



April 21, 2016

## DRAFT FOUNDATION INVESTIGATION AND DESIGN REPORT

**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**

**Submitted to:**

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# **PART A**

**FOUNDATION INVESTIGATION REPORT**

**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**



### 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by The Ministry of Transportation, Ontario (MTO), Northeastern Region to provide foundation engineering services for proposed retained soil structure (RSS) walls at the Coniston Canadian Pacific Railway (CPR) Overhead, located on Highway 17 approximately 2.8 km west of the Highway 17 and Highway 537 junction in the Sudbury Area. This work has been carried out under the Retainer Assignment Agreement #5013-E-0034.

The purpose of this investigation is to establish the subsurface conditions at the locations of the proposed RSS wall, adjacent to the Coniston CPR Overhead by methods of borehole drilling, in situ testing and laboratory testing of selected soil samples. The proposed RSS walls will be part of the rehabilitation of the overhead structure to be carried out by Morrison Hershfield Ltd. (MH). As part of the Terms of Reference (TOR), the MTO provided the General Arrangement and Conceptual RSS Wall design drawings, which were completed by MH. The approximate location of the Coniston CPR Overhead (the Site) is shown on Drawing 1.

### 2.0 SITE DESCRIPTION AND BACKGROUND INFORMATION

Based on the design drawings, provided to us on October 6, 2015, we understand the RSS walls will be used to support a crash wall, constructed behind the east and west piers of the existing overhead structure, at track level. Two RSS walls are proposed, one at each of the east and west bridge piers, with each wall approximately 3.0 m in height and 32 m long.

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 areas of standing water and various types of vegetation and organic soils. The CP railway right of way within the vicinity of the overhead structure, appear 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. The ground surface at the proposed RSS Wall Locations is relatively flat and ranges in elevation from 253.9 m to 254.1 m at the west pier and 254.0 m to 254.4 m at the east pier, based on ground surface elevations measured at the borehole locations.

A previous foundation investigation and design report for the site indicates the native material at the site consists of varved silty clay to clayey silt underlain by silty sand to sandy silt with gravel, which is in turn underlain by bedrock<sup>1</sup>.

### 3.0 INVESTIGATION PROCEDURE

The investigation for the RSS walls was carried out between January 18 and 21, 2016, during which time a total of four boreholes, denoted BH1 through BH4, were advanced within the footprint of the proposed walls at the locations shown on Drawing 1. The Record of Borehole sheets are presented in Appendix A.

Prior to the beginning of the field investigation Golder completed a site reconnaissance to assess access for drilling equipment and layout the borehole locations. The boreholes were staked in the field based on the details provided on the General Arrangement drawings and existing site features. Given the locations of boreholes, Golder was required to arrange access to the CPR corridor, coordinated the investigation work with CPR and arranged to have

<sup>1</sup> 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. 411-140.



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CPR flaggers on site while completing the investigation. In addition Golder prepared a site specific health and safety plan and contacted Ontario-One-Call and CPR to clear the borehole locations of buried services/utilities.

The field investigation was carried out using a buggy-mounted CME 550 drill rig and portable tri-pod drilling equipment supplied and operated by Landcore Drilling Ltd. of Chelmsford, Ontario. Boreholes BH1 and BH2, at the west piers, were advanced using 108 mm inner diameter hollow-stem augers. Boreholes BH3 and BH4, at the east piers, were advanced using 76 mm inner diameter NW casing and wash boring techniques. 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 for Boreholes BH1 and BH2 and a manual hammer for Borehole BH3 and BH4 and carried out in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586, Standard Test Method for Standard Penetration Test). Field vane shear tests were completed, where ground conditions warranted such tests, in accordance with ASTM D2573 (Standard Test method for Filed Vane Shear Test in Fine-Grained Soils). Dynamic Cone Penetration Tests (DCPT) were also completed from the bottom of each borehole. All boreholes were backfilled with bentonite and cuttings upon completion in accordance with Ontario Regulation 903 Wells (as amended).

All boreholes were sampled to a depth of 6.7 to 7.3 m below ground surface; and the DCPTs were advanced beyond the sampling depth in all boreholes to refusal which ranges from 8.3 m to 11.4 m below ground surface.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets provided in Appendix A.

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 cared for the soil 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. The results of the laboratory testing on samples from the boreholes are presented on the Record of Borehole sheets and are included in Appendix B.

The location of the boreholes was measured relative to known points (the bridge piers). The elevation and coordinates of the borehole locations were obtained by referencing the measured borehole locations to the digital terrain mapping shown on the drawings. The borehole locations given in the Record of Borehole sheets and shown on Drawing 1 are positioned relative to MTM NAD 83 northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and borehole and DCPT depths are as follows:

Borehole	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)	DCPT Termination Depth (m)
	Northing	Easting			
BH1	5149845.7	318819.3	253.9	7.3	8.3
BH2	5149825.5	318800.2	254.1	6.7	10.2
BH3	5149846.5	318844.3	254.0	7.2	8.7
BH4	5149824.0	318822.5	254.4	7.3	11.4





## 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 A. The results of geotechnical laboratory testing are contained in Appendix B. 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 Drawing 1 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<sup>2</sup>. 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<sup>3</sup>.

### 4.2 Subsoil Conditions

In general, the subsoil conditions encountered at the borehole location consist of a surface layer of variable fill material underlain by native deposits of organic clay, in places, sandy silt or clayey silt which in turn are underlain by a stratum of varved clayey silt to clay underlain by a deposit of silt and sand in places. A more detailed description of the soil deposits and groundwater conditions encountered in the boreholes is provided below.

Deposit/Layer Description	Boreholes	Deposit Thickness (m)	Deposit Surface Elevation (m)	N Values (blows)	Laboratory Testing
				Field Vane Results (kPa)	
				Consistency or Relative Density	
<b>(FILL) Gravelly Silty Sand to Sand and Gravel<sup>1</sup></b> , trace organics; trace to some cobbles <sup>2</sup> ; brown to black; Frozen to wet	BH1 to BH4	0.7 – 1.5	254.4 – 253.9	N = 7 – 13 <sup>3</sup> (where not frozen)	w = 12%, 23% 1 - M (Fig. B1)
				N/A	
				<b>Loose to Compact</b>	
<b>Organic Clay, Clayey Silt, Sandy Silt</b> , trace to some fibrous organics, trace to some gravel; brown to black to grey; moist to wet	BH1 to BH3	0.7 – 1.5	253.4 – 252.5	N = 9 to 23	w = 24% - 35% 1 – MH (Fig. B2)
				N/A	
				<b>Very stiff/loose</b>	

<sup>2</sup> Garnet, J.F., 1980. Sudbury Area (NTS 411/SE) District of Nipissing, Parry Sound and Sudbury; Ontario Geologic Society, Northern Ontario Engineering Geology Terrain Study 100.

<sup>3</sup> Ibid.



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Deposit/Layer Description	Boreholes	Deposit Thickness (m)	Deposit Surface Elevation (m)	N Values (blows)	Laboratory Testing
				Field Vane Results (kPa)	
				Consistency or Relative Density	
<b>Clayey Silt to Clay</b> , trace sand, varved; brown/grey to grey; moist to wet	BH1 to BH4	4.6 – >5.8 <sup>4</sup>	252.9 – 251.7	N = 0 (weight of hammer) to 9 S <sub>u</sub> = 30 to 98 Sensitivity = 2 to 5 <b>Firm to Stiff</b>	w = 11% - 49% w <sub>l</sub> = 29% - 53% w <sub>p</sub> = 20% - 24% I <sub>p</sub> = 9% - 30% 3 – MH (Fig. B3) 10 – AL (Fig. B4)
<b>Silt and Sand</b> , trace clay; brown-grey; wet	BH2	> 0.7	248.1	4 N/A <b>Very Loose to Loose</b>	w = 24% 1 – MH (Fig. B5)

### Where:

N = SPT 'N'-value; number of blows for 0.3 m of penetration

w = Natural Moisture Content (%)

MH = Combined Sieve and Hydrometer analysis

M = Sieve analysis for particle size

AL = Atterberg Limits Test

w<sub>p</sub> = Plastic Limit (%)

w<sub>l</sub> = Liquid Limit (%)

I<sub>p</sub> = Plasticity Index (%)

NP = Non-Plastic test result

SPT Refusal = greater than 100 blows per 0.3 m of penetration

### Notes:

<sup>1</sup> Silty clay layers 75 mm and 125 mm thick were observed in the fill material encountered at BH1

<sup>2</sup> Cobbles were observed in drill cutting (spoil) and inferred from observation of drilling progress.

<sup>3</sup> SPT N-values of 46 and 58 blows per 0.3 m of penetration were measured at the ground surface in Boreholes BH1 and BH4 and are attributed to frozen conditions.

<sup>4</sup> Sampling in BH1, BH3 and BH4 was terminated in the clayey silt to clay stratum. The deposit thickness reported in the above table may not be representative of the full thickness of the deposit as sampling did not extend beyond the end of the stratum. Dynamic Cone Penetration Tests (DCPT) were extended beyond the bottom of the boreholes to refusal (actual or inferred 100 blow per, or for less than, 0.30 m of penetration).

Based on the results of the DCPTs, previous geotechnical investigations at the site, and published geological information, the DCPTs were driven to "Refusal" on the inferred bedrock surface, as summarized below for each borehole.





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Borehole	Depth of DCPT Depth to Refusal (m)	Elevation of DCPT Refusal (m)
BH1	8.3	245.6
BH2	10.2	243.9
BH3	8.7	245.3
BH4	11.4	243.0

### 4.3 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 as part of the tripod rig drilling process in Boreholes BH3 and BH4 likely impacted the measured groundwater levels. Water levels may vary depending on the time of year and precipitation events.

Borehole	Ground Surface Elevation (m)	Groundwater Depth (mbgs)	Groundwater Elevation (m)
BH1	253.9	1.5	252.4
BH2	254.1	3.1	251.0
BH3	254.0	0.6	253.4
BH4	254.4	0.3	254.1

### 5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. David Marmor E.I.T., 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 Principal of 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**

**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**



## **6.0 DISCUSSION AND ENGINEERING RECOMMENDATION**

### **6.1 General**

This section of the report provides foundations engineering recommendations for the detail design of the retaining soil system (RSS) walls to be constructed as part of the rehabilitation of the Coniston Canadian Pacific Railway (CPR) Overhead Structure on Highway 17, in the Sudbury area. The Ministry of Transportation, Ontario (MTO) retained Morrison Hershfield (MH) to complete the design of rehabilitation works of the CPR Coniston overhead; and retained Golder Associates Ltd. (Golder) under a retainer assignment (Northeastern Region Retainer Agreement 5013-E-0034) to complete a foundation investigation and provide foundation engineering recommendations for the design of the RSS walls. Based on the drawings provided by MH, Golder has assumed that the structure rehabilitation design is being completed to the Canadian Highway Bridge Design Code (CHBDC) 2006 edition. As such, Golder has completed this report to CHBDC 2006 as well.

The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to carry out the detail design of the RSS retaining walls. Where comments are made on construction, they are provided to highlight those aspects which could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing CPR Overhead is a three span, concrete and steel beam bridge, supported on abutments and concrete piers and is generally oriented east-west. The bridge supports Highway 17 (two lanes) as it passes over the CPR tracks and right-of-way. As part of the rehabilitation of the Coniston CPR overhead an RSS wall will be constructed at each of the east and west piers of the bridge as shown in the general arrangement drawings of the rehabilitation design provided to Golder as part of the Terms of Reference (TOR).

Based on the subsurface conditions encountered and the design drawings, an RSS wall is considered geotechnically feasible at both the east and west piers as this wall type should be able to accommodate the magnitude of the estimated settlement at the site. The incorporation of slip joints in the RSS wall facing panels are recommended to accommodate potential differential settlement that may develop along the length of the walls if the proprietary wall design/manufacturing allows for such joint details. The subexcavation of unsuitable material below the footprint of the wall front facing footing and reinforced soil mass along the toe of slope for this wall option is not expected to be as deep as would be required for shallow foundations of other retaining wall options, such as concrete retaining walls. However, some excavation must still be completed within the zone of the reinforced soil mass and facing footing. Based on the design drawings provided, the maximum fill height behind the wall is approximately 3.0 m and the width of the reinforced soil mass is about 2.4 m for the lower portion of the RSS wall and stepped to a width of about 4.4 m above the lower portion; the maximum width of backfill behind the wall panels is 8 m, including the reinforced soil mass, placed, beyond the reinforced soil mass in steps about 2 m wide and 0.7 m to 0.9 m high.

Based on the requirements set out in the TOR for this project, the design drawings provided, and the feasibility of constructing RSS walls at this site, detailed design recommendations are not provided herein for soil retaining systems other than RSS walls. The design of the RSS Wall system should be consistent with MTO's RSS Design Guidelines (2008).



## **6.2 Foundation Recommendations for RSS Walls**

### **6.2.1 Founding Elevations**

A typical RSS wall has front facing panels supported on a concrete footing (concrete leveling pad as may be required by the RSS supplier) constructed on a compacted granular pad founded at a shallow depth below the ground surface in front of the reinforced soil mass. The granular pad should consist of compacted OPSS.PROV 1010 (*Aggregates*) Granular 'A' material that should extend at least 1.0 m beyond the outside edge of the concrete leveling pad (footings), then outward/downward at 1 horizontal to 1 vertical (1H:1V). At this specific site, the need for the concrete facing panels should be evaluated by the Owner/Design Engineer and proprietary RSS Wall Supplier as reportedly a crash wall is to be constructed in front of the reinforced soil mass. In addition, due to the presence of pile caps at the bridge piers, the orientation and final dimensions of the concrete leveling pad may need to be adjusted. Excavation in front (track side) of pile caps is not recommended.

The facing panels footing/levelling pad and the reinforced soil mass should be founded below any existing topsoil, existing fill and any unsuitable soils. Where present, such materials should be subexcavated and replaced with compacted granular fill, prior to constructing the reinforced soil mass and levelling pad, to improve the performance of the walls since differential settlement is anticipated to occur as discussed in Section 6.2.2. Once subexcavation and backfilling is completed, the RSS wall levelling pad/footing should be founded at a minimum depth of 0.5 m below the backfilled grade, on a minimum 0.3 m thick granular pad.

The unstabilized water levels along the proposed wall alignment as measured in open holes at the completion of drilling, ranges from Elevations 254.1 m to 251.0 m. The depth of subexcavation will likely extend to below the groundwater level at the site depending on stabilized groundwater levels at the time of construction. It is recommended that once the existing fill, organics and any unsuitable fill is removed, the excavation should be backfilled immediately with OPSS.PROV 1010 (*Aggregates*) Granular 'B' Type II material below the groundwater table as the excavation progresses (see Section 6.3).

Based on the borehole information, the following subexcavation base levels are recommended to remove existing fill materials and organics to improve the settlement performance of the walls:

<b>Retaining Wall Location</b>	<b>Corresponding Boreholes</b>	<b>Ground Surface Elevations at Borehole Locations (m)</b>	<b>Depth of Subexcavation Below Ground Surface (at Borehole Locations) (m)</b>	<b>Subexcavation/ Granular Pad Founding Elevation (m)</b>
West Pier	BH1, BH2	253.9 to 254.1	2.0 to 1.0	251.9 to 253.0
East Pier	BH3, BH4	254.0 to 254.4	1.1 to 1.5	252.9

To maintain the stability of the existing bridge embankments front slopes, the subexcavation and backfilling operations should be completed in strips of limited width, as per the example NSSP provided in Appendix C. The subexcavation should be backfilled with OPSS.PROV 1010 Granular B Type II material in wet subgrade conditions, and Granular A material in dry conditions, to ground surface and the reinforced soil mass, concrete leveling pad and facing panels may then be constructed as described above – i.e., facing leveling pad/footing founded 0.5 m below the finished grade in front of the wall, with a minimum 300 mm thick granular pad below the facing footing.



For this site, the RSS walls may/could be designed for low performance and appearance in accordance with MTO Special Provision (SP) 599S22 (*Retained Soil System*). Further, depending on the configuration/location of the crash wall, the need for and type of facing panel should be discussed with the Owner/Designer and Proprietary product supplier.

### 6.2.2 Settlement

The estimated settlement of the subgrade due to the loading imposed by the approximately 3.0 m of new fill behind the RSS walls has been estimated using both hand calculations and the commercially available computer program Settle-3D from Rocscience. These 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 and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Elastic Modulus (MPa)	Undrained Shear Strength(kPa)	Recompression Index	Coefficient of Consolidation (cm <sup>2</sup> /sec)
Reinforced Soil Mass	22	-	-	-	-
New Granular Backfill for sub-excavation areas	21	80	-	-	-
Varved Clayey Silt to Clay	17	15 - 5	75 to 35	0.05	0.012 to 0.015

The estimated settlement of the subgrade under the reinforced soil mass and facing footing is estimated to be between 40 mm and 65 mm at the west wall and 55 mm to 60 mm at the east wall, with the higher magnitude of settlement occurring in areas where the varved clayey silt to clay deposit is thicker. It is recommended that vertical slip joints be incorporated into the RSS wall facing panels to accommodate potential 25 mm and 5 mm differential settlement along the west and east wall face, respectively.

The majority of the subgrade settlement is attributed to the consolidation of the varved clayey silt to clay subgrade soil which underlies the existing fill at the site. Based on the analysis for the 3.0 m of fill, the clayey silt to clay deposit is overconsolidated and settlement is estimated to reach 90% consolidation within 6 months to 7 months of the completion of placement of the reinforced soil mass for the RSS wall.

In addition, settlement of the sub-aqueously placed granular replacement fill, if such condition is encountered, should occur during placement of the RSS wall fill, and is expected to be less than 25 mm.

### 6.2.3 Geotechnical Resistance

For the RSS concrete leveling pad footing constructed on a compacted granular pad after the required sub-excavation and backfilling is complete, the wall design may be completed based on a factored geotechnical resistance at ULS of 150 kPa and a geotechnical reaction at SLS (for 25 mm of settlement) of 150 kPa.

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass (which can be taken as 4.4 m, for the first two steps of the wall as shown on the design drawings), and an average elastic modulus (E) for the ground of 10 MPa, a factored geotechnical axial resistance at ULS of 150 kPa and a geotechnical reaction at SLS (for 25 mm of settlement) of 35 kPa may be used for design of the RSS wall founded on the properly prepared subgrade as discussed above. However, it must be noted that the post-construction settlement of the





RSS walls is estimated to be between 40 mm and 65 mm for the west wall and 55 mm to 60 mm for the east wall, for the actually required wall height of 3.0 m imposing a loading of about 66 kPa per unit area.

### 6.2.4 Resistance to Lateral Loads / Sliding Resistance

Resistance to lateral forces/sliding resistance between the compacted fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the *Canadian Highway Bridge Design Code and its Commentary 2006 (CHBDC 2006)*. The coefficient of friction,  $\tan \phi'$ , between the compacted granular fill of the RSS wall mass and the properly prepared fill subgrade as outlined in Section 6.2.1 may be taken as 0.67. This represents an unfactored value. The actual values used should be reviewed and revised, if necessary, by the proprietary RSS supplier for their detail design of the wall. Similarly, the coefficient of friction  $\tan \delta$  between the concrete levelling pad/footing and the granular pad may be taken as 0.60.

### 6.2.5 Global Stability

The static global stability analyses for the embankment front slopes incorporating an RSS wall at the toe of slope, was carried out by the commercially available computer program SLOPE/W from Geo-Slope International Ltd. using the parameters outlined below and assume that all existing topsoil and organics are completely removed prior to constructing the RSS walls.

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Effective Friction Angle (Degrees)	Undrained Shear Strength (kPa)
Existing Embankment Fill	22	30	-
Existing Fill and Surficial Silt, Clayey Silt, and Organic Soil at Toe of Slope	20	29	-
Varved Clayey Silt to Clay	17	-	75 to 35
New Granular B Backfill (to replace subexcavated soil)	21	34	-
New Granular A Backfill	22	35	-
RSS Wall Facing Panel	24	100	-
Reinforced Soil Mass	22	100	-

The design drawings provided show an embankment front slope inclined at approximately 3 horizontal to 1 vertical (3H:1V) and a plan cross-section skewed to the bridge abutments and piers. The global stability analysis has been completed for an embankment front slope inclined at 2H:1V, which is assumed to be the critical case where the slope is perpendicular to the bridge abutments and piers.

The results of the static global stability analyses indicate that a Factor of Safety of 1.5 is achieved for a RSS wall up to 3.0 m high (retained soil height), as shown on Figure 1. It should be noted that the internal stability of the reinforced soil wall is to be designed and assessed by the proprietary product supplier.

### 6.2.6 Loading on Existing Foundations

Based on the previously completed geotechnical report for this site completed by the Ministry of Transportation and Communication in 1977 (Geocres #: 41I-140), the existing foundations of the overhead structure consist of



HP 12x102 Steel H-Piles driven to bedrock. The bridge piers are supported by a pile cap which connects the Steel H-Piles. Based on the General Arrangement drawings provided by MH it appears that the bottom of the pile caps at the west and east piers are at approximately Elevations 252.0 m and 252.5 m, respectively. Based on the referenced MTO geotechnical report, the lowest elevation that bedrock was encountered was at approximately Elevation 241.1 m.

Based on the general arrangement drawings provided by MH, the RSS Walls will be constructed on the outside of the existing piers. The additional load from the retained soil mass may cause unbalanced loads to develop on the existing pile caps. The structural engineer should verify the capacity of the pile cap is not exceed by the addition of the proposed RSS Wall loadings.

As discussed in Section 6.2.2 the retained soil mass will cause differential settlement of the underlying native soil which in turn will cause differential movement between the native soils and the piles resulting in downdrag (negative skin friction) on the piles. A preliminary analysis of the dragloads for the HP 12x102 piles was completed based on the "alpha" method as described in Chapter 18 of the Canadian Foundation Engineering Manual (CFEM), 4<sup>th</sup> Edition, 2006. The assumptions and result of the analysis are presented below.

Undrained Shear Strength of Compressible Layer (kPa)	Estimated Maximum Length of Pile (m)	Perimeter Length of Pile (m)	Unfactored Dragload, Negative Skin Friction (kN)
75 to 35	11.4	1.3	525

The dragload provided above is an unfactored value. In accordance with CHBDC, 2006 the structural designer should verify that the structural axial capacity of the pile at ULS is not exceeded when the pile is subjected to the exiting dead (permanent) load plus the dragload noted above. It should be noted that the dragload is to be used for evaluating the structural axial capacity only and that it has no effect on the geotechnical axial resistance of the pile.

## 6.3 Construction Considerations

### 6.3.1 Excavation and Temporary Protection Systems

The excavations for front facing strip footing/levelling pad and subexcavation for the RSS wall mass will extend through the existing fill and into loose sandy silt and very stiff organic clay and clayey silt deposits, as well as potentially into the stiff to firm clayey silt to clay deposit. 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. Organic soil deposits would be classified as Type 4 soils. 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 excavation walls.

As noted in Section 6.2.1, the excavation may extend below the groundwater level and the excavated materials should be replaced with OPSS.PROV 1010 (*Aggregates*) Granular 'B' Type II material to the founding elevation of the wall. This sub-excavation should be backfilled immediately as noted in OPSS.PROV 209 (*Embankments over Swamps*) for fill placement in wet conditions or below water.



Excavations to prepare for the construction of the reinforced soil mass will extend behind the face of the RSS wall for a distance of approximately 2.4 m (based on design drawings), extending into the toe of the embankment front slope and resulting in temporary steepening (i.e. temporary cut slope) to, locally, about 1H:1V. To maintain stability, the subexcavation should be limited to short strips of no greater than about 3 m wide and the subexcavation and backfilling operations should be carried out essentially simultaneously. An NSSP should be included in the Contract Documents to require such operations; an example is included in Appendix C.

Should the overall temporary cut into the front slope of the abutment embankments be required to be inclined steeper than about 1H:1V, then temporary protection systems will be required to maintain stability of the excavation walls and embankment slope during subexcavation and construction of the RSS wall. 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 1b as specified in OPSS.PROV 539, to avoid excessive movement of the abutments.

### 6.3.2 Groundwater Control

The subexcavation required at the west and east RSS wall locations will extend about 0.5 m and 1.2 m, respectively, below the groundwater table, depending on stabilized groundwater levels at the time of construction. As discussed in Section 6.3.1, the excavation and backfilling operation are planned to be carried out subaqueously and essentially simultaneously.

In the event that stabilized groundwater levels at the RSS wall locations are below the depth of excavation at the time of construction, some seepage from granular fill materials will likely be encountered. This near-surface water seepage is expected to be relatively small, such that the water inflow can be handled by pumping from filtered sumps placed at the base of the excavation. A Permit to Take Water (PTTW) is not anticipated to be required for the temporary groundwater control at this site.

## 7.0 CLOSURE

This Foundation Design Report was prepared by Mr. David Marmor E.I.T., 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 Principal of Golder and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.



## Report Signature Page

### GOLDER ASSOCIATES LTD.

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DPM/SEMP/JMAC/nh

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\\golder.gds\gal\whitby\active\\_2014\1181- geotechnical & pavement\14-1181-0014 mto eoi 5013-e-0034 ner retainer\assignment 15\report\14-1181-0014.15000 draft fidr 2016\04'21.docx



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Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.

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

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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.

Geocon Ltd., 1977. Foundation Investigation Report for CPR Overhead at Coniston W.P. 158-74-01, Site 46-123, Hwy. 17, District 17, Sudbury. Prepared by the Ministry of Transportation and Communication, Ontario. Geocres No. 411-140.

Kulhawy, F.H. and Mayne, P.W. 1990. Manual on Estimating Soil Properties for Foundation Design. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

Ministry of Transportation, Ontario, RSS Design Guidelines, Engineering Standards Branch, September 2008.

Hanson, Walter E., Peck, Ralph B., and Thornburn, Thomas H., 1974. *Foundations Engineering*, 2<sup>nd</sup> Edition, John Wiley and Sons, New York.

### Ontario Provincial Standard Specifications (OPSS)

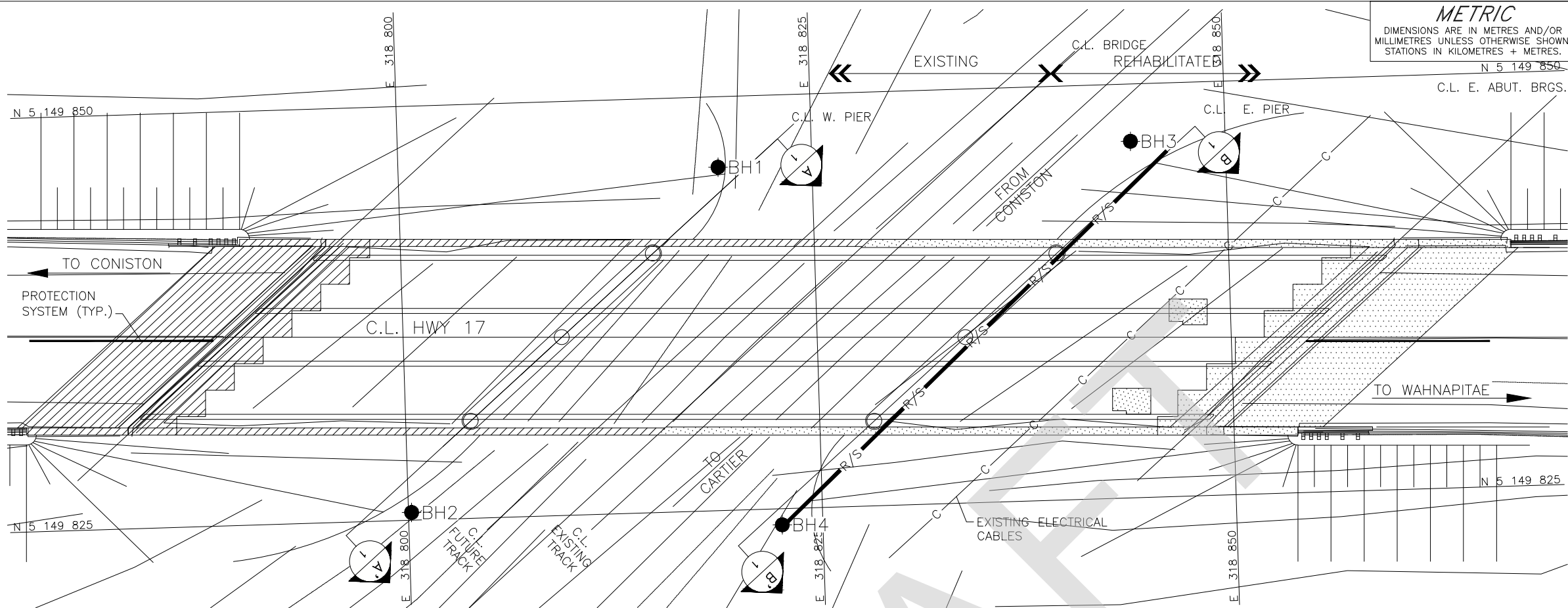
OPSS.PROV 209	Construction Specification for Embankments over Swamps and Compressible Soils
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
SP 599S22	Special Provision for Retained Soil System; Wall/Slope

### Commercial Software

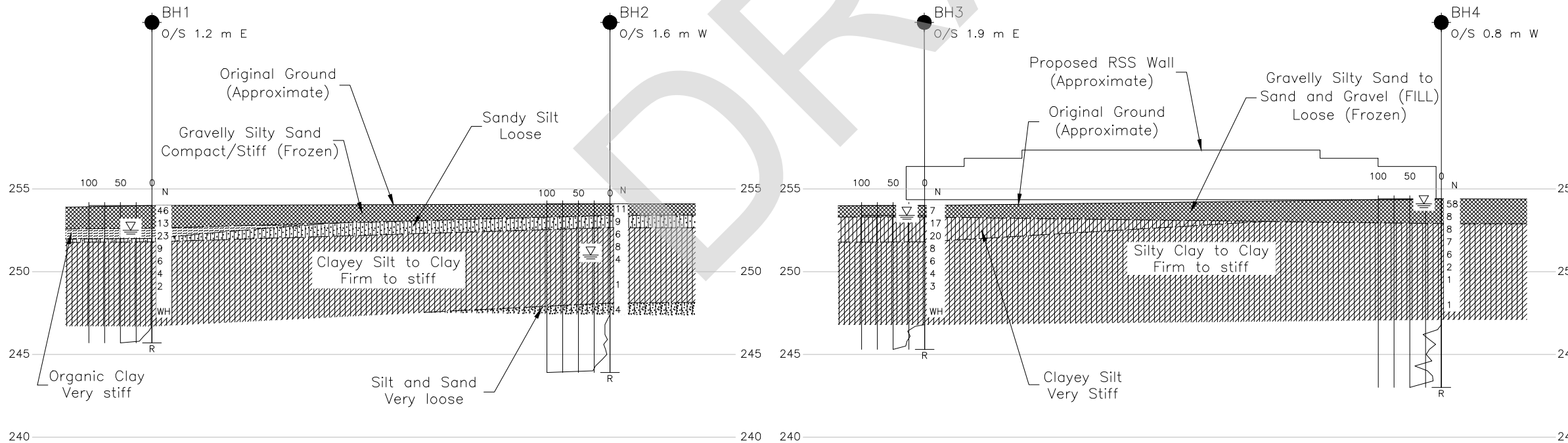
SLOPE/W (Version 7.23) by Geo-Slope International Ltd.

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





PLAN SCALE  
0 3 6 m



PROFILE A-A'  
1  
HORIZONTAL SCALE  
0 3 6 m  
VERTICAL SCALE  
0 3 6 m

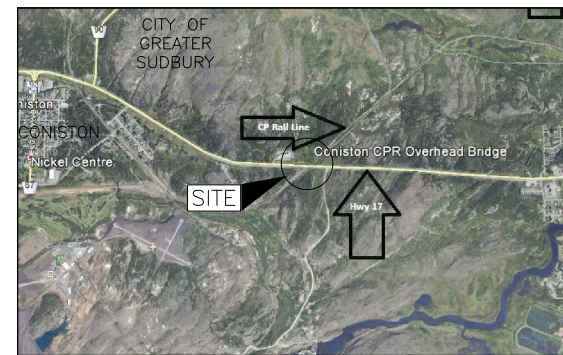
PROFILE B-B'  
1  
HORIZONTAL SCALE  
0 3 6 m  
VERTICAL SCALE  
0 3 6 m

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

CONT No. 5165-10-01  
WP No. 5165-10-01



**HIGHWAY 17**  
CONISTON CPR OVERPASS RETAINING WALLS  
**BOREHOLE LOCATION PLAN AND SOIL STRATA**



KEY PLAN SCALE  
0 500 1000 m

**LEGEND**

- Borehole - Current Investigation
- R/S — R.S.S. Wall
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
- ∇ 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

**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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

**REFERENCE**

Base plans provided in digital format, drawing file no. 46-123\_01.dwg, dated SEP, 2015, received JAN, 2016.

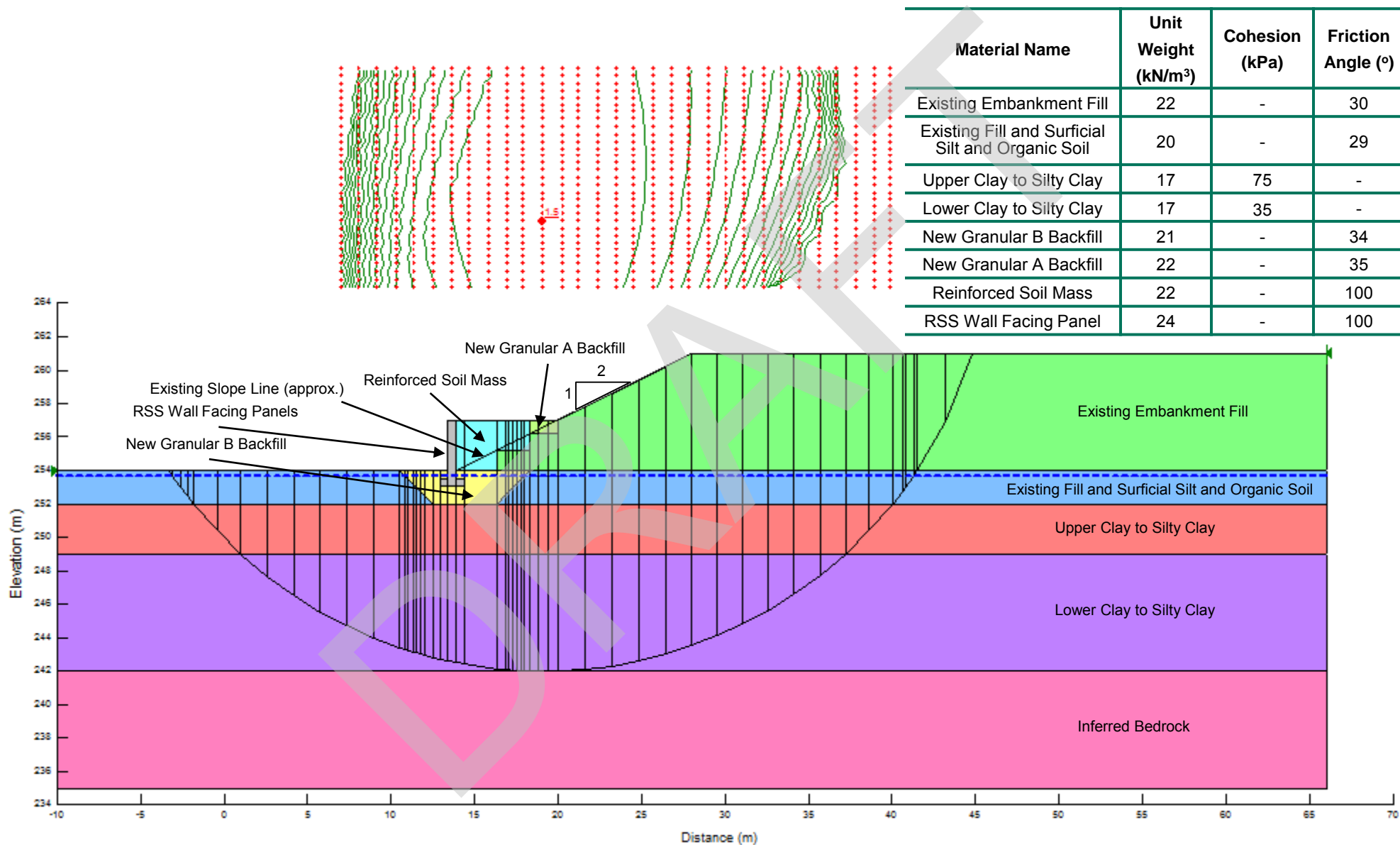
NO.	DATE	BY	REVISION
Geocres No.,			
HWY. 17		PROJECT NO. 14-1181-0014	DIST. .
SUBM'D.,	CHKD. DM	DATE: 4/18/2016	SITE: 46-123
DRAWN: TB	CHKD. SEMP	APPD. JMAC	DWG. 1

**DRAFT**



# Stability Analysis RSS Wall – East Embankment Front Slope

Figure 1



Date: April, 2016

Project No: 14-1181-0014 Phase 15000

Analysis By: DPM Reviewed By: SEMP



# **APPENDIX A**

## **Borehole Records**

DRAFT



## LIST OF SYMBOLS

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

### I. GENERAL

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

### II. STRESS AND STRAIN

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

### III. SOIL PROPERTIES

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

### (a) Index Properties (continued)

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

### (b) Hydraulic Properties

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

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

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

### (d) Shear Strength

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

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

Notes: 1  
2

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



## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### II. PENETRATION RESISTANCE

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

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

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

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

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

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

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

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

### III. SOIL DESCRIPTION

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

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

#### (b) Cohesive Soils Consistency

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

### IV. SOIL TESTS




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

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

### V. MINOR SOIL CONSTITUENTS

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

PROJECT 14-1181-0014			RECORD OF BOREHOLE No BH1			1 OF 1 METRIC															
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			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W <sub>p</sub> — W — W <sub>L</sub>			γ			GR SA SI CL		
253.9	GROUND SURFACE							20 40 60 80 100													
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		253														
252.5	- 125 mm thick silty clay layer encountered at 1.2 m depth.																				
1.4	ORGANIC CLAY, some sand, trace fibrous peat Very stiff		A	SS	23		252														
251.9	Grey to black Moist		3	B																	
2.2	Sandy SILT, trace gravel Grey Wet		4	SS	9		251														
	CLAYEY SILT to CLAY, trace to some sand, varved Firm to stiff Brown/grey Moist to wet		5	SS	6		250														
			6	SS	4		249														
			7	SS	2		248														
	Becoming grey below 6.1 m depth.		8	SS	WH		247														
246.6	END OF BOREHOLE START OF DCPT						246														
7.3																					
245.6	END OF DCPT DCPT REFUSAL (50 blows/0.08m)																				
8.3	Note(s):  1. Water level at a depth of 1.5 m below ground surface (Elev. 252.4 m) upon completion of drilling.																				

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										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100	20
254.1	GROUND SURFACE						254											
0.0	Gravelly silty sand (FILL) Compact Brown Frozen		1	SS	11		254											
253.4							253											
0.7	Sandy SILT, some gravel, trace organics Loose Brown-grey to black Moist to wet		2	SS	9		253											
252.7							252											
1.4	CLAYEY SILT to SILTY CLAY, trace sand, varved Firm to stiff Brown/grey Moist to wet		3	SS	6		252											
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							150											



[illegible]

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

PROJECT <u>14-1181-0014</u>		<b>RECORD OF BOREHOLE No BH4</b>		1 OF 2 <b>METRIC</b>																			
G.W.P. _____		LOCATION <u>N 5149824.0; E 318822.5</u>		ORIGINATED BY <u>DM</u>																			
DIST _____ HWY <u>17</u>		BOREHOLE TYPE <u>NW Casing, Portable Equipment</u>		COMPILED BY <u>TB</u>																			
DATUM <u>GEODETIC</u>		DATE <u>January 20, 2016</u>		CHECKED BY <u>SEMP</u>																			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)									
254.4	GROUND SURFACE						20 40 60 80 100			20 40 60			kN/m <sup>3</sup>	GR SA SI CL									
0.0	Sand and gravel, some fines, trace organics, some cobbles (FILL) Loose Dark brown to brown Frozen to wet		1	SS	58	254				○				38 49 (13)									
252.9			2	SS	8	253																	
1.5	SILTY CLAY, trace sand, varved Firm to stiff Brown/grey Moist to wet		3	SS	8	252				○													
			4	SS	7	251				○													
			5	SS	6	250				○													
			6	SS	2	249				○													
			7	SS	1	248				○													
			8	SS	1	247				○													
247.1	END OF BOREHOLE START OF DCPT					246																	
7.3						245																	
						244																	
243.0						243																	
11.4																							

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

Continued Next Page

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



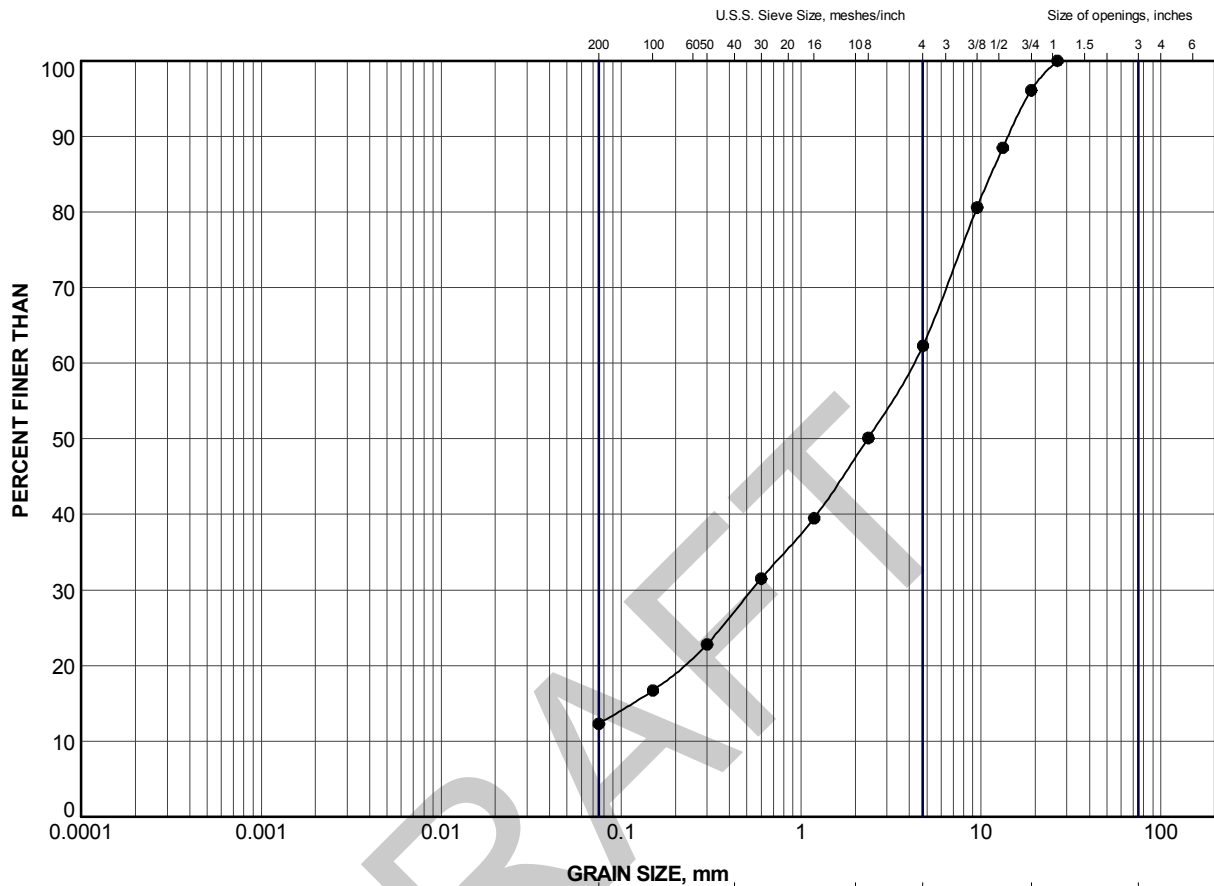
+3, ×3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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



## **APPENDIX B**


### **Geotechnical Laboratory Test Results**

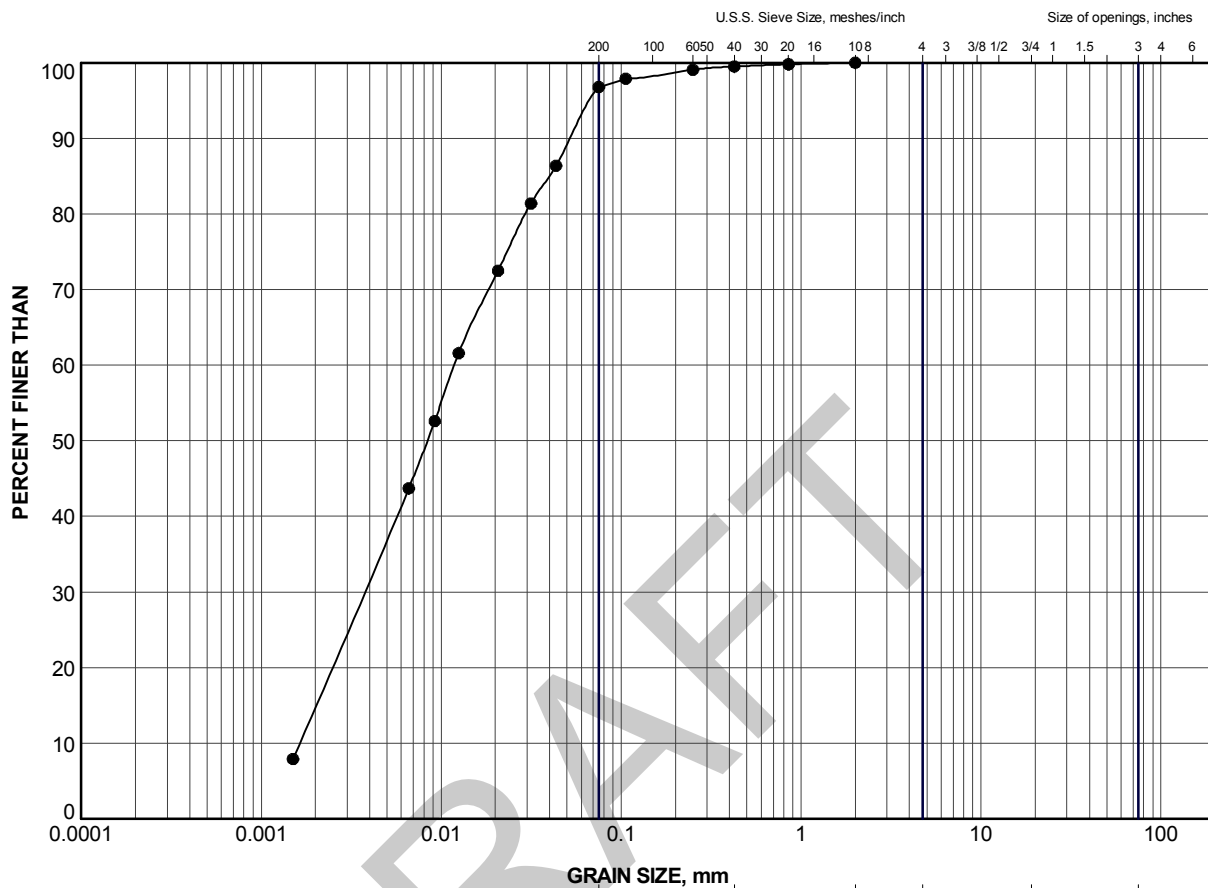


CLAY AND SILT	SAND SIZE, mm						Cobble Size
	fine	medium	coarse	fine	coarse		
	SAND SIZE			GRAVEL SIZE			

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH4	1	254.1

PROJECT				
HIGHWAY 17 CONISTON CPR OVERPASS - ASSIGNMENT 15				
TITLE				
GRAIN SIZE DISTRIBUTION SAND and GRAVEL (FILL)				
PROJECT No.		14-1181-0014		14-1181-0014 CONISTON A15.GPJ
DRAWN	TB	Apr 2016	SCALE	N/A
CHECK	DM	Apr 2016	REV.	
APPR	JH	Apr 2016		
 <b>Golder Associates</b> SUDBURY, ONTARIO			<b>FIGURE B1</b>	



CLAY AND SILT	SAND SIZE, mm			GRAVEL SIZE, mm		Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH3	2	252.9

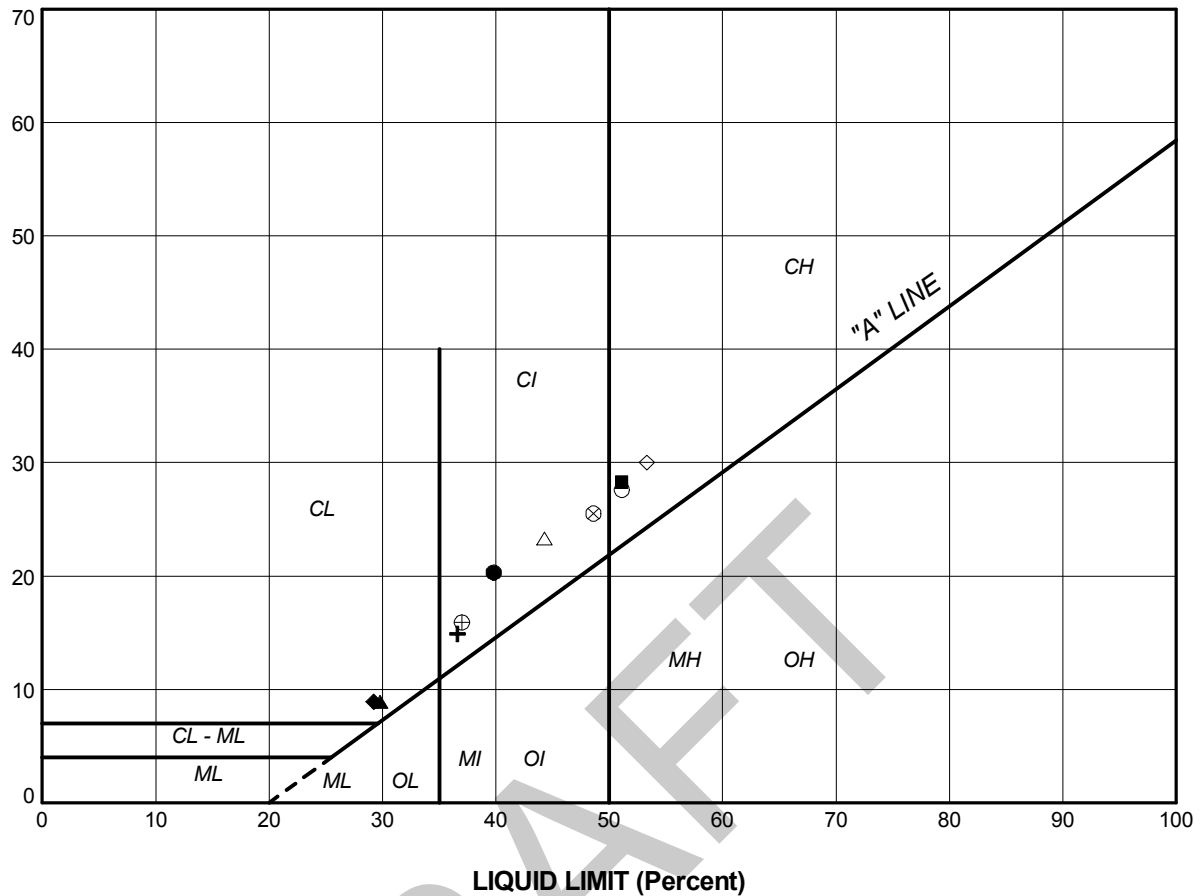
PROJECT					HIGHWAY 17 CONISTON CPR OVERPASS - ASSIGNMENT 15							
TITLE					GRAIN SIZE DISTRIBUTION CLAYEY SILT							
PROJECT No.		14-1181-0014		14-1181-0014 CONISTON A15.GPJ		DRAWN		TB	Apr 2016	SCALE	N/A	REV.
CHECK		DM		Apr 2016		APPR		JH	Apr 2016	FIGURE B2		







PLASTICITY INDEX (Percent)



**SOIL TYPE**  
C = Clay  
M = Silt  
O = Organic

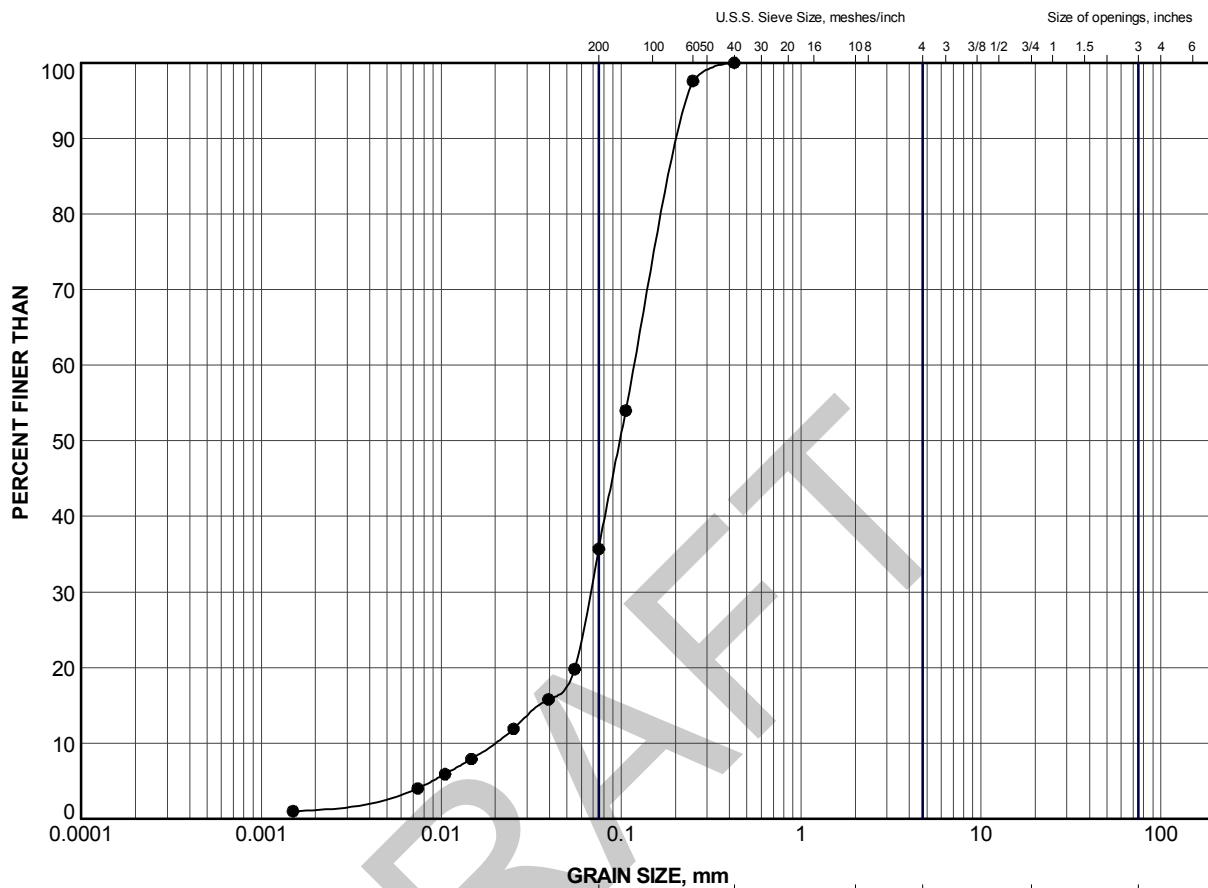
**PLASTICITY**  
L = Low  
I = Intermediate  
H = High

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	BH1	4	39.8	19.5	20.3
■	BH1	6	51.1	22.8	28.3
▲	BH1	8	29.8	20.9	8.9
+	BH2	4	36.6	21.7	14.9
◆	BH2	6	29.2	20.3	8.9
◇	BH3	4	53.3	23.3	30.0
○	BH3	7	51.1	23.5	27.6
△	BH4	4	44.3	21.0	23.3
⊗	BH4	6	48.6	23.1	25.5
⊕	BH4	8	37.0	21.1	15.9

PROJECT				
HIGHWAY 17 CONISTON CPR OVERPASS - ASSIGNMENT 15				
TITLE				
PLASTICITY CHART CLAYEY SILT to CLAY				
PROJECT No.		14-1181-0014		FILE No. 11810014 CONISTON A15.GPJ
DRAWN	TB	Apr 2016	SCALE	N/A
CHECK	DM	Apr 2016	REV.	
APPR	JH	Apr 2016	FIGURE B4	






CLAY AND SILT	SAND SIZE, mm			GRAVEL SIZE, mm		Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	BH2	7	247.7

PROJECT					
HIGHWAY 17 CONISTON CPR OVERPASS - ASSIGNMENT 15					
TITLE					
GRAIN SIZE DISTRIBUTION SILT and SAND					
		PROJECT No.		14-1181-0014	
		DRAWN		TB	Apr 2016
		CHECK		DM	Apr 2016
		APPR		JH	Apr 2016
		SCALE		N/A	
		REV.			
		<b>FIGURE B5</b>			



## **APPENDIX C**

### **Non-Standard Special Provisions**

DRAFT

## **EARTH EXCAVATION, GRADING – Item No.**

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### **Special Provision**

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This special provision outlines the procedure to be used for excavation of the existing fill and surficial soil deposits containing organics under the RSS wall footprint along the toe of the front slopes of the Coniston CPR Overhead structure. The depth/elevation of subexcavation along the various areas of the RSS walls are shown on the Contract Drawings.

Staged excavation in strips of limited width shall be carried out to maintain the stability of the excavation and the existing bridge approach embankment front slope during the subexcavation and backfilling operations. The staged excavation procedures are outlined as follows:

- a) The work should be carried out from one end of area to be subexcavated and progressively towards the other end.
- b) Removal of the existing fill and surficial native deposits containing organic soil from within the footprint of the reinforced soil mass of the RSS wall shall be carried out in short “strip” sections excavated perpendicular to the railway tracks, with the base of the excavation (as measured parallel to the railway tracks) not wider than approximately 3 m.
- c) Temporary excavation side slopes or back slopes through the existing fill and native deposits containing organic material 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. Organic soil deposits would be classified as Type 4 soils. 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.
- d) Excavation and backfilling operations shall be carried out simultaneously in a manner that the excavation is not left open for more than the approximately 3 m wide “strip” width at any given time.

The Contractor shall maintain the operation of the Highway 17 and the railway line during excavation and backfilling operations including and not limited to the utilization of temporary concrete barriers and/or traffic control, as required.

Payment for the Contractor to provide the above requirements, including all equipment, labour and materials shall be deemed to be included in the contract bid price for the various tender items.

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