September 7, 2017

FOUNDATION INVESTIGATION AND DESIGN REPORT

HIGHWAY 17 CONISTON CPR OVERHEAD
TEMPORARY DETOUR STRUCTURE, SITE NO. 46-123
SUDBURY DISTRICT, TOWNSHIP OF DRYDEN
AGREEMENT NO. 5015-E-0045 - WORK ORDER 1

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GEOCRES NO.: 41I-352

REPORT

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Golder Associates

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PART A

FOUNDATION INVESTIGATION REPORT
HIGHWAY 17 CONISTON CPR OVERHEAD
TEMPORARY DETOUR, SITE NO. 46-123
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO), to provide foundation engineering services for a temporary modular bridge (TMB) and detour embankment widening associated with the rehabilitation of the Canadian Pacific Railway (CPR) Overhead structure located on Highway 17 in Coniston, Ontario, approximately 2.8 km west of the Highway 17-Highway 537 junction in the Sudbury Area. This work has been carried out under the Retainer Assignment Agreement # 5015-E-0045 – Work Order #1. The highway and structural engineering aspects of the project are being carried out under separate contract between Morrison Hershfield (MH) and MTO.

The purpose of this investigation is to establish the subsurface conditions at the locations of the foundation element of the temporary modular bridge and along the proposed detour embankment widening, adjacent to the Coniston CPR Overhead by methods of borehole drilling, in situ testing and laboratory testing of selected soil samples.

2.0 SITE DESCRIPTION AND BACKGROUND INFORMATION

We understand that the existing CPR Overhead is to be rehabilitated which requires a temporary detour (i.e. widening of the existing approach embankments) and a temporary modular bridge (TMB). We understand that a three-span TMB will be located about 4 m to the north of the existing bridge and the existing approach embankments will require widening along the north side slope.

The existing west approach embankment is about 10 m high and may have been constructed of a combination of granular fill layers and cohesive fill layers. Based on information presented in the previous bridge General Arrangement (GA) drawings and previous borehole information we understand that the east approach embankment is comprised of rock fill. Northeast of the existing east abutment there is a visible bedrock outcrop and the rock is dipping to the west (towards the rail tracks) and is 9 m high at the abutment front slope. Blast rock fill is visually noted along the north side of the east approach embankment.

In general, the topography in the area of the Overhead structure consists of rolling terrain, including densely treed areas, bedrock outcrops, and low-lying swamps containing organic soils and areas of standing water and various types of vegetation. The CPR right-of-way appears to be aligned within a natural valley between bedrock outcrops. The railway tracks are aligned in a northeast-southwest direction, while the Overhead structure and Highway 17 are aligned in an east-west direction, skewed to the track alignment. The existing ground surface along the proposed detour and TMB alignment varies greatly as the centreline of the detour is positioned approximately along the mid-slope of the north side of the existing Highway 17 approach embankments and due to the exposed sloping bedrock noted above. Select site photographs are attached following the text of this report.

2.1 Previous Investigations

Previous foundation investigations for the existing bridge at the site carried out in 1975 indicates that the native material at the site consists of deposits of varved silty clay to clayey silt underlain by a deposit of silty sand to gravelly sandy silt, which is in turn underlain by bedrock. The details of this subsurface investigation are presented in:

- Geocon Ltd., 1977. Foundation Investigation Report for CPR Overhead at Coniston W.P. 158-74-01, Site 46-123, Hwy. 17, District 17, Sudbury. Ministry of Transportation and Communications, Ontario. Geocres No. 41I-140.

The locations of these boreholes have been converted from previous station and offset to approximate coordinates in MTM NAD83 (Zone12) along with the ground surface elevations as follows:

Borehole	Location (MTM NAD 83, Zone12)		Ground Surface Elevation (m)
	Northing	Easting	
1	5149839.2	318797.0	262.1
2	5149851.8	318825.8	252.6
3	5149838.1	318849.8	262.4
4	5149837.4	318866.3	262.6
5	5149832.3	318785.0	262.1
6	5149832.5	318797.7	262.2
7	5149819.1	318822.0	253.8
8	5149830.3	318856.5	262.5

In 2016, Golder conducted a foundation investigation to support retaining walls adjacent to the tracks.

- Golder Associates Ltd., 2016. Foundation Investigation Report for RSS Walls at the Coniston CPR Overhead 2.8 km West of Highway 537/17 Junction, Site # 46-123, Sudbury Area, Assignment No. 15, Agreement No. 5013-E-0034, W.P. 5165-10-01. Geocres No. 41J-342

In summary, these boreholes encountered a 0.7 m to 1.5 m thick layer of gravelly silt sand to sand and gravel fill from ground surface in places underlain by layers of organic clay and/or sandy silt between about 0.2 to 0.8 m thick; in turn underlain by a deposit of varved clayey silt to clay between 4.6 m and 5.8 m thick, which is in places underlain by a 0.7 m thick and potentially up to 4.2 m thick deposits of silt and sand.

The locations of these boreholes in MTM NAD83 (Zone12) coordinates and ground surface elevations referenced to Geodetic datum are as follows:

Borehole	Location (MTM NAD 83, Zone12)		Ground Surface Elevation (m)
	Northing	Easting	
BH1	5149845.7	318819.3	253.9
BH2	5149825.5	318800.2	254.1
BH3	5149846,5	318844.3	254.0
BH4	5149824.0	318881.4	254.4

The locations of the 1975 and 2016 Foundations Investigations are shown on Drawing 1. The pertinent subsurface information from the 1975 and 2016 Foundations Investigations in presented in Appendices A and B, respectively.

3.0 INVESTIGATION PROCEDURE

The current investigation for the detour and TMB was carried out between April 18 and May 1, 2017, during which time a total of nine boreholes (C17-1 to C17-9) and four dynamic cone penetration tests (DCPT) were advanced at the locations shown on Drawing 1. The Record of Borehole and Drillhole sheets are presented in Appendix C.

The field investigation was carried out using a buggy-mounted CME 55 drill rig and portable Hilti core drilling equipment supplied and operated by Landcore Drilling Ltd. of Chelmsford, Ontario. Boreholes C17-1 and C17-2, were advanced using a 50 mm inside diameter core barrel advanced by a Hilti coring machine. Boreholes C17-3 to C17-9 were advanced using 108 mm inside diameter hollow stem augers with NW casing and wash boring techniques (where required). In general, soil samples were obtained at depth intervals of 0.75 m and 1.5 m, using a 50 mm O.D. split-spoon sampler driven by an automatic hammer, carried out in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Samples of the cohesive soils were obtained using 76 mm O.D. thin walled Shelby Tubes (ASTM D1587). Field vane shear tests were completed within cohesive deposits in accordance with ASTM D2573, using MTO Standard 'N' size vanes. All boreholes were backfilled with bentonite and cuttings upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The boreholes were sampled to depths between 2.3 m and 17.3 m below ground surface. In addition, dynamic cone penetration tests (DCPTs) were advanced 10 m west and 10 m east of Boreholes C17-8 and C17-9 along the existing embankment toe of slope to depths between 0.2 m and 2.7 m below ground surface for delineation of refusal/bedrock surface.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and a standpipe piezometer was installed in Borehole 17-4 to permit monitoring of the groundwater level. The piezometer consists of a 38 mm diameter polyvinyl chloride (PVC) pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled with bentonite pellets and/or bentonite grout to ground surface. The piezometer installation details and water level readings are indicated on the borehole records contained in Appendix A. All other boreholes were backfilled upon completion in accordance with Ontario Regulation 903 (Wells*,* as amended).

The fieldwork was supervised by a member of our engineering and technical staff, who observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and took custody of the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury Geotechnical Laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, grain size distribution and Atterberg limits) was carried out on selected samples. In addition two one-dimensional consolidation (oedometer) testes were carried out on selected soil samples. Unconfined compression strength (UCS) tests were carried out on selected bedrock core samples. The results of the laboratory testing on samples from the boreholes are presented on the Record of Borehole and Drillhole sheets and are included in Appendix D.

The approximate locations of the boreholes were determined based on preliminary drawings provided to Golder during the planning phase by MH as foundation element locations were not known at that time and confirmed with AECOM/MTO prior to drilling. The as-drilled locations and elevations of the boreholes were surveyed using a Trimble Geo7 GPS survey unit. A summary of the borehole locations (northing and easting coordinates given relative to NAD83 MTM Zone 12, as well as latitude and longitude) and Geodetic elevations are provided on the borehole records and together with the drilling depths are summarized below.

Borehole	Location (MTM NAD 83, Zone12)		Location (WGS84)		Ground Surface Elevation (m)	Borehole/ DCPT Depth (m)
	Northing	Easting	Latitude	Longitude		
C17-1	5149846.8	318881.4	46.488138	-80.816580	260.2	3.0*
C17-2	5149856.2	318871.2	46.488223	-80.816713	254.0	3.0*
C17-3	5149852.7	318815.9	46.488192	-80.817433	253.5	11.7*
C17-4	5149860.2	318800.5	46.488260	-80.817633	253.5	11.9*
C17-5	5149839.0	318787.1	46.488070	-80.817809	262.6	17.3
C17-6	5149865.0	318776.1	46.488304	-80.817951	253.1	10.7
C17-7	5149841.5	318720.8	46.488094	-80.818672	261.7	9.2*
C17-8	5149860.2	318720.0	46.488262	-80.818682	255.9	0.3
C17-8D1	5149860.2	318710.0	46.488262	-80.818812	256.2	0.2
C17-8D2	5149860.2	318730.0	46.488262	-80.818552	255.0	1.0
C17-9	5149857.5	318671.9	46.488239	-80.819309	257.1	2.3
C17-9D1	5149857.5	318661.9	46.488239	-80.819439	257.1	2.7
C17-9D2	5149857.5	318681.9	46.488238	-80.819178	257.1	2.1

*Includes between 1.0 and 3.1 m of bedrock core length.

The relevant borehole logs from this investigation used to supplement the current investigation are provided in Appendix C.

4.0 SUBSURFACE CONDITIONS

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are provided on the Record of Borehole sheets contained in Appendix C. The results of geotechnical laboratory testing are contained in Appendix D. The results of the in situ tests (i.e., SPT 'N'-values and field vanes) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profiles on Drawings 1 and 2 are inferred 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.

4.1 Regional Geology

The site is located within a glaciolacustrine plain, with low relief and a suspected high water table[1]. The published information indicated the site borders on areas characterized by bedrock knobs generally covered by a thin veneer (1 to 3 m in thickness) of bouldery sandy glacial till, with low relief and undulating topography[1].

4.2 Subsoil Conditions

In general, the subsoil conditions encountered at the borehole locations consist of embankment fill or a surface layer of topsoil, underlain by a native deposit of clayey silt to clay, which is underlain by a granular deposit ranging in composition from silt to sand in turn underlain by a till deposit comprised of gravelly silty sand to sand and gravel overlying bedrock. Generally, the stratigraphy noted in the current investigation is consistent with the previous investigations. A more detailed description of the soil deposits and groundwater conditions encountered in the boreholes is provided below.

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[1] Garnet, J.F., 1980. Sudbury Area (NTS 41i/SE) District of Nipissing, Parry Sound and Sudbury; Ontario Geologic Society, Northern Ontario Engineering Geology Terrain Study 100.

Deposit/Layer Description	Boreholes	Deposit Thickness (m)	Deposit Surface Elevation (m)	N Values (blows) Field Vane Results (kPa) Consistency or Relative Density	Laboratory Testing
Asphalt	C17-5, C17-7	0.10	262.6, 261.7	n/a	n/a
Concrete[1]	C17-5	0.25	262.5	n/a	n/a
Sandy/Silty Topsoil and/or Peat	C17-3, C17-4, C17-6, C17-8, C17-9	0.1 – 0.2	257-1 – 253.1	n/a	n/a
(FILL) Sand and Gravel, Sand, Silty Sand, Sandy Silt[2], trace organics, brown; moist to wet,	C17-3 to C17-7 and C17-9	0.6 – 4.0 0.9 (lower fill in C17-5)	262.2 – 253.0	N = 6 - 57; 67/0.16 n/a **Loose to very dense**	w = 7% – 10% 3 – M (Fig. D1) 1 – MH (Fig. D1)
(FILL) Clayey Silt, some to with sand, some gravel; brown; moist to wet	C17-4, C17-5, C17-7	0.9 – 4.9	258.3 – 252.8	N = 4 - 57 n/a **Firm to hard**	w = 18% – 20% 2 - MH (Fig. D2) 2 – AL (Fig. D3) w_l = 26% – 30% w_p= 17% I_p = 9% – 13%
Clayey Silt to Clay, trace sand, trace gravel, varved; brown to grey; wet	C17-3 to C17-6	4.7 – 7.8	252.4 – 251.5	N = WH – 14 s_u = 24 – >100 S = 4 - 9 **Soft to very stiff**	w = 28% – 47% 9 - MH (Fig. D4) 13 -AL (Fig. D5) w_l = 26% – 54% w_p= 19% – 24% I_p = 6% – 31% 2 – Oedometer (Fig. B6 and B7)
Sandy Silt, Silt and Sand, Silt, Sand[2], trace gravel, trace clay; grey; wet	C17-3, C17-4, C17-6, C17-9	0.4 – 2.6	256.0 – 243.8	N = 1 – 36 n/a **Very loose to dense**	w = 14% – 31% 4 - MH (Fig. D8) 1 – AL (Fig. DB9) including 1-AL(N.P.) w_l = 18% w_p= 15% I_p = 3%
TILL - Sand and Gravel to Gravelly Silty Sand[2], trace clay; dark brown to grey; wet	C17-5, C17-7 to C17-9	0.2 - 2.6	256.1 – 246.4	N = 16 – 74; 10/0.2 n/a **Compact to very dense**	w = 6% – 18% 3 – MH (Fig. D10)

Where:
N = SPT ‘N’-value; number of blows for 0.3 m of penetration
s_u = Undrained Shear Strength from in situ field ‘N’-vane (kPa)
S = calculated sensitivity
w = Natural Moisture Content (%)
MH = Combined Sieve and Hydrometer analysis
M = Sieve analysis for particle size
AL = Atterberg Limits Test
w_p = Plastic Limit (%)
w_l = Liquid Limit (%)
I_p = Plasticity Index (%)
NP = Non-Plastic test result

Notes:
1. Concrete encountered in Borehole C17-5 is likely part of the concrete approach slab.
2. Cobbles were encountered in Boreholes C17-3, C17-5 within the native sandy silt to silt and in the sand and gravel to gravelly silty sand deposits up 110 mm in diameter. Cobbles were also encountered within the embankment fill and within the gravelly silty sand Till deposit in Borehole C17-7.

Laboratory consolidation (oedometer) tests were carried out on two Shelby Tube samples of the clayey silt to clay deposit, obtained from Shelby tube samples in Boreholes C17-3 and in a separate borehole drilled adjacent to Borehole C17-4. The preconsolidation stress was estimated from the void ratio versus logarithmic pressure plot and from the total work versus pressure plot. A bulk unit weight of 18.0 kN/m^3 and 18.7 kN/m^3 and a specific gravity of 2.77 were measured on the consolidation test samples. The detailed results of the oedometer tests are shown on Figures D6 and D7 in Appendix D, and the test results are summarized below:

Borehole/ Sample No.	Sample Depth / Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	σ_p' - σ_{vo}' (kPa)	OCR	e_o	C_c	C_r	c_v* (cm^2/s)
C17-3/ Sample 7	5.5 m/ 248.0 m	65	140	75	2.2	0.96	0.19	0.02	6.3×10^{-3}
Adjacent to C17-4/ Sample 1	5.5 m/ 248.0 m	70	190	120	2.7	1.09	0.16	0.03	2.7×10^{-3}

*For the consolidation stress range 130 kPa to 250 kPa

where:	σ_{vo}'	is the effective overburden stress in kPa
	σ_p'	is the preconsolidation stress in kPa
	OCR	is the overconsolidation ratio
	e_o	is the initial void ratio
	C_c	is the compression index
	C_r	is the recompression index
	c_v	is the coefficient of consolidation in cm^2/s

4.3 Bedrock/Refusal

On the northeast end of the existing bridge, an exposed bedrock knob is present dipping westerly towards the rail right-of-way. Another bedrock knob is present approximately 20 m north of Borehole 17-8 located beyond the toe of the embankment slope.

Based on the results of the DCPTs, previous geotechnical investigations at the site, and published geological information, the DCPTs are considered to have achieved “refusal” on the inferred bedrock surface. Further, boreholes where bedrock was not cored were terminated on “refusal” conditions on the inferred bedrock surface as indicated by auger refusal, refusal to further casing advancement and/or split-spoon refusal. Bedrock was cored in Boreholes C17-1 to C17-4 and C17-7 and the depth/elevation of the actual/inferred bedrock surface is presented below.

Borehole No.	Depth to Bedrock/Refusal (below ground surface at borehole location) (m)	Bedrock Surface/DCPT Refusal Elevation (m)	Refusal Condition (m)
C17-1	Ground Surface	260.2	3.0 m bedrock core length
C17-2	Ground Surface	254.0	3.0 m bedrock core length
C17-3	8.6	244.9	3.1 m bedrock core length
C17-4	8.9	244.6	3.0 m bedrock core length
C17-5	17.3	245.3	Casing and split-spoon refusal
C17-6	10.7	242.4	Auger and split-spoon refusal
C17-7	8.2	253.5	1.0 m bedrock core length
C17-8	0.3	255.6	Auger and split-spoon refusal
C17-8D1	0.2	256.0	Hammer bouncing
C17-8D2	1.0	254.0	Hammer bouncing
C17-9	2.3	254.8	Auger and split-spoon refusal
C17-9D1	2.7	254.4	Hammer bouncing
C17-9D2	2.1	255.0	Hammer bouncing

The retrieved bedrock core from Boreholes C17-1 to C17-3 and C17-7 is described as slightly weathered to fresh, very fine grained, grey arkosic greywacke. In Borehole C17-4, the bedrock is described as very fine grained, grey to pink meta quartzite. More detailed descriptions of the bedrock cores are presented on the Record of Drillhole sheets in Appendix C. Photographs of the bedrock core samples are shown on Figure D11 in Appendix D. The bedrock properties, as encountered in the boreholes, are summarized below.

Borehole No.	Total Core Recovery (TCR)	Rock Quality Designation (RQD)	Quality Classification (Table 3.10 of CFEM 2006[2])	UCS (MPa)	Strength Classification (Table 3.5 of CFEM 2006[3])
C17-1	75% - 100%	18% - 67%	Very Poor to Fair	91	(R4) Strong
C17-2	100%	22% - 69%	Very Poor to Fair	142	(R5) Very Strong
C17-3	93% - 100%	45% - 81%	Poor to Good	87	(R4) Strong
C17-4	100%	81% - 100%	Good to Excellent	151	(R5) Very Strong
C17-7	100%	0%	Very Poor	-	-

4.4 Groundwater Conditions

Unstabilized groundwater levels measured in the open boreholes upon completion of drilling are summarized below. It should be noted that the introduction of drilling water to advance NW casing in the boreholes may impact the measured groundwater levels. Water levels may vary depending on the time of year and precipitation events.

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[2] Canadian Geological Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

Borehole	Ground Surface Elevation (m)	Depth to Groundwater (mbgs)	Groundwater Elevation (m)
C17-3	253.5	2.1	251.4
C17-4 (Piezometer)	253.5	1.6 (April 27, 2017 and July 4, 2017)	251.9
C17-5	262.6	6.9	255.7
C17-6	253.1	5.9	247.2
C17-7	261.7	6.6	255.1
C17-9	257.1	1.8	255.3

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Tibor Berecz, and the technical aspects were reviewed by Ms. Sarah E.M. Poot, P.Eng. a senior geotechnical engineer and Associate of Golder. Mr. Jorge M.A. Costa, P.Eng., a Senior Consultant with Golder and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.

Report Signature Page

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PART B

FOUNDATION DESIGN REPORT
HIGHWAY 17 CONISTON CPR OVERHEAD
TEMPORARY DETOUR, SITE NO. 46-123
SUDBURY DISTRICT, TOWNSHIP OF DRYDEN
AGREEMENT No. 5015-E-0045 – WORK ORDER 1

6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the temporary modular bridge (TMB) and widening of the detour approach embankments associated with the overall rehabilitation of the Canadian Pacific Railway (CPR) Overhead structure located on Highway 17 near Coniston, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigations. The discussion and recommendations presented are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the foundations for the replacement structure. The foundation design report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the contract or Design-Build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the MTO to provide recommendations on the foundation aspects for the design of the TMB detour structure and widening of the adjacent high fill approach embankments to the west of the existing structure in support of the overall rehabilitation of the CPR overhead structure near Coniston, Ontario. Morrison Hershfield (MH) are conducting the highway and structural design engineering services under separate contract with MTO.

Based on a preliminary General Arrangement Drawing provided by MH on May 26, 2017, we understand that construction of a temporary detour embankments with a grade raise of approximately 4 m to the north of the existing embankments and to a height of up to 5 m is required to support the rehabilitation of the existing structure without disrupting traffic flow along the Highway 17 corridor. We understand that the proposed TMB will consist of a three-span, non-skewed Acrow Double-wide deck with spans of 15.2 m, 82.4 m and 18.3 m from west to east, respectively, to be supported on concrete footings and/or deep foundation units.

The existing bridge is shown in plan on Drawing 1 and consists of a three-span, cast-in-place concrete structure founded on a combination of shallow and deep foundations and skewed relative to the alignment of Highway 17. The existing west approach embankment is approximately 10 m high, and based on the boreholes advanced for the current investigation and the previous investigations the west approach embankment is constructed of cohesive fill underlying granular fills; and the west abutment is supported on deep foundations consisting of steel H-piles driven to bedrock. The existing piers are also supported on piles driven to bedrock. The existing east abutment is supported on a spread footing founded on a rock fill platform over the bedrock surface which outcrops nearby and dips steeply westerly towards the east pier. The east approach embankment is reportedly constructed of rock fill with a sand and gravel cover layer.

At this site, the configuration of the proposed TMB will present constructability challenges as the west abutment is located across the mid-slope of the existing approach embankment granular/cohesive fill; and the east pier is located across the mid-slope of the existing rock fill embankment/rock outcrop adjacent to the existing east abutment. Further, the bedrock is dipping from east to west towards the existing east pier and rail line.

Additional challenges affecting abutment/pier and approach embankment design are presented with the presence of the underlying soft to very stiff, varved, clayey silt to clay subsoils which have experienced and will continue to undergo consolidation settlement as a result of the past and new loading.

We understand that together with rehabilitation of the existing structure, RSS walls are also being incorporated into the final approach embankment construction at the toe of the front slopes adjacent to the CPR tracks as outlined in GEOCRES 41J-342. This will result in the addition of fill to the existing fill mass of the front slopes.

6.2 Temporary Modular Bridge Foundation Options

The proposed TMB will be separated from the existing bridge by an approximately 4 m shift of the TMB to the north. Given the existing site topography (i.e. existing embankment side slopes) and the proposed detour abutment/pier locations (mid-slope of the existing embankments) it was not possible to drill boreholes at/near the locations of the new foundation elements. Further the exact location of the TMB piers and abutments were unknown at the time of the geotechnical investigation. As such subsurface information from the following boreholes are being utilized for the development of design parameters and recommendation to provide information for the temporary detour structure:

TMB Foundation Element	Borehole	Borehole Location
West Abutment	C17-4	Toe of Slope – Existing Embankment
	C17-5	West Abutment – Existing Embankment
West Pier	C17-3	Toe of Slope – Existing Embankment
East Pier	C17-2	Toe of Slope – Existing Embankment
	BH3 (GEOCRES 41J-342)	Toe of Slope – Existing Embankment – 2016 Drilling
	3 (GEOCRES 41I-140)	Existing Embankment – 1977 Drilling
	4 (GEOCRES 41I-140)	Existing Embankment – 1977 Drilling
East Abutment	C17-1	Proposed TMB East Abutment

Generally the subsurface conditions vary from west to east comprising: bedrock at shallow depth along the west of the west approach embankment; 8.5 m to 10.5 m thick deposits of fill, cohesive soil/non-cohesive soil underlain by bedrock at the west abutment, along the TMB crossing; and a bedrock outcrop at the east abutment and beyond. At the proposed TMB east abutment, the bedrock dips steeply to the west towards the rail tracks and east pier.

Based on the above, it is likely that a combination of shallow (spread footings on granular pad and/or bedrock) and deep (pile) foundations will be required to support the temporary modular bridge.

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

- **Shallow Footings:** Shallow foundations are considered feasible at the proposed TMB east abutment either founded on a granular pad or on mass concrete over bedrock or directly on bedrock. Spread footings, although technically feasible, are not considered practical for the west abutment, west pier and east pier as temporary protection systems would be required to support the highway in order to excavate and install large engineered fill pads to achieve the resistances required to support the relatively large TMB structure. Further, the associated embankment settlement of the cohesive deposit due to loading from the adjacent embankment fills, presence of underlying compressible native soils and would result in poor performance of the structure.

- **Driven Steel H-piles:** Driven steel H-piles end-bearing piles on bedrock are feasible for support of the west abutment, west pier and the east pier. Special measures such as pre-drilling into the bedrock, need to be taken at the east pier to account for the sloping bedrock surface such as pre-drilling into the bedrock and be able to seat the piles. Additionally, the composition of the embankment in the northeast corner of the existing structure reportedly consists of rock fill and this may require pre-drilling to advance the piles through this material. Consideration of the batter of the existing piles will need to be addressed.

- **Micropiles:** Micropiles socketed into the bedrock could be considered for support of the abutments. Micropiles have the advantage of requiring lighter weight equipment for installation, which may be advantageous at this site, however difficulty may also be encountered advancing through the rock fill at the east pier. Installing micropiles through rock fill requires a sacrificial casing to be used such that the grout will not migrate into the rock fill. The cost of micropiles is typically higher than conventional H-piles.

- **Drilled steel casings (small diameter):** Drilled steel casings, which are typically between 305 mm and 750 mm in diameter, have the advantage over driven piles or micropiles or being able to penetrate strata where frequent obstructions are present in overburden soil deposits, and have an advantage over larger diameter drilled shafts where drilling a bedrock socket is required in strong and/or sloping bedrock. The cost premium for this type of foundation may not be warranted given that it is a temporary structure.

- **Drilled shafts/caissons (large diameter):** Drilled shafts socketed into the bedrock are also considered to be feasible for a deep foundation option at this site. However, caissons are not commonly constructed in Northern Ontario due to constructability issues associated with large-diameter drill holes through wet subgrade soils, obstructions such as rock fill, and challenges associated with seating/sealing large diameter elements at the interface with the sloping bedrock, and the costs associated with creating a socket in the strong to very strong bedrock.

The following sections provide recommendations for shallow and deep foundation options to support the proposed TMB structure. From a foundations perspective, driven steel H-piles are considered more practical and economical in terms of initial construction costs for the west abutment, west pier and east pier, although pre-drilling at the east pier may be required. Spread footings are preferred at the east abutment.

6.2.1 Consequence and Site Understanding Classification

It is understood that the TMB is to be designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC).

As the proposed TMB will carry traffic along Highway 17 and will carry large volumes of traffic with the potential to impact alternative transportation corridors, a “typical consequence level” is considered appropriate as outlined in Section 6.5 of the *Canadian Highway Bridge Design Code* (CHBDC 2014) and its *Commentary*. Further, given the scope of work of the foundation field investigation and laboratory testing program as outlined in Sections 3.0 and 4.0, 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, respectively, of the CHBDC (2014) have been used for design as indicated in Section 6.3 to 6.6.

6.2.2 Seismic Site Classification

For seismic design purposes, a site classification for seismic site response of E (for soft soil) should be used, based on Table 4.1 in Section 4.4.3 of the 2014 CHBDC (2014).

6.3 Deep Foundations

The west abutment, west pier and east pier of the TMB may be founded on driven steel H-piles end bearing on the bedrock. The following pile tip elevations can be used for design purposes, based on the results of the foundation investigation:

TMB - Foundation Element	Borehole	Estimated Design Driven Pile Tip Elevation / Top of Bedrock (m)
West Abutment	C17-4 and C17-5 Current Investigation	244.5
West Pier	C17-3 – Current Investigation	244.5
East Pier	C17-2 – Current Investigation BH3 (GEOCRES 41J-342); 3 (GEOCRES 41I-140); and 4 (GEOCRES 41I-140)	254.0 – 1.5 = 252.5 on north side to 205.5 – 1.5 = 249.0 on south side

At the east pier, the piles should be installed in pre-drilled holes advanced through the rock fill embankment and into the bedrock for a depth of 1.5 m due to the steeply sloping bedrock surface across the pier as noted in the above table. Additionally, there should be a provision made in the Contract for dealing with varying pile lengths due to the variability of the depth to bedrock in particular at the east pier.

It should be noted that obstructions (i.e. cobbles/boulders) were encountered within the native soils in Boreholes C17-3, C17-5 and C17-7 and within the embankment fill in Borehole C17-7 and should be considered during pile driving. Further based on the previous available information, we understand that the existing east abutment embankment is constructed from blast rock fill at which a pile foundation is not considered suitable; refer to Section 6.4 for foundation design recommendations.

6.3.1 Geotechnical Axial Resistances

A factored ultimate geotechnical resistance of 2,000 kN per pile may be used for the design of steel HP 310x110 piles driven to the surface of the arkosic greywacke or meta quartzite bedrock or socketed 1.5 m into bedrock. The factored serviceability geotechnical resistance (for 25 mm of settlement) will be greater than the factored ultimate geotechnical resistance. As the bedrock is considered to be an unyielding material, ULS conditions will govern for this foundation type. Since cobbles were encountered within the fill/native soils at some borehole locations and could be encountered between boreholes, which could impede pile driving and cause the piles to "hang up" or be deflected from their intended vertical alignment, consideration should be given to using a heavier H-pile section, such as HP310x132, to reduce the potential for damage to the piles during driving to the required tip elevation. The risk of hang up or deflection on cobbles/boulders is low for this site.

6.3.1.1 Set Criteria and Pile Driving Note

All pile installation/driving should be in accordance with OPSS 903 (Deep Foundations). The piles should be fitted with driving shoes or flange plates (reinforced tips) in accordance with OPSD 3000.100 (Steel H-Driving Shoe) to minimize damage to the pile tip during driving.

For piles driven to bedrock, set criteria are highly dependent on pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to also avoid overdriving and possibly damaging the piles. Based on our experience, consideration should be given to the following preliminary criteria for piles driven to bedrock:

- The piles should be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules, but not exceeding 60 kilojoules.
- On reaching the required set, the hammer energy should be reduced to 75 per cent and the pile should be re-driven in 2 sets of 10 blows to improve the process of seating the pile on the bedrock.
- A final set of no less than 10 blows per 12 mm of penetration should be obtained at the maximum hammer energy. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.

A NSSP, which outlines the above criteria for seating the piles on bedrock, should be included in the Contract; an example is included in Appendix C.

The pile driving note that should be added to the drawings for this project is Note 5 in Clause 3.3.3 of the MTO's Structural Manual (MTO 2008), as follows:

- "Piles to be driven to bedrock."

The piles should be tapped to confirm they are seated on the bedrock. For the east pier foundation, the standard pile driving notes do not apply and the piles are to be installed/founded on bedrock at the bottom of the pre-drilled holes into bedrock.

6.3.1.2 Resistance to Lateral Loads

The design of steel pile foundations subjected to lateral loads should take into account such factors as the batter of the pile (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. Lateral loading could be resisted fully or partially by the use of battered piles.

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

For cohesionless soils:

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

Where: n_h is the constant of horizontal subgrade reaction (kPa/m), 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: s_u is the undrained shear strength of the soil (kPa); and,

B is the pile diameter/width (m).

The following values of n_h and s_u (Terzaghi, 1995) and s_u may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection).

Foundation Element (Relevant Boreholes)	**Soil Unit**	**Elevation (m)**	n_h **(kPa/m)**	Su (kPa)
West Abutment (C17-4)	New/Existing Fill	Assumed u/s of pile cap (262 m) to 251.9 m	18,000	-
	Varved Clayey Silt to Clay	251.9 to 247.3	-	75 above Elev. 248 m 30 below Elev. 248 m
	Silt and Sand	247.3 to Bedrock	4,400	-
West Pier (C17-3)	Varved Clayey Silt to Clay	Assumed u/s of pile cap (251.5 m) to 246.8 m		75 above Elev. 248 m 30 below Elev. 248 m
	Sandy Silt	246.8 to Bedrock	4,400	-
East Pier (4 and C17-2)	Uniformly graded loose sand (in the predrilled hole)	Varies across pier. Bedrock is dipping from east to west. Assumed u/s of pile cap (258 m) to bedrock ranging from 254.0 m to 250.5 m	1,300	-

Note: U/s of pile caps assumed from cross sections provided. U/s of pile caps on preliminary General Arrangement drawing provided by MH are above the proposed detour ground surface elevation.

For a single vertical HP310X110 (or HP310X132) pile advanced to the design tip elevations provided in Section 6.3, the estimated factored lateral resistance at ULS and the lateral reaction at SLS (for 10 mm of horizontal deflection at the pile cap) are presented below. These values are based on analysis carried out using the commercially available program LPile Plus 2016 (Version 2016.9.09), produced by Ensoft Inc.

Foundation Element	Lateral Resistance/Reaction (kN)	
	Factored ULS	Factored SLS (10 mm of deflection)
West Abutment	300	60
West Pier	70	60
East Pier (1.5 m socket into bedrock)	See Note	See Note

Note: For the steel H-piles socketed into bedrock at the east pier, the lateral resistance will be developed primarily from the fixity (in concrete) within the drilled sockets. In this case, the structural resistance of the steel H-pile will govern the ultimate lateral resistance.

The lateral resistances given above are based on an assumed free-head condition of 1,400 kN unfactored axial load applied at the top of the pile for HP310X110 and HP310X132 piles. The lateral resistance should be reviewed if greater vertical loads are anticipated.

Given the preliminary nature of the general arrangement drawing provide, P-Y curves can be completed if required, once the pile cap elevations for the TMB abutment/ pier elevations are known.

6.3.2 Potential for Conflict of Deep Foundation Systems

Given the proximity of the existing structure to the proposed TMB structure, the structural engineer shall exercise caution to ensure that there are no conflicts with the proposed deep foundations for the TMB with the existing deep foundations (steel H-piles driven to bedrock for west abutment and piers) of the existing structure. We understand that battered piles were installed on the existing structure (unknown direction) and that battered piles may be utilized for the TMB structure.

It is recommended that the proposed deep foundations for the TMB structure be positioned (i.e. remain isolated) a minimum of 10 pile diameters away from any existing piles. Further, it is recommended that the TMB foundation elements be positioned as far as practical from the existing foundations to eliminate the potential for conflict with/disturbance of the existing foundations.

6.3.3 Downdrag Loads

Based on the previously completed geotechnical report for this site (Geocres No. 41I-140), the existing foundations of the overhead structure consist of HP 12x102 (equivalent HP 310x152) Steel H-Piles driven to bedrock for the west abutment, west pier and east pier and a shallow foundation on a rock fill pad over bedrock for the east abutment. The bridge piers are supported by a pile cap which connects the steel H-Piles.

The placement of new embankment fill for the proposed detour widening will result in settlement of the underlying approximately 5 m to 8 m thick silty clay deposit and cause downdrag loads on the new and existing piles at the west abutment, west pier and east pier. An estimated unfactored downdrag load acting on a HP 310 x 110 pile of 350 kN per pile may be used for design at the piers and 600 kN per pile may be used for design at the west abutment. A preliminary analysis of the dragloads for the existing HP 12x102 piles constituting the foundation element of the existing piers and abutments was completed based on the "alpha" method (where α = adhesion coefficient) interpolated from Figure 18.1 of the Canadian Foundation Engineering Manual (CFEM), 4th Edition, 2006, and results in an estimated unfactored downdrag load of 350 kN and 600 kN for the piers and west abutment, respectively. 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 2014 CHBDC and its Commentary for factored ultimate and serviceability conditions.

It should also be noted that downdrag loading from the proposed RSS wall fill (GEOCRES 41J-342) may also impact the existing piles and this should be considered in the evaluation of downdrag. The structural engineer should verify the capacity of the piles are not exceed by the additional embankment fill as part of the TMB detour and proposed RSS Walls.

6.3.4 Frost Protection

Should the temporary detour be subject to winter conditions (depending on construction schedule), then the pile caps should be provided with a minimum of 2.0 m of soil cover for frost protection as per OPSD 3090.100 (Foundation, Frost Penetration Depths for Northern Ontario) or a combination of soil cover and rigid insulation. For polystyrene insulation, the MTO has adopted an equivalency of 25 mm of insulation for every 0.3 m reduction in soil cover.

6.4 Shallow Foundation Recommendations

6.4.1 Geotechnical Axial Resistances

The proposed TMB east abutment could be supported on a strip footing founded on a granular pad/mass concrete overlying bedrock or directly on the bedrock surface. The bedrock surface at the east abutment (Borehole C17-1) is at Elevation 260.2 m, below the current founding Elevation 263 m shown on the GA drawing. It is recommended that the footing level be raised to the bedrock surface to avoid excavation or, if a lower founding elevation is required by the designer, that bedrock excavation and mass concrete/granular pad be used to level the area. However, while the abutment is located in an area at the crest of the bedrock outcrop, the bedrock surface may also vary across the foundation element.

For strip footings, a factored ultimate geotechnical resistance at ULS of 800 kPa may be used for design for a footing placed directly on the bedrock. The factored serviceability geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored geotechnical axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern. Mass concrete could be used to level the bedrock surface, an example NSSP outlining the requirements has been included in Appendix C.

The bedrock surface should be inspected following sub-excavation to ensure that the rock mass integrity was preserved during excavation and that the bedrock surface is properly cleaned, scaled and loosened debris removed prior to placing the concrete for footings in accordance with OPSS 902 (Excavating and Backfilling Structures).

Alternatively, footings can be constructed on a granular pad consisting of OPSS.PROV 1010 Granular A compacted in nominal lifts of 200 mm or less, compacted to 100 % Maximum Dry Density in accordance with OPSS.PROV 501 (Compacting). For footings constructed on a properly prepared granular pad directly overlying bedrock, the factored ultimate geotechnical resistance at ULS and factored serviceability geotechnical resistance at SLS for 25 mm of settlement may be taken as 400 kPa.

6.4.2 Resistance to Lateral Loads

The resistance to lateral forces/sliding resistance between mass concrete and the bedrock at the east abutment should be calculated in accordance with Section 6.10.5 of the CHBDC (2014) applying the appropriate consequence and degree of site understanding factors as noted in Section 6.2.1. A coefficient of friction, tan φ’, of 0.70 may be used for the interface between the concrete and bedrock or 0.55 for concrete footing on properly prepared granular pad.

Dowels should be incorporated into the design for footings constructed directly on the sloping bedrock at this site or if additional horizontal resistance is required to resist sliding. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. Where the rock mass is stronger than the concrete, the design of the dowels into the rock may be handled in the same way as the dowel embedment into the concrete for uniaxial compressive strength of the grout is similar to that of the concrete. The dowels should have a minimum embankment length within the strong bedrock of 1 m, and the structural strength of the grout should not be exceeded. An example NSSP for dowels (anchors) into bedrock is provided in Appendix E, if required.

6.4.3 Frost Protection

The east abutment footing founded directly on the bedrock or mass concrete over bedrock does not required soil cover for frost protection. Footings founded on a granular pad require a minimum frost cover of 2.0 m.

6.5 Embankment Stability and Settlement

The existing west approach embankment in the vicinity of the west abutment is approximately 10 m high with current side slopes of approximately 1.75 to 1.5 Horizontal to 1 Vertical (1.75 - 1.5H:1V). Based on cross-sections provided by the designer, a temporary detour widening with a grade equivalent to approximately 5 m of fill placed on the existing embankment side slope will result in an overall embankment height of approximately 8 m. The global stability analysis has been completed for a new detour embankment either constructed of granular fill with side slopes inclined at 2H:1V and constructed of rock fill with side slopes inclined at 1.25H:1V.

For the purposes of settlement and stability analyses, the critical section (i.e. largest fill height and thickest cohesive deposit) is located adjacent to the existing west abutment. At the east approach, bedrock knobs are visible and the embankments are relatively low (less than about 2 m) and as such, stability and settlement is not a concern at the east approach.

6.5.1 Stability

Slope stability analyses have been carried out for the proposed embankment configuration using the commercially available program SlopeW by (Version 7.23), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. The Factor of Safety (FoS) is defined as the ratio of forces tending to resist failure to the driving forces tending to cause the failure. For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, ψ, and the geotechnical resistance factor ϕ_{gu} (i.e. $FoS = 1/(\psi * \phi_{gu})$). Accordingly, a target minimum FoS of 1.3 have been used for design of the temporary embankment side slopes, and FoS of 1.5 for the design of the final embankment configuration as per Table 6.2 of CHBDC (2014) for both the total stress (short term undrained) and effective stress (long term drained), as applicable. The static global stability analyses assume that all existing topsoil and organics are completely removed prior to constructing the embankments.

6.5.2 Parameter Selection

The overburden encountered in the various areas is generally composed of embankment fill underlain by interlayered deposits of either granular soils (sand, silty sand, sandy silt, silt and sand, silt) or a combination of cohesive deposits (clayey silt, silty clay and/or clay). For granular soils, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle and effective cohesion) for the organics and granular soils were estimated from empirical correlations using the results of in situ SPTs, in conjunction with engineering judgement based on experience in similar soil conditions.

The effective stress parameters employed the cohesive deposit in the analyses assuming drained conditions were estimated from empirical correlations using the results of laboratory testing in conjunction with engineering judgement based on experience in similar soil conditions.

For cohesive deposits, total stress parameters were employed in the analyses assuming undrained conditions. The total stress parameters (i.e., average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of in situ field vane shear tests, inferred from the laboratory consolidation tests results, and estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content), where appropriate. For the consolidation tests, the following correlation proposed by Mesri (1975) was employed to estimate the undrained shear strength:

$$s_u = 0.22\sigma'_p$$

where: s_u = average mobilized undrained shear strength (kPa)
σ'_p = preconsolidation pressure (kPa)

Where appropriate, Bjerrum's correction factor for plasticity was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests as follows:

$$s_{u(mob)} = \mu s_{u(FV)} \qquad \text{(after Bjerrum, 1973)}$$

where: $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
$s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
μ = Bjerrum's correction factor based on Plasticity Index

Where varved clay was encountered, an additional reduction factor of 25 per cent was employed to account for the angle of minimum shearing resistance (Milligan and Lo, 1967).

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed north approach area.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (Degrees)	Undrained Shear Strength (kPa)
New Embankment Fill - Granular	21	35	-
Rock Fill	19	40	-
Existing Embankment Fill – Granular	21	33	-
Existing Embankment Fill - Cohesive	19	29	-
Varved Clayey Silt to Clay	18.7	29[1]	75 to 30 (Toe of Slope) 75 (Under Existing Embankment)
Sandy Silt to Silt and Sand	19	28	-
Sand and Gravel	20	33	-

1. Effective stress (i.e. drained condition).
2. Total stress (i.e. undrained condition).

6.5.3 Results of Analysis

The results of the static global stability analyses indicate that a Factor of Safety of 1.53 is achieved for the existing west approach embankment side slope in the drained condition as shown on Figure 1.

The results of the static global stability analyses indicate that Factors of Safety of 1.31 and 1.82 are achieved for the temporary detour west approach granular embankment slopes for both the undrained and drained condition, respectively, as shown on Figures 2 and 3. Factors of Safety greater than 1.3 is achieved for a temporary rock fill embankment in both the drained and undrained condition. As the factor of safety is greater than 1.3, stability mitigation measures are not required as part of the detour embankment construction.

The results of the static global stability analyses indicate that a Factor of Safety of 1.63 is achieved for the final granular or rock fill embankment configuration (at 2H:1V) in the drained condition, as shown on Figure 4 (for the granular embankment).

6.5.4 Settlement

To estimate the magnitude of the expected settlements of the detour embankments, analyses were carried out on a critical section at the west approach embankment using the commercially available program Settle^{3D} produced by Rocscience Inc. as well as hand calculations. The rate of settlement of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory. The model geometry and stratigraphy at the west approach embankment is shown on Figures 2 and 3, as used for the stability analyses. For the settlement analyses at the temporary detour, the critical section was assessed for the new embankment height and geometry and an average thickness of 6.3 m of the cohesive deposit based on Borehole C16-4, C17-5 and C17-6). The sources of settlement were considered to include:

- immediate settlement of the cohesionless and cohesive deposits;
- time-dependent consolidation of the cohesive deposits; and
- self-weight compression of the embankment fill materials.

If rock fill is used for the construction of the proposed detour embankments, there will be settlement due to compression of the rock fill itself under self-weight, in addition to the settlement of the underlying foundation soils. The magnitude of settlement of the rock fill depends on the following factors:

- type of rock/strength of particles;
- size and shape of rock particles;
- gradation of rock fill;
- total height/thickness of rock fill (stress level); and
- method of construction and sequence of placement (including lift thickness, compactive effort and state of packing).

The settlement of rock fill occurs as a result of re-arrangement of rock particles under load and wetting and as a result of localized crushing of rock particles at point contacts. The magnitude of both the short-term and long-term post-construction settlement of the rock fill is a function of the height of fill as well as the method of fill placement (i.e., compacted versus dumped rock fill) as outlined in "*MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates*" (MTO, September 2010).

Rock fill should be placed, whenever possible, in a controlled manner (i.e., not end-dumped) in accordance with OPSS.PROV 206 (Grading). Blading, dozing and 'chinking' the rock fill to form a dense, compact mass is required to minimize voids and bridging and, reduce settlements and should be used to construct rock fill embankments above the existing groundwater table. Where rock fill cannot be placed in a controlled manner (i.e., below the groundwater table), the post-construction settlement of the rock fill is expected to be greater.

Short-Term Rock Fill Settlement

The magnitude of short-term post-construction settlement associated with compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (September 2010), as follows:

Height of Rock Fill, H	Short-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 5 m	0.5% H	1.0% H
>5 m to 10 m	0.75% H	1.5% H
>10 m to 15 m	1.0% H	2.0% H

Approximately 90 per cent of the short-term settlement may be expected to occur within the first six (6) months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one (1) year following the completion of embankment construction to full height.

Long-Term Rock Fill Settlement

The magnitude of long-term post-construction settlement for compacted and end-dumped rock fill may be estimated in accordance with the MTO Foundations Guideline (September 2010), as follows:

Total Height of Rock Fill, H	Long-Term Rock Fill Settlement	
	Compacted Rock Fill	Dumped Rock Fill
Up to 15 m	0.1% H	0.2% H

The long-term rock fill settlement is expected to occur from one (1) year following the completion of construction over the life of the embankment.

Based on MTO's "Embankment Settlement Criteria for Design" dated March 2, 2010, the following post-construction settlement and differential settlement criteria are considered acceptable for settlements to occur within 20 years post-paving for the bridge approach embankments at this site.

Location	Distance from Transition Point (i.e., Abutment)	Total Post-Construction Settlement (mm)
Transition/Taper to Bridge Abutments	0 m to 20 m	25
	20 m to 50 m	50
	50 m to 75 m	75
Embankment Widening (non-freeway)	-	75

These criteria have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments.

The simplified stratigraphy together with the associated strengths and unit weights employed for the different soil types at the approach embankments are summarized below.

6.5.4.1 Parameter Selection

The settlement analyses have been completed using estimated elastic deformation moduli and consolidation indices as given below, based on correlations with the SPT "N"-values, field vanes, oedometer testing and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Type	γ (kN/m^3)	Settlement Parameters
Existing Embankment Fill - Granular	21	Es = 40 MPa
Existing Embankment Fill - Cohesive	19	Es = 32 MPa
Varved Clayey Silt to Clay	18.7	σ_p' = 140 kPa OCR = 2.2 e_o = 1.0 C_c = 0.2 C_r = 0.02 Eu = 21 MPa
Sandy Silt to Silt and Sand	19	Es = 11 MPa
Sand and Gravel	20	Es = 40 MPa

The coefficient of consolidation, c_v, required in the time-rate settlement analysis was estimated from the correlation with liquid limit (NAVFAC, 1986) assuming over consolidated clays. A c_v equal to 6.2 x 10^{-3} cm^2/s is considered appropriate for the normally consolidated range as also interpreted from the results of the laboratory consolidation test results in Section 4.2.

6.5.4.2 Results of Analysis

The proposed widening for the east approach embankment is to be constructed over visible bedrock knobs and settlement is estimated to be negligible.

At the detour west approach embankment constructed of granular material, the factored settlement along the centreline of the proposed detour embankment is estimated to be about 75 mm in the vicinity of the existing west abutment, transitioning to negligible settlements approximately 75 m west of the west abutment (near Borehole C17-8). Along the outside (northern) edge of the detour, the estimated factored settlement is about 130 mm. The north edge of the existing pavement will have an estimated factored settlement of about 35 mm diminishing to about 15 mm at the centreline of existing Highway 17. Therefore, in the transverse direction, there will be up to about 100 mm of settlement between existing north edge of pavement and north edge of the detour (i.e. across the detour). There will be between 0 mm of settlement between the south edge of existing pavement and 35 mm at the north edge of pavement. In the longitudinal direction along the detour, the up to 130 mm of settlement will occur along the approach, while the abutment, assumed to be supported on piles to bedrock, will not settle. A similar 35 mm of settlement is estimated to occur in the longitudinal direction at the existing west abutment.

At the detour west approach, if rock fill is utilized for construction of the detour embankment, the factored settlement along the centreline of the proposed detour embankment is estimated to be approximately 125 mm (comprised of 75 mm settlement of the native soils and 50 mm of rock fill settlement) in the vicinity of the west abutment, transitioning to negligible settlements approximately 75 m west of the detour west abutment (near Borehole C17-8). Along the outside northern edge of the detour the estimated factored settlement is about 190 mm (comprised of 130 mm of settlement of the native soils and 60 mm of rock fill settlement). The north edge of the existing pavement will undergo a similar estimated settlement of 35 mm as noted above, diminishing to about 15 mm at the centreline of the highway. There will be differential settlement occurring in the transverse direction between the existing north edge of the existing embankment pavement and north edge of the new embankment detour of approximately 155 mm. In summary, there will be a more pronounced effect of differential settlement if rock fill is utilized in the construction of the detour embankment.

Given that the post-construction settlement does not meet the MTO settlement criteria the vicinity of the temporary detour structure for both total and differential conditions, consideration could be given to allowing for additional quantities of temporary asphalt in the contract package to allow the contractor to “pad” the bridge approach should the deformation be noticeable (i.e. in a “bump”). In addition, assuming that the existing Highway 17 pavement structure will be replaced as part of the rehabilitation works, paving should be delayed until a minimum of 6 months has occurred after placing the detour embankment fill to allow settlement to occur and avoid differential post-construction settlement of the re-instated pavement.

Mitigation in advance of construction of the detour west approach embankment could be considered but is likely impractical for the estimated magnitudes of settlement and limited length of time the detour will be in place. As an example, preloading the embankment to allow for consolidation of the compressible cohesive soils such that the allowable settlements are less than 20 mm at the bridge approach prior to moving traffic to the detour would take about 4 to 6 months. If rock fill is utilized in construction of the detour embankment, a large portion of the settlement will occur within the first 6 months post-construction. Additionally, utilizing granular fill instead of rock fill will result in reduced settlement of the detour embankment fill itself. Alternatively, the use of lightweight fill, such as the incorporation of EPS into the embankment mass, could be considered to reduce the magnitude of settlement on both the detour and the existing west approach embankment, but is likely not economical for a temporary detour.

6.6 Lateral Earth Pressures

The lateral earth pressures acting on the TMB abutment walls and any associated wing walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

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

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular ‘A’ or Granular B Type II, should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and 3190.100 (Walls, Retaining and Abutment, Wall Drain).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northern Region Directive (2002) for backfill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (Walls, Abutment, Backfill, Rock).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 2.0 m behind the back of the wall (as shown on Figure C6.20(a) of the Commentary to the CHBDC (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or pile cap (as shown on Figure C6.20(b) of the Commentary to the CHBDC (2014).

The following guidelines and recommendations are provided regarding the lateral earth pressures for static loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- For restrained wall, the pressures are based on the proposed embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill or OPSS.PROV 1010 Select Subgrade Material (SSM):

Material	Earth Fill or SSM
Soil Unit Weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_P	3.0

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

Material	Granular 'A'	Granular 'B' Type II
Soil Unit Weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_P	3.7	3.7

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
  - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
  - Horizontal translation of 0.001 times the height of the wall; or,
  - A combination of both.

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

6.7 Construction Considerations

6.7.1 Subgrade Preparation and Embankment Construction

As noted in Section 6.1, the existing west approach embankment is constructed of granular material and cohesive fill and the existing east approach is constructed of rock fill pad over bedrock that dips steeply from east to west. For the proposed Highway 17 temporary detour, removal of the organic soils from below the footprint of the embankment is recommended prior to widening the existing embankment. Where new fill is to tie into existing fill along and beyond the approaches for the detour, the new fill should be "keyed-in" or benched into the existing fills, in accordance with OPSD 208.010 (Benching of Earth Slopes). Side slopes for granular fill should be no steeper than 2H:1V. Side slopes for rock fill embankments should be no steeper than 1.25H:1V.

Fill for detour embankment construction should consist of Granular 'B' Type I, II or III meeting the specifications of OPSS.PROV 1010 (Aggregates). The embankment fill for the detour should be placed and compacted in accordance with OPSS 501 (Compacting) and OPSS.PROV 206 (Grading).

Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

For embankments constructed using earth fill, the incorporation of 2 m wide benches (or successive berms) into the uniform side slope profile is required wherever the embankment will exceed a height of 8 m such that the uninterrupted slope does not exceed a height of 8 m as per OPSD 202.010 (Slope Flattening). As such, we recommend incorporating berms into the detour west approach embankment and final regrading of the embankment slope at approximately Elevation 254 m. The bench should tied into the existing embankment approximately 50 m to the west of the west abutment or where the embankment height is less than 8 m.

6.7.2 Control of Groundwater and Surface Water

Due to the assumed elevation of the west abutment and east pier within the embankments, groundwater control is not anticipated to be required. At the west pier, the pile cap should be set as high as possible to allow construction in-the-dry. However, depending on the final founding elevation and the groundwater level at the time of construction, unwatering may be required and can likely be handled using properly filtered sumps within the excavation.

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation but all surface water should be directed away from the excavations. Seepage from the granular fills should be expected, particularly after precipitation events. It is anticipated that minor surface water seepage and seepage from the granular fills can be controlled by using properly filtered sumps within the excavation.

An application under the Environment Activity Section Registry (EASR) of the Ontario Ministry of Environment and Climate Change (MOECC) is not likely to be required as the pumping volumes should not exceed 50,000 L/day. However, as the final details of elevation are not known, it may be prudent to submit an application in the event it is required. Under the EASR, a Permit to Take Water (PTTW) is not required for water taking for construction site dewatering for volumes less than 400,000 L/day.

6.7.3 Excavation and Temporary Protection Systems

The excavations for abutment and piers pile cap construction will extend through the existing fill and potentially into the clayey silt to clay deposits. Open-cut excavations into these materials 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. The existing fill and native soils would be classified as Type 3 soil above the groundwater table and Type 4 soil below the groundwater table, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V. In Type 4 soils, the temporary excavation side slopes should be formed no steeper than 3H:1V. If spaces constraints do not allow for these excavation slopes to be achievable then temporary protection systems will be required to maintain stability of the existing embankment.

Should steeper slopes be required, then temporary protection systems will be required to maintain stability of the excavation walls and embankment slope during subexcavation and construction of the proposed detour abutments. Where temporary protection systems are implemented, they should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, to avoid excessive movement of the existing bridge abutments. The contractor is responsible for the complete detailed design of the temporary shoring/protection systems.

The temporary support system could consist of soldier piles and lagging (temporary roadway protection) where the H-piles would be driven or installed in pre-drilled holes to a suitable depth, followed by horizontal lagging installed as the excavation proceeds. Soldier pile installation should be in accordance with OPSS 903 (Deep Foundations). Support to the wall, if required depending on the height, would likely require the use of tie backs. As either or both the soldier piles and tie backs could conflict with the existing bridge abutment piles and this should be checked by the structural designer.

The support systems may be designed using the following parameters:

Soil Type	**Coefficient of Earth Pressure**			**Internal Angle of Friction**	**Undrained Shear Strength**	**Unit Weight**
	Active, K_a	**At Rest, K_o**	**Passive, K_p**	**(ϕ, degrees)**	**(kPa)**	**(γ, kN/m^3)**
Existing Embankment Fill - Granular	0.30	0.46	3.37	33	-	20
Existing Embankment Fill - Cohesive	0.35	0.52	2.88	29	-	19
Clayey Silt to Clay	0.35	0.52	2.88	29	75 - 30	18.7
Sandy Silt Silt to and Sand	0.36	0.53	2.77	28	-	19
Sand and Gravel	0.30	0.46	3.37	33	-	20

The temporary shoring design should be assessed for both the drained (ϕ) and undrained (c_u) cases and the design should be based on the more conservative earth pressure conditions. Further, the total passive resistance of the temporary protection system below the base of the excavation should be calculated based on the values of K_p given above and then reduced by an appropriate factor of safety that considers the allowable wall movement as extrapolated from Figure C6.18 of the CHBDC (2014) to account for the fact that a large strain would be required for full mobilization of the passive resistance.

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are present, the coefficient of earth pressure should be adjusted accordingly.

6.7.4 Site Constraints on Construction

Given the proximity of the existing structure to the proposed TMB structure, the structural engineer shall exercise caution to ensure that there are no potential conflicts with the proposed deep foundations for the TMB or temporary shoring piles with the battered piles (unknown direction) of the existing structure.

Consideration will also need to be given to the construction considerations identified in GEOCRES 41J-342 for the proposed RSS walls, should the RSS walls be constructed while the TMB structure and detour are in place. An estimated unfactored downdrag load acting on a HP 310 x 110 pile of 350 kN per pile may be used for design at the piers.

Other site constraints include overhead electrical lines, which run underneath the east side of bridge, parallel to the rail tracks, which may impact construction accessibility. Underground utilities, belonging to the railway company or others, are often located within a railway right-of-way (ROW). The location of buried infrastructure within the ROW should be identified prior to finalizing the design and should be located in the field prior to any excavation / construction activities.

6.7.5 Vibration Monitoring

Vibrations induced on a structure up to a maximum peak particle velocity (PPV) of 100 mm/s are generally considered applicable for bridge structures in good condition. However, as the existing Highway 17 CPR Overhead structure is in poor condition, it is recommended that a lower peak particle velocity be adopted for this site, at least during the start of any pile driving operations and for the piles driven closest to the existing bridge. Based on vibration monitoring experience, it is considered unlikely that the vibrations induced by conventional construction activities (such as pile driving) will affect the performance of the existing structures, but may reach this threshold level. Therefore, vibration monitoring should be carried out during construction at this site adopting a PPV of 50mm/s initially.

6.7.6 Existing Structure Monitoring

We recommend that the abutments and piers of the existing structure be monitored for settlement and lateral movement during the new construction, especially during construction works adjacent to the existing structure, such as excavation operations, installation of temporary protection systems and pile driving, for the following reasons:

- the proximity of the proposed TMB with the existing structure;
- the age and poor condition of the existing structure; and
- the requirement for the existing structure to carry traffic during construction of the TMB.

The foundation monitoring should be carried out by a qualified foundations consultant reporting to the Contract Administrator.

6.7.7 Obstructions

Blast rock fill (cobble and boulder sizes) were observed on the surface of the embankment in the northeast quadrant of the existing structure which could affect the installation of deep foundations, excavations for foundations and installation of temporary roadway protection systems (if required), in particular at the proposed TMB east pier. Further the previous investigations and drawings indicate the potential for the presence of blast rock fill in the vicinity of the existing east abutment and proposed TMB east pier. Further, cobbles were encountered in the native soils at this site in Boreholes C17-5 and C17-7. An NSSP should be included in the Contract Documents to identify to the contractor the possible presence of cobbles and/or boulders within the embankment fill; an example of which is included in Appendix E.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Adam Core, P.Eng., and the technical aspects were reviewed by Ms. Sarah E.M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M.A. Costa, P.Eng., Golder's Designated MTO Foundations Contact for this project and Senior Consultant of Golder, conducted an independent quality review of the report.

Report Signature Page

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Terzaghi, K. and Peck, R.B. 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil

Commercial Software

SLOPE/W (Version 7.23) by Geo-Slope International Ltd.

Settle-3D (Version 2.003) by RocScience Inc.

LPile 2013 (Version 7.05) by Ensoft Inc.

Ontario Occupational Health and Safety Act

Ontario Regulation 213/91 Construction Projects

Ontario Provincial Standard Drawings

OPSD 202.010	Slope Flattening using surplus excavated material on earth or rock embankments
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario
OPSD 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3101.200	Walls Abutment, Backfill Rock
OPSD 3121.150	Walls Retaining, Backfill Minimum Granular Requirement

Ontario Provincial Standard Specifications

OPSS. 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling – Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act

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

Table 1: Evaluation of TMB Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Spread Footings on Bedrock or Granular Pad Over Bedrock (East Abutment)	1	■ Can minimize bedrock excavation depending on design footing level. ■ Bedrock is exposed or present at relatively shallow depth at the east abutment. ■ Adequately high axial resistances for footings on bedrock.	■ Variable bedrock surface may require bedrock excavation or mass concrete placement to achieve level footing. Controlled blasting will be required adjacent to existing structure. ■ Associated settlements of the temporary detour, cohesive embankment fill and native soils make this option not feasible for supporting west abutment, west pier and east pier.	■ Typically lower cost than deep foundations.	■ If bedrock is higher than anticipated, bedrock removal is required. ■ Variability in bedrock surface will impact mass concrete quantities and excavation depths.
Steel H-piles driven to bedrock (West Abutment, West Pier, East Pier)	1	■ Conventional construction (excluding at the east pier, which will likely encounter steeply dipping bedrock surface) ■ Higher axial resistance compared to spread footings founded on soil subgrade at west abutment and west pier. ■ Shallower excavation for pile cap compared to spread footings which will eliminate the need for dewatering. ■ Minimum pile length should be achievable without bedrock socketing.	■ Not practical at the east abutment as bedrock is present at shallow depth or exposed. ■ East pier will require pre-drilled holes through embankment rock fill and socketing into the sloping bedrock.	■ Relative costs lower than other deep foundation options.	■ Potential for “hanging-up” on obstructions within the fill. ■ Experiencing difficulties seating the piles at the east pier due to the sloping bedrock surface.
Micropiles	3	■ Lighter weight equipment to construct micropiles may be advantageous at this site. ■ Can drill into sloping bedrock.	■ Requires detailed micropile design/drawings/specifications. ■ Pile load tests required to confirm capacity for design. ■ Can pre-drill through rock fill utilizing a sacrificial casing at east pier	■ Additional cost associated with detail micropile design. ■ Cost for specialist contractor. Typically higher than for driven steel H-piles and similar to drilled steel casings; may be less expensive than larger diameter drilled shafts. ■ Additional cost for the micropile pile load tests.	■ Low risk of not achieving design resistance. ■ Lower risk of impacting existing bridge due to lower vibration levels.

Foundation Type	Rank	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Drilled Shafts (Small Diameter)	4	■ Higher axial resistances compared to steel H-piles. ■ Easier to penetrate obstructions and rock fill and socket into sloping bedrock compared to larger diameter caissons, or H-Piles.	■ Temporary liners may be required at some locations (i.e. west pier) to control groundwater. ■ May still encounter potential for difficulties penetrating through rock fill or obstructions, although less than for larger diameter caissons. ■ Potential for installation difficulty due to sloping bedrock, although less than for larger diameter caissons or driven H-Piles.	■ Relative costs higher than for steel H-piles (similar cost to micropiles).	■ Lower risk of difficulties during installation through rock fill or on sloping bedrock. ■ Potential for construction problems associated with groundwater during caisson installation.
Drilled Shafts (Large Diameter Caissons)	NR	■ Higher axial resistances compared to steel H-piles. ■ Could consider extending caissons as columns to underside of the structure.	■ Temporary liners may be required at some locations (i.e. west pier) to control groundwater inflow. ■ Potential for difficulties penetrating through rock fill or obstructions compared to piles or drilled shaft, micropiles. ■ Potential for installation difficulty due to sloping bedrock surface.	■ Relative costs much higher than for steel H-piles, micropiles or smaller diameter drilled casings.	■ Highest potential risk of difficulties being encountered during installation due to the presence of rock fill and sloping bedrock surface. ■ Potential for construction problems associated with groundwater inflow during caisson installation.

NR: Not Recommended

Site Photographs

North Side of Hwy 17 Overhead Looking East Across CPR Tracks

North Side of Hwy 17 Overhead Looking West Across CPR Tracks

Project No.	1651997-WO1	**Golder Associates Ltd.**	Inputted by:	TB
Date:	September, 2017		Checked by:	SP

Site Photographs

Looking North at South side of Hwy 17 Overhead

Looking South from Location of Borehole C17-3

MINISTRY OF TRANSPORTATION, ONTARIO

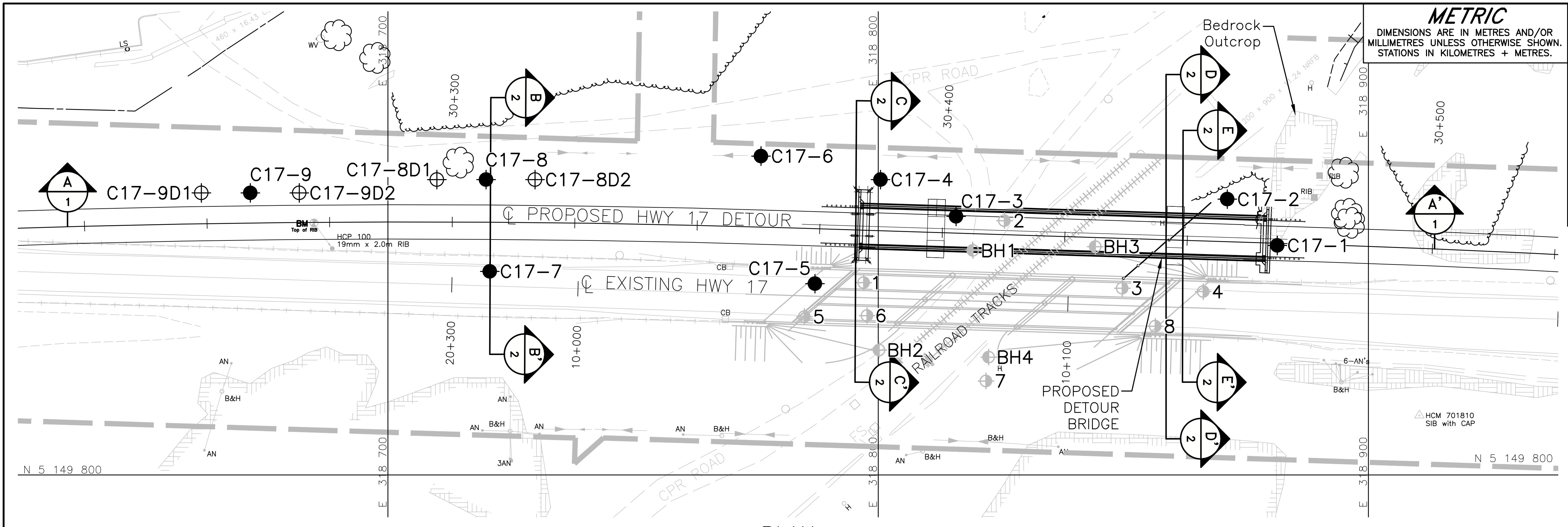
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5165-10-01

HIGHWAY 17
CONISTON CPR OVERHEAD
BOREHOLE LOCATIONS AND SOIL STRATA

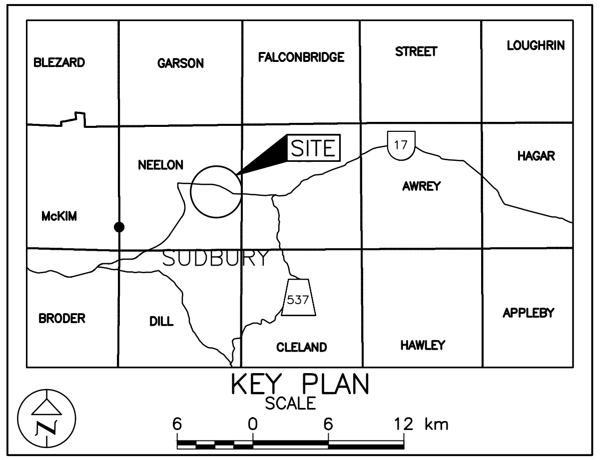
SHEET
.

Golder Associates



PLAN

SCALE
10 0 10 20 m



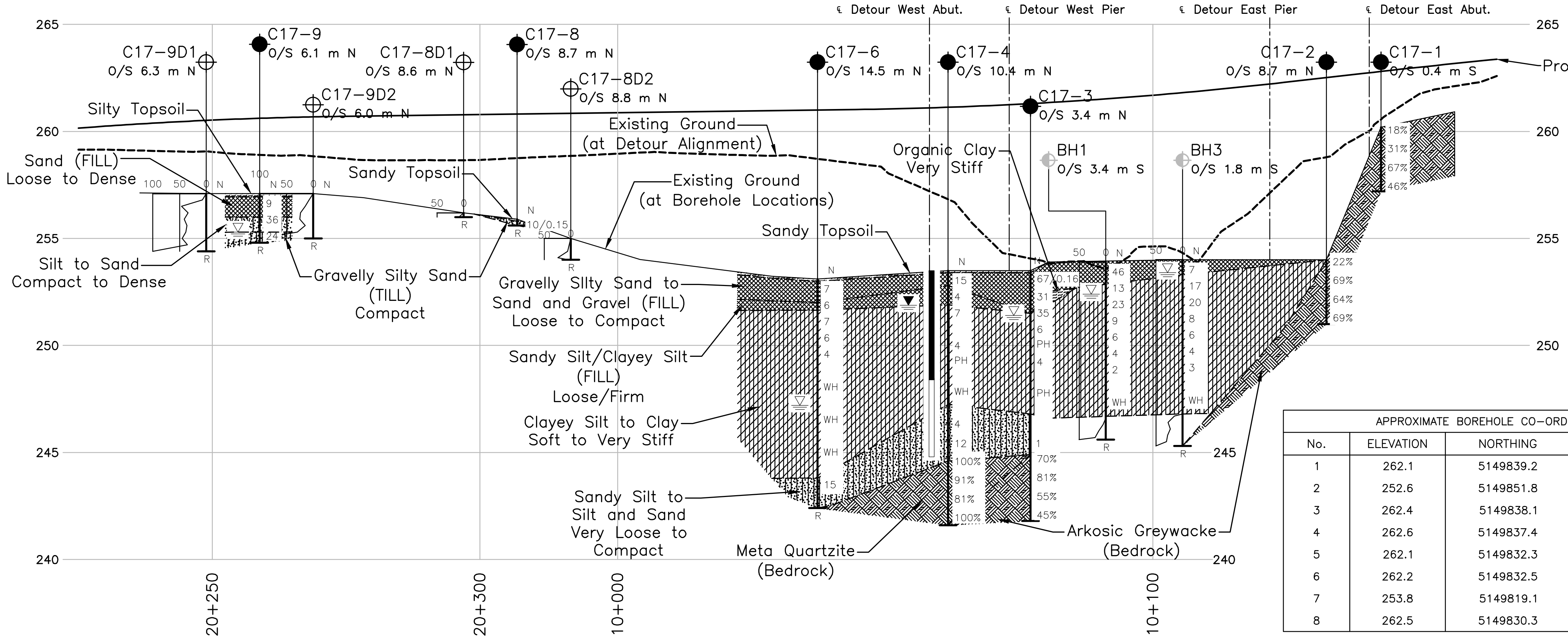
KEY PLAN
SCALE
6 0 6 12 km

LEGEND

●	Borehole – 2017
◐	Borehole – 2016 (Previous Investigation – Golder)
◐	Borehole – 1975 (Previous Investigation – GEOCON)
⊕	Dynamic Cone Penetration Test
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
R	Refusal
100%	Rock Quality Designation (RQD)
▼	WL in piezometer, measured on APR 27, 2017
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
BH1	253.9	5149845.7	318819.3
BH2	254.1	5149825.5	318800.2
BH3	254.0	5149846.5	318844.3
BH4	254.4	5149824.0	318822.5
C17-1	260.2	5149846.8	318881.4
C17-2	254.0	5149856.2	318871.2
C17-3	253.5	5149852.7	318815.9
C17-4	253.5	5149860.2	318800.5
C17-5	262.6	5149839.0	318787.1
C17-6	253.1	5149865.0	318776.1
C17-7	261.7	5149841.5	318720.8
C17-8	255.9	5149860.2	318720.0
C17-8D1	256.2	5149860.2	318710.0
C17-8D2	255.0	5149860.2	318730.0
C17-9	257.1	5149857.5	318671.9
C17-9D1	257.1	5149857.5	318661.9
C17-9D2	257.1	5149857.5	318681.9



A–A'
1

OVERHEAD DETOUR – CENTRELINE PROFILE

HORIZONTAL SCALE
10 0 10 20 m
2.5 0 2.5 5 m
VERTICAL SCALE

APPROXIMATE BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
1	262.1	5149839.2	318797.0
2	252.6	5149851.8	318825.8
3	262.4	5149838.1	318849.8
4	262.6	5149837.4	318866.3
5	262.1	5149832.3	318785.0
6	262.2	5149832.5	318797.7
7	253.8	5149819.1	318822.0
8	262.5	5149830.3	318856.5

NOTES

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

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

REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. Base.dwg and x1124160_46-123_Detour Alignment.dwg, received MAY 31, 2017 and 46-123TMB_01 – Option 2.dwg, received June 15, 2017.

S.E.M. POOT
90471921
AUG. 31, 2017

J.M.A. COSTA
AUG. 31, 2017

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 41I-352

HWY. 17	PROJECT NO. 1651997		DIST. .
SUBM'D. .	CHKD. AC	DATE: 8/31/2017	SITE: 46-123
DRAWN: TB	CHKD. SEMP	APPD. JMAC	DWG. 1

PLOT DATE: September 1, 2017
FILENAME: S:\Clients\MTO\1651997 [illegible] Coniston.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

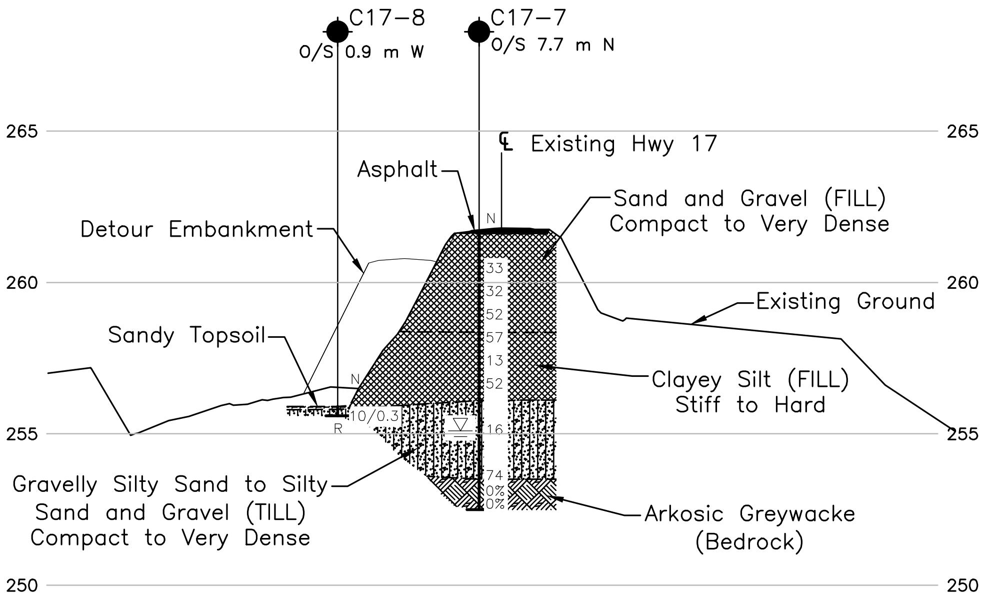
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5165-10-01

HIGHWAY 17
CONISTON CPR OVERHEAD
SOIL STRATA

SHEET

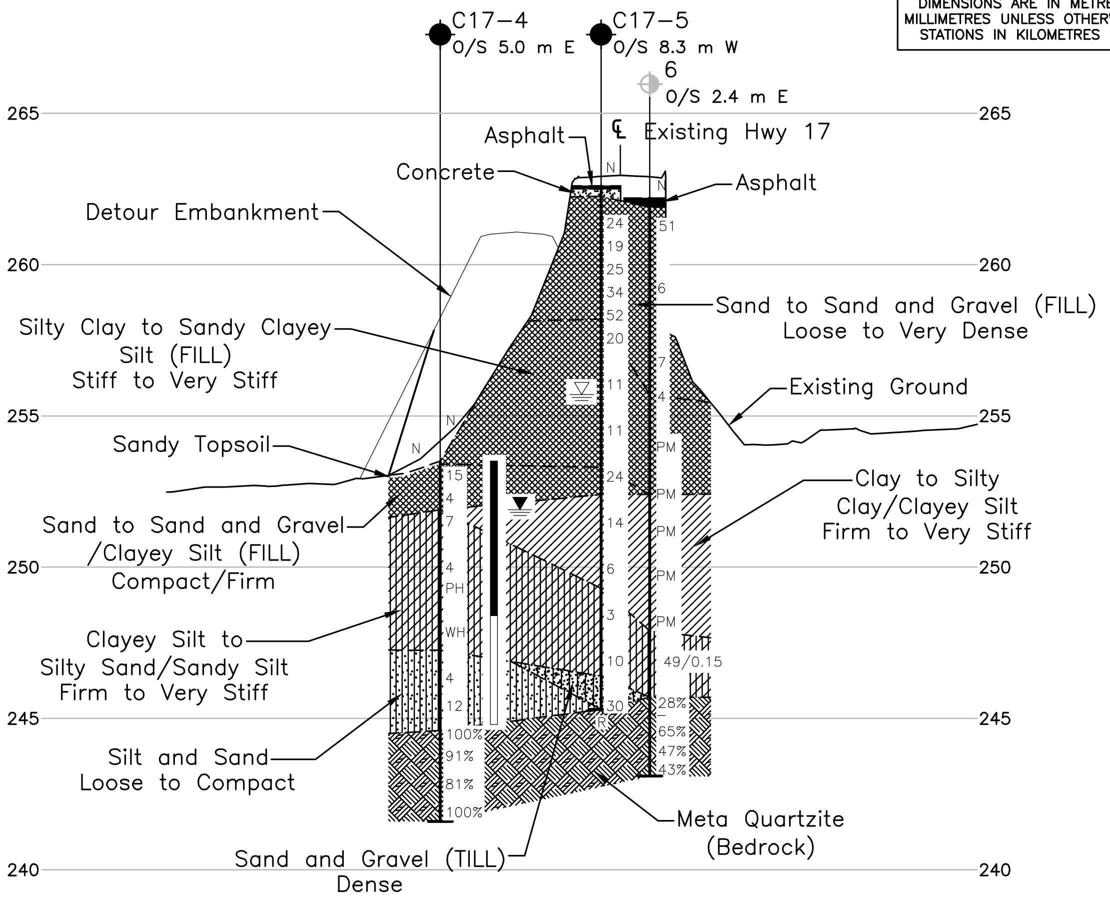
Golder Associates



SECTION AT STA 20+308 (30+308 DETOUR)

B-B'
1

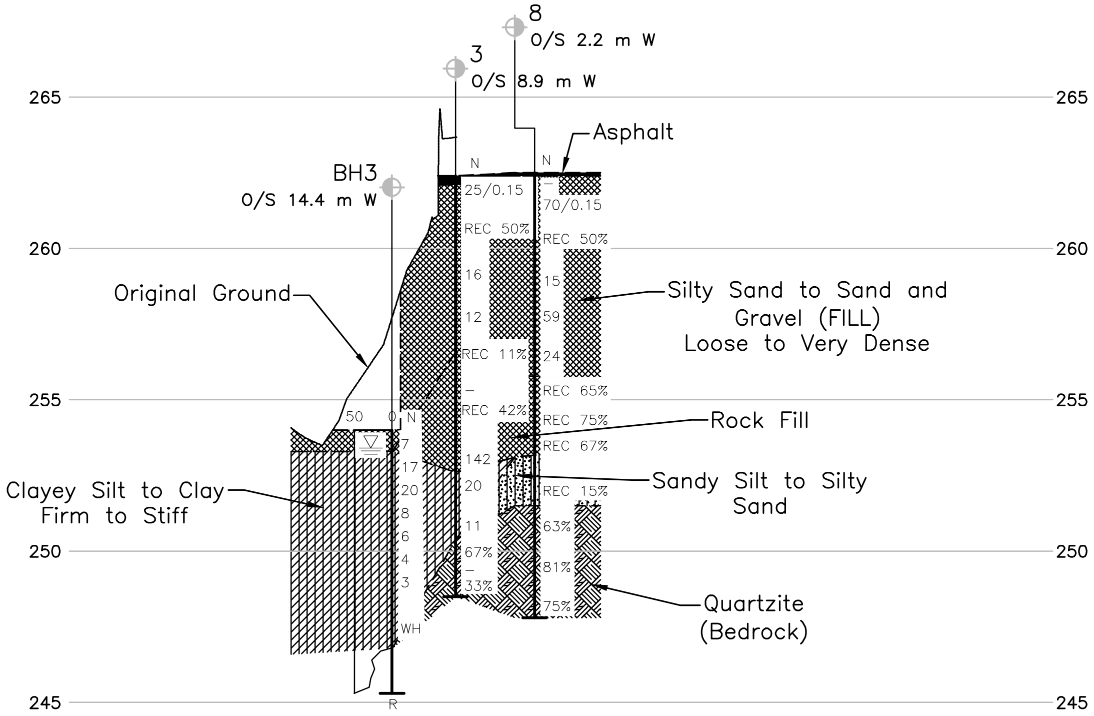
HORIZONTAL SCALE 10 0 10 20 m
2.5 0 2.5 5 m VERTICAL SCALE



SECTION AT STA 10+057 (30+381 DETOUR)

C-C'
1

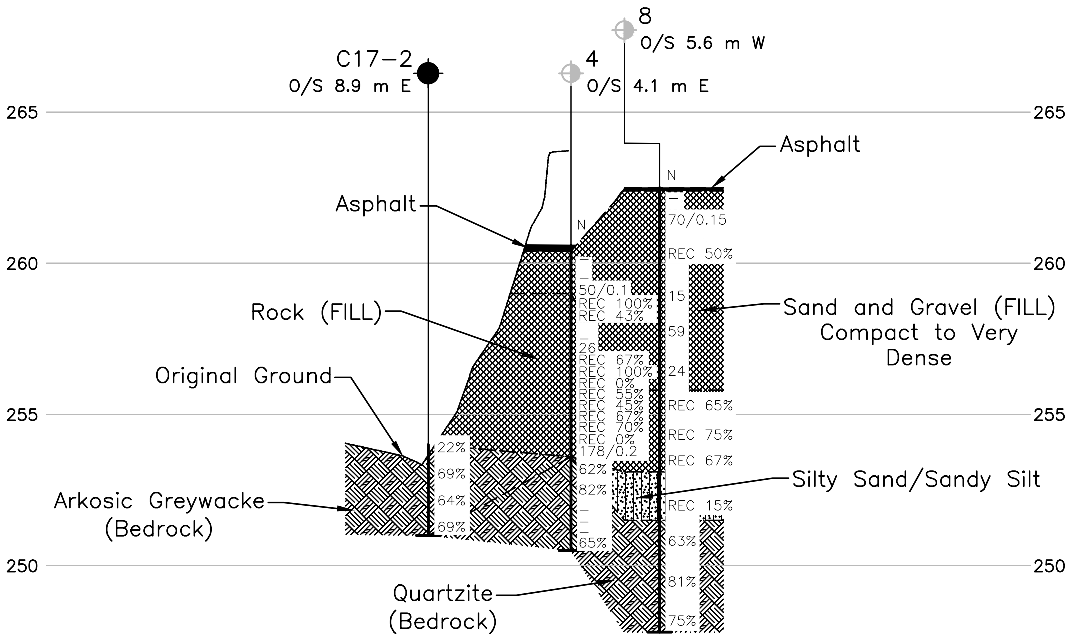
HORIZONTAL SCALE 10 0 10 20 m
2.5 0 2.5 5 m VERTICAL SCALE



SECTION AT STA 10+120 (30+446 DETOUR)

D-D'
1

HORIZONTAL SCALE 10 0 10 20 m
2.5 0 2.5 5 m VERTICAL SCALE



SECTION AT STA 10+123 (30+449 DETOUR)

E-E'
1

HORIZONTAL SCALE 10 0 10 20 m
2.5 0 2.5 5 m VERTICAL SCALE

LEGEND

- ● Borehole – 2017
- ◐ Borehole – 2016 (Previous Investigation – Golder)
- ◐ Borehole – 1975 (Previous Investigation – GEOCON)
- ▮ Seal
- ▯ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
- REC Recovery (%)
- 100% Rock Quality Designation (RQD)
- ▼ WL in piezometer, measured on APR 27, 2017
- ▽ WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
3	262.4	5149838.1	318849.8
4	262.6	5149837.4	318866.3
6	262.2	5149832.5	318797.7
8	262.5	5149830.3	318856.5
BH3	254.0	5149846.5	318844.3
C17-2	254.0	5149856.2	318871.2
C17-4	253.5	5149860.2	318800.5
C17-5	262.6	5149839.0	318787.1
C17-7	261.7	5149841.5	318720.8
C17-8	255.9	5149860.2	318720.0

NOTES

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

REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. Base.dwg and x1124160_46-123_Detour Alignment.dwg, received MAY 31, 2017 and 46-123TMB_01 – Option 2.dwg, received June 15, 2017.

S.E.M. POOT
90471921
AUG. 31, 2017

J.M.A.COSTA
AUG. 31, 2017

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 41I-352

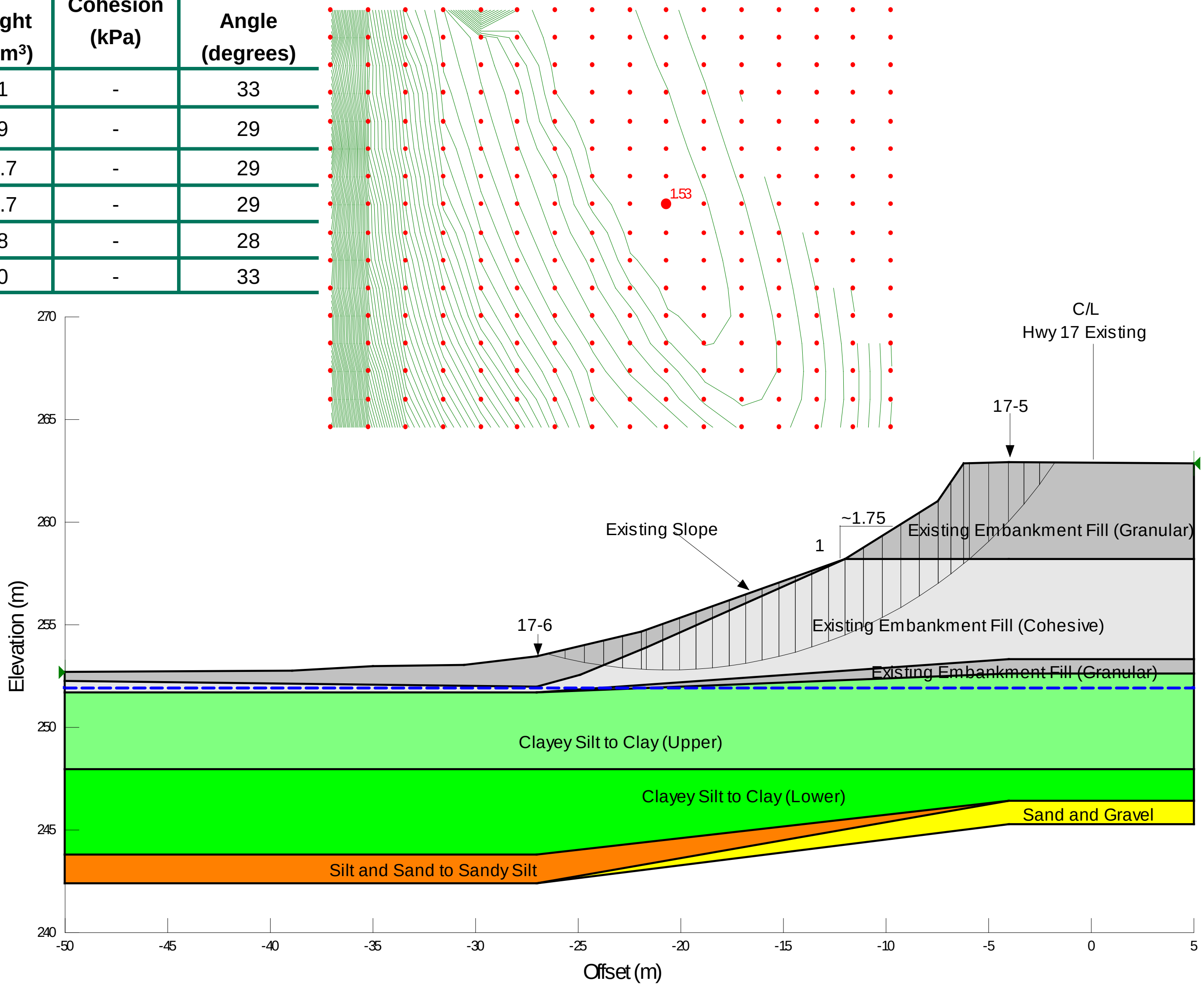
HWY. 17		PROJECT NO. 1651997	DIST. .
SUBM'D. .	CHKD. AC	DATE: 8/31/2017	SITE: 46-123
DRAWN: TB	CHKD. SEMP	APPD. JMAC	DWG. 2

PLOT DATE: September 1, 2017
FILENAME: S:\Clients\MTO\1651997 aecom_5015-e-0045_ne retainer\99_PROJ\1651997_Coniston\40_PROD\1651997_Coniston.dwg

Stability Analysis
West Approach
Existing Conditions – Drained

Figure 1

Material Name	Unit Weight (kN/m³)	Cohesion (kPa)	Friction Angle (degrees)
Existing Embankment Fill	21	-	33
Existing Embankment Fill – Cohesive	19	-	29
Clayey Silt to Clay (Upper)	18.7	-	29
Clayey Silt to Clay (Lower)	18.7	-	29
Silt and Sand to Sandy Silt	18	-	28
Sand and Gravel	20	-	33



Date: June 2017

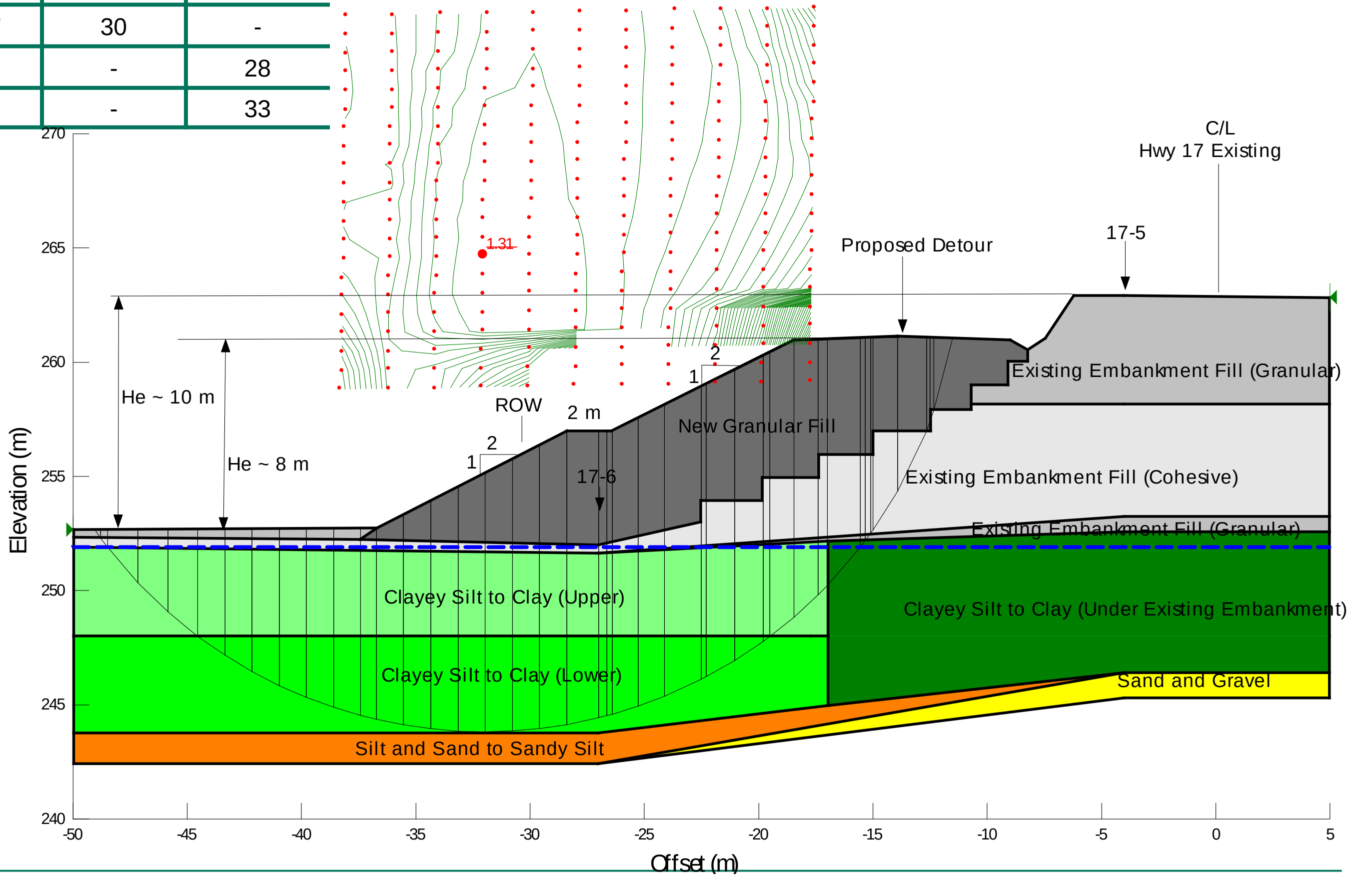
Project No: 1651997 – Coniston CPR Detour

Analysis By: AC **Reviewed By:** SEMP

Stability Analysis
West Approach
Temporary Detour Embankment – Undrained

Figure 2

Material Name	Unit Weight (kN/m³)	Cohesion (kPa)	Friction Angle (degrees)
New Embankment Fill – Granular	21	-	35
Existing Embankment Fill – Granular	21	-	33
Existing Embankment Fill – Cohesive	19	-	29
Clayey Silt to Clay (Under Existing Embankment)	18.7	75	-
Clayey Silt to Clay (Upper – Toe of Slope)	18.7	75 – 30	-
Clayey Silt to Clay (Lower – Toe of Slope)	18.7	30	-
Silt and Sand to Sandy Silt	18	-	28
Sand and Gravel	20	-	33



Date: June 2017

Project No: 1651997 – Coniston CPR Detour

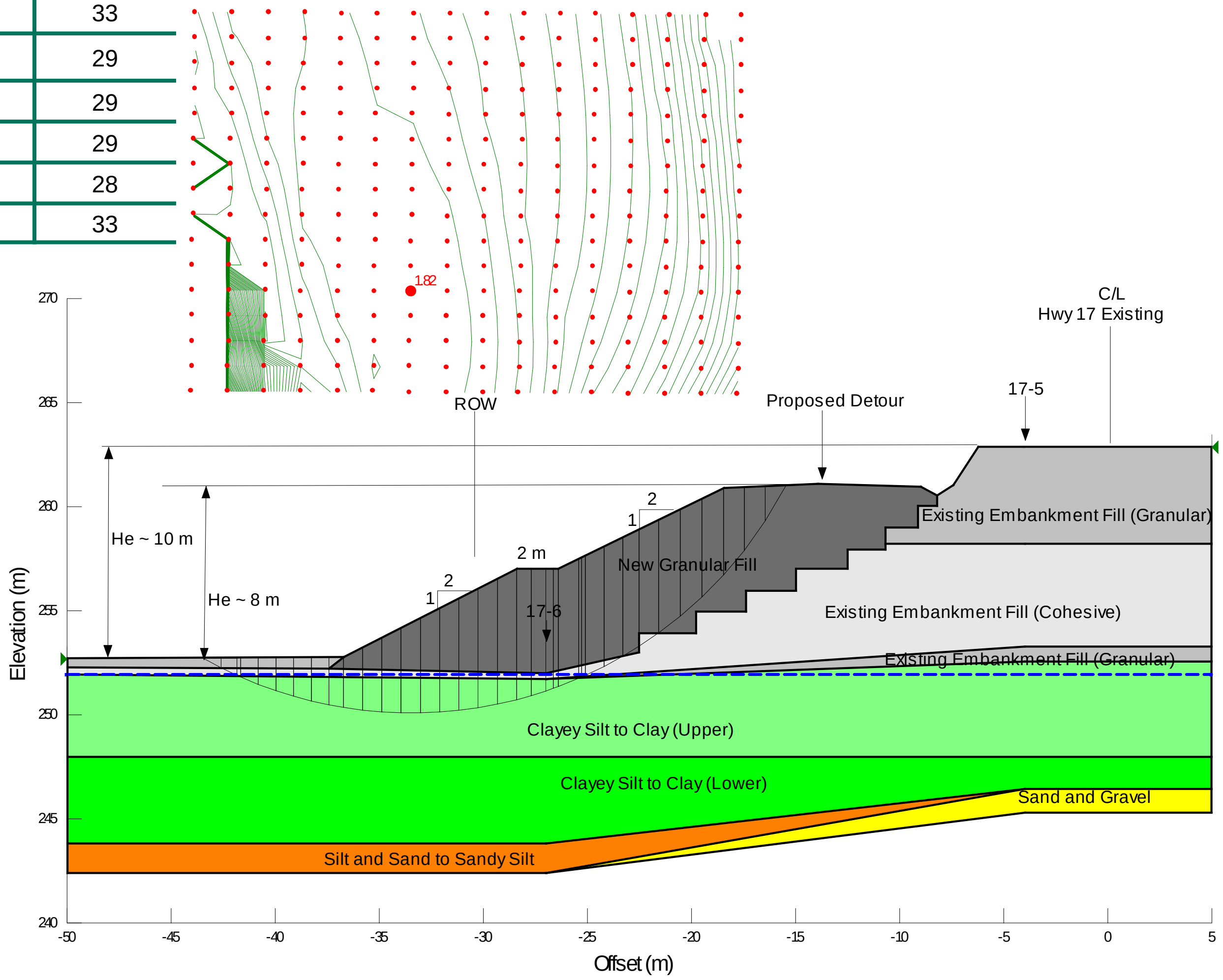
Analysis By: AC **Reviewed By:** SEMP

Golder Associates

Stability Analysis
West Approach
Temporary Detour Embankment – Drained

Figure 3

Material Name	Unit Weight (kN/m³)	Cohesion (kPa)	Friction Angle (degrees)
New Embankment Fill – Granular	21	-	35
Existing Embankment Fill – Granular	21	-	33
Existing Embankment Fill – Cohesive	19	-	29
Clayey Silt to Clay (Upper)	18.7	-	29
Clayey Silt to Clay (Lower)	18.7	-	29
Silt and Sand to Sandy Silt	18	-	28
Sand and Gravel	20	-	33



Date: June 2017

Project No: 1651997 – Coniston CPR Detour

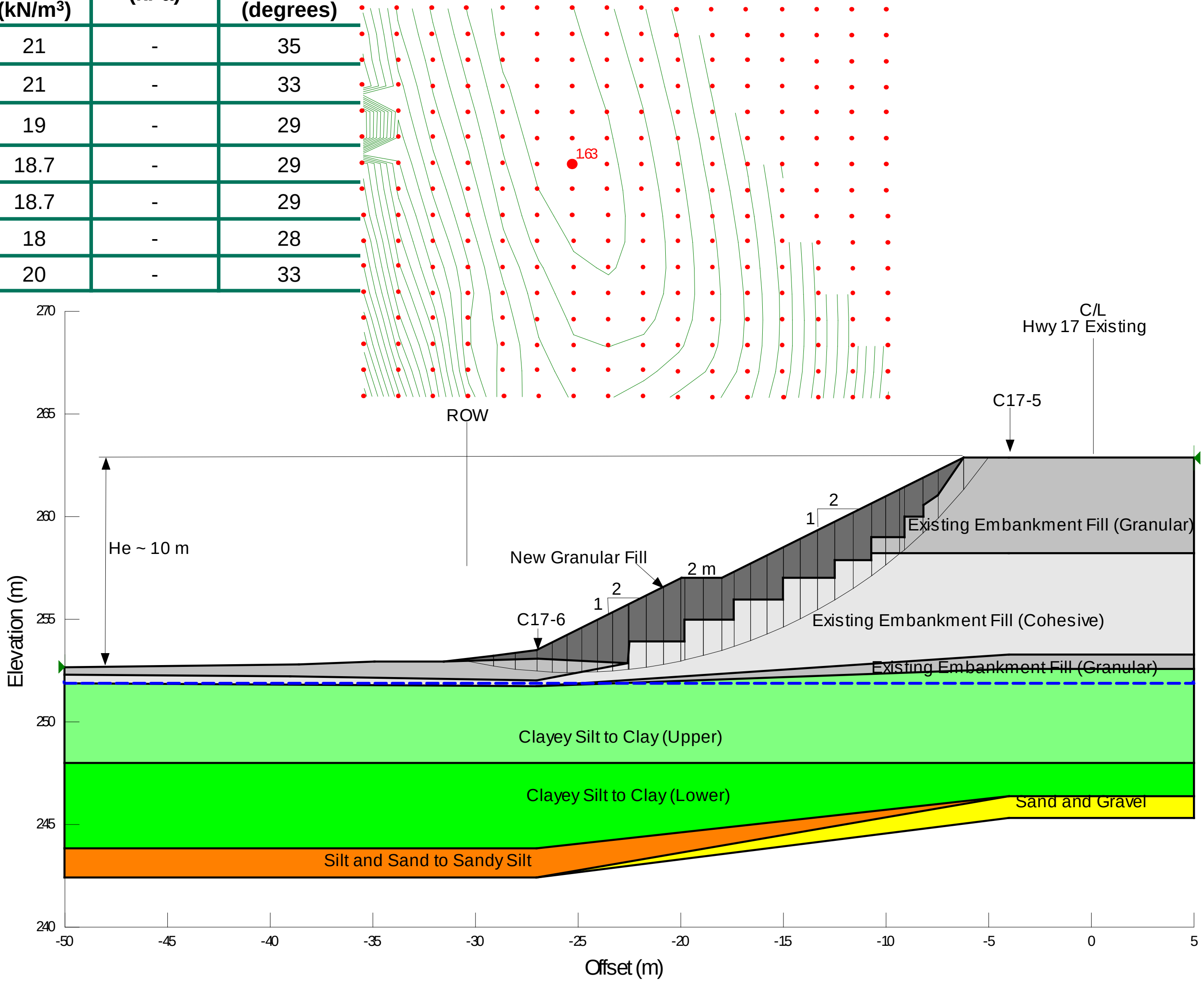
Analysis By: AC **Reviewed By:** SEMP

Golder Associates

Stability Analysis
West Approach
Final Embankment Configuration – Drained

Figure 4

Material Name	Unit Weight (kN/m³)	Cohesion (kPa)	Friction Angle (degrees)
New Embankment Fill – Granular	21	-	35
Existing Embankment Fill – Granular	21	-	33
Existing Embankment Fill – Cohesive	19	-	29
Clayey Silt to Clay (Upper)	18.7	-	29
Clayey Silt to Clay (Lower)	18.7	-	29
Silt and Sand to Sandy Silt	18	-	28
Sand and Gravel	20	-	33



Date: June 2017

Project No: 1651997 – Coniston CPR Detour

Analysis By: AC **Reviewed By:** SEMP

APPENDIX A

Borehole Records – GEOCON 1975

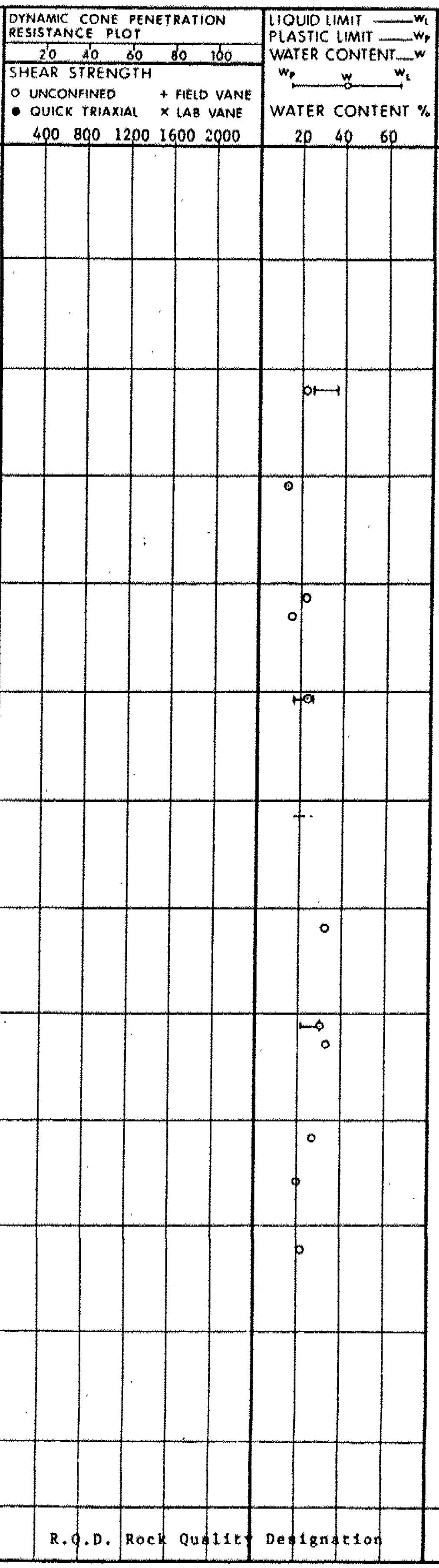
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS–ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 1

WP 158-74-01
DIST 17 HWY 17
DATUM Geodetic
LOCATION Sta. 101 + 95 o/s 9.0 Lt. ¢ Hwy. 17
BORING DATE January 13, 1977
BOREHOLE TYPE NW, NX & BX Casing, AXT Rock Core
ORIGINATED BY AEL
COMPILED BY RAH
CHECKED BY RGC

SOIL PROFILE		SAMPLES			GROUND WATER	REMARKS
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEV	% GR SA SI CL
860.0	Ground Level					
0.0	Asphalt	1A	WS	-		
0.8	Fill Sand & gravel Compact	1B	WS	-		
		2	WS	-		
		3	WS	-		
		4	SS	30/3"	855	
	grey brown	5	SS	23		
851.0 9.0	Fill Silty clay/clayey silt Stiff	6	SS	8		0 12 58 30
	grey brown	7	TW	PM	845	
					840	
		8	SS	14		0 54 38 8 0 12 60 20
		9	SS	13	835	
831.3 28.7	Silty sand & gravel				830	
828.4	Compact grey brown	10	SS	30		0 77 19 4
31.6	Silty clay/clayey silt Varved Stiff	11	WS	-		
	brown	12	SS	13	825	
819.0					820	
41.0	Silty clay/clayey silt Varved	13	SS	13		
816.0	Stiff to firm grey					
44.0	Silty sand/sandy silt with occasional gravel				815	
	Compact to very dense	14	SS	19		0 50 47 3
		15	SS	14		
	grey				810	
807.8		16	SS	53		2 46 48 4
52.2	Bedrock	17	AXT RC	93%		RQD [illegible]
	Note:				805	
	Medium to fine grained grey, hard quartzite. Joint	18	AXT RC	98%		RQD 41%
	spacing close to very close. Core generally fractured.	19	AXT RC	91%	800	RQD 32%
796.9						
63.1	End of Borehole Note: W.L. Not Established					



UNIT WEIGHT γ

R.Q.D. Rock Quality Designation

15 ⌖ 5 (20 above, 10 below) % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

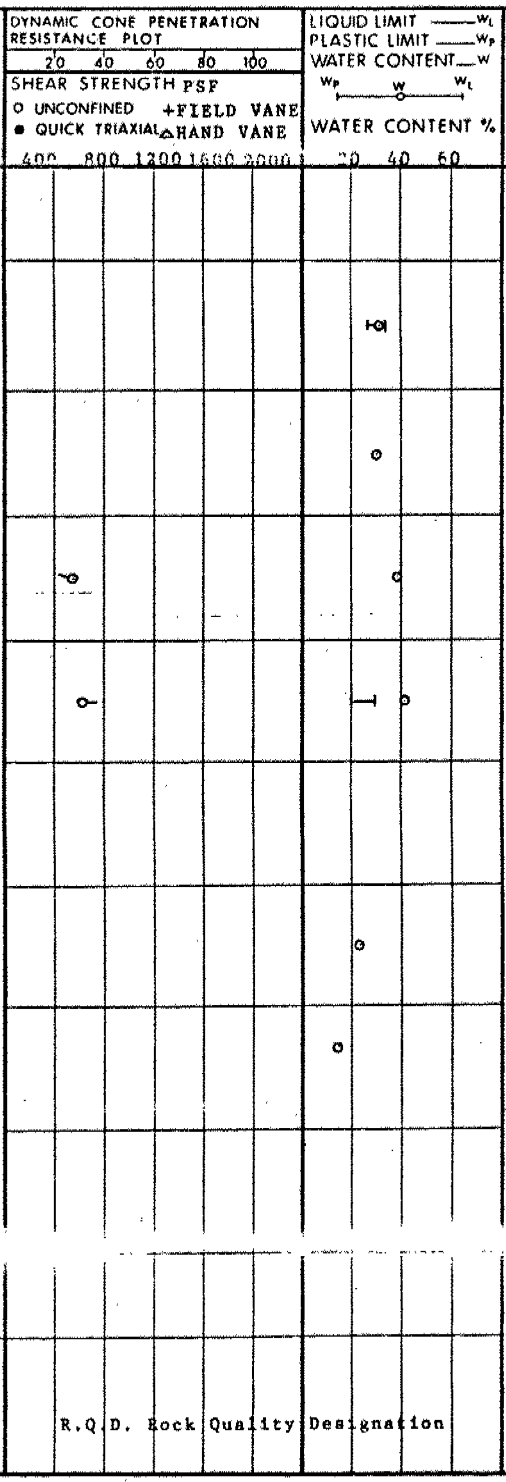
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 2

WP 158-74-01
LOCATION Sta. 102 + 86 o/s 48.5 Lt. ∅ Hwy. 17
ORIGINATED BY AEL
DIST 17 HWY 17
BORING DATE December 10, 13, & 19, 1976.
COMPILED BY RAH
DATUM Geodetic
BOREHOLE TYPE BX & AX Cased & Cored
CHECKED BY RGC

SOIL PROFILE		SAMPLES					
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	UNIT WEIGHT γ	REMARKS % GR SA SI CL
828.6	Ground Level						
0.0	Silty Sand with gravel Compact brown	1	SS	23	▼		
825.6 3.0	Silty clay/clayey silt Varved Stiff to firm brown				825		
		2	SS	4			0 4 67 29
					820		
		3	SS	4			
815.1 13.5	Silty clay/clayey silt Varved Firm grey				815		
		4	TW	PM		115	0 2 74 24
					810		
		5	TW	PM		111	0 2 78 20
					805		
803.6 25.0	Silty sand/sandy silt with occasional gravel Loose brown	6	WS	-			
					800		
		7	SS	5			3 34 48 15
					795		
		8	SS	50/4"			28 75 (3)
792.1 36.5	Bedrock Note: Medium to fine grained, grey. Joint spacing close to very close. Core generally fractured.	9	BX RC	100%			RQD 22%
		10	BX RC	92%	790		RQD 69%
		11	BX RC	75%			RQD -
		12	BX RC	63%			RQD -
		13	AX RC	75%			RQD -
		14	AX RC	80%	785		RQD 43%
781.6 47.0	End of Borehole						



R.Q.D. Rock Quality Designation

20
15 ◇ 5 % STRAIN AT FAILURE
10

TT

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 3

WP 158-74-01
DIST 17 HWY 17
DATUM Geodetic

LOCATION Sta. 103 + 65 o/s 10.0 Lt. ℄ Hwy. 17
BORING DATE January 20 - 22, 1977.
BOREHOLE TYPE NX, BX & AX Casing, AXT Rock Core

ORIGINATED BY AEL
COMPILED BY RAH
CHECKED BY RGC

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER	REMARKS
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEV	% GR SA SI CL
860.8	Ground Level					
0.0	Asphalt					
1.0	Fill Sand and gravel Compact to very dense grey brown Note: Cobbles and boulders up to 6 inches in size, encountered from 3 to 7 feet. NXCA drilled from 0 to 5 feet. BXCA drilled from 5 to 7 feet.	1	SS	25/6	860	
		2	BXL RC	50%	855	
		3	SS	16	850	
		4	SS	12	845	
		5	BXL RC	11%		
841.8 19.0	Fill Rock Note: Rock up to 18 inches in size encountered. BXCA drilled from 19 to 21.8 feet. AXCA drilled from 21.8 to 29,8 feet.				840	
		6	BXL RC	-		
		7	BXL RC	42%	835	
		8	SS	142	830	
828.8 32.0	Silty clay/clayey silt Stiff brown	9	SS	20		
					825	
		10	SS	11		0 2 58 40
821.3 39.5	Bedrock Note: Medium to fine grained, grey hard quartzite. Joint spacing close. Core reasonably sound.	11	AXT RC	88%	820	RQD 67%
		12	AXT RC	91%		RQD -
815.3 45.5	End of Borehole Note: W.L. Not Established	13	AXT RC	100%		RQD 33%

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH: ○ UNCONFINED +FIELD VANE ● QUICK TRIAXIAL ×LAB VANE: 400 800 1200 1600 2000

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w
w_P — w — w_L

WATER CONTENT %: 20 40 60

UNIT WEIGHT γ

R.Q.D. Rock Quality Designation

20
15 ◇ 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 4

WP 158-74-01
DIST 17 HWY 17
DATUM Geodetic
LOCATION Sta. 104 + 19 o/s 10.0 Lt. ℄ Hwy. 17
BORING DATE January 3 - 6, 1977
BOREHOLE TYPE NX & BX Casing, BX & AX Rock Core
ORIGINATED BY AEL
COMPILED BY RAH
CHECKED BY RGC

OFFICE REPORT ON SOIL EXPLORATION

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 400 800 1200 1600 2000
LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w
w_P —○— w —— w_L
WATER CONTENT %: 20 40 60
UNIT WEIGHT γ
REMARKS: % GR SA SI CL

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	DYNAMIC CONE PENETRATION / SHEAR STRENGTH	REMARKS
861.5	Ground Level						
0.0	Asphalt						
0.5	Fill Sand and gravel Very dense grey brown	1	WS	-	860	Note: Fill from 0.5 to 5.4. Cobbles and boulders encountered up to 6 inches in size from 3 to 5 feet. NXCA drilled from 1.6 to 5.9 feet.	
		2	WS	-			
856.1		3	SS	50/4"			
5.4	Fill Rock Note: Rock up to 18 inches in size encountered. NXCA drilled from 5.4 to 8.5 feet. BXCA drilled from 8.5 to 22.9 feet.	4	BX,RC	100%	855		
		5	BX RC	43%			
		6	BX,RC	-			
		7	SS	26	850		
		8	BX,RC	67%			
		9		100%			
		10	BX,RC	0%			
		11	BX RC	55%			
		12	BX RC	45%	845		
		13	BX RC	67%			
		14	BX RC	70%			
		15	BX,RC	0%	840		
838.6		16	SS	178/8"			
22.9	Bedrock Note: Medium to fine grained grey, hard quartzite. Joint spacing close. Loss of return water down to 28'2". Core generally sound.	17	BX,RC	67%			RQD 62%
		18	BX RC	94%	835		RQD 82%
		19	BX	100%			RQD -
		20		[illegible]			RQD -
		21	RC	100%			RQD -
828.3		22	AX RC	100%	830		RQD 65%
33.2	End of Borehole						

Note: W.L. Not Established

R.Q.D. Rock Quality Designation

20
15 ⌖ 5 % STRAIN AT FAILURE
10

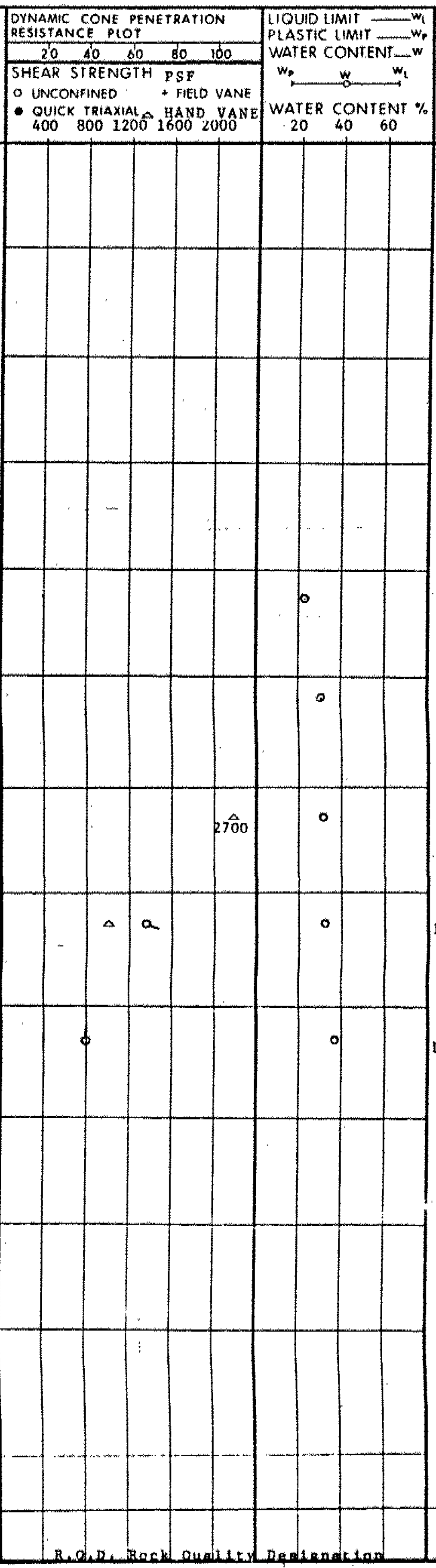
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS – ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 5

WP 158-74-01
DIST 17 HWY 17
DATUM Geodetic
LOCATION Sta.101+56 O/S 12.5 Lt. ₵ Hwy.17
BORING DATE January 26-29, 1977
BOREHOLE TYPE H.S. Augers & AX Casing, AXT Rock Core
ORIGINATED BY AEL
COMPILED BY RAH
CHECKED BY RGC

SOIL PROFILE		SAMPLES			GROUND WATER	UNIT WEIGHT	REMARKS
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEV	γ	% GR SA SI CL
859.8	Ground level						
0.0	Asphalt						
0.5	Fill Sand and gravel Very dense to compact				855		
					850		
		1	SS	14			
846.3 / 13.5	Fill Silty clay/clayey silt Stiff grey & brown				845		
		2	TW	PM			
					840		
		3	TW	PM			
					835		
		4	TW	PM			
832.8 / 27.0	Silty clay/clayey silt Varved Very stiff to stiff brown				830		
		5	TW	PM			
					825		
		6	TW	PM		116	
821.8 / 38.0	Siltyclay/clayey silt Varved Firm grey				820		
		7	TW	PM		122	
					815		
		8	TW	PM			
813.4 / 46.4	Silty sand/sandy silt with occasional gravel Very dense grey brown				810		
		9	SS	51			
		10	SS	50/3"	805		
[illegible] / 57.7	Bedrock Note: Medium to fine grained grey, hard quartzite, joint spacing close to very close. Core generally fractured.	11	AXT RC	88%	800		RQD 42%
797.6		12	AXT RC	100%			RQD -
62.2	End of Borehole						
	Note: W.L. Not Established						



R.Q.D. Rock Quality Designation

20
15 ◇ 5 % STRAIN AT FAILURE
10

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 6

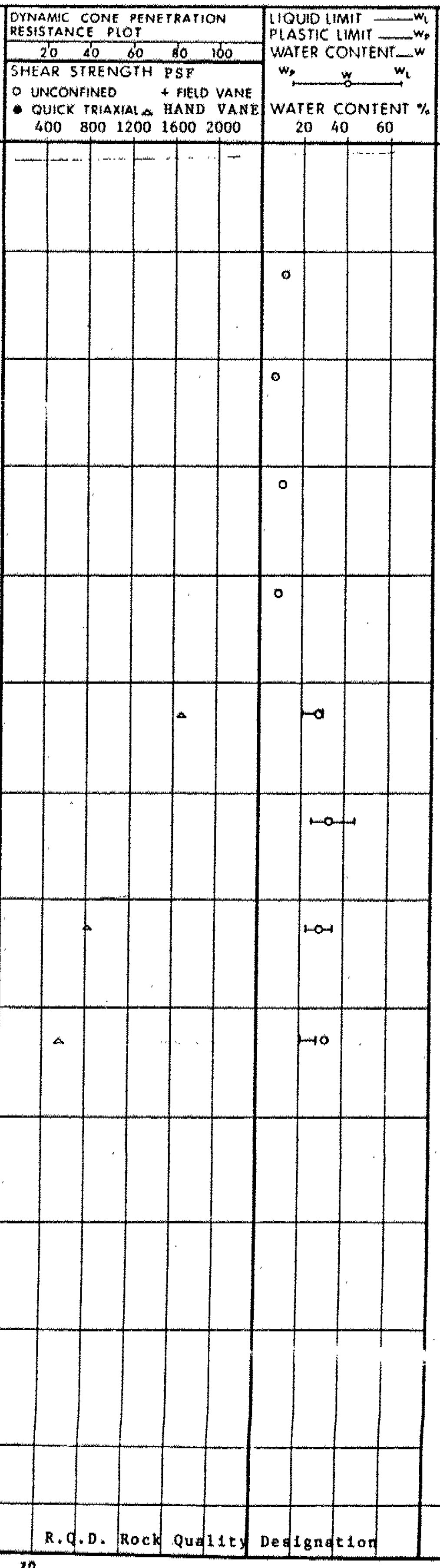
WP 158-74-01
DIST 17 HWY 17
DATUM Geodetic

LOCATION Sta. 101 + 98 o/s 11.0' Rt. ₵ Hwy. 17
BORING DATE January 25 - 28, 1977
BOREHOLE TYPE H.S. Augers & AX Casing, AXT Rock Core

ORIGINATED BY AEL
COMPILED BY RAH
CHECKED BY RGC

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE		SAMPLES			GROUND WATER				REMARKS
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES		ELEV	UNIT WEIGHT γ		% GR SA SI CL
860.1	Ground Level								
0.0	Asphalt								
0.9	Fill Sand and gravel Very dense to loose grey brown	1	SS	51		855			
		2	SS	6		850			
		3	SS	7		845			
		4	SS	4		840			
838.8 21.3	Fill Silty clay/clayey silt Stiff grey & brown	5	TW	PM		835			
		6	TW	PM		830			
828.1 32.0	Silty clay/clayey silt Varved Stiff to firm brown	7	TW	PM		825			
819.1 41.0	Silty clay/clayey silt Varved Firm grey	8	TW	PM		820			
		9	TW	PM		815			
813.1 47.0	Silty sand/sandy silt with occasional gravel Very dense grey	10	SS	49/7"		810			
806.1 54.0	Bedrock Note: Medium to fine grained grey,hard quartzite. Joint spacing close to very close.	11	AXT RC	94%		805			RQD 28%
		12	AXT RC	100%					
		13	AXT RC	82%					RQD 65%
		14	AXT RC	69%					RQD 47%
		15	AXT RC	92%		800			RQD 43%
797.5 62.6	End of Borehole Note: W.L. Not Established								



15 ◇ 5 % STRAIN AT FAILURE
20 / 10

OFFICE REPORT ON SOIL EXPLORATION

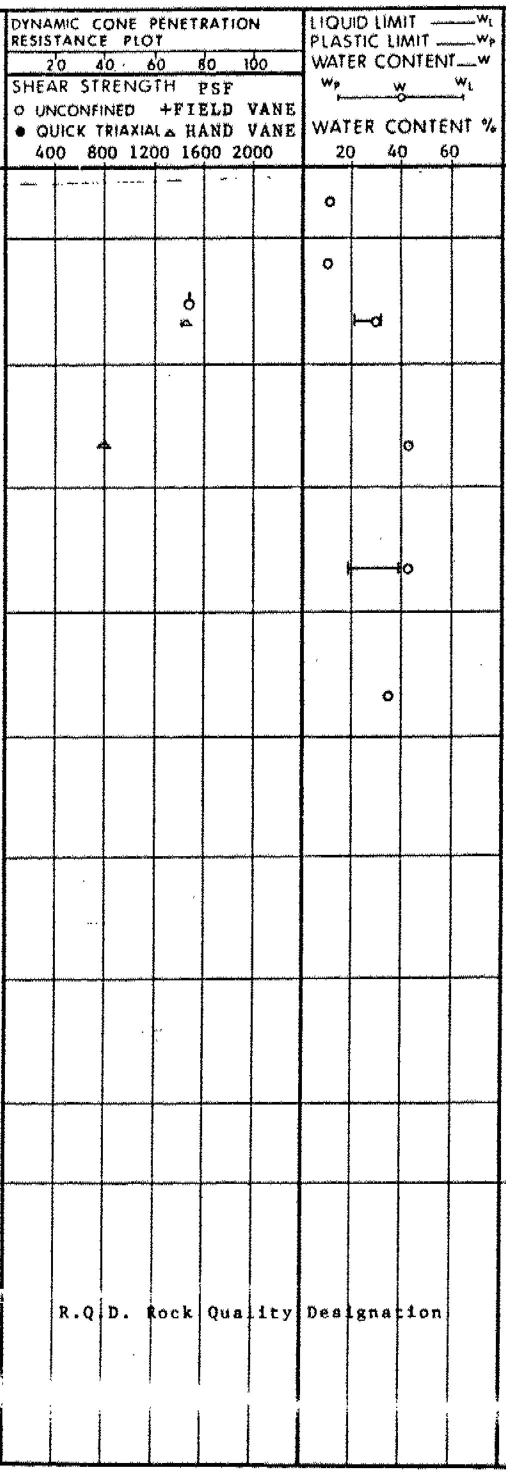
MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 7

WP 158-74-01
LOCATION Sta. 102 + 78 o/s 47.5 Rt. ¢ Hwy. 17
ORIGINATED BY AEL
DIST 17 HWY 17
BORING DATE December 15 & 16, 1976.
COMPILED BY RAH
DATUM Geodetic
BOREHOLE TYPE BX Casing, AX Rock Core
CHECKED BY RGC

SOIL PROFILE		SAMPLES			GROUND WATER	UNIT WEIGHT	REMARKS
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	ELEV	γ	% GR SA SI CL
832.8	Ground Level						
0.0 829.8	Silty sand with gravel Loose brown	1	SS	6	▼830		
3.0	Silty clay/clayey silt Varved Stiff to firm brown	2	SS	9			
		3	TW	PM		120	4 15 59 22
					825		
		4	TW	PM			
819.8 13.0	Silty clay/clayey silt Varved Firm grey				820		
		5	TW	PM			0 6 54 40
					815		
		6	TW	PM			0 2 80 18
809.8 23.0	Silty sand/sandy silt with occasional gravel Loose grey				810		
		7	SS	6			0 52 44 4
					805		
802.7 30.1	Bedrock Note: Medium to fine grained grey, hard quartzite. Joint spacing close to very close. Core generally fractured.	8	AX RC	97%			RQD 50%
					800		
		9	AX RC	100%			RQD -
		10	AX RC	83%			RQD 17%
					795		
791.8		11	AX RC	93%			RQD 40%
41.0	End of Borehole						



R.Q.D. Rock Quality Designation

15 ◇ 5 (20, 10) % STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS - ONTARIO

HIGHWAY ENGINEERING DIVISION - ENGINEERING MATERIALS OFFICE - SOIL MECHANICS SECTION

RECORD OF BOREHOLE Nº 8

WP 158-74-01
DIST 17 HWY 17
DATUM Geodetic
LOCATION Sta. 103 + 88 o/s 11.5' Rt. ¢ Hwy. 17
BORING DATE January 25 - 26, 1977
BOREHOLE TYPE BX & AX Casing, AXT & BX Rock Core
ORIGINATED BY AEL
COMPILED BY RAH
CHECKED BY RGC

SOIL PROFILE			SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	LIQUID LIMIT ——w_L PLASTIC LIMIT ——w_P WATER CONTENT —— w w_P —— w —— w_L	UNIT WEIGHT	REMARKS
ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER ELEV	400 800 1200 1600 2000	WATER CONTENT % 20 40 60	γ	% GR SA SI CL
861.2	Ground Level									
0.3	Asphalt		1	WS	-	860				
	Fill Sand and gravel Compact to very dense grey brown		2	SS	70/6"					
			3	BX RC	50%	855				
	Note:		4	BX RC	50%					
	Cobbles and boulders up to 6 inches in size encountered from 3 to 9 feet. BXCA drilled from 0.8 to 16 feet. AXCA drilled from 16 to 22 feet.		5	SS	15	850				
			6	SS	59	845				
			7	SS	24	840				
839.2 22.0	Fill Rock		8	AXT RC	65%					
	Note:		9	AXT RC	75%	835				
	Rock up to 18 inches encountered AXCA drilled to 31 feet.		10	AXT RC	67%					
830.2 31.0	Silty sand/sandy silt (probable) with occasional gravel					830				
			11	AXT RC	15%					
825.1 36.1	Bedrock		12	AXT RC	100%	825				RQD 63%
	Note: Medium to fine grained grey, hard quartzite. Joint spacing close. Loss of return water 38.5 to 43.5 feet.		13	AXT RC	97%	820				RQD 81%
	Core generally sound.		14	AXT RC	100%	815				RQD 75%
813.0 48.2	End of Borehole Note: W.L. Not Established						R.Q.D. Rock Quality Designation			

20
15 ◇ 5 % STRAIN AT FAILURE
10

APPENDIX B

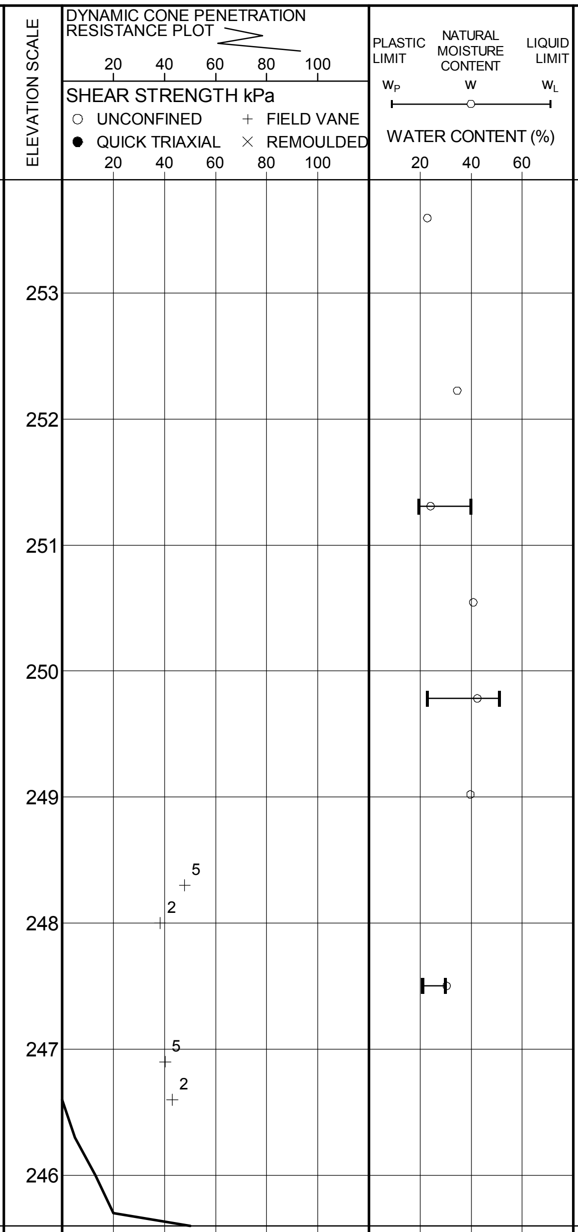
Borehole Records – Golder 2016

RECORD OF BOREHOLE No BH1

1 OF 1 **METRIC**

PROJECT 14-1181-0014			
G.W.P.	LOCATION N 5149845.7; E 318819.3		ORIGINATED BY DM
DIST	HWY 17	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers	COMPILED BY TB
DATUM GEODETIC		DATE January 18, 2016	CHECKED BY SEMP

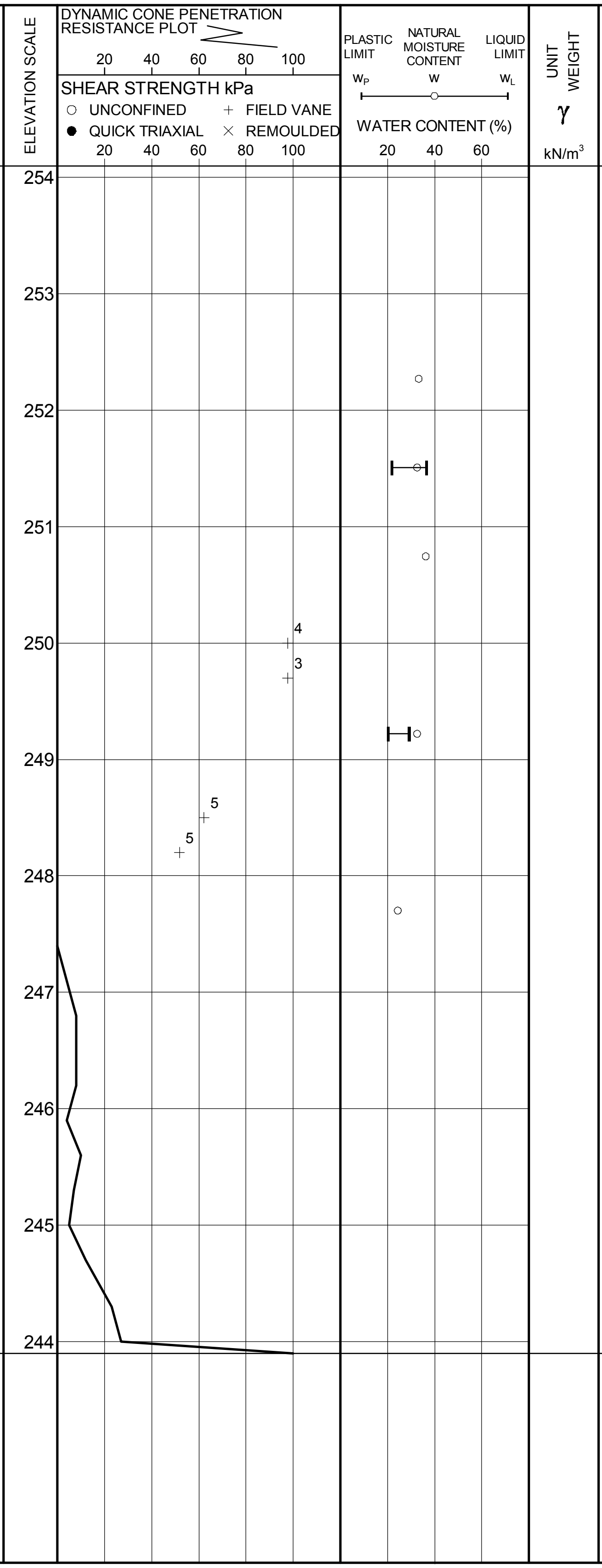
SOIL PROFILE			SAMPLES					REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
253.9	GROUND SURFACE										
0.0	Gravelly silty sand, some cobbles (FILL) Compact/stiff below frozen material Brown Frozen to wet		1	SS	46						
	- 75 mm thick silty clay layer encountered at 0.8 m depth.		2	SS	13						
252.5 1.4	- 125 mm thick silty clay layer encontered at 1.2 m depth.					∇					
	ORGANIC CLAY, some sand, trace fibrous peat Very stiff Grey to black Moist		3 A 3 B	SS	23						
251.9 2.2	Sandy SILT, trace gravel Grey Wet										
	CLAYEY SILT to CLAY, trace to some sand, varved Firm to stiff Brown/grey Moist to wet		4	SS	9			0	6	66	28
			5	SS	6						
			6	SS	4						
			7	SS	2						
	Becoming grey below 6.1 m depth.		8	SS	WH						
246.6 7.3	END OF BOREHOLE START OF DCPT										
245.6 8.3	END OF DCPT DCPT REFUSAL (50 blows/0.08m) Note(s): 1. Water level at a depth of 1.5 m below ground surface (Elev. 252.4 m) upon completion of drilling.										



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

PROJECT 14-1181-0014	**RECORD OF BOREHOLE No BH2**	1 OF 1	**METRIC**
G.W.P.	LOCATION N 5149825.5; E 318800.2		ORIGINATED BY DM
DIST HWY 17	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers		COMPILED BY TB
DATUM GEODETIC	DATE January 19, 2016		CHECKED BY SEMP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
254.1	GROUND SURFACE								
0.0	Gravelly silty sand (FILL) Compact Brown Frozen	1	SS	11					
253.4 0.7	Sandy SILT, some gravel, trace organics Loose Brown-grey to black Moist to wet	2	SS	9					
252.7 1.4	CLAYEY SILT to SILTY CLAY, trace sand, varved Firm to stiff Brown/grey Moist to wet	3	SS	6					
		4	SS	8		0	1	70	29
					∇				
		5	SS	4					
	Becoming grey below 4.3 m depth.								
		6	SS	1					
248.1 6.0	SILT and SAND, trace clay Very loose Brown/grey Wet	7	SS	4		0	65	33	2
247.4 6.7	END OF BOREHOLE START OF DCPT								
243.9 10.2	END OF DCPT DCPT REFUSAL (100 blows/0.18 m) Note(s): 1. Water level at a depth of 3.1 m below ground surface (Elev. 251.0 m) upon completion of drilling.								



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

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

RECORD OF BOREHOLE No BH3

1 OF 1 **METRIC**

PROJECT 14-1181-0014

G.W.P.

DIST

HWY 17

DATUM GEODETIC

LOCATION N 5149846.5; E 318844.3

BOREHOLE TYPE NW Casing, Portable Equipment

DATE January 19, 2016

ORIGINATED BY DM

COMPILED BY TB

CHECKED BY SEMP

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
254.0	GROUND SURFACE
0.0	Gravelly silty sand, trace organics, trace cobbles (FILL) Loose Black to brown Wet
253.3 0.7	CLAYEY SILT, trace sand, trace gravel, trace organics Very stiff Brown to grey Wet
251.8 2.2	CLAY, varved Firm to stiff Brown/grey Moist to wet Trace organics encountered in Sample 4. Becoming grey below 5.8 m depth.
246.8 7.2	END OF BOREHOLE START OF DCPT
245.3 8.7	END OF DCPT DCPT REFUSAL (50 blows/0.15 m)

Note(s):

1. Water level at a depth of 0.6 m below ground surface (Elev. 253.4 m) upon completion of drilling and maybe influenced by introduction of drilling water.

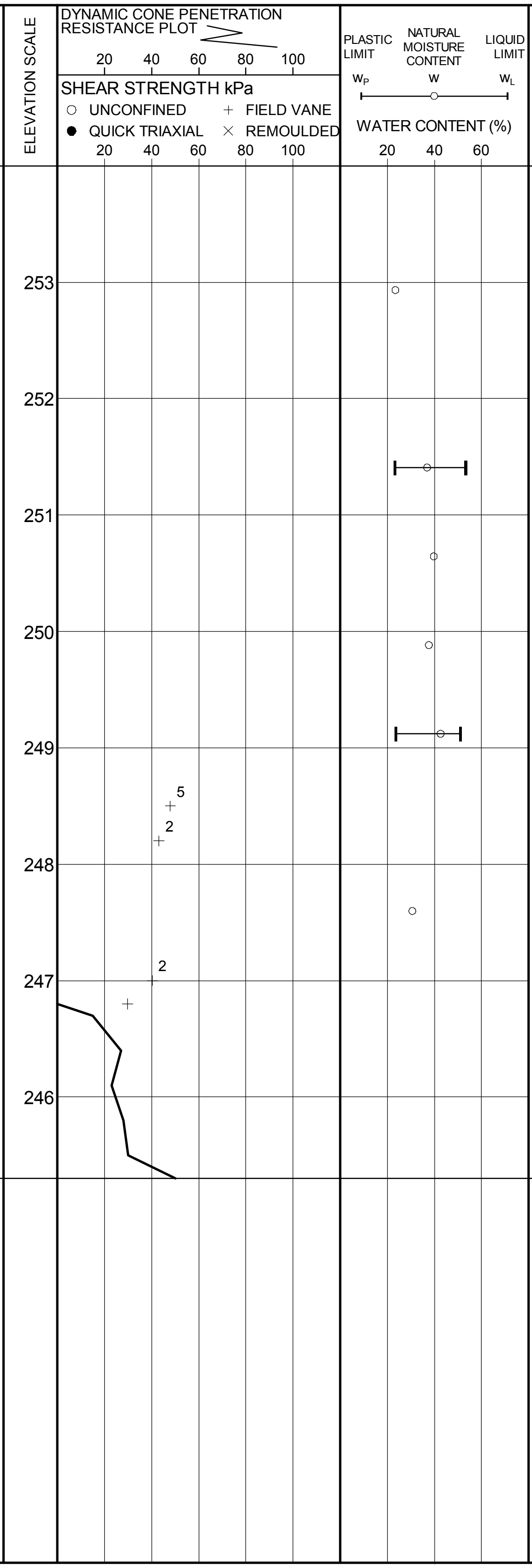
SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	7				
2	SS	17	0	3	82	15
3	SS	20				
4	SS	8				
5	SS	6				
6	SS	4				
7	SS	3	0	0	49	51
8	SS	WH				

GROUND WATER CONDITIONS: ∇ (water level at Elev. 253.4)

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)



+3, ×3: Numbers refer to Sensitivity

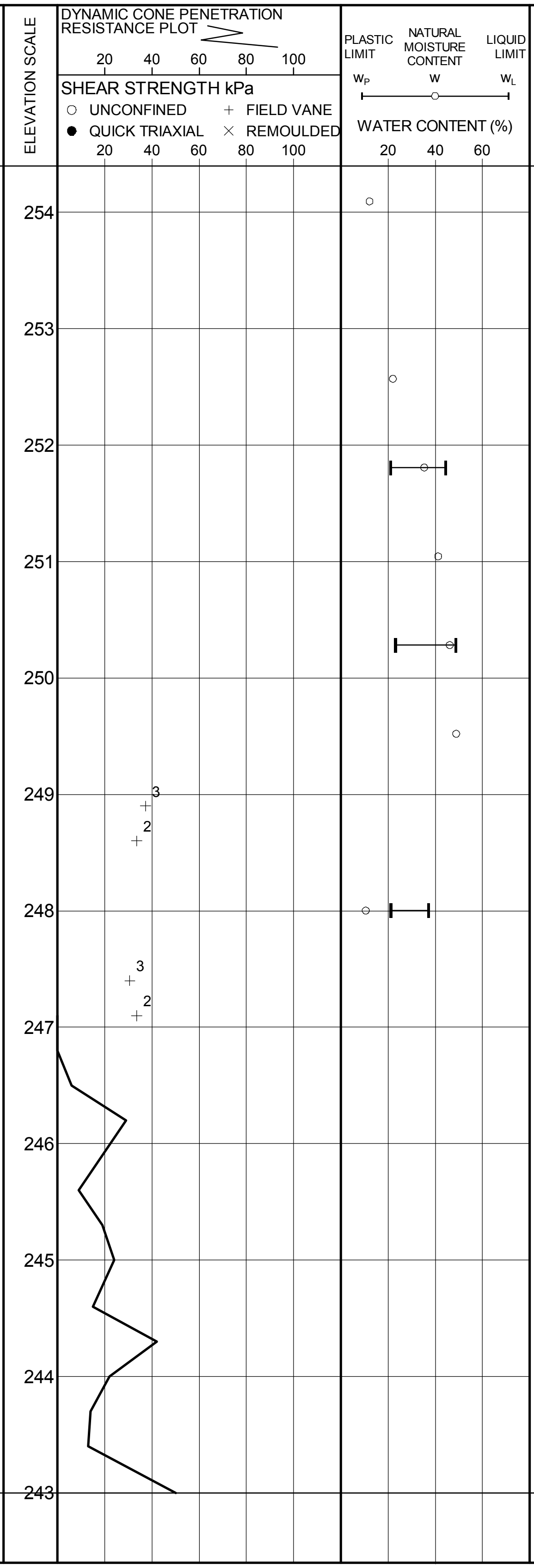
○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No BH4

1 OF 2 **METRIC**

PROJECT 14-1181-0014
G.W.P.
DIST HWY 17
DATUM GEODETIC
LOCATION N 5149824.0; E 318822.5
BOREHOLE TYPE NW Casing, Portable Equipment
DATE January 20, 2016
ORIGINATED BY DM
COMPILED BY TB
CHECKED BY SEMP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
254.4	GROUND SURFACE										
0.0	Sand and gravel, some fines, trace organics, some cobbles (FILL) Loose Dark brown to brown Frozen to wet		1	SS	58	∇		38	49	(13)	
			2	SS	8						
252.9 1.5	SILTY CLAY, trace sand, varved Firm to stiff Brown/grey Moist to wet		3	SS	8						
			4	SS	7						
			5	SS	6						
			6	SS	2						
	Becoming grey below 4.6 m depth.		7	SS	1						
			8	SS	1						
247.1 7.3	END OF BOREHOLE START OF DCPT										
243.0 11.4											



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

SUD-MTO 001 1411810014 CONISTON A15.GPJ GAL-MISS.GDT 15/04/16 DATA INPUT:

RECORD OF BOREHOLE No BH4

2 OF 2 **METRIC**

PROJECT 14-1181-0014
G.W.P.
LOCATION N 5149824.0; E 318822.5
ORIGINATED BY DM
DIST HWY 17
BOREHOLE TYPE NW Casing, Portable Equipment
COMPILED BY TB
DATUM GEODETIC
DATE January 20, 2016
CHECKED BY SEMP

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"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 (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	END OF DCPT DCPT REFUSAL (50 blows/0.15 m) Note(s): 1. Water level at a depth of 0.3 m below ground surface (Elev. 254.1 m) upon completion of drilling and maybe influenced by introduction of drilling water.										

SUD-MTO 001 1411810014 CONISTON A15.GPJ GAL-MISS.GDT 15/04/16 DATA INPUT:

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

APPENDIX C

Current Investigation – Borehole Records

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

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[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 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

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
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, 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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_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 $= \sigma'_p / \sigma'_{vo}$

(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 $= \tan\delta$
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes:

1 $\tau = c' + \sigma' \tan\phi'$

2 shear strength = (compressive strength)/2

---

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

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^{o} 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
FLT	Fault
SH	Shear
VN	Vein
FR	Fracture
SY	Stylolite
BD	Bedding
FO	Foliation
CO	Contact
AXJ	Axial Joint
KV	Karstic Void
MB	Mechanical Break

PL	Planar
CU	Curved
UN	Undulating
IR	Irregular
K	Slickensided
PO	Polished
SM	Smooth
SR	Slightly Rough
RO	Rough
VR	Very Rough

Golder Associates

RECORD OF BOREHOLE No C17-1

1 OF 1 **METRIC**

PROJECT 1651997			
W.P. 5165-10-01	LOCATION N 5149846.8; E 318881.4 (LAT. 46.488138; LONG. -80.81658)		ORIGINATED BY SA
DIST	HWY 17	BOREHOLE TYPE 2" Hilti Core	COMPILED BY AC
DATUM GEODETIC	DATE April 26, 2017		CHECKED BY SEMP

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
260.2	BEDROCK OUTCROP										
0.0	ARKOSIC GREYWACKE (BEDROCK) For coring details see Record of Drillhole C17-1.		1	RC	REC 93%		260				RQD = 18%
			2	RC	REC 100%		259				RQD = 31%
			3	RC	REC 100%		258				RQD = 67%
			4	RC	REC 75%						RQD = 46%
257.2 3.0	END OF BOREHOLE Note: 1. Borehole dry upon completion of coring.										

SUD-MTO 001 LAT/LONG 1651997.GPJ GAL-MISS.GDT 05/07/17 DATA INPUT:

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

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

PROJECT: 1651997

RECORD OF DRILLHOLE: C17-1

SHEET 1 OF 1

LOCATION: N 5149846.8 ;E 318881.4 (LAT. 46.488138; LONG. -80.81658)

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 26, 2017

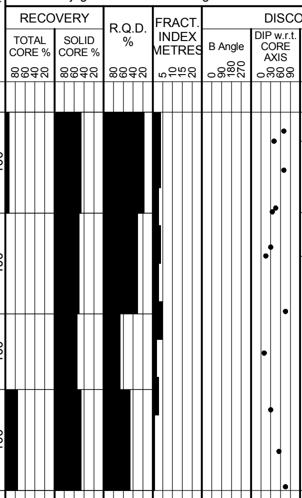
DRILL RIG: Hilti

DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q' AVG.	
		BEDROCK OUTCROP													
0	2" Core Bit Manual Hilti	ARKOSIC GREYWACKE Strong Slightly weathered Very fine grained Grey Oxidized joints throughout. Weathered between 0.7 m and 0.8 m depth. Weathered, missing core between 2.7 m and 3.0 m depth.		260.2 0.0	1		Grey 100	BDPORo BDPOK BDPORo JNPLRo BDPOK							
1					2		Grey 100	BDPOK BDPOSM JNIRRo							UCS = 91 MPa
2					3		Grey 100	MB JNIRRo							
					4		Grey 100	JNUNRo JNIRRo JNIRRo							
3		END OF DRILLHOLE		257.2 3.0											
4															
5															
6															
7															
8															
9															
10															
11															
12															



DEPTH SCALE

1 : 60

Golder Associates

LOGGED: SA

CHECKED: SEMP

Golder Associates

RECORD OF BOREHOLE No C17-2

1 OF 1 **METRIC**

PROJECT 1651997

W.P. 5165-10-01

LOCATION N 5149856.2; E 318871.2 (LAT. 46.488223; LONG. -80.816713)

ORIGINATED BY SA

DIST ____ HWY 17

BOREHOLE TYPE 2" Hilti Core

COMPILED BY AC

DATUM GEODETIC

DATE April 28 and May 1, 2017

CHECKED BY SEMP

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"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 (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
254.0	BEDROCK OUTCROP										
0.0	ARKOSIC GREYWACKE (BEDROCK) For coring details see Record of Drillhole C17-2.	▨	1	RC	REC 100%						RQD = 22%
		▨	2	RC	REC 100%		253				RQD = 69%
		▨	3	RC	REC 100%		252				RQD = 64%
251.0		▨	4	RC	REC 100%						RQD = 69%
3.0	END OF BOREHOLE Note: 1. Borehole dry upon completion of coring.						251				

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

SUD-MTO 001 LAT/LONG 1651997.GPJ GAL-MISS.GDT 05/07/17 DATA INPUT:

PROJECT: 1651997

RECORD OF DRILLHOLE: C17-2

SHEET 1 OF 1

LOCATION: N 5149856.2 ;E 318871.2 (LAT. 46.488223; LONG. -80.816713)

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 28 and May 1, 2017

DRILL RIG: Hilti

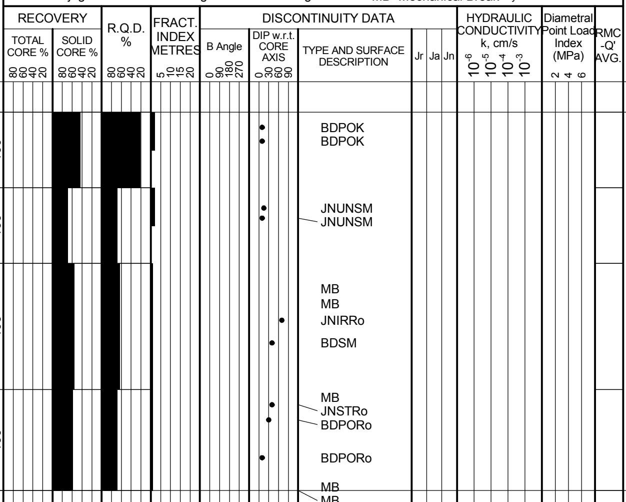
DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	COLOUR / % RETURN (FLUSH)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	
		BEDROCK OUTCROP	254.0				
0	2" Core Bit Manual Hilti	ARKOSIC GREYWACKE Very strong Fresh Very fine grained Grey Joint with silt fil at a depth of 0.8 m. Oxidized joints to 2.4 m depth, then partially healed quartz carbonate joints. Heavily jointed zone from between 2.6 m and 2.7 m depth.	0.0	1	Grey 100	BDPOK BDPOK	
				2	Light Grey 100	JNUNSM JNUNSM	
1				3	Light Grey 100	MB MB JNIRRo BDSM	UCS = 142 MPa
2				4	Light Grey 100	MB JNSTRo BDPORo BDPORo	
3		END OF DRILLHOLE	251.0 3.0			MB MB	
4							
5							
6							
7							
8							
9							
10							
11							
12							



DEPTH SCALE

1 : 60

Golder Associates

LOGGED: SA

CHECKED: SEMP

RECORD OF BOREHOLE No C17-3

1 OF 3 **METRIC**

PROJECT 1651997

W.P. 5165-10-01

DIST HWY 17

DATUM GEODETIC

LOCATION N 5149852.7; E 318815.9 (LAT. 46.488192; LONG. -80.817433)

BOREHOLE TYPE NW Casing, Wash Boring and NQ Coring

DATE April 25, 2017

ORIGINATED BY SA

COMPILED BY AC

CHECKED BY SEMP

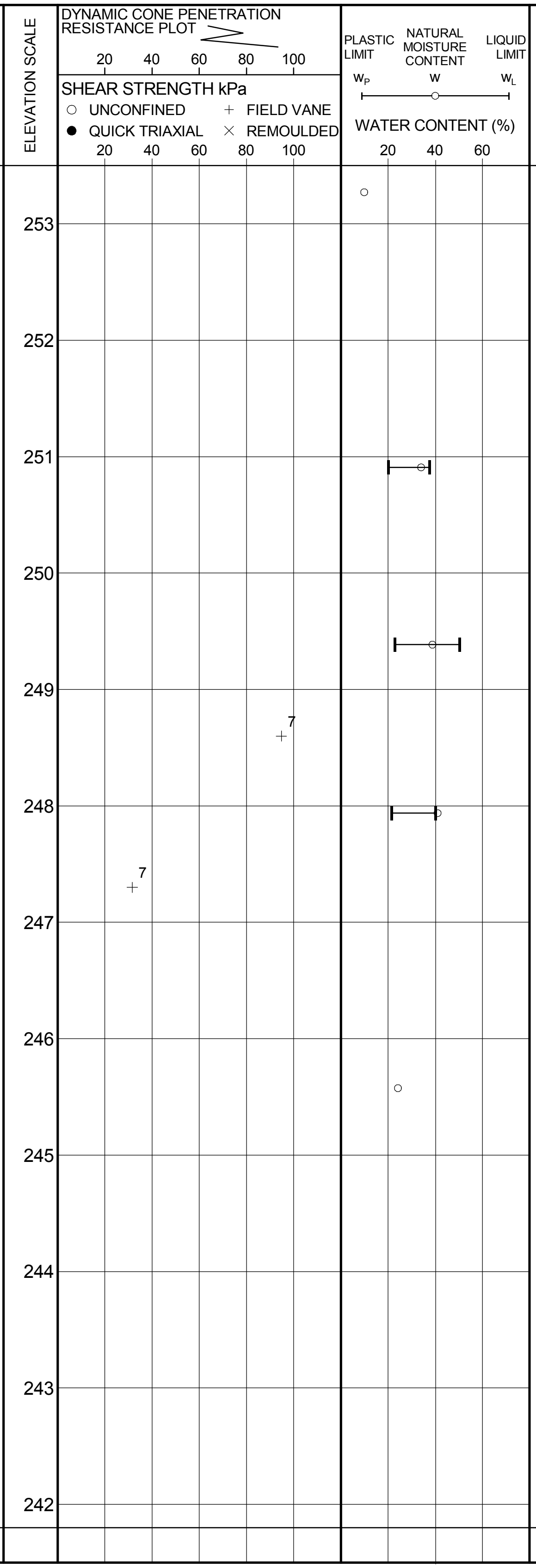
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
253.5	GROUND SURFACE
0.0 0.1	PEAT (Fibrous) Brown Moist
252.9 0.6	Sand and gravel, trace to some silt (FILL) Very dense Brown Moist 75 mm cobble at 0.45 m depth
	Silty sand (FILL) Dense Brown Wet
251.5 2.0	SILTY CLAY to CLAY, trace gravel, trace sand Firm to stiff Brown to grey Wet
246.8 6.7	Sandy SILT, trace clay Very loose Grey Wet A 110 mm cobble was encountered at 8.4 m depth
244.9 8.6	ARKOSIC GREYWACKE (BEDROCK) For coring details see Record of Drillhole C17-3.
241.8 11.7	END OF BOREHOLE

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	67/0.16		43	43	11	3
2	SS	31					
3	SS	35					
4	SS	6		0	2	57	41
5	TO	PH					
6	SS	4		1	1	50	48
7	TO	PH	18.7				
8	SS	1		0	30	66	4
1	RC	REC 100%		RQD = 70%			
2	RC	REC 93%		RQD = 81%			
3	RC	REC 97%		RQD = 55%			
4	RC	REC 100%		RQD = 45%			

GROUND WATER CONDITIONS: ▽ (at approx. elev. 251.4)



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No C17-3

2 OF 3 **METRIC**

PROJECT 1651997			
W.P. 5165-10-01	LOCATION N 5149852.7; E 318815.9 (LAT. 46.488192; LONG. -80.817433)		ORIGINATED BY SA
DIST	HWY 17	BOREHOLE TYPE NW Casing, Wash Boring and NQ Coring	COMPILED BY AC
DATUM GEODETIC		DATE April 25, 2017	CHECKED BY SEMP

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"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 (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Note: 1. Water level at a depth of 2.1 m below ground surface (Elev. 251.4 m) upon completion of drilling.										

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

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

PROJECT: 1651997

RECORD OF DRILLHOLE: C17-3

SHEET 3 OF 3

LOCATION: N 5149852.7 ;E 318815.9 (LAT. 46.488192; LONG. -80.817433)

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 25, 2017

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

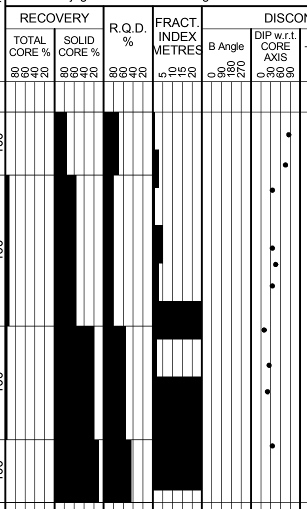
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	COLOUR / % RETURN (FLUSH)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	
		REFER TO PREVIOUS PAGE		244.9				
9	CME 55 NW	ARKOSIC GREYWACKE Strong Slightly weathered Very fine grained Grey		8.6	1	Grey 100	JNUNRo JNUNRo	
10	CME 55 NQ	Sand Seams at 10.3 m and 10.7 m depth. Heavily jointed between 10.3 m and 10.7 m depth. Partially healed quartz carbonate joints			2	Grey 100	BDPOSM BDPOSM JNUNRo BDPOK	UCS = 87 MPa
11					3	Grey 100	BDPOK BDPOK BDPOK	
				241.8 11.7	4	Grey 100	BDPOK	
12		END OF DRILLHOLE						
13								
14								
15								
16								
17								
18								
19								
20								



HYDRAULIC CONDUCTIVITY k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) | Jr | Ja | Jn | Diametral Point Load Index (MPa) (2 4 6) | RMC -Q' AVG.

DEPTH SCALE

1 : 60

LOGGED: SA

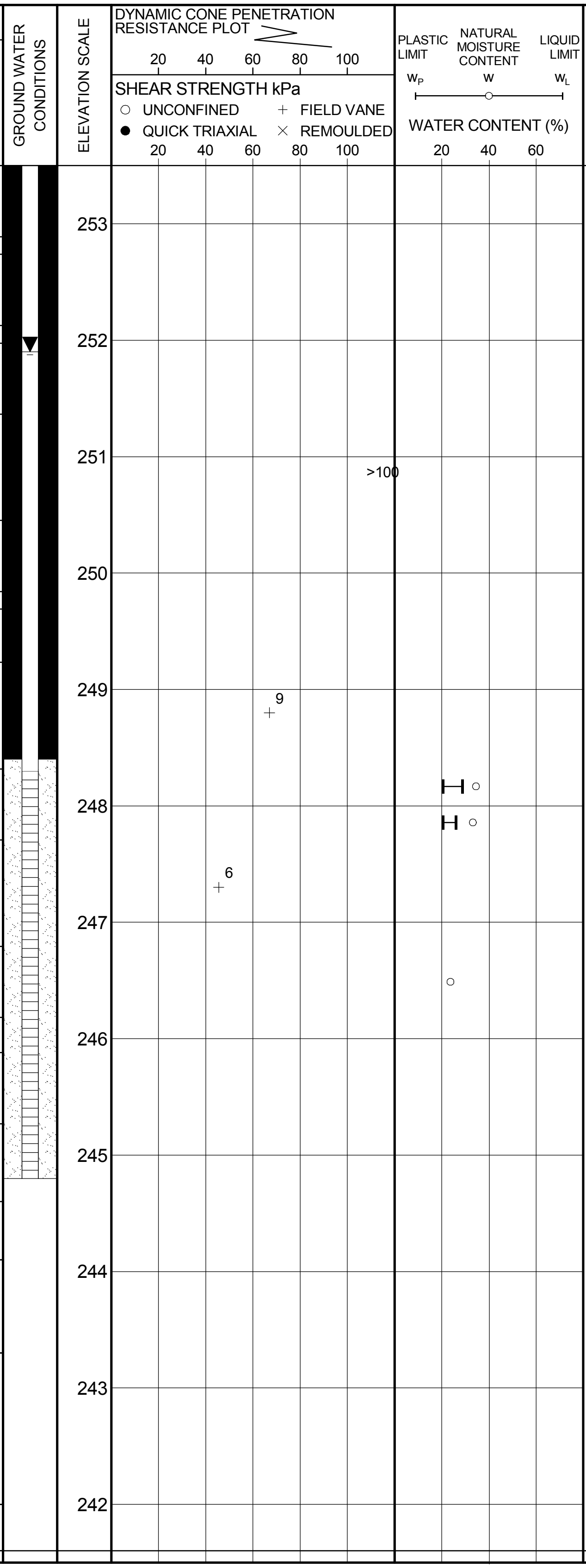
CHECKED: SEMP

RECORD OF BOREHOLE No C17-4

1 OF 3 **METRIC**

PROJECT 1651997			
W.P. 5165-10-01	LOCATION N 5149860.2; E 318800.5 (LAT. 46.48826; LONG. -80.817633)	ORIGINATED BY SA	
DIST	HWY 17	BOREHOLE TYPE NW Casing, Wash Boring and NQ Coring	COMPILED BY AC
DATUM GEODETIC	DATE April 24, 2017	CHECKED BY SEMP	

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
253.5	GROUND SURFACE									
0.0	Sandy TOPSOIL									
0.1	Sand and gravel, trace organics (FILL) Compact Brown Moist	1	SS	15						
252.8 0.7	Clayey silt, some sand, some gravel (FILL) Firm Brown Wet	2	SS	4						
251.9 1.6	CLAYEY SILT, trace sand Firm to very stiff Brown to grey Wet	3	SS	7						
					>100					
	Varved below 3.0 m depth	4	SS	4						
		5	TO	PH						
					9					
		6A	SS	WH			0	0	75	25
	Shelby tube sample taken in adjacent borehole at 5.5 m depth (Elev. 248.0 m) for consolidation testing	6B				18.0				
					6					
247.2 6.3	SILT and SAND, trace gravel, trace clay Loose to compact Grey Wet	7	SS	4		NP	2	46	47	5
		8	SS	12						
244.6 8.9	META QUARTZITE (BEDROCK) For coring details see Record of Drillhole C17-4.	1	RC	REC 100%			RQD = 100%			
		2	RC	REC 100%			RQD = 91%			
		3	RC	REC 100%			RQD = 81%			
241.6		4	RC	REC 100%			RQD = 100%			



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

SUD-MTO 001 LAT/LONG 1651997.GPJ GAL-MISS.GDT 05/07/17 DATA INPUT:

RECORD OF BOREHOLE No C17-4

2 OF 3 **METRIC**

PROJECT 1651997

W.P. 5165-10-01

LOCATION N 5149860.2; E 318800.5 (LAT. 46.48826; LONG. -80.817633)

ORIGINATED BY SA

DIST HWY 17

BOREHOLE TYPE NW Casing, Wash Boring and NQ Coring

COMPILED BY AC

DATUM GEODETIC

DATE April 24, 2017

CHECKED BY SEMP

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- *CONTINUED FROM PREVIOUS PAGE* ---										
11.9	END OF BOREHOLE Notes: 1. Water level at a depth of 1.2 m below ground surface (Elev. 252.3 m) upon completion of drilling. 2. An additional shelby tube was obtained 2 m northeast of borehole at 5.2 m depth for consolidation testing. 3. Water level in piezometer mesured at a depth of 1.6 m below ground surface (Elev. 251.9 m) on April 27, 2017 and on July 4, 2017.										

SUD-MTO 001 LAT/LONG 1651997.GPJ GAL-MISS.GDT 05/07/17 DATA INPUT:

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

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

PROJECT: 1651997

RECORD OF DRILLHOLE: C17-4

SHEET 3 OF 3

LOCATION: N 5149860.2 ;E 318800.5 (LAT. 46.48826; LONG. -80.817633)

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 24, 2017

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

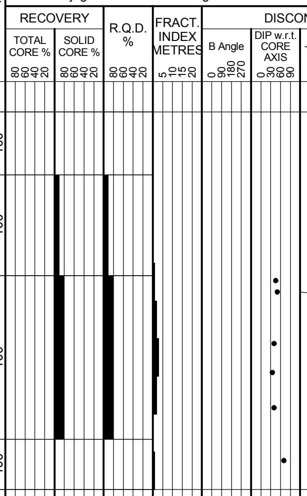
DATUM: GEODETIC

JN - Joint	BD - Bedding	PL - Planar	PO - Polished
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	COLOUR / % RETURN	FLUSH	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	
		REFER TO PREVIOUS PAGE	244.6					
9	CME 55 HQ	META QUARTZITE Very strong Fresh Very fine grained Grey-black Partially healed quartz carbonate filled joints	8.9	1	Grey 100			UCS = 151 MPa
	NQ			2	Grey 100			
10								
				3	Grey 100		JNUNRo BDPORo JNUNRo BDPORo BDPORo	
11								
			241.6	4	Grey 100		BDPORo	
12		END OF DRILLHOLE	11.9					
13								
14								
15								
16								
17								
18								
19								
20								



Jr Ja Jn

HYDRAULIC CONDUCTIVITY k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}

Diametral Point Load Index (MPa) 2 4 6

RMC -Q' AVG.

SUD-RCK LAT/LONG 1651997.GPJ GAL-MISS.GDT 05/07/17 DATA INPUT:

DEPTH SCALE

1 : 60

Golder Associates

LOGGED: SA

CHECKED: SEMP

RECORD OF BOREHOLE No C17-5

1 OF 2 **METRIC**

PROJECT 1651997

W.P. 5165-10-01

DIST

HWY 17

DATUM GEODETIC

LOCATION N 5149839.0; E 318787.1 (LAT. 46.48807; LONG. -80.817809)

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers and NW Casing

DATE April 19, 2017

ORIGINATED BY SA

COMPILED BY AC

CHECKED BY SEMP

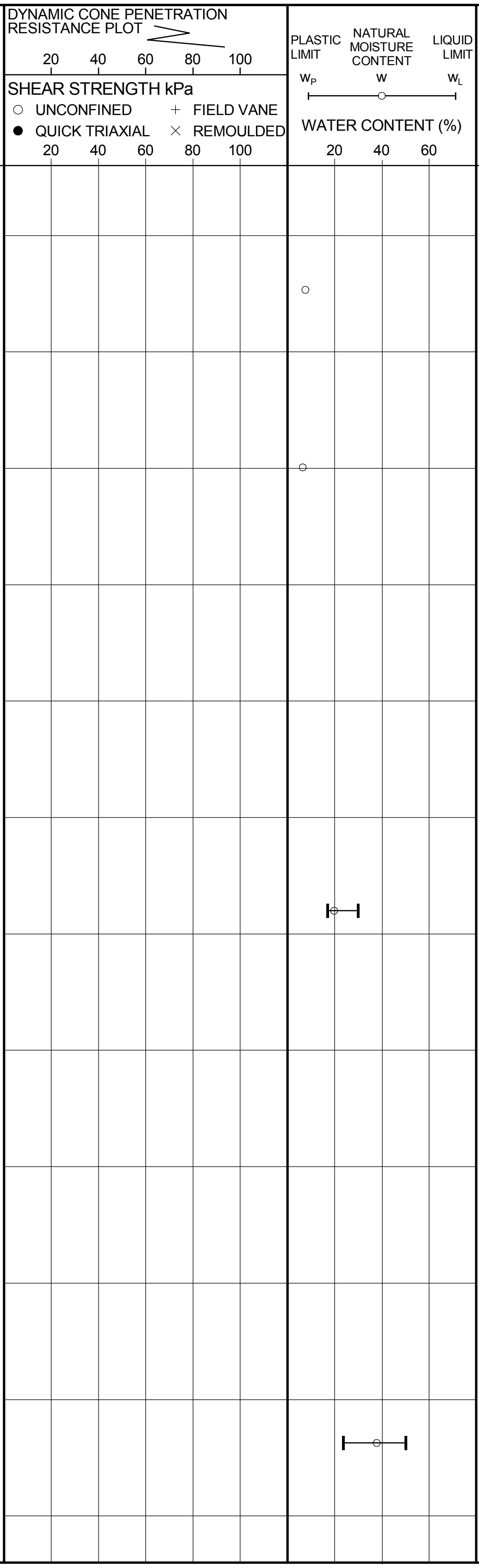
ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
262.6	GROUND SURFACE							
0.0	ASPHALT (100 mm)							
262.2	CONCRETE (250 mm)							
0.4	Sand, trace to some gravel, trace to some silt (FILL) Compact to very dense Brown Moist							
		1	SS	24	8	84	(8)	
		2	SS	19				
		3	SS	25	19	71	(10)	
		4	SS	34				
		5	SS	52				
258.2 4.4	Clayey silt with sand, trace gravel (FILL) Stiff to very stiff Brown Moist to wet							
		6	SS	20				
		7	SS	11	0	20	55	25
		8	SS	11				
253.3 9.3	Sand, trace to some gravel (FILL) Compact Brown Wet	9	SS	24				
252.4 10.2	CLAY Very stiff Brown to grey Wet							
		10	SS	14	0	0	42	58

GROUND WATER CONDITIONS: ▽ (at approx. elevation 255.8)

ELEVATION SCALE: 262, 261, 260, 259, 258, 257, 256, 255, 254, 253, 252, 251

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)



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+^{3}, ×^{3}: Numbers refer to Sensitivity

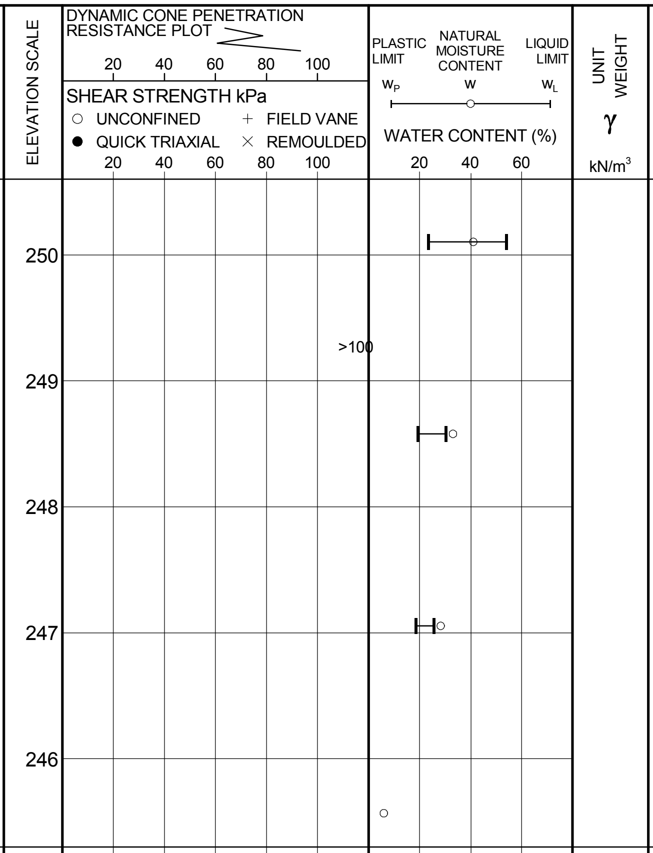
○$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No C17-5

2 OF 2 **METRIC**

PROJECT 1651997			
W.P. 5165-10-01	LOCATION N 5149839.0; E 318787.1 (LAT. 46.48807; LONG. -80.817809)		ORIGINATED BY SA
DIST	HWY 17	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers and NW Casing	COMPILED BY AC
DATUM GEODETIC		DATE April 19, 2017	CHECKED BY SEMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	CLAY Very stiff Brown to grey Wet		11	SS	6						
249.3 13.3	CLAYEY SILT, trace sand, silt seams throughout Very stiff to firm Grey Wet						>100				
			12	SS	3			0	1	72	27
			13	SS	10			0	0	76	24
246.4 16.2	SAND and GRAVEL, trace to some silt, trace clay (TILL) Dense Brown Wet Cobble encountered at 16.9 m depth.		14	SS	30			40	50	8	2
245.3 17.3	END OF BOREHOLE REFUSAL TO FURTHER CASING AND SPLIT-SPOON ADVANCEMENT Note: 1. Water level at a depth of 6.9 m below ground surface (Elev. 255.7 m) upon completion of drilling.										



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

RECORD OF BOREHOLE No C17-6

1 OF 1 **METRIC**

PROJECT 1651997

W.P. 5165-10-01

LOCATION N 5149865.0; E 318776.1 (LAT. 46.488304; LONG. -80.817951)

ORIGINATED BY SA

DIST HWY 17

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

COMPILED BY AC

DATUM GEODETIC

DATE April 20, 2017

CHECKED BY SEMP

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
253.1	GROUND SURFACE
0.0	Sandy TOPSOIL
0.1	Sand and gravel, some silt, trace organics (FILL) Loose Brown Moist
252.0 1.1	Sandy silt, trace gravel, trace organics (FILL) Loose Grey Wet
251.6 1.5	CLAYEY SILT to SILTY CLAY, trace sand, varved Soft to stiff Grey to red-grey Wet Silt seams below 7.6 m depth.
243.8 9.3	SILT and SAND, trace gravel, trace clay Compact Grey Wet Attempted spoon at 10.7 m depth, hammer bouncing.
242.4 10.7	END OF BOREHOLE AUGER AND SPLIT-SPOON REFUSAL

Note:

1. Water level at a depth of 5.9 m below ground surface (Elev. 247.2 m) upon completion of drilling.

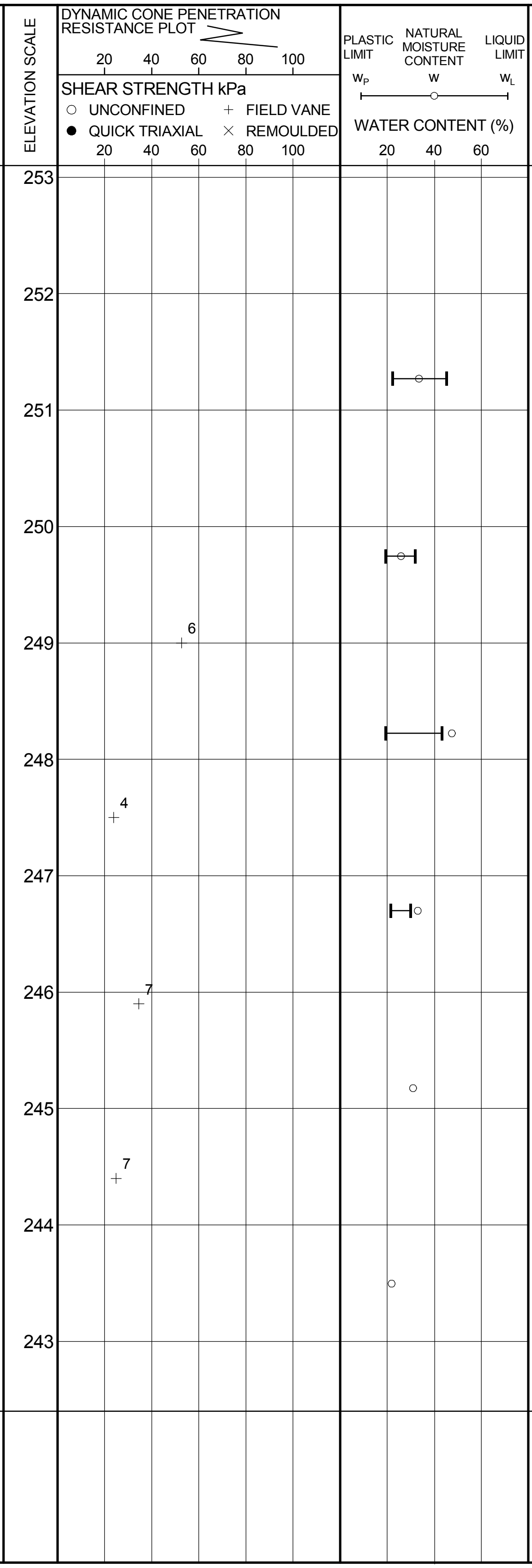
SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	7				
2	SS	6				
3	SS	7	0	3	48	49
4	SS	6				
5	SS	4	0	2	60	38
6	SS	WH				
7	SS	WH	0	0	76	24
8	SS	WH				
9A 9B	SS	15	5	41	50	4

REMARKS & GRAIN SIZE DISTRIBUTION (%)

UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS: ▽ at Elev. 247.2



+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No C17-7

1 OF 2 **METRIC**

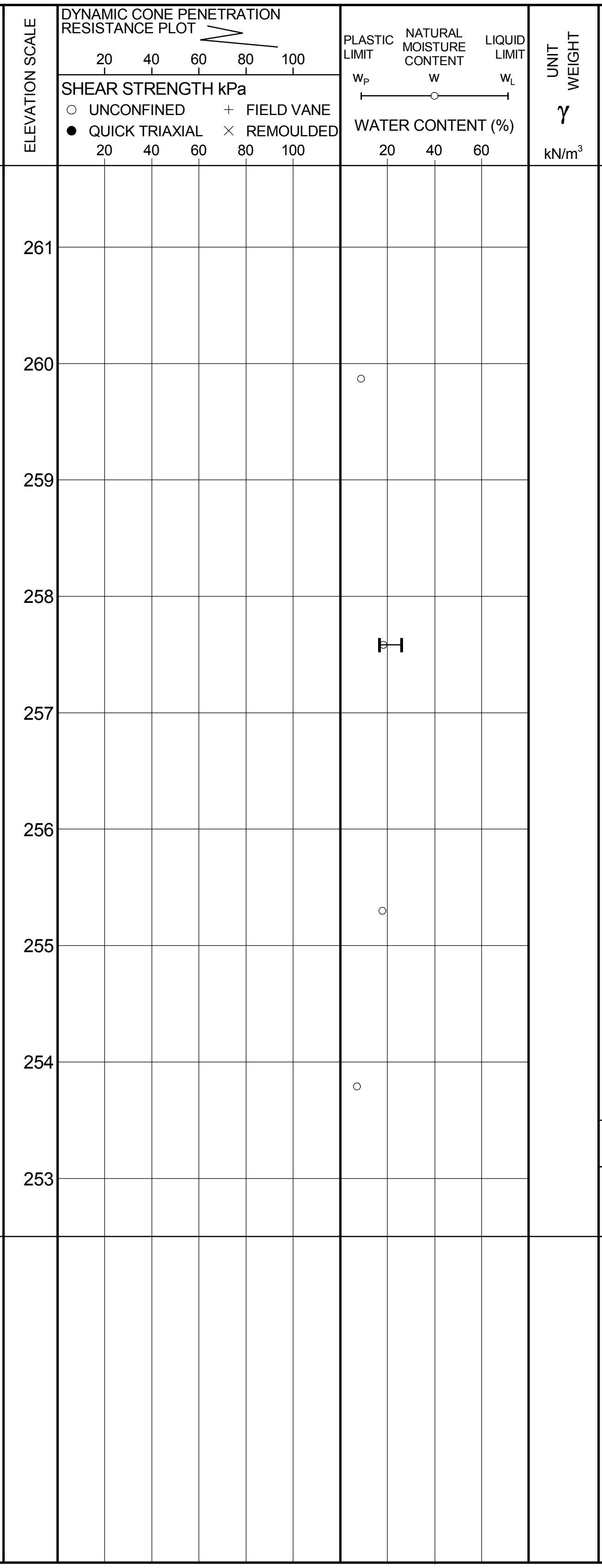
PROJECT 1651997			
W.P. 5165-10-01	LOCATION N 5149841.5; E 318720.8 (LAT. 46.488094; LONG. -80.818672)	ORIGINATED BY SA	
DIST	HWY 17	BOREHOLE TYPE NW Casing, Wash Boring and NQ Coring	COMPILED BY AC
DATUM GEODETIC	DATE April 18, 2017	CHECKED BY SEMP	

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
261.7	GROUND SURFACE
0.0	ASPHALT (100 mm)
0.1	Sand and gravel to gravelly sand, trace to some silt (FILL) Compact to very dense Brown Moist Cobbles between 0.3 m and 3.4 m depth.
258.3 3.4	Clayey silt, some sand to sandy silt (FILL) Stiff to hard Brown Moist
256.1 5.6	Silty SAND and GRAVEL, trace clay (TILL) Compact to very dense Reddish brown to grey Wet Trace organics in Sample 7 Cobbles below 6.1 m depth
253.5 8.2	ARKOSIC GREYWACKE (BEDROCK) For coring details see Record of Drillhole C17-7.
252.5 9.2	END OF BOREHOLE Note: 1. Water level at a depth of 6.6 m below ground surface (Elev. 255.1 m) upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	33		
2	SS	32		32 51 (17)
3	SS	52		
4	SS	57		
5	SS	13		0 15 59 26
6	SS	52		
7	SS	16	∇	30 38 29 3
8	SS	74		31 45 20 4
1	RC	REC 100%		RQD = 0%
2	RC	REC 100%		RQD = 0%



+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

PROJECT: 1651997

RECORD OF DRILLHOLE: C17-7

SHEET 2 OF 2

LOCATION: N 5149841.5 ;E 318720.8 (LAT. 46.488094; LONG. -80.818672)

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: April 18, 2017

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling

DATUM: GEODETIC

JN - Joint FLT - Fault SHR- Shear VN - Vein CJ - Conjugate	BD- Bedding FO- Foliation CO- Contact OR- Orthogonal CL - Cleavage	PL - Planar CU- Curved UN- Undulating ST - Stepped IR - Irregular	PO- Polished K - Slickensided SM- Smooth Ro - Rough MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.25 m (5 10 15 20)	DISCONTINUITY DATA B Angle (0 90 180 270)	DIP w.r.t. CORE AXIS (0 30 60 90)	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3})	Diametral Point Load Index (MPa) (2 4 6)	RMC -Q' AVG.
		REFER TO PREVIOUS PAGE		253.5															
	CME 55 NQ Coring NW	ARKOSIC GREYWACKE Strong Fresh Fine grained Grey-black		8.2	1	Grey 100		■	■	■			BR						
9		Highly fractured with oxidized joints		252.5 9.2	2	Grey 100		■	■	■									
		END OF DRILLHOLE																	
10																			
11																			
12																			
13																			
14																			
15																			
16																			
17																			
18																			
19																			
20																			

DEPTH SCALE

1 : 60

Golder Associates

LOGGED: SA

CHECKED: SEMP

RECORD OF BOREHOLE No C17-8

1 OF 1 **METRIC**

PROJECT 1651997			
W.P. 5165-10-01	LOCATION N 5149860.2; E 318720.0 (LAT. 46.488262; LONG. -80.818682)	ORIGINATED BY SA
DIST HWY 17	BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers	COMPILED BY AC
DATUM GEODETIC	DATE April 20, 2017	CHECKED BY SEMP

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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
255.9	GROUND SURFACE										
0.0	Sandy TOPSOIL		1	SS	10/0.2						
0.3	Gravelly Silty SAND (TILL) Compact Reddish brown										
	END OF BOREHOLE AUGER AND SPLIT-SPOON REFUSAL										

Note:

1. Borehole dry upon completion of drilling.

2. Advanced dynamic cone penetration tests 10 m west (C17-8D1) and 10 m east (C17-8D2) of borehole.

3. A bedrock outcrop noted approxymately 22 m northwest of borehole.

SUD-MTO 001 LAT/LONG 1651997.GPJ GAL-MISS.GDT 05/07/17 DATA INPUT:

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

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

RECORD OF PENETRATION TEST No C17-8D1

1 OF 1 **METRIC**

PROJECT 1651997

W.P. 5165-10-01

LOCATION N 5149860.2; E 318710.0 (LAT. 46.488262; LONG. -80.818812)

ORIGINATED BY SA

DIST ______ HWY 17

BOREHOLE TYPE Dynamic Cone Penetration Test

COMPILED BY AC

DATUM GEODETIC

DATE April 20, 2017

CHECKED BY SEMP

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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
256.2	GROUND SURFACE										
0.0							256				
0.2	END OF DCPT REFUSAL TO FURTHER PENETRATION (HAMMER BOUNCING)										

SUD-MTO 001 LAT/LONG 1651997.GPJ GAL-MISS.GDT 05/07/17 DATA INPUT:

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

RECORD OF PENETRATION TEST No C17-8D2

1 OF 1 **METRIC**

PROJECT 1651997

W.P. 5165-10-01

DIST

HWY 17

DATUM GEODETIC

LOCATION N 5149860.2; E 318730.0 (LAT. 46.488262; LONG. -80.818552)

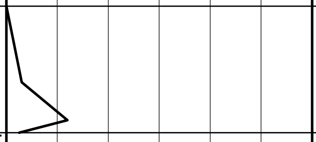
BOREHOLE TYPE Dynamic Cone Penetration Test

DATE April 20, 2017

ORIGINATED BY SA

COMPILED BY AC

CHECKED BY SEMP

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
255.0	GROUND SURFACE										
0.0											
254.0 1.0	END OF DCPT REFUSAL TO FURTHER PENETRATION (HAMMER BOUNCING)						254				

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

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

Golder Associates

RECORD OF BOREHOLE No C17-9

1 OF 1 **METRIC**

PROJECT 1651997

W.P. 5165-10-01

LOCATION N 5149857.5; E 318671.9 (LAT. 46.488239; LONG. -80.819309)

ORIGINATED BY SA

DIST ____ HWY 17

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

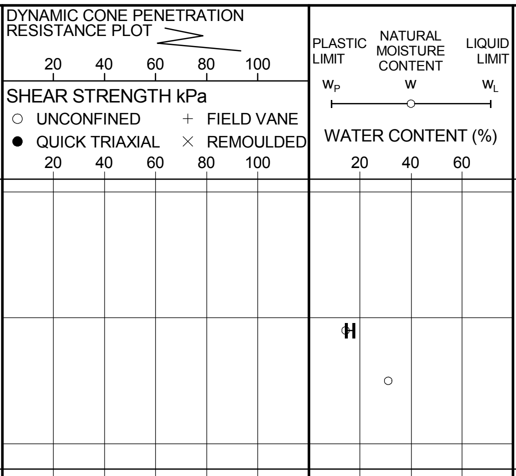
COMPILED BY AC

DATUM GEODETIC

DATE April 20, 2017

CHECKED BY SEMP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
257.1	GROUND SURFACE											
0.0	Silty TOPSOIL						257					
0.2	Sand, some gravel, some silt (FILL) Loose to dense Brown Moist		1	SS	9							
256.0 1.1	SILT, trace to some sand, trace organics		2	SS	36		256					
255.6 1.5	Dense Brown Moist											
255.3 1.8	SAND, trace gravel, trace to some silt Compact Reddish brown Wet		3	SS	24	∇			1	88	9	2
254.8 2.3	Gravelly Silty SAND (TILL) Compact Dark brown to grey Wet						255					



END OF BOREHOLE
AUGER AND SPLIT-SPOON REFUSAL

Note:

1. Water level at a depth of 1.8 m below ground surface (Elev. 255.3 m) upon completion of drilling.

2. Advanced dynamic cone penetration tests 10 m west (C17-9D1) and 10 m east (C17-9D2) of borehole.

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

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

RECORD OF PENETRATION TEST No C17-9D1

1 OF 1 **METRIC**

PROJECT 1651997

W.P. 5165-10-01

LOCATION N 5149857.5; E 318661.9 (LAT. 46.488239; LONG. -80.819439)

ORIGINATED BY SA

DIST HWY 17

BOREHOLE TYPE Dynamic Cone Penetration Test

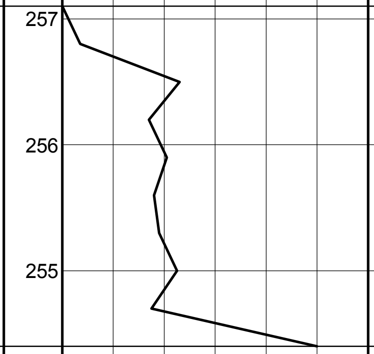
COMPILED BY AC

DATUM GEODETIC

DATE April 20, 2017

CHECKED BY SEMP

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"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 (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.1	GROUND SURFACE										
0.0							257 256 255				
254.4 2.7	END OF DCPT REFUSAL TO FURTHER PENETRATION (HAMMER BOUNCING)										



+^{3}, ×^{3}: Numbers refer to Sensitivity

○$^{3\%}$ STRAIN AT FAILURE

SUD-MTO 001 LAT/LONG 1651997.GPJ GAL-MISS.GDT 05/07/17 DATA INPUT:

RECORD OF PENETRATION TEST No C17-9D2

1 OF 1 **METRIC**

PROJECT 1651997

W.P. 5165-10-01

LOCATION N 5149857.5; E 318681.9 (LAT. 46.488238; LONG. -80.819178)

ORIGINATED BY SA

DIST HWY 17

BOREHOLE TYPE Dynamic Cone Penetration Test

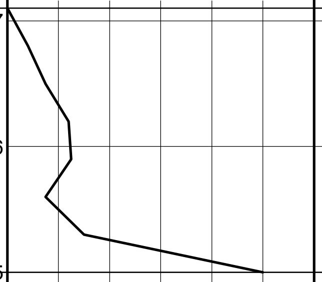
COMPILED BY AC

DATUM GEODETIC

DATE April 20, 2017

CHECKED BY SEMP

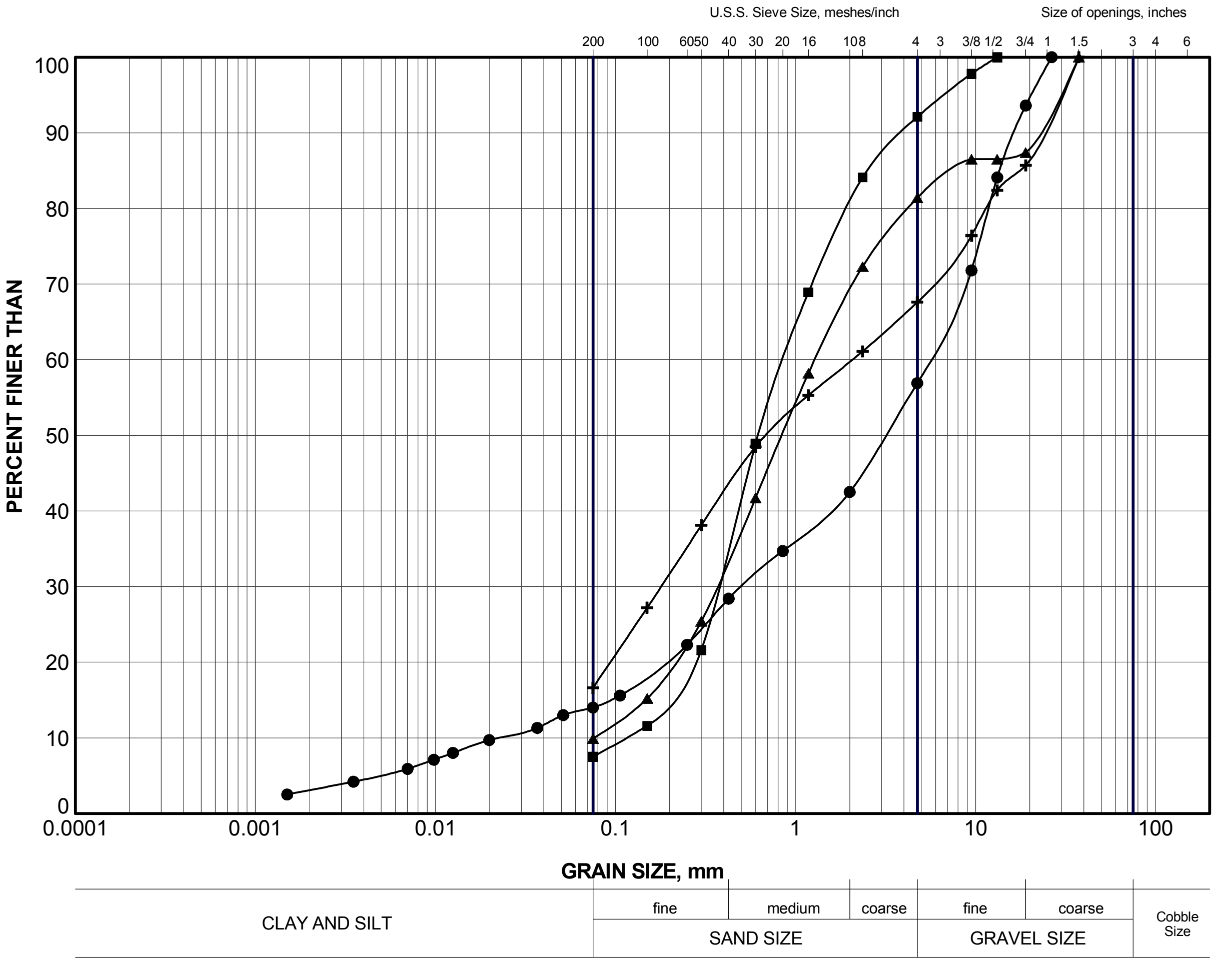
SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
257.1	GROUND SURFACE										
0.0							257				
							256				
255.0 2.1	END OF DCPT REFUSAL TO FURTHER PENETRATION (HAMMER BOUNCING)						255				



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

APPENDIX D

Current Investigation – Geotechnical Laboratory Test Results



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C17-3	1	253.3
■	C17-5	1	261.5
▲	C17-5	3	260.0
✚	C17-7	2	259.9

PROJECT

HIGHWAY 17
CONISTON CPR OVERHEAD BRIDGE

TITLE

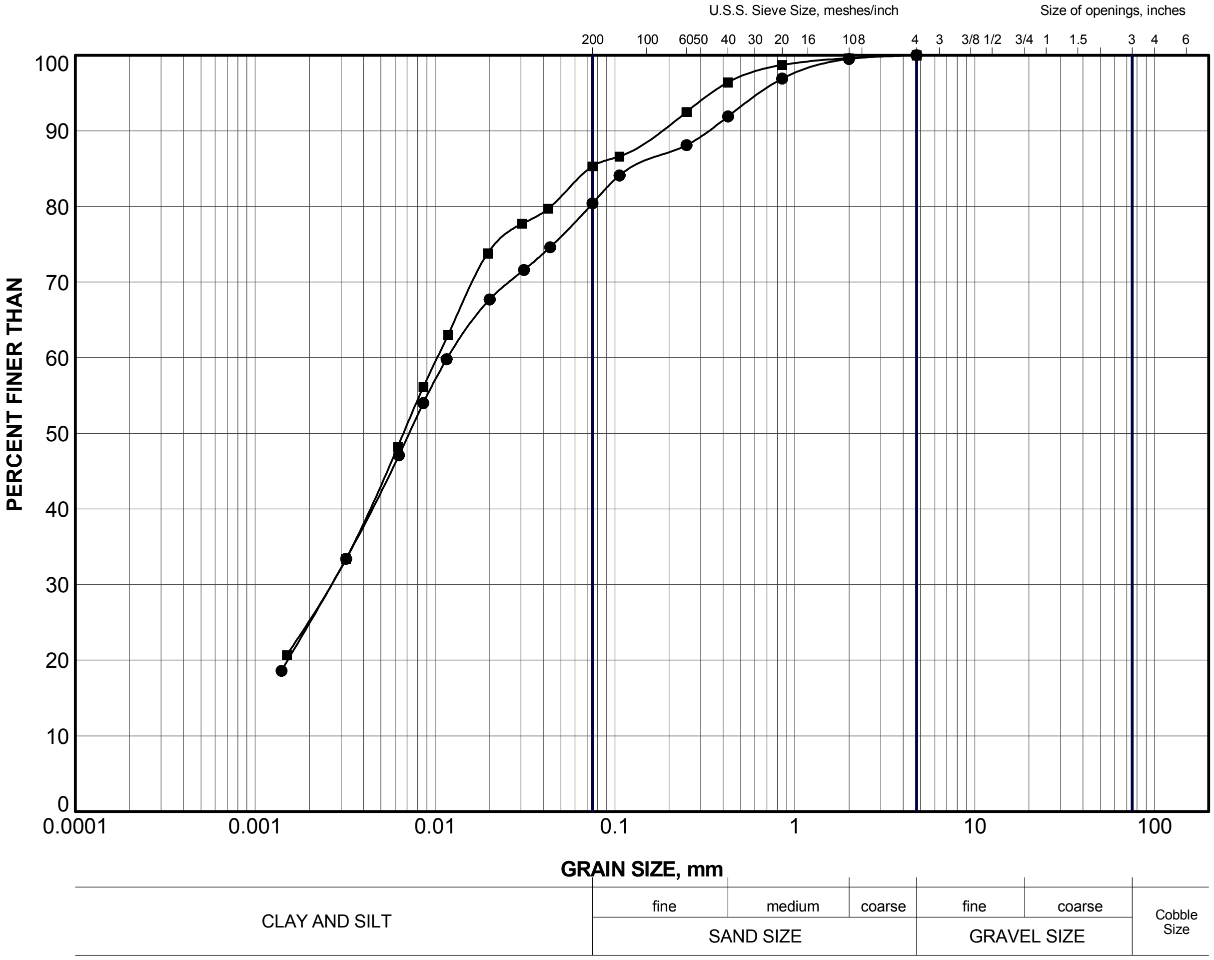
GRAIN SIZE DISTRIBUTION

SAND to SAND and GRAVEL (FILL)

PROJECT No.		1651997	FILE No.	1651997.GPJ	
DRAWN	TB	Sept 2017	SCALE	N/A	REV.
CHECK	SEMP	Sept 2017			
APPR	JMAC	Sept 2017	**FIGURE D1**		

Golder Associates
SUDBURY, ONTARIO

SUD-MTO GSD (2016) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C17-5	7	256.2
■	C17-7	5	257.6

PROJECT

HIGHWAY 17
CONISTON CPR OVERHEAD BRIDGE

TITLE

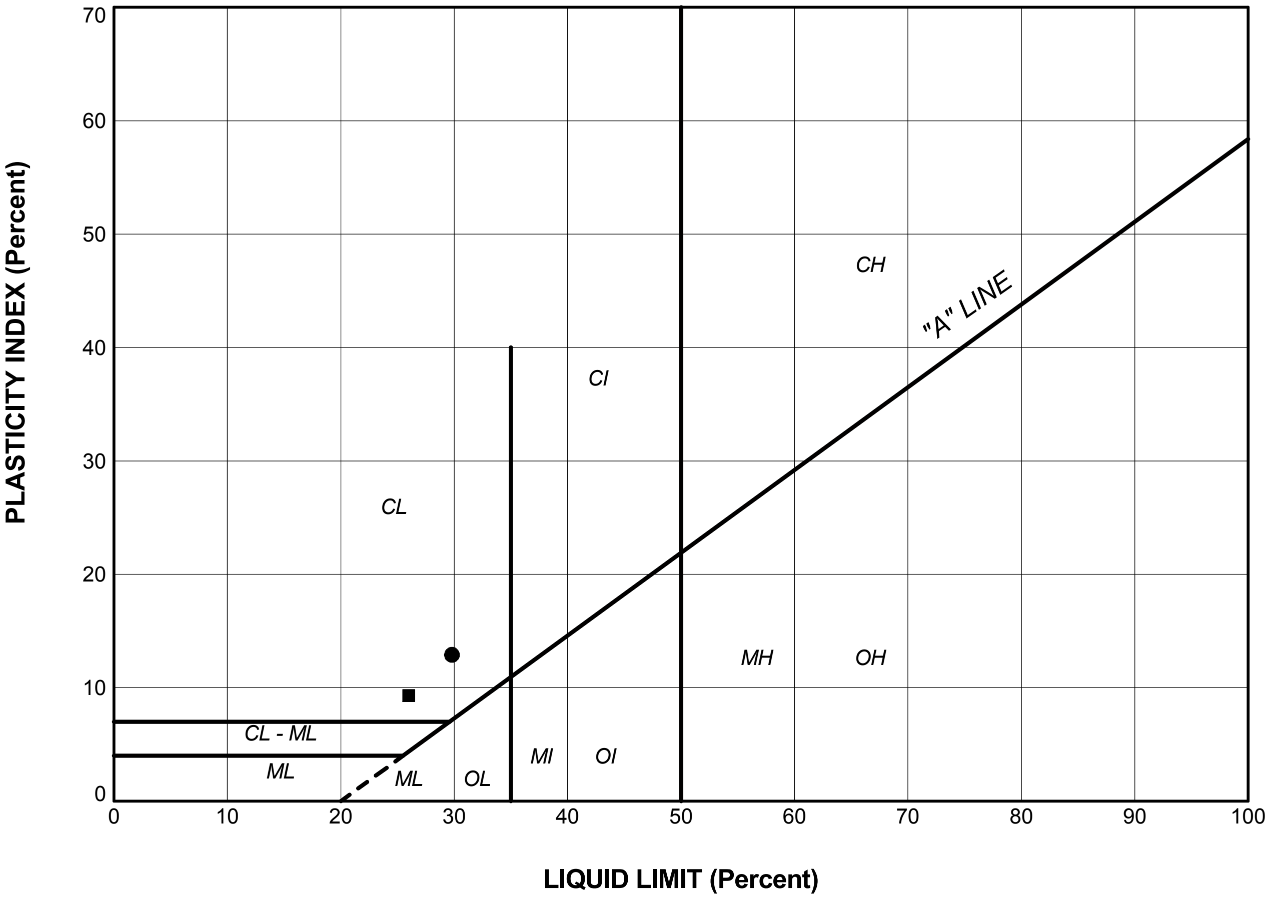
GRAIN SIZE DISTRIBUTION

CLAYEY SILT (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1651997	FILE No.	1651997.GPJ
DRAWN	TB	Sept 2017	SCALE N/A	REV.
CHECK	SEMP	Sept 2017		
APPR	JMAC	Sept 2017	FIGURE D2	

SUD-MTO GSD (2016) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	C17-5	7	29.8	16.9	12.9
■	C17-7	5	26.0	16.7	9.3

PROJECT

HIGHWAY 17
CONISTON CPR OVERHEAD BRIDGE

TITLE

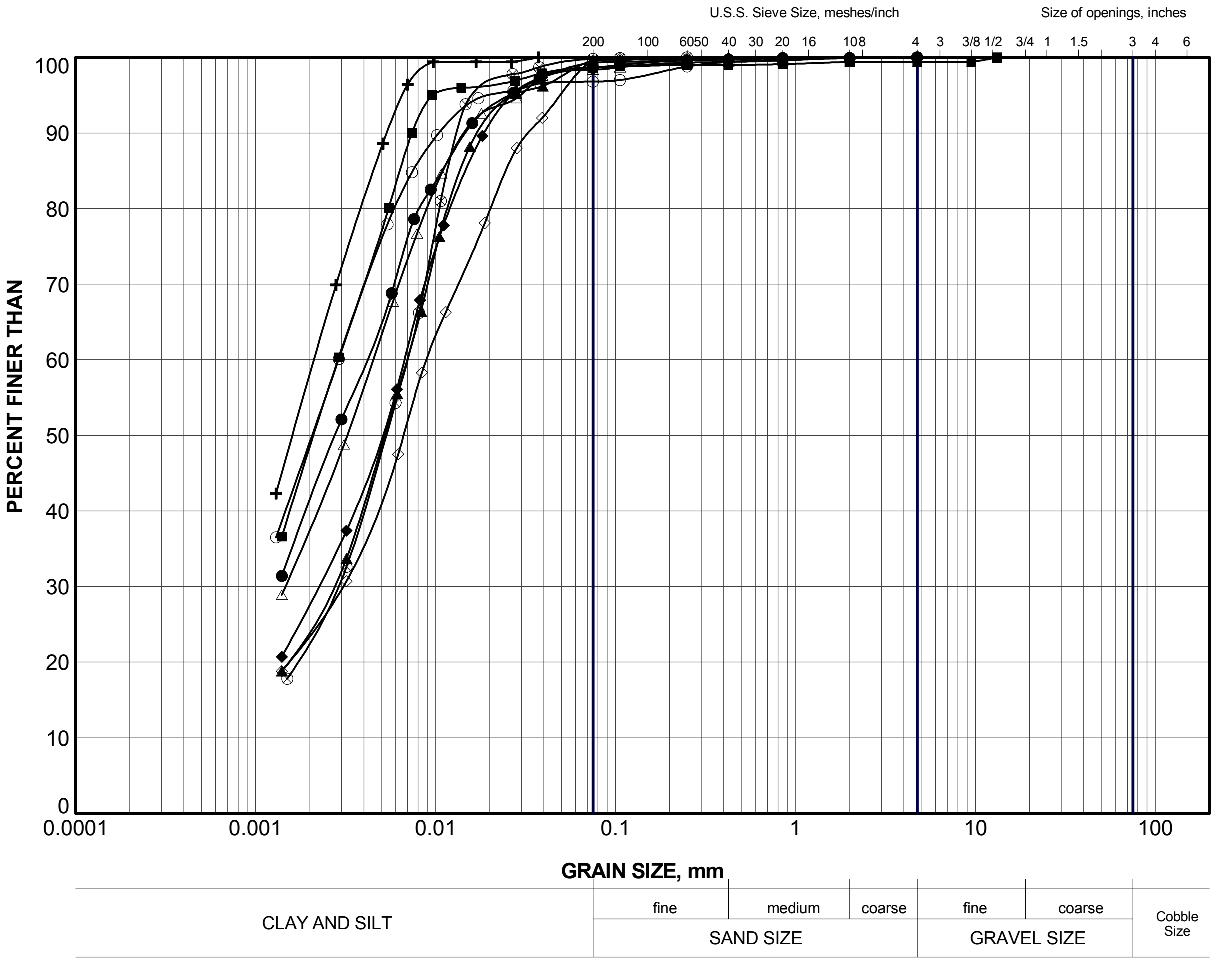
PLASTICITY CHART

CLAYEY SILT (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1651997	FILE No. 1651997.GPJ
DRAWN	TB	Sept 2017	SCALE N/A REV.
CHECK	SEMP	Sept 2017	
APPR	JMAC	Sept 2017	**FIGURE D3**

SUD-MTO PL (2016) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C17-3	4	250.9
■	C17-3	6	249.4
▲	C17-4	6A	248.2
✚	C17-5	10	251.6
◆	C17-5	12	248.6
◇	C17-5	13	247.1
○	C17-6	3	251.3
△	C17-6	5	249.8
⊗	C17-6	7	246.7

PROJECT

HIGHWAY 17
CONISTON CPR OVERHEAD BRIDGE

TITLE

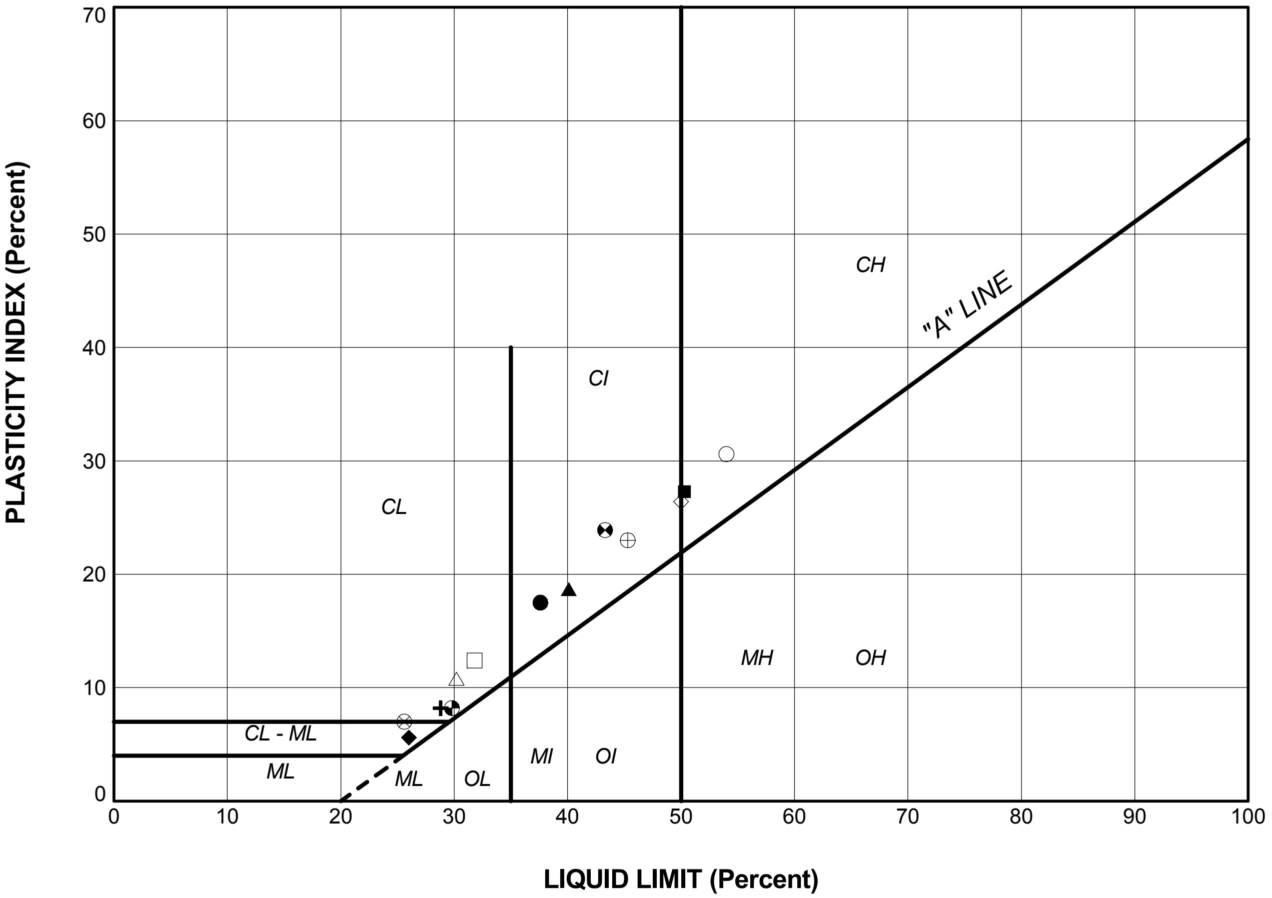
GRAIN SIZE DISTRIBUTION

CLAYEY SILT to CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1651997	FILE No. 1651997.GPJ
DRAWN	TB	Sept 2017	SCALE N/A REV.
CHECK	SEMP	Sept 2017	
APPR	JMAC	Sept 2017	**FIGURE D4**

SUD-MTO GSD (2016) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	C17-3	4	37.6	20.1	17.5
■	C17-3	6	50.3	23.0	27.3
▲	C17-3	7	40.1	21.4	18.7
✚	C17-4	6A	28.8	20.6	8.2
◆	C17-4	6B	26.0	20.4	5.6
◇	C17-5	10	50.0	23.6	26.4
○	C17-5	11	54.0	23.4	30.6
△	C17-5	12	30.2	19.4	10.8
⊗	C17-5	13	25.6	18.6	7.0
⊕	C17-6	3	45.3	22.3	23.0
□	C17-6	5	31.8	19.4	12.4
◓	C17-6	6	43.3	19.4	23.9
◕	C17-6	7	29.8	21.6	8.2

PROJECT
HIGHWAY 17
CONISTON CPR OVERHEAD BRIDGE

TITLE

PLASTICITY CHART

CLAYEY SILT to CLAY

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1651997	FILE No.	1651997.GPJ
DRAWN	TB	Sept 2017	SCALE N/A	REV.
CHECK	SEMP	Sept 2017	**FIGURE D5**	
APPR	JMAC	Sept 2017		

SUD-MTO PL25 (2016) GLDR_LDN.GDT

CONSOLIDATION TEST SUMMARY

FIGURE D6

Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Number	1651997-1203	Sample Number	7
Borehole Number	C17-3	Sample Depth, m	5.5

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Numbe	2		
Date Started	May 17, 2017		
Date Completed	May 25, 2017		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.522	Unit Weight, kN/m^3	18.72
Sample Diameter, cm	6.358	Dry Unit Weight, kN/m^3	13.86
Area, cm^2	31.74	Specific Gravity, Measured	2.766
Volume, cm^3	80.06	Solids Height, cm	1.289
Water Content, %	35.09	Volume of Solids, cm^3	40.91
Wet Mass, g	152.85	Volume of Voids, cm^3	39.15
Dry Mass, g	113.15		

TEST COMPUTATIONS

Pressure kPa	Primary Consolidation mm	Corr. Height cm	End of Primary Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s	Total Work kJ/m3
0	0	2.522	0.957	2.522					
4	0.07	2.510	0.952	2.516	375	0.0036	6.17E-04	2.16E-07	0.006
13	0.04	2.500	0.945	2.505	240	0.0055	4.18E-04	2.27E-07	0.038
31	0.06	2.481	0.936	2.491	135	0.0097	2.50E-04	2.39E-07	0.136
66	0.11	2.453	0.917	2.467	135	0.0096	2.71E-04	2.54E-07	0.606
137	0.18	2.403	0.890	2.428	135	0.0093	2.00E-04	1.81E-07	2.063
277	0.44	2.326	0.831	2.365	375	0.0032	2.14E-04	6.63E-08	8.506
558	0.31	2.265	0.781	2.295	240	0.0047	9.11E-05	4.16E-08	19.921
1140	0.28	2.205	0.736	2.235	60	0.0176	3.90E-05	6.75E-08	41.105
558	-0.05	2.210	0.715	2.207					
137	-0.14	2.224	0.726	2.217					
31	-0.16	2.241	0.739	2.232					
4	-0.12	2.252	0.748	2.246					

Note:
k calculated using cv based on t_{90} values.
Void ratio for unloading (or rebound) calculated for the end of incremen

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.423	Unit Weight, kN/m^3	18.31
Sample Diameter, cm	6.36	Dry Unit Weight, kN/m^3	14.42
Area, cm^2	31.74	Specific Gravity, Measured	2.766
Volume, cm^3	76.93	Solids Height, cm	1.289
Water Content, %	26.95	Volume of Solids, cm^3	40.91
Wet Mass, g	143.64	Volume of Voids, cm^3	36.02
Dry Mass, g	113.15		

Prepared By: TG

Golder Associates

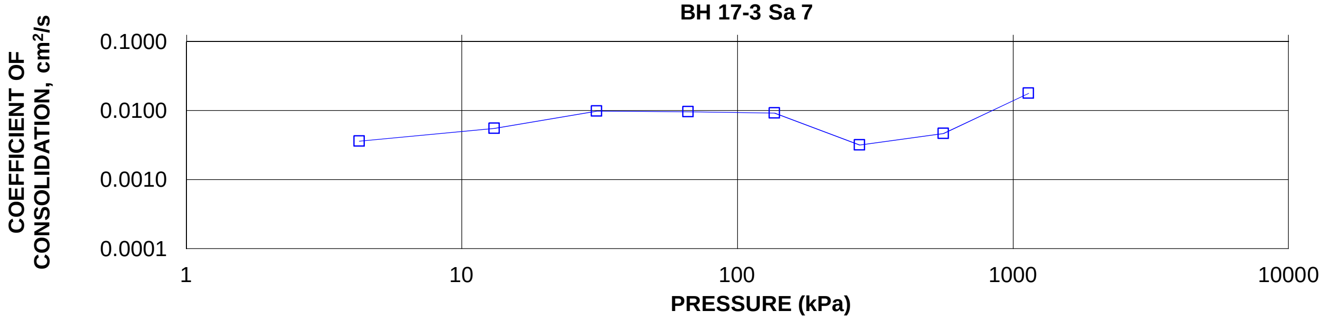
Checked By: MT

CONSOLIDATION TEST SUMMARY

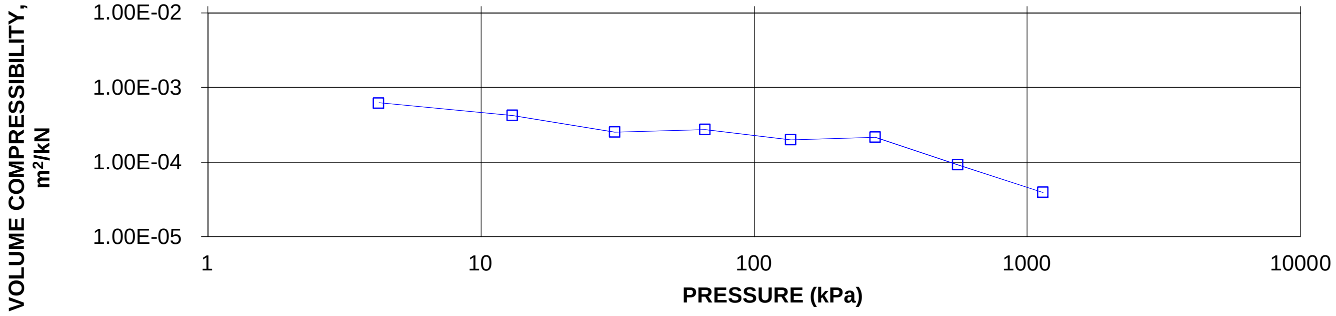
FIGURE D6
Pg. 2 of 4

C17-3

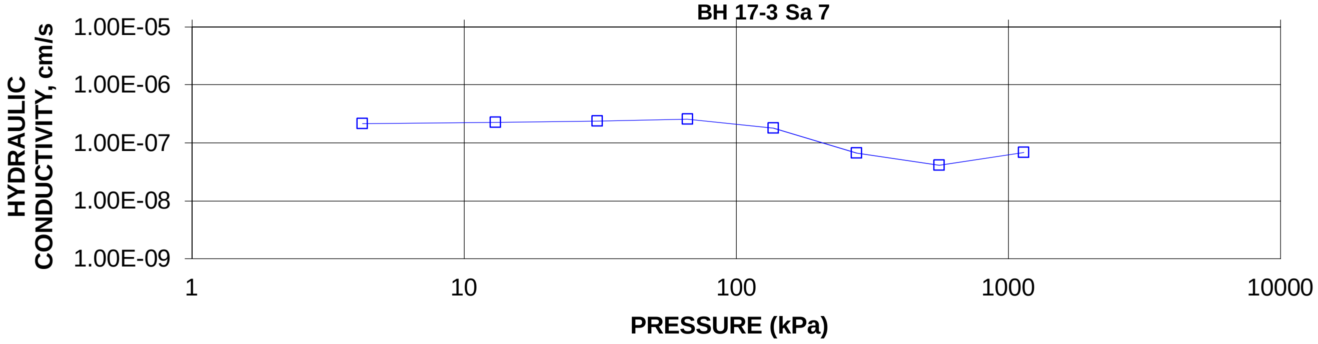
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH 17-3 Sa 7



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 17-3 Sa 7



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 17-3 Sa 7



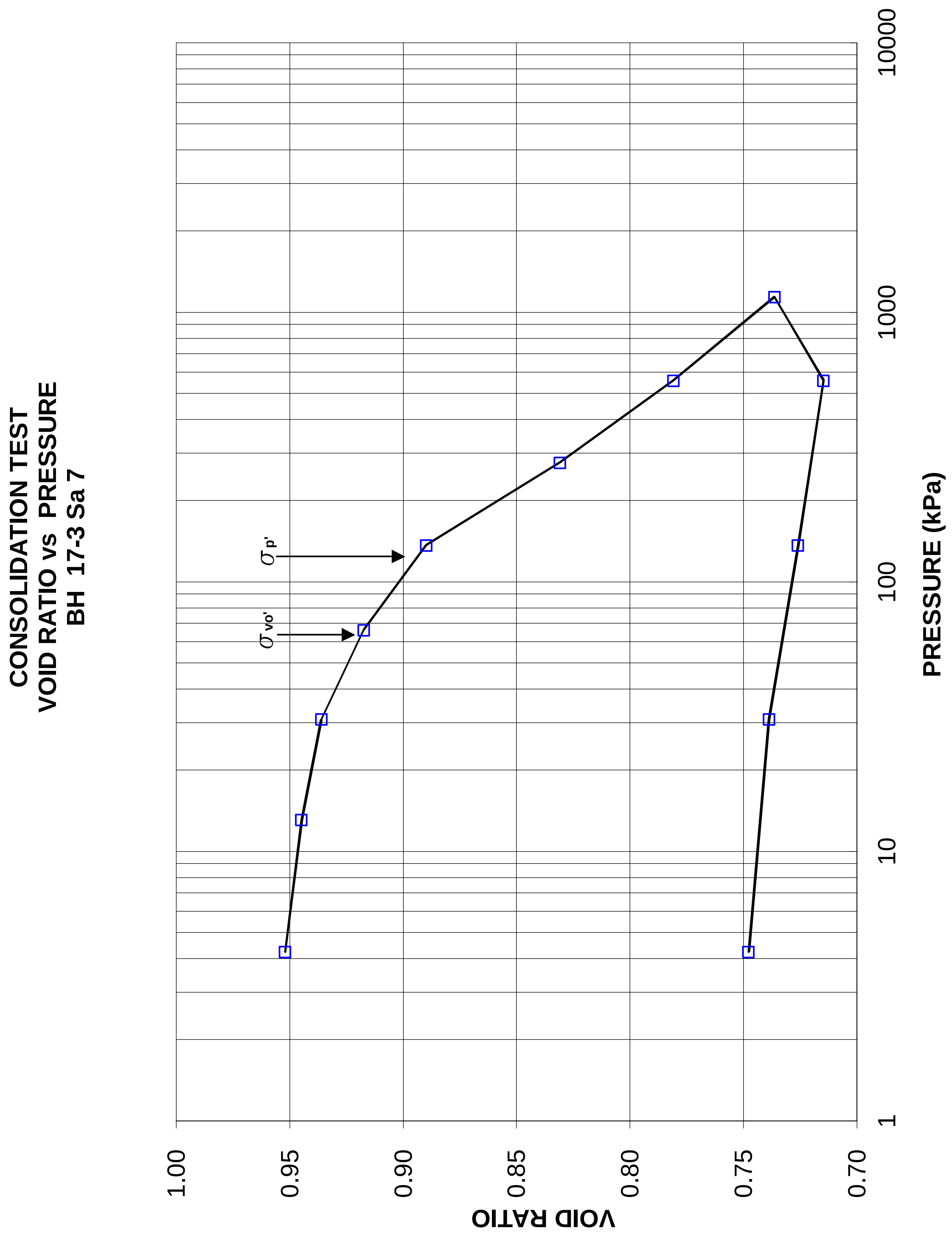
Project No.1651997-1203
Prepared By: TG

Golder Associates

Checked By: MT

CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

FIGURE D6
Pg. 3 of 4



Project No. 1651997-1203

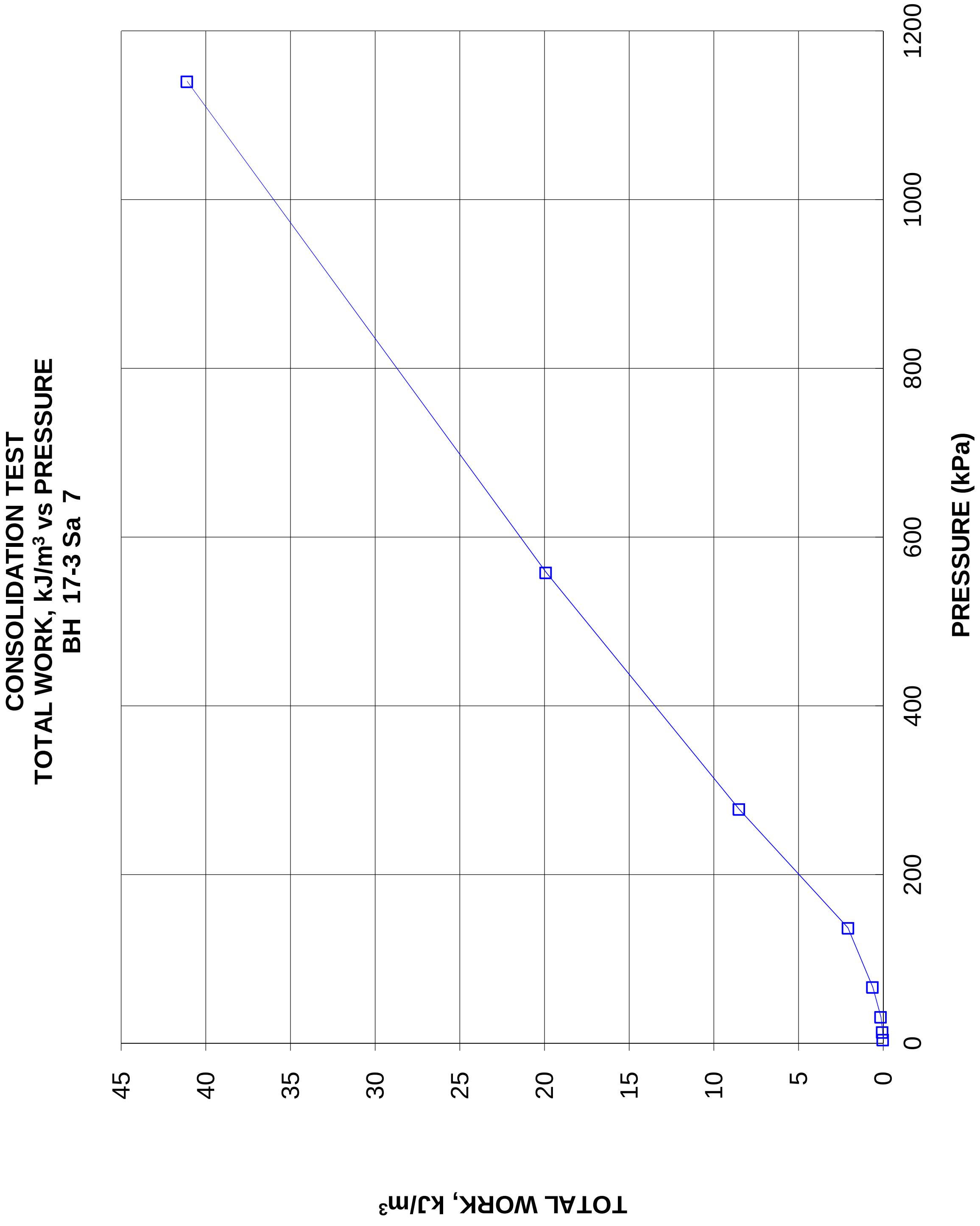
Prepared By: TG

Golder Associates

Checked By: MT

CONSOLIDATION TEST
TOTAL WORK VS PRESSURE

FIGURE D6
Pg. 4 of 4



Project No: 1651997-1203

Prepared By: TG

Golder Associates

Checked By: MT

CONSOLIDATION TEST SUMMARY

FIGURE D7
Pg. 1 of 4

SAMPLE IDENTIFICATION

Project Numbe	1651997-1203	Sample Numbe	1
Borehole Numbe	C17-4	Sample Depth, m	5.5

TEST CONDITIONS

Test Type	Standard	Load Duration, hı	24
Oedometer Numbe	1		
Date Startec	May 9,2017		
Date Completec	May 25,2017		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.544	Unit Weight, kN/m^3	18.00
Sample Diameter, cm	6.357	Dry Unit Weight, kN/m^3	12.99
Area, cm^2	31.74	Specific Gravity, measurec	2.765
Volume, cm^3	80.75	Solids Height, cm	1.219
Water Content, %	38.53	Volume of Solids, cm^3	38.69
Wet Mass, g	148.20	Volume of Voids, cm^3	42.06
Dry Mass, g	106.98		

TEST COMPUTATIONS

Pressure kPa	Primary Consolidation mm	Corr. Height cm	End of Primary Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s	Total Work kJ/m3
0	0.00	2.544	1.087	2.544					
9	0.01	2.526	1.086	2.535	375	0.00363	4.40E-05	1.57E-08	0.002
18	0.04	2.512	1.068	2.519	540	0.00249	9.59E-04	2.34E-07	0.117
35	0.14	2.485	1.050	2.499	844	0.00157	5.21E-04	8.01E-08	0.356
69	0.17	2.447	1.025	2.466	470	0.00274	3.42E-04	9.19E-08	0.975
143	0.24	2.396	0.988	2.422	540	0.00230	2.45E-04	5.52E-08	2.936
285	0.33	2.311	0.939	2.353	375	0.00313	1.66E-04	5.09E-08	8.238
570	0.65	2.206	0.842	2.258	540	0.00200	1.63E-04	3.19E-08	29.574
1140	0.48	2.121	0.770	2.163	240	0.00413	6.00E-05	2.43E-08	62.723
570	-0.05	2.126	0.744	2.123					
143	-0.18	2.144	0.759	2.135					
35	-0.20	2.164	0.775	2.154					
9	-0.19	2.182	0.790	2.173					

Note:
k calculated using c_v based on t_{90} values.
Void ratio for unloading (or rebound) calculated for the end of increme

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

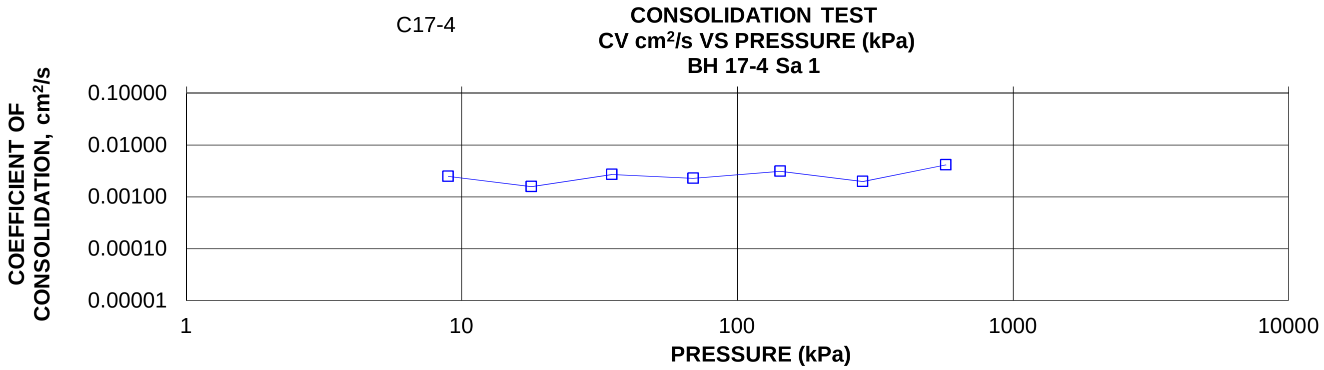
Sample Height, cm	2.400	Unit Weight, kN/m^3	17.57
Sample Diameter, cm	6.36	Dry Unit Weight, kN/m^3	13.77
Area, cm^2	31.74	Specific Gravity, measurec	2.765
Volume, cm^3	76.17	Solids Height, cm	1.219
Water Content, %	27.58	Volume of Solids, cm^3	38.69
Wet Mass, g	136.49	Volume of Voids, cm^3	37.48
Dry Mass, g	106.98		

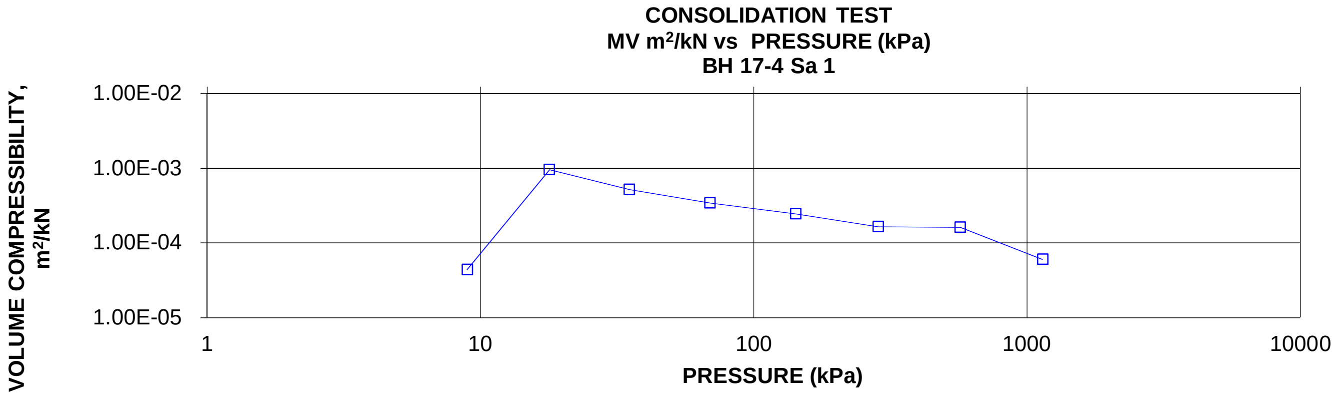
Prepared By: TG **Golder Associates** Checked By: MT

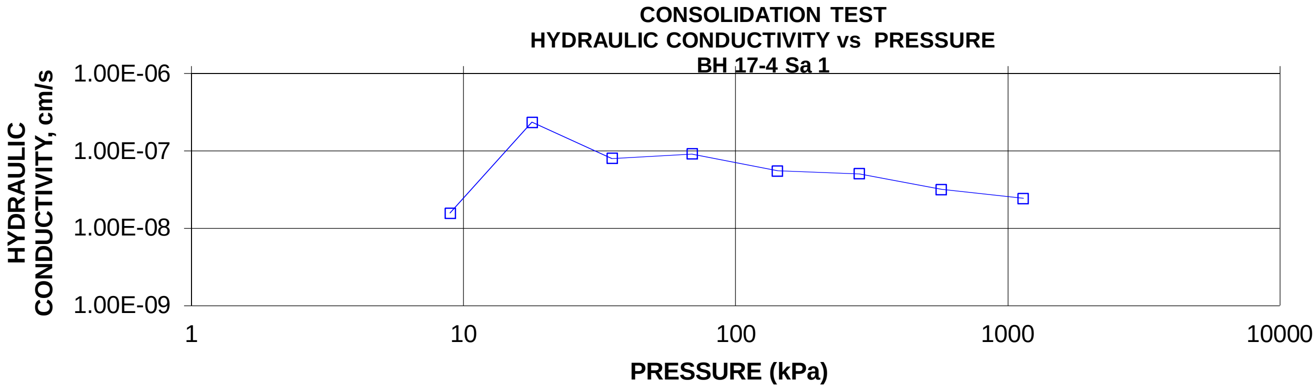
CONSOLIDATION TEST SUMMARY

FIGURE D7
Pg. 2 of 4

C17-4







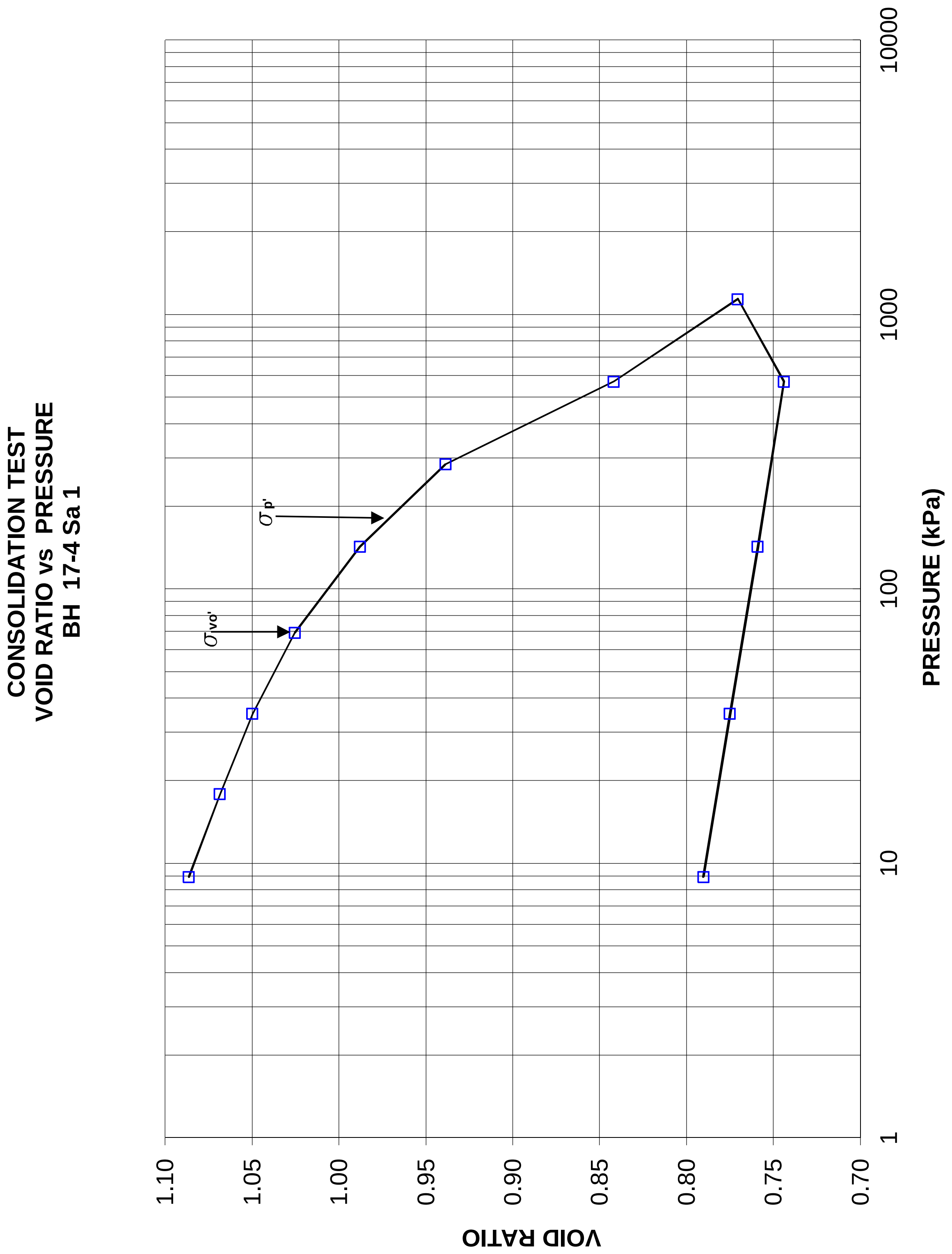
Project No. 1651997-1203
Prepared By: TG

Golder Associates

Checked By: MT

CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

FIGURE D7
Pg. 3 of 4



Project No. 1651997-1203

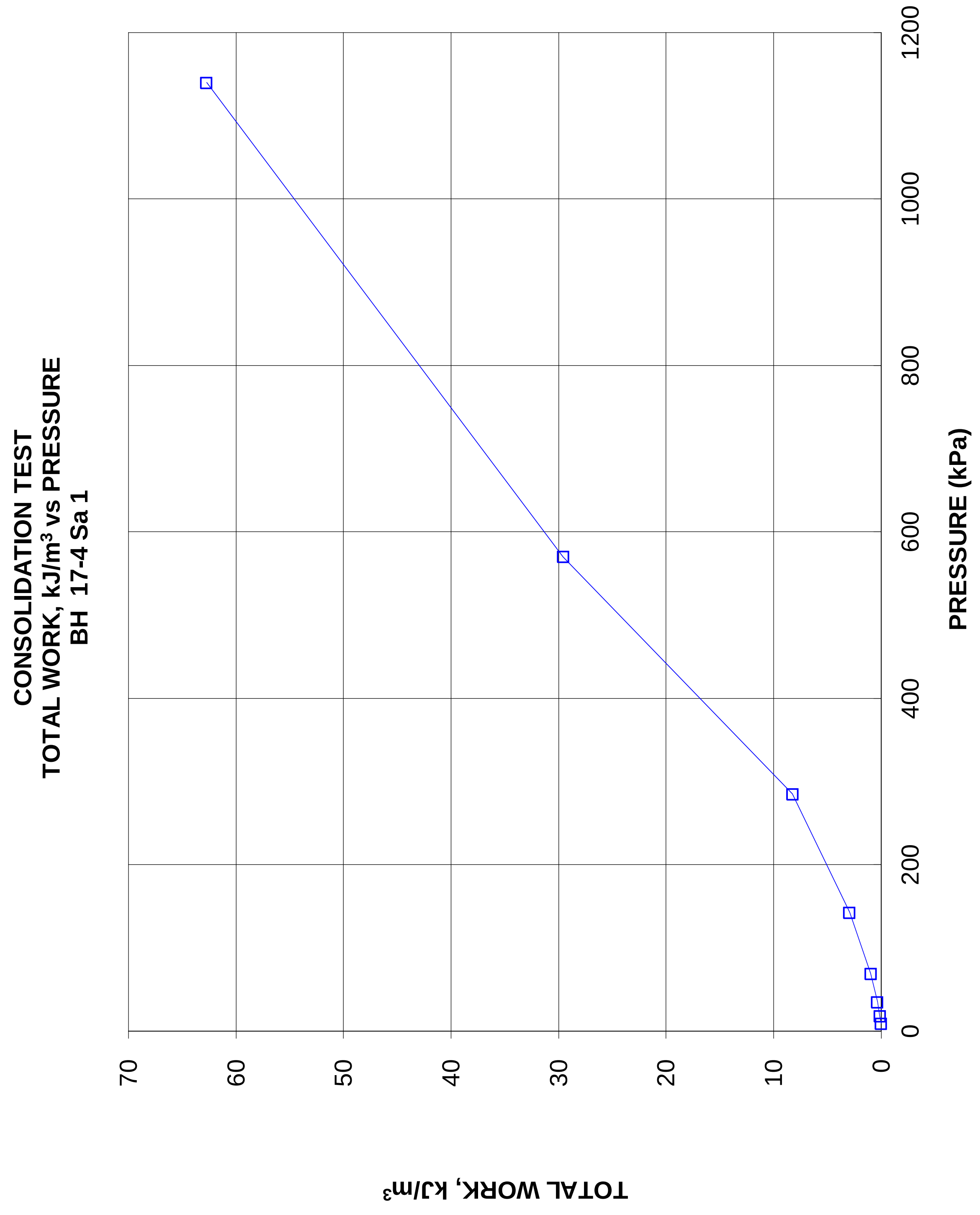
Prepared By: TG

Golder Associates

Checked By: MT

CONSOLIDATION TEST
TOTAL WORK VS PRESSURE

FIGURE D7
Pg. 4 of 4

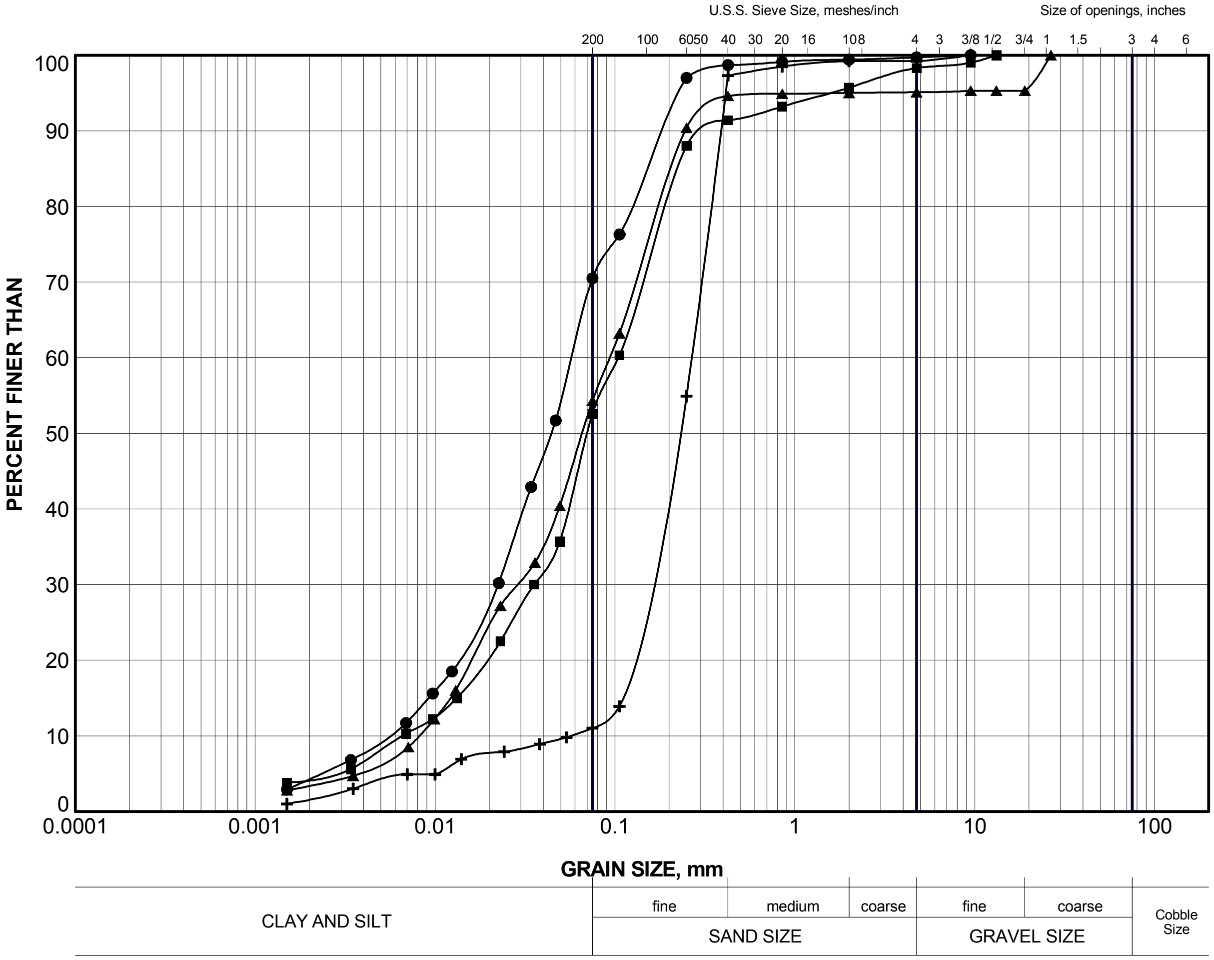


Project No: 1651997-1203

Prepared By: TG

Golder Associates

Checked By: MT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C17-3	8	245.6
■	C17-4	7	246.5
▲	C17-6	9B	243.7
+	C17-9	3	255.5

PROJECT

HIGHWAY 17
CONISTON CPR OVERHEAD BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

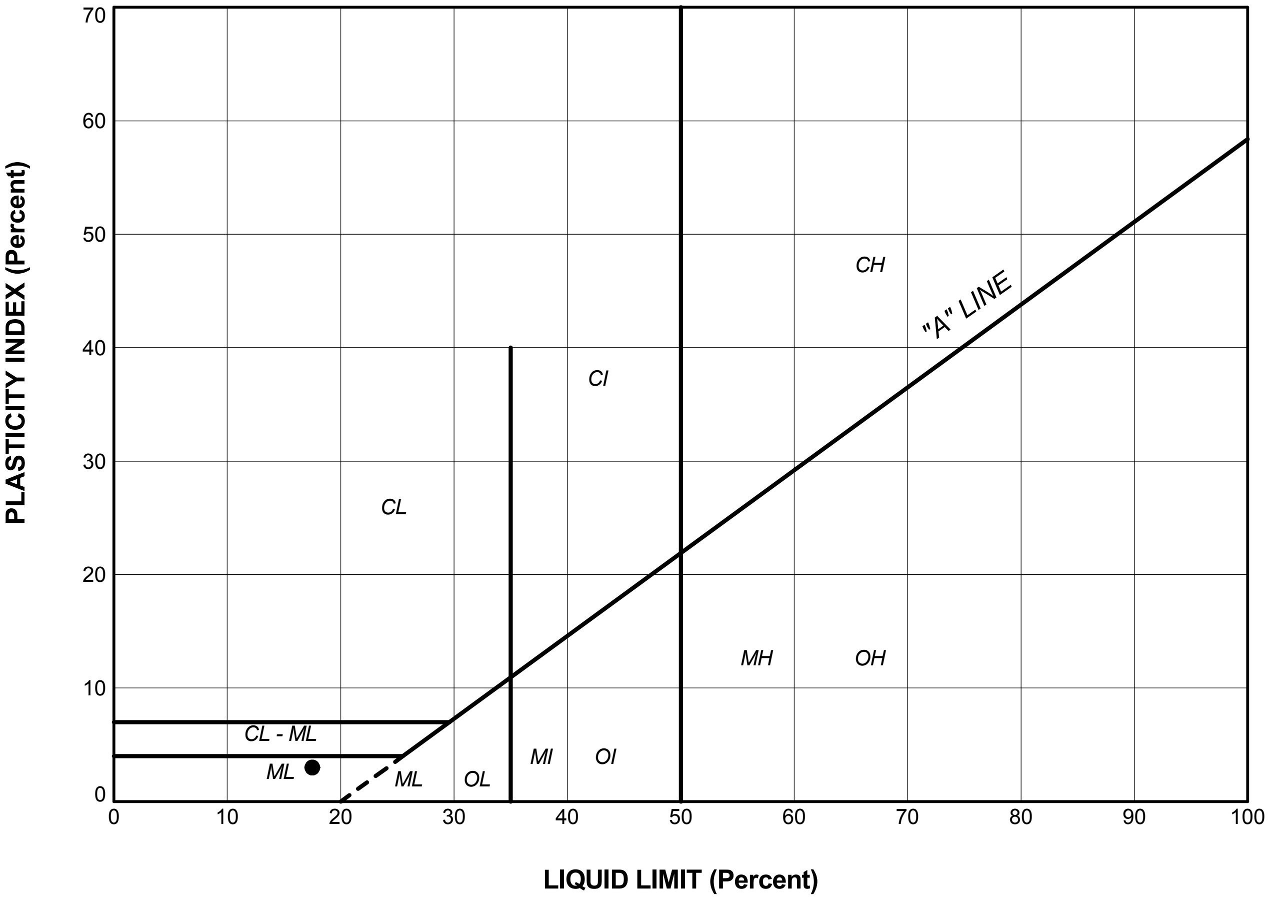
SANDY SILT to SILT and SAND to SAND

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1651997	FILE No.	1651997.GPJ
DRAWN	TB	Sept 2017	SCALE N/A	REV.
CHECK	SEMP	Sept 2017		
APPR	JMAC	Sept 2017		

FIGURE D8

SUD-MTO GSD (2016) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	C17-9	2	17.5	14.5	3.0

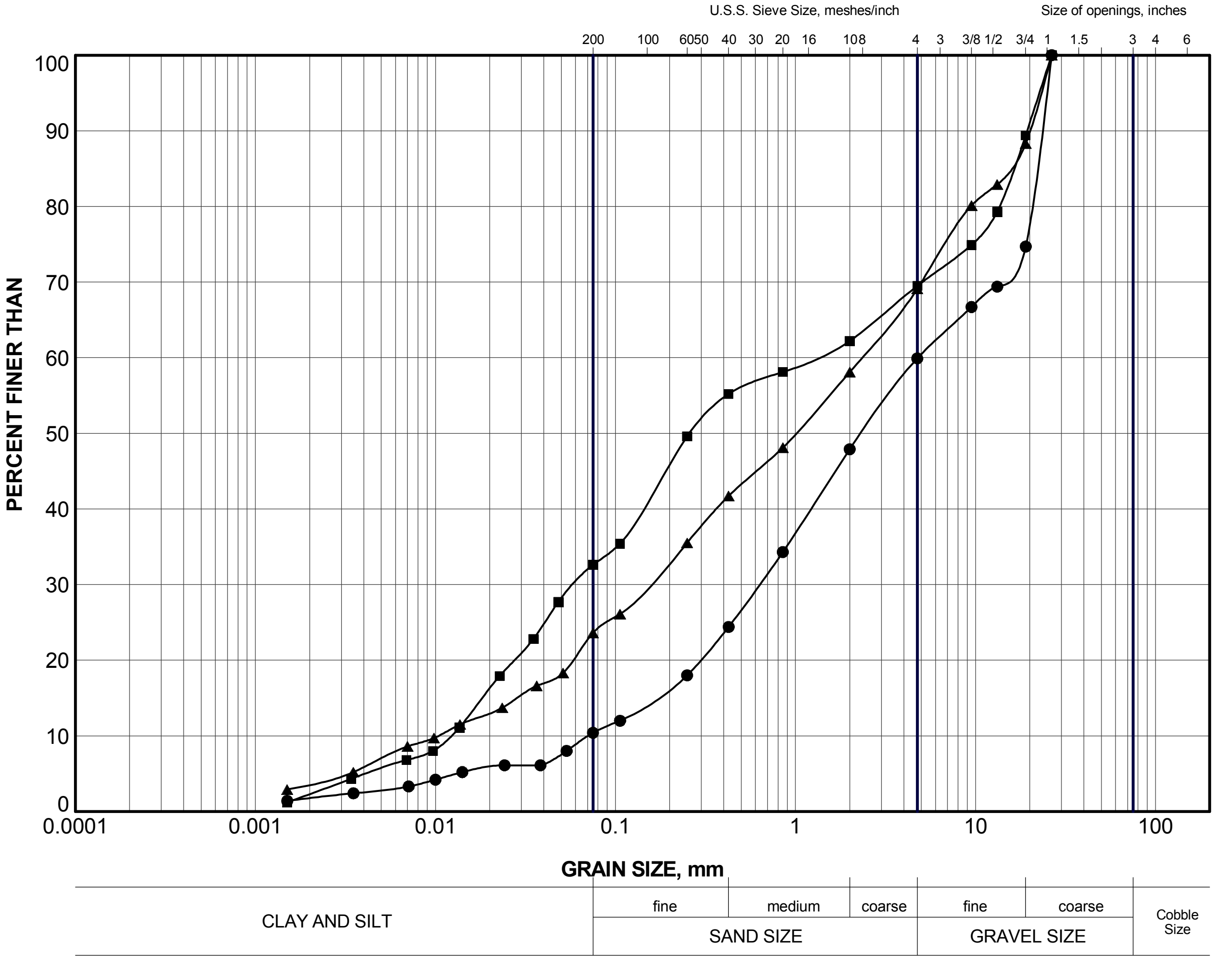
PROJECT
HIGHWAY 17
CONISTON CPR OVERHEAD BRIDGE

TITLE
PLASTICITY CHART
SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1651997	FILE No.	1651997.GPJ
DRAWN	TB	Sept 2017	SCALE N/A	REV.
CHECK	SEMP	Sept 2017		
APPR	JMAC	Sept 2017	FIGURE D9	

SUD-MTO PL (2016) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C17-5	14	245.6
■	C17-7	7	255.3
▲	C17-7	8	253.8

PROJECT

HIGHWAY 17
CONISTON CPR OVERHEAD BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

SILTY SAND and GRAVEL to SAND and GRAVEL (TILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.		1651997	FILE No.		1651997.GPJ	
DRAWN	TB	Sept 2017	SCALE	N/A	REV.	
CHECK	SEMP	Sept 2017	FIGURE D10			
APPR	JMAC	Sept 2017				

SUD-MTO GSD (2016) GLDR_LDN.GDT

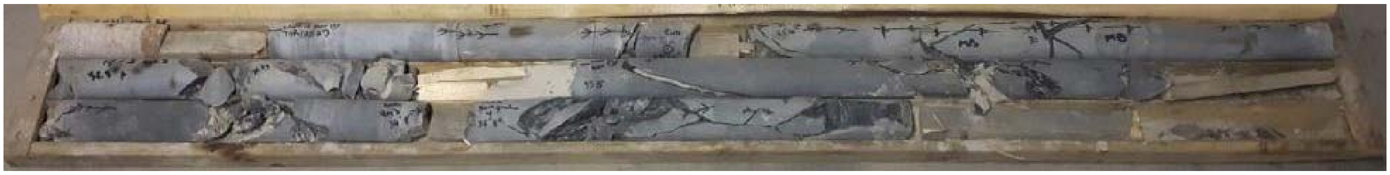
Borehole C17-1

Box 1: 0.0 m – 3.0 m

Borehole C17-2

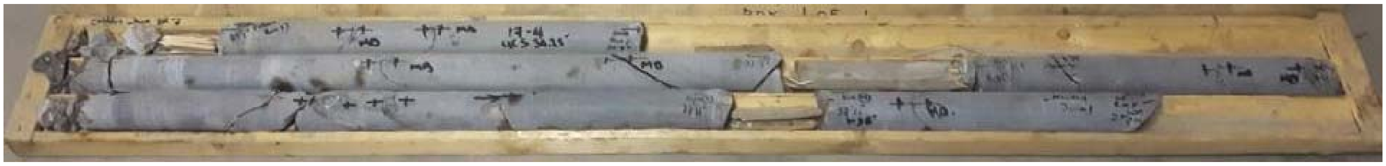
Box 1: 0.0 m – 3.0 m

Borehole C17-3



Box 1: 8.6 m – 11.7 m

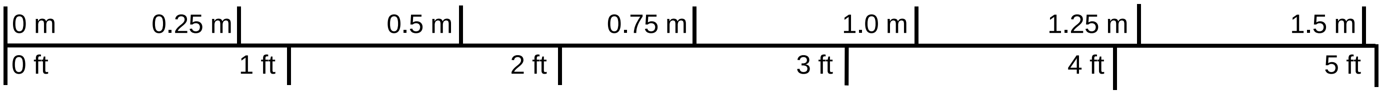
Borehole C17-4



Box 1: 8.9 m – 11.9 m

Borehole C17-7

Box 1: 8.2 m – 9.2 m



Scale

PROJECT

CPR Overhead on Highway 17 at Coniston
Sudbury Area, Ontario

TITLE

Bedrock Core Photographs
Borehole C17-1, C17-2, C17-3, C17-4, C17-7

Golder Associates

PROJECT No.	1651997		FILE No.	----
DESIGN	AC	JUN 17	SCALE NTS	REV.
CADD	-- --		**FIGURE D11**	
CHECK	SEMP	JUN 17		
REVIEW	SEMP	JUN 17		

REVISION DATE: June 12, 2017 BY: AC Project: 1651997

APPENDIX E

NSSPs

DOWELS INTO ROCK - Item No.

Non-Standard Special Provision

Scope of Work

This special provision covers the requirements for the placement and field testing of dowels into rock.

Construction

Dowels into rock shall be constructed in accordance with OPSS.PROV 904 Concrete Structures[i]. All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 Steel Reinforcement for Concrete[ii] (dowel bars conforming to CAN/CSA G30.18, Grade 400).

Where dowels are to be placed in rock, hole shall be drilled to the required depth and size. Hole diameter shall be two times the nominal diameter of the dowel. Each hole shall be cleaned out, grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete or at least 30 MPa at 28 days.

If the dowel hole contains water, the Contractor shall remove the water, otherwise a tremie procedure shall be used to completely fill the hole with grout. The dowel shall be forced into the hole after the grout has been placed and while it is still fresh.

Rock Dowel Testing

All proposed testing procedures shall be in general conformance with ASTM D3689-07, ASTM D1143-07 and ASTM D4435-08. Field testing must be carried out in the presence of, and the results reviewed and approved by, the Contract Administrator.

Performance Tests

The following table summarizes the number of rock dowels where performance testing shall be carried out to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

Bridge	Foundation	Number of Dowels for Performance Testing
Temporary Detour Structure	East Abutment	2 per footing

Performance test shall be by axial tensioning using a hydraulic jack with a capacity of at least 1.5 times the ultimate strength of the dowels.

Rock dowels shall be loaded and unloaded in 3 cycles and measurements of the displacement of the dowel shall be carried out at each load increment (step) in accordance with the following schedule:

Cycle-Step	1-1	1-2	1-3	2-1	2-2	2-3	2-4
% Design Load	50	75	25	50	75	100	25

Cycle-Step	3-1	3-2	3-3	3-4	3-5
% Design Load	50	75	100	110	25

The design load shall be taken as 360 kN for 35M dowels, 252 kN for 30M dowels, 180 kN, for 25M dowels, and 108 kN for 20M dowels.

Displacement measurements shall be carried out at each load increment using calibrated displacement gauges capable of measuring movements of 0.0025 cm. Measurements shall be referenced to an independent fixed referenced pint.

Rock dowels which fail to meet the acceptance criteria shall be replaced at the Contractor's expense and re-tested. If a rock dowel fails, three (3) additional rock dowels shall be tested at the same abutment and pier footing as directed by the Contract Administrator.

Acceptance criteria for the rock dowels will be in accordance with the Post-Tensioning Institute (1985) as follows:

- The dowels are acceptable if the total elastic movement is greater than 80 percent of the theoretical elastic elongation of the free stressing and is less than the theoretical elongation of the free stressing length plus 50 percent of the bond length.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

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[i] OPSS.PROV 904 Construction Specification for Concrete Structures

[ii] OPSS.PROV 1440 Material Specification for Steel Reinforcement for Concrete

OBSTRUCTIONS

Non-Standard Special Provision

The Contactor is hereby notified that the native soils and embankment fill at the site of the Coniston CPR Overhead structure site and as inferred from available information should be expected to contain cobbles and boulders as encountered overlying bedrock in some boreholes, which could affect excavations and the installation of deep foundations. Further the existing east approach embankment is reportedly constructed of blast rock fill. Consideration of the presence of these obstructions must be made in selection of appropriate equipment and procedures for installation of the foundations.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

MASS CONCRETE - Item No.

Non-Standard Special Provision

Scope of Work

The scope of work for the above noted tender item includes mass concrete under the footings at the east abutments at the Highway 17 – CPR overpass temporary detour structure.

Construction

Concrete shall be of the same strength as the footing concrete and placed in accordance with OPSS 904. Concrete shall be placed directly over the properly prepared bedrock surface.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

H-PILES - Item No.

Non-Standard Special Provision

903.07.02 Driven Piles

903.07.02.07.03.03 Driving to Bedrock

Section 903.07.02.07.03.03 of OPSS 903 is deleted and replaced with the following:

When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile.

In order to avoid overdriving and possibly damaging the piles when seating onto bedrock, the piles shall be driven to an initial set equal to or greater than 10 blows per 12 mm of penetration (unless abrupt peaking occurs) using a hammer with rated energy of about 50 kilojoules but not exceeding 60 kilojoules. The bedrock elevation shall be recorded. On reaching the required set, the hammer energy shall be reduced to 75 percent of the maximum energy and the pile shall then be re-driven in 2 sets of 10 blows and the penetration recorded after each set of 10 blows. The hammer energy shall then be increased to 100 percent and the pile re-driven for 10 blows and the penetration recorded. A final set of no less than 10 blows per 12 mm of penetration shall be obtained at the maximum hammer energy.

If unusually excessive penetration per blow is observed, driving shall be stopped and this excessive penetration immediately reported to the Contract Administrator.

The Quality Verification Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

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