



## Foundation Investigation and Design Report

*Highway 28, Eel's Creek Bridge Replacement, North Kawartha, Ontario  
MTO Agreement No. 4014-E-0012, Assignment No. 15*

Submitted to:

**Ministry of Transportation, Ontario - Foundations Section**

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July 15, 2019

**GEOCRES No. 31D-727**

*Latitude: 44.640834*

*Longitude: -78.135905*



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# PART A

FOUNDATION INVESTIGATION REPORT  
HIGHWAY 28, EEL'S CREEK BRIDGE REPLACEMENT (SITE NO. 26-117)  
NORTH KAWARTHA, ONTARIO  
MINISTRY OF TRANSPORTATION, ONTARIO

## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the detail design of the Eel's Creek Bridge replacement project (Site No. 26-117) located on Highway 28, approximately 200 m south of Eel's Creek Park Road in North Kawartha, Ontario.

The purpose of this investigation is to establish subsurface soil, bedrock and groundwater conditions at the proposed structure replacement by borehole/probehole drilling, vertical seismic profiling (VSP) and geotechnical/analytical laboratory testing on selected soil and bedrock samples.

The Terms of Reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal titled "To Provide Foundations Engineering Services on Retainer, Various Locations, Assignment No. 4014-E-0012 & 4014-E-0013, Eastern Region" dated October 2014 and associated clarifications. The detailed Scope of Work for this assignment is presented in the Golder's *Understanding of Scope* documents for Assignment Number 4014-E-0012, Work Item No. 15 and the associated Work Item Quote Form. Supplementary foundation investigation services were provided under a similar understanding of scope document prepared for Assignment Number 4017-E-0048, Work Item No. 003. Authorization to proceed with both work order assignments was provided by MTO via signed Work Item Quote Forms for each assignment.

## 2.0 SITE DESCRIPTION

The site of the bridge structure to be replaced is located on Highway 28 in North Kawartha, Ontario. Highway 28 is currently an undivided two-lane highway with northbound and southbound traffic at this location. Eel's Creek flows in an east to west direction below the existing bridge. Bedrock outcrops were observed in the vicinity of the bridge during visits to the site in 2017 and 2018 (see Drawing 1).

Based on the General Plan drawing (Dwg. No. 320-2-1 prepared by DHO, dated December 3, 1952), the existing Eel's Creek Bridge is slightly skewed from the highway alignment and is a 17.8 m long single-span structure (as measured parallel to highway alignment). Based on the General Plan drawing, the north abutment is shown to be supported on a stepped shallow footing (about 13.5 m long by 1.8 m wide) founded below the existing stream bed on sand and gravel between about Elevation 254.6 m and 252.8 m. The south abutment footing (about 12.8 m long by 1.8 m wide) is shown to have been founded on bedrock, after removal / excavation of a minimum 0.3 m of the surficial rock within the foundation footprint. Within the south abutment foundation footprint, the bedrock surface is shown to be sloping from about Elevation 258.8 m on the east side to about Elevation 256.4 m on the west side, with a similar slope from the south side to the north side.

Concrete wingwalls are shown to be present each at each quadrant of the abutments, with wingwall foundations shown to match the founding level of the adjacent abutment wall at each location. The wing wall foundations are shown to extend about 5 m to 6 m back from the face of the abutment walls and range from about 2 m to 3 m wide. Cantilevered portions of the wingwalls extend back from the foundations and are shown to terminate about 7 m to 9 m back from the face of the existing abutment walls.

During the investigations performed in 2017 and 2018, the existing bridge abutments and approach embankments were observed and showed no visual signs of major distress or movement. The side slopes of the approach embankment were observed to be covered in rock fill in some areas, especially near the creek. Photographs taken of the bridge from Highway 28 and near the creek level are provided below.



**Photograph 1 – Looking at bridge skew from Highway 28 – looking south**



**Photograph 2 – Looking at face of south abutment from north side of creek (photo courtesy of MTO, 2016)**





**Photograph 3 – Looking at face of north abutment from south side of creek (photo courtesy of MTO, 2016)**



**Photograph 4 – South abutment and bedrock outcrop – looking southwest from north side of creek**





**Photograph 5 – West side of Highway 28 looking north. Rock fill observed on surface of northwest approach embankment side slope.**

The water level of Eel's Creek upstream and east of the existing bridge structure, as measured by Golder in June 2017, was at about Elevation 257.9 m. The water level shown on the General Plan (Dwg. No. 320-2-1) at the Eel's Creek structure was at Elevation 257.0 m and a high-water level at Elevation 258.5 m is also shown on the 1952 drawing.

### 3.0 FIELD INVESTIGATION PROCEDURES

The subsurface investigation for the proposed bridge replacement was carried out in three phases. The first phase was carried out between December 10 and 15, 2016, at which time three boreholes (designated as Boreholes 16-1, 16-2 and 16-6) were advanced at the locations shown on Drawing 1. A second phase of investigation was carried out between June 19 and 21, 2017 during which time a total of seven boreholes (designated as Boreholes 17-3, 17-4, 17-5, 17-7, 17-8, 17-9 and 17-10) were advanced at the locations shown on Drawing 1. A third investigation phase was carried out between November 12 and 15, 2018 to provide additional information to address alternate skews for the abutment foundations, during which time a total of five boreholes (designated as Boreholes 18-1, 18-2, 18-6, 18-8 and 18-10) and four probeholes (designated as Probeholes 18-3, 18-5, 18-9 and 18-11) were advanced at the locations shown on Drawing 1.

The boreholes and probeholes were advanced near the proposed replacement structure foundation elements as described below:

Proposed Foundation Element	Nearest Relevant Borehole / Probehole
South Approach Embankment	17-10, 18-1
South Abutment Foundations	16-6, 17-7, 17-8, 17-9, 18-2, 18-3
South Approach Temporary Protection System	16-6, 17-8, 17-10, 18-1
North Approach Embankment	17-5, 18-8
North Abutment Foundations	16-1, 16-2, 17-3, 17-4, 18-5, 18-6, 18-9, 18-10, 18-11
North Approach Temporary Protection System	16-2, 17-4, 17-5, 18-8

The location of the boreholes and probeholes are shown on Drawing 1 and the borehole, probehole and drillhole records are presented in Appendix A. Lists of abbreviations and symbols and rock descriptions are also provided in Appendix A to assist in the interpretation of the borehole, probehole and drillhole records.

The boreholes completed in 2016 and 2017 (Boreholes 16-1, 16-2, 16-6, 17-3, 17-4, 17-5, 17-7, 17-8, 17-9 and 17-10) were advanced using a CME-55 track-mounted drill rig supplied and operated by George Downing Estate Drilling Ltd. of Grenville-sur-la-Rouge, Quebec. The boreholes and probeholes completed in 2018 were advanced using an Acker Renegade track-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario (Boreholes 18-1, 18-2, and 18-10 and Probeholes 18-3, 18-9 and 18-11); and using a CME-55 truck mounted drill rig supplied and operated by Geo-Environmental Drilling Inc. of Acton, Ontario (Boreholes 18-6 and 18-8 and Probehole 18-5).

The boreholes from the 2016 investigation were generally advanced through the overburden using 114 mm outer diameter (O.D.) HW casing with wash boring techniques. The boreholes from the 2017 investigation, and Boreholes 18-1, 18-2, 18-6, 18-8 and 18-10 were advanced through the overburden using 203 mm O.D. and 108 mm inner diameter (I.D.) hollow-stem augers. Probeholes 18-3, 18-9 and 18-11 were advanced using 102 mm O.D. solid-stem augers and Probehole 18-5 was advanced using 152 mm O.D. and 57 mm I.D. hollow-stem augers. Core samples of the bedrock in Boreholes 16-1, 16-2, 17-7, 18-1, 18-2, 18-6, 18-8 and 18-10 were obtained using either an 'HQ' or 'NQ' size rock core barrel, as noted on the drillhole records.

The boreholes and probeholes were advanced to depths between approximately 2.0 m and 13.3 m below existing ground surface, including coring of bedrock for core lengths of between 0.9 m and 5.2 m in select boreholes. Photographs of the recovered bedrock core samples are provided in Appendix B.

At borehole locations, soil samples were generally obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm outer diameter split-spoon sampler advanced by an automatic hammer mounted on the drill rig, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586<sup>1</sup>). Bedrock quality and discontinuity

<sup>1</sup> ASTM D1586-11 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils, ASTM International, West Conshohocken, PA, 2011

data were recorded in the field based on visual inspection of the recovered bedrock core extracted from the core barrel. Probeholes were advanced by continuously drilling from ground surface, without sampling, until auger refusal was reached. At each probehole location, a split-spoon sample was attempted at the auger refusal depth with limited to no penetration observed / measured.

The groundwater conditions in the open boreholes / probeholes were observed during drilling operations and prior to wash boring / rock coring and are noted on the borehole and probehole records in Appendix A. All boreholes were backfilled upon completion of drilling / coring in accordance with Ontario Regulation 903 (Wells) (as amended).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination. Geotechnical classification testing (i.e. water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. Uniaxial Compressive Strength (UCS) tests were carried out on selected rock core samples. All the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Higher complexity Unconfined Compressive Strength (UCS) tests including measurement of Young's modulus were carried out on two selected specimens of the bedrock core samples by Geomechanica Inc., a specialist rock testing company, on behalf of Golder. The results of the geotechnical laboratory testing on the soil and rock core samples are included in Appendices B and D.

Borehole 16-2 was grouted and cased with a PVC pipe after completion of geotechnical drilling to allow for Vertical Seismic Profile (VSP) testing, which was carried out on June 20, 2017. Details of the VSP investigation methodology and results are outlined in the Technical Memorandum titled "Vertical Seismic Profiling Test Results – Eels Creek Bridge and Highway 28, North Kawartha, Ontario", dated July 4, 2017 and attached in Appendix C, following the text of this report.

Two selected soil samples were submitted, under chain-of-custody procedures, to Maxxam Analytics of Mississauga, Ontario (a Standards Council of Canada accredited laboratory) for corrosivity testing. The soil samples were analyzed for a suite of parameters, including conductivity, resistivity, soluble chloride, soluble sulphate and pH. The testing procedures and results of the analytical tests are presented in Appendix E.

The as-drilled locations and ground surface elevations at Boreholes 16-2, 17-3, 17-4, 17-5, 17-7, 17-8, 17-9 and 17-10 were established on-site by Golder personnel using a Global Positioning System unit (i.e. Trimble Geo7 GPS unit) with a horizontal accuracy of 2 cm or less, and a vertical accuracy of 4 cm or less. The remainder of the borehole and probehole locations were measured in the field relative to existing site features, and the ground surface elevations and coordinates were generated from the digital terrain model / CAD files provided by MTO. The locations provided on the borehole and probehole records and shown on Drawings 1 and 2 are positioned relative to the MTM NAD 83 (Zone 10) coordinate system, and the ground surface elevations are referenced to Geodetic datum. The as-drilled borehole/probehole locations, coordinates, ground surface elevations and drilled depths/elevations are summarized below.

Borehole / Probehole Designation	Coordinates, MTM NAD83 Zone 10 (Geographic)		Ground Surface Elevation (m)	Borehole / Probehole Termination Depth (m)	Borehole / Probehole Termination Elevation (m)
	Northing (Latitude, °)	Easting (Longitude, °)			
16-1	4,945,466.9 (44.641072)	413,005.0 (-78.136013)	262.0	13.3*	248.7
16-2	4,945,465.5 (44.641058)	413,010.4 (-78.135945)	262.0	11.8*	250.2
16-6	4,945,440.7 (44.604834)	413,014.0 (-78.135905)	262.4	4.3	258.1
17-3	4,945,468.7 (44.641087)	413,011.5 (-78.135930)	262.0	5.6	256.4
17-4	4,945,470.1 (44.641100)	413,006.4 (-78.135995)	262.0	9.3	252.7
17-5	4,945,475.9 (44.640847)	413,008.9 (-78.135962)	261.9	5.2	256.7
17-7	4,945,442.0 (44.640847)	413,008.9 (-78.135969)	262.4	8.8*	253.6
17-8	4,945,438.9 (44.640890)	413,009.7 (-78.135960)	262.4	3.9	258.5
17-9	4,945,435.3 (44.640786)	413,015.5 (-78.135884)	262.5	3.1	259.4
17-10	4,945,431.5 (44.640752)	413,012.8 (-78.135921)	262.5	2.0	260.5
18-1	4,945,424.4 (44.640688)	413,010.4 (-78.135953)	262.6	5.7*	256.9
18-2	4,945,432.7 (44.640763)	413,008.9 (-78.135971)	262.2	6.6*	255.6
18-3	4,945,437.1 (44.640803)	413,009.6 (-78.135961)	262.3	3.6	258.7
18-5	4,945,470.5 (44.641104)	413,003.4 (-78.136032)	261.9	9.8	252.2
18-6	4,945,475.5 (44.641149)	413,002.8 (-78.136038)	261.8	8.6*	253.3
18-8	4,945,479.4 (44.641148)	413,006.6 (-78.133599)	261.8	8.8*	253.0

Borehole / Probehole Designation	Coordinates, MTM NAD83 Zone 10 (Geographic)		Ground Surface Elevation (m)	Borehole / Probehole Termination Depth (m)	Borehole / Probehole Termination Elevation (m)
	Northing (Latitude, °)	Easting (Longitude, °)			
18-9	4,945,470.6 (44.641103)	413,016.9 (-78.135862)	260.7	2.9	257.8
18-10	4,945,473.4 (44.641128)	413,017.6 (-78.135852)	260.7	6.2*	254.5
18-11	4,945,478.3 (44.641173)	413,015.9 (-78.135873)	261.0	3.2	257.8

\*Includes coring of bedrock for core lengths of between 0.9 m and 5.2 m

## 4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*<sup>2</sup> (Chapman and Putnam, 1984), this site lies within the physiographic region known as Georgian Bay Fringe, located north of the Drummer moraines.

The Georgian Bay Fringe is characterized by shallow deposits of glacial till and bare rock knobs and ridges. The underlying bedrock in this physiographic region is typically Precambrian igneous and metamorphic rock.

### 4.2 Subsurface Conditions

The subsurface and groundwater conditions encountered in the boreholes advanced at this site as part of the foundation investigation together with the results of in-situ and geotechnical and analytical laboratory testing, are presented on the borehole records (provided in Appendix A) and laboratory test figures / tables (provided in Appendices B and D). The results of the in-situ field tests (i.e., SPT 'N'-values) as presented on the borehole records are uncorrected, and are based on sampling procedures carried out with an automatic hammer.

The stratigraphic boundaries shown on the borehole/probehole records and on the stratigraphic profiles on Drawings 1 and 2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations; however, the factual data presented in the borehole records governs any interpretation of the site conditions. It should be noted that the interpreted stratigraphic profiles shown on Drawings 1 and 2 represent a simplification of the subsurface conditions.

The subsurface conditions encountered at the Site generally consist of the highway pavement structure underlain by non-cohesive embankment fill underlain by bedrock comprised of a combination of granite and migmatite gneiss. Interlayers of silty sand to silty sand and gravel were encountered between the fill and bedrock interface at some

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



locations. A more detailed description of the subsurface conditions encountered in the boreholes from the current investigation is provided in the following sections.

#### **4.2.1 Topsoil**

A 100 mm thick layer of topsoil was encountered at the ground surface in Borehole 18-10. Topsoil was observed at the ground surface in Probeholes 18-9 and 18-11.

#### **4.2.2 Asphalt**

All boreholes, with the exception of Borehole 18-10 and Probeholes 18-9 and 18-11, were advanced from within the travelled lanes of Highway 28 and through the pavement structure. Asphalt ranging in thickness between approximately 100 mm and 200 mm was encountered at ground (roadway) surface.

#### **4.2.3 Non-Cohesive Embankment Fill**

The asphalt and topsoil are underlain by deposits of non-cohesive fill extending to depths ranging from 1.5 m to 9.3 m below existing ground surface (Elevation 261.1 m to 252.7 m) in all boreholes. The upper 0.3 m to 0.7 m of fill is generally comprised of sand and gravel to gravelly sand (i.e. pavement structure) at most borehole locations. The lower portions of the fill range in composition from silt and sand to sand to gravelly sand to sand and gravel containing trace to some clay and trace to some organics. The presence of cobbles and potentially boulders within the fill at some borehole locations was inferred based on auger grinding during borehole advancement at varying depths/elevations as noted on the borehole records. The cobble to boulder sized fragments could potentially be rock fill that was previously used for embankment construction. Boreholes 16-6, 17-3, 17-4, 17-5, 17-8 and 17-9 were terminated after encountering auger refusal and split-spoon refusal on the inferred fill/bedrock interface.

Standard Penetration Test (SPT) 'N' values measured in the fill range from 2 blows per 0.3 m of penetration to 50 blows for 0.05 m of penetration, but generally less than 20 blows per 0.3 m of penetration, indicating a very loose to very dense, but generally loose to compact state of compactness.

The results of grain size distribution testing carried out on 25 samples of the fill are provided on Figures B1A-B, B2A-C and B3 in Appendix B. It should be noted that the grain size results do not reflect the cobble or boulder content of the fill material due to the 50 mm outer diameter sampler used to retrieve the samples. An Atterberg Limits test was carried out on one sample of the fill deposit from Borehole 18-10 and returned a result of non-plastic. The natural water content measured on selected samples of the fill ranges between 2 per cent and 26 per cent.

#### **4.2.4 Silty Sand and Gravel to Sand to Silty Sand (Containing Organics)**

The fill soils were underlain by a deposit of silty sand and gravel to sand to silty sand in Boreholes 16-1, 16-2, 17-10 and 18-6 at depths between 1.5 m and 7.5 m below ground surface (Elevation 261.1 m and 254.5 m). Variable amounts of organics were encountered within the deposit in Borehole 16-1, 16-2 and 17-10. In Borehole 16-1, wood fragments were observed in the upper portion of the deposit and the laboratory organic content measured on one sample was approximately 10 per cent.

Standard Penetration Test (SPT) 'N' values measured within the deposit range from 1 blow to 5 blows per 0.3 m of penetration, indicating very loose to loose compactness condition. Higher SPT 'N' values ranging from 50 blows for no penetration to 50 blows for 0.13 m of penetration were recorded at the bottom of the deposit in Borehole 16-1, 17-10 and 18-6, but are indicative of sampler refusal on the underlying bedrock interface.

The results of grain size distribution testing carried out on three samples of this deposit are provided on Figure B4 in Appendix B. The natural water content typically measured on selected samples of the deposit range between 12



per cent and 34 per cent, with one higher value of 58 per cent measured in the upper portion of the deposit in Borehole 16-1 where significant organics were present.

#### 4.2.5 Bedrock

Bedrock was inferred and/or confirmed to be encountered underlying the fill and sandy deposits in all boreholes / probeholes advanced during the current investigation. Bedrock was confirmed by core samples recovered from Boreholes 16-1, 16-2, 17-7, 18-1, 18-2, 18-6, 18-8 and 18-10 where bedrock was encountered at depths ranging between 1.8 m and 10.3 m below ground surface. Auger and split-spoon refusal on inferred bedrock was encountered in all of the remaining boreholes and probeholes between a depth of 2.0 m and 9.8 m below ground surface. The approximate depths to the top of bedrock below ground surface and corresponding top of bedrock surface elevation are summarized below and shown on the borehole / probehole / drillhole records in Appendix A.

Borehole / Probehole Designation	Existing Ground Surface Elevation (m)	Approximate Depth to Bedrock Surface (m)	Approximate Bedrock Surface Elevation (m)	Notes
16-1	262.0	10.3	251.7	Bedrock cored
16-2	262.0	6.5	255.5	Bedrock cored
16-6	262.4	4.3	258.1	Auger / split-spoon refusal on inferred bedrock
17-3	262.0	5.6	256.4	Auger / split-spoon refusal on inferred bedrock
17-4	262.0	9.3	252.7	Auger / split-spoon refusal on inferred bedrock
17-5	261.9	5.2	256.7	Auger / split-spoon refusal on inferred bedrock
17-7	262.4	5.1	257.3	Bedrock cored
17-8	262.4	3.9	258.5	Auger / split-spoon refusal on inferred bedrock
17-9	262.5	3.1	259.4	Auger / split-spoon refusal on inferred bedrock
17-10	262.5	2.0	260.5	Auger / split-spoon refusal on inferred bedrock
18-1	262.6	1.8	260.9	Bedrock cored
18-2	262.2	2.6	259.6	Bedrock cored
18-3	262.3	3.6	258.7	Auger / split-spoon refusal on inferred bedrock
18-5	261.9	9.8	252.2	Auger / split-spoon refusal on inferred bedrock

Borehole / Probehole Designation	Existing Ground Surface Elevation (m)	Approximate Depth to Bedrock Surface (m)	Approximate Bedrock Surface Elevation (m)	Notes
18-6	261.8	7.7	254.2	Bedrock cored
18-8	261.8	5.3	256.5	Bedrock cored
18-9	260.7	2.9	257.8	Auger / split-spoon refusal on inferred bedrock
18-10	260.7	2.6	258.1	Bedrock cored
18-11	261.0	3.2	257.8	Auger / split-spoon refusal on inferred bedrock

Based on the review of the recovered bedrock core samples, the bedrock consists predominantly of black and grey gneiss, pink granite and black and red migmatite bedrock. The bedrock is generally fresh, fine to medium grained and non-porous. Photographs of the bedrock cores are shown on Figures B9 to B14 in Appendix B.

The Rock Quality Designation (RQD) measured on the core samples generally ranges from about 58 per cent to 100 per cent, indicating a rock mass of fair to excellent quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM), 2006<sup>3</sup>. At Boreholes 18-1 and 18-8, RQD values of 30 per cent to 40 per cent were measured near the bedrock surface at depths between 1.8 m to 4.4 m (Elevation 260.8 m to 258.2 m) and 5.3 m to 5.7 m (Elevation 256.5 m to 256.1 m) below ground surface respectively, indicating that this portion is a rock mass of poor quality (CFEM), 2006<sup>3</sup>.

The Total Core Recovery (TCR) ranges from about 48 per cent to 100 per cent and the Solid Core Recovery (SCR) ranges from about 32 per cent to 100 per cent.

Uniaxial Compressive Strength (UCS) tests (including assessment of Young's modulus) were carried out on two selected specimens of the bedrock core samples by Geomechanica Inc. on behalf of Golder and the detailed test report is included in Appendix D. UCS tests were also carried out on four samples of the bedrock core by Golder and the laboratory test results and photographs of the condition of the tested specimens are summarized in Figures B5, B6, B7 and B8 in Appendix B. The results of the rock core strength testing are summarised below:

Borehole No.	Sample Depth (m)	Sample Elevation (m)	Bedrock Type	Bulk Unit Weight (kN/m <sup>3</sup> )	UCS (MPa)	Young's Modulus (GPa)
16-1	10.47 – 10.75	251.5 – 251.2	Migmatite	27.0	47.0	not measured
16-2	6.48 – 6.75	255.5 – 255.2	Migmatite	25.5	99.4	not measured
17-7	5.87 – 6.00	256.5 – 256.4	Migmatite	25.8	45.5	not measured

<sup>3</sup> Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual (CFEM), 4<sup>th</sup> Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.

Borehole No.	Sample Depth (m)	Sample Elevation (m)	Bedrock Type	Bulk Unit Weight (kN/m <sup>3</sup> )	UCS (MPa)	Young's Modulus (GPa)
17-7	8.62 – 8.80	253.8 – 253.6	Migmatite	27.3	55.1	not measured
18-1	3.79 – 3.93	258.8 – 258.7	Migmatite	28.2	149.1	78.3
18-8	5.76 – 5.99	256.0 – 255.8	Gneiss	27.4	93.0	42.8

Based on the laboratory testing results, the intact strength of the migmatite (granite and gneiss) bedrock varies from medium strong (25 MPa < UCS < 50 MPa) to strong (50 MPa < UCS < 100 MPa) to very strong (100 MPa < UCS < 250 MPa) in accordance with Table 3.5 in CFEM (2006).

#### 4.2.6 Groundwater Conditions

The groundwater level, if present, in the open boreholes or within the drilling casing was typically measured upon completion of drilling operations and / or prior to rock coring. In some boreholes, drilling mud / water was added during drilling operations and groundwater levels were not recorded in these boreholes. The details of the groundwater level measurements are shown on the borehole records contained in Appendix A; however, it is noted that these measurements may not represent the long-term, stabilized groundwater levels at the site.

The measured groundwater levels in the open boreholes upon completion of drilling operations are summarized below:

Borehole / Probehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date (dd/mm/yyyy)
16-1	262.0	3.3	258.7	10/12/2016
17-3	262.0	Dry	N/A	20/06/2017
17-4	262.0	6.1	255.9	20/06/2017
17-5	261.9	Dry	N/A	20/06/2017
17-7	262.4	Dry	N/A	21/06/2017
17-8	262.4	Dry	N/A	21/06/2017
17-9	262.5	Dry	N/A	20/06/2017
17-10	262.5	Dry	N/A	20/06/2017
18-1	262.6	Dry	N/A	14/11/2018
18-2	262.2	1.6	260.6	15/11/2018
18-3	262.3	Dry	N/A	14/11/2018
18-5	261.9	7.6	254.3	14/11/2018

Borehole / Probehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date (dd/mm/yyyy)
18-6	261.8	5.2	256.6	15/11/2018
18-8	261.8	5.1	256.7	14/11/2018
18-9	260.7	Dry	N/A	15/11/2018
18-11	261.0	2.7	258.3	15/11/2018

Note: Water levels were not recorded in boreholes 16-2, 16-6 and 18-10

Based on the General Arrangement (GA) drawing provided by MTO and dated April 2016, the Eel's Creek water level is at about Elevation 257.0 m and the high-water level (HWL) is shown to be at Elevation 258.5 m. It is noted that the same creek water levels are shown on the original 1952 design drawing. In June 2017, the Eel's Creek water level east of the bridge site was measured to be at Elevation 257.9 m during the foundation investigation. It is noted that the Eel's Creek streambed gradient within this reach is significant based on the observed fast flowing water and rapids observed during the investigation in June 2017.

It should be noted that groundwater levels and the Eel's Creek water level are subject to seasonal fluctuations and precipitation events and are expected to be higher during wet seasons and sustained periods of precipitation. Perched groundwater conditions are anticipated to be present at / near the overburden / bedrock interface when this interface is located above the creek water level.

### 4.3 Analytical Testing of Soil

A soil sample from each of Boreholes 16-1 and 17-7 was selected during the field investigation programs and submitted to Maxxam Analytics of Mississauga, Ontario for analysis of parameters used to assess the potential corrosivity of the site soils to steel and concrete. The analytical laboratory test results are provided on the Certificate of Analysis presented in Appendix E and summarized below.

Borehole No.	Sample ID.	Depth (m)	Elevation	Material Type	Parameters				
					Resistivity (ohm-cm)	Electrical Conductivity (µmho/cm)	pH	Chloride (Cl) Content (µg/g)	Soluble Sulphate (SO <sub>4</sub> ) Content (µg/g)
16-1	SS 12	8.4 – 9.0	253.6 – 253.0	Sand	3,200	312	5.85	45	200
17-7	SS 5 & SS 6	3.0 – 4.4	259.4 – 258.0	Sand and Gravel (Fill)	1,600	621	7.72	250	140

## 5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Katelyn Nero, and was reviewed by Mr. Matthew Kelly, P.Eng. Kevin Bentley, M.E.Sc., P.Eng., an Associate and MTO Designated Foundations Contact of Golder, and Lisa Coyne, P.Eng., a Principal and MTO Designated Foundations Contact of Golder, conducted a technical and quality control review of the report.

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# PART B

FOUNDATION DESIGN REPORT  
HIGHWAY 28, EEL'S CREEK BRIDGE REPLACEMENT (SITE NO. 26-117)  
NORTH KAWARTHA, ONTARIO  
MINISTRY OF TRANSPORTATION, ONTARIO



## 6.0 DISCUSSION AND RECOMMENDATIONS

This section of the report provides geotechnical recommendations for the design of the Eel's Creek bridge replacement (Site No. 26-117) on Highway 28 in North Kawartha, Ontario. These recommendations are based on interpretation of the factual data obtained from the boreholes and probeholes advanced during the subsurface investigation at this site and from site observations.

This Foundation Investigation and Design Report and its interpretations 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 construction contractor or the design-build contractor. The contractor must make their own interpretation of the subsurface conditions based on the factual data in Part A (Foundation Investigation) of this report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

### 6.1 General

Based on the General Plan drawing (Dwg. No. 320-2-1 prepared by DHO, dated December 3, 1952), the existing Eel's Creek Bridge is slightly skewed from the highway alignment and is a 17.8 m long single span structure (parallel to highway alignment). Based on the General Plan drawing, the north abutment is shown to be supported on a stepped shallow footing (about 13.5 m long by 1.8 m wide) founded below the existing stream bed on sand and gravel between about Elevation 254.6 m and 252.8 m. The south abutment footing (about 12.8 m long by 1.8 m wide) is shown to have been founded on bedrock after removal / excavation of a minimum 0.3 m of the surficial rock. Concrete wingwalls are shown to be present each at each quadrant of the bridge, with wingwall foundations shown to match the founding level of the adjacent abutment wall at each location. The wing wall foundations are shown to extend about 5 m to 6 m back from the face of the abutment walls and range from about 2 m to 3 m wide. Cantilevered portions of the wingwalls extend back from the foundations and are shown to terminate about 7 m to 9 m back from the face of the existing abutment walls.

Based on the preliminary General Arrangement (GA) drawing provided by MTO, dated April 2016 (revised September 28, 2018), the proposed Eel's Creek bridge replacement will consist of a single-span structure with a total span length of about 31.6 m and a width of about 13.5 m. The new bridge is to be approximately 1.7 m wider on the east side (with associated embankment widening) and will accommodate new widened shoulders in both northbound and southbound directions. Two new abutment configuration options are shown on the GA drawing, with new north and south abutments either perpendicular to the highway alignment or skewed at about 20 degrees to the highway alignment (approximately parallel to Eel's Creek and similar to the existing bridge structure configuration).

Based on the preliminary Staging drawing provided by MTO, dated April 2016, the new structure is proposed to be replaced in two stages. In this regard, the Highway 28 traffic will travel on one-half of the existing bridge while removal and replacement/widening of the other half is carried out using temporary protection systems, following which the traffic will be switched over to the new half of the bridge and removal and replacement of the second half will be carried out.

## 6.2 Bridge Foundation Options

Based on the subsurface investigation results, there is generally cohesionless fill overlying bedrock at the north and south abutment locations. Given the variability of the fill type and range in compactness with a significant portion of the fill being loose, the existing fill soils are considered not suitable for support the new bridge foundations. Foundations will need to be supported on the competent bedrock present at the site. There is significant variation in the elevation of the bedrock surface across the site and specifically across the footprint of each foundation element.

The following summarizes the range of bedrock elevations encountered in the nearest boreholes at each abutment location.

Foundation Element	Relevant Boreholes / Probeholes	Bedrock Elevation / Depth below ground surface (m)	
		Minimum (Lowest) Elevation	Maximum (Highest) Elevation
North Abutment	16-1, 16-2, 17-3, 17-4, 17-5, 18-5, 18-6, 18-9, 18-10, 18-11	251.7 / 10.3	258.1 / 2.6
South Abutment	16-6, 17-7, 17-8, 17-9, 18-2, 18-3	257.3 / 5.1	259.6 / 2.6

At the north abutment, the bedrock generally slopes down from north to south and from east to west at angles up to about 40 degrees. At the south abutment, the bedrock generally slopes down from south to north and from east to west at angles up to about 25 degrees. Several foundation options have been considered and are discussed in the sections below. A comparison of the advantages, disadvantages, risks/consequences and relative costs for each option is provided in Tables 1 and 2 (for the south and north abutment locations respectively) following the text of this report.

The preferred foundation option from a geotechnical / foundation perspective is as follows:

- South Abutment – spread footings founded on bedrock and/or on mass concrete placed on bedrock.
- North Abutment – drilled steel casings (filled with concrete) socketed into bedrock.

### 6.2.1 Option No. 1 – Spread Footings Founded on Bedrock and/or on Mass Concrete Placed on Bedrock

Consideration could be given to subexcavating the existing fill and any native overburden soils within each foundation footprint to expose the bedrock. In low areas, replacement with mass concrete (or controlled low strength material depending on required load capacity) can be considered, to raise the surface up to the design founding elevation and ultimately minimize the foundation depth/height of abutment walls.

This option is not considered to be practical at most of the north abutment location where bedrock was typically encountered at depths between 5 m and over 10 m below ground surface, and below the groundwater / creek water level. This option is considered suitable for the east limit (including east wingwall) of the north abutment and at the entire south abutment and corresponding wingwall locations. In order to minimize bedrock subexcavation and create a level design founding surface, consideration can be given to placing mass concrete where the bedrock surface is lower, as described above.

Excavation of the bedrock may be required at shallow depths to remove any weathered or fractured rock (possibly from construction of the existing foundations) and/or to create a level founding surface where spread footings or mass concrete can be placed. Due to the risk of disturbing the existing or new foundations through vibrations or rock over-break, any blasting techniques must be carefully designed and controlled by a blasting engineer to limit over-excavation and minimize impacts to the new or existing structure foundations as the new abutment foundations will be in close proximity to the existing abutments during staged construction. Bedrock excavation could also be carried out using line drilling and hydraulic splitting or using non-explosive chemical rock-breaking compounds. Where bedrock excavation is not considered to be practical, spread footings / mass concrete could be anchored using rock dowels to resist sliding on the sloping bedrock surface.

At the north abutment, groundwater was measured between about Elevation 258.7 m and 254.3 m, with an average of about Elevation 257 m which is the Eel's Creek water level shown on the GA drawing. At the south abutment, the majority of the boreholes were dry upon completion of drilling with the exception of one borehole that measured groundwater at Elevation 260.6 m. Although these water levels were recorded during drilling and may not represent the stabilized groundwater level at the site, it is anticipated that perched groundwater will be present near the overburden / bedrock interface with a stabilized groundwater level at the creek water level (i.e. Elevation 257 m).

Subexcavation to the bedrock surface will require dewatering of the perched groundwater within the non-cohesive fill and native deposits above the creek water level (i.e. at the south abutment and east limit of the north abutment). For excavations below the creek water level, a combined dewatering system using sheetpile cut-off walls and well points or the use of sheetpile cut-off walls and tremie concrete plug may be required (as discussed in Section 6.10.3). Difficulties achieving a watertight seal between the sheetpiles and the sloping bedrock should be anticipated. As a result, deep subexcavation at the north abutment is not recommended and not considered practical.

## 6.2.2 Option No. 2 – Drilled Steel Casings Socketed into Bedrock

Consideration could be given to the use of drilled steel casings socketed into the bedrock. The procedure typically uses rotary duplex drilling techniques with a sacrificial ring bit on the bottom of a permanent steel casing. A down-the-hole hammer and flush system will likely be necessary to advance the casing through the fill soils containing cobbles and rock fragments (possible boulders) and seat the casing within the bedrock and also to create an uncased rock socket within the bedrock below the bottom of the casing.

Information from various product suppliers and experience on previous MTO contracts indicate that this type of drilling system allows relatively accurate and straight penetration in steeply sloping bedrock surfaces (typically up to 60 degrees) and can also readily penetrate cobbles, boulders, and rock fragments. This technique can also be used to drill rock socket in very strong granitic bedrock.

In order to develop sufficient capacity in compression and tension, an uncased rock socket with a Length / Diameter (L/D) ratio of at least 3 is recommended. The permanent steel casing must be embedded at least 1 m below the lowest point of contact with the bedrock surface and a minimum of 1 m into fair quality bedrock, however, additional casing embedment length may be required to satisfy the lateral loads on the abutments, and also achieve a proper seal in the bedrock prior to socket construction, if the upper bedrock at the pile location is of poor quality. A seal will be required to prevent inflow of the surrounding sands and silts during cleaning, rock drilling and placement of concrete.

The pile is designed to develop the majority of its axial capacity based on the shear resistance along the rock socket wall (i.e. between the concrete and bedrock interface) rather than on end-bearing at the base of the socket. As

such, the requirement to thoroughly clean and inspect the base of the socket will be lessened, however a thorough and proper flushing of the side walls of the rock socket is required. A reinforcing steel cage would likely have to be lowered through the casing and into the rock socket prior to placement of concrete by tremie methods.

As described in detail in Section 6.5, it is assumed that 300 mm to 600 mm diameter casings installed using the specialized down-the-hole hammer drilling technique will be sufficient for the anticipated loadings, although this needs to be confirmed when structural loads are finalized. This option is preferred in lieu of larger diameter caissons which are less likely to achieve the required socket and/or watertight seal for rock anchors within the strong to very strong sloping bedrock without more specialized equipment.

As discussed in the previous section, groundwater is anticipated to be encountered during the pile installation within the cohesionless soils and thus the requirement for an adequate casing seal. It is anticipated that subexcavation for pile caps would be above the creek water level; however, some perched water may be present and may require dewatering of the non-cohesive fill deposit. The dewatering effort will depend on the pile cap level and groundwater conditions at the time of construction.

### **6.2.3 Option No. 3 - Combined Spread Footings Founded on Bedrock and Drilled Steel Casings Socketed into Bedrock**

Alternately, consideration could be given to combining spread footings founded on bedrock / mass concrete with drilled steel casings socketed into bedrock. This option allows for optimizing open-cut excavation and deep foundation options in order to allow for conventional shallow foundation construction in some areas but minimize extensive subexcavation and placement of mass concrete below the groundwater table / creek water level.

This option may be considered as an option at the North Abutment location but is not practical at the South Abutment location as the bedrock surface is more consistent. For the North Abutment, subexcavation depths are anticipated to be limited to less than about 3 m to 5 m below ground surface to avoid excavating below the creek / groundwater level and allow for conventional equipment to be used. Based on this assumption, spread footings could only be considered along the wingwall and the eastern limit (about 10% or less than 2 m of abutment length) of the North Abutment location.

The design and construction considerations given above for the shallow footing and steel casing options are also applicable for this alternative.

### **6.2.4 Option No. 4 – Steel H-piles Driven to Found on Bedrock**

Consideration could also be given to supporting the abutments on steel H-piles driven to found on bedrock. As per the table in [Section 6.6](#), estimated pile lengths from the assumed bottom of pile cap to the bedrock surface range from 2 m to 9 m at the North Abutment and 1 m to 3 m at the South Abutment.

The minimum required design pile embedment length is 3 m and 5 m for conventional and integral abutments, respectively, as per MTO guidelines. As a result, steel H-piles are not considered practical at the South Abutment but may be considered at the North Abutment. In order to achieve the minimum pile embedment lengths, pre-drilling or advanced excavation / trenching into the very strong bedrock (and subsequent backfill with granular fill) would be required at the east side of the North Abutment in order to achieve the minimum design pile embedment length.

The obstructions (i.e. cobbles, rock fragments and/or boulders) encountered in the fill during the drilling investigation may pose challenges during driving operations, and the sloping bedrock surface may present difficulties in seating of the driven piles. In this regard, piles should be fitted with appropriate rock points (i.e. Titus "Rock Injector Design",

Oslo Points as per OPSD 3000.201 or equivalent); however, even with the rock points, seating of the piles may be difficult due to the combination of sloping bedrock and relatively thin overburden soils (i.e. short pile embankment lengths), and as such this option is considered to have higher risks and lower practicability for this site. Assuming bedrock excavation and granular backfill to provide a minimum 3 m pile embedment, the proposed pile embedment lengths would range from about 3 m to 9 m at the North Abutment.

It is anticipated that subexcavation for pile caps would be above the creek water level; however, some perched water may be present and may require dewatering of the non-cohesive fill deposit. The dewatering effort will depend on the groundwater conditions at the time of construction.

### **6.2.5 Option No. 5 – Combination of Spread Footings and Steel H-piles Driven to Found on Bedrock**

Consideration could also be given to supporting the North Abutment on a combination of spread footings and steel H-piles founded on bedrock. This option is not practical for the South Abutment. Shallow spread footings may be constructed on bedrock where driven pile embedment lengths are less than 3 m, typically the east wingwall and east limit (about 2 m length) of the North Abutment.

The design and construction considerations for the shallow foundation founded on bedrock option and steel H-pile driven to bedrock option are also applicable for this alternative.

### **6.2.6 Option No. 6 – Micropiles**

Consideration was given to using micropiles for support of the bridge abutments. For the purpose of this report, a micropile is defined as a small diameter (typically less than 300 mm), drilled and grouted replacement pile that is typically reinforced with a central reinforcing bar. Contrary to conventional pipe piles (concrete filled) or caissons where most of the applied load is resisted by the reinforced concrete, micropile structural capacities rely on high-capacity steel elements (typically threaded bars or reinforcing steel) to resist most or all of the applied load. The special drilling (down-the-hole hammer) and grouting methods used in micropile installation allow for high grout/ground bond values along the grout/ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter, any end-bearing contribution in micropiles is generally neglected. Vertical micropiles may be limited in lateral capacity and cost effectiveness, especially in the gravelly fill soils where grout viscosity will be critical in achieving an effective micropile design.

For this site, considering the fill soils and the fact that bedrock is generally encountered at shallow depth, there is not enough overburden thickness to develop sufficient axial capacity and the micropiles would require socketing into the bedrock. As noted above, a threaded central bar is typically placed into the cased borehole for both structural and lateral support and the borehole casing fully or partially removed. The lateral capacity is based entirely on the bending stiffness of the steel bars with limited resistance offered from the grouted zone. As a result, this option is not preferred from a foundations perspective and will not be discussed further.

### **6.2.7 Option No. 7 – Caissons**

Consideration was given to using caissons for support of the abutments; however, due to the sloping strong to very strong bedrock and the potential difficulties associated with achieving an adequate seal and seating into the bedrock at the base of the relatively large diameter caisson (i.e. to prevent groundwater and cohesionless soil inflow into the casing at the base) and the specialized large equipment that would be required, caissons are not considered to be a practical option and are not discussed further.

## 6.3 General Foundation Design Context

### 6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code and its Commentary* (CHBDC, 2014), the proposed bridge and its foundation system are classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the typical project specific foundation investigation carried out at this site (as presented in Part A of the report), in comparison to the degree of site understanding in Section 6.5 of the CHBDC (2014), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding”. Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor,  $\Psi$ , from Table 6.1 and geotechnical resistance factors,  $\Phi_{gu}$  and  $\Phi_{gs}$ , from Table 6.2 of the CHBDC (2014) have been used for design.

### 6.3.2 Seismic Site Classification

It is understood that the proposed replacement structure has an Importance Category of “Major Route” bridge in accordance with Section 4.4.2 of the CHBDC (2014).

Based on the results of the VSP testing (refer to Appendix C), the average shear wave velocity from ground surface to a depth of 30 m below ground surface was measured / estimated to be 729 m/s. Referring to Table 4.1 (CFEM, 2014) and the shear wave velocity calculated from the VSP testing, the site classification is near the limit between Site Class B and Site Class C. However, given the fact that there is greater than 3 m of overburden between the assumed underside of the foundation / pile cap and bedrock surface in some areas of the North Abutment, the site is classified as “Site Class C”.

## 6.4 Spread Footings

The bridge abutments may be supported on shallow spread footings founded on mass concrete or directly on the properly prepared migmatite/gneiss bedrock at the founding elevation range provided in the table presented in Section 6.3.

For design of spread footings on bedrock, based on the borehole results and exposed bedrock outcrops, there is a high variability in the bedrock surface elevation within the limits of each foundation element. Prior to construction of the footings, or placement of mass concrete, it will be necessary to clean, scale and remove any loose or fractured rock or surficial debris to ensure a proper bond to bedrock, which may result in lower founding elevations than those presented above; bedrock “knobs” or ridges may also be present, such that portions of the bedrock surface are higher than those presented above. Spread footings should be placed either directly on the properly prepared bedrock surface or placed on mass concrete placed on the properly prepared bedrock surface which will minimize the bedrock excavation required. An NSSP should be included in the Contract Documents to allow for placement of additional mass concrete to accommodate variations in the bedrock surface; an example is provided in Appendix F.

All bedrock excavation within and near the foundation footprints should be carried out using controlled blasting (under the direction of an experienced blasting engineer), line drilling and hydraulic splitting techniques or chemical rock breaking compounds to minimize shattering/over-break and reduce vibration levels and potential impacts to the existing or new foundations. Additional recommendations on bedrock excavation are provided in Sections 6.10.6 and 6.10.7.



### 6.4.1 Geotechnical Axial Resistance

For the abutments, spread footings constructed on the surface of the properly prepared migmatite/gneiss bedrock, or on mass concrete placed on the properly prepared bedrock surface, should be designed based on the factored ultimate and serviceability (for 25 mm of settlement) geotechnical resistances given below.

Spread Footing Location	Geotechnical Axial Resistance	
	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa)
North Abutment	10,000	n/a
South Abutment	10,000	n/a

The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate axial geotechnical resistance, because the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

All loose, broken and/or fractured rock within the foundation footprint and at the footing level should be cleaned and scaled prior to replacement with concrete and in accordance with OPSS 902 (Excavating and Backfilling – Structures) and SP109S12 (Amendment to OPSS 902).

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.10.2 and 6.10.4 of the *Canadian Highway Bridge Design Code* (CHBDC) and its *Commentary*.

The UCS test results of the bedrock core samples and field description of the recovered bedrock core indicate that the bedrock is generally strong to very strong, and where excavations for the abutment foundations extend into this formation (for levelling purposes), appropriate construction equipment and procedures (such as controlled blasting, line drilling and hydraulic splitting or using a non-explosive chemical rock breaking compounds) will be required. It is recommended that an NSSP be included in the Contract Documents to warn the contractor of the strength and sloping characteristics of the bedrock; an example is included in Appendix F.

### 6.4.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the base of the concrete footing and the underlying bedrock should be calculated in accordance with Section 6.10.5 of the *CHBDC (2014)*.

For mass concrete placed on the bedrock surface, the design must check the sliding resistance between the base of the concrete footing and the top of the mass concrete, and between the base of the mass concrete and the bedrock. The coefficient of friction,  $\tan \delta$ , may be taken as 0.62 between the base of the cast-in-place concrete footings and mass concrete, and as 0.70 between the base of the mass concrete/concrete footings and bedrock. These values represent an unfactored value; in accordance with the *CHBDC (2014)*, the appropriate load and resistance factors should be applied in calculating the horizontal resistance.

If necessary, the sliding resistance can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the intact rock mass

is essentially as strong or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of concrete. The dowels should have a minimum embedded length within the unfractured (intact) bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded.

An ultimate grout-to-rock bond strength of 1,700 kPa can be used for design, applying the appropriate load and resistance factors (according to Table 6.2 of the CHBDC) to calculate the Ultimate Limit States (ULS) resistance. The geotechnical resistance at Serviceability Limit State (SLS) for 25 mm of displacement will be greater than the factored resistance at ULS; as such, ULS conditions will govern for this installation. The upper 0.3 m of the bond length should be ignored in the calculation of required bond length as the rock near surface may be weathered or disturbed. The actual bond strength for the rock-grout interface may vary from the typical design value given and should be verified by testing in the field. Dowels should be checked to ensure that the rock mobilization around the anchor can support the design load (i.e. check against conical rock mass failure). Closely spaced dowels should be checked for group interaction. If doweling into bedrock is adopted at this site, an NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels (an example is provided in Appendix F).

### 6.4.3 Frost Protection

For spread footings or mass concrete founded on the properly prepared intact bedrock at this site, frost protection is not required.

## 6.5 Drilled Steel Casings (Concrete Filled) Socketed into Bedrock

Consideration could be given to the use of drilled steel casings socketed into the bedrock for foundation design at both abutment locations. The procedure uses rotary duplex drilling techniques using a sacrificial ring bit on the bottom of a permanent steel casing. A down-the-hole hammer and flush system is employed to advance and seat the casing within the rock and also to create an uncased rock socket within the bedrock below the bottom of the casing. The steel casing can then be filled with tremie concrete (if there is water inflow through the bedrock) and reinforcing steel, as required.

### 6.5.1 Axial Geotechnical Resistance

The drilled steel casing pile is designed to develop the majority of its axial capacity based on the shear resistance along the rock socket wall (i.e. between the concrete and bedrock interface) rather than on end-bearing at the base of the socket. As such, the requirement to thoroughly clean and inspect the base of the socket will be lessened; however a thorough and proper flushing of the side walls of the rock socket is required.

The factored ultimate and serviceability (for 25 mm of settlement) axial geotechnical resistance that may be used for design are provided below:

Steel Casing Type	Factored Ultimate Axial Geotechnical Resistance(kN) <sup>1,2</sup>	Factored Serviceability Axial Geotechnical Resistance(kN) <sup>3</sup>
600 mm Diameter Drilled Steel Casings (tremie concrete filled)	5,000	n/a
324 mm Diameter Drilled Steel Casings (tremie concrete filled)	2,500	n/a

**Notes:**

<sup>1</sup> Values depend on structural capacity of the composite casing / concrete / pile and may need to be adjusted depending on final configuration, pipe steel grade, concrete strength, bedrock socket details, and reinforcing steel, if applicable

<sup>2</sup> Uncased rock socket length = 2 m (minimum)

<sup>3</sup> For pile founded in bedrock the factored serviceability resistance for 25 mm of settlement is greater than the factored ultimate resistance and therefore ULS condition governs

## 6.5.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the predominantly fill soils in front of the vertical drilled casings, and the reductions due to group effects, may be determined as per Section 6.6.2.

## 6.5.3 Frost Protection

The drilled casing / pile caps should be provided with a minimum of 1.7 m of soil cover for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically and perpendicular from the face of the abutment slope or ground surface to the edge of the underside of the pile cap. Alternatively, rigid insulation could be used to reduce the thickness of soil cover needed. As a guide, the MTO has adopted a 25 mm thickness of rigid polystyrene foam insulation as equivalent to a 0.3 m reduction in conventional soil cover. A layer of granular material (75 mm of Granular 'A') should be provided as a bedding and cover where such insulation is placed adjacent to rock fill.

## 6.6 Steel H-Pile Foundations

At the abutment locations, the foundation may be partially or completely supported on steel H-piles driven to bedrock, depending on which option is chosen (i.e. partially combined with spread footings or with bedrock excavated to provide sufficient minimum pile length and replaced with granular material).

The following anticipated bottom of pile cap, driven pile lengths, and bedrock elevations may be assumed for design at the abutment locations:

Foundation Element	Assumed Underside of Pile Cap Elevation (m)	Estimated Depth to Bedrock from Underside of Pile Cap		Estimated Bedrock Elevation (m)	
		West Side	East Side	Minimum (lowest)	Maximum (Highest)
North Abutment	260.4	8.7 m	2.3 m	251.7	258.1
South Abutment	260.6	3.3 m	1.0 m	257.3	259.6

Note: Depths and elevations shown represent the maximum anticipated range within each foundation element based on boreholes.

In order to provide a minimum pile embedment length of 3 m (as per MTO requirement), up to 2 m of bedrock would need to be pre-drilled / subexcavated and replaced with granular fill in some areas. Alternatively, the road could be

raised to allow for sufficient pile embedment depth. Given that the depth to bedrock ranges from about 1 m to 3.3 m (less than or near the minimum pile embedment depth) at the south abutment, this option is not considered practical at the south abutment location, as discussed above under Section 6.2 (Bridge Foundation Options).

Depending on whether the abutments are conventional, semi-integral or integral, the required pile embedment depth will vary. Based on the sloping bedrock, there should be a provision made in the contract to address the varying pile lengths due to the variable bedrock surface elevation. An example NSSP alerting the Contractor of the sloping bedrock has been included in Appendix F.

The presence of cobbles, rock fragments, and boulders was inferred at some borehole locations based on auger grinding and difficulty in auger advancement within the fill and native soil deposits, as noted on the borehole records. Consideration must be given to potential difficulties driving the piles to bedrock and potential for piles being deflected away from vertical and/or twisted due to the presence of cobbles and boulders, and difficulty in seating the piles due to the relatively steeply sloping bedrock at this site. In this regard, piles should be fitted with appropriate rock points (i.e. Titus "Rock Injector Model", Oslo Points as per OPSD 3000.201 or equivalent) however, even with the rock points, alignment and seating of the piles may be difficult due to the combination of obstructions in the fill, sloping bedrock and relatively thin overburden soils (i.e. short pile embankment lengths). An NSSP should be included in the contract to alert the Contractor of these conditions and is included in Appendix F for reference. Pile installation and rock points should be in accordance with OPSS.PROV 903 (Deep Foundations) and Special Provisions SP903S06 and SP109F57.

### 6.6.1 Axial Geotechnical Resistance

Based on the measured uniaxial compressive strength of the rock at this site and the bedrock quality, for a steel HP 310 x 110 pile driven to practical refusal, the factored ultimate axial geotechnical resistance of 1,200 kN may be assumed for design. The ULS value of 1,200 kN has been reduced to account for the combination of the following factors at this site

- The potential in difficulties in dealing with the sloping bedrock and potential for the piles sliding along the bedrock surface; and
- To account for the relatively low pile embedment lengths (typically ranges between 3 m and 8.7 m, assuming pre-drilling / bedrock subexcavation and granular backfill is required at some locations).

The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than the factored ultimate axial resistance, because the bedrock is considered to be an unyielding material; as such, ULS considerations will govern for this foundation type.

### 6.6.2 Resistance to Lateral Loads

Resistance to lateral loading may be derived using vertical piles, with enhanced support offered by inclined (battered) piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in front of the piles, whereas inclined piles derive lateral resistance from the soil in front of the piles as well as the horizontal component of the axial load present in the inclined pile. If integral or semi-integral abutments are considered, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections. However, given that the depth to bedrock is fairly shallow at some locations and pre-drilling / bedrock subexcavation is not preferred, integral abutments are likely not a practical option (i.e., it would be difficult to achieve a minimum 5 m pile length as required for integral abutments) as it would require additional bedrock excavation.

It should be noted that the response of a pile to lateral loads is highly non-linear and methods that assume linear behaviour (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than about 1 percent of the pile diameter, where the loading is static (non-cyclic) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear behaviour of the soil should be considered by the use of P-y curves.

The factored serviceability geotechnical response of the soil from 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,  $k_h$  (MPa/m) is based on the following equation for granular soils:

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

The following table provides the recommended range for the value of  $n_h$  (Terzaghi, 1995) to be used in the structural analysis. The range in values reflects the variability in the subsurface conditions and values used will depend on the design elevation of the pile cap. Design values are provided for the full stratigraphic sequence at the site.

Foundation Element	Soil Unit	$n_h$ (MPa/m)
North Abutment	New embankment fill, if applicable, assumed to be compacted Granular 'A' or 'B' fill above the groundwater / creek level	12
	New embankment fill or backfill in bedrock trench excavation / pre-drilled hole, assumed to be saturated compacted granular fill	8
	Existing very loose to very dense silty sand to sand to gravelly sand to sand and gravel (FILL) above the groundwater / creek level	10
	Existing very loose to very dense silty sand to sand to gravelly sand to sand and gravel (FILL) below the groundwater / creek level	6
	Existing loose native silty sand below the groundwater / creek level	4
South Abutment	New embankment fill, if applicable, assumed to be compacted Granular 'A' or 'B' fill above the groundwater / creek level	12
	New embankment fill or backfill in bedrock trench excavation / pre-drilled hole, assumed to be saturated compacted granular fill	8
	Existing very loose to very dense silty sand to gravelly sand to sand and gravel (FILL) above the groundwater / creek level	10
	Existing very loose to very dense silty sand to gravelly sand to sand and gravel (FILL) below the groundwater / creek level	6
	Existing saturated very dense silty sand and gravel	10

The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (see Section C6.11.2.2.2 of the *Commentary to the CHBDC, 2014*). The structural resistance of the piles should be evaluated to establish the governing case at ULS.

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than six to eight pile diameters between rows of driven steel H-piles. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor,  $R$  (NAVFAC DM7.1, 1986) as follows:

Pile Spacing in Direction of Loading ( $D$ = Pile Diameter)	Reduction Factor ( $R$ )
8D	1.00
6D	0.70
4D	0.40
3D	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above summary. Reduction for group effects is negligible when the centre-to-centre pile spacing exceeds three pile diameters measured in the direction perpendicular to loading.

### 6.6.3 Frost Protection

The pile caps should be provided with a minimum of 1.7 m of soil cover for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically and perpendicular from the face of the abutment slope or ground surface to the edge of the underside of the pile cap.

## 6.7 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the wall, the coefficient of lateral earth pressure will need to be adjusted to account for the slope as per the Commentary to the CHBDC (2014), Section C6.12.1.

- 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 subdrains 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 OPSD 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3 and Figure 6.6. Hand operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design, as required.



- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.7 m behind the back of the wall, per Figure C6.20(a) of the *Commentary to the CHBDC* (2014). For unrestrained walls, fill should be placed in a 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, per Figure C6.20(b) of the *Commentary to the CHBDC* (2014).
- For a restrained wall, the pressures are based on the existing and proposed embankment fill and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM):

Parameter	SSM (Granular Fill)
Soil Unit Weight	20 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:	
Active, $K_a$	0.33
At rest, $K_o$	0.50
Passive, $K_p$	3.00

- For an unrestrained wall, the pressures are based on the properties of the granular backfill and the following parameters (unfactored) may be used:

Parameter	Granular 'A'	Granular 'B' Type II
Soil Unit Weight	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
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 required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the *Commentary of the CHBDC*, 2014.
- 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.

## 6.8 Approach Embankment Design and Construction

Based on the investigation results, the subgrade soils within the existing and proposed approach embankment locations consist of a surficial layer of asphalt or topsoil underlain by fill consisting of silt and sand to sand to gravelly sand to sand and gravel, underlain by native silty sand to silty sand and gravel at some locations, underlain by strong to very strong bedrock. A bedrock outcrop was exposed at the ground surface southwest of the south approach embankment location. Rock fill was also observed covering the existing approach embankment side

slopes at the southeast, southwest, and northwest quadrants. The existing embankments are about 5 m high and their side slopes are generally inclined at about 2H:1V, becoming 1.5H:1V near the south abutment.

Referring to the General Arrangement (GA) drawing provided by MTO, the existing highway and approach embankments will be widened by about 1.7 m to the east. The road grade is not anticipated to change significantly (i.e. no grade raise). The proposed bridge span is to be increased to allow construction of the new abutments behind the existing abutments, shown to be about 7 m to 12 m away at the north abutment and about 2 m to 7 m away at the south abutment. As a result, the approach embankments will be excavated / cut back from the existing location (essentially 'unloading' the foundation soils) with a small amount of embankment fill added to the east side. The existing road grade is at about Elevation 262.3 m at the south approach embankment and about Elevation 262.1 m at the north approach embankment. The existing ground surface on the east side of the highway embankment is at about Elevation 259.0 m at the south approach and 260.5 m at the north approach and existing side-slopes are at about a 2H:1V slope. As a result, it is expected that less than 1 m thickness of new embankment fill will be added on the east side as part of the widening.

The General Arrangement drawing indicates that the new front slopes will be approximately 5 m high and will be inclined at 2H:1V at the north abutment and 1.5H:1V at the south abutment (similar to existing condition). The side-slope on the west side is assumed to remain unchanged and the new side-slope is assumed to be inclined at 2H:1V.

### 6.8.1 Subgrade Preparation and Embankment Construction

Prior to placement of engineered fill for the approach embankment widening, all topsoil/organic soils and loosened/softened fills must be removed from the footprint of the embankment widening (including on the existing embankment side slopes).

Engineered fill for construction of the new embankment widening should consist of Granular 'A' or 'B' or Select Subgrade Material (SSM) meeting the specifications of OPSS.PROV 1010 (Aggregates). Consideration could be given to using well graded rockfill if being placed on top of the existing rock fill-lined slopes. The embankment fill should be placed and compacted in accordance with OPSD 208.010 (Benching of Earth Slopes), OPSS.PROV 501 (Compacting) and OPSS.PROV 206 (Grading). Embankment side slopes should be designed and constructed no steeper than 2H:1V, except where rock fill is used, permitting slopes oriented at 1.5H:1V as proposed in the GA Drawing.

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments. Alternatively, and if this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with gravel sheeting or rock protection as per OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting), and OPSS.PROV 1004 (Aggregates – Miscellaneous) can be considered.

### 6.8.2 Global Stability

#### 6.8.2.1 Methodology

Although we understand that grades will not be raised and embankment heights / slopes will match the existing configuration, a global stability check at the approach embankments was carried out. Limit equilibrium slope stability analysis was performed using the commercially available program Slide (2018), produced by Rocscience Inc., employing the Morgenstern-Price method for analysis. For all analysis, the factor of safety of numerous potential failure surfaces were computed in order to establish the minimum Factor of Safety. 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 purposes of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor,  $\Psi$ , and

the geotechnical resistance factor  $\Phi_{gu}$  (i.e.  $FoS = 1/(\Psi * \Phi_{gu})$ ). Accordingly, a target minimum FoS of 1.5 has been used for the final embankment configuration.

### 6.8.2.2 Parameter Selection

The simplified stratigraphy together with the associated strength and unit weight employed for the different soil types at the critical embankment sections are summarized below.

Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Effective Friction Angle (°)	Effective Cohesion (c') (kPa)
Rock Fill	19	39	0
Compact to Very Dense Sand and Gravel to Gravelly Sand, contain cobbles	20	36	0
Very Loose to Dense Silty Sand to Sand to Sand and Gravel (Fill)	20	30	0
Very Loose Sand	20	27	0
Granitic Bedrock	25	>45	0

The overburden soils are generally comprised of loose to very dense cohesionless soils. For these soils, effective stress parameters were employed in the analysis assuming drained conditions using the results of the in situ Standard Penetration Tests (SPT) as suggested by Bowles (1997), in conjunction with engineering judgement based on experience in similar soil conditions.

### 6.8.2.3 Stability Analysis

Analyses were performed on critical sections of the approach embankment at the south and north sides (i.e. greatest fill height, steepest slope and thickest overburden deposit) to assess the global stability of the proposed configuration. The stability analyses were carried out using the existing embankment profiles assuming that the proposed embankment will be reconstructed to have side slopes at 2H:1V. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the Factor of Safety is equal to the inverse of the product of the consequence factor,  $\Psi$ , and the geotechnical resistance factor,  $\phi_{gu}$  (i.e.,  $FoS = 1/(\Psi \cdot \phi_{gu})$ ). Accordingly, minimum Factors of Safety of 1.33 and 1.54 have been targeted for the design of the embankment slopes for the short term/temporary and long term/permanent static conditions, as per Table 6.2 of CHBDC (2014).

The results of the stability analysis for the critical northwest approach embankment (i.e. highest embankment with the thickest overburden and the creek located directly near the toe) is shown in Figure 1.

Referring to Figure 1, the minimum calculated FoS against global instability was 1.3, where the critical slip surface was located within the side slope of the embankment; and, a minimum Fos of greater than 1.54 was calculated for a deep-seated failure that would affect the operation / shoulder of the highway. At the south abutment, for an embankment constructed out of granular fill with a front slope of 1.5H:1V below the pavement structure, the FoS

was calculated to be greater than 1.5 for a deep-seated failure that would affect the operation of the roadway. These factors of safety are considered to be appropriate for the design of the embankment widening at this site.

### 6.8.3 Settlement

Some settlement of the founding soils under the widened north and south approach embankment areas can be expected as a result of the loading from the new embankment fills placed for the embankment widening. The foundation soils typically consisted of a 3 m to 4 m thick deposit of loose to very dense silty sand to sand and gravel (fill) soils overlying bedrock. Assuming all surficial topsoil/organic soils and any loosened / softened fill soils will be removed prior to placing the new embankment fill (less than 1 m thick), settlement of the widened approach embankment is expected to be less than 25 mm, and this settlement is expected to be immediate and occur during construction.

## 6.9 Analytical Testing for Construction Materials

The results of an analytical testing on two soil samples are presented in Section 4.3 and in Appendix E. The analytical test results were compared to CSA A23.1 Table 3 (*Additional requirements for concrete subjected to sulphate attack*) for potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, based on the two samples of soil tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

The analytical test results of the soil samples were also compared to Table 2 of the U.S. Criteria for Assessing Ground Corrosion Potential (as derived from Federal Highways Administration (FHWA) 2003) for the potential attack on buried steel. The sulphate and chloride concentrations and the resistivity measured in sample SS12 from Borehole 16-1 indicate "Mild to no corrosion potential". In sample SS5&SS6 from Borehole 17-7, the sulphate concentrations and the resistivity measured indicates "Mild to no corrosion potential" and the chloride concentrations measured indicate "Strong corrosion potential". Based on the results of the samples tested, and given that the structure is located adjacent to the roadway and will be exposed to de-icing salt, consideration should be given by the designer to designing for a "C" type exposure class as defined by CSA A23.1 Table 1.

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

## 6.10 Design and Construction Considerations

### 6.10.1 Temporary Excavations

All temporary excavations must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OSHA), as amended.

The majority of the fill and native soils above the groundwater / creek water level at this site are classified according to OSHA as Type 3 soils, with the exception of the very loose layers (typically encountered at depth or below the creek water level) that would be classified as Type 4 soil. Temporary excavations (i.e. those that are open only for a relatively short period of time) in Type 3 soils should be made with side slopes no steeper than 1H:1V above the water table. For Type 4 soils, or if the excavations extend below the water table with adequate groundwater control not in place, temporary protection systems should be used, or the excavations should be made with side slopes no steeper than 4H:1V. Water seepage / inflow from perched groundwater near the overburden / bedrock interface is likely to occur which could cause temporary localised sloughing of the side slopes, until such time as the perched water drains and/or is adequately dewatered. For excavations into bedrock, if necessary, the overall

slope to the cut face may be formed vertically, or near vertically (i.e. about 0.25H:1V). Temporary excavations should be observed and reviewed during construction to confirm that the soil and groundwater conditions are as anticipated. If unexpected conditions are encountered, the Contractor's geotechnical engineer and dewatering specialist should review the excavation plan considering the actual groundwater and soil conditions at that time.

Depending on the foundation option that is chosen, subexcavation into the underlying bedrock may be required. It is noted that the bedrock is generally classified as strong to very strong. This will make rock excavation potentially difficult, particularly in areas where only small depths and narrow zones of removal are needed. Bedrock excavation in the vicinity of the proposed structure foundations should be carried out using controlled line drilling and pre-shearing techniques (as discussed in Sections 6.10.6 and 6.10.7).

### 6.10.2 Temporary Protection Systems

Based on the GA drawing for staging provided by MTO, we understand that the bridge is proposed to be replaced in halves in order to maintain traffic during construction. It is expected that temporary protection systems will be required along the mid-line of the highway to maintain a single lane of traffic along Highway 28 during construction of the new bridge structure. The temporary excavation support systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection System*) and Special Provision S105S19. The lateral movement should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent utilities can tolerate this magnitude of deformation. The selection and design of the protection system will be the responsibility of the contractor.

### 6.10.3 Groundwater and Surface Water Control

At the north abutment, groundwater was measured between about Elevation 258.7 m to 254.3 m, with an average of about Elevation 257 m which is the Eel's Creek water level shown on the GA drawing. At the south abutment, the majority of the boreholes were dry upon completion of drilling with the exception of one borehole that measured groundwater at Elevation 260.6 m. Although these water levels were recorded during drilling and may not represent the stabilized groundwater level at the site, it is anticipated that perched groundwater will be present near the overburden / bedrock interface with a stabilized groundwater level at or slightly above the creek water level (i.e. Elevation 257 m).

Depending on the selected foundation type for each abutment location, the amount / extent of dewatering will vary significantly. Excavations that extend below the groundwater / creek level will require more elaborate dewatering measures designed by dewatering specialist to ensure that foundation elements / backfill can be constructed in dry conditions. For excavations above the creek level, perched groundwater is expected to be encountered along the bedrock / overburden interface and may range from some water seepage to significant dewatering until such time as the perched zone is adequately drained. Filtered sump pumps combined with perimeter trenches are expected with more elaborate dewatering systems required at locations where the existing Highway 28 grade and surface water collection ditches are intercepted.

For excavations below the creek water level (such as at the North Abutment and possibly near the west limit of the South Abutment, depending on the foundation option selected), a combined dewatering system using sheetpile cut-off walls and well points or the use of sheetpile cut-off walls and tremie concrete plug may be required. Difficulties achieving a watertight seal between the sheet piles and the sloping bedrock should be anticipated and dewatering efforts could be significant to keep excavations dry and minimize the loss of soils (with associated settlement) from behind the sheetpile wall. A seal will need to be placed at the sheet pile and soil/rock interface in order to minimize the potential for fines migrating into the base of the excavation and control water inflow.

Dewatering operations to allow for construction in the dry should be managed in accordance with OPSS.PROV 517 (Dewatering) and OPSS 902 (Excavating and Backfilling – Structures), as modified by the NSSP FOUN0003 (Dewatering of Structure Excavation). A copy of NSSP FOUN0003 is provided in Appendix F, with the applicable foundation-related fill-in section completed.

#### 6.10.4 Obstructions

The presence of cobbles, rock fragments and/or boulders was inferred based on grinding of the augers during borehole advancement within the embankment fill, as noted on the borehole records. It should be noted that cobbles and boulders could affect excavation, installation of the temporary protection systems, dewatering systems, and deep foundations. An NSSP should be included in the contract documents to alert the contractor of such potential construction difficulties; an example is included in Appendix F for reference.

#### 6.10.5 Removal of Existing Foundations

The bridge replacement will require removal of the existing abutment walls, wingwalls, and possibly portions of the foundations. It is recommended that foundation removals be limited to removal of the abutment walls and wingwalls only and no extraction / removal of spread footings be required unless there is a specific conflict, which may be the case at the south abutment depending on the final location/skew of the new abutment.

#### 6.10.6 Bedrock Excavation

Excavation of the bedrock may be required at shallow depths to ensure a level founding surface where spread footings or mass concrete is to be used. Due to the risk of impacting the existing or new foundations through vibrations or rock over-break, blasting must be carefully designed and controlled by a blasting engineer to limit over-excavation and minimize impacts to the new or existing structure foundations as the new abutment foundations will be in close proximity to the existing abutments during staged construction. Bedrock excavation could also be carried out using line drilling and hydraulic splitting or using a non-explosive chemical rock-breaking compounds.

For excavations into the bedrock, the overall slope to the cut face may be formed vertical or at a steep near-vertical slope (i.e. 0.25H:1V). The use of controlled blasting techniques (such as pre-shearing or cushion blasting) are recommended, particularly along footing areas, in order to provide a neat excavation line and minimize face instabilities resulting from damage to the rock mass.

All blasting and/or bedrock excavation should be carried out in accordance with OPSS.PROV 120 and OPSS.PROV 202, which also reference OPSS.PROV 206.

#### 6.10.7 Bedrock Subgrade Inspection

Immediately following completion of excavation of the bedrock to the founding level, the footing subgrade should be inspected by a Foundation Engineer, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) as amended by SP109S12, to check that all existing fill materials, concrete and fractured or loosened portions of the bedrock are removed prior to construction of the footings for the abutments.



## 7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Katelyn Nero and reviewed by Mr. Mathew Kelly, P.Eng., a geotechnical engineer with Golder. Mr. Kevin Bentley, P.Eng., an MTO Designated Foundations Contact and Associate of Golder, and Ms. Lisa Coyne, P.Eng., a Principal and MTO Designated Foundations Contact of Golder, conducted a technical and quality control review of the report.

### Golder Associates Ltd.



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KN/MK/KJB/LCC/rb

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## REFERENCES

Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual (CFEM), 4<sup>th</sup> Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.

Canadian Highway Bridge Design Code (CHBDC (2014)) and Commentary on CAN/CSA-S6-14. Canadian Standard Association. (CSA) Group.

Canadian Standards Association (CSA). 2014. Concrete materials and methods of concrete construction / Test methods and standard practices for concrete (CSA A23.1-14/A23.2-14).

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

National Research Council of Canada. 2015. National Building Code of Canada

Natural Resources Canada. 2015. Geological Survey of Canada – Seismic Hazard Model for Canada

Terzaghi, K., 1955. Evaluation of Coefficients of Subgrade Reaction/ Geotechnique, Vol. 5, No. 4, pp. 297-326. Discussion in Vol. 6, No. 2, pp. 94-98.

Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manual 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

Federal Highway Administration (Federal Highway Administration, 2015), National Highway Institute Publication No. FHWA/NHI14007, 1978.

### ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D7012	Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

### Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91 Construction Projects (as amended)

### Ontario Provincial Standard Specification and Special Provisions:

OPSS.PROV 120	General Specification for the Use of Explosives
OPSS.PROV 202	Construction Specification for Rock Removal by Manual Scaling, Machine Scaling, Trim Blasting, or Controlled Blasting
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems

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OPSS.PROV 802	Construction Specification for Topsoil
OPSS.PROV 803	Construction Specification for Sodding
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
SP105S19	Amendment to OPSS.PROV 539
SP109F57	Amendment to OPSS.PROV 903
SP109S12	Amendment to OPSS 902
SP FOUN0003	Dewatering Structure Excavations
SP 903S06	Amendment to OPSS.PROV 903

**Ontario Provisional Standard Drawing:**

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.201	Foundation, Piles, Steel HP 310 Oslo Pint
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutments, Walls

**Ontario Water Resources Act:**

Ontario Regulation 903 Wells (as amended)

**Table 1: Comparison of Foundation Alternatives for South Abutment – Eel’s Creek Bridge Replacement**

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Spread Footings founded on bedrock and/or on mass concrete placed on bedrock	1	<ul style="list-style-type: none"> <li>Can minimize or eliminate bedrock excavation;</li> <li>Conventional construction techniques.</li> </ul>	<ul style="list-style-type: none"> <li>Sloping bedrock surface will likely require some bedrock excavation (e.g. controlled blasting) and/or installation of rock dowels.</li> <li>Subexcavation of 3 m to 5 m below road surface anticipated to reach bedrock surface, resulting in the need for robust temporary protection system</li> </ul>	<ul style="list-style-type: none"> <li>Lowest cost option (assuming similar temporary protection system used for other options)</li> </ul>	<ul style="list-style-type: none"> <li>Variability in bedrock surface will impact excavation depths and mass concrete quantities;</li> <li>Risk of difficulties installing dowels or bedrock excavation in relatively strong bedrock</li> <li>Risk of disturbing / damaging existing bridge foundations during bedrock excavation / removal process (e.g. controlled blasting)</li> </ul>
Drilled Steel Casings (Concrete Filled) socketed into bedrock (diameter typically greater than about 300 mm)	2	<ul style="list-style-type: none"> <li>With down hole hammer, allows for better control of the socketing into strong to very strong bedrock;</li> <li>Decreases likelihood of piles deflecting off of sloping bedrock as compared to driven piles;</li> <li>Increased capacity when compared to steel H-piles or smaller diameter casings/micropiles;</li> <li>With pile cap maintained as high as possible, excavation and dewatering requirements are minimized.</li> <li>Smaller equipment compared to caisson / pile driving operation</li> </ul>	<ul style="list-style-type: none"> <li>Specialized drilling equipment required, and sloping bedrock may cause difficulties;</li> </ul>	<ul style="list-style-type: none"> <li>Higher relative costs than spread footings, although savings in excavation and temporary protection system efforts</li> <li>Lower dewatering costs for construction of pile cap compared to spread footings founded on bedrock and/or mass concrete placed on bedrock.</li> <li>Comparable costs to smaller diameter casing sizes / micropiles</li> </ul>	<ul style="list-style-type: none"> <li>Design elevations can be achieved by drilling through any possible cobbles/boulders/rock fragments encountered and into the bedrock</li> <li>Risk of disturbing existing foundations due to vibrations using down the hole hammer</li> </ul>
Micropiles (diameter typically less than about 300 mm) socketed into bedrock	3	<ul style="list-style-type: none"> <li>With down hole hammer, allows for better control of the socketing into strong to very strong bedrock;</li> <li>Decreases likelihood of piles deflecting off of sloping bedrock as compared to driven piles;</li> <li>With pile cap maintained as high as possible, excavation and dewatering requirements are minimized.</li> <li>With pile cap maintained as high as possible, excavation and dewatering requirements are minimized.</li> <li>Smaller equipment compared to caisson / pile driving operation</li> </ul>	<ul style="list-style-type: none"> <li>Specialized drilling equipment required, and sloping bedrock may cause difficulties.</li> <li>Lower geotechnical resistance for smaller diameter friction or end-bearing pile design, however this can be offset by adding more piles;</li> <li>Lower lateral geotechnical resistance, especially for micropiles.</li> </ul>	<ul style="list-style-type: none"> <li>Higher relative costs than spread footings</li> <li>Comparable costs to drilled steel casings (&gt;300 mm diameter)</li> </ul>	<ul style="list-style-type: none"> <li>Design elevations can be achieved by drilling through any possible cobbles/boulders/rock fragments encountered and into the bedrock</li> <li>Risk of disturbing existing foundations due to vibrations using down the hole hammer, although risk is less for smaller diameter installations;</li> <li>High risk of difficulties achieving required lateral resistance with small diameter piles / micropiles.</li> </ul>
Caissons founded on bedrock	NP	<ul style="list-style-type: none"> <li>High geotechnical resistance if socketed into bedrock</li> </ul>	<ul style="list-style-type: none"> <li>Difficulties achieving clean base on sloping bedrock would result in socketing large diameter caissons into strong to very strong rock or use of rock dowels</li> <li>Large diameter sockets or installation of rock dowels within caissons not considered practical compared to other options</li> </ul>	<ul style="list-style-type: none"> <li>Not economical due to large diameter bedrock sockets</li> </ul>	<ul style="list-style-type: none"> <li>High risk of difficulties achieving bedrock socket in strong to very strong bedrock with sloping surface</li> <li>Large caisson rig required and limited work zone assuming half-and-half construction staging</li> </ul>
Steel H-Piles driven to found on bedrock  (sub excavate / drill bedrock and replace with compacted granular fill where required to achieve a minimum 3 m pile embedment length)	NP	<ul style="list-style-type: none"> <li>Allows for integral abutment design; however significant pre-drilling / rock excavation would be required for and not considered practical</li> </ul>	<ul style="list-style-type: none"> <li>Low geotechnical resistance for short piles on sloping bedrock</li> <li>Assuming pile cap bottom is below frost depth, pile lengths anticipated to range from about 1 m to 3.3 m, typically near or less than preferred minimum pile embedment length of 3 m</li> <li>Significant amount of bedrock excavation or pre-drilling into bedrock would be required to reach minimum pile lengths, especially for integral abutment design</li> </ul>	<ul style="list-style-type: none"> <li>Higher costs compared to spread footings for bedrock subexcavation / pre-drilling to achieve minimum pile length of 3 m or more</li> </ul>	<ul style="list-style-type: none"> <li>Sloping bedrock will reduce geotechnical resistances and given short pile lengths, make this option not practical</li> <li>Potential difficulties predrilling or subexcavating through strong to very strong bedrock;</li> <li>Potential for piles to be deflected away from vertical (i.e. seating problems) during driving (especially with low overburden thickness).</li> <li>Potential for piles to “hang-up” on cobbles, boulders, rock fragments or be deflected away from vertical during driving (especially with low overburden thickness).</li> </ul>
Spread footings supported on overburden soils	NP	-	<ul style="list-style-type: none"> <li>Existing fill soils are not considered competent to support spread footings</li> </ul>	-	<ul style="list-style-type: none"> <li>Compaction / control of existing fill placement is unknown</li> <li>High risk of settlement and low geotechnical resistance values make this option not practical</li> </ul>

NP = Not Practical

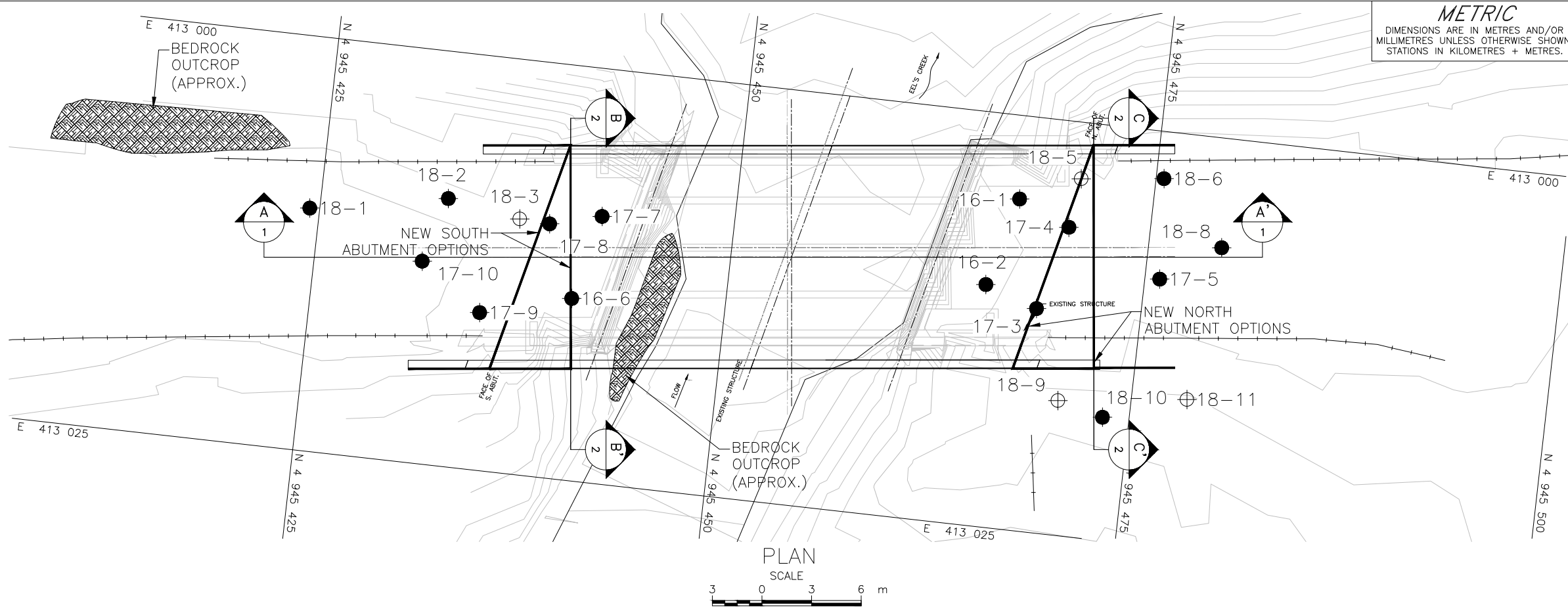
**Table 2: Comparison of Foundation Alternatives for North Abutment – Eel’s Creek Bridge Replacement**

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Drilled Steel Casings (Concrete Filled) socketed into bedrock (diameter typically greater than about 300 mm)	1	<ul style="list-style-type: none"> <li>With down hole hammer, allows for better control of the socketing into strong to very strong bedrock;</li> <li>Decreases likelihood of piles deflecting off of sloping bedrock as compared to driven piles;</li> <li>Increased capacity when compared to steel H-piles or smaller diameter casings / micropiles;</li> <li>With pile cap maintained as high as possible, excavation and dewatering requirements are minimized.</li> <li>Smaller equipment compared to caisson / pile driving operation</li> </ul>	<ul style="list-style-type: none"> <li>Specialized drilling equipment required, and sloping bedrock may cause difficulties;</li> </ul>	<ul style="list-style-type: none"> <li>Lower costs compared to full subexcavation and replacement with mass concrete / spread footings, except near east limit where bedrock is shallower</li> <li>Higher costs than driven piles</li> <li>Comparable costs to smaller diameter casing sizes / micropiles</li> </ul>	<ul style="list-style-type: none"> <li>Design elevations can be achieved by drilling through any possible cobbles/boulders/rock fragments encountered within the fill and into the bedrock</li> <li>Risk of disturbing existing foundations due to vibrations using down the hole hammer</li> </ul>
Combined Spread Footings founded on bedrock and Drilled Steel Casings (Concrete Filled) socketed into bedrock (diameter typically greater than about 300 mm)	2	<ul style="list-style-type: none"> <li>Conventional construction techniques for shallow footings / excavation near east limit of abutment and east wingwall only (where bedrock is shallow above creek water level);</li> <li>For remaining portion of abutment and west wingwall, drilled steel casings with down hole hammer allows for better control of the socketing into strong to very strong bedrock;</li> <li>Decreases likelihood of piles deflecting off of sloping rock as compared to driven piles;</li> <li>With pile cap maintained as high as possible, excavation and dewatering requirements are minimized.</li> </ul>	<ul style="list-style-type: none"> <li>Different construction procedures / sequencing required for use of shallow and deep foundations within single foundation footprint;</li> <li>Specialized drilling equipment required, and sloping bedrock may cause difficulties;</li> <li>Spread footing excavation depths in the order of 3 m to 5 m are anticipated (so as not to excavate below creek water level and allow for minimum 3 m pile embedment) and will only cover east wingwall foundation footprint and about 10% of abutment foundation footprint with remaining area needing deep foundations;</li> </ul>	<ul style="list-style-type: none"> <li>Costs comparable to using drilled steel casings for entire foundation footprint.</li> <li>Higher relative costs than driven pile foundations since more specialized equipment required;</li> <li>Lower costs compared to full subexcavation and replacement with mass concrete / spread footings.</li> </ul>	<ul style="list-style-type: none"> <li>Design elevations / founding subgrade can be achieved / verified by exposing bedrock surface for shallow foundation component and by drilling through any possible cobbles/boulders/rock fragments encountered in the fill and socketing into the bedrock for steel casing component</li> <li>Risk of differential settlement / movement using both shallow and deep foundation options across length of single foundation element.</li> <li>Risk of disturbing existing foundations due to vibrations using down the hole hammer</li> </ul>
Micropiles (diameter typically less than about 300 mm) socketed into bedrock	3	<ul style="list-style-type: none"> <li>With down hole hammer, allows for better control of the socketing into strong to very strong bedrock;</li> <li>Decreases likelihood of piles deflecting off of sloping bedrock as compared to driven piles;</li> <li>With pile cap maintained as high as possible, excavation and dewatering requirements are minimized.</li> <li>With pile cap maintained as high as possible, excavation and dewatering requirements are minimized.</li> <li>Smaller equipment compared to caisson / pile driving operation</li> </ul>	<ul style="list-style-type: none"> <li>Specialized drilling equipment required, and sloping bedrock may cause difficulties.</li> <li>Lower geotechnical resistance for smaller diameter friction or end-bearing pile design, however this can be offset by adding more piles;</li> <li>Lower lateral geotechnical resistance, especially for micropiles.</li> </ul>	<ul style="list-style-type: none"> <li>Lower costs compared to full subexcavation and replacement with mass concrete / spread footings, except on east side</li> <li>Comparable costs to drilled steel casings (&gt;300 mm diameter)</li> <li>Higher costs than driven piles</li> </ul>	<ul style="list-style-type: none"> <li>Design elevations can be achieved by drilling through any possible cobbles/boulders/rock fragments encountered and into the bedrock</li> <li>Risk of disturbing existing foundations due to vibrations using down the hole hammer, although risk is less for smaller diameter installations;</li> <li>High risk of difficulties achieving required lateral resistance with small diameter piles / micropiles.</li> </ul>
Combined Spread Footings founded on bedrock and Steel H-Piles driven to found on bedrock	4	<ul style="list-style-type: none"> <li>Conventional construction techniques for shallow footings near east limit of abutment and east wingwall only (where bedrock is shallow and above creek water level);</li> <li>For remaining portion of abutment and west wingwall, driven steel H-piles (minimum 3 m embedment) with specialized driving shoes to bite into strong to very strong bedrock;</li> <li>With pile cap maintained as high as possible, excavation and dewatering requirements are minimized.</li> </ul>	<ul style="list-style-type: none"> <li>Different construction procedures / sequencing required for use of shallow and deep foundations within single foundation footprint.</li> <li>Spread footing excavation depths in the order of 3 m to 5 m are anticipated (so as not to excavate below creek water level and allow for minimum 3 m pile embedment) and will only cover east wingwall foundation footprint and about 10% of abutment foundation footprint with remaining area needing deep foundations;</li> <li>Lower design geotechnical resistance for short piles on sloping bedrock compared to conventional design</li> </ul>	<ul style="list-style-type: none"> <li>Lower cost than combined spread footing and steel casing socketed into bedrock option</li> <li>Lower cost than all spread footing or all pile driving option</li> </ul>	<ul style="list-style-type: none"> <li>Design elevation / founding subgrade can be achieved / verified by exposing bedrock surface for shallow foundation component;</li> <li>Potential for piles to be deflected away from vertical (i.e. seating problems) during driving (especially in with lower overburden thickness).</li> <li>Potential for piles to “hang-up” on cobbles, boulders, rock fragments or be deflected away from vertical during driving (especially with low overburden thickness).</li> <li>Risk of differential settlement / movement using both shallow and deep foundation options across length of single foundation element.</li> <li>Risk of disturbing existing foundations due to vibrations from pile driving</li> </ul>

Foundation Option	Rank	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Steel H-Piles driven to found on bedrock  (sub excavate / drill bedrock and replace with compacted granular fill where required to achieve a minimum 3 m pile embedment length)	5	<ul style="list-style-type: none"><li>■ Allows for integral abutment design; however significant pre-drilling / rock excavation would be required for minimum 5 m embedment length and not considered practical</li><li>■ Conventional construction techniques for semi-integral design in areas where pile embedment is more than 3 m</li></ul>	<ul style="list-style-type: none"><li>■ Lower design geotechnical resistance for short piles on sloping bedrock compared to conventional design</li><li>■ Assuming bottom of pile cap is below frost depth, pile lengths anticipated to range from 2.5 m to 10 m to reach bedrock; which is less than or near the accepted minimum pile embedment length of 3 m</li><li>■ Some bedrock excavation or pre-drilling into bedrock would be required to reach minimum pile lengths on east side, especially for integral abutment design</li></ul>	<ul style="list-style-type: none"><li>■ Lower costs compared to other options in area to achieve minimum pile length of 3 m or more, as required</li><li>■ Cost will increase if pre-drilling required to achieve minimum pile embedment lengths.</li></ul>	<ul style="list-style-type: none"><li>■ Potential difficulties pre-drilling or subexcavating through strong to very strong bedrock;</li><li>■ Potential for piles to be deflected away from vertical (i.e. seating problems) during driving (especially in with lower overburden thickness).</li><li>■ Potential for piles to “hang-up” on cobbles, boulders, rock fragments or be deflected away from vertical during driving (especially with low overburden thickness).</li><li>■ Risk of disturbing existing foundations due to vibrations from pile driving</li></ul>
Caissons founded on bedrock	6	<ul style="list-style-type: none"><li>■ High geotechnical resistance if socketed into bedrock</li></ul>	<ul style="list-style-type: none"><li>■ Difficulties achieving clean, dry base on sloping bedrock would result in socketing large diameter caissons into strong to very strong rock or use of rock dowels</li><li>■ Large diameter sockets or installation of rock dowels within caissons not considered practical compared to other options</li><li>■ Specialized installation using drilling mud and tremie concrete required unless sealed into bedrock</li></ul>	<ul style="list-style-type: none"><li>■ Not economical for large diameter bedrock sockets</li></ul>	<ul style="list-style-type: none"><li>■ High risk of difficulties achieving bedrock socket in strong to very strong bedrock with sloping rock surface</li><li>■ Large caisson rig required and limited work zone assuming half-and-half construction staging</li></ul>
Spread Footings founded on bedrock and/or on mass concrete placed on bedrock	NP	<ul style="list-style-type: none"><li>■ Only practical near east limit of abutment and east wing wall</li><li>■ Can minimize or eliminate bedrock excavation;</li><li>■ Conventional construction techniques for shallow excavation above groundwater / creek water level on east limit only, specialized equipment required elsewhere.</li></ul>	<ul style="list-style-type: none"><li>■ Subexcavation depths through existing fills anticipated to range from about 5 m to greater than 10 m below highway surface for majority of abutment footprint</li><li>■ Tremied concrete placement and/or dewatering required for majority of footprint, except near east limit. Sheet pile box or formwork likely required to limit size of excavation, control groundwater and control mass concrete quantities;</li><li>■ Variable bedrock surface may require bedrock excavation or installation of rock dowels (below groundwater level).</li><li>■ Sealing sheet pile box below groundwater on strong sloping bedrock will be difficult.</li></ul>	<ul style="list-style-type: none"><li>■ Higher costs compared to other options due to large amount of subexcavation, dewatering requirements and/or placement of mass concrete below water table.</li></ul>	<ul style="list-style-type: none"><li>■ Variability in bedrock surface will impact excavation depths and mass concrete quantities;</li><li>■ Risk of difficulty in achieving seal in sloping bedrock for sheet piles for dewatering;</li><li>■ Risk of difficulties installing dowels or bedrock excavation below groundwater level.</li></ul>
Spread footings supported on overburden soils	NP	-	<ul style="list-style-type: none"><li>■ Existing fill soils are not considered competent to support spread footings for new bridge</li></ul>	-	<ul style="list-style-type: none"><li>■ Compaction / control of existing fill placement is unknown</li><li>■ High risk of settlement and low geotechnical resistance values make this option not practical</li></ul>

NP = Not Practical



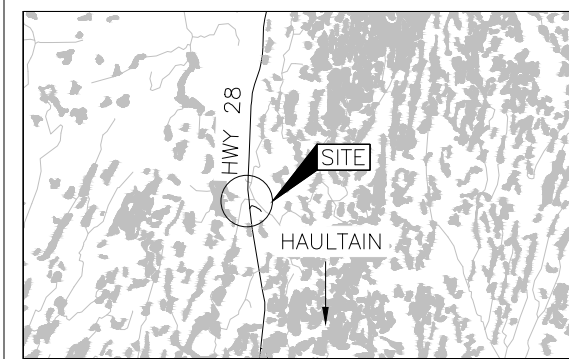


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No.

HWY 28  
EEL'S CREEK BRIDGE REPLACEMENT  
BOREHOLE LOCATION AND SOIL  
STRATA

SHEET

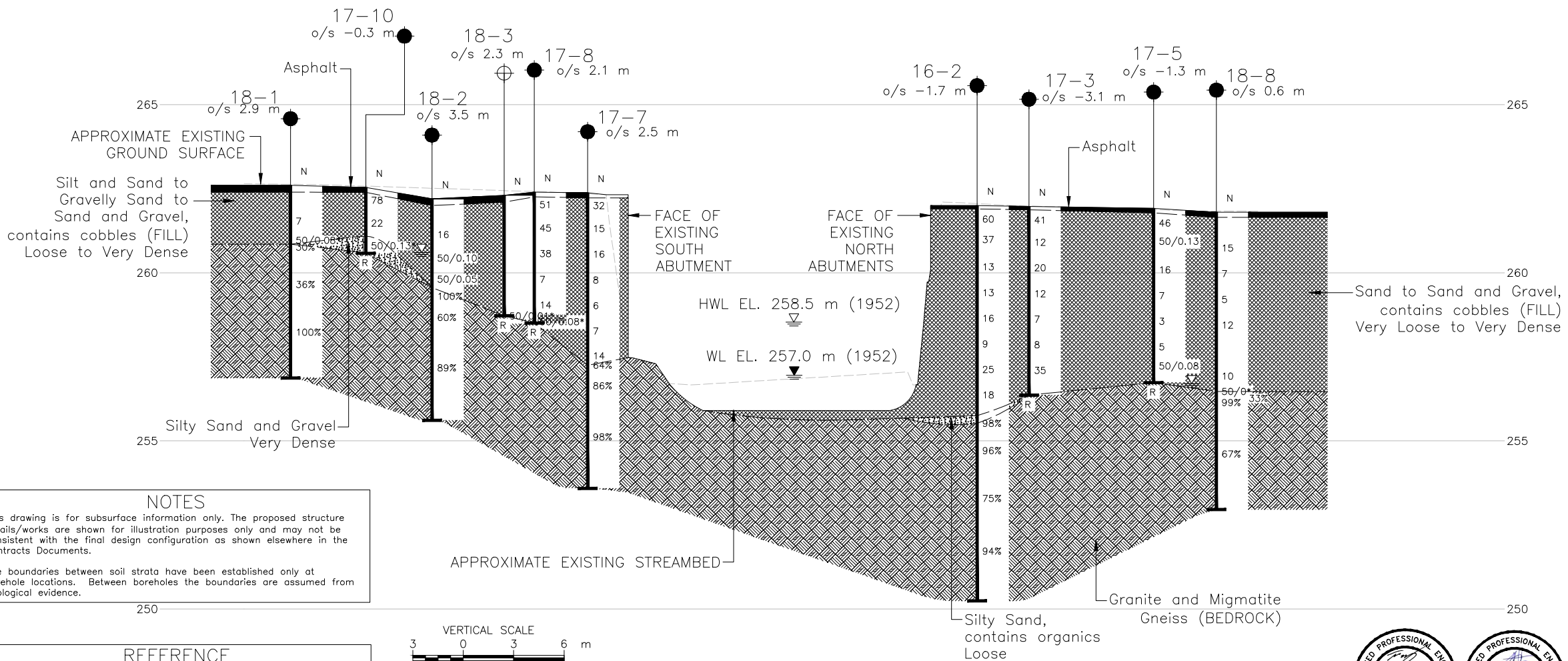


KEY PLAN  
SCALE  
1 0 1 2 km

#### LEGEND

- Borehole - Current Investigation
- ⊕ Probehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL upon completion of drilling
- R Auger refusal on inferred bedrock

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16-6	262.4	4945440.7	413014.0
16-1	262.0	4945466.9	413005.0
16-2	262.0	4945465.5	413010.4
17-10	262.5	4945431.5	413012.8
17-9	262.5	4945435.3	413015.5
17-8	262.4	4945438.9	413009.7
17-7	262.4	4945442.0	413008.9
17-5	261.9	4945475.9	413008.9
17-4	262.0	4945470.1	413006.4
17-3	262.0	4945468.7	413011.5
18-5	261.9	4945470.5	413003.4
18-3	262.3	4945437.1	413009.6
18-9	260.7	4945470.6	413016.9
18-11	261.0	4945478.3	413015.9
18-10	260.7	4945473.4	413017.6
18-8	261.8	4945479.4	413006.6
18-6	261.8	4945475.5	413002.8
18-2	262.2	4945432.7	413008.9
18-1	262.6	4945424.4	413010.4



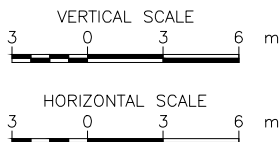
#### NOTES

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 MTO, drawing file no. BC320281.dwg, received October 31, 2018.  
General arrangement plan provided in digital format by MTO, drawing file no. 26-117 GENERAL ARRANGEMENT.DWG, received October 22, 2018.  
General arrangement plan provided by MTO, DWG NO. 1, SITE 26-117 EEL'S CREEK BRIDGE GENERAL ARRANGEMENT, HWY 28, dated April 2016.



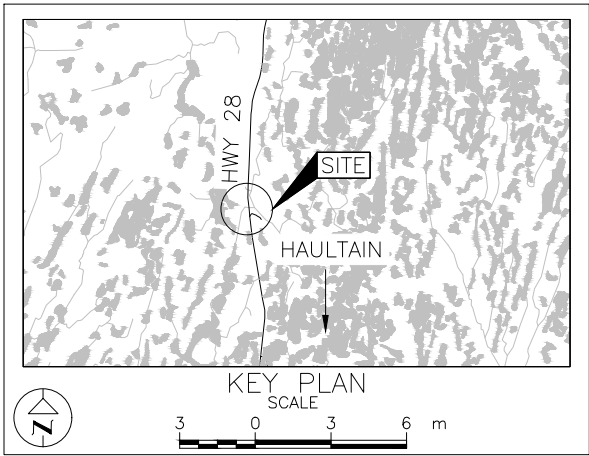
#### CROSS-SECTION A-A'



NO.	DATE	BY	REVISION
Geocres No. 31D-727			
HWY. 28	PROJECT NO. 1413191/1895756		DIST. EASTERN
SUBM'D. KN	CHKD. MWK	DATE: 05/10/2019	SITE: 26-117
DRAWN: DD	CHKD. MWK	APPD. KJB	DWG. 1

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

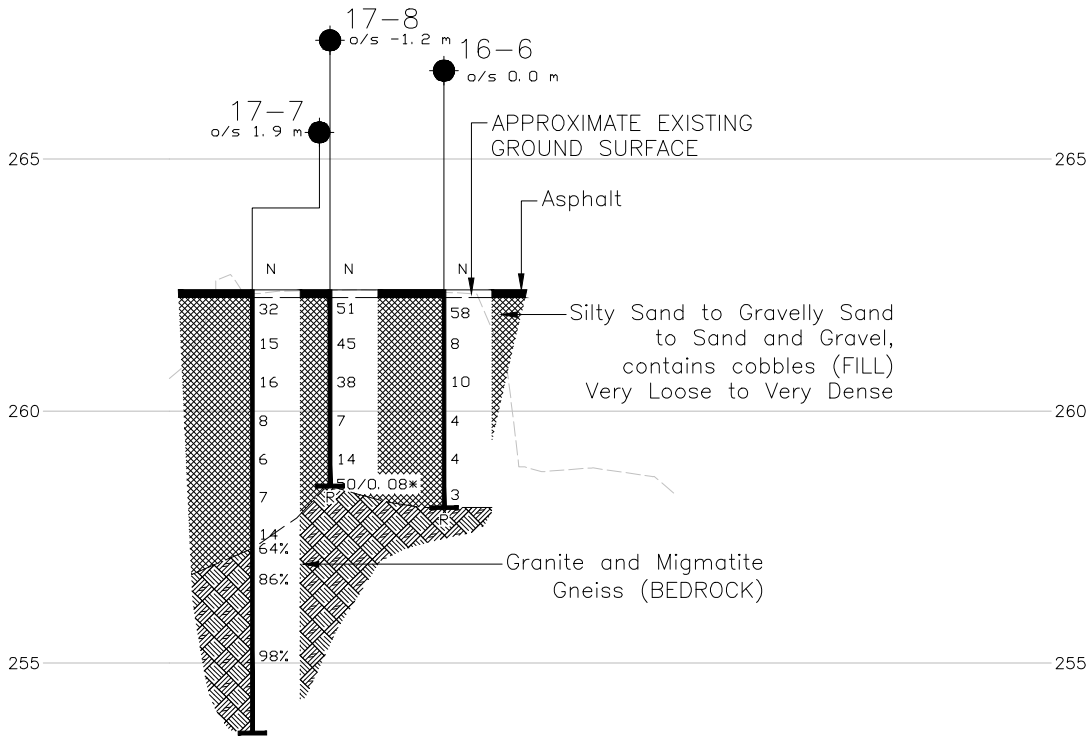
CONT No.		SHEET
WP No.		
HWY 28 EEL'S CREEK BRIDGE REPLACEMENT SOIL STRATA		



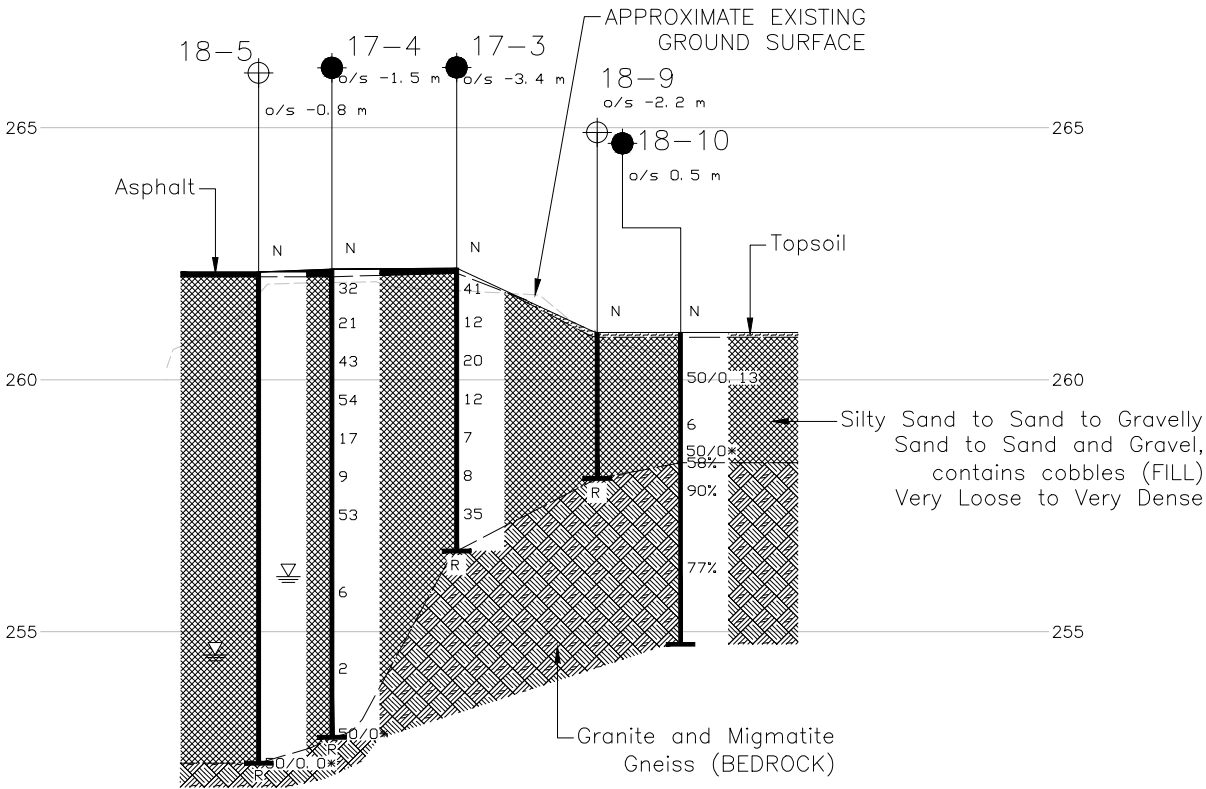
LEGEND

- Borehole – Current Investigation
- ⊕ Probehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ≡ WL upon completion of drilling
- R Auger refusal on inferred bedrock

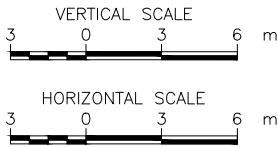
BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16-6	262.4	4945440.7	413014.0
16-1	262.0	4945466.9	413005.0
16-2	262.0	4945465.5	413010.4
17-10	262.5	4945431.5	413012.8
17-9	262.5	4945435.3	413015.5
17-8	262.4	4945438.9	413009.7
17-7	262.4	4945442.0	413008.9
17-5	261.9	4945475.9	413008.9
17-4	262.0	4945470.1	413006.4
17-3	262.0	4945468.7	413011.5
18-10	260.7	4945473.4	413017.6
18-8	261.8	4945479.4	413006.6
18-6	261.8	4945475.5	413002.8
18-2	262.2	4945432.7	413008.9
18-1	262.6	4945424.4	413010.4
18-5	261.9	4945470.5	413003.4
18-3	262.3	4945437.1	413009.6
18-9	260.7	4945470.6	413016.9
18-11	261.0	4945478.3	413015.9



CROSS-SECTION B-B' – SOUTH ABUTMENT



CROSS-SECTION C-C' – NORTH ABUTMENT



NOTES

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REFERENCE

Base plans provided in digital format by MTO, drawing file no. BC320281.dwg, received October 31, 2018.

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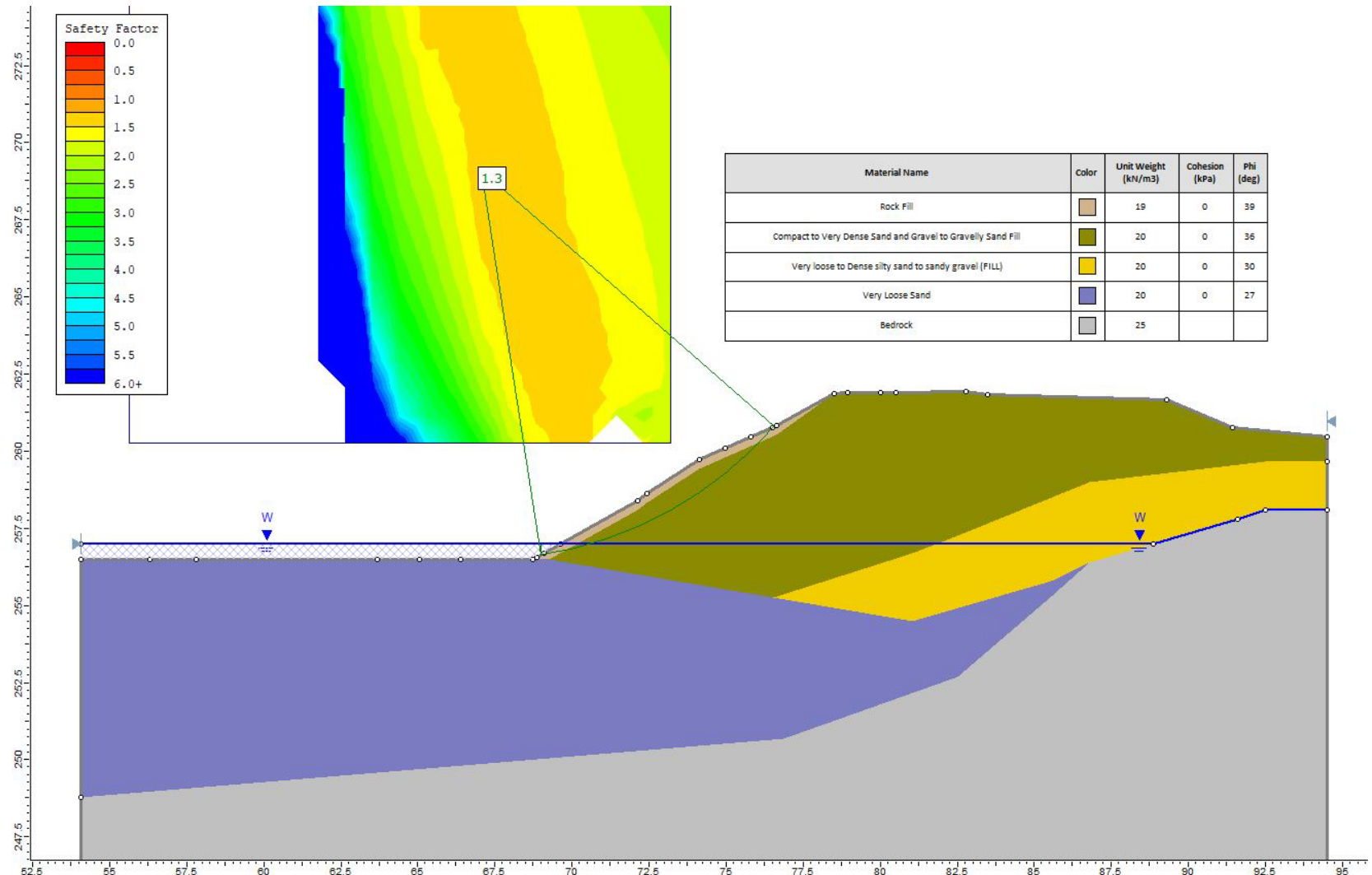


NO.	DATE	BY	REVISION
Geocres No. 31D-727			
HWY. 28	PROJECT NO. 1413191/1895756		DIST. EASTERN
SUBM'D. KN	CHKD. MWK	DATE: 05/10/2019	SITE: 26-117
DRAWN: DD/SW	CHKD. MWK	APPD. KJB	DWG. 2

# Global Stability Analysis

Figure 1

## Replacement of Eel's Creek Bridge (Site No. 26-117) Long-Term (Drained) Analysis



**APPENDIX A**  
**Record of Boreholes**

## LIST OF SYMBOLS

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

### I. GENERAL

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

### II. STRESS AND STRAIN

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

### III. SOIL PROPERTIES

#### (a) Index Properties

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

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

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

#### (b) Hydraulic Properties

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

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

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

#### (d) Shear Strength

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

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

Notes: 1  
2

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

## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPE

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

### II. PENETRATION RESISTANCE

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

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

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

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

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

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

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

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

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

### III. SOIL DESCRIPTION

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

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

#### (b) Cohesive Soils Consistency

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

### IV. SOIL TESTS

w	water content
$w_p$	plastic limit
$w_l$	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
$SO_4$	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



## 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

<u>Description</u>	<u>Bedding Plane Spacing</u>
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

<u>Description</u>	<u>Spacing</u>
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

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

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

## CORE CONDITION

### Total Core Recovery (TCR)

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

### Solid Core Recovery (SCR)

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

### Rock Quality Designation (RQD)

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

## DISCONTINUITY DATA

### Fracture Index

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

### Dip with Respect to Core Axis

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

### Description and Notes

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

### Abbreviations

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

PROJECT		1413191 (1150)		RECORD OF BOREHOLE No 16-1		SHEET 1 OF 2		METRIC															
G.W.P.				LOCATION		N 4945466.9; E 413005.0 MTM NAD 83 ZONE 10 (LAT. 44.641072; LONG. -78.136013)		ORIGINATED BY															
DIST		Eastern HWY 28		BOREHOLE TYPE		114 mm O.D. HW Casing and Wash Boring		COMPILED BY															
DATUM		Geodetic		DATE		December 10, 2016		CHECKED BY															
								MWK															
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100	20	40	60	80	100	10
262.0	GROUND SURFACE																						
0.0	ASPHALT (150 mm)																						
261.5	Sand and gravel, trace to some silt (FILL)		1A	SS	60																		
0.5	Grey Moist		1B																				
	Sand, some gravel, some silt (FILL)		2	SS	28																		
	Compact																						
	Brown Moist		3	SS	18																		
259.8																							
2.2	Sandy gravel, some silt to silty, trace clay (FILL)		4	SS	18																		
	Compact to dense																						
	Brown Moist to wet		5	SS	17																		
			6	SS	35																		
			7	SS	12																		
256.7																							
5.3	Sand, some silt, trace gravel, trace clay (FILL)		8	SS	5																		
	Loose																						
	Brown Moist		9	SS	8																		
			10	SS	5																		
254.5																							
7.5	SAND, trace to some silt, trace gravel, trace clay, contains organics matter including wood fragments to 9 m depth		11	SS	5																		
	Very loose to loose																						
	Brown to dark brown Moist to wet		12	SS	2																		
			13	SS	3																		
			14	SS	54/0.23																		
251.7																							
10.3	MIGMATITE (BEDROCK)		1	RC	REC 100%																		
	Rock cored between depths of 10.29 m to 13.30 m below ground surface.																						
	See Record of Drillhole 16-1 for details.		2	RC	REC 100%																		
248.7																							
13.3																							

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\TOHWY\_28\_EELS\_CREEK\G02\_DATA\GINT\HWY\_28\_EELS\_CREEK.GPJ GAL-GTA.GDT 19-5-10



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

GTA-MTO 001 S:\CLIENTS\MTOWHY 28 EELS CREEK\02 DATA\GINT\HWY 28 EELS CREEK.GPJ GAL-GTA.GDT 19-5-10

LOCATION: N 4945466.9 ;E 413005.0

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90°      AZIMUTH: ---

DRILL RIG: CME 55 Track Mount

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

[illegible]

## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



# GOLDER

LOGGED: DG

CHECKED: AK

PROJECT		RECORD OF BOREHOLE No 16-2				SHEET 1 OF 1		METRIC								
1413191 (1150)		G.W.P.		LOCATION		N 4945465.5; E 413010.4 MTM NAD 83 ZONE 10 (LAT. 44.641058; LONG. -78.135945)		ORIGINATED BY DG								
DIST Eastern HWY 28		BOREHOLE TYPE		114 mm O.D. HW Casing and Wash Boring		COMPILED BY AK										
DATUM Geodetic		DATE		December 10-11, 2016		CHECKED BY MWK										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
262.0	GROUND SURFACE															
0.0	ASPHALT (100 mm)		1A	SS	60											
261.4	Sand and gravel (FILL) Very dense Grey Moist		1B													
0.6			2	SS	37											3 89 7 1
260.6	Sand, trace to some silt, trace clay, trace gravel (FILL) Dense Brown Moist															
1.5	- Contains 25 mm thick silt pocket Sand and gravel, trace to some silt, trace clay (FILL) Loose to compact Brown Moist to wet - Contains cobbles/rock fragments from 3.0 m to 6.4 m depth		3	SS	13											
			4	SS	13											
			5	SS	16											
			6	SS	9											
			7	SS	25											34 57 8 1
			8	SS	18											
255.8	Silty SAND, contains organic matter Loose Brown to dark brown Wet MIGMATITE (BEDROCK)		9A	SS	36											
6.5			9B													
			1	RC	REC 100%											RQD = 98%
	Rock cored between depths of 6.48 m to 11.76 m below ground surface.  See Record of Drillhole 16-2 for details.		2	RC	REC 100%											RQD = 96%
			3	RC	REC 100%											RQD = 75%
			4	RC	REC 100%											RQD = 94%
250.2	END OF BOREHOLE															
11.8	NOTE:  1. PVC pipe (63 mm diameter) installed and grouted in place from ground surface to 10.8 m depth for future VSP testing.  2. Water level not measured due to influence of using wash boring methods.															

PROJECT: 1413191 (1150)

**RECORD OF DRILLHOLE: 16-2**

SHEET 1 OF 1

LOCATION: N 4945465.5 ; E 413010.4

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Track Mount

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	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.																FEATURES	R0/R1 ZONES	NOTES PIEZOMETER OR STANDPIPE INSTALLATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
						RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA					ROCK STRENGTH INDEX			WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
						TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	R4	R3	R2	R1	W1	W2	W3				W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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7		Continued from Record of Borehole 16-2		255.52																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				

UCS = 99.4 MPa  
 $\gamma = 25.5 \text{ KN/m}^3$ 

## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: DG

CHECKED: AK

GTA-RCK 054 S:\CLIENTS\MT01HWY\_28 EELS CREEK\DATA\GINT01HWY\_28 EELS CREEK\GPJ GAL-MISS.GDT 19-7-17



PROJECT		RECORD OF BOREHOLE No 16-6				SHEET 1 OF 1		METRIC									
1413191 (1150)																	
G.W.P.		LOCATION				N 4945440.7; E 413014.0 MTM NAD 83 ZONE 10 (LAT. 44.604834; LONG. -78.135905)		ORIGINATED BY DG									
DIST Eastern HWY 28		BOREHOLE TYPE				114 mm O.D. HW Casing and Wash Boring		COMPILED BY AK									
DATUM Geodetic		DATE				December 14-15, 2016		CHECKED BY MWK									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
262.4	GROUND SURFACE																
0.0	ASPHALT (150 mm)																
0.2	Sand and gravel, trace to some silt, trace clay (FILL)		1	SS	58												40 50 7 3
261.8	Very dense																
0.6	Grey Moist		2	SS	8												
	Silty sand, some gravel, containing silt pockets, trace clay (FILL)																
	Loose to very loose																
	Brown		3	SS	10												
	Moist to wet																
	- Gravelly between depths of 1.5 m and 3.0 m below ground surface		4	SS	4												
	- Wet below 2.3 m depth (Elev. 260.1 m)																
			5	SS	4												
258.1	END OF BOREHOLE - AUGER AND SPOON REFUSAL Possible Bedrock		6	SS	3												
4.3	NOTE: 1. Sample 3 had no sample recovery when split-spoon retrieved. 2. Split-spoon measured >50 blows/0.05 m of penetration at a depth of 4.3 m (Elev. 258.1 m).																



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

PROJECT		1413191 (1150)		RECORD OF BOREHOLE No 17-4		SHEET 1 OF 1		METRIC								
G.W.P.				LOCATION		N 4945470.1; E 413006.4 MTM NAD 83 ZONE 10 (LAT. 44.641100; LONG. -78.135995)		ORIGINATED BY								
DIST		Eastern HWY 28		BOREHOLE TYPE		203 mm O.D. Hollow Stem Power Augers		COMPILED BY								
DATUM		Geodetic		DATE		June 20, 2017		CHECKED BY								
								MWK								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
262.0	GROUND SURFACE															
0.0	ASPHALT (150 mm)															
0.2	Sand and gravel, some silt (FILL)		1	SS	32											
261.3	Dense Grey Moist															
0.7	Silty sand, trace gravel (FILL)		2	SS	21											
260.6	Compact Brown Moist															
1.5	Gravelly sand, some silt, trace clay, contains cobbles (FILL)		3	SS	43											
	Compact to very dense Brown Moist		4	SS	54											
	- Grinding of augers from depth of 1.5 m to 2.2 m															
			5	SS	17											
258.3	Sand, some gravel, trace to some silt, trace clay, contains cobbles (FILL)		6	SS	9											
3.7	Loose to very dense Brown Moist															
	- Grinding of augers from depth of 3 m to 4 m		7	SS	53											
256.4	Gravelly sand, some silt, contains cobbles (FILL)		8	SS	6											
5.6	Loose Brown Wet															
	- Grinding of augers from depth of 6.5 m to 7.5 m															
254.8	Sand, trace silt, trace clay, some organics at a depth of 7.9 m (FILL)		9	SS	2											
7.2	Very loose Grey-brown to brown Wet															
252.7	END OF BOREHOLE - AUGER AND SPOON REFUSAL		10	SS	50/0*											
9.3	Possible Bedrock															
<p>NOTE:</p> <p>1. Water level measured at a depth of about 6.1 m below ground surface (Elev. 255.9) upon completion of drilling.</p> <p>2. * N-value not representative of soil due to assumed bedrock contact.</p> <p>3. Sample 4 had zero sample recovery in split-spoon.</p> <p>4. Limited sample recovery in sample 7.</p>																

PROJECT		1413191 (1150)		RECORD OF BOREHOLE No 17-5				SHEET 1 OF 1		METRIC							
G.W.P.				LOCATION		N 4945475.9; E 413008.9 MTM NAD 83 ZONE 10 (LAT. 44.640847; LONG. -78.135962)		ORIGINATED BY		BC							
DIST		Eastern HWY 28		BOREHOLE TYPE		203 mm O.D. Hollow Stem Power Augers		COMPILED BY		KN							
DATUM		Geodetic		DATE		June 20, 2017		CHECKED BY		MWK							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
261.9	GROUND SURFACE																
0.0	ASPHALT (100 mm)																
0.1	Sand and gravel, trace to some silt, trace to some clay (FILL)		1	SS	46												34 53 6 7
261.1	Dense Brown Moist		2	SS	50/0.13												
0.8	Sand, some silt, trace gravel, trace clay, contains cobbles (FILL)																
	Very loose to very dense Brown Moist		3	SS	16												
	- Auger grinding from depth of 4.6 m to 5.2 m		4	SS	7												
			5	SS	3												2 81 14 3
			6	SS	5												
			7	SS	50/0.08												
256.7	END OF BOREHOLE - AUGER AND SPOON REFUSAL																
5.2	Possible Bedrock																
NOTES: 1. Borehole caved to a depth of about 4.3 m below ground surface on removal of augers. 2. Borehole dry upon completion of drilling. 3. SPT "N" value measured 50 blows with zero penetration at a depth of 5.2 m (Elev. 256.7 m) below ground surface.																	

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PROJECT		RECORD OF BOREHOLE No 17-7				SHEET 1 OF 1		METRIC									
1413191 (1150)		LOCATION N 4945442.0; E 413008.9 MTM NAD 83 ZONE 10 (LAT. 44.640847; LONG. -78.135969)				ORIGINATED BY BC											
G.W.P.		DIST Eastern HWY 28				BOREHOLE TYPE 203 mm O.D. Hollow Stem Power Augers				COMPILED BY KN							
DATUM Geodetic		DATE June 21, 2017				CHECKED BY MWK											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
262.4	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALT (150 mm)																
0.2	Sand and gravel to gravelly sand, trace to some silt, trace clay, contains cobbles (FILL) Loose to dense Brown Moist		1	SS	32		262										
			2	SS	15												30 58 7 5
			3	SS	16		261										
	- Auger grinding between depths of 0.9 m and 1.5 m		4	SS	8		260										
	- Auger grinding between depths of 2.3 m and 3.7 m		5	SS	6		259										23 69 4 4
	- Split-spoon refusal at 5.1 m depth below ground surface		6	SS	7		258										
			7	SS	14												
257.3	MIGMATITE (BEDROCK)		1	RC	REC 95%		257										RQD = 64%
5.1	Rock cored between depths of 5.10 m to 8.8 m below ground surface.  See Record of Drillhole 17-7 for details.		2	RC	REC 100%		256										RQD = 86%
			3	RC	REC 98%		255										RQD = 98%
							254										
253.6	END OF BOREHOLE																
8.8	NOTES: 1. Borehole dry upon completion of drilling.																

PROJECT: 1413191 (1150)

**RECORD OF DRILLHOLE: 17-7**

SHEET 1 OF 1

LOCATION: N 4945442.0 ; E 413008.9

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Track Mount

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	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.																FEATURES	R0/R1 ZONES	NOTES PIEZOMETER OR STANDPIPE INSTALLATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
						RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA						ROCK STRENGTH INDEX		WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
						TOTAL CORE %	SOLID CORE %			B Angle 0 to 90 N to S	DIP w.r.t CORE AXIS 0 to 90 S to N	TYPE AND SURFACE DESCRIPTION	Jr	Ja	R4 R3 R2 R1	W1 W2 W3 W4 W5 W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
		Continued from Record of Borehole 16-7		257.25																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														

## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: BC

CHECKED: AK

GTA-RCK 054 - S:\CLIENTS\MT01HWY\_28\_EELS\_CREEK\DATA\GINT01HWY\_28\_EELS\_CREEK.GPJ GAL-MISS.GDT 19-7-17



PROJECT		1413191 (1150)		RECORD OF BOREHOLE No 17-8				SHEET 1 OF 1		METRIC							
G.W.P.				LOCATION		N 4945438.9; E 413009.7 MTM NAD 83 ZONE 10 (LAT. 44.640890; LONG. -78.135960)		ORIGINATED BY		BC							
DIST		Eastern HWY 28		BOREHOLE TYPE		203 mm O.D. Hollow Stem Power Augers		COMPILED BY		KN							
DATUM		Geodetic		DATE		June 21, 2017		CHECKED BY		MWK							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
262.4	GROUND SURFACE																
0.0	ASPHALT (150 mm)																
0.1	Sand and gravel, trace to some silt, trace to some clay, contains cobbles (FILL) Loose to very dense Brown Moist  - Auger grinding between depths of 0.8 m and 1.5 m  - Fresh broken rock/gravel fragments observed in sample 3		1	SS	51												39 47 6 8
			2	SS	45												
			3	SS	38												
			4	SS	7												
			5	SS	14												
258.5	END OF BOREHOLE - AUGER AND SPOON REFUSAL Possible Bedrock		6	SS	50/0.08												
3.9	NOTES:  1. Borehole caved to a depth of about 3.6 m below ground surface on removal of augers.  2. Borehole dry upon completion of drilling.  3. *N-value not representative of soil due to bedrock contact.																

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PROJECT		RECORD OF BOREHOLE No 17-9				SHEET 1 OF 1		METRIC										
1413191 (1150)																		
G.W.P.		LOCATION				N 4945435.3; E 413015.5 MTM NAD 83 ZONE 10 (LAT. 44.640786; LONG. -78.135884)		ORIGINATED BY BC										
DIST Eastern HWY 28		BOREHOLE TYPE				203 mm O.D. Hollow Stem Power Augers		COMPILED BY KN										
DATUM Geodetic		DATE				June 20, 2017		CHECKED BY MWK										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
262.5	GROUND SURFACE							20	40	60	80	100						
0.0	ASPHALT (100 mm)																	
	Sand and gravel to sand, some gravel, trace to some silt, trace clay (FILL) Compact to very dense Grey to brown Moist		1	SS	69		262											
			2	SS	16													19 69 7 5
			3	SS	14		261											
260.3	Sandy gravel, some silt (FILL) Compact Brown Wet		4	SS	11		260											
259.4	END OF BOREHOLE - AUGER AND SPOON REFUSAL Possible Bedrock		5	SS	50/0.02													
3.1	NOTES:  1. Borehole caved to a depth of about 1.5 m below ground surface.  2. Borehole dry upon completion of drilling.  3. *N-value not representative of soil due to bedrock contact.																	

PROJECT		RECORD OF BOREHOLE No 17-10				SHEET 1 OF 1		METRIC									
1413191 (1150)																	
G.W.P.		LOCATION				N 4945431.5; E 413012.8 MTM NAD 83 ZONE 10 (LAT. 44.640752; LONG. -78.135921)		ORIGINATED BY BC									
DIST Eastern HWY 28		BOREHOLE TYPE				203 mm O.D. Hollow Stem Power Augers		COMPILED BY KN									
DATUM Geodetic		DATE				June 20, 2017		CHECKED BY MWK									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
262.5	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALT (150 mm)																
0.2	Sand and gravel, trace to some silt, trace to some clay (FILL) Compact to very dense Grey to brown Moist		1	SS	78		262										35 50 8 7
			2	SS	22												
261.1							261										
1.5	Silty SAND and GRAVEL, trace organics Very dense																
260.5	Brown to dark brown Wet		3	SS	50/0.13												
2.0	END OF BOREHOLE - AUGER REFUSAL Possible Bedrock																
NOTE: 1. Borehole dry upon completion of drilling. 2. *N-value not representative of soil due to assumed bedrock contact.																	

PROJECT		RECORD OF BOREHOLE No 18-1				SHEET 1 OF 1		METRIC									
1895756																	
G.W.P.		LOCATION				N 4945424.4; E 413010.4 MTM NAD 83 ZONE 10 (LAT. 44.640688; LONG. -78.135953)		ORIGINATED BY SK									
DIST Eastern HWY 28		BOREHOLE TYPE				203 mm O.D. Hollow Stem Power Augers		COMPILED BY AK									
DATUM Geodetic		DATE				November 14, 2018		CHECKED BY MWK									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
262.6	GROUND SURFACE																
0.0	ASPHALT (200 mm)																
0.2	Silt and sand, some gravel, trace clay (FILL) Loose Brown Moist		1	SS	7												12 55 31 2
260.9			2	SS	50/0.08												
1.8	MIGMATITE (BEDROCK)																
	Rock cored between depths of 1.8 m to 5.7 m below ground surface.  See Record of Drillhole 18-1 for details.		1	RC	REC 69%												RQD = 30%
			2	RC	REC 77%												RQD = 36%
			3	RC	REC 94%												RQD = 100%
256.9	END OF BOREHOLE																
5.7	NOTES:  1. Borehole dry in hollow stem augers upon completion of drilling, prior to rock coring.  2. *N-value not representative of soil due to bedrock contact.																

PROJECT: 1895756

**RECORD OF DRILLHOLE: 18-1**

SHEET 1 OF 1

LOCATION: N 4945424.40 ;E 413010.40

DRILLING DATE: November 14, 2018

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Acker Renegade Track Mount

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY												FEATURES	PIEZOMETER				
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA				WEATH- ERING INDEX	Diametral Point Load Index (MPa)							
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION									Jr	Ja	Jzon	
						80000000 80000000 80000000 80000000	80000000 80000000 80000000 80000000				80000000 80000000 80000000 80000000	80000000 80000000 80000000 80000000	80000000 80000000 80000000 80000000	80000000 80000000 80000000 80000000						80000000 80000000 80000000 80000000			
		Continued from Record of Borehole 18-1		260.85																			
2		Fresh, crystalline, foliated, black and red, fine to medium grained, non-porous, very strong MIGMATITE		1.75														Lost Core					
					1													Broken Core					
3																		Broken Core					
																		Broken Core					
					2													Lost Core					
4																		$\gamma = 28.8 \text{ KN/m}^3$					
																		UCS=149.1 MPa					
																		E = 78.3 GPa					
5					3																		
		END OF DRILLHOLE		256.87																			
6				5.73																			
7																							
8																							
9																							
10																							
11																							

PROJECT		1895756		RECORD OF BOREHOLE No 18-2				SHEET 1 OF 1		METRIC							
G.W.P.				LOCATION				N 4945432.7; E 413008.9 MTM NAD 83 ZONE 10 (LAT. 44.640763; LONG. -78.135971)		ORIGINATED BY SK							
DIST		Eastern HWY 28		BOREHOLE TYPE				203 mm O.D. Hollow Stem Power Augers		COMPILED BY AK							
DATUM		Geodetic		DATE				November 15, 2018		CHECKED BY MWK							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
262.2	GROUND SURFACE																
0.0	ASPHALT (200 mm)																
0.2	Gravelly sand to sand, some gravel, trace to some silt, trace clay (FILL) Compact to very dense Brown Moist to wet - Auger grinding at a depth of 1.1 m		1	SS	16												17 72 8 3
	- Wet below a depth of 2.1 m		2	SS	50/0.10												
	- Auger grinding/refusal at a depth of 2.6 m below ground surface		3	SS	50/0.05												28 56 13 3
259.6	MIGMATITE (BEDROCK)																
2.6	Rock cored between depths of 2.90 m to 6.59 m below ground surface.  See Record of Drillhole 18-2 for details.		1	RC	REC 100%												RQD = 100%
			2	RC	REC 100%												RQD = 60%
			3	RC	REC 100%												RQD = 89%
255.6	END OF BOREHOLE																
6.6	NOTES:  1. Water level in hollow stem augers measured at a depth of about 1.6 m below ground surface (Elev. 260.6 m) prior to rock coring.																

PROJECT: 1895756

## RECORD OF DRILLHOLE: 18-2

SHEET 1 OF 1

LOCATION: N 4945432.70 ;E 413008.90

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Acker Renegade Track Mount  
DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY																				FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
				ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA						WEATH- ERING INDEX						Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
							TOTAL CORE % 000000	SOLID CORE % 000000			DIP W/L CORE AXIS 000000	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jzon	W1	W2	W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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DEPTH SCALE

1 : 50



GOLDER

LOGGED: SK

CHECKED: AK

GTA-RCK 046 S:\CLIENTS\MT\Hwy\_28 EELS CREEK\DATA\GINT\HWY\_28 EELS CREEK.GPJ GAL-MISS.GDT 19-5-10



PROJECT		RECORD OF BOREHOLE No 18-3				SHEET 1 OF 1		METRIC										
G.W.P. _____		LOCATION N 4945437.1; E 413009.6 MTM NAD 83 ZONE 10 (LAT. 44.640803; LONG. -78.135961)				ORIGINATED BY SK												
DIST Eastern HWY 28		BOREHOLE TYPE 102 mm O.D. Solid Stem				COMPILED BY AK												
DATUM Geodetic		DATE November 14, 2018				CHECKED BY MWK												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
262.3	GROUND SURFACE							20	40	60	80	100						
0.0	ASPHALT (200 mm)							20	40	60	80	100						
0.2	- Continuous auger drilling from ground surface-no sampling						262											
							261											
							260											
							259											
258.7																		
3.6	END OF PROBEHOLE - AUGER REFUSAL Possible Bedrock  NOTES:  1. Borehole dry upon completion of drilling.  2. SPT 'N' value measured >50 blows for 0.01 m of penetration on possible bedrock. A small piece of granite was retrieved from tip of split-spoon.		1	SS	50/0.01													

GTA-MTO 001 S:\CLIENTS\MTOWHY\_28\_EELS\_CREEK\DATA\GINT\HWY\_28\_EELS\_CREEK.GPJ GAL-GTA.GDT 19-5-10



GTA-MTO001 S:\CLIENTS\IMTO\HWY 28 EELS CREEK\02 DATA\GINT\HWY 28 EELS CREEK.GPJ GAL-GTA.GDT 19-5-10

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

PROJECT		RECORD OF BOREHOLE No 18-6				SHEET 1 OF 1		METRIC									
G.W.P. _____		LOCATION N 4945475.5; E 413002.8 MTM NAD 83 ZONE 10 (LAT. 44.641149; LONG. -78.136038)				ORIGINATED BY BC											
DIST Eastern HWY 28		BOREHOLE TYPE 203 mm O.D. Hollow Stem Power Augers				COMPILED BY AK											
DATUM Geodetic		DATE November 15, 2018				CHECKED BY MWK											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
261.8	GROUND SURFACE																
0.0	ASPHALT (150 mm)																
0.2	Gravelly sand, trace to some silt (FILL) Compact Brown Moist		1	SS	11												
260.4																	
1.5	Sand, trace to some gravel, trace to some silt, trace clay (FILL) Very loose to compact Brown Moist		2	SS	8												
			3	SS	6												
			4	SS	13												
	- Auger grinding at a depth of 3.7 m																
			5	SS	4												
256.2																	
5.6	SAND, trace silt, trace clay Very loose Brown Wet		6	SS	1*												
	- Possible blowing sand condition in Sample 6																
	- Auger and spoon refusal at a depth of 7.7 m below ground surface		7	SS	50/0*												
254.2	GNEISS (BEDROCK)																
7.7	Rock cored between depths of 7.7 m to 8.6 m below ground surface.		1	RC	REC 100%												
253.3																	
8.6	See Record of Drillhole 18-6 for details. END OF BOREHOLE																
NOTES: 1. * N-value not representative of soil due to bedrock contact. 2. Water level measured at a depth of 5.2 m below ground surface on completion of drilling (Elev. 256.6 m).																	

PROJECT: 1895756

**RECORD OF DRILLHOLE: 18-6**

SHEET 1 OF 1

LOCATION: N 4945475.50 ;E 413002.80

DRILLING DATE: November 15, 2018

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Track Mount

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY															FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA				WEATH- ERING INDEX			Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION				Jr	Ja	Jzon		W1	W2			W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
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8		Continued from Record of Borehole 18-6		254.15																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: BC

CHECKED: AK

GTA-RCK 046 S:\CLIENTS\MT\HWY 28 EELS CREEK\02 DATA\GINT\HWY 28 EELS CREEK.GPJ GAL-MISS.GDT 19-5-10



GTA-MTO 001 S:\CLIENTS\MTOWHWY 28 EELS CREEK\02 DATA\GINT\HWY 28 EELS CREEK.GPJ GAL-GTA.GDT 19-5-10

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

PROJECT: 1895756

## RECORD OF DRILLHOLE: 18-8

SHEET 1 OF 1

LOCATION: N 4945479.4 ;E 413006.6

DRILLING DATE: November 14, 2018

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Track Mount

DRILLING CONTRACTOR: Geo-Environmental Drilling Inc.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	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.																FEATURES	R0/R1 ZONES	NOTES PIEZOMETER OR STANDPIPE INSTALLATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
						RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA				ROCK STRENGTH INDEX			WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
						TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION				Jr	Ja	R4	R3	R2	R1	W1	W2				W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
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## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



GOLDER

LOGGED: BC

CHECKED: AK

GTA-RCK 054 - S:\CLIENTS\MT01HWY\_28\_EELS\_CREEK\DATA\GINT01HWY\_28\_EELS\_CREEK.GPJ GAL-MISS.GDT 19-7-17

PROJECT		RECORD OF BOREHOLE No 18-9				SHEET 1 OF 1		METRIC								
G.W.P. _____		LOCATION N 4945470.6; E 413016.9 MTM NAD 83 ZONE 10 (LAT. 44.641103; LONG. -78.135862)				ORIGINATED BY SK										
DIST Eastern HWY 28		BOREHOLE TYPE 102 mm O.D. Solid Stem Power Augers				COMPILED BY AK										
DATUM Geodetic		DATE November 15, 2018				CHECKED BY MWK										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
260.7	GROUND SURFACE															
0.0	- Continuous auger drilling from ground surface-no sampling  - Auger grinding encountered at a depth of 2 m															
						260										
						259										
						258										
257.8	END OF PROBEHOLE - AUGER AND SPOON REFUSAL Bedrock		1	SS	50/0*											
2.9	NOTES:  1. SPT 'N' value measured >50 blows for zero penetration on possible bedrock.  2. Open borehole dry upon completion of drilling.  3. Topsoil observed at ground surface.															



PROJECT		RECORD OF BOREHOLE No 18-10				SHEET 1 OF 1		METRIC									
1895756																	
G.W.P.		LOCATION				N 4945473.4; E 413017.6 MTM NAD 83 ZONE 10 (LAT. 44.641128; LONG. -78.135852)		ORIGINATED BY SK									
DIST Eastern HWY 28		BOREHOLE TYPE				203 mm O.D. Hollow Stem Power Augers		COMPILED BY AK									
DATUM Geodetic		DATE				November 15, 2018		CHECKED BY MWK									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
260.7	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL (100 mm)																
	Gravelly sand to sand, trace gravel, some silt, trace clay, trace organics, some silty sand pockets (FILL) Loose to very dense Brown Moist - Auger grinding between depths of 1.0 m and 1.2 m		1	SS	50/0.13		260										23 64 11 2
			2	SS	6		259										3 77 17 3
258.1	- Auger grinding below depth of 2.1 m and auger refusal 2.6 m																
2.6	GNEISS (BEDROCK)		1	RC	REC 98%		258										RQD = 58%
	Rock cored between depths of 2.6 m to 6.2 m below ground surface.  See Record of Drillhole 18-10 for details.		2	RC	REC 100%		257										RQD = 90%
			3	RC	REC 93%		256										RQD = 77%
254.5	END OF BOREHOLE						255										
6.2	NOTES:  1. Water level not recorded prior to rock coring.																

GTA-MTO 001 S:\CLIENTS\MTOWHY\_28\_EELS\_CREEK\DATA\GINT\HWY\_28\_EELS\_CREEK.GPJ GAL-GTA.GDT 19-5-10

PROJECT: 1895756

**RECORD OF DRILLHOLE: 18-10**

SHEET 1 OF 1

LOCATION: N 4945473.40 ;E 413017.60

DRILLING DATE: November 15, 2018

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Acker Renegade Track Mount

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA	WEATH- ERING INDEX	Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
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3		Continued from Record of Borehole 18-10		258.11																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		</

DEPTH SCALE

1 : 50



LOGGED: BC

CHECKED: AK

GTA-RCK 046 S:\CLIENTS\MT\Hwy\_28 EELS CREEK\DATA\GINT\HWY\_28 EELS CREEK\GPJ GAL-MISS.GDT 19-5-10

PROJECT		RECORD OF BOREHOLE No 18-11				SHEET 1 OF 1		METRIC								
1895756		LOCATION N 4945478.3; E 413015.9 MTM NAD 83 ZONE 10 (LAT. 44.641173; LONG. -78.135873)				ORIGINATED BY SK										
G.W.P.		DIST Eastern HWY 28				BOREHOLE TYPE 102 mm O.D. Solid Stem Power Augers				COMPILED BY AK						
DATUM Geodetic		DATE November 15, 2018				CHECKED BY MWK										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
261.0	GROUND SURFACE						20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>		
0.0	- Continuous auger drilling from ground surface-no sampling  - Auger grinding between depths of 2.1 m and 2.9 m															
257.8	END OF PROBEHOLE - AUGER AND SPOON REFUSAL Possible Bedrock		1	SS	50/0.0*											
3.2	NOTES:  1. SPT 'N' value measured >50 blows for 0.01 m of penetration on possible bedrock.  2. Water level measured in open borehole at a depth of 2.7 m below ground surface (Elev. 258.3 m) upon completion of drilling.  3. Topsoil observed at ground surface.															

GTA-MTO 001 S:\CLIENTS\MTOWHY\_28\_EELS\_CREEK\DATA\GINT\HWY\_28\_EELS\_CREEK.GPJ GAL-GTA.GDT 19-5-10

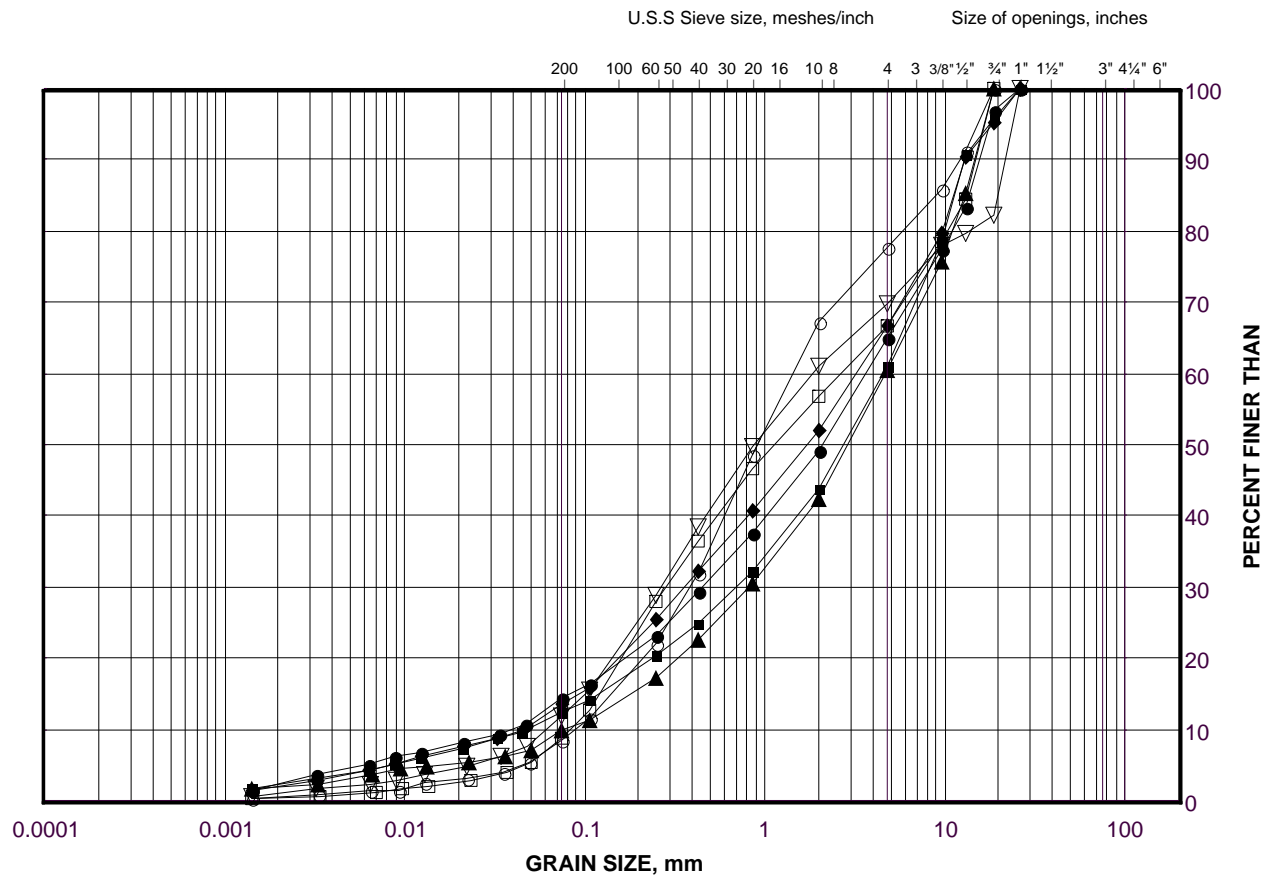
**APPENDIX B**

**Geotechnical Laboratory Test Results**

# GRAIN SIZE DISTRIBUTION

Gravelly Sand to Sand and Gravel (FILL)

FIGURE B1 A



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	17-10	SS1	262.1
■	17-8	SS1	261.9
◆	17-5	SS1	261.5
▲	16-6	SS1	261.9
▽	17-7	SS2	261.3
○	17-7	SS5	259.0
□	16-2	SS7	257.1

Project Number: 1895756,1413191

Checked By: MWK

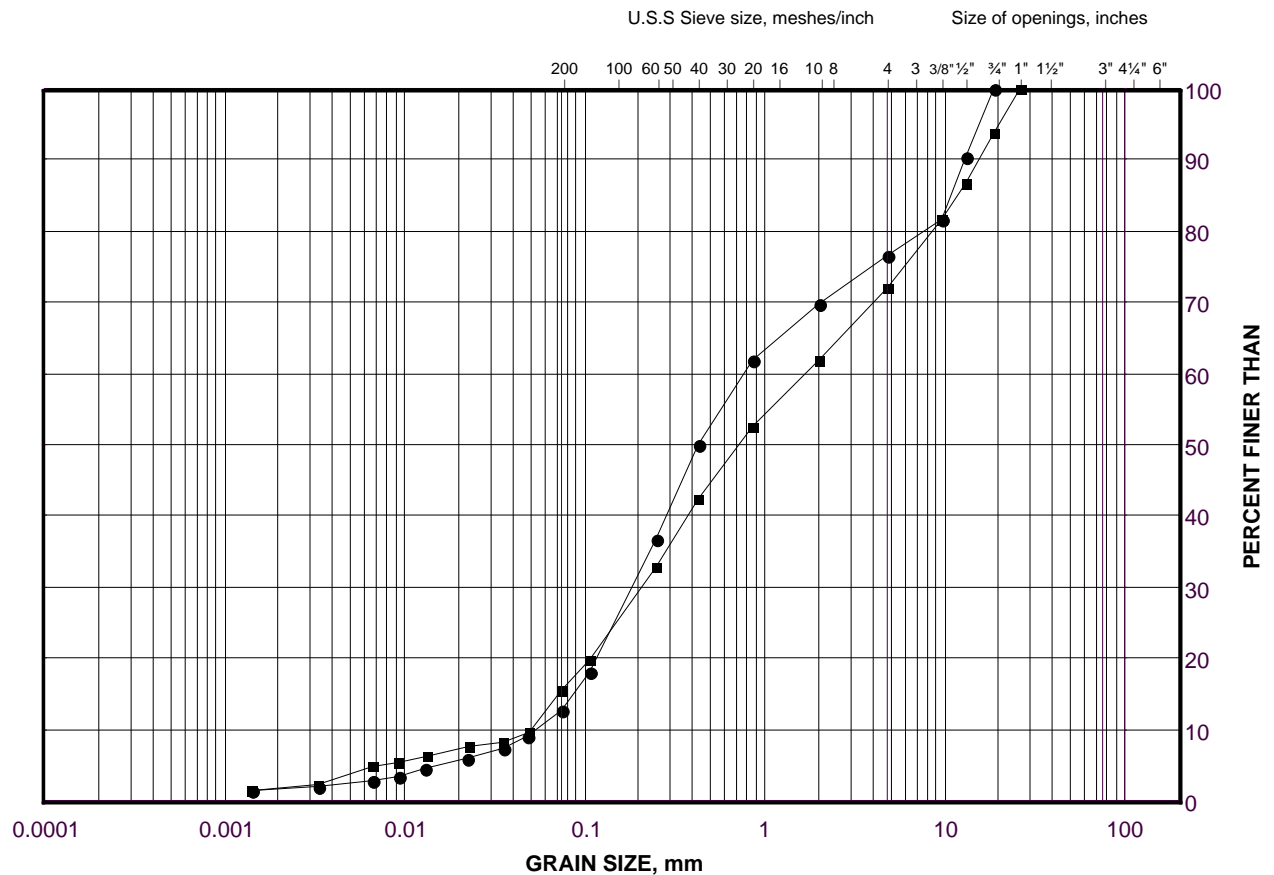
**Golder Associates**

Date: 08-Mar-19

# GRAIN SIZE DISTRIBUTION

Gravelly Sand to Sand and Gravel (FILL)

FIGURE B1 B



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	18-10	SS1	259.8
■	18-2	SS3	259.8

Project Number: 1895756,1413191

Checked By: MWK

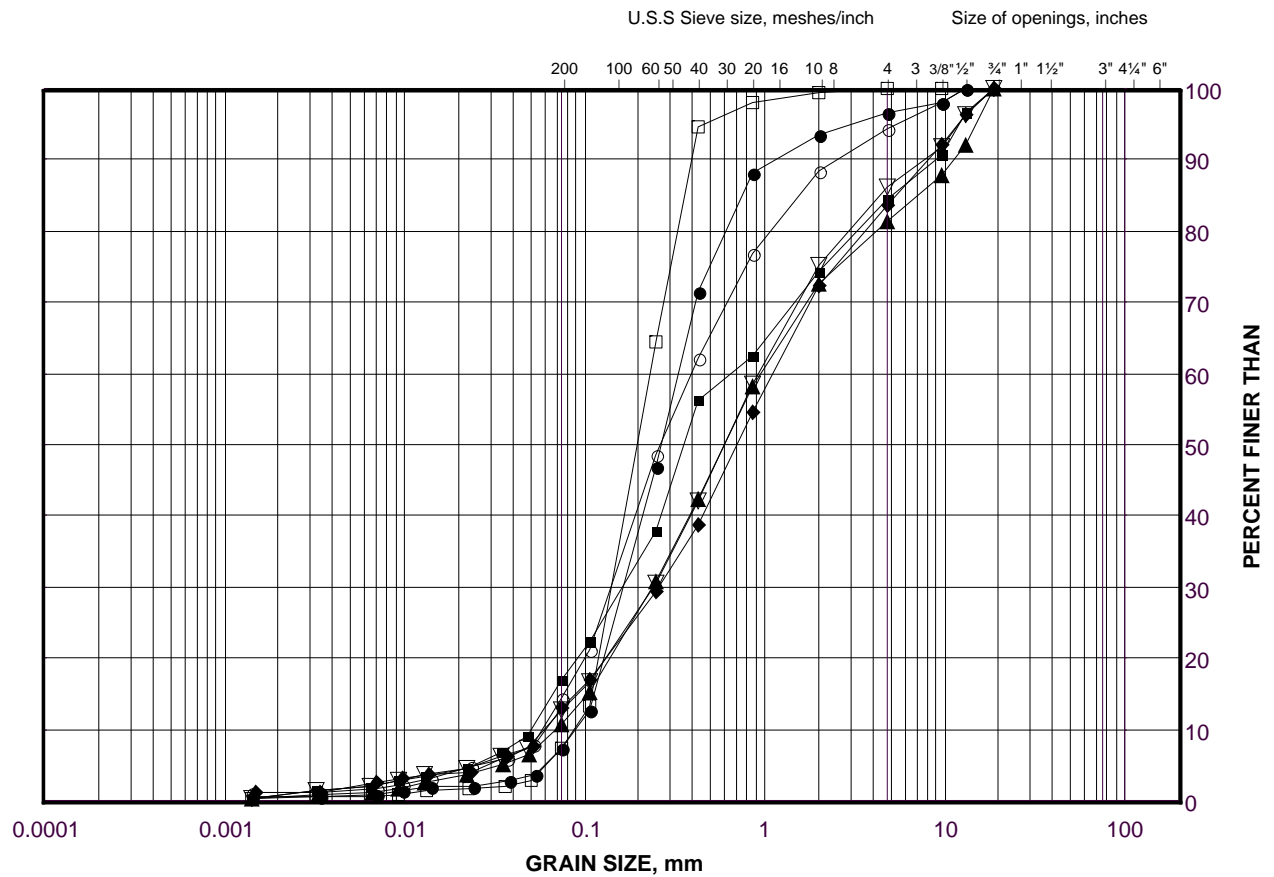
**Golder Associates**

Date: 08-Mar-19

# GRAIN SIZE DISTRIBUTION

Sand (Fill)

FIGURE B2 A



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-2	SS2	261.0
■	17-3	SS3	260.2
◆	16-1	SS3	260.2
▲	17-3	SS5	258.6
▽	17-4	SS6	257.9
○	16-1	SS8	256.4
□	17-4	SS9	254.1

Project Number: 1895756,1413191

Checked By: MWK

**Golder Associates**

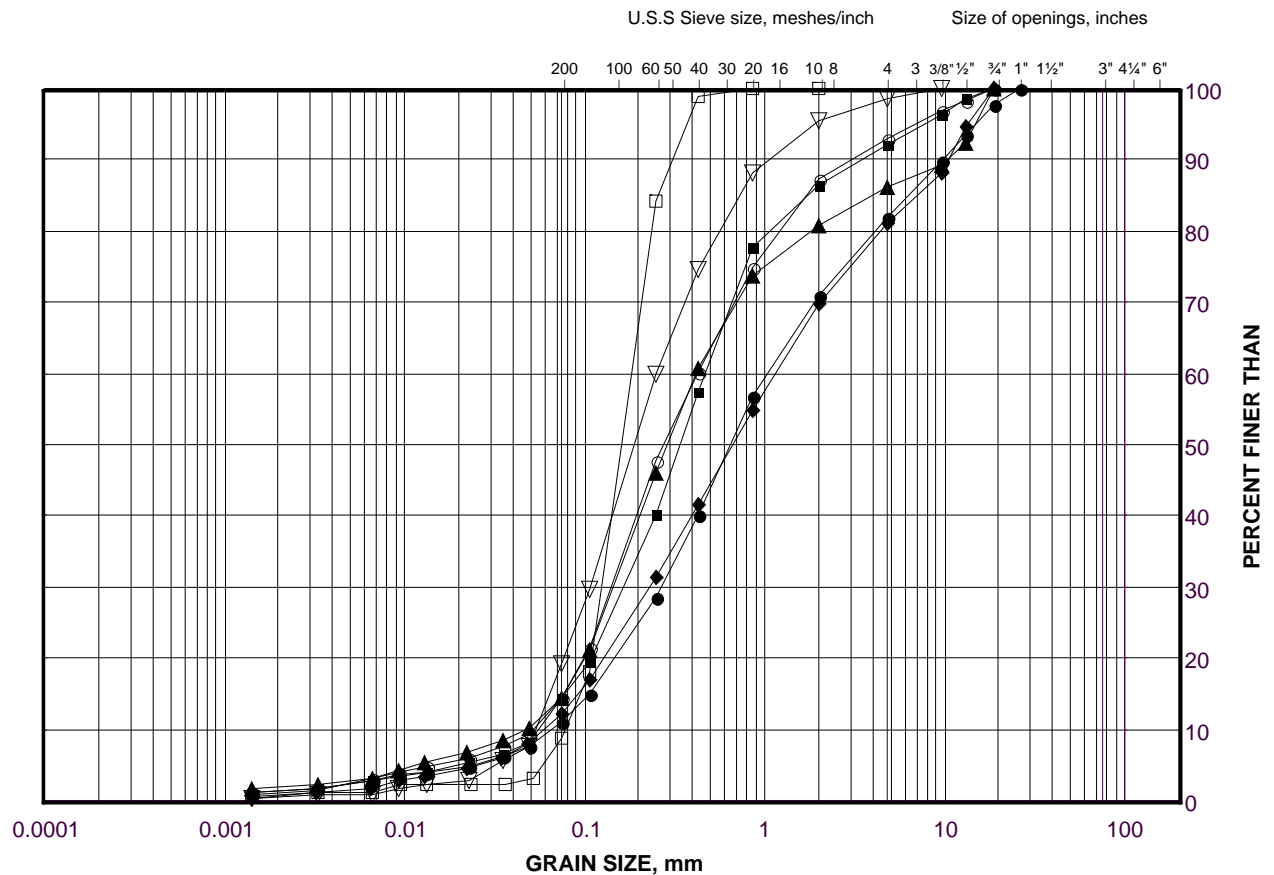
Date: 08-Mar-19



# GRAIN SIZE DISTRIBUTION

Sand (Fill)

FIGURE B2 B



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	18-2	SS1	261.1
■	18-8	SS2	260.0
◆	17-9	SS2	261.4
▲	18-6	SS3	259.2
▽	17-5	SS5	258.5
○	18-6	SS5	256.9
□	18-8	SS5	256.9

Project Number: 1895756,1413191

Checked By: MWK

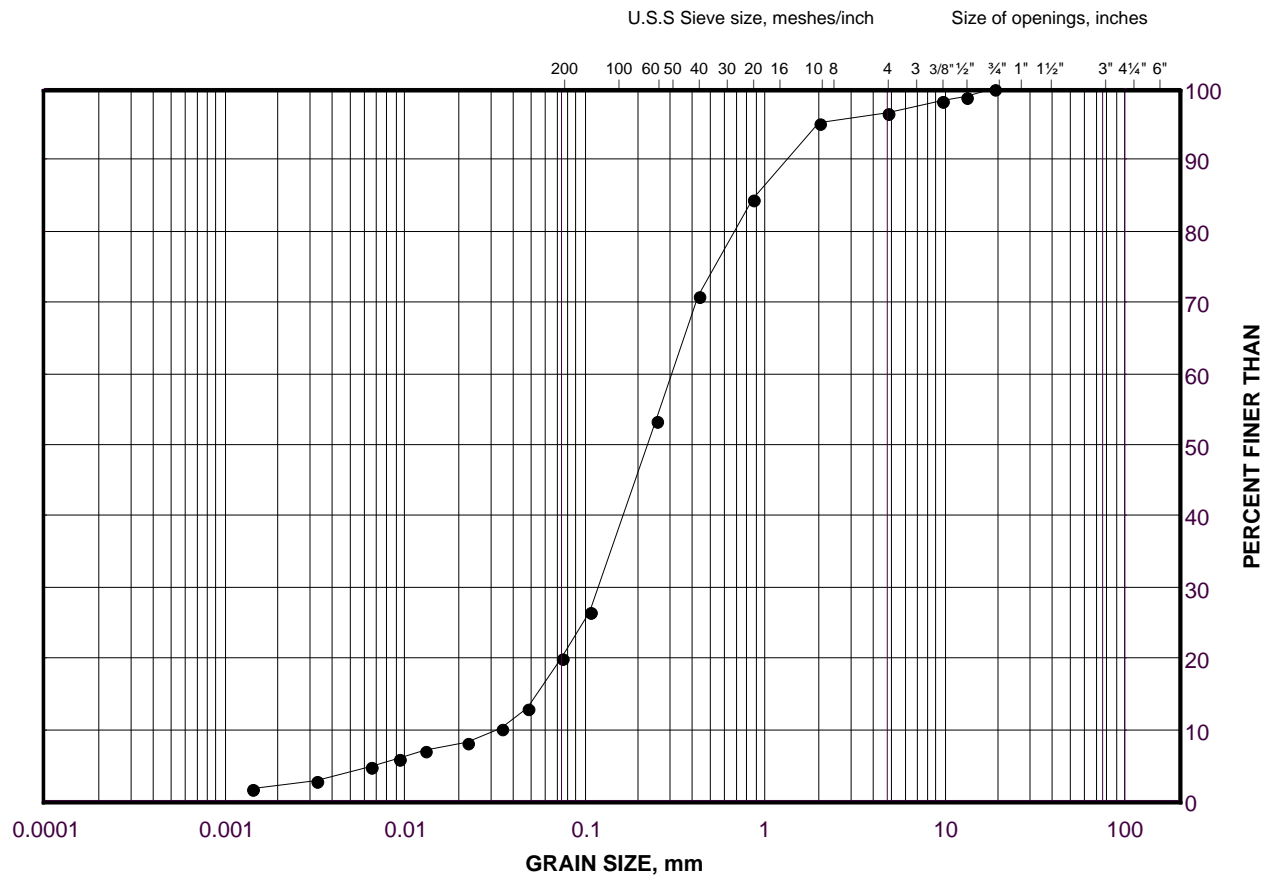
**Golder Associates**

Date: 08-Mar-19

# GRAIN SIZE DISTRIBUTION

Sand (Fill)

FIGURE B2 C



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	18-10	SS2	258.9

Project Number: 1895756,1413191

Checked By: MWK

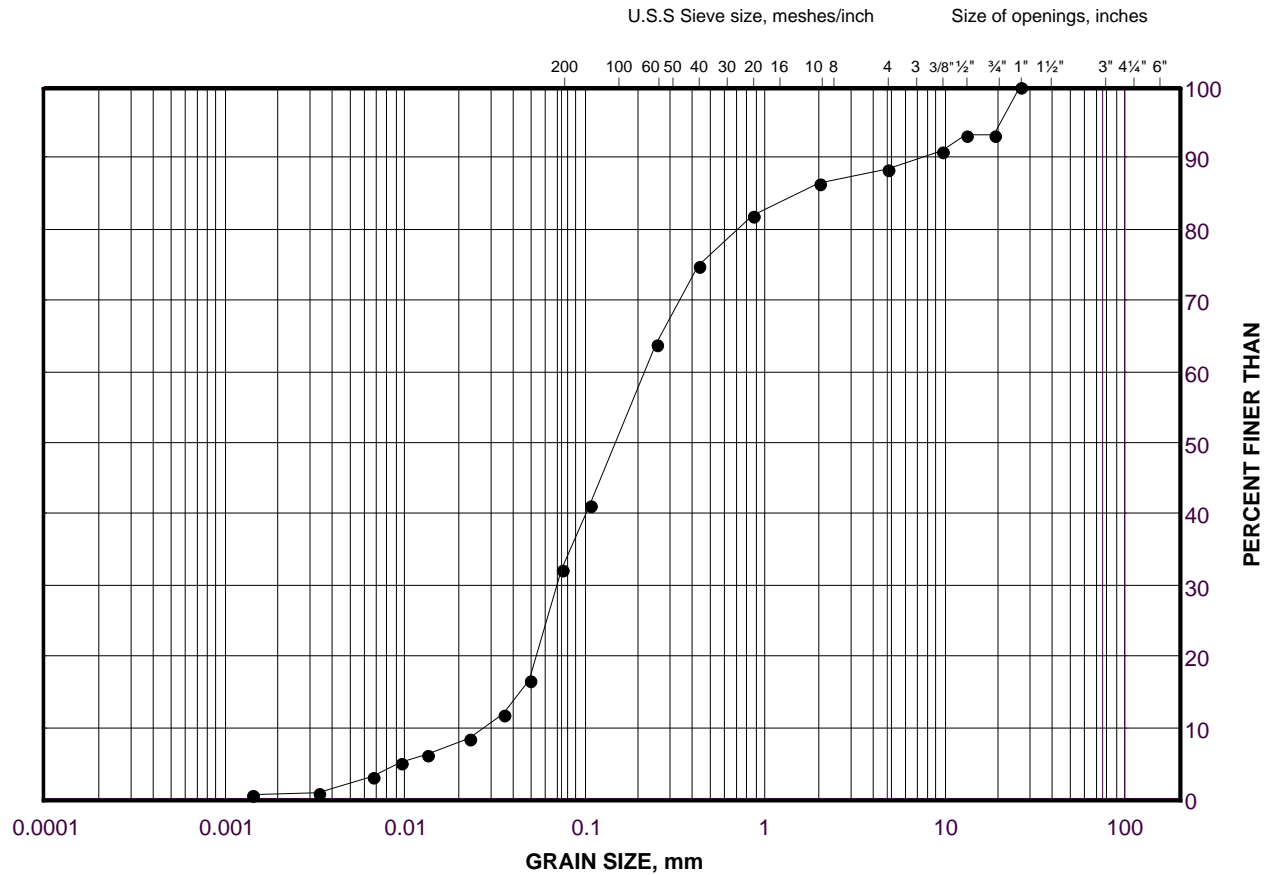
**Golder Associates**

Date: 08-Mar-19

# GRAIN SIZE DISTRIBUTION

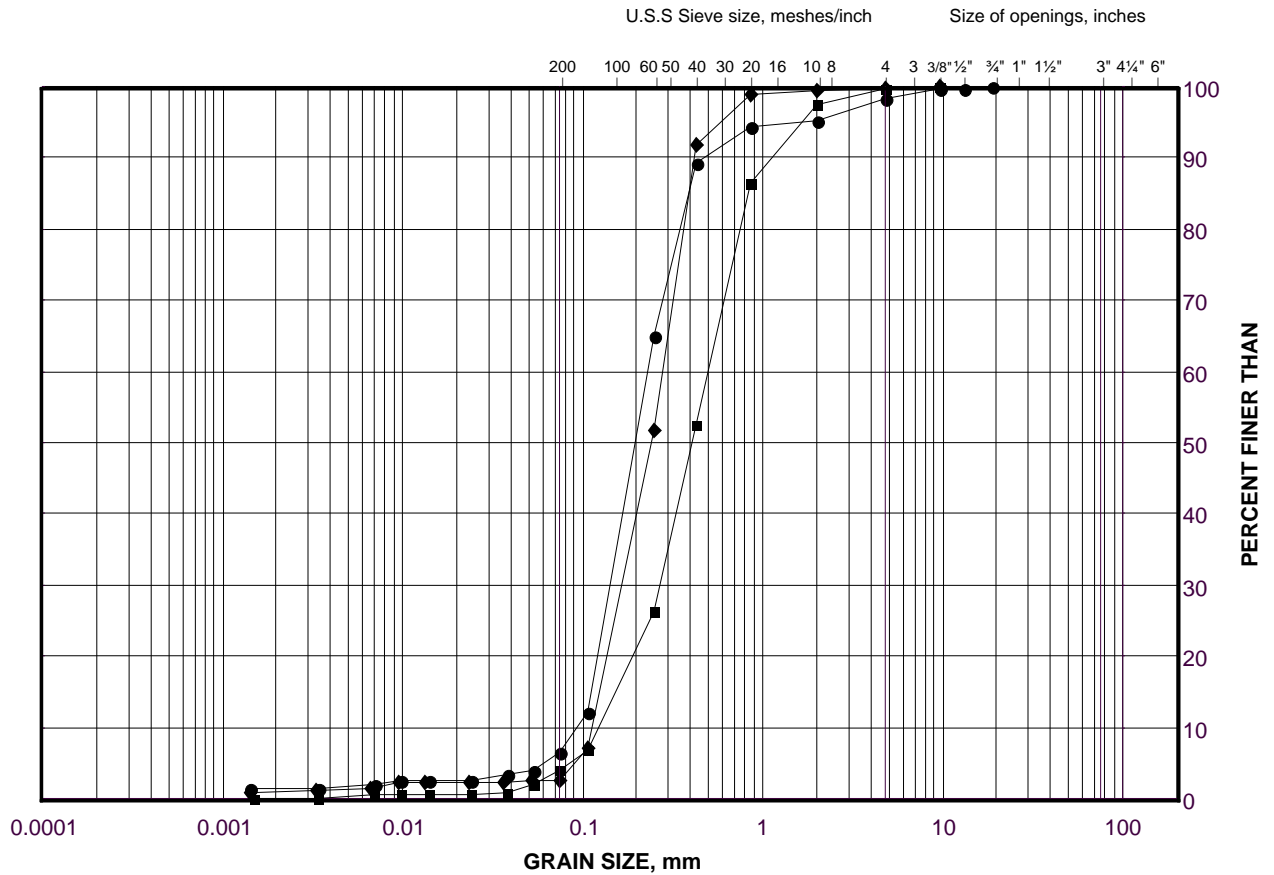
Silt and Sand (Fill)

FIGURE B3



SAND

FIGURE B4



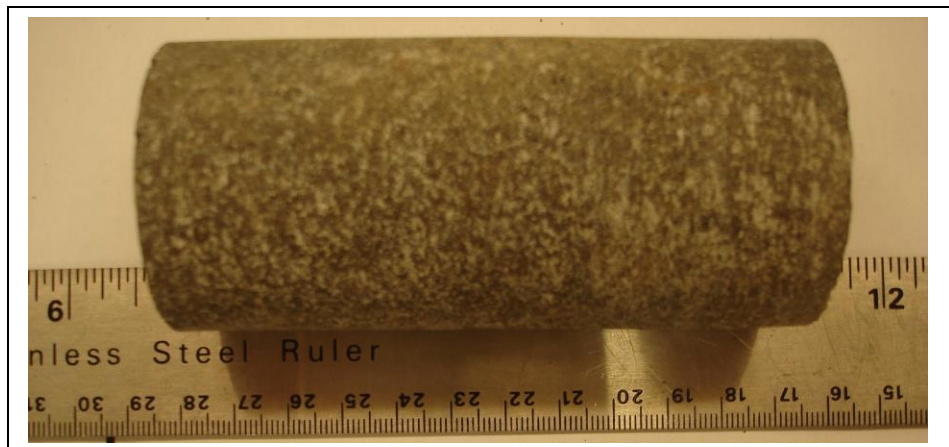
SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	16-1	SS11	254.1
■	16-1	SS13	252.6
◆	18-6	SS6	255.4

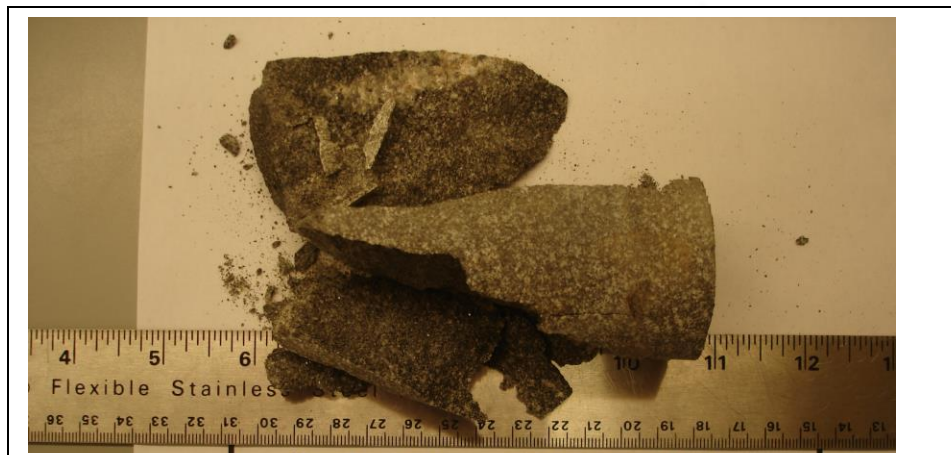
UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS  
ASTM D7012

FIGURE B5



Project No.: 1413191/1150  
Borehole No.: 16-1  
Sample No.: RC 1  
Depth: 10.47-10.75m

BEFORE COMPRESSION



Project No.: 1413191/1150  
Borehole No.: 16-1  
Sample No.: RC 1  
Depth: 10.47-10.75m

AFTER COMPRESSION

Date Aug. 22, 2017  
Project 1413191

**Golder Associates**

Drawn Frank  
Chkd. MM

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

FIGURE B6



Project No.: 1413191/1150  
Borehole No.: 16-2  
Sample No.: RC 1  
Depth: 6.48-6.75m

BEFORE COMPRESSION



Project No.: 1413191/1150  
Borehole No.: 16-2  
Sample No.: RC 1  
Depth: 6.48-6.75m

AFTER COMPRESSION

Date Aug. 22, 2017  
Project 1413191

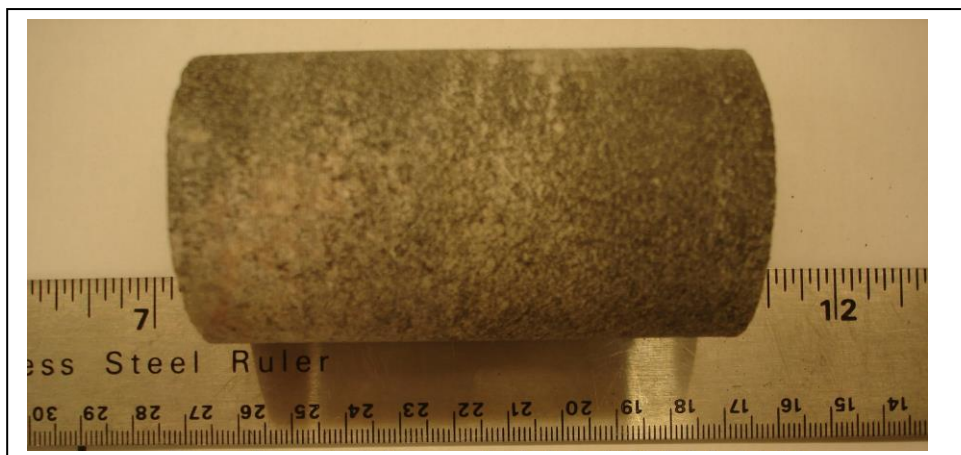
**Golder Associates**

Drawn Frank  
Chkd. MM



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

FIGURE B7



Project No.: 1413191/1150  
Borehole No.: 17-7  
Sample No.: RC 2  
Depth: 5.87-6.0m

BEFORE COMPRESSION



Project No.: 1413191/1150  
Borehole No.: 17-7  
Sample No.: RC 2  
Depth: 5.87-6.0m

AFTER COMPRESSION

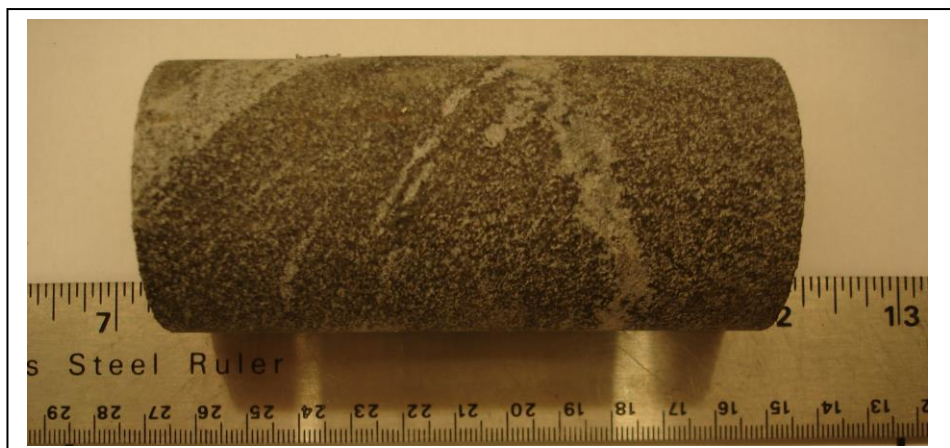
Date Aug. 22, 2017  
Project 1413191

**Golder Associates**

Drawn Frank  
Chkd. MM

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

FIGURE B8



Project No.: 1413191/1150  
Borehole No.: 17-7  
Sample No.: RC 3  
Depth: 8.62-8.8m

BEFORE COMPRESSION



Project No.: 1413191/1150  
Borehole No.: 17-7  
Sample No.: RC 3  
Depth: 8.62-8.8m

AFTER COMPRESSION

Date Aug. 22, 2017  
Project 1413191

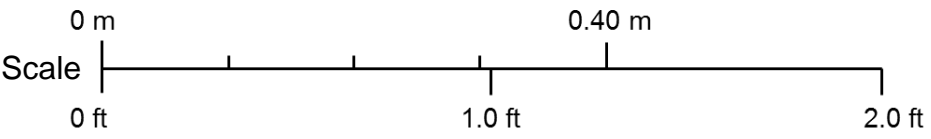
**Golder Associates**


Drawn Frank  
Chkd. MM





**Borehole 16-1:** Bedrock cored between depths of about 10.29 m to 13.30 m



PROJECT		Highway 28 Eel's Creek Bridge Replacement (Site No. 26-117) North Kawartha, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPHS BOREHOLE 16-1			
	PROJECT No.1413191 /1895756		FILE No. ----		
	DESIGN	AK	20181126	SCALE	NTS
	CADD	--		FIGURE B9	
	CHECK	MWK	20171214		
	REVIEW	KJB	20180209		
				VER. 1.	

Rock fragments and cobbles recovered  
between depths of 3.05 m and 6.48 m

Start of Run No. 1 (6.48 m)  
Top of Bedrock

Start of Run No. 2 (7.29 m)



Start of Run No. 3 (8.71 m)

**Borehole 16-2:** Rock fragment and cobbles cored between depths of about 3.05 m and 6.48 m  
and Bedrock cored between depths of about 6.48 m to 8.92 m

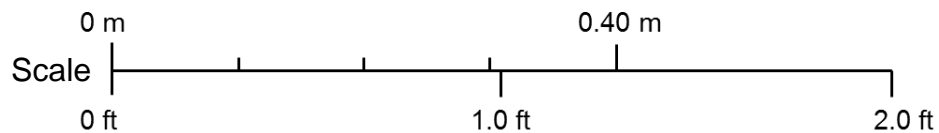
Continuation of Run No. 3 (8.92 m)




Start of Run No. 4 (10.28 m)

End of Borehole (11.76 m)

**Borehole 16-2:** Bedrock cored between depths of about 8.92 m to 11.76 m



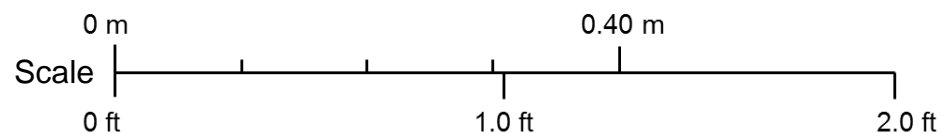
PROJECT		Highway 28 Eel's Creek Bridge Replacement (Site No. 26-117) North Kawartha, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPHS BOREHOLE 16-2			
	PROJECT No.1413191 /1895756			FILE No. ----	
	DESIGN	AK	20181126	SCALE	NTS
	CADD	--		FIGURE B10	
	CHECK	MWK	20171214		
	REVIEW	KJB	20180209		
				VER.	1.




**Borehole 17-7: Bedrock cored between depths of about 5.13 m to 7.26 m**

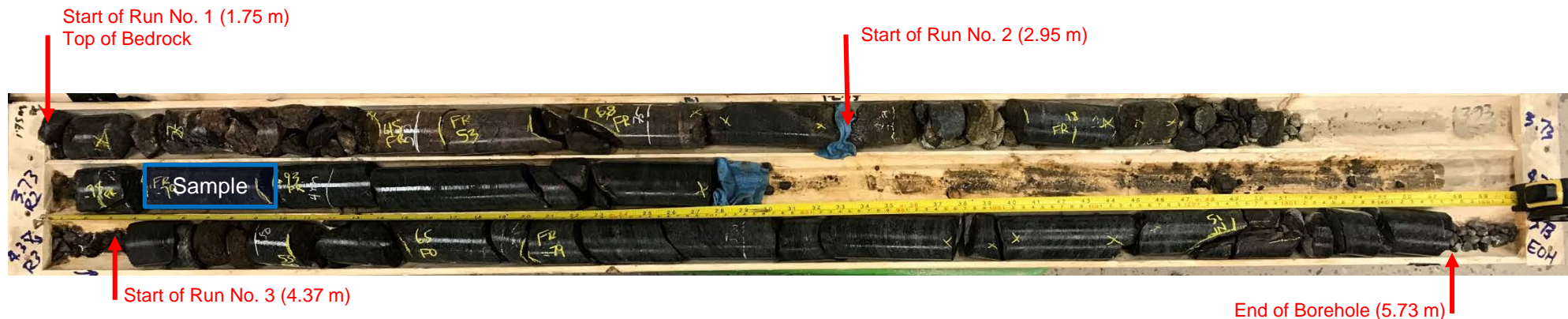


**Borehole 17-7: Bedrock cored between depths of about 7.26 m to 8.79 m**



PROJECT		Highway 28 Eel's Creek Bridge Replacement (Site No. 26-117) North Kawartha, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPHS BOREHOLE 17-7			
	PROJECT No.1413191 /1895756		FILE No. ----		
	DESIGN	AK	20181126	SCALE	NTS
	CADD	--		FIGURE B11	
	CHECK	MWK	20171214		
	REVIEW	KJB	20180209		
				VER. 1.	

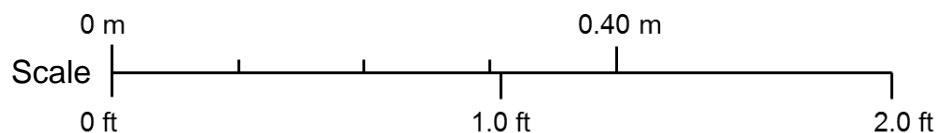





**Borehole 18-1:** Bedrock cored between depths of about 1.75 m to 5.73 m



**Borehole 18-2:** Bedrock cored between depths of about 2.96 m to 6.59 m



PROJECT		Highway 28 Eel's Creek Bridge Replacement (Site No. 26-117) North Kawartha, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPHS BOREHOLES 18-1 AND 18-2			
	PROJECT No.1413191 /1895756			FILE No. ----	
	DESIGN	AK	20181126	SCALE	NTS
	CADD	--		FIGURE B12	
	CHECK	MWK	20171214		
	REVIEW	KJB	20180209	VER. 1.	

Start of Run No. 1 (7.65 m)  
Top of Bedrock



End of Borehole (8.55 m)

**Borehole 18-6:** Bedrock cored between depths of about 7.65 m to 8.55 m

Start of Run No. 1 (5.50 m) – Lost core from 5.33 m to 5.50 m

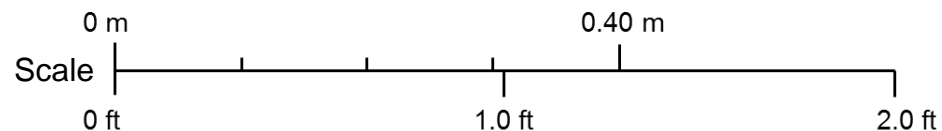
Start of Run No. 2 (5.66 m)




Start of Run No. 3 (7.19 m)

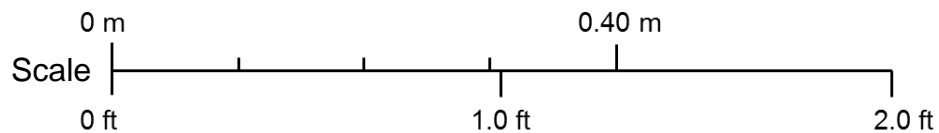
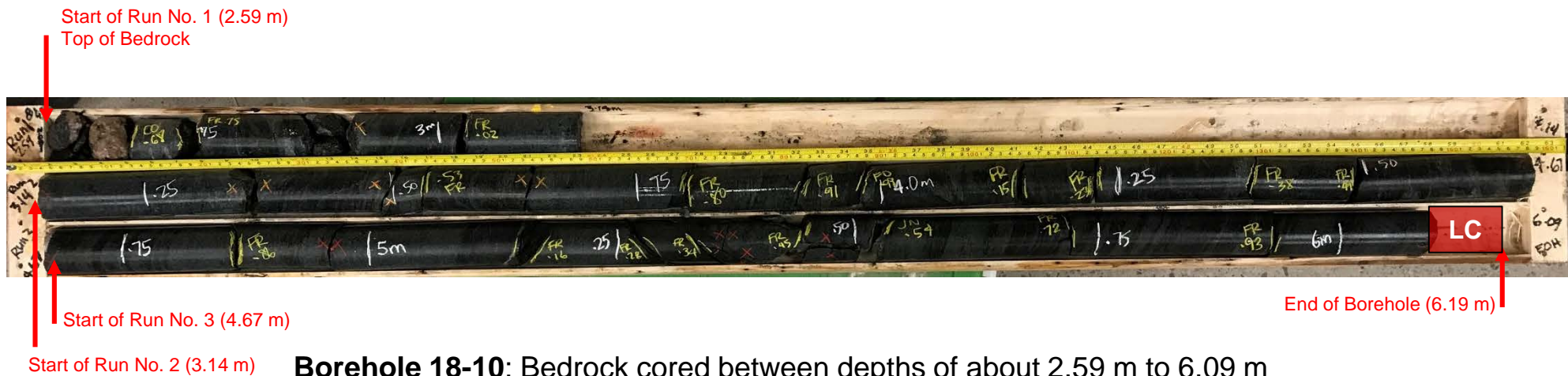
End of Borehole (8.84 m)


**Borehole 18-8:** Bedrock cored between depths of about 5.50 m to 8.84 m



PROJECT	Highway 28 Eel's Creek Bridge Replacement (Site No. 26-117) North Kawartha, Ontario					
	TITLE BEDROCK CORE PHOTOGRAPHS BOREHOLES 18-6 AND 18-8					
	PROJECT No.1413191 /1895756			FILE No. ----		
	DESIGN	AK	20181126	SCALE	NTS	VER. 1.
	CADD	--		FIGURE B13		
	CHECK	MWK	20171214			
	REVIEW	KJB	20180209			





PROJECT		Highway 28 Eel's Creek Bridge Replacement (Site No. 26-117) North Kawartha, Ontario			
TITLE		BEDROCK CORE PHOTOGRAPHS BOREHOLE 18-10			
	PROJECT No.1413191 /1895756			FILE No. ----	
	DESIGN	AK	20181126	SCALE	NTS
	CADD	--		FIGURE B14	
	CHECK	MWK	20171214		
	REVIEW	KJB	20180209		
			VER. 1.		

**APPENDIX C**

**Vertical Seismic Profiling Test Results**

**DATE** July 04, 2017**PROJECT No.** 1413191 - 1150**TO** Kevin Bentley  
Golder Associates**FROM** Stephane Sol, Christopher Phillips**EMAIL** ssol@golder.com, cphillips@golder.com**VERTICAL SEISMIC PROFILING TEST RESULTS  
EELS CREEK BRIDGE AND HIGHWAY 28, NORTH KAWARTHA, ONTARIO**

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This memorandum presents the results of the Vertical Seismic Profiling (VSP) testing carried out at the Eels Creek Bridge located along Highway 28 in North Kawartha, Ontario. VSP testing was carried out on June 20, 2017. Borehole 16-2 was drilled to an approximate depth of 11.8 m below the existing pavement surface and then cased with a PVC pipe grouted in place. Borehole 16-2 was located at the center of the northbound lane. The borehole consisted of approximately 6.6 m of silty sand fill over granite bedrock.

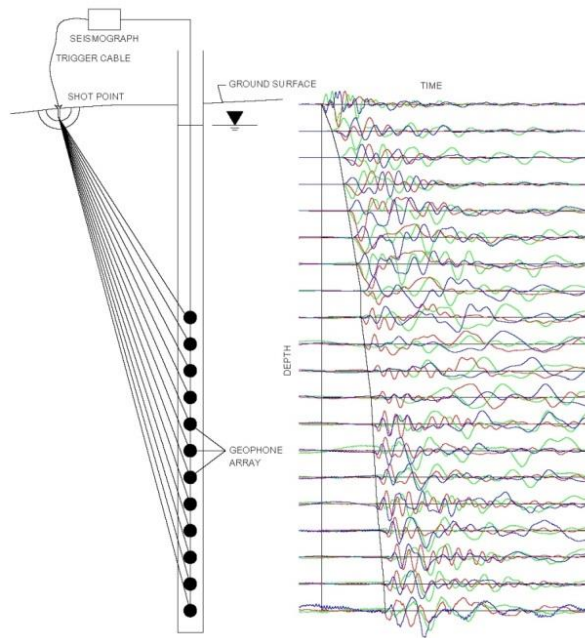
**Methodology**

For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth. The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear-wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole.

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the 2010 National Building Code of Canada.







*Example 1: Layout and resulting time traces from a VSP survey.*

## Fieldwork

The fieldwork was carried out on June 20, 2017, by personnel from the Golder Mississauga office.

Both compression and shear-wave seismic sources were used and both were located 2 m from the borehole. The seismic source for the compression wave test consisted of a 9.9 kilogram sledge hammer vertically impacted on a metal plate. The seismic source for the shear-wave test consisted of a 2.4 metre long, 150 millimetre by 150 millimetre wooden beam, weighted by a vehicle and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. The shear source was coupled to the ground surface by parking a vehicle on top of it. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced at 0.5-metre intervals below the ground surface to a maximum depth of the casing (10.3 m).

The seismic records collected for each source location were stacked a minimum of five times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.341 seconds was collected for each seismic shot.

## Data Processing

Processing of the VSP test results consisted of the following main steps:

- 1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;
- 2) Low Pass Filtering of data to remove spurious high frequency noise;
- 3) First break picking of the compression and shear-wave arrivals; and,
- 4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records are presented on the following four plots and show the first break picks of the compression wave (Figure 1) and shear wave arrivals (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths for Borehole 16-2. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

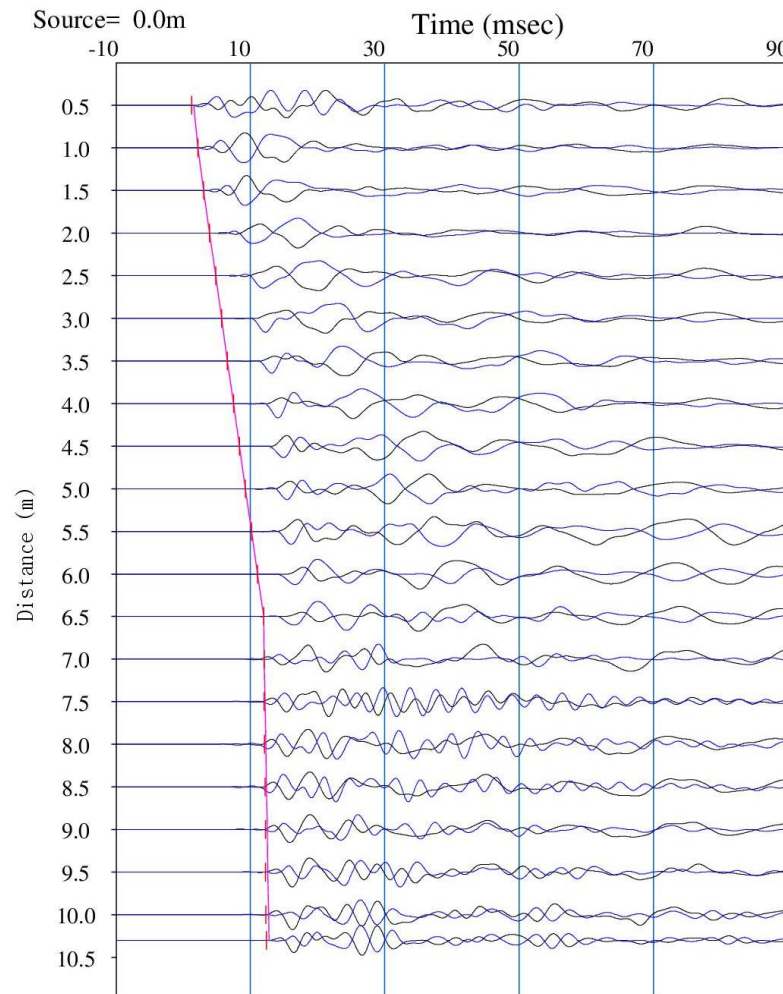


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 16-2.

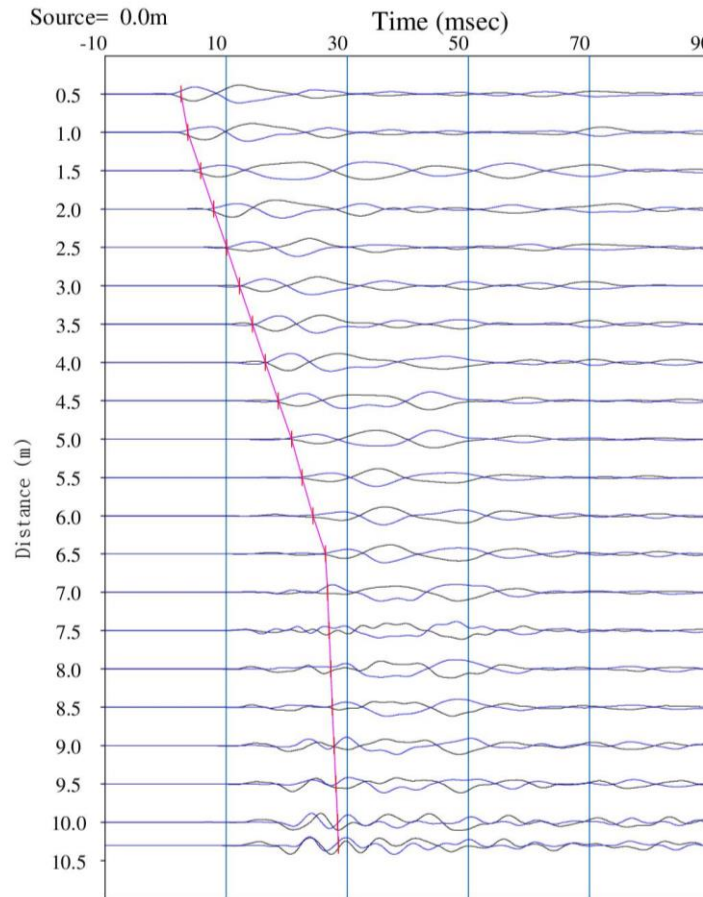


Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 16-2.

## Results

The VSP results are summarized in Table 1. The shear wave and compression wave layer velocities were calculated by best fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. For the overburden a bulk density of 1,650 kg/m<sup>3</sup> was used. For the granite bedrock down to the bottom of the borehole at 10.3 metres, a bulk density of 2,600 kilogram per cubic metre was used.

The average shear wave velocity from ground surface to a depth of 30 metres was measured to be 729 metres per second. The average velocity was calculated assuming that the velocity from 10.3 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 1,500 m/s which is equal to the velocity of the granite bedrock at the bottom of the borehole.

## Limitations

This technical memorandum, which specifically includes all tables, figures and attachments, is based on data and information collected by Golder Associates Ltd. and is based solely on the conditions of the properties at the time of the work, supplemented by historical information and data obtained by Golder Associates Ltd. as described in this memo.

Golder Associates Ltd. has relied in good faith on all information provided and does not accept responsibility for any deficiency, misstatements, or inaccuracies contained in the reports as a result of omissions, misinterpretation, or fraudulent acts of the persons contacted or errors or omissions in the reviewed documentation.

The services performed, as described in this memo, were conducted in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practicing under similar conditions, subject to the time limits and financial and physical constraints applicable to the services.

Any use which a third party makes of this memo, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this memo.

The findings and conclusions of this memo are valid only as of the date of this memo. If new information is discovered in future work, including excavations, borings, or other studies, Golder Associates Ltd. should be requested to re-evaluate the conclusions of this memo, and to provide amendments as required.

## Closure

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

### GOLDER ASSOCIATES LTD.



Stephane Sol, Ph.D., P.Geo  
Senior Geophysicist



Christopher Phillips, M.Sc., P.Geo  
Principal, Senior Geophysicist

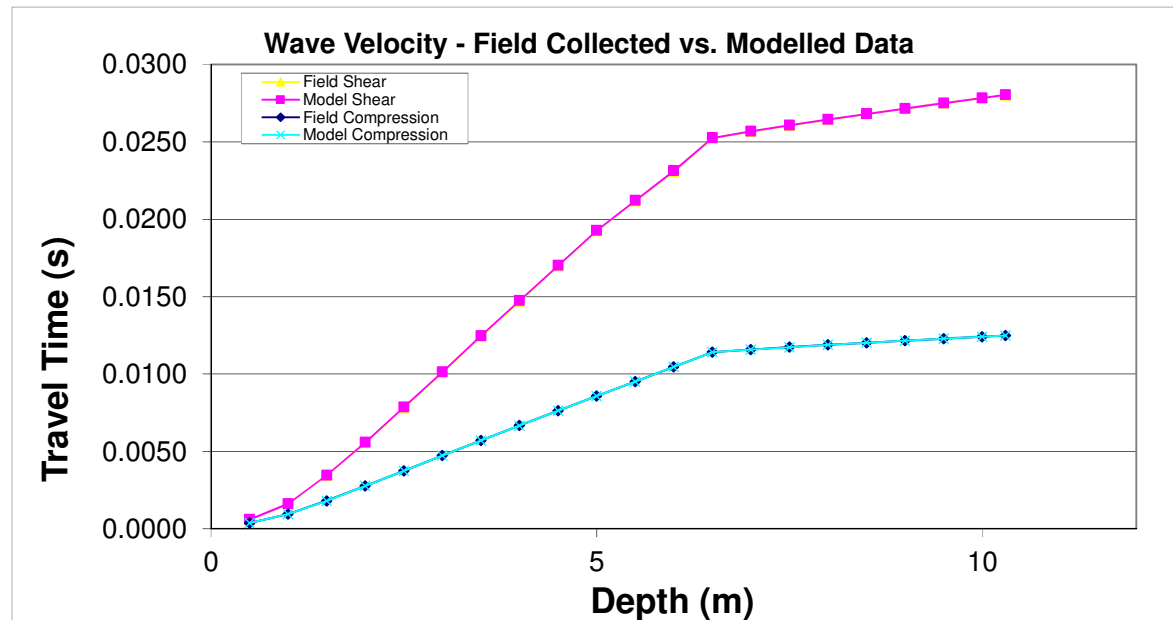
SS/CRP/jl

\\golder.gds\gal\mississauga\active\2014\1111\1413191 mto - foundations eng retainer - east on\15 - hwy 28 eels creek\3 - field work management\geophysics\report final\1413191 tech memo 2017july04 vsp.docx

Attachment: Table 1 – Shear Wave Velocity Profile at BH-16-2

**TABLE 1**  
**SHEAR WAVE VELOCITY PROFILE AT BH-16-2**

Layer Depth (m)		Velocities (m/s)		Estimated Bulk Density (kg/m <sup>3</sup> )	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	0.5	1350	835	1650	0.19	1150	2738	1473
0.5	1.0	850	495	1650	0.24	404	1005	653
1.0	1.5	590	270	1650	0.37	120	329	414
1.5	2.0	520	235	1650	0.37	91	250	325
2.0	2.5	510	220	1650	0.39	80	221	323
2.5	3.0	510	220	1650	0.39	80	221	323
3.0	3.5	510	215	1650	0.39	76	212	327
3.5	4.0	520	220	1650	0.39	80	222	340
4.0	4.5	520	220	1650	0.39	80	222	340
4.5	5.0	525	220	1650	0.39	80	223	348
5.0	5.5	535	260	1650	0.35	112	300	324
5.5	6.0	535	260	1650	0.35	112	300	324
6.0	6.5	525	235	1650	0.37	91	251	333
6.5	7.0	3000	1200	2600	0.40	3744	10519	18408
7.0	7.5	3200	1270	2600	0.41	4194	11797	21033
7.5	8.0	3200	1300	2600	0.40	4394	12313	20765
8.0	8.5	3600	1400	2600	0.41	5096	14380	26901
8.5	9.0	3700	1450	2600	0.41	5467	15408	28305
9.0	9.5	3800	1450	2600	0.41	5467	15468	30255
9.5	10.0	4000	1500	2600	0.42	5850	16593	33800
10.0	10.3	4000	1500	2600	0.42	5850	16593	33800

**Notes**

1. Depth Presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.

**APPENDIX D**

**Rock Laboratory Test Results**

December 14, 2018

Ms. Alysha Kobylinski  
Golder Associates Ltd.  
6925 Century Avenue, Suite #100  
Mississauga, Ontario  
Canada L5N 7K2

Re: UCS+E testing  
(Golder Project No. 1895756)

Dear Ms. Kobylinski:

On November 30, 2018, one (1) HQ-sized and one (1) NQ-sized core samples were received by Geomechanica Inc. via drop-off by Golder Personnel. These samples were identified as being from Golder project 1895756 (Eel's Creek) From these samples, two (2) UCS tests were completed.

Details regarding the steps of specimen preparation and testing along with the test results and photographs of the test specimens before and after testing are presented in the accompanying laboratory report and spreadsheet.

Sincerely,



Bryan Tatone Ph.D., P. Eng.

Geomechanica Inc.  
Tel: (647) 478-9767  
Email: [bryan.tatone@geomechanica.com](mailto:bryan.tatone@geomechanica.com)

# Rock Laboratory Testing Results

**A report submitted to:**

Alysha Kobylinski  
Golder Associates Ltd.  
6925 Century Avenue, Suite #100  
Mississauga, Ontario  
Canada L5N 7K2

**Prepared by:**

Bryan Tatone, PhD, PEng  
Omid Mahabadi, PhD, PEng  
Geomechanica Inc.  
#900-390 Bay St.  
Toronto ON  
M5H 2Y2 Canada  
Tel: +1-647-478-9767  
lab@geomechanica.com

**December 14, 2018**

Project number: 1895756

**Abstract**

This document summarizes the results of rock laboratory testing, including the result of uniaxial compressive strength (UCS) tests. Results including the uniaxial compressive strength (UCS) and Young's modulus along with photographs of test specimens before and after testing are presented herein.

**In this document:**

1 Uniaxial Compressive Strength Tests	1
Appendices	4



# 1 Uniaxial Compressive Strength Tests

## 1.1 Overview

This section summarizes the results of uniaxial compressive strength testing of an NQ- and HQ-sized specimen. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial strain rate of approximately 0.050 mm/min (Figure 1). The specimen preparation and testing procedure included the following:

1. Unwrapping of the core sample and inspecting it for damage.
2. Diamond cutting of core samples to obtain cylindrical specimens with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding of specimens to obtain flat (within  $\pm 0.025$  mm) and parallel end faces (within  $0.25^\circ$ ).
4. Placement of the specimens into the loading frame and axially loading them to rupture while continuously recording the axial force and axial deformation to determine the peak strength (UCS) and tangent Young's modulus.



Figure 1: Forney loading frame setup for uniaxial compressive strength testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-08. The side straightness

criteria, as checked with a feeler gauge, was met for all samples and the minimum length:diameter criteria was met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 with the following exceptions:

- Rather than a spherical seat diameter equal to 1 to 2 times the specimen diameter, the setup used here employed a 25.4 mm diameter high precision ball bearing and seat. Despite the smaller diameter, this seat could move freely to accommodate small angular rotations in any direction, as needed, and therefore did not appreciably influence the results.
- The tests reported herein included the measurement of the UCS and elastic modulus, but not the Poisson's ratio. This represents a hybrid between Methods C and D of ASTM D7012-14.

## 1.2 Results

The results of the tests are summarized in Table 1. Additional specimens measurements and details are provided in the summary spreadsheet that accompanies this report. The corresponding stress-strain curves for the uniaxial compression tests are presented in Figure 2. Young's modulus is the tangent modulus, calculated as the slope of the best fit line through  $\pm 300$  data points on either side of the point representing 50.0% of the peak strength.

Table 1: Summary of Uniaxial Compression test results.

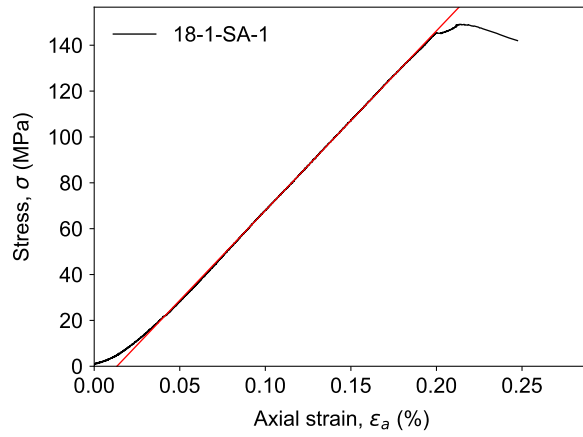
Sample	Depth (m)	Bulk density $\rho$ (g/cm <sup>3</sup> )	UCS (MPa)	Young's modulus $E$ (GPa)	Lithology	Failure description
18-1-SA-1	3.79 - 3.93	2.877	149.1	78.3	Biotite-hornblende gneiss	1
18-8-SA-1	5.76 - 5.99	2.794	93.0	42.8	Biotite-hornblende gneiss	2
Average		2.836	121.1	60.6		
Standard deviation		0.041	28.1	17.8		

<sup>1</sup> Inclined shear failure

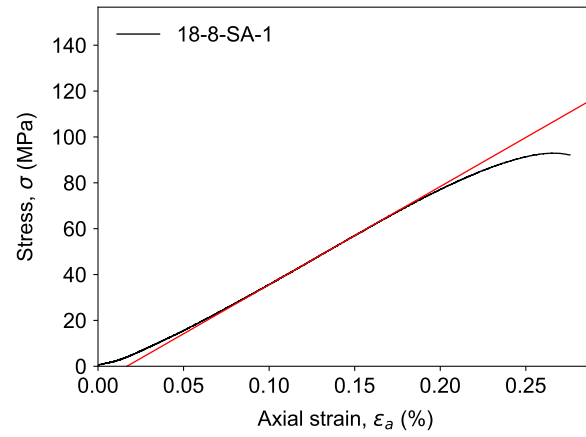
<sup>2</sup> Partial hourglass failure

## 1.3 Specimen photographs

Photographs of the specimens prior to and after testing are presented in the Appendix of this report.



(a) 18-1-SA-1



(b) 18-8-SA-1



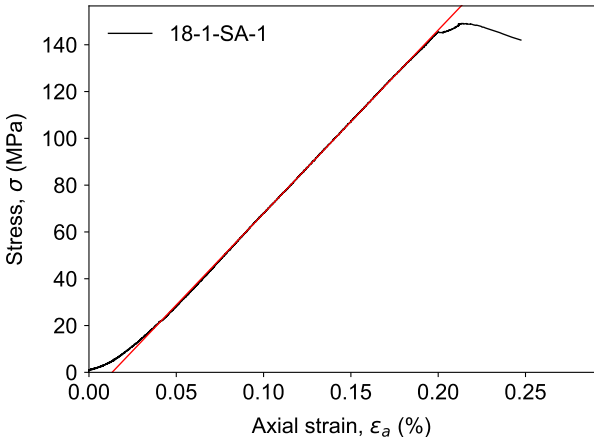
Figure 2: Measured stress-strain curves.

# Appendices



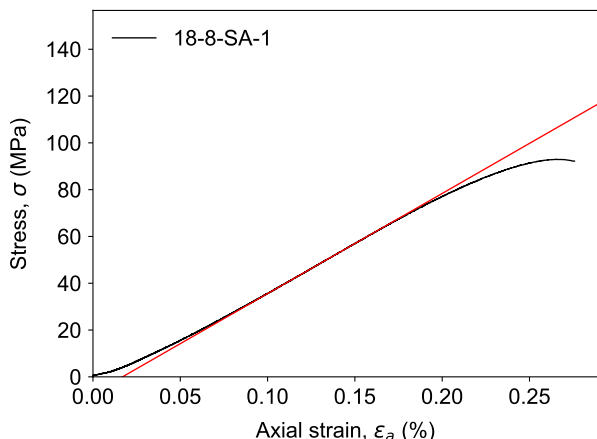
## Specimen sheets

- 18-1-SA-1
- 18-8-SA-1

## Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1895756														
Sample	18-1-SA-1	Depth	3.79 - 3.93														
<div>Specimen parameters</div> <table><tr><td>Diameter (mm)<sup>a</sup></td><td>47.10</td></tr><tr><td>Length (mm)<sup>a</sup></td><td>98.04</td></tr><tr><td>Bulk density ρ (g/cm<sup>3</sup>)</td><td>2.877</td></tr><tr><td>UCS (MPa)</td><td>149.1</td></tr><tr><td>Young's modulus E (GPa)<sup>b</sup></td><td>78.3</td></tr><tr><td>Lithology</td><td>Biotite-hornblende gneiss</td></tr><tr><td>Failure description<sup>c</sup></td><td>1</td></tr></table>		Diameter (mm) <sup>a</sup>	47.10	Length (mm) <sup>a</sup>	98.04	Bulk density ρ (g/cm <sup>3</sup> )	2.877	UCS (MPa)	149.1	Young's modulus E (GPa) <sup>b</sup>	78.3	Lithology	Biotite-hornblende gneiss	Failure description <sup>c</sup>	1	<div>Prior to testing</div> <div></div>	<div>After testing</div> <div></div>
Diameter (mm) <sup>a</sup>	47.10																
Length (mm) <sup>a</sup>	98.04																
Bulk density ρ (g/cm <sup>3</sup> )	2.877																
UCS (MPa)	149.1																
Young's modulus E (GPa) <sup>b</sup>	78.3																
Lithology	Biotite-hornblende gneiss																
Failure description <sup>c</sup>	1																
<div><div><div><sup>a</sup> Additional specimen measurement/details provides in accompanying summary spreadsheet.</div><div><sup>b</sup> Tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 50.0% of the peak strength.</div><div><sup>c</sup> Failure description: <sup>1</sup> Inclined shear failure;</div></div><div></div></div>																	
Remarks:																	
Performed by	BSAT	Date	2018-12-14														

## Uniaxial Compression Test

Client	Golder Associates Ltd.	Project	1895756
Sample	18-8-SA-1	Depth	5.76 - 5.99
<div>Specimen parameters</div>		Prior to testing	After testing
Diameter (mm) <sup>a</sup>	63.15		
Length (mm) <sup>a</sup>	128.82		
Bulk density $\rho$ (g/cm <sup>3</sup> )	2.794		
UCS (MPa)	93.0		
Young's modulus $E$ (GPa) <sup>b</sup>	42.8		
Lithology	Biotite-hornblende gneiss		
Failure description <sup>c</sup>	2		
<div><div><div><sup>a</sup> Additional specimen measurement/details provides in accompanying summary spreadsheet.</div><div><sup>b</sup> Tangent modulus, calculated as the slope of the best fit line through <math>\pm 300</math> data points on either side of the point representing 50.0% of the peak strength.</div><div><sup>c</sup> Failure description: <sup>2</sup> Partial hourglass failure;</div></div><div></div></div>			
Remarks:			
Performed by	BSAT	Date	2018-12-14

**APPENDIX E**

**Analytical Laboratory Test Results**

Your Project #: 1413191-1150

Site Location: EEL'S CREEK

Your C.O.C. #: na

**Attention:Kevin Bentley**

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

**Report Date: 2017/08/23**

Report #: R4671474

Version: 2 - Revision

**CERTIFICATE OF ANALYSIS – REVISED REPORT**

**MAXXAM JOB #: B7H5074**

**Received: 2017/08/15, 14:16**

Sample Matrix: Soil  
# Samples Received: 2

Analyses	Date		Date Analyzed	Laboratory Method	Reference
	Quantity	Extracted			
Chloride (20:1 extract)	2	N/A	2017/08/21	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2017/08/21	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2017/08/18	2017/08/18	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2017/08/15	2017/08/21	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	2	N/A	2017/08/21	CAM SOP-00464	EPA 375.4 m

**Remarks:**

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported: unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

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

Results relate to samples tested.

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

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

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



Your Project #: 1413191-1150

Site Location: EEL'S CREEK

Your C.O.C. #: na

**Attention:Kevin Bentley**

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

**Report Date: 2017/08/23**

Report #: R4671474

Version: 2 - Revision

**CERTIFICATE OF ANALYSIS – REVISED REPORT**

**MAXXAM JOB #: B7H5074**

**Received: 2017/08/15, 14:16**

**Encryption Key**

Please direct all questions regarding this Certificate of Analysis to your Project Manager.

Ema Gitej, Senior Project Manager

Email: EGitej@maxxam.ca

Phone# (905)817-5829

=====

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

### RESULTS OF ANALYSES OF SOIL

<b>Maxxam ID</b>		EYD320	EYD321		
<b>Sampling Date</b>		2016/12/10	2017/06/21		
<b>COC Number</b>		na	na		
	<b>UNITS</b>	<b>BH16-1</b>	<b>BH17-7</b>	<b>RDL</b>	<b>QC Batch</b>
<b>Calculated Parameters</b>					
Resistivity	ohm-cm	3200	1600		5119234
<b>Inorganics</b>					
Soluble (20:1) Chloride (Cl)	ug/g	45	250	20	5127419
Conductivity	umho/cm	312	621	2	5127604
Available (CaCl2) pH	pH	5.85	7.72		5124333
Soluble (20:1) Sulphate (SO4)	ug/g	200	140	20	5127430
RDL = Reportable Detection Limit					
QC Batch = Quality Control Batch					

Maxxam Job #: B7H5074  
Report Date: 2017/08/23

Golder Associates Ltd  
Client Project #: 1413191-1150  
Site Location: EEL'S CREEK  
Sampler Initials: DG

## TEST SUMMARY

**Maxxam ID:** EYD320  
**Sample ID:** BH16-1  
**Matrix:** Soil

**Collected:** 2016/12/10  
**Shipped:**  
**Received:** 2017/08/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5127419	N/A	2017/08/21	Deonarine Ramnarine
Conductivity	AT	5127604	N/A	2017/08/21	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5124333	2017/08/18	2017/08/18	Tahir Anwar
Resistivity of Soil		5119234	2017/08/21	2017/08/21	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5127430	N/A	2017/08/21	Deonarine Ramnarine

**Maxxam ID:** EYD321  
**Sample ID:** BH17-7  
**Matrix:** Soil

**Collected:** 2017/06/21  
**Shipped:**  
**Received:** 2017/08/15

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5127419	N/A	2017/08/21	Deonarine Ramnarine
Conductivity	AT	5127604	N/A	2017/08/21	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5124333	2017/08/18	2017/08/18	Tahir Anwar
Resistivity of Soil		5119234	2017/08/21	2017/08/21	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5127430	N/A	2017/08/21	Deonarine Ramnarine

### GENERAL COMMENTS

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

Package 1	3.3°C
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Revised report (2017/08/23): Sample ID updated as requested.

Sample EYD320 [BH16-1] : Sample submitted and analyzed past the recommended hold time as per client consent.

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

Maxxam Job #: B7H5074  
Report Date: 2017/08/23

## QUALITY ASSURANCE REPORT

Golder Associates Ltd  
Client Project #: 1413191-1150  
Site Location: EEL'S CREEK  
Sampler Initials: DG

QC Batch	Parameter	Date	Matrix Spike		SPIKED BLANK		Method Blank		RPD	
			% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5124333	Available (CaCl <sub>2</sub> ) pH	2017/08/18			99	97 - 103			0.99	N/A
5127419	Soluble (20:1) Chloride (Cl)	2017/08/21	111	70 - 130	103	70 - 130	<20	ug/g	NC	35
5127430	Soluble (20:1) Sulphate (SO <sub>4</sub> )	2017/08/21	124	70 - 130	109	70 - 130	<20	ug/g	NC	35
5127604	Conductivity	2017/08/21			100	90 - 110	<2	umho/cm	1.5	10

N/A = Not Applicable

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

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

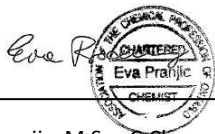
Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

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

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference  $\leq 2 \times \text{RDL}$ ).

### VALIDATION SIGNATURE PAGE

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



Ewa Pranjić, M.Sc., C.Chem, Scientific Specialist

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

**CHAIN OF CUSTODY RECORD 76787**

Page 1 of 1

Invoice Information		Report Information (if differs from invoice)		Project Information (where applicable)		Turnaround Time (TAT) Required	
Company Name: <u>Golder Associates Ltd.</u>		Company Name:		Quotation #:		<input checked="" type="checkbox"/> Regular TAT (5-7 days) Most analyses	
Contact Name: <u>Kevin Bentley</u>		Contact Name:		P.O. #/ AF#: <u>1413191-1150</u>		PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS	
Address: <u>6925 Century Ave.</u>		Address:		Project #: <u>1413191-1150</u>		Rush TAT (Surcharges will be applied)	
Suite #100, Mississauga ON				Site Location: <u>Eel's Creek</u>		<input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3-4 Days	
Phone: <u>(905) 567 4444</u> Fax: <u>(905) 567 6561</u>		Phone: Fax:		Site #:		Date Required:	
Email: <u>Kevin.Bentley@golder.com</u>		Email:		Sampled By: <u>DG and BC</u>		Rush Confirmation #:	
MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY							
Regulation 153		Other Regulations		Analysis Requested		LABORATORY USE ONLY	
<input type="checkbox"/> Table 1 <input type="checkbox"/> Res/Park <input type="checkbox"/> Med/ Fine <input type="checkbox"/> Table 2 <input type="checkbox"/> Ind/Comm <input type="checkbox"/> Coarse <input type="checkbox"/> Table 3 <input type="checkbox"/> Agri/ Other <input type="checkbox"/> Table _____ FOR RSC (PLEASE CIRCLE) Y / N		<input type="checkbox"/> CCME <input type="checkbox"/> Sanitary Sewer Bylaw <input type="checkbox"/> MISA <input type="checkbox"/> Storm Sewer Bylaw <input type="checkbox"/> PWQO <input type="checkbox"/> Region <input type="checkbox"/> Other (Specify) _____ <input type="checkbox"/> REG 558 (MIN. 3 DAY TAT REQUIRED)		REFER TO BACK OF COC <input type="checkbox"/> REG 153 METALS & INORGANICS <input type="checkbox"/> REG 153 ICPMS METALS <input type="checkbox"/> REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B) <input type="checkbox"/> Corrosivity		CUSTODY SEAL Y / N Present Intact COOLING MEDIA PRESENT: <u>Y</u> COMMENTS: <u>2017/06/21</u>	
Include Criteria on Certificate of Analysis: Y / N							
SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM							
SAMPLE IDENTIFICATION		DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	HOLD - DO NOT ANALYZE
1 BH16-1		2016/12/10	Soil	1	1		X
2 BH16-5/6		<del>2017/06/21</del>	Soil	1	1		X
3		2017/06/21	Soil	1	1		X
4							
5							
6							
7							
8							
9							
10							
RELINQUISHED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)		DATE: (YYYY/MM/DD)	TIME: (HH:MM)
<u>Kate New</u>		2017/08/15	2:15 PM	<u>Tamara A. Thompson</u>		2017/08/15	14:16

15-Aug-17 14:16

Ema Gitej



B7H5074

MNI ENV-871

**APPENDIX F**

**Non-Standard Special Provisions**



**MASS CONCRETE – Item No.**

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**Non-Standard Special Provision**

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**Scope of Work**

The scope of work for the above noted tender item includes the mass concrete under the footings for the North Abutment and South Abutment, including associated wingwalls.

**Construction**

Concrete shall be of the same strength as the footing concrete and shall be placed in accordance with OPSS.PROV 904.

**Basis of Payment**

Payment at the contract price for the above noted tender item includes full compensation for all labour, equipment and materials to do the required work.

END OF SECTION

## **DOWELS INTO ROCK - Item No.**

---

Non-Standard Special Provision

---

### **1.0 Scope**

Where required, the Contractor shall provide dowels into the bedrock at the foundations for the Eel's Creek Bridge structure.

### **2.0 Construction**

Concrete to fill-in depressions in the bedrock or to otherwise raise the grade level to the underside of the footing shall be of the same strength as the footing concrete and placed in accordance with OPSS 904. All reinforcing steel supplied shall be in accordance with OPSS.PROV 1440 (dowel bars conforming to CAN/CSA G30.18, Grade 400).

Where dowels are to be set in to 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 grouted and the dowel set in place. Grout shall be of the same strength as the footing concrete or at least 25 MPa at 28 days which is greater.

If the drill 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.

### **3.0 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 provided to the Contract Administrator. The contract Administrator must provide concurrence that the testing results meet the specifications prior to the contractor continuing with any other related works.

#### **3.01 Performance Tests**

Performance testing shall be carried out at a minimum of 10 per cent of the dowels at each foundation element, and at a minimum of two dowels per foundation element, to confirm that the design load of the rock dowels can be achieved. The Contract Administrator will select the rock dowels to be tested.

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.

#### **4.0 Basis of Payment**

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

**END OF SECTION**

## **AMENDMENT TO OPSS 903, APRIL 2016**

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Special Provision No. 109F57

April 2018

### **903.03 DEFINITIONS**

Section 903.03 of OPSS 903 is amended by the deletion of the definitions for Certificate of Conformance and Quality Verification Engineer.

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

#### **903.04.02.04.02.01 Milestone Inspections**

Clause 903.04.02.04.02.01 of OPSS 903 is deleted in its entirety.

#### **903.04.02.06 Review of Splice Test Results and Permission to Proceed**

Clause 903.04.02.06 of OPSS 903 is deleted in its entirety.

### **903.07 CONSTRUCTION**

#### **903.07.02.07.01 General**

Clause 903.07.02.07.01 of OPSS 903 is amended by deleting the first paragraph in its entirety and replacing it with the following:

The driving of piles shall be carefully monitored and controlled and pile driving records produced for each pile under the direction of the Contractor. A pile driving record shall be submitted to the Contract Administrator.

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

##### **903.07.02.07.03.01 General**

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

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be determined using the [\* Designer Fill-In, See Notes to Designer] at end of initial driving as specified in the Contract Documents. If the specified ultimate resistance is not achieved, retap/restrike shall be conducted after initial driving as specified in the Contract Documents.

A Request to Proceed shall be submitted to the Contract Administrator after the design ultimate resistance is achieved.

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

##### **903.07.02.07.03.03 Driving to Bedrock**

Clause 903.07.02.07.03.03 of OPSS 903 is amended by deleting the last sentence in its entirety.

#### **903.07.02.07.04            Wave Equation Analysis**

Clause 903.07.02.07.04 of OPSS 903 is deleted in its entirety and replaced with the following:

When requested by the Contract Administrator, all equipment, material, and personnel shall be supplied to conduct the wave equation analysis procedure.

#### **903.07.03.07                Concrete**

##### **903.07.03.07.01            General**

Clause 903.07.03.07.01 of OPSS 903 is deleted in its entirety and replaced with the following:

A Request to Proceed shall be submitted to the Contract Administrator before the concrete placement.

The reinforcement shall not be displaced or distorted during the construction of the caisson.

The placement of concrete shall not proceed until the Contract Administrator has inspected the caisson hole and issued to the Contractor a Notice to Proceed.

Concrete shall be placed immediately after the Notice to Proceed has been received and shall be placed in the caisson according to OPSS 904 and as specified herein.

Arching of concrete during casing withdrawal shall be prevented.

##### **903.07.03.07.05            Founding Elevation**

Clause 903.07.03.07.05 of OPSS 903 is amended by deleting the last paragraph in its entirety and replacing it with the following:

Complete access to inspect the bearing area of the caisson pile prior to the placement of concrete shall be given to the Contract Administrator.

##### **903.07.06                    Load Test**

Subsection 903.07.06 of OPSS 903 is amended by deleting the first paragraph in its entirety and replacing it with the following:

When a load test is specified in the Contract Documents, the testing shall be according to ASTM D 1143M for piles under vertical static load, ASTM D 3689 for piles under tensile load, and ASTM D 3966 for piles under lateral loads. The Contract Administrator shall witness the pile load test. All records and results of the pile load test shall be submitted to the Contract Administrator.

##### **903.07.08.01.02            Visual Inspection of Welds**

Clause 903.07.08.01.02 of OPSS 903 is deleted in its entirety and replaced with the following:

Complete access to visually inspect the welds shall be given to the Contract Administrator.

A representative sample of not less than 30% of the welds, as determined by the Contract Administrator, shall be visually inspected for conformance to the requirements of CSA W59 and the Contract Documents.

**903.07.08.01.03            Non-Destructive Testing of Welds**

Clause 903.07.08.01.03 of OPSS 903 is deleted in its entirety and replaced with the following:

Radiographic or ultrasonic testing shall be carried out using procedures according to CSA W59.

Ultrasonic or radiographic testing shall be carried out on the entire length of selected splice welds chosen at random by the Contractor's welding inspector assigned to carry out visual inspections.

Selection shall be based on the following criteria:

- a) For pile groups other than at integral abutments, 10% of the splice welds, rounded to the next highest number, but no fewer than two.
- b) For pile groups at integral abutments, 10% of the splice welds, rounded to the next highest number, but no fewer than two of when the welds are below 6 m of the pile cut-off elevation.
- c) For pile groups at integral abutments, all splice welds within 6 m of the pile cut-off elevation.

**903.07.08.03            Certificate of Conformance**

Clause 903.07.08.03 of OPSS 903 is deleted in its entirety.

**903.10                    BASIS FOR PAYMENT**

**903.10.01                Supply Equipment for Installing Driven Piles - Item  
Supply Equipment for Installing Caisson Piles - Item  
Supply Equipment for Installing Displacement Caisson Piles - Item**

Subsection 903.10.01 of OPSS 903 is amended by deleting the second paragraph in its entirety and replacing it with the following:

For payment purposes, 50% of the work under this item shall be paid when the satisfactory performance of the equipment has been demonstrated to the Contract Administrator by the installation of 1% of piles.

Another 40% shall be paid by progress payments proportional to the work completed. The remaining 10% shall be paid on the satisfactory completion of the installation of piles.

**OBSTRUCTIONS - Item No.**

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Non-Standard Special Provision

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The overburden soils at the site contain obstructions such as cobbles and rock fragments (possibly boulder sized) as indicated and inferred from difficulties advancing and grinding of the augers during the geotechnical investigation as shown in the Record of Borehole sheets. The Contractor is alerted to the presence of these obstructions such that the selection of appropriate equipment and procedures required to penetrate / remove cobbles/boulders during excavation and/or installation of temporary protection systems, piles, sheetpiles, drilled casings, etc. can be made and to ensure design tip/excavation levels are achieved.

**Basis of Payment**

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

**END OF SECTION**

## **DEWATERING STRUCTURE EXCAVATIONS - Item No.**

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Special Provision No. FOUN0003

March 8, 2018

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### **Amendment to OPSS 902, November 2010**

OPSS 902, November 2010, Construction Specification for Excavating and Backfilling - Structures is amended as follows:

#### **902.02 REFERENCES**

Section 902.02 of OPSS 902 is amended by the addition of the following:

#### **Ontario Provincial Standard Specifications, Construction**

OPSS 517      Dewatering  
OPSS 805      Temporary Erosion and Sediment Control Measures

#### **902.03 DEFINITIONS**

Section 903.03 of OPSS 902 is amended by the addition of the following:

**Automatic Transfer Switch** means as defined in OPSS 517.

**Cofferdam** means as defined in OPSS 539.

**Cut-Off Wall** means as defined in OPSS 517.

**Design Storm Return Period** means as defined in OPSS 517.

**Dewatering System** means as defined in OPSS 517.

**Groundwater Control System** means as defined in OPSS 517.

**Plug** means as defined in OPSS 517.

**Sediment** means as defined in OPSS 517.

**Sediment Control Measure** means as defined in OPSS 517.

**Temporary Flow Passage System** means as defined in OPSS 517.

**Unwatering** means as defined in OPSS 517.

**Vegetated Discharge Area** means as defined in OPSS 517.

**Waterbody** means as defined in OPSS 517.

**Watercourse** means as defined in OPSS 517.



## **902.04 DESIGN AND SUBMISSION REQUIREMENTS**

### **902.04.01 Design Requirements**

#### **902.04.01.01 Dewatering**

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a [\* Designer Fill-In, See Notes to Designer] year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

### **902.04.02 Submission Requirements**

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

#### **902.04.02.01 Working Drawings**

Working Drawings for the dewatering system shall be according to OPSS 517.

#### **902.04.02.02 Preconstruction Survey**

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, utilities, and structures, within a distance of [300] metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

#### **902.04.02.03 Milestone Inspections**

Clause 902.04.02.02 of OPSS 902 is deleted in its entirety.

## **902.07 CONSTRUCTION**

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

#### **902.07.04                      Dewatering Structure Excavation**

##### **902.07.04.01                      General**

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

##### **902.07.04.02                      Discharge of Water**

The discharge of water shall be according to OPSS 517.

##### **902.07.04.03                      Monitoring**

Monitoring shall be according to OPSS 517.

##### **902.07.04.04                      System Amendments**

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

##### **902.07.04.05                      Removal**

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:

Designer Fill-Ins

- \* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.
- \*\* Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this standard tender item only on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.

**Very Strong Sloping Bedrock - Item No.**

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Non-Standard Special Provision

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**Scope**

The Contractor shall be alerted that the bedrock surface at the Highway 28 Eel's Creek Bridge site is variable and steeply sloping up to angles of 45 degrees. The bedrock is also classified as strong to very strong with uniaxial compressive strengths measured in the order of 50 MPa to 250 MPa. Any excavations / removals / installations related to construction of the foundations and temporary support systems must account for the unique sloping and high strength bedrock conditions.

**Basis of Payment**

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

**END OF SECTION**



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