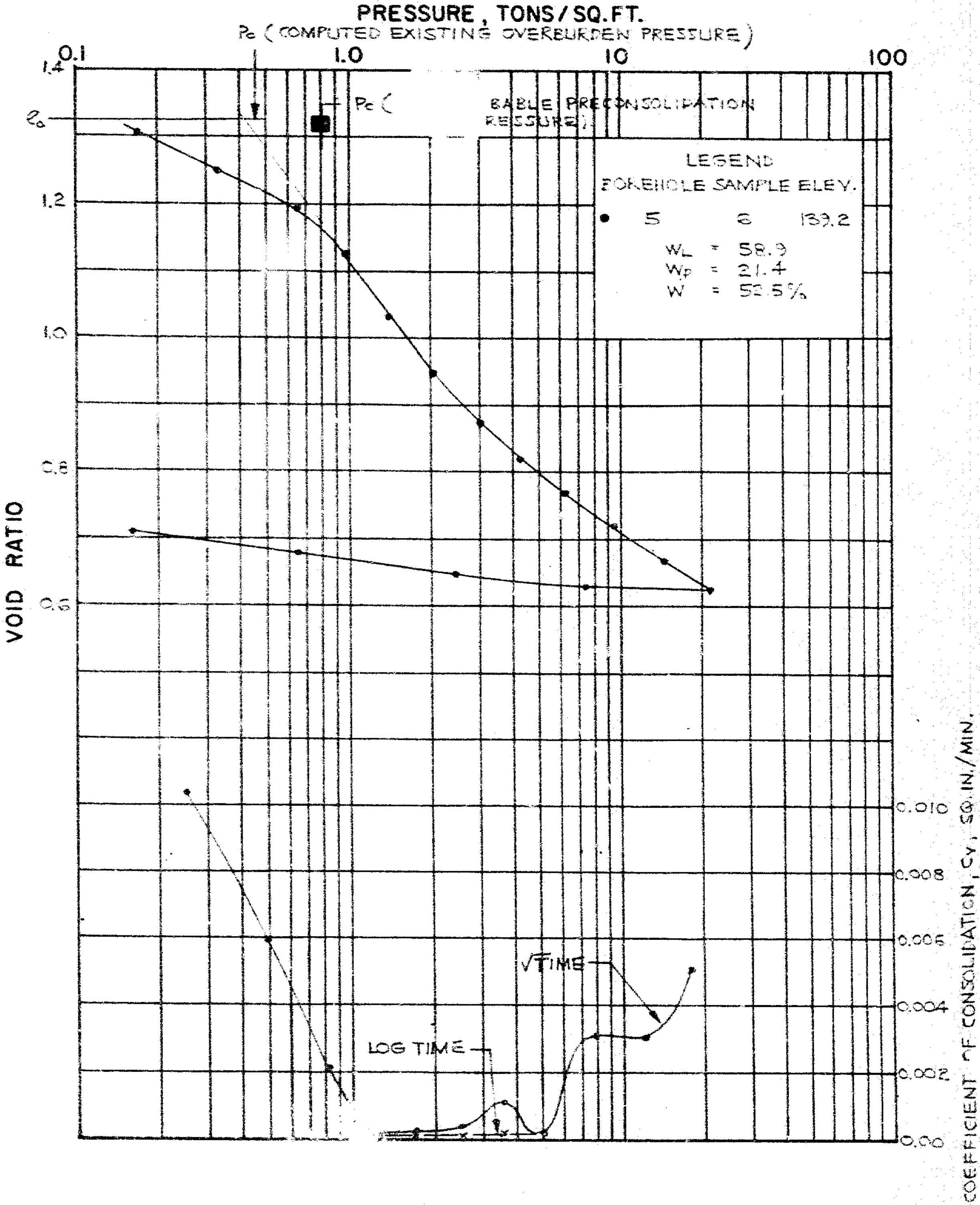


GOLDER & ASSOCIATES

PROJECT No. ...

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 14



GOLDER & ASSOCIATES

APPENDIX D

Selected Site Photographs

Photograph 1: Site 31-230, North approach embankment, looking south (July 17, 2018).

Photograph 2: Site 31-230, Borehole 18-1101, adjacent to existing bridge (July 17, 2018).

CLIENT DILLON CONSULTING LIMITED			PROJECT REPLACEMENT OF FRASER ROAD UNDERPASS AT HIGHWAY 401, UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ONTARIO				
CONSULTANT GOLDER	YYYY-MM-DD	2018/11/28	TITLE **SELECTED SITE PHOTOGRAPHS**				
	PREPARED	CK					
	DESIGN	CK					
	REVIEW	WC	PROJECT No. **1899802**	Phase **1100**	Rev. **0**	Figure **D1**	
	APPROVED	FJH					

Photograph 3: Site 31-230, West side of Fraser Road Bridge, looking south (July 17, 2018).

Photograph 4: Site 31-230, East side of Fraser Road Bridge, looking south (July 17, 2018).

CLIENT
DILLON CONSULTING LIMITED

PROJECT
REPLACEMENT OF FRASER ROAD UNDERPASS AT HIGHWAY 401, UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ONTARIO

CONSULTANT

GOLDER

YYYY-MM-DD	2018/11/28
PREPARED	CK
DESIGN	CK
REVIEW	WC
APPROVED	FJH

TITLE
SELECTED SITE PHOTOGRAPHS

PROJECT No.	Phase	Rev.	Figure
1899802	1100	0	D2

Photograph 5: Site 31-230, South Approach Embankment, looking northeast (July 17, 2018).

Photograph 6: Site 31-230, Borehole 18-1103, adjacent to existing bridge (July 17, 2018).

CLIENT
DILLON CONSULTING LIMITED

PROJECT
REPLACEMENT OF FRASER ROAD UNDERPASS AT HIGHWAY 401, UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ONTARIO

CONSULTANT

GOLDER

YYYY-MM-DD	2018/11/28
PREPARED	CK
DESIGN	CK
REVIEW	WC
APPROVED	FJH

TITLE
SELECTED SITE PHOTOGRAPHS

PROJECT No.	Phase	Rev.	Figure
1899802	1100	0	**D3**

APPENDIX E

ConTec Investigation Report

CPT Report for CPT 18-1101 and CPT 18-1103

PRESENTATION OF SITE INVESTIGATION RESULTS

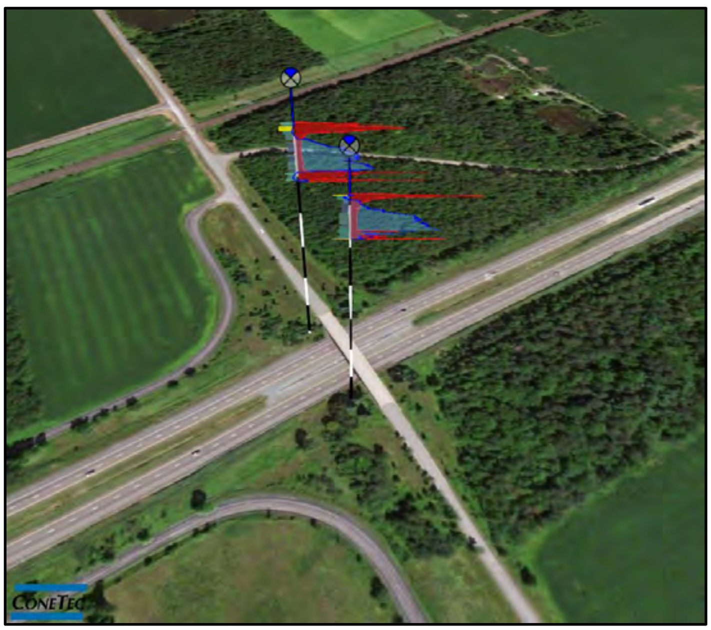
Fraser Road Bridge Replacement

Prepared for:

Golder Associates

ConeTec Job No: 18-05055

Project Start Date: 06-Sep-2018
Project End Date: 07-Sep-2018
Report Date: 17-Sep-2018



Prepared by:

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

Tel: (905) 886-2663
Fax: (905) 886-2664
Toll Free: (800) 504-1116

Email: conetecon@conetec.com
www.conetec.com
www.conetecdataservices.com

Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates at the Fraser Road Bridge. The program consisted of two cone penetration tests (CPT).

Project Information

Project	
Client	Golder Associates
Project	Fraser Road Bridge Replacement
ConeTec project number	18-05055

An image from Google Earth including the CPT test location is presented below.

Rig Description	Deployment System	Test Type
CPT track rig (CME 75)	14 ton rig cylinder	CPT

Coordinates		
Test Type	Collection Method	EPSG Number
CPT	Consumer grade GPS	32618

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Advanced CPT plots with Ic, Su(Nkt), Phi, $N_1$60Ic and Soil Behaviour Type (SBT) scatter plots have been included in the data release package

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm^2)	Sleeve Area (cm^2)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
271:T375F10U200	AD271	15	225	375	10	200
Cone 271 was used for all CPT soundings.						

Calculated Geotechnical Parameter Tables	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPT parameters have been generated and are provided in Excel format files in the release folder. The CPT parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Soils were classified as either drained or undrained based on Normalized Soil Behaviour Type Chart (SBT Q_{tn}) (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures – clayey silt to silty clay (zone 4) 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.

Limitations

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

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

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

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

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in 5 cm^2, 10 cm^2 and 15 cm^2 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^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

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

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

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

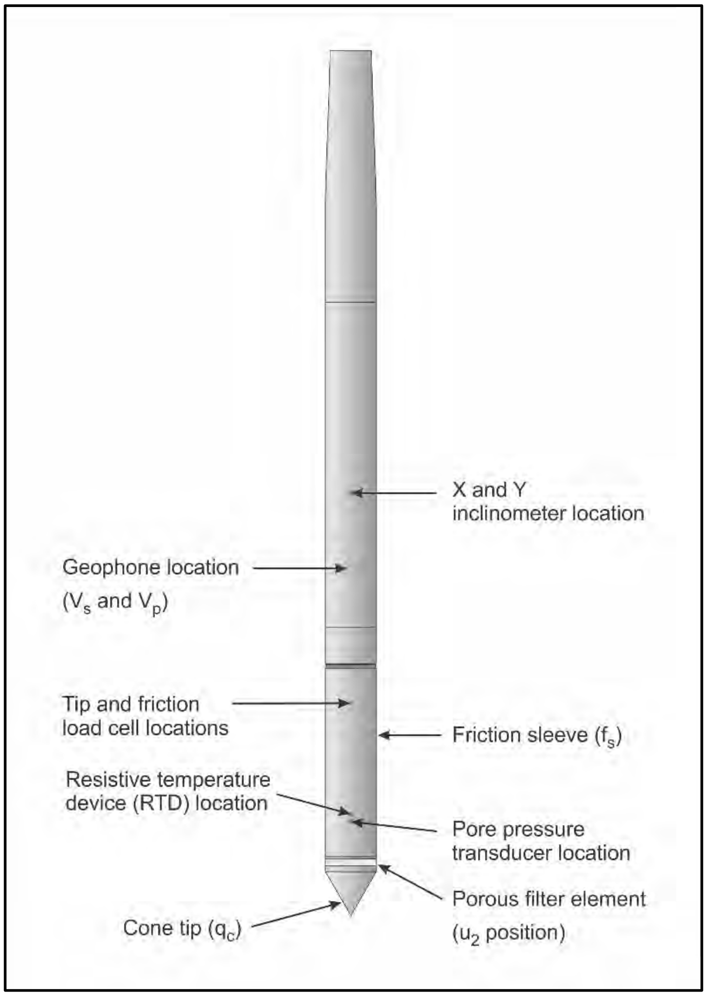


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording interval is 2.5 cm; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

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

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

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

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 38.1 mm are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

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

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

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson et al. (1986) and Robertson (1990, 2009). It should be noted that it is not always possible to accurately identify a soil behaviour type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

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

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

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 (Rf) 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).

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

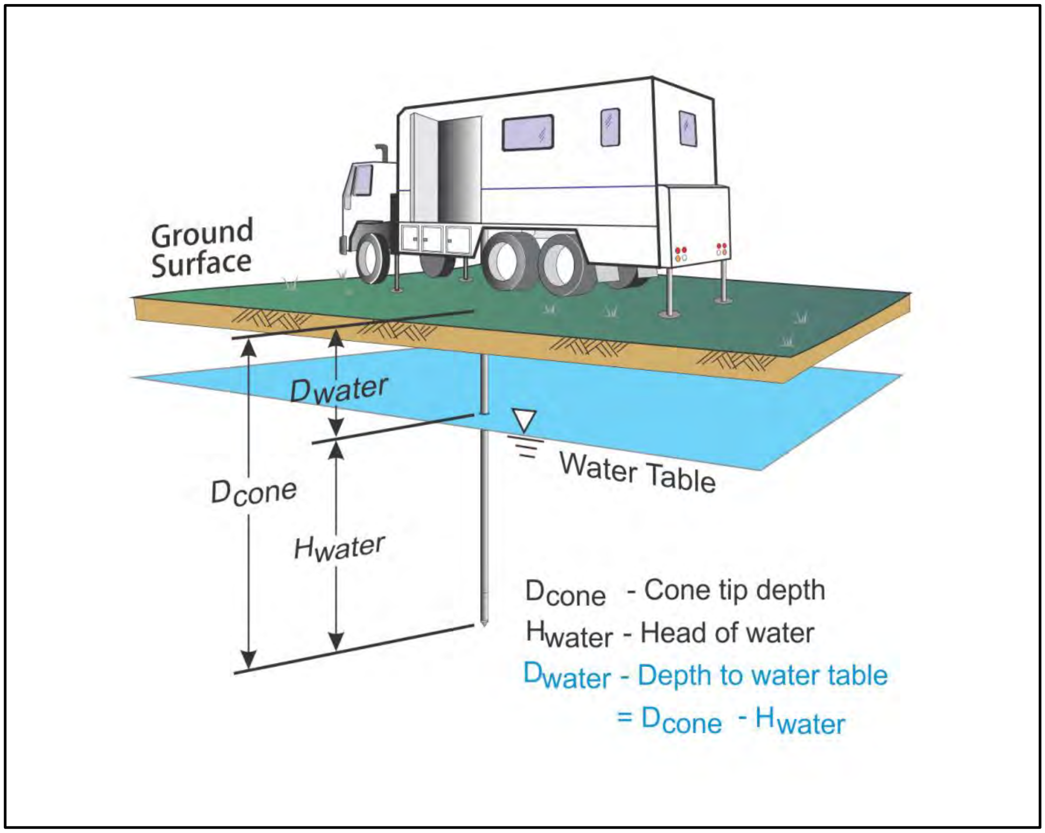


Figure PPD-1. Pore pressure dissipation test setup

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

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

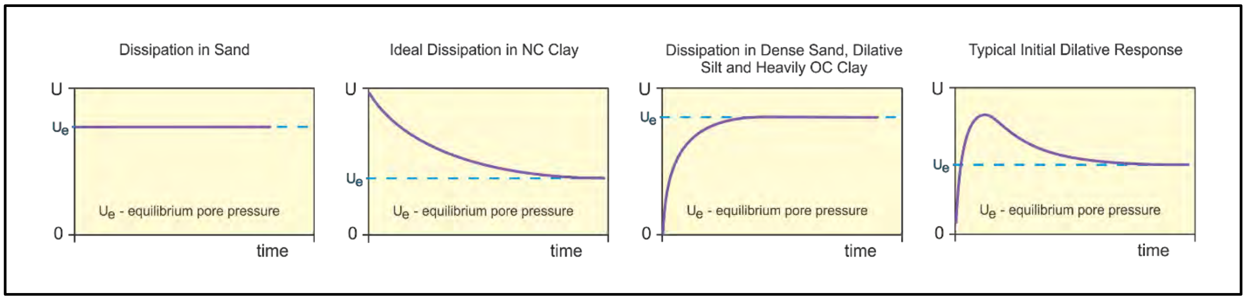


Figure PPD-2. Pore pressure dissipation curve examples

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

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

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

Where:

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

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

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

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

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

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

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

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

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

ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.

Burns, S.E. and Mayne, P.W., 1998, “Monotonic and dilatory pore pressure decay during piezocone tests”, Canadian Geotechnical Journal 26 (4): 1063-1073.

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Jones, G.A. and Van Zyl, D.J.A., 1981, “The piezometer probe: a useful investigation tool”, Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm: 489-495.

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.

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

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.

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.

Robertson, P.K., 2009, “Interpretation of cone penetration tests – a unified approach”, Canadian Geotechnical Journal, Volume 46: 1337-1355.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, “Estimating coefficient of consolidation from piezocone tests”, Canadian Geotechnical Journal, 29(4): 551-557.

Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, “An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils”, Canadian Geotechnical Journal, 36(2): 369-381.

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

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic
- Soil Behaviour Type (SBT) Scatter Plots
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and
Standard Cone Penetration Test Plots

ConeTec

Job No:	18-05055
Client:	Golder Associates
Project:	Fraser Road Bridge Replacement
Start Date:	06-Sep-2018
End Date:	07-Sep-2018

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface[1] (m)	Final Depth (m)	Northing[2] (m)	Easting (m)	Refer to Notation Number
CPT18-1101	18-05055_CP01	06-Sep-2018	271:T375F10U200	4.2	10.600	4995741	535764	
CPT18-1103	18-05055_CP02	07-Sep-2018	271:T375F10U200	4.2	8.925	4995683	535794	3

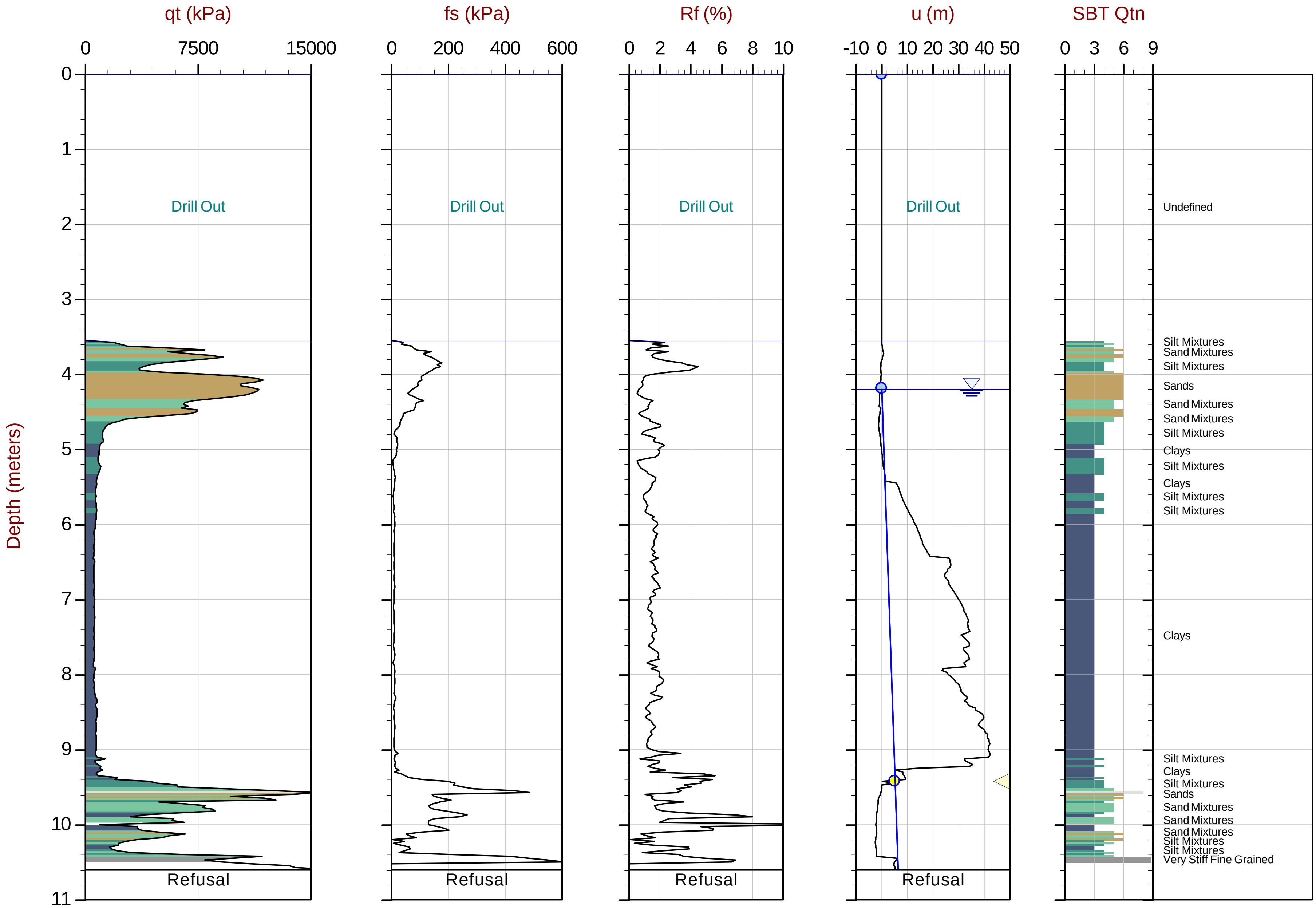
1. The assumed phreatic surface was based on pore pressure dissipation tests unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device with datum WGS84/UTM Zone 18 North.
3. The assumed phreatic surface was based on an adjacent CPT

CONETEC

Golder

Job No: 18-05055
Date: 2018-09-06 10:08
Site: Cornwall, ON

Sounding: CPT18-1101
Cone: 271:T375F10U200



Max Depth: 10.600 m / 34.78 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05055_CP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM: 18N N 4995741m E: 535764m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

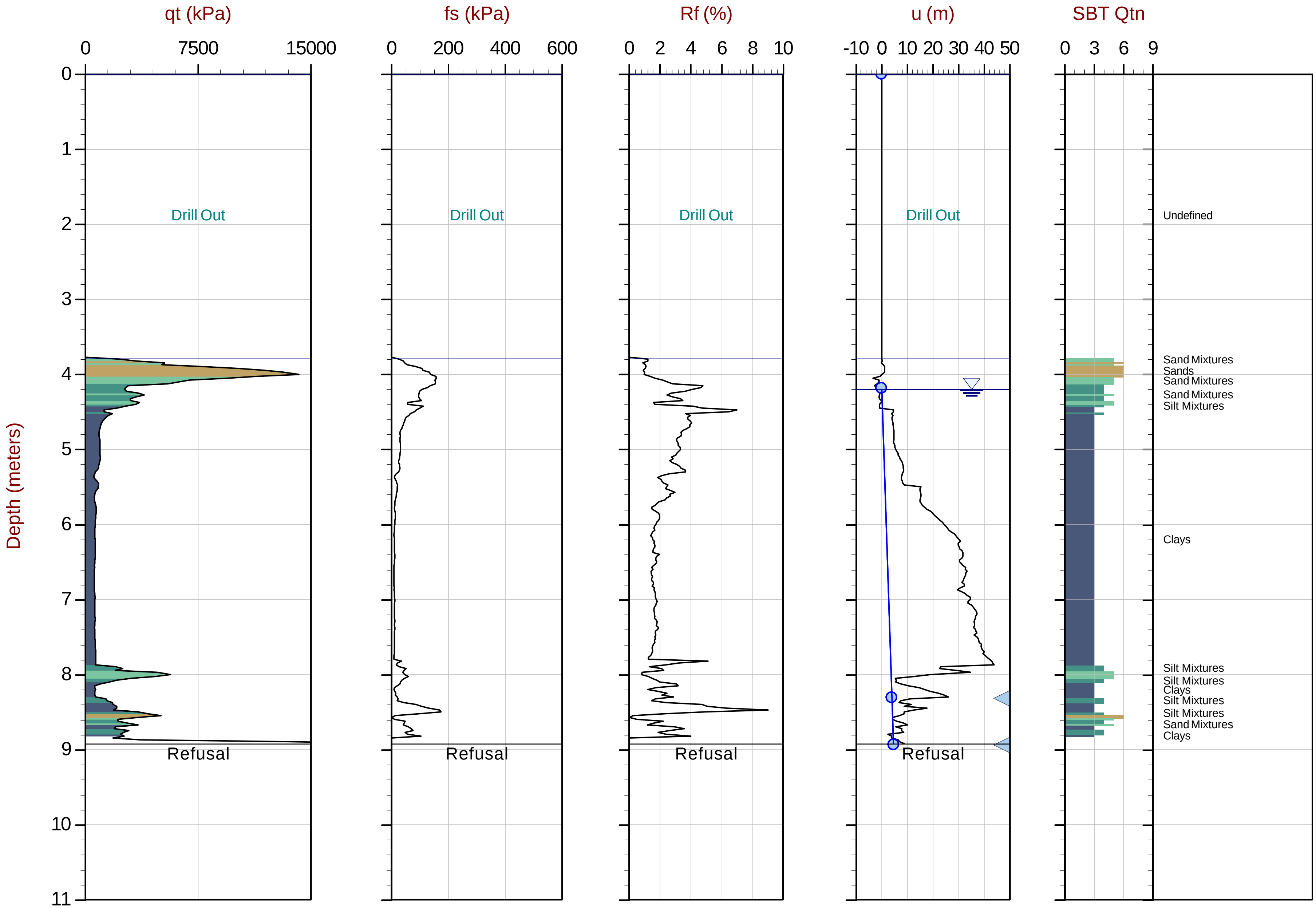
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Golder

Job No: 18-05055
Date: 2018-09-07 06:12
Site: Cornwall, ON

Sounding: CPT18-1103
Cone: 271:T375F10U200



Max Depth: 8.925 m / 29.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05055_CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM: 18N N 4995683m E: 535794m
Sheet No: 1 of 1

Overplot Item: Ueq, Assumed Ueq, Dissipation, Ueq achieved, Dissipation, Ueq not achieved, Hydrostatic Line

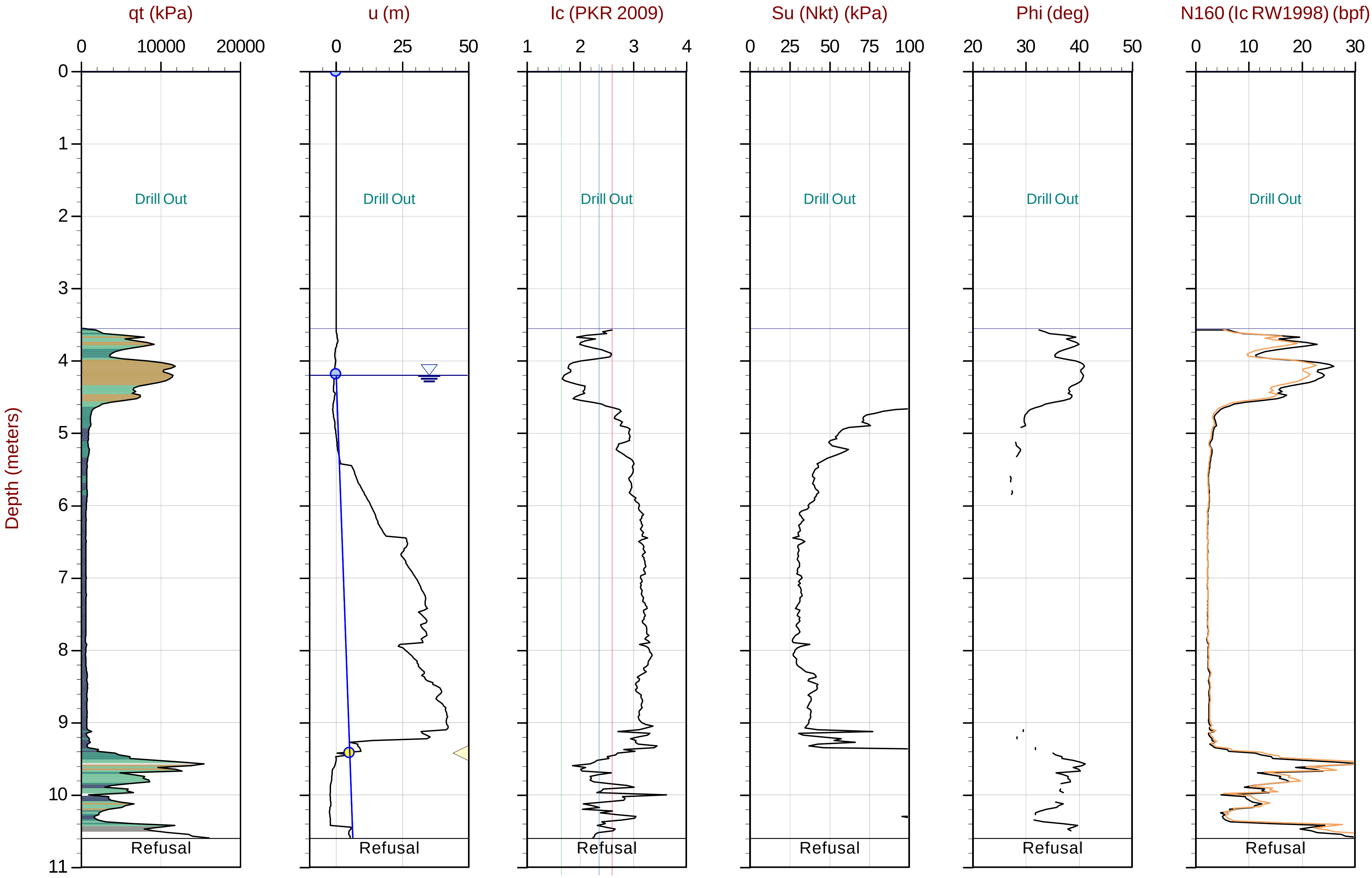
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic

Golder

Job No: 18-05055
Date: 2018-09-06 10:08
Site: Cornwall, ON

Sounding: CPT18-1101
Cone: 271:T375F10U200



Max Depth: 10.600 m / 34.78 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

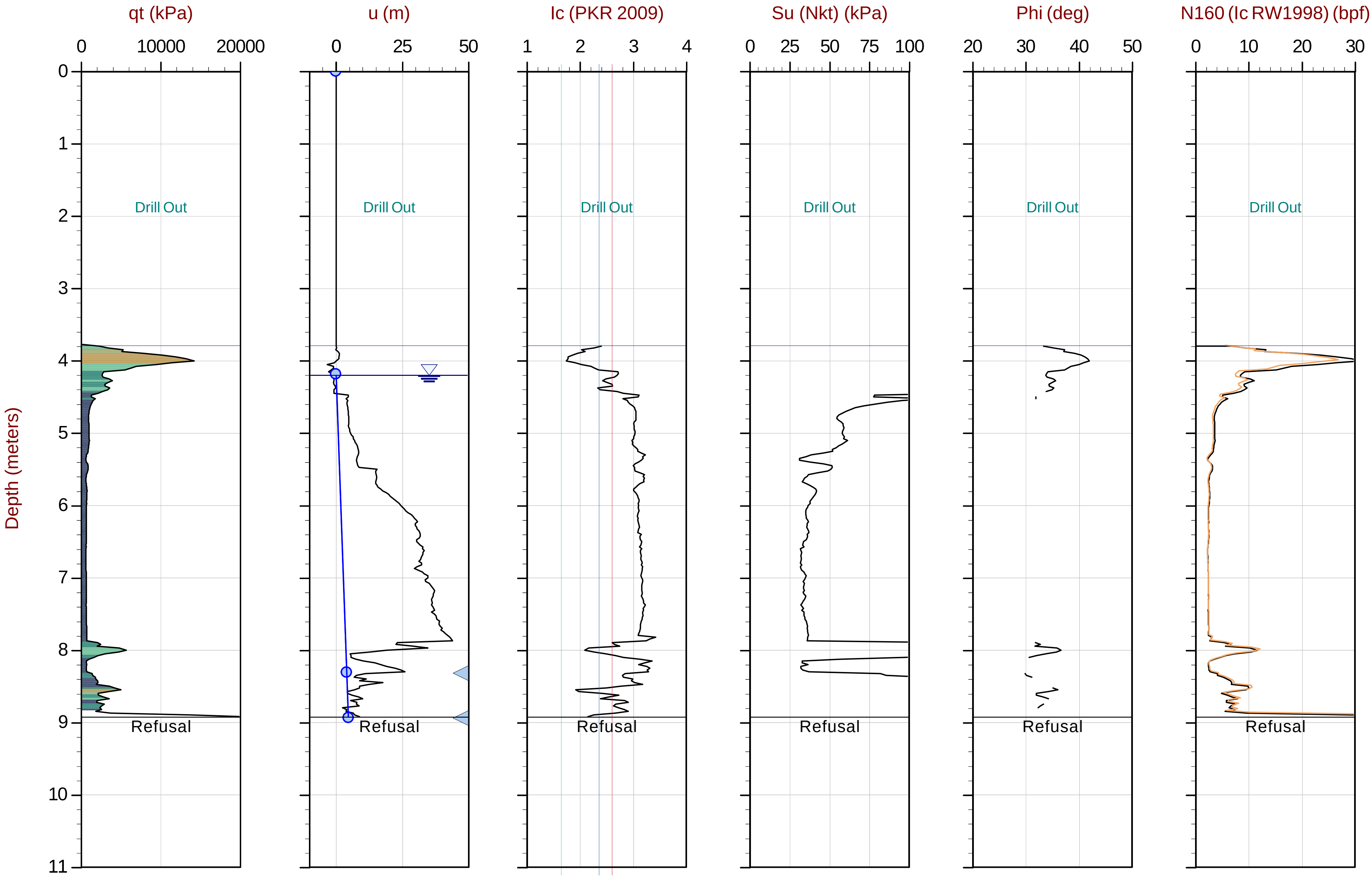
File: 18-05055_CP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM: 18N N 4995741m E: 535764m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

Golder	Job No: 18-05055 Date: 2018-09-07 06:12 Site: Cornwall, ON	Sounding: CPT18-1103 Cone: 271:T375F10U200



Max Depth: 8.925 m / 29.28 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-05055_CP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt: 15.0

SBT: Robertson, 2009 and 2010
Coords: UTM: 18N N 4995683m E: 535794m
Sheet No: 1 of 1

Overplot Item: Ueq Assumed Ueq Dissipation, Ueq achieved Dissipation, Ueq not achieved Hydrostatic Line

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

Soil Behaviour Type (SBT) Scatter Plots

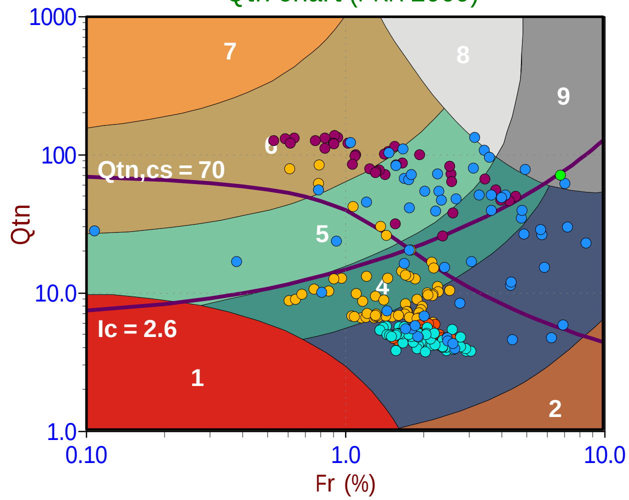
CONETEC

Golder

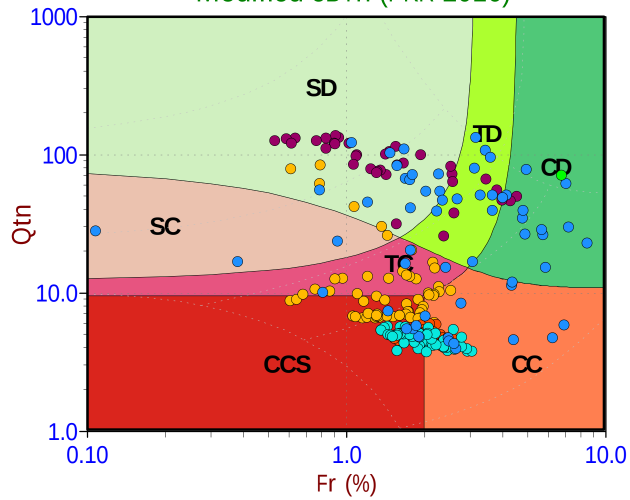
Job No: 18-05055
Date: 2018-09-06 10:08
Site: Cornwall, ON

Sounding: CPT18-1101
Cone: 271:T375F10U200

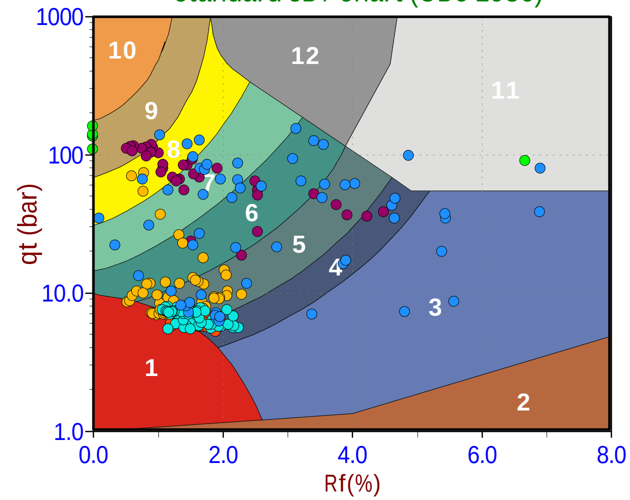
Qtn Chart (PKR 2009)



Modified SBTn (PKR 2016)



Standard SBT Chart (UBC 1986)



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)

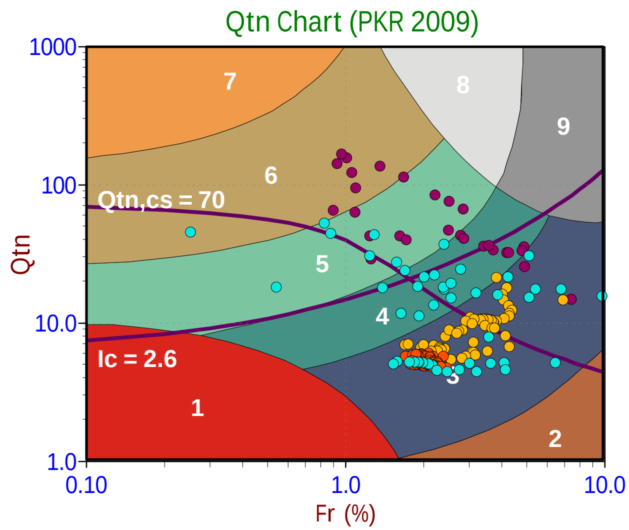
Legend
- Sensitive Fines
- Organic Soil
- Clay
- Silty Clay
- Clayey Silt
- Silt
- Sandy Silt
- Silty Sand/Sand
- Sand
- Gravelly Sand
- Stiff Fine Grained
- Cemented Sand

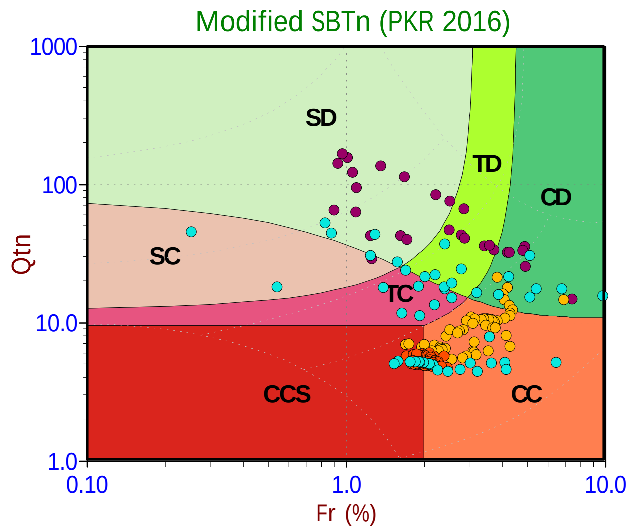
CONETEC

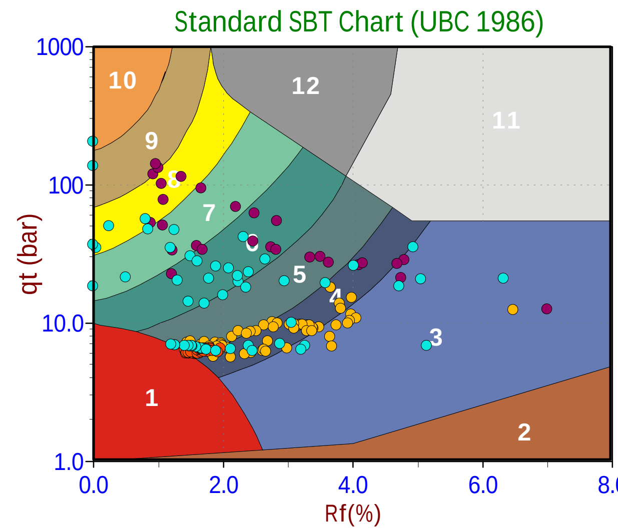
Golder

Job No: 18-05055
Date: 2018-09-07 06:12
Site: Cornwall, ON

Sounding: CPT18-1103
Cone: 271:T375F10U200







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

Pore Pressure Dissipation Summary and
Pore Pressure Dissipation Plots

Job No: 18-05055
Client: Golder Associates
Project: Fraser Road Bridge Replacement
Start Date: 06-Sep-2018
End Date: 07-Sep-2018

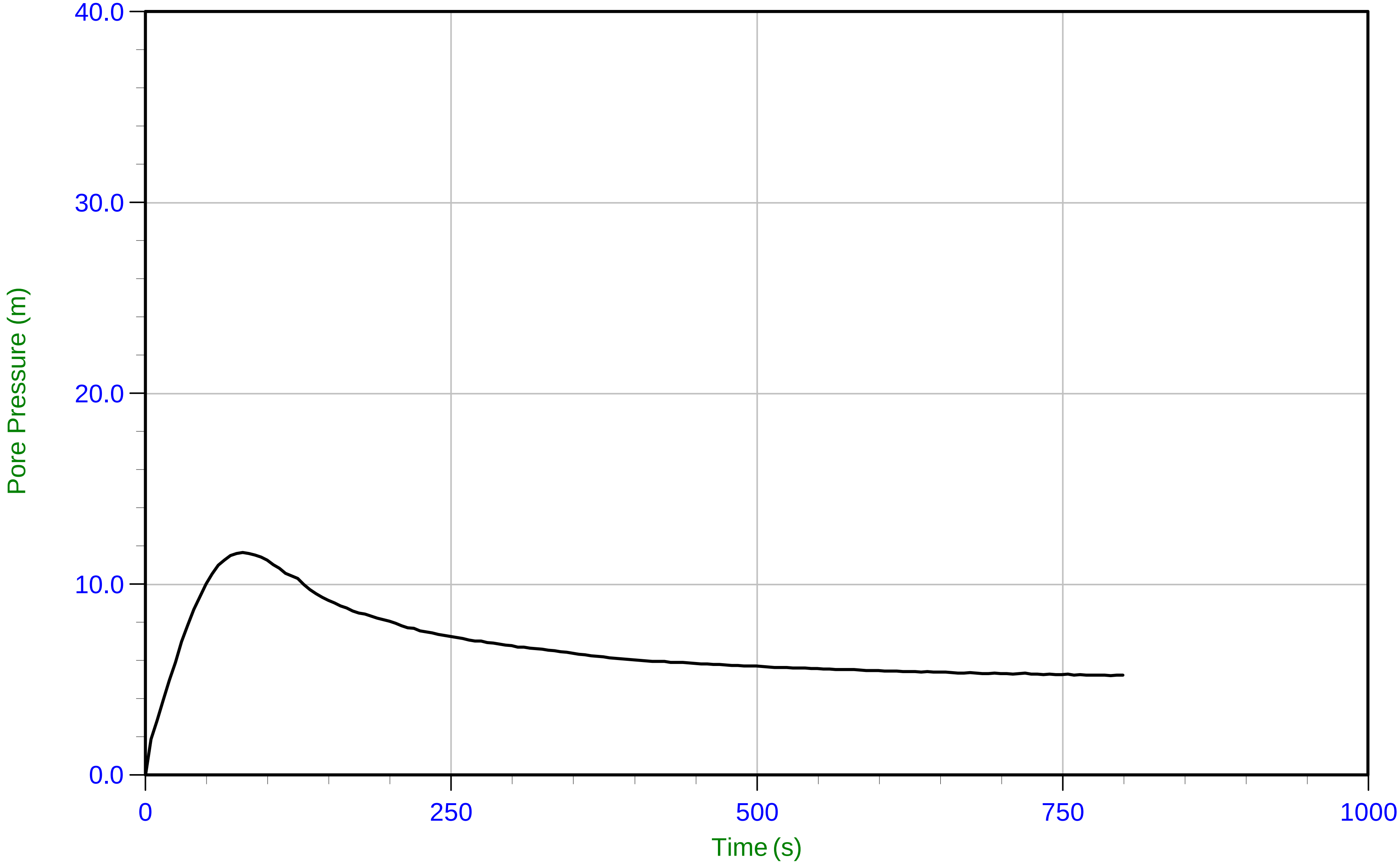
CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm^2)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U_{eq} (m)	Calculated Phreatic Surface (m)	Estimated Phreatic Surface (m)	t_{50} [a] (s)	Assumed Rigidity Index (I_r)	c_h [b] (cm^2/min)
CPT18-1101	18-05055_CP01	15	800	9.425	5.2	4.2				
CPT18-1103	18-05055_CP02	15	1200	8.315	Not Achieved		4.2	718	100	1.0
CPT18-1103	18-05055_CP02	15	900	8.925	Not Achieved		4.2	229	100	3.1

a. Time is relative to where umax occurred

b. Houlsby and Teh, 1991

ConeTec	Golder	Job No: 18-05055 Date: 09/06/2018 10:08 Site: Cornwall, ON	Sounding: CPT18-1101 Cone: 271:T375F10U200 Area=15 cm²



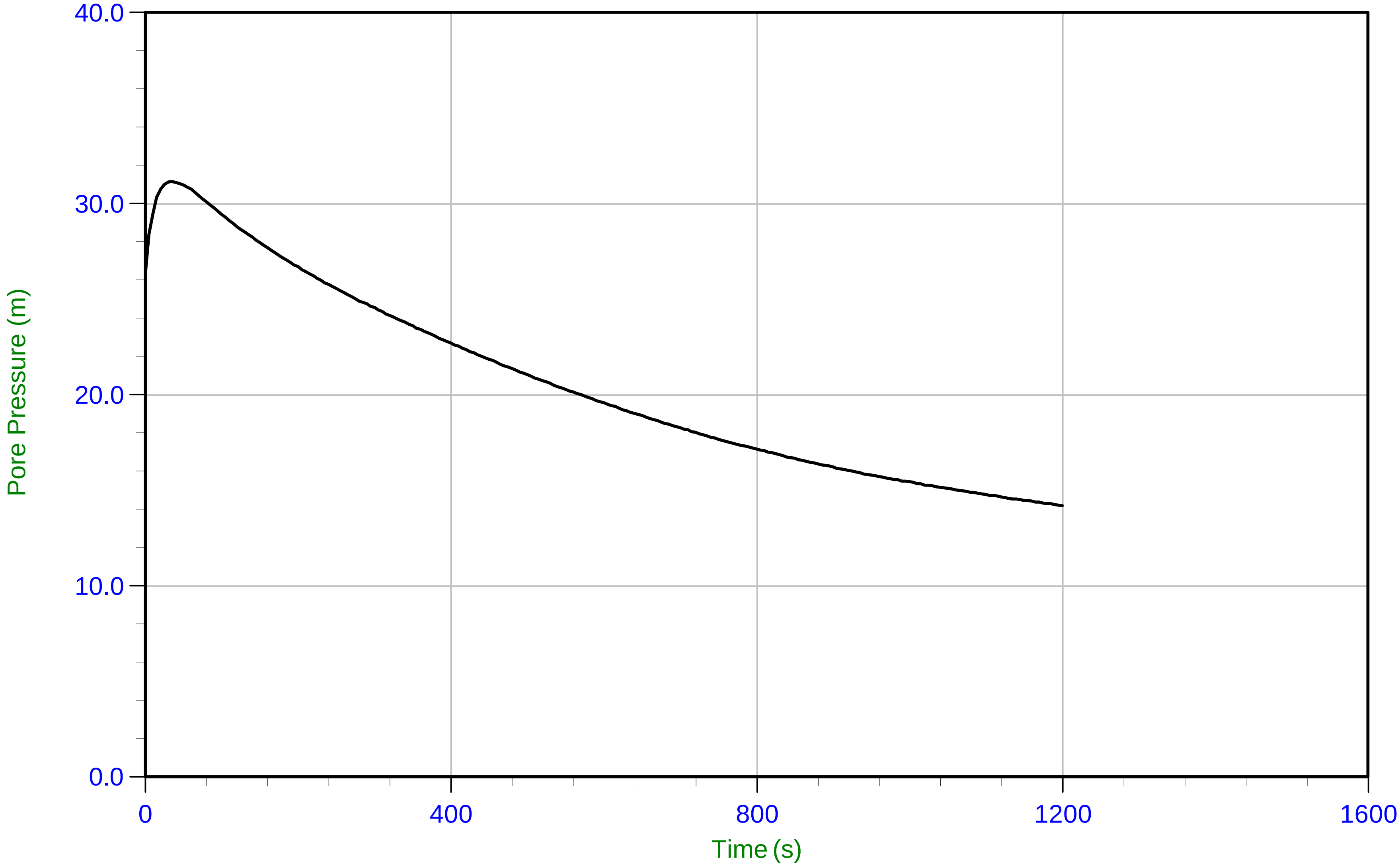
Trace Summary:

Filename: 18-05055_CP01.PPF	U Min: -0.1 m	WT: 4.189 m / 13.743 ft
Depth: 9.425 m / 30.922 ft	U Max: 11.7 m	Ueq: 5.2 m
Duration: 800.0 s		

Golder

Job No: 18-05055
Date: 09/07/2018 06:12
Site: Cornwall, ON

Sounding: CPT18-1103
Cone: 271:T375F10U200 Area=15 cm²



Trace Summary:

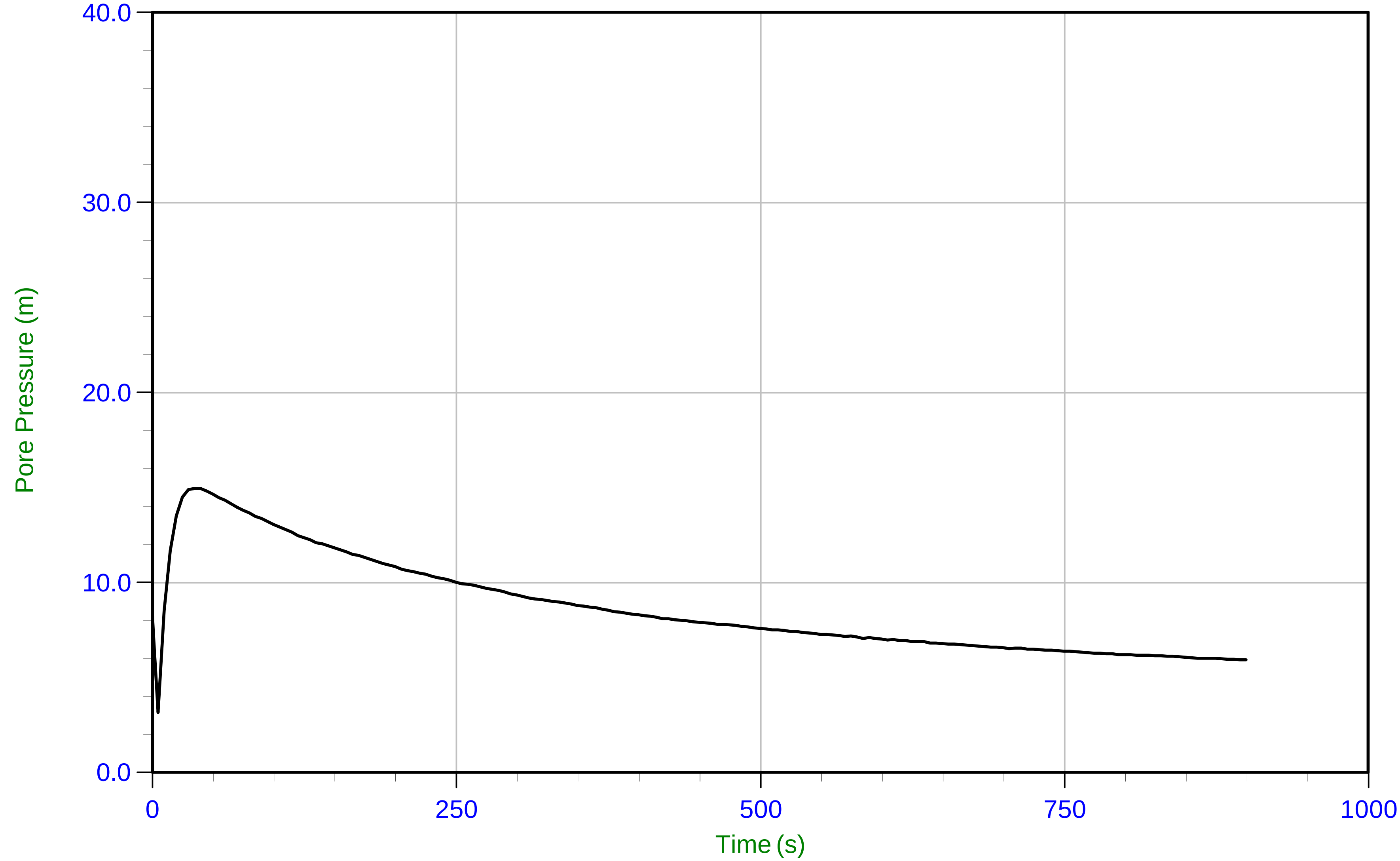
Filename: 18-05055_CP02.PPF
Depth: 8.315 m / 27.280 ft
Duration: 1200.0 s

U Min: 14.2 m
U Max: 31.1 m

WT: 4.189 m / 13.743 ft
Ueq: 4.1 m
U(50): 17.64 m

T(50): 718.2 s
Ir: 100
Ch: 1.0 sq cm/min

CONETEC	Golder	Job No: 18-05055 Date: 09/07/2018 06:12 Site: Cornwall, ON	Sounding: CPT18-1103 Cone: 271:T375F10U200 Area=15 cm²



Trace Summary:

Filename: 18-05055_CP02.PPF	U Min: 3.2 m	WT: 4.189 m / 13.743 ft	T(50): 229.1 s
Depth: 8.940 m / 29.330 ft	U Max: 15.0 m	Ueq: 4.8 m	Ir: 100
Duration: 900.0 s		U(50): 9.85 m	Ch: 3.1 sq cm/min

APPENDIX F

Vertical Seismic Profiling (VSP) Testing Results

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

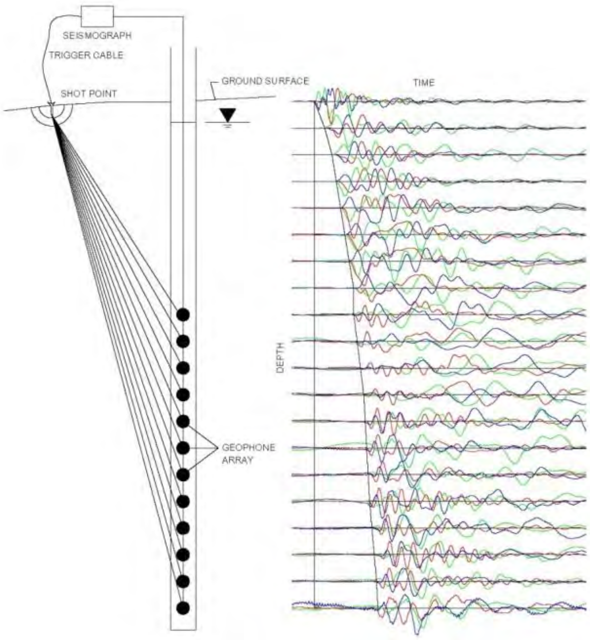
VERTICAL SEISMIC PROFILING TEST RESULTS
HWY 401 FRASER ROAD UNDERPASS, LANCASTER, ONTARIO

This memorandum presents the results of two Vertical Seismic Profiling (VSP) testing carried out at the Fraser Raod 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.

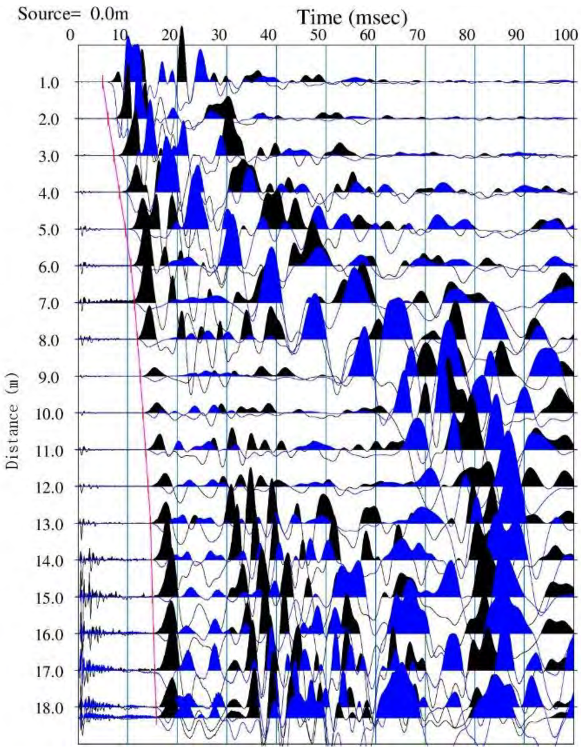


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole BH18-1101.

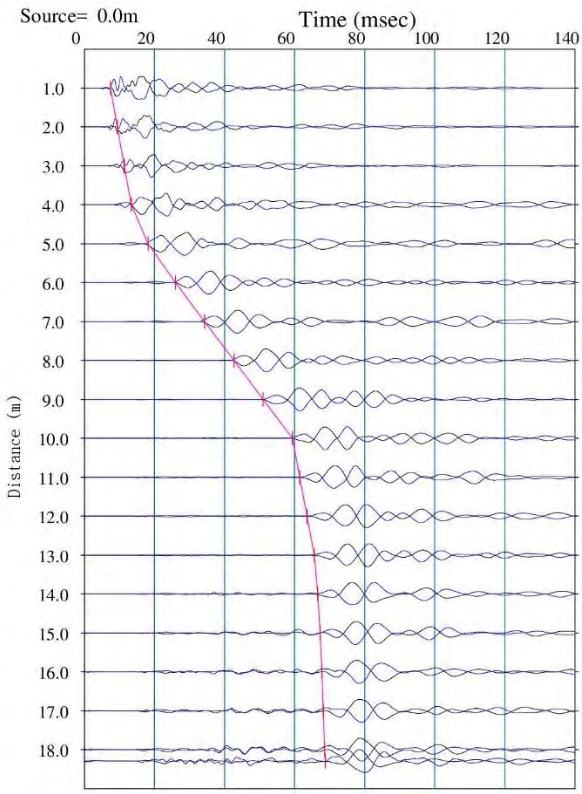


Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 18-1101.

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.

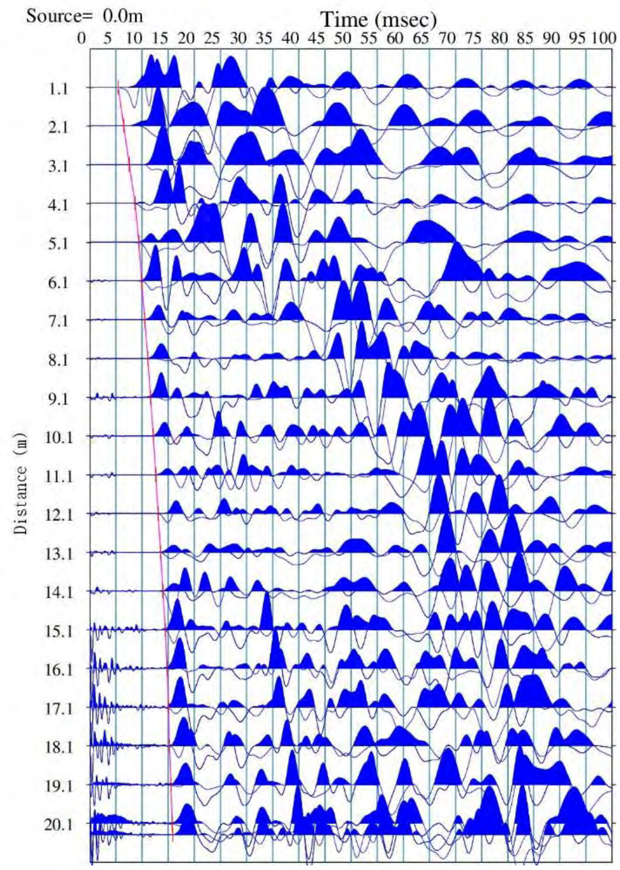


Figure 3: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 18-1103.

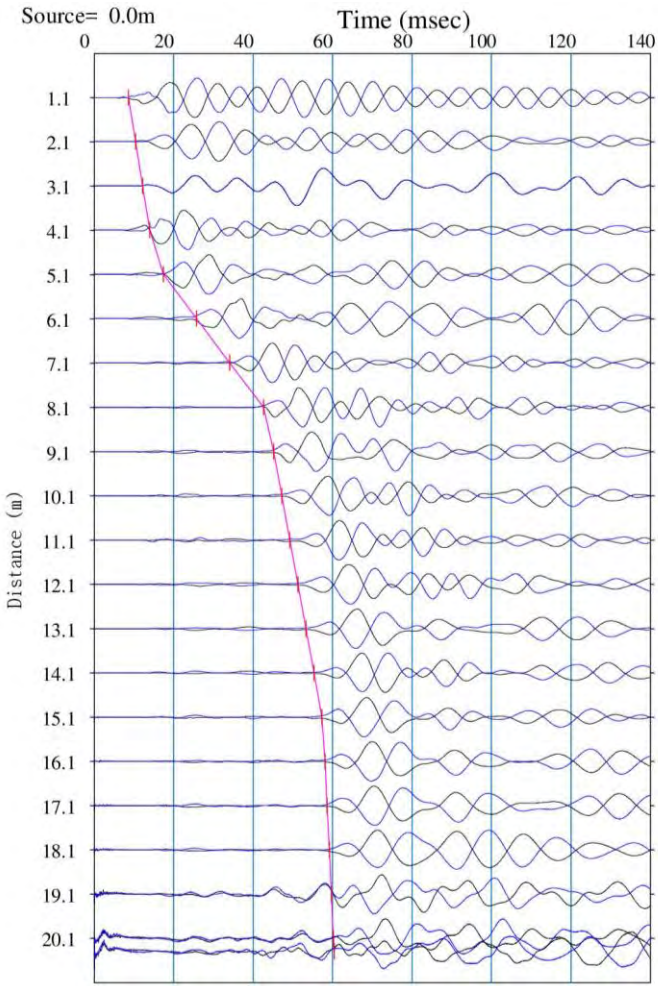


Figure 4: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 18-1103.

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^3 was used for silty sand, 1,550 kg/m^3 for silty clay, and an estimated bulk density of 2,600 kg/m^3 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

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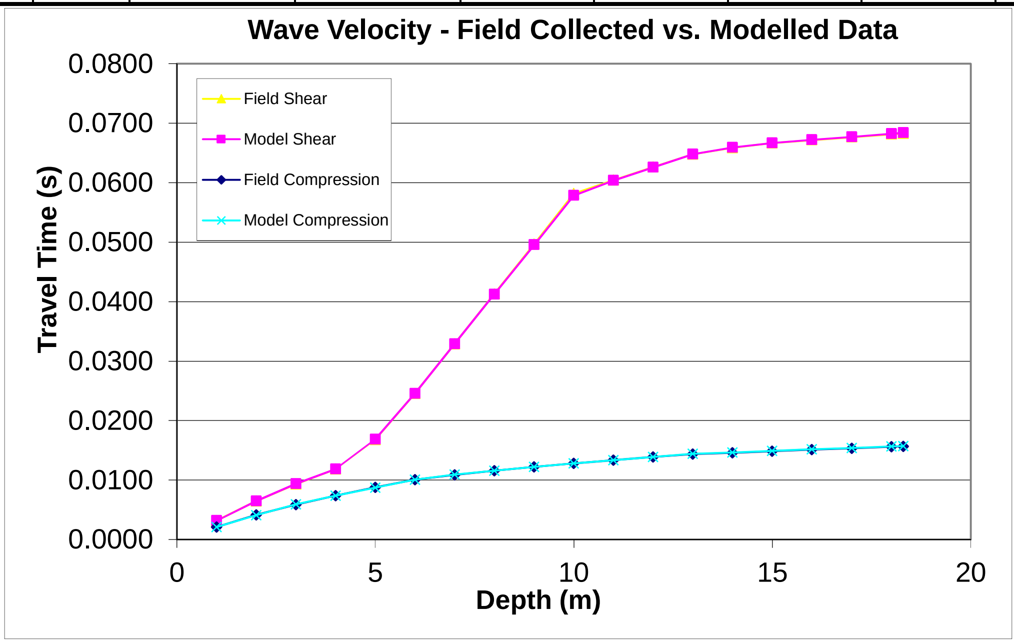
TABLE 1
SHEAR WAVE VELOCITY PROFILE AT BOREHOLE BH18-1101

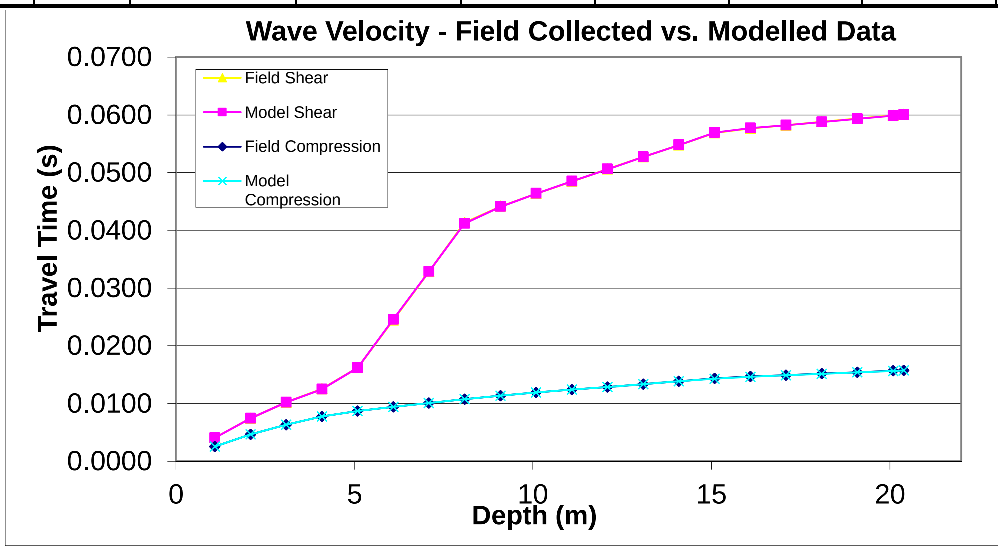
Layer Depth (m)		Velocities (m/s)		Estimated Bulk Density (kg/m^3)	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1.0	480	316	2000	0.12	200	446	195
1.0	2.0	500	305	2000	0.20	186	448	252
2.0	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^3)	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 G

Results of Chemical Analysis

Eurofins Environment Testing Report No. 1818195

Certificate of Analysis

eurofins Environment Testing

Client:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7	Report Number:	1818195
Attention:	Ms. Christine Ko	Date Submitted:	2018-10-04
PO#:		Date Reported:	2018-10-12
Invoice to:	Golder Associates Ltd. (Ottawa)	Project:	1899802/1100
		COC #:	836335

Group	Analyte	MRL	Units	Guideline	1391697	1391698	1391699
				Lab I.D.	1391697	1391698	1391699
				Sample Matrix	Soil	Soil	Soil
				Sample Type			
				Sampling Date	2018-10-04	2018-09-18	2018-09-10
				Sample I.D.	18-1101 SA 5	18-1102 SA 5	18-1103 SA11
Anions	Cl	0.002	%		0.087	0.049	0.006
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.54	1.00	0.46
	pH	2.00			8.11	8.13	8.04
	Resistivity	1	ohm-cm		1850	1000	2220
Subcontract	S2-	0.2	ug/g		3.1	<0.2	<0.2
	SO4	20	ug/g		360	30	240

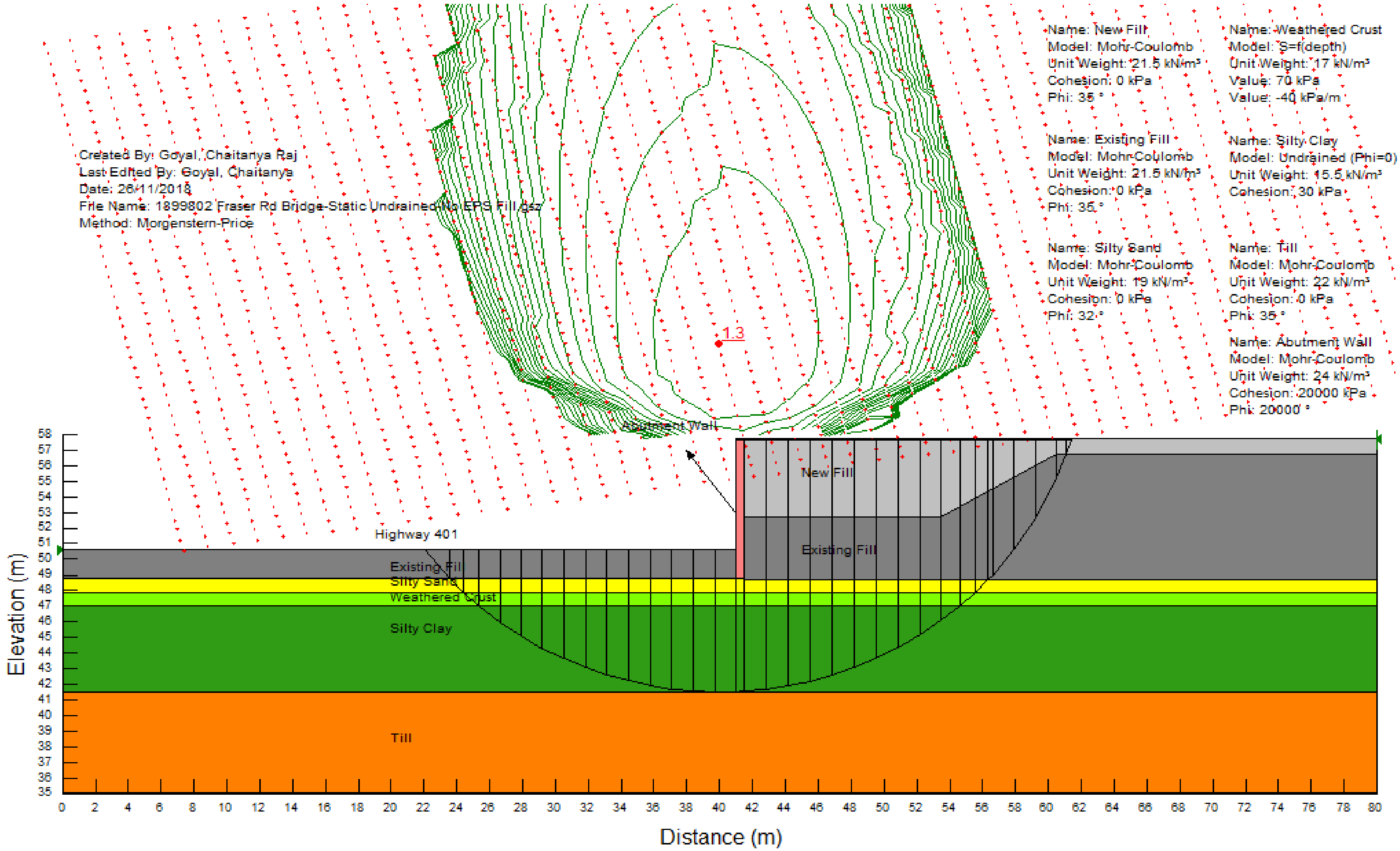
Guideline = *** = Guideline Exceedence**

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 H

Results of Slope Stability Analysis



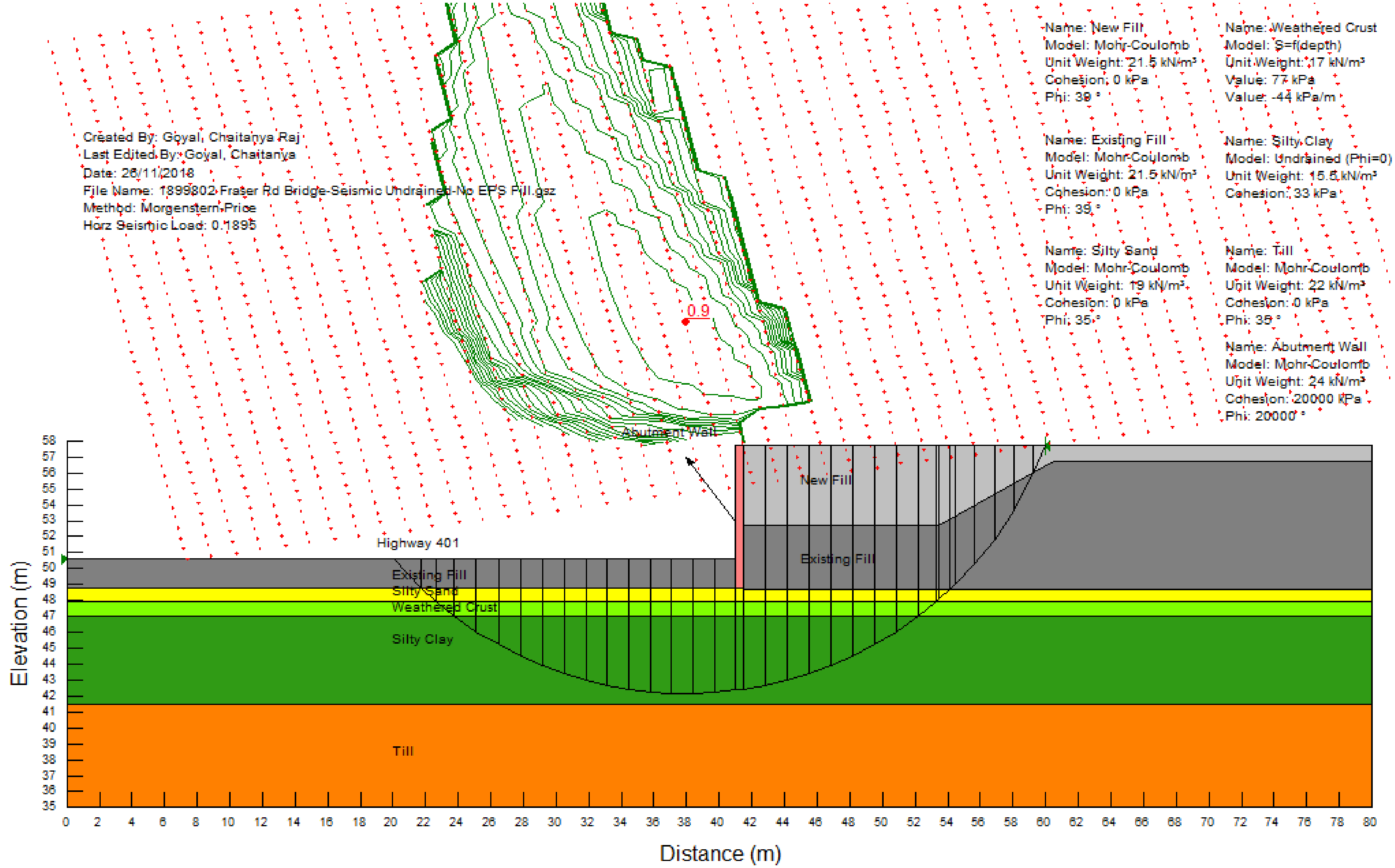
GOLDER

FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN
REPLACEMENT OF FRASER ROAD UNDERPASS AT HIGHWAY 401
UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON

SLOPE STABILITY ANALYSIS FOR STATIC UNDRAINED CONDITIONS
NO EPS LIGHTWEIGHTFILL

Project No.	1899802
Drawn:	CRG
Date:	28/11/2018
Checked:	CK
Review:	WC

FIGURE H1A



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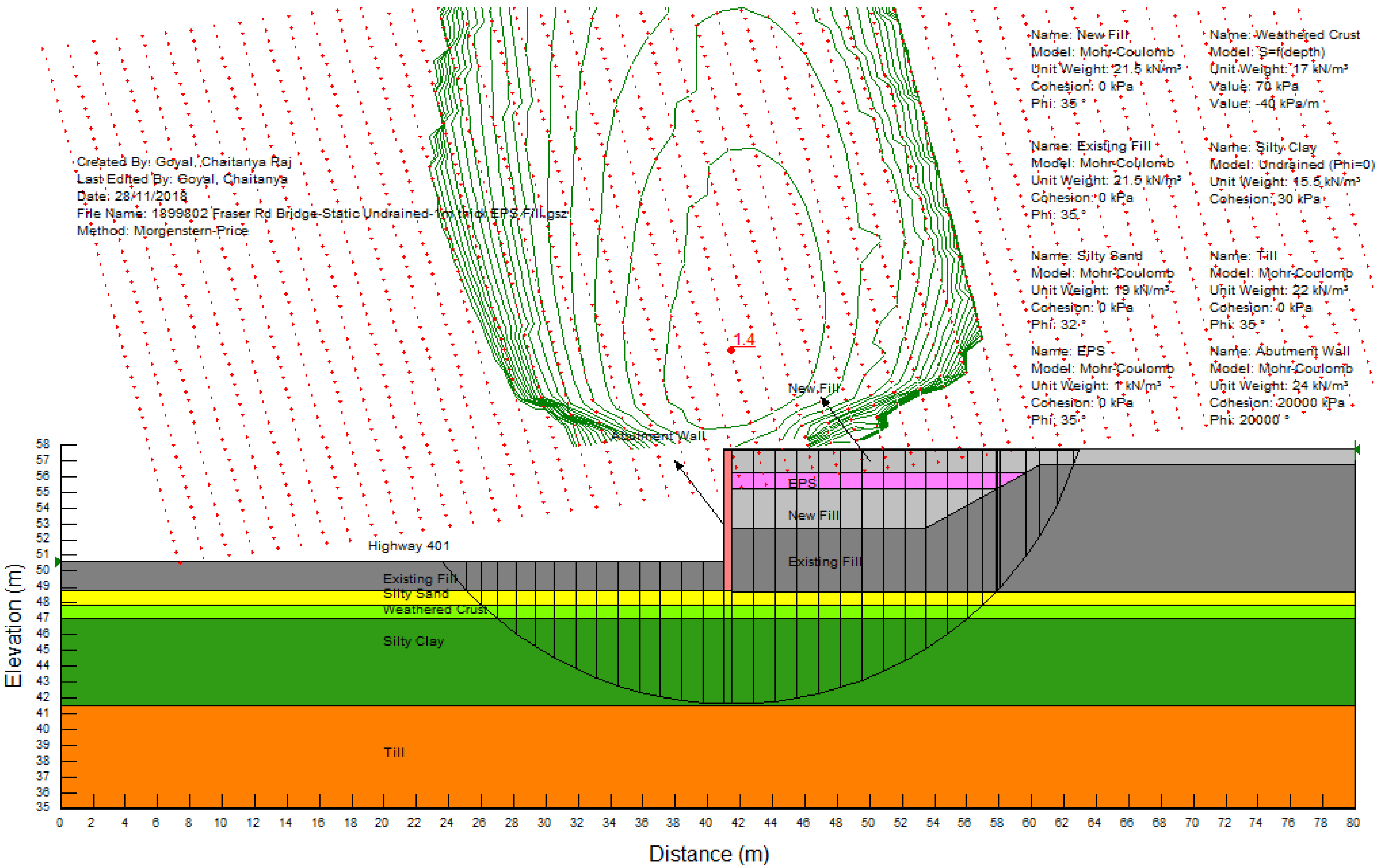
FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN
REPLACEMENT OF FRASER ROAD UNDERPASS AT HIGHWAY 401
UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON

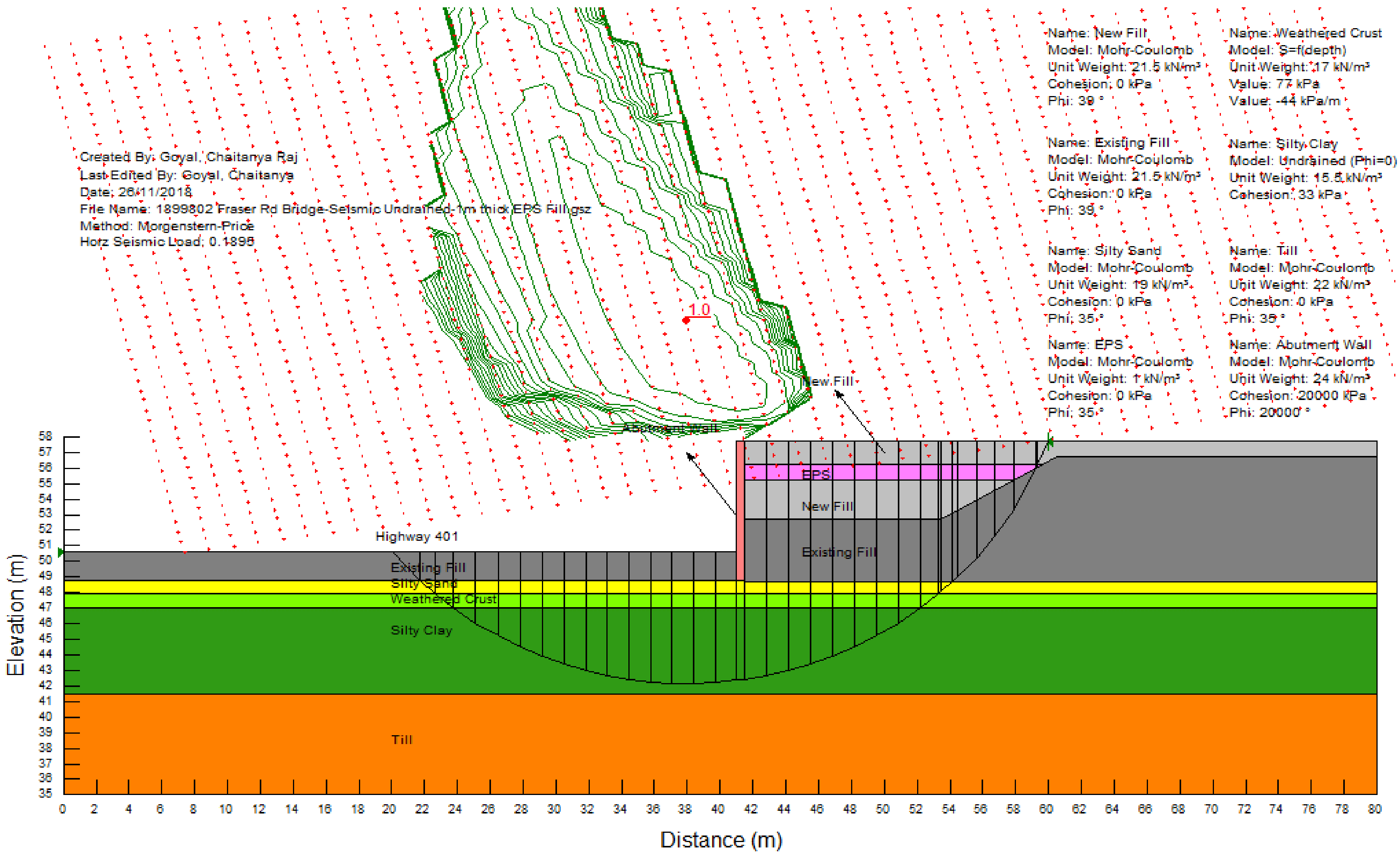
SLOPE STABILITY ANALYSIS FOR SEISMIC UNDRAINED CONDITIONS
NO EPS LIGHTWEIGHTFILL

Project No.	1899802
Drawn:	CRG
Date:	28/11/2018
Checked:	CK
Review:	WC

FIGURE H1B



GOLDER	FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN REPLACEMENT OF FRASER ROAD UNDERPASS AT HIGHWAY 401 UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON SLOPE STABILITY ANALYSIS FOR STATIC UNDRAINED CONDITIONS EPS LIGHTWEIGHTFILL = 1 M THICK	Project No. 1899802 Drawn: CRG Date: 28/11/2018 Checked: CK Review: WC	**FIGURE H2A**

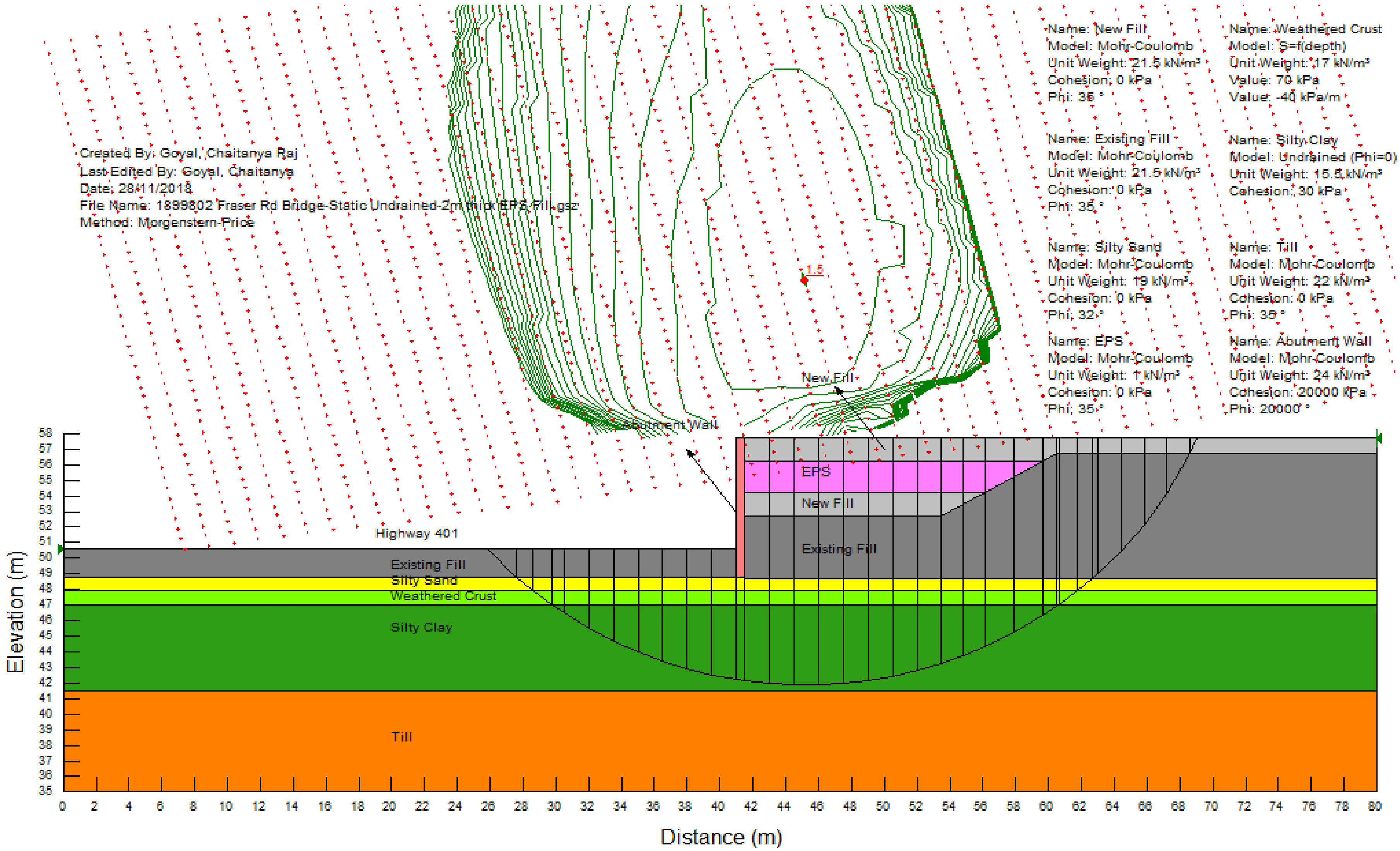


FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN
REPLACEMENT OF FRASER ROAD UNDERPASS AT HIGHWAY 401
UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON

SLOPE STABILITY ANALYSIS FOR SEISMIC UNDRAINED CONDITIONS
EPS LIGHTWEIGHTFILL = 1 M THICK

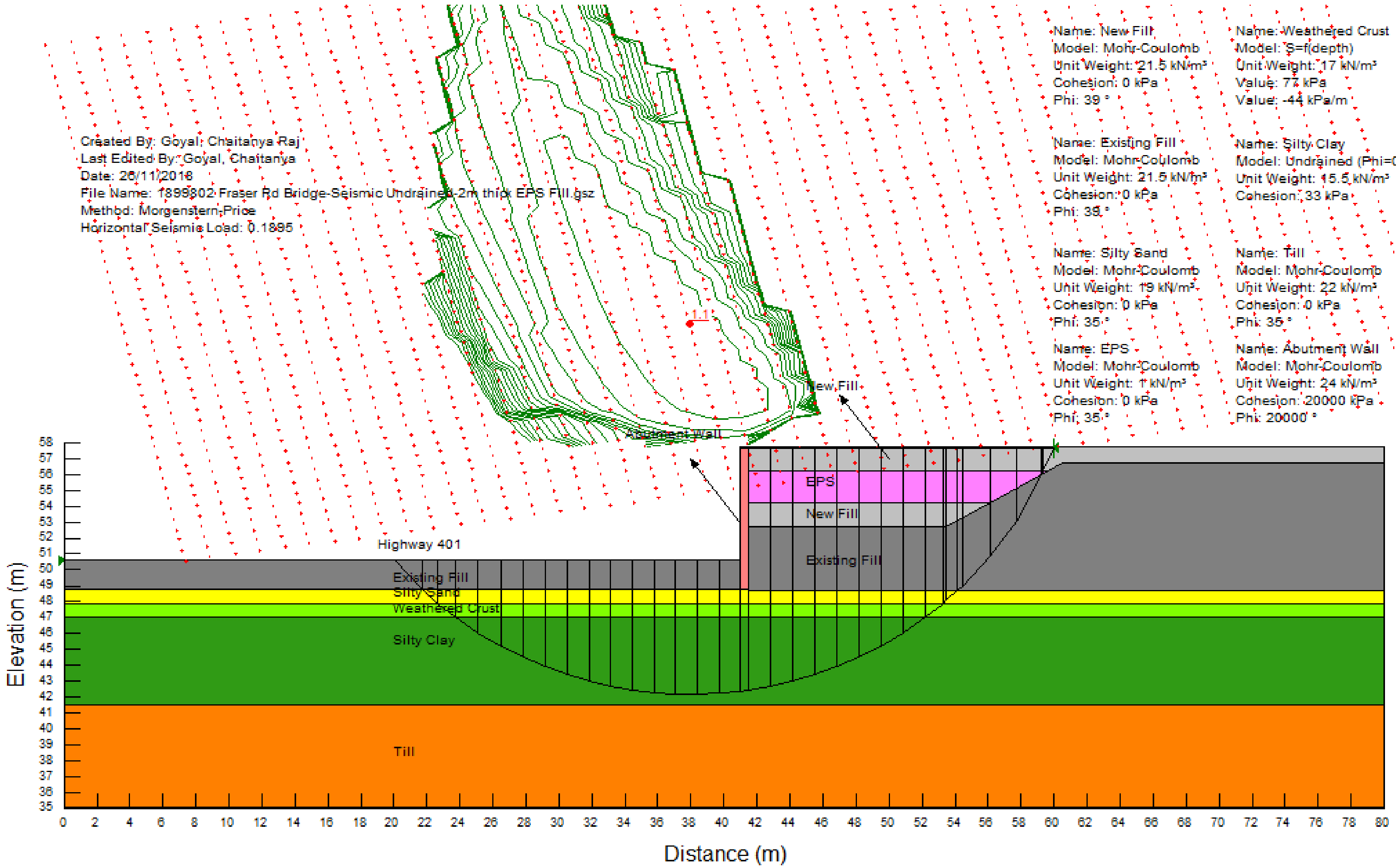
Project No.	1899802
Drawn:	CRG
Date:	28/11/2018
Checked:	CK
Review:	WC

FIGURE H2B



GOLDER	FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN REPLACEMENT OF FRASER ROAD UNDERPASS AT HIGHWAY 401 UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON SLOPE STABILITY ANALYSIS FOR STATIC UNDRAINED CONDITIONS EPS LIGHTWEIGHTFILL = 2 M THICK	Project No. 1899802 Drawn: CRG Date: 28/11/2018 Checked: CK Review: WC

FIGURE H3A



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FOUNDATION INVESTIGATION AND PRELIMINARY DESIGN
REPLACEMENT OF FRASER ROAD UNDERPASS AT HIGHWAY 401
UNITED COUNTIES OF STORMONT, DUNDAS AND GLENGARRY, ON

SLOPE STABILITY ANALYSIS FOR SEISMIC UNDRAINED CONDITIONS
EPS LIGHTWEIGHTFILL = 2 M THICK

Project No.	1899802
Drawn:	CRG
Date:	28/11/2018
Checked:	CK
Review:	WC

FIGURE H3B

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