FOUNDATION INVESTIGATION AND DESIGN REPORT

Skootamatta River Bridge Replacement (Site No. 11X-0076/B0)
Highway 7 East of Madoc, Ontario
MTO GWP 4077-14-00, WP 4091-14-01

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PART A

FOUNDATION INVESTIGATION REPORT
SKOOTAMATTA RIVER BRIDGE REPLACEMENT (SITE NO. 11X-0076/B0)
HIGHWAY 7, EAST OF MADOC, ONTARIO
MTO GWP 4077-14-00, WP 4091-14-01

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Parson Inc. (Parsons) on behalf of the Ministry of Transportation, Ontario (MTO) as part of the Eastern Region 4017-E-0023 Mega 12 Retainer Assignment 17 to provide detail foundation engineering services for the Design Build (DB)-Ready assignment for the proposed replacement of the existing Skootamatta River Bridge (Site No. 11X-0076/B0) along Highway 7, east of Madoc, Ontario. Foundation engineering services for this assignment are required for the following structures:

- Skootamatta River Bridge – permanent bridge replacement on the existing alignment;
- Skootamatta River Bridge – temporary modular bridge (TMB) for the north detour alternative; and
- Skootamatta River Bridge – TMB for the south detour alternative.

The purpose of this investigation is to establish the subsurface soil and bedrock conditions at the proposed Skootamatta River Bridge replacement, associated approach embankments and TMB detour structures and detour embankments, by borehole drilling, rock coring and laboratory testing on selected soil and rock core samples. The information obtained was used to support foundation engineering input for use by Parsons and MTO to assess strategies for the bridge replacement, which will be incorporated into the design-build-ready package.

2.0 SITE DESCRIPTION

The existing Skootamatta River Bridge is located along Highway 7, approximately 13 km east of the Highway 7/ Highway 62 intersection in Madoc, Ontario, as shown on the Key Plan on Drawing 1. It should be noted that the orientation (i.e., north, south, east, west) stated in the text of the report is typically referenced to project north and therefore may differ slightly from magnetic north shown on Drawings 1 to 4. For the purpose of this report, Highway 7 is described as oriented in an east-west direction with the Skootamatta River flowing in a north-south direction at the bridge location.

Highway 7 is a two-lane divided highway, carrying a single lane of traffic in each direction (i.e., westbound and eastbound directions) at the bridge location. The existing single-span arch concrete bridge constructed in 1933 is approximately 10 m wide, has an arch clear span of approximately 23 m and carries Highway 7 over the Skootamatta River. Based on available design drawings from 1933, the existing bridge is supported by spread footings on bedrock. The drawing indicates a design founding level of Elevation 525.7 ft. (approximately Elevation 160.2 m), potentially after blasting a rock knoll out to the required elevation; the drawing further indicates the footings were to be founded at least 0.3 m into “solid rock” with the heel (back edge) of the footings “built up against solid rock”.

It is interpreted that the east and west abutments are supported by spread footings founded at about Elevation 160 m and the existing highway grade at the bridge is at Elevation 165.5 m, resulting in embankments up to about 5 m to 6 m in height. A minor rehabilitation was carried out in 1978 and a structural rehabilitation of the bridge was carried out in 1999. The 1999 structural rehabilitation also included the installation of a new gabion retaining wall at the southeast approach embankment. It is understood that the bridge is to be replaced as it is nearing the end of its lifecycle.

In general, the topography at the site is rolling/valley terrain with relatively dense tree cover on the south side of the bridge beyond the highway right-of-way. There are bedrock outcrops present along the riverbanks on both sides of the bridge and both sides of the highway. The ground surface conditions at the existing bridge location

are shown on Photographs 1 to 4. Existing older bridge abutments are noted on the north side of the bridge and are shown on Photographs 5 and 6. Based on field observations during this investigation, there is no evidence of movement, tilted vegetation, or tension cracks that would suggest instability, erosion, or settlement at the existing bridge foundations and approach embankments.

Skootamatta River is a river within the Lake Ontario drainage basin which flows from north to south, originating from Joeperry Lake in Bon Echo Provincial Park in Addington Highlands. The River joins Moira River in the Municipality of Tweed which discharges into the Bay of Quinte on Lake Ontario. At the structure site, the river is approximately 22 m wide and its base is located at approximately Elev. 160.0 m. Based on the 1999 General Arrangement drawings, the water level in the Skootamatta River was measured at Elev. 159.9 m in January 1999. Golder surveyed the river water level to be at approximately Elevation 160.3 m on September 28, 2020, during the field investigation at this site. It is understood that the 50-year high water level is at Elevation 163.0 m.

A retail development occupies the property at the northwest corner of the bridge. Electrical transmission lines are located within the footprint of the proposed east abutment and cross the River at a skew from the northeast corner of the existing bridge to the southwest corner.

3.0 INVESTIGATION PROCEDURES

The field work for the current foundation investigation was carried out between September 21 and October 21, 2020, during which time a total of 20 boreholes, designated as Boreholes S1 to S18, S20 and S21, were advanced at the site. The boreholes were advanced at the location of the proposed replacement bridge and approach embankments and at the proposed TMB locations and detour alignments on both the north and south sides of the existing bridge, at the locations shown on Drawings 1 to 5. The borehole and drillhole records from this investigation are provided in Appendix A. Lists of abbreviations and symbols are also provided in this appendix to assist in the interpretation of the borehole and drillhole records.

The borehole investigation was carried out using track-mounted CME 45 and truck-mounted CME 55 drill rigs and portable drilling equipment supplied and operated by Marathon Underground Constructors Corporation of Greely, Ontario. The boreholes drilled by the truck-mounted drill rig were advanced through the overburden and rock fill using 210 mm outer diameter and 108 mm inner diameter hollow-stem augers or ‘HQ’ casing with wash boring and coring techniques. The boreholes advanced using portable equipment were advanced through the overburden using HQ-size or NQ-size casing with coring and wash boring techniques. Water was pumped from the Skootamatta River for use in coring and wash boring techniques; filter fabric and silt fences were used for environmental protection of the river during these operations. Traffic protection was provided where necessary in accordance with MTO’s Book 7 Manual of Temporary Conditions.

Where possible, soil samples were obtained at 0.75 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer on the drill rigs or with a manual cathead and standard weight hammer where portable equipment was employed, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)[1]. However, at some borehole locations split-spoon samples were not able to be obtained at regular intervals due to the presence of rock fill. The SPT “N” values as presented on the borehole records and in Section 4 are uncorrected.

[1] ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

All boreholes were advanced to split-spoon refusal and/or auger or casing refusal, and bedrock was confirmed by NQ coring in selected boreholes. The boreholes were advanced to depths ranging from about 0.3 m to 8.4 m below existing ground surface, including coring of bedrock for core lengths of between 1.7 m and 4.6 m in boreholes located at the proposed replacement bridge and TMB foundation elements. Photographs of the recovered bedrock core samples are provided in Appendix B.

The groundwater conditions and water levels in the open boreholes were observed during and immediately following drilling operations provided that no water was added during the drilling operations; at most locations, the open borehole water levels were not obtained due to the addition of water associated with wash boring techniques through the overburden. A standpipe piezometer was installed in Borehole S2 to permit monitoring of the stabilized groundwater level at that borehole location. The standpipe piezometer consists of a 19 mm diameter PVC pipe with a slotted screen sealed at a selected depth within the borehole. The borehole annulus surrounding the piezometer screen was backfilled with sand and the remainder of the borehole was then backfilled with cement-bentonite grout to ground surface. Details of the piezometer installation and water level readings are presented on the borehole records in Appendix A. The remainder of the boreholes located at proposed replacement bridge abutments were backfilled with a bentonite cement grout while boreholes at the approach embankments were backfilled with bentonite upon completion 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 and logged the boreholes and drillholes. The samples were identified in the field, placed in labelled containers and transported to Golder's Mississauga and Whitby geotechnical laboratories where the samples underwent further visual examination and laboratory testing in accordance with MTO and/or ASTM Standards, as applicable. Index and classification testing consisting of water content, Atterberg limits and grain size distributions were carried out on selected soil samples. In addition, unconfined compression (UC) tests (including assessment of Young's modulus and bulk density) were carried out on selected specimens of the bedrock core samples by Geomechanica Inc. The results of the laboratory testing on the soil and rock samples are included in Appendix B.

The as-drilled borehole locations and the ground surface elevations were obtained using a GPS (Trimble Geo 7X), having an accuracy of 0.1 m in the vertical and horizontal directions. The locations given on the borehole/drillhole records and shown on the drawings are positioned relative to MTM NAD 83 (Zone 9) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, geographic (latitude/longitude) coordinates, ground surface elevations and borehole depths are summarized below.

Location	Borehole Type	Borehole Number	MTM NAD83 Northing (m) (Latitude)	MTM NAD83 Easting (m) (Longitude)	Ground Surface Elevation (m)	Borehole Depth (m)
Proposed Replacement Structure						
West Approach	Drill Rig	S5	4934674.5 (44.549079)	238938.9 (-77.328914)	165.2	2.0
	Drill Rig	S6	4934670.5 (44.549044)	238948.8 (-77.328789)	165.2	5.0
West Abutment	Drill Rig	S1	4934688.2 (44.549206)	238970.0 (-77.328525)	165.4	7.7(1)
	Drill Rig	S2	4934680.2 (44.549133)	238969.5 (-77.328530)	165.3	7.8
	Drill Rig	S17	4934681.4 (44.549144)	238971.6 (-77.328504)	165.4	7.7
	Drill Rig	S18	4934686.8 (44.549192)	238966.6 (-77.328568)	165.4	6.3
East Abutment	Drill Rig	S3	4934695.1 (44.549270)	239000.1 (-77.328147)	165.5	8.4(2)
	Portable Tripod	S4	4934705.2 (44.549360)	238991.7 (-77.328254)	160.5	2.9 m
	Drill Rig	S20	4934696.8 (44.549286)	239003.4 (-77.328105)	165.5	7.2
	Drill Rig	S21	4934701.3 (44.549326)	239000.6 (-77.328141)	165.6	6.2
East Approach	Drill Rig	S7	4934713.0 (44.549433)	239019.4 (-77.327907)	165.7	1.6
	Drill Rig	S8	4934710.5 (44.549412)	239035.6 (-77.327701)	165.8	1.3
Temporary Detour and Modular Bridge - North						
West Detour Embankment	Drill Rig	S11	4934662.1 (44.548964)	238898.1 (-77.329427)	165.3	1.6
West Abutment	Drill Rig	S9	4934693.9 (44.549256)	238955.6 (-77.328706)	164.0	4.6
East Abutment	Drill Rig	S10	4934719.1 (44.549487)	239004.9 (-77.328090)	164.7	7.4
East Detour Embankment	Drill Rig	S12	4934738.3 (44.549664)	239053.8 (-77.327477)	165.4	5.5
Temporary Detour and Modular Bridge - South						
West Detour Embankment	Drill Rig	S15	4934638.6 (44.548753)	238896.0 (-77.329449)	164.8	1.6
West Abutment	Drill Rig	S13	4934670.9 (44.549050)	238973.6 (-77.328477)	163.1	2.5
East Abutment	Portable Tripod	S14	4934691.9 (44.549243)	239020.6 (-77.327888)	163.0	3.5
East Detour Embankment	Drill Rig	S16	4934737.1 (44.549657)	239093.9 (-77.326971)	166.1	0.3

Note(s):
1. Concrete encountered at a depth of 3.7 m below ground surface (Elev. 161.7 m) and is likely associated with existing footings.
2. Concrete encountered at a depth of 4.6 m below ground surface (Elev. 160.9 m) and is likely associated with existing footings.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The project area is located within the Georgian Bay Fringe physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putman, 1984)[2]. The Georgian Bay Fringe which is characterized by very shallow soil and bare rock knobs and ridges, extends along the east side of Georgian Bay through the Parry Sound and Muskoka areas, then eastward from Muskoka in patches into the area north of the Kawartha Lakes.

This part of the Georgian Bay Fringe physiographic region was never submerged during periods of glacial recession. As a result, the surficial soils in this area consist of very shallow deposits of sand, silt and clay underlain by crystalline bedrock and numerous bare knobs and ridges of bedrock are present throughout the area. Localized low-lying swampy areas, containing peat and/or organic soils overlying soft/loose native soils, are present in valleys between the bedrock knobs and ridges.

The bedrock in this area consists of mafic to felsic metavolcanics rocks of the Grenville Supergroup and Flinton Group. Some flows, tuffs, breccias, minor iron formation, and minor metasedimentary rocks can be expected with the possible inclusion of reworked pyroclastic units and amphibolite. Outcrops of this formation are commonly found along water courses.

4.2 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions encountered in the boreholes advanced during the current investigation, together with the results of the laboratory tests and in-situ testing carried out, are presented on the borehole/drillhole records and geotechnical laboratory test sheets in Appendices A and B, respectively.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profiles on Drawings 1 to 5 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests and in-situ 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. It should be noted that the interpreted stratigraphy shown on the drawings is a simplification of the subsurface conditions.

In general, the subsurface conditions encountered at the site consist of asphalt, concrete or topsoil, overlying granular fill of various gradations (i.e., sandy silt to gravel), underlain by native deposits of non-cohesive silty sand and sandy silt. Rock fill was also encountered in Boreholes S6, S18 and S21. Where the bedrock is not exposed at the ground surface, the overburden soil is underlain by amphibolite and gneiss bedrock at relatively shallow depths, between 0.3 m and 5.5 m below ground surface. A more detailed description of the subsurface conditions encountered at the site is provided in the following sub-sections.

[2] Chapman, L.J. and Putman, D.F., 1984, *The Physiography of Southern Ontario*, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.)

4.2.1 Asphalt and Concrete

Approximately 100 mm to 178 mm of apshalt was encountered in Boreholes S1 to S3, S5 to S7, S17, S18, S20 and S21. A concrete layer with a thickness of approximately 225 mm and 330 mm was encountered below the asphalt in Boreholes S1 and S3, respectively.

4.2.2 Topsoil

Topsoil was encountered in Boreholes S9 to S12 (north TMB detour) and S13, S14 and S16 (south TMB detour) at ground surface and ranged in thickness from about 50 mm to 700 mm, and typically less than 175 mm.

4.2.3 SILTY SAND (SM) (FILL) to GRAVEL (GP) (FILL)

Non-cohesive fill consisting of silty sand to gravel was encountered in all boreholes except Boreholes S4, S13, S14 and S16. The surface of the fill was encountered below the asphalt/concrete or topsoil or at the ground surface at Elevations ranging from 163.8 m to 165.8 m, and its thickness ranges from 0.3 m to 3.8 m. The fill contained organic inclusions in Boreholes S10, S11, and S12. Borehole S12 contains a 0.3 m thick layer of organic silty sand fill which was encountered at Elevation 162.7 m and which is underlain by a 0.7 m thick layer of silty sand with organic inclusions.

A concrete layer with a thickness of approimately 600 mm was encountered below the fill at Elevation 161.7 m in Borehole S1 which is likely associated with the existing footing.

The SPT “N” values measured within the fill generally range from 1 blow to 48 blows per 0.3 m of penetration, indicating that the fill layer has a very loose to dense compactness condition. One SPT “N” value of 100 blows per 0.07 m of penetration was recorded in Borehole S6, but it is inferred that the value may be affected by the split-spoon penetrating rock fill fragments. In Boreholes S3, S7, S9, S11, S12, S15, S16 and S17, the last split-spoon sample near the bottom of the topsoil, fill or native silty sand was greater than 100 blows per 0.3 m of penetration likely due to the presence of the underlying bedrock/inferred bedrock.

Grain size distribution tests were carried out on thirteen samples of the fill material and the results are shown on Figures B1a and B1b in Appendix B. Atterberg limits testing carried out on three samples of the fill materials returned non-plastic results. The water content measured on samples of the fill range between about 2% and 15% with two samples measuring 29% and 32% likely as a result of the presence of organics. An organic content test completed on a sample from the organic silty sand fill layer encountered in Borehole S12 measured approximately 24%.

4.2.4 Rock (FILL)

Rock fill was encountered and cored in Boreholes S6, S18 and S21. The surface of the rock fill was encountered below the fill at depths between 0.4 m and 3 m (between Elevations 162.4 m and 164.8 m) and its thickness ranges from 0.1 m to 1.2 m.

A 0.6 m thick layer of cobbles and boulders was encountered in Borehole S10 below the fill and above the bedrock at 3.8 m depth (Elevation 160.9 m) as evidenced from the split-spoon recovery of rock fragments and the recovery from the casing.

4.2.5 SILTY SAND (SM)

In Boreholes S8, S12, S14 and S20, a deposit of silty sand was encountered underlying the topsoil or fill. The surface of the silty sand deposit was encountered at depths of between 0.1 m and 3.7 m below ground surface

(between Elevations 161.7 m and 165.1 m) and it ranges in thickness from about 0.5 m to 1.8 m. The silty sand deposit was noted to be gravelly in Borehole S8.

The SPT “N” values measured within the silty sand deposit range from 6 blows to 98 blows per 0.3 m of penetration, indicating a loose to very dense compactness condition. In Borehole S14, the spoon in the silty sand was greater than 100 blows per 0.3 m of penetration likely due to the presence of the underlying bedrock.

Two grain size distribution tests were carried out on selected samples of the silty sand deposit and the results are shown on Figure B2 in Appendix B. Atterberg limits testing carried out on two samples of the silty sand deposit returned non-plastic results. The water content measured on two samples of this deposit are about 18% and 24%.

4.2.6 SILT to Sandy SILT (ML)

A 0.5 m to 0.7 m thick layer of non-cohesive silt to sandy silt were encountered in Boreholes S5 and S13 beneath the fill and topsoil at Elevations 163.0 m and 163.7 m, respectively.

The SPT “N”-values measured within the silt to sandy silt deposit range from 4 blows to 12 blows per 0.13 m of penetration, indicating a loose to compact compactness condition.

Atterberg limits tests were carried out on two samples of the deposit and returned non-plastic results. The natural moisture content measured on one sample of the silt is about 43%.

4.2.7 Bedrock

The bedrock surface was determined by casing refusal, auger and/or split-spoon refusal as well as by bedrock coring in selected boreholes. Bedrock was encountered and core samples were recovered in all boreholes advanced at the proposed replacement bridge and TMB foundation elements. Refusal to casing, auger and/or split-spoon advancement was encountered in Boreholes S5, S7, S8, S11, S12, S15, and S16, while bedrock was exposed at ground surface in Borehole S4.

In Boreholes S2 and S17 weathered/fractured bedrock fragments were recovered in the last split-spoon sample taken in the boreholes prior to rock coring.

The depths to bedrock or refusal below ground surface, and the corresponding bedrock surface elevation or refusal elevation, and the cored depth are summarized below.

Location	Borehole No.	Depth to Bedrock (m)	Bedrock Surface Elevation (m)	Comments
Proposed Replacement Structure				
West Approach	S5	2.0	163.2	Casing refusal
	S6	1.6	163.6	Bedrock cored 3.4 m
West Abutment	S1	4.3(1)	161.1(1)	Bedrock cored 3.5 m
	S2	3.7	161.7	Bedrock cored 4.0 m
	S17	4.1	161.3	Bedrock cored 3.5 m
	S18	3.1	162.2	Bedrock cored 3.1 m
East Abutment	S3	4.0(2)/4.9	161.6(2)/160.7	Bedrock cored 4.4 m
	S4	0.0	160.5	Bedrock cored 2.9 m
	S20	3.7	161.8	Bedrock cored 3.7 m
	S21	2.7	162.9	Bedrock cored 3.5 m
East Approach	S7	1.6	164.1	Casing refusal
	S8	1.3	164.5	Casing refusal
Temporary Detour and Modular Bridge - North				
West Detour Embankment	S11	1.6	163.7	Casing refusal
West Abutment	S9	1.7	162.3	Bedrock cored 2.9 m
East Abutment	S10	4.4	160.3	Bedrock cored 3.0 m
East Detour Embankment	S12	5.5	159.9	Auger and split-spoon refusal
Temporary Detour and Modular Bridge - South				
West Detour Embankment	S15	1.6	163.2	Auger and split-spoon refusal
West Abutment	S13	0.8	162.3	Bedrock cored 1.7 m
East Abutment	S14	0.5	162.6	Bedrock cored 3.0 m
East Detour Embankment	S16	0.3	165.8	Auger and split-spoon refusal

Note(s):
1. An approximately 225 mm thick layer of concrete was encountered at a depth of 3.7 m below ground surface (Elev. 161.7 m), on top of the bedrock surface.
2. An approximately 330 mm thick layer of concrete was encountered at a depth of 4.6 m below ground surface (Elev. 160.9 m) and on top of the bedrock surface. Bedrock or a possible rock fill/rock slab was encountered above the concrete layer at a depth of 4.0 m below ground surface (Elev. 161.5 m), with a thickness of about 0.8 m. The bedrock depth/elevation shown in the table above for S3 could be lower (i.e., at Elev. 160.6 m) if the bedrock layer above the concrete is a bedrock slab and/or rock fill.

Based on a review of the bedrock core samples, the retrieved bedrock core is described as fine- to medium-grained, dark grey, fresh to slightly weathered amphibolite with quartz-calcite veins, and medium-grained, dark grey, fresh to moderately weathered gneiss with brecciated quartz-calcite and calcite-hematite veins.

The details of the bedrock descriptions are presented on the drillhole records in Appendix A and photographs of the recovered bedrock core samples are shown on Figures B3 to B15 in Appendix B. The degree of weathering of the bedrock samples (i.e., fresh to moderately weathered – W1 to W3), and the strength classification of the intact

rock mass based on field identification (i.e., strong to very strong – R3 to R4) are described in accordance with the International Society for Rock Mechanics (ISRM)[3] standard classification system.

The Rock Quality Designation (RQD) measured on the core samples ranges from 43% to 100%, indicating a rock mass of poor to excellent quality, and generally fair to excellent quality, as per Table 3.10 of CFEM (2006)[4]. The Total Core Recovery (TCR) is between 49% and 100% and the Solid Core Recovery (SCR) is between 42% and 100%. These measurements, as encountered in the cored boreholes, are summarized below.

Borehole Number	Total Core Recovery (%)	Solid Core Recovery (%)	Rock Quality Designation (%)	Quality Classification Table 3.10 of CFEM 2006[4]
S6	100	82 – 99	96 – 100	Excellent
S1	100	82 – 96	80 – 92	Good to Excellent
S2	100	90 – 98	97 – 100	Excellent
S17	100	79 – 95	93 – 100	Excellent
S18	100	87 – 97	100	Excellent
S3	100	86 – 100	75 – 100	Good to Excellent
S4	100	93 – 100	94 – 100	Excellent
S20	78 – 100	45 – 83	52 – 81	Fair to Good
S21	70 – 100	54 – 95	61 – 97	Fair to Excellent
S9	100	87 – 99	92 – 100	Excellent
S10	100	75 – 93	67 – 91	Fair to Excellent
S13	100	45 – 96	58 – 100	Fair to Excellent
S14	49 – 100	42 – 100	43 – 100	Poor to Excellent

Unconfined Compression (UC) tests (ASTM D7012)[5] and other tests were carried out on selected core samples of the amphibolite and gneiss bedrock. The results are summarised below and the details are presented on the UCS Rock Laboratory Test Result report from Geomechanica in Appendix B.

[3] International Society for Rock Mechanics Commission on Test Methods, 1985. *Int. J. Rock Mech.Min. Sci. & Geomech*. Abstr. Vol 22, No. 2, pp. 51-60.

[4] Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual (CFEM)*, 4th Edition. BiTech Publications Ltd.

[5] ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Borehole No.	Sample Depth Interval (m)	Sample Elevation Interval (m)	Uniaxial Compressive Strength (UCS) (MPa)	Bulk Density (g/cm^3)	Young's Modulus (E) (GPa)	Rock Type
S1	4.7 – 5.0	160.6 – 160.3	106.9	3.06	68.1	Amphibolite
S2	3.9 – 4.1	161.4 – 161.2	173.4	3.03	100.2	Amphibolite
S17	7.3 – 7.6	158.1 – 157.8	220.1	3.01	101.5	Amphibolite
S18	4.4 – 4.7	161.0 – 160.7	188.7	3.05	109.1	Amphibolite
S3	6.9 – 7.3	158.6 – 158.2	85.9	2.90	61.8	Gneiss
S4	2.5 – 2.7	158.0 – 157.7	124.4	2.93	74.8	Gneiss
S20	6.7 – 7.0	158.8 – 158.5	145.2	2.82	67.2	Gneiss
S21	3.7 – 4.2	161.9 – 161.4	248.6	3.01	106.1	Gneiss
S9	2.2 – 2.5	161.7 – 161.5	166.9	3.07	102.8	Amphibolite
S10	5.4 – 5.7	159.3 – 159.0	186.5	3.03	109.2	Amphibolite
S13	2.1 – 2.3	161.0 – 160.8	103.9	2.93	68.9	Gneiss
S14	0.5 – 0.7	162.5 – 162.3	136.7	2.87	64.2	Amphibolite

A total of eight axial or diametral Point Load Tests (PLTs) were carried out on four samples of the amphibolite and gneiss bedrock, and the results are summarized below.

Borehole No.	Sample Depth Interval (m)	Sample Elevation Interval (m)	Orientation	Axial $I_{s(50mm)}$ (MPa)	Estimated Uniaxial Compressive Strength (UCS) (MPa)	Rock Type
S1	6.4 – 6.6	159.0 – 158.8	Axial	5.9	123.2	Amphibolite
			Diametral	21.1	485.4	
S17	6.3 – 6.5	159.1 – 158.9	Axial	7.2	151.5	Amphibolite
			Diametral	15.6	358.6	
S3	6.2 – 6.4	159.3 – 159.1	Axial	4.1	85.5	Gneiss
			Diametral	10.4	239.6	
S4	1.8 – 2.1	158.7 – 158.5	Axial	6.3	133.1	Gneiss
			Diametral	13.4	309.1	

The estimated uniaxial compressive strength (UCS) values for each sample tested for point load strength are based on a relationship between Is_{50} and UCS which is given by a correlation factor (C) in accordance with ASTM D573108[6], which may vary depending on the size of the core sample and the strength of the rock. For this

[6] ASTM D573108 Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification

site, the UCS values are based on an estimated average correlation factor (C) of 19 to 23 calculated from the average Is_{50}.

Based on the laboratory UCS and point load tests, in accordance with Table 3.5 in CFEM (2006)[4], the amphibolite and gneiss bedrock is generally classified as strong (R4, 50 MPa < UCS < 100 MPa) to very strong (R5, 100 MPa < UCS < 250 MPa) with the one lower value attributed to pre-existing structures (i.e. joints) in the core sample.

4.2.8 Groundwater Conditions

A standpipe piezometer was installed in Borehole S2 and the groundwater level was measured on two occasions as presented in the table below. The measured water level on November 11, 2020 was approximately 0.3 m above the surface of the bedrock in Borehole S2. It should be noted that the water levels in the remaining boreholes were affected by water introduced into the boreholes during wash boring for HQ casing advancement and/or during NQ coring operations.

Borehole Number	Stratum Well Sealed Into	Groundwater Depth (m)	Groundwater Elevation (m)	Date of Piezometer Reading
S2	Amphibolite Bedrock	4.8	160.5	October 13, 2020
		3.4	161.9	November 11, 2020

The above-noted groundwater levels appear to be slightly above the adjacent river level. The Skootamatta River water level, as surveyed by Golder on September 28, 2020, was at approximately Elevation 160.3 m. It is understood that the 50-year high water level in Skootamatta River is at Elevation 163.0 m.

It should be noted that the groundwater levels in the area are subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Mo'oud Nasr, P.Eng., a geotechnical engineer of Golder, and was reviewed by Ms. Sarah Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Ms. Lisa Coyne, P.Eng., a Principal and MTO Designated Foundations Contact for Golder, completed a technical and quality review of this report.

Signature Page

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PART B

FOUNDATION DESIGN REPORT
SKOOTAMATTA RIVER BRIDGE REPLACEMENT (SITE NO. 11X-0076/B0)
HIGHWAY 7, EAST OF MADOC, ONTARIO
MTO GWP 4077-14-00, WP 4091-14-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering recommendations for the proposed replacement of the Skootamatta River Bridge (Site No. 11X-0076/B0) and associated approach embankments as well as the temporary detours and temporary modular bridges (TMBs). The Foundation Investigation and Design Report and these interpretations and recommendations are intended solely for the use of MTO's design team and shall not be used or relied upon for any other purpose or by any other parties.

The recommendations provided herein are based on an interpretation of the factual data obtained from the boreholes and drillholes advanced during the subsurface investigation. The interpretation and recommendations are intended to provide MTO's design team with sufficient information to confirm the selected design and construction alternatives and to design the bridge foundations and approach embankments to a DB-ready procurement stage.

This report was prepared in the context of a design-build ready contract. Therefore, general recommendations related to construction are also provided within this report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design and execution of the design-build project. Those requiring information on aspects of construction must make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods and scheduling.

6.1 General

The existing Skootamatta River Bridge is located along Highway 7, approximately 13 km east of the Highway 7/ Highway 62 intersection in Madoc, Ontario. The existing bridge consists of an approximately 23 m long by 10 m wide single-span arch concrete bridge, which was constructed in 1933.

Based on available design drawings, it is interpreted that the east and west abutments are supported by spread footings founded on bedrock at about Elevation 160 m. The highway grade at the bridge is at Elevation 165.5 m resulting in embankments up to about 5 m to 6 m in height. Abandoned rail bridge abutments, comprised of stone, are located approximately 4 m north of the existing bridge on both the east and west side of the Skootamatta River; they are fenced off and surrounded by trees and other minor vegetation. Based on field observations during this investigation, there is no evidence of movement, tilted vegetation, or tension cracks on the existing Highway 7 bridge and approach embankments that would suggest instability, erosion, or settlement at the existing bridge foundations and approach embankments. There is limited information regarding the abandoned rail bridge abutments, and Golder and Parsons take no responsibility for their condition nor their potential for reuse.

We understand that the proposed replacement structure is to consist of a two-lane, single-span structure constructed along the same alignment as the existing bridge. The replacement bridge will be approximately 48.0 m long by 13.5 m wide (overall), with the proposed west and east abutments located at about Station 15+153 and 15+185, respectively, which are slightly behind the existing abutments. The presence of the existing abandoned rail bridge abutments to the north must be considered in excavations for the replacement structure and associated staging/detours.

To facilitate the replacement of the existing bridge, traffic is being staged onto a detour alignment with the use of Temporary Modular Bridges (TMBs) over the Skootamatta River. Two options are being considered for the TMB as follows: (1) single-lane or two-lane bridges located about 13 m north of the existing bridge (centreline to centreline), and (2) single-lane or two-lane bridge located about 13 m south of the existing bridge (centreline to centreline). It is understood that a north side detour alignment is the preferred option.

6.2 Foundation Options

Based on the proposed structure configuration and the subsurface conditions encountered at this site, both shallow and deep foundation options have been considered for support of the new abutments. A summary comparison of the feasibility, advantages and disadvantages, risks/ consequences, and relative costs associated with each option is provided below, and a comparison of the viable options is presented in Table 1 following the text of this report.

- **Spread / Strip Footings:** Shallow foundations comprised of spread or strip footings, supported on bedrock, are considered feasible for support of the new abutments, and represent the preferred option from a foundations perspective. This option is expected to require temporary protection systems and groundwater control.
- **Driven Steel H-piles or Pipe Piles:** Driven H-piles or pipe piles founded on bedrock have been considered for support of the new abutments. However, due to the presence of rock fill and the shallow depth to bedrock, there is a high risk associated with the piles not being able to penetrate to sufficient depths to develop the required axial and lateral geotechnical resistances prior to reaching effective refusal, and for the piles being damaged or deflected from their alignment tolerance. Further, it may be difficult to achieve pile toe fixity given the shallow depth and strong to very strong nature of the bedrock at this site. Therefore, driven piles are not considered a viable option for the new abutments.
- **Drilled Shafts (Caissons):** Drilled shafts (caissons) founded on or socketed into the bedrock may be considered for the support of the new abutments. The rig and tooling could be equipped to address penetration through rock fill/obstructions and socketing into the strong to very strong bedrock, and temporary liners could be used to support the sidewalls during advancement through water-bearing overburden soils. However, due to the small working area it may be challenging to set up the full-size caisson drilling equipment. Therefore, caissons are likely not a viable option for the new abutments; however, consideration could be given to smaller diameter “drilled steel casings” as presented below.
- **Drilled Steel Casings**: Drilled steel casings, which are typically on the order of 450 mm to 750 mm in diameter, could also be considered as a deep foundation option for support of the replacement bridge structure or detour structure. This foundation option would have similar advantages to steel H-piles or drilled shafts in terms of minimizing excavation depth, protection system requirements and groundwater control requirements. Drilled steel casings also handle obstructions better than driven steel piles or larger diameter caissons. If required for pile toe fixity, bedrock sockets for drilled steel casings are typically more cost effective to construct than for larger diameter caissons and would not require a separate operation to form the rock socket as would be the case for driven steel H-piles.

Based on the subsurface conditions at this site, shallow foundations comprised of strip/spread footings founded on the slightly weathered to fresh, strong to very strong amphibolite and gneiss bedrock are considered to be the most practical and most economical foundation option for the proposed replacement structure and for a temporary modular bridge, if adopted. Given the shallow depth to bedrock, driven piles and drilled shafts are not considered to be practical at this site and therefore are not discussed further in this report; however, smaller diameter drilled steel casings are a feasible alternative, and discussion and recommendations for this option are included in this report.

Based on the proposed footing and ground surface elevations, it is anticipated that excavation depths up to about 6 m will be required through existing fill, including rock fill, native soils and into the bedrock for the construction of the footings. In addition, temporary protection systems will potentially be required between the proposed footings and existing bridge footings which are in close proximity. The recommendations for temporary protection systems are discussed further in Section 6.8.2.

6.3 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code CAN/CSA S6-19* (*CHBDC, 2019*) and its Commentary, the proposed bridge and its foundation system are expected to carry medium to high traffic volumes and its performance will have potential impacts on other transportation corridors; hence, the structure is 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, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding", in accordance with Section 6.5 of the *CHBDC* (2019). Accordingly, the appropriate corresponding Ultimate Limit State (ULS) and Serviceability Limit State (SLS) consequence factor Ψ, and geotechnical resistance factors, Φ_{gu} and Φ_{gs}, from Tables 6.1 and 6.2 of the *CHBDC* have been used for design.

6.4 Seismic Design

6.4.1 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the borehole investigation. In the absence of any geophysical testing and considering the anticipated foundation levels, the site may be classified as Site Class C in accordance with Table 4.1 of the *CHBDC* (2019). Geophysics testing (i.e., shear wave velocity measurements), if carried out, could potentially provide a more favourable Site Class designation.

6.4.2 Spectral Response Values and Seismic Performance Category

Based on the location of the Skootamata River Bridge, the reference Site Class C spectral acceleration values were obtained based on the 5th generation seismic hazard maps published by the Geological Survey of Canada (GSC). In accordance with Section 4.4.3.1 of *CHBDC* (2019), the peak ground acceleration (PGA), peak ground velocity (PGV) and 5 per cent damped spectral response acceleration ($S_a(T)$) values for Site Class C are presented below.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
PGA (g)	0.039	0.057	0.089
PGV (m/s)	0.037	0.056	0.088
$S_a(0.2)$ (g)	0.069	0.099	0.148
$S_a(0.5)$ (g)	0.049	0.070	0.103
$S_a(1.0)$ (g)	0.029	0.042	0.062
$S_a(2.0)$ (g)	0.014	0.021	0.032
$S_a(5.0)$ (g)	0.003	0.005	0.009
$S_a(10.0)$ (g)	0.001	0.002	0.004

Given that the seismic hazard values above reference ground conditions associated with Site Class C, further modification to obtain site-specific seismic hazard values are not required. The design spectral response acceleration ($S(T)$) values are equal to the 5 per cent damped spectral response acceleration ($S_a(T)$) values.

In accordance with Table 4.10 of the *CHBDC* (2019), the bridge structure (Importance Category of "Major-Route"), falls within Seismic Performance Category 1 and therefore, analysis for seismic loads is not required as per Section 4.4.5.1 of the *CHBDC*.

6.5 Spread Footings

6.5.1 Founding Elevations and Geotechnical Resistances

The footings for the replacement bridge and temporary modular bridges can be founded directly on the surface of the slightly weathered to fresh, fair to excellent quality bedrock (i.e., rock quality designation (RQD) > 50 per cent), although it is noted that the igneous/metamorphic bedrock surface undulates and/or slopes within the footprint of the foundation elements. Some bedrock excavation would be required to create a level surface, or alternatively, to minimize bedrock excavation, the footings can be constructed on mass concrete that is placed over the fair to excellent quality bedrock following excavation, cleaning and inspection of the rock subgrade. In this regard, the table below summarizes the range in bedrock surface elevation encountered in the boreholes within/near each foundation footprint, from which the structural designers may select founding levels that either require bedrock excavation, placement of mass concrete, or a combination of the two. It is also understood that with the removal of the existing bridge footings and some regrading at the site, it may be necessary to place the abutment footings at a lower elevation with more bedrock excavation, to avoid expositing the top edge of the footings in the regraded abutment foreslope/river bank. The footings can be designed based on the factored ultimate geotechnical resistance and the factored serviceability geotechnical resistance (for 25 mm of settlement) as outlined in the table below.

Foundation Element	Reference Boreholes	Founding Material	Maximum (Highest) Founding Elevation Based on Approximate Surface Elevation of Fair to Excellent Quality Bedrock (m)(1,2)	Design Founding Elevation (m)(5)	Factored Ultimate Geotechnical Resistance (MPa)	Factored Serviceability Geotechnical Resistance (for 25 mm of Settlement) (MPa)
Proposed Replacement Structure						
West Abutment	S1, S2, S17 & S18	Good to excellent quality, very strong, amphibolite bedrock	162.2 to 161.1	160.2	60	N/A
East Abutment	S3, S4, S20 & S21	Fair to excellent quality, very strong, gneiss bedrock	162.9 to 161.6(3)	159.8	50	N/A
Temporary Modular Bridge – North Detour						
West Abutment	S9	Excellent quality, very strong, amphibolite bedrock	162.3	160.0	60	N/A
East Abutment	S10	Fair quality, very strong, amphibolite bedrock	160.3(4)	160.0	50	N/A
Temporary Modular Bridge –South Detour						
West Abutment	S13	Fair to excellent quality, very strong, gneiss bedrock	162.3	160.0	45	N/A
East Abutment	S14	Fair to excellent quality, very strong, amphibolite bedrock	162.6	160.0	60	N/A

Note(s):
1. Founding elevations are based on the highest fair to excellent quality bedrock elevation encountered in the boreholes across a foundation element, which generally corresponds to the surface of the bedrock at this site, although minor subexcavation of fractured rock may be required at some locations.
2. To minimize bedrock excavation, higher founding levels may be adopted by the structural designer with the use of thicker mass concrete, following excavation to and cleaning of the variable bedrock surface.
3. The founding elevation on the south side of the east abutment could be lower (at Elevation 160.7 m), as possible rock fill and/or bedrock slab was encountered in Borehole S3 above the concrete layer at Elevation 161.6 m.
4. Borehole S10 encountered cobbles and boulders above the bedrock. This borehole could not be located directly on the foundation element due to site constraints so the elevation of the bedrock at the east detour abutment may be different than encountered in this borehole.
5. The design founding level given in this column is understood to be based on setting an elevation that limits the potential for exposure of the front edge of the footing adjacent to the Skootamatta River.

The factored serviceability geotechnical resistances for 25 mm of settlement, which are based on footing sizes up to 4 m wide, will be greater than the factored ultimate geotechnical resistances given in the table above and as such, the ULS conditions will govern the design. Footing levels lower than the elevations given above can utilize the same geotechnical resistances. If higher founding levels are used in conjunction with mass concrete, the geotechnical resistances provided also assume that any mass concrete required to level the footing bearing surface (i.e., up to the elevation of the highest point of bedrock as encountered in the boreholes at each abutment and as indicated above) will have a uniaxial compressive strength (UCS) no less than that of the footing concrete.

In addition, the geotechnical resistances provided above are dependent on the footing size and will have to be re-evaluated and modified if larger footing sizes are considered. These geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.10.5 of the *CHBDC* (2019) and its Commentary.

6.5.2 Subgrade Preparation

All existing overburden soils (i.e., fill, rock fill, topsoil, silt to sandy silt and silty sand), moderately weathered bedrock (if encountered), and very poor to poor quality fractured bedrock (i.e., RQD < 50 per cent) should be sub-excavated, and the founding surface properly cleaned and prepared, prior to placing/pouring the footings or mass concrete.

The bedrock surface was noted to be variable across the footprint of the proposed replacement bridge abutments with a top of bedrock elevation difference of about 1.1 m across the west abutment and about 1.4 m across the east abutment (possibly up to about 2.2 m of elevation difference, if rock fill and/or bedrock slab is encountered above the concrete at Elevation 161.6 m in Borehole S3 and following sub-excavation of this material). Further, if the location of the foundation elements change from that investigated, the bedrock elevation will likely vary from that in the boreholes, particularly at the north detour east abutment as the borehole could not be drilled directly on the foundation element due to site constraints. Previously blasted rock may also be present at the location of old bridge elements. The design-build team should be aware of the variation of the bedrock surface across each foundation element and the potential for fractured bedrock and cobbles and boulders above the good quality bedrock surface, with differences occurring between and beyond the boreholes. Given the variable bedrock conditions, it is recommended that a Notice to Contractor be included in the DB Ready specifications to alert the Contractor to the variability in the bedrock surface elevations at this site.

Consideration may need to be given to dowelling the footings to augment sliding resistance on sloping bedrock, and/or to levelling the bedrock surface with mass concrete to create a horizontal bearing surface for the footings. Alternatively, consideration could be given to lowering the footing founding elevation to the lowest point of bedrock within the footprint and sub-excavating the upper portion of the exposed bedrock, as required, although this increased rock excavation will also increase the potential for noise and vibration impacts on the creek and neighbouring properties.

The bedrock is classified as strong to very strong and pre-drilling and hoe-ramming techniques alone may not be adequate to excavate the bedrock at this site. As such, consideration could be given to controlled blasting techniques as per OPSS.PROV 120 *(Explosives)* and OPSS.PROV 202 *(Rock Removal - Manual or Blasting)* in order to preserve the integrity of the rock mass in the area of the footing excavations. Pre-shearing, line-drilling or other specialized techniques may be required to maintain the excavation lines and preserve the integrity of the rock mass along the footprint of the footings. If adopted, the effect of blasting on the existing roadway, existing bridge and temporary protection systems, the river and fisheries, and surrounding properties must be considered by the designer and by the blasting contractor. The geotechnical resistances provided above assume that the bedrock at and below the founding level has not been adversely fractured or damaged by blasting or other means of excavation, and that all loose/shattered rock is removed prior to placement of mass concrete or the structural concrete for the footings.

The subgrade (excavated bedrock surface) should be inspected by a Foundation Engineering Specialist following subexcavation and cleaning to check that the rock mass integrity was preserved during excavation and that the bedrock surface is properly cleaned, scaled with all loosened debris removed prior to placing/pouring the concrete for footings in accordance with DBSP 0902 *(Excavating and Backfilling Structures),* as amended by FOUN0003 *(Dewatering of Structure Excavation)* and the NSSP addressing *rock excavation for structures,* copies of which are included in Appendix C.

6.5.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the concrete footings and the properly cleaned and prepared bedrock surface should be calculated in accordance with Section 6.10.4 of the *CHBDC* (2019) applying the appropriate consequence and degree of site understanding factor as noted in Section 6.2. For footings founded directly on properly prepared, slightly weathered to fresh bedrock, the unfactored coefficient of friction ($tan\ \varphi$') may be taken as 0.70 for cast-in-place footings. For cast-in-place footings founded on mass concrete, the unfactored coefficient of friction ($tan\ \varphi$') may be taken as 0.70.

Dowels connecting the concrete footing to the bedrock should be incorporated into the design if additional horizontal resistance is required on sloping bedrock, as discussed in Section 6.5.2. In this regard, it is generally recommended that dowels be incorporated into the design where bedrock is estimated to slope at greater than 10 degrees and/or if additional horizontal resistance is required. If dowels into bedrock are required, Special Provision (SP) FOUN0002 *(Dowels into Rock)* should be included in the specifications, a copy of which is included in Appendix C. Sloping foundations should be checked against sliding failure by the Structural Designer.

The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout, and steel. Where the rock mass is stronger than the concrete (which is the case at this site), the design of the dowels into the rock may be handled in the same way as the dowel embedment into the concrete, for uniaxial compressive strength of the grout similar to that of the concrete. Dowels should have a minimum 1 m embedment into the fair quality (i.e., RQD > 50 per cent) bedrock and the structural strength of the dowels and compressive strength of the grout should not be exceeded.

6.5.4 Frost Protection

The estimated frost penetration depth in the area of the Skootamatta River Bridge is 1.6 m as interpreted from OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).* However, for footings founded on bedrock or mass concrete over bedrock, soil cover for protection from frost penetration is not necessary.

6.6 Drilled Steel Casings

Drilled steel casings socketed into the amphibolite and gneiss bedrock may be used for support of the abutments. It is recommended that MTO's Special Provision DBSP 0903 be included in the procurement-ready package, in the event that the DB proponent elects to use drilled steel casing foundations.

The RQD values on bedrock core recovered at the site are between 43% and 100% and the bedrock is considered strong to very strong in accordance with Tables 3.10 and 3.5 of the *Canadian Foundation Engineering Manual* based on the drillhole data and the laboratory unconfined compressive strengths typically in the range of about 85 MPa to 250 MPa.

6.6.1 Founding Level and Axial Geotechnical Resistance

It is recommended that the 600 mm diameter drilled steel casings be socketed a minimum of at least 1 m into good quality bedrock. The surface of the bedrock was encountered in the boreholes between Elevations 162.9 m and 160.5 m in the vicinity of the proposed main bridge abutments. Steel casings with a diameter of 600 mm have been considered herein for ease of installation compared to larger casing sizes, although the design-build proponent may elect to optimize their design for alternate equipment and casing sizes.

Drilled steel casings with a diameter of 600 mm socketed a minimum of 1 m into good or better quality bedrock should be designed based on a factored ultimate geotechnical sidewall resistance of 2,900 kN/m length of rock

socket. If the bottom of the drilled casing hole is thoroughly cleaned and the bedrock quality verified to be good or better by camera inspection, then the drilled steel casings can be designed for a combination of sidewall resistance and end bearing. In this case, a factored ultimate geotechnical end bearing resistance of 30 MPa should be used. Serviceability Limit States resistances will not apply to drilled casings founded within the fresh to slightly weathered, good to excellent quality (RQD) bedrock, since the factored SLS resistance for 25 mm of settlement is greater than the factored ultimate axial geotechnical resistance.

6.6.2 Resistance to Lateral Loads

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

The resistance to lateral loading in front of a single casing may be estimated using subgrade reaction theory and the coefficient of horizontal subgrade reaction, k_h (kPa/m). However, the response of a casing to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum casing deflections are less than 1 percent of the casing diameter, where the loading is static (no cycling) and where the casing material is linear (CFEM, 2006). If one or more of these conditions are not satisfied, then it is recommended that the lateral casing analysis be carried out using p-y curves which can be provided upon request.

The factored ultimate lateral resistance for 600 mm diameter casings in moderately weathered to fresh bedrock is 2 MN/m length.

The following equations (CFEM, 1992 as referenced in the CHBDC Commentary, 2006) may be used to calculate values of k_h. The k_h value should be reduced by the appropriate factor where there is sloping ground in vicinity of the casing caps. Group effects on the lateral resistance should be taken into account in accordance with CHBDC 2019 Section 6.11.3.4, Figures C6-22 to C6-24, as applicable.

For bedrock:

$$k_h = \frac{K_h}{B}$$

Where: k_h= is the coefficient of horizontal subgrade reaction (kPa/m);

K_h= spring constant (kN/m/m)

B= casing diameter or width (m)

For moderately weathered to fresh bedrock a spring constant value of K_h = 12,000 MN/m/m may be used for 600 mm diameter casings.

6.7 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls and any associated wingwalls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutment walls and wingwalls:

- 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 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- For structures that are not comprised of integral or semi-integral abutments, rock fill may be used as backfill behind the walls and the material should meet the specification as outlined in the Northern Region Directive (2002) for back fill of structures adjacent to rock embankments. Other aspects of rock backfill requirements should be in accordance with OPSD 3101.200 (*Wall, Abutment, Backfill, Rock*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with the *CHBDC* (2019) Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501 *(Compacting).* 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 1.6 m behind the back of the wall in accordance with Figure C6.31(a) of the *Commentary to the CHBDC* (2019). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:<1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.31(b) of the *Commentary to the CHBDC* (2019).

6.7.1 Static Lateral Earth Pressures

The following guidelines and recommendations are provided regarding the lateral earth pressures for static loading conditions. The parameters below assume level backfill and ground surface behind the abutment walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

For restrained walls, the pressures are based on the proposed new embankment fill behind the granular backfill zone, and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM) for the general embankment fill:

Fill Type	Unit Weight of Material (kN/m^3)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Earth Fill / Select Subgrade Material	20	0.50	0.33

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

Fill Type	Unit Weight of Material (kN/m^3)	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22	0.43	0.27
Granular 'B' Type II	21	0.43	0.27

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 C6.12 of the *Commentary to the CHBDC* (2019).

¡ 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

As outlined in Section 6.1, the road surface of the replacement bridge crossing Skootamatta River will be at about Elevation 166 m, requiring placement of up to about 6 m of fill to construct the east and west approach embankments. Outside of the abutment wingwalls, the embankment is proposed to be constructed from granular fill (OPSS.PROV 1010 Granular B Type I or II) with 2 horizontal to 1 vertical (2H:1V) side slopes. Alternatively, the embankments could be constructed using rock fill with 1.25 horizontal to 1 vertical (1.25H:1V) side slopes.

At both the west and the east approaches, the ground surface surrounding the bridge is relatively flat and then slopes down to the river. The boreholes advanced in the area generally encountered silty sand to gravel fill and rock fill on top of amphibolite and gneiss and amphibolite bedrock.

For the detour embankments, the ground surface is relatively flat to the TMB abutments with assumed minimal grade raise and these embankments should be constructed out of granular fill (Granular B Type I or II).

6.8.1 Global Stability

6.8.1.1 Analysis Methods

Limit equilibrium slope stability analysis was carried out for the new higher/wider highway embankments using the commercially available program Slide 2018, produced by Rocscience, employing the Morgenstern-Price method of analysis. For the analyses, the Factor of Safety (FoS) of numerous potential surfaces was computed in order to establish the minimum FoS. The stability analyses were performed to check that the target minimum FoS was

achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design.

The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, Φ_{gu} (i.e., FoS = 1 / (Ψ * Φ_{gu})). Accordingly, a target minimum FoS of 1.5 has been used for the design of the permanent, final embankment configuration as per Table 6.2 of the *CHBDC* (2019) using effective stress (drained) conditions, as applicable.

The stability analyses carried out for bridge design includes assessment of the front abutment slopes as well as the side slopes. The stability analyses were completed based on the subsurface conditions as encountered in Boreholes S1, S2, S3, S4, S17, S18, S20, and S21.

6.8.1.2 Parameter Selection

For the new granular fill, rock fill and the existing fill, effective stress parameters were employed in the analysis assuming drained conditions, and the strength parameters were estimated from empirical correlations based on the in-situ SPT 'N'-values. The correlations proposed by Terzaghi and Peck (1967) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

The foundation engineering parameters for the soil types encountered in the boreholes at the approach embankments are summarized below. For the stability and settlement analyses, the groundwater level at the approach embankments was assumed to be at the river level (Elevation 160.3 m, September 28, 2020) and following the native soil/bedrock interface away from the river as evidenced from the installed piezometer.

Stratigraphic Unit	γ (kN/m^3)	φ' (°)
New Granular B Type I or II Fill	21	35
Rock Fill	19	48
Existing Silty Sand to Gravel Fill	21	35
Bedrock	Impenetrable	

6.8.1.3 Analysis Results

The results of the analyses indicate that for an approximately 6 m high embankment constructed using OPSS.PROV 1010 Granular 'B' (Type I or II) fill material with side slopes inclined at 2H:1V, the FoS for long-term global stability is greater than 1.5, which satisfies the minimum target FoS for the respective conditions (see Figure 1).

The results of the analyses indicate that for an approximately 6 m high embankment constructed using rock fill material with side slopes inclined at 1.25H:1V, the FoS for long-term global stability is greater than 1.5, which satisfies the minimum target FoS for the respective conditions (see Figure 2).

As the footings for the proposed replacement bridge are to be founded directly on the properly prepared bedrock with the front slope embankment fill being supported by the abutment wall, slope stability issues are not anticipated for the proposed front slopes.

Based on the geometry of the detour abutment embankments, slope stability issues are not anticipated. However further analysis will be required if the detour embankments exceed what is anticipated based on the existing information.

6.8.2 Settlement

Settlement of the east and west approach embankments can be expected as a result of the loading from the new fills, on the native cohesionless deposits (if not removed) although given their minor thickness (typically less than 1 m) this settlement is expected to be minimal and would occur during construction. Similarly, any existing fills left in place (not containing organics) are expected to experience less than 25 mm of settlement, which would occur during construction.

If granular fill is used for embankment construction and it is properly placed and compacted, settlement is anticipated to be minimal and would occur during construction. The use of earth fill for embankment construction is not recommended as there could be post-construction settlement of the fill.

The settlement is expected to meet the criteria of less than 25 mm at the bridge abutments in accordance with the MTO Guideline, Embankment Settlement Criteria for Design (2010).

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

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

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

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

For a rock fill thickness of 6 m, the estimated short-term and long-term settlement is 45 mm and about 5 mm, respectively, for a total of 50 mm. Approximately 90 per cent of the short-term settlement may be expected to occur within the first six months following construction of the embankment to full height. The short-term settlement is expected to be fully completed within one year following the completion of embankment construction to full

height. The long-term rock fill settlement is expected to occur from one year following the completion of construction over the life of the embankment. Settlement of the rock fill will be differential with respect to the existing embankment. In this case, delaying of final paving for 3 to 6 months would be required.

All new fill should be “keyed-in” or benched into the existing fills, in accordance with OPSD 208.010 (Benching of Earth Slopes). Where granular fill is placed over rock fill, a geotextile separator would be required to avoid migration of fines into the rock fill.

6.9 Construction Considerations

6.9.1 Subgrade Preparation and Embankment Construction

All existing organics (i.e., topsoil, and/or mixed organic soil) are recommended to be removed below the footprint of the proposed widened embankments. Fill for construction of the proposed approach embankments should consist of OPSS.PROV 1010 *(Aggregates)* Granular ‘A’, Granular ‘B’ (Type I or II), or rock fill. For portions of the embankment and/or abutment backfill extending below the groundwater level, it is recommended that Granular ‘B’ Type II or rock fill be used. The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 *(Compacting)* and OPSS.PROV 206 *(Grading).* Granular fill embankment side slopes should be constructed no steeper than 2H:1V. Rock fill embankments side slopes should be no steeper than 1.25H:1V.

The embankment front slopes and side slopes adjacent to the river require erosion protection in accordance with OPSS.PROV 511 *(Rip Rap, Rock Protection, and Granular Sheeting).* Erosion protection should be placed on the slopes up to at least 0.5 m above the design high water level. Erosion protection could consist of a minimum 0.6 m thick layer of R-10 Rip Rap (300 mm diameter as per OPSS.PROV 1004 *(Aggregates),* or rock protection.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding as per OPSS 802 *(Topsoil)* and OPSS 804 *(Seed and Cover)* should be carried out as soon as possible after construction of the embankments (unless rock fill is used). If this slope protection is not in place before winter, then alternate erosion protection measures, such as covering the slopes with straw or granular sheeting as per OPSS.PROV 511 *(Rip Rap, Rock Protection, and Granular Sheeting)* will be required to reduce the potential for remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

6.9.2 Excavations and Temporary Protection Systems

Excavations of up to about 6 m depth for construction of the replacement bridge foundations are anticipated to extend through the existing overburden soils (i.e., fill, rock fill, topsoil, silt to sandy silt and silty sand), cobbles and boulders (where encountered), moderately weathered bedrock (where encountered), and any very poor to poor quality rock to expose the underlying slightly weathered to fresh bedrock with an RQD > 50 per cent. Additional excavation into the strong to very strong, slightly weathered to fresh bedrock may also be required depending on the final foundation elevation selected for the footings.

Open-cut excavations, if feasible at this site, shall be carried out in accordance with Ontario Regulation 213, Ontario Occupational Health and Safety Act for Construction Projects (as amended). The overburden soils are considered to be Type 3 soils above the groundwater table and Type 4 soils below the groundwater table. Temporary open-cut excavations in Type 3 soils should remain stable if side slopes are formed no steeper than 1H:1V. In Type 4 soils, the excavation side slopes should be formed no steeper than 3H:1V, although steeper slopes may be adopted if dewatering is successfully implemented. Excavations into the bedrock, where required, may be cut vertically or near-vertically depending on the degree of weathering and jointing/fracturing at the excavation face.

It is recommended that a Notice to Contractor be included in the DB-Ready package to alert the Design-Build Contractor to the presence of obstructions within the existing fill and at the surface of the bedrock. Where required, temporary protection systems are expected to consist of soldier piles and lagging. The installation of sheet-piles is expected to be impeded by the presence of cobbles and/or boulders or rock fill, although a heavier sheet pile section may successfully penetrate shallower fill thicknesses. Given the shallow depth to bedrock at this site, sheet pile installations would also require the drilling/placement of toe-pins to fix the base of the sheeting to the top of the bedrock. Support to the sheet-pile system, if required, could be in the form of struts and wales and rakers or anchors.

Soldier piles and lagging would likely be more suitable to penetrate through the obstructions but would still require pre-drilling to socket the H-piles into bedrock. Support to the soldier pile and lagging system could also be in the form of struts and wales and rakers or anchors.

All temporary protection systems shall be designed and constructed in accordance with DBSP 539 (*Temporary Protection Systems),* as amended by the Non-Standard Special Provision for Dewatering of Structure Excavation (Cofferdams), a copy of which is included in Appendix C*.* The lateral movement of the temporary shoring system shall meet Performance Level 2 as specified in DBSP 539. The Design-Build Contractor is responsible for the selection and complete detail design of the temporary protection systems at this site.

6.9.3 Control of Groundwater and Surface Water

Temporary excavations along the proposed new alignment will be required to facilitate sub-excavation of the fill, rock fill and native soils and bedrock in advance of the footing construction. The elevation of the groundwater table at the site is generally anticipated to be at the river level and then locally at or just above the interface of the native soils/bedrock surface. As such, groundwater seepage into the excavation should be expected from the relatively permeable native soils, fill and from joints/fractures within the bedrock. Therefore, control of groundwater will be necessary to allow for levelling of the bedrock surface and/or construction of the footings in dry conditions.

As noted in Section 6.8.2, given the shallow bedrock conditions at this site, a sheet pile cofferdam would require the drilling/placement of toe-pins to fix the base of the sheeting to the top of the bedrock. In addition, sandbags or a concrete plug would likely be required to develop a seal between the base of the sheeting/shoring and the uneven bedrock surface in order to facilitate unwatering.

Consideration could be given to the use of a sandbag or inflatable bladder cofferdam to isolate the footing excavations from the river channel/ groundwater table and allow unwatering and footing construction in the dry. To minimize seepage, the sandbags or inflatable bladder should be placed directly on the bedrock surface. Groundwater seepage should still be anticipated between the base of the cofferdam system and the exposed bedrock surface and from fractures within the bedrock. As such, consideration may also be given to the use of a tremie concrete plug to seal the base of excavation(s) prior to dewatering; however, appropriate techniques and subgrade cleaning would be required to demonstrate that no overburden soils are trapped between the tremie plug and the top of bedrock, which would compromise the geotechnical resistances of the footings.

Surface water should be directed away from the excavation areas to prevent ponding of water that could impede footing construction. Unwatering of all excavations should be carried out in accordance with OPSS.PROV 517 *(Dewatering),* as modified by Special Provision (SP) 517F01 *(Dewatering System; Temporary Flow Passage System)* and the Non-Standard Special Provision FOUN0003 (*Dewatering of Structure Excavation*). A copy of FOUN0003 has been provided in Appendix C. Given the presence of existing infrastructure in the vicinity of the site, it is recommended that a pre-construction condition survey be carried out to capture existing wells within a

150 m radius from the site (consistent with that radius recommended for pre-construction surveys associated with vibration impacts); this has been reflected in the foundation designer fill-in in Table A of SP 517F01. In addition, the foundation insert requiring a minimum of 5 years experience for the dewatering system design engineer and design-checking engineer should be included in (SP) 517F01. These fill-ins should be completed by the design team during preparation of the DB Ready procurement package. The design, implementation and decommissioning of the groundwater control systems is the responsibility of the Design-Build Contractor.

Based on available design drawings, it is interpreted that the existing bridge structure footings are founded directly on bedrock. As such, the risk of potential settlement impacts to the existing bridge as a result of a temporary groundwater lowering are considered to be low.

Based on the final footing elevations and the river water level at the time of construction, an Environmental Activity Section Registry (EASR), and potentially a permit to take water (PTTW), may be required as per the Environmental Protection Act by the Ontario Ministry of the Environment, Conservation and Parks (MECP). The Design-Build Contractor must evaluate the estimated seepage and groundwater removal quantity, based on their design elevations and proposed construction methods/procedures, to make the final assessment/determination whether an EASR or PTTW is ultimately required.

6.9.4 Obstructions

Cobble, boulder and/or rock fill obstructions (as encountered in Boreholes S6, S10, S18 and S21) could affect the installation of temporary protection systems and/or temporary cofferdams (if required). It is recommended that an NSSP (or Notice to Contractor) be included in the specifications to warn the Design-Build Contractor of the possible presence of such obstructions. A copy of an NSSP is included in Appendix C. Note that the extent and depth of the obstructions may vary beyond and between the borehole locations.

As outlined in section 6.1, old bridge abutments are present on the north side of the existing bridge. The old bridge abutments will not have an impact on the footprint of the proposed bridge replacement however they may impact the installation of protection systems. The old bridge abutments are in the direct vicinity of the proposed north TMB alternative. Additional investigation would be necessary to determine if the old abutments are suitable for use with a TMB. The contractor should verify whether the old abutments will impact the main bridge or TMB construction and assess their impact on the TMB placement and design.

6.9.5 Vibration Monitoring

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities such as line drilling and/or hoe-ramming or temporary protection systems installation using vibratory methods will reach this threshold level and, therefore, vibration monitoring for the existing bridge is not expected to be required during construction at this site, if blasting operations are not used for bedrock excavation. It is recommended that vibration monitoring is carried out at the existing bridge in the event that blasting is used to excavate bedrock at the abutment locations.

Commercial properties are located within about 150 m of the proposed abutment locations. Lower PPV thresholds of 20 mm/s to 50 mm/s are generally considered applicable for vibration impacts on buildings. While the vibrations induced by construction activities will likely attenuate to below the PPV thresholds at distances of well below 150 m, humans are sensitive to vibrations at much lower levels. Therefore, it is recommended that vibration monitoring be carried out between the bridge replacement site and the properties located within 150 m

during shoring/cofferdam installation and bedrock excavation operations, if relatively limited bedrock excavation will be completed using line drilling and hoe-ramming techniques. In cases where blasting is used, the vibration monitoring should be extended to a 250 m radius consistent with the requirements of OPSS 120 *(Explosives)*. An NSSP describing the requirements for vibration monitoring is presented in Appendix C.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Mo'oud Nasr, P.Eng., a geotechnical engineer of Golder, and was reviewed by Ms. Sarah Poot, P.Eng., a senior geotechnical engineer and Associate of Golder.
Ms. Lisa Coyne, P.Eng., a Principal and MTO Designated Foundations Contact for Golder, completed a technical and quality review of this report.

Signature Page

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YS/MN/SEMP/LCC/ljv

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REFERENCES

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

Canadian Standards Association (CSA). 2019. *Canadian Highway Bridge Design Code* (CHBDC (2019)) and *Commentary on* CAN/CSA-S6-19. CSA Special Publication.

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

International Society for Rock Mechanics Commission on Test Methods. 1985. Int. J. Rock Mech.Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

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

MTO Foundations Guideline, Embankment Settlement Criteria for Design, July 2010.

MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates (2010).

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D7012	Standard Test Method for Compressive Strength and Elastic moduli of Intact Rock Core Specimens under Varying States of Stress and Temperature
ASTM D573108	Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification

Commercial Software:

Slide (Version 8) by Rocscience Inc.

Ontario Provisional Standard Drawing:

OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirement
OPSD 3101.200	Wall, Abutment, Backfill, Rock.
OPSD 3121.150	Walls, Retaining, Backfill Minimum Granular Requirement

Ontario Provincial Standard Specification:

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 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 802	Construction Specification for Topsoil
OPSS.PROV 804	Construction Specifications for Seed and Cover
OPSS.PROV 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1004	Material Specification for Aggregates - Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects (as amended)

Table 1: Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Risk / Consequence	Relative Costs
Spread / Strip Footings founded on bedrock	¡ Feasible for support of the abutments and is considered the preferred foundation option.	¡ Conventional construction techniques ¡ Presence of suitable stratum at shallow depths to provide required axial resistance.	¡ Dewatering and protection systems likely to be required, although will be a similar disadvantage for pile cap construction for deep foundations unless pile cap can be perched higher.	¡ Deeper excavation with more bedrock excavation. Depending on time of year, more significant dewatering effort may be required to allow construction in dry conditions.	¡ Lower relative cost than deep foundations.
Drilled Steel Casings founded socketed within bedrock	¡ Feasible in diameters ranging from about 450 mm to 750 mm; smaller diameters may be feasible subject to design by DB proponent.	¡ Reasonably conventional construction technique; may allow use of smaller equipment, for which access at abutments is reasonable throughout construction staging. ¡ Allows pile cap to be maintained relatively high, minimizing excavation and dewatering at abutments. ¡ Proven effective where rock sockets are required; this relatively smaller option will be more constructable and cost effective than larger diameter drilled shafts; better able to handle abrasive rock conditions and sloping bedrock conditions than driven piles or larger diameter caissons. ¡ Will minimize the amount of bedrock to be excavated compared to a strip footing option. ¡ Drilled steel casings will readily handle obstructions if present within the proposed abutment areas	¡ Requires more/larger specialized equipment compared to that required for footing excavation. ¡ Still requires excavation and protection systems to allow for construction of pile cap. ¡ Does not allow for integral abutment design.	¡ Some risk of individual drilled steel casings needing to extend deeper to encounter good quality bedrock, unless design is completed on more conservative geotechnical resistances.	¡ Higher installation cost compared to spread/strip footings based on specialized equipment required; however, offset by some savings in protection systems, excavation and dewatering.

MINISTRY OF TRANSPORTATION, ONTARIO

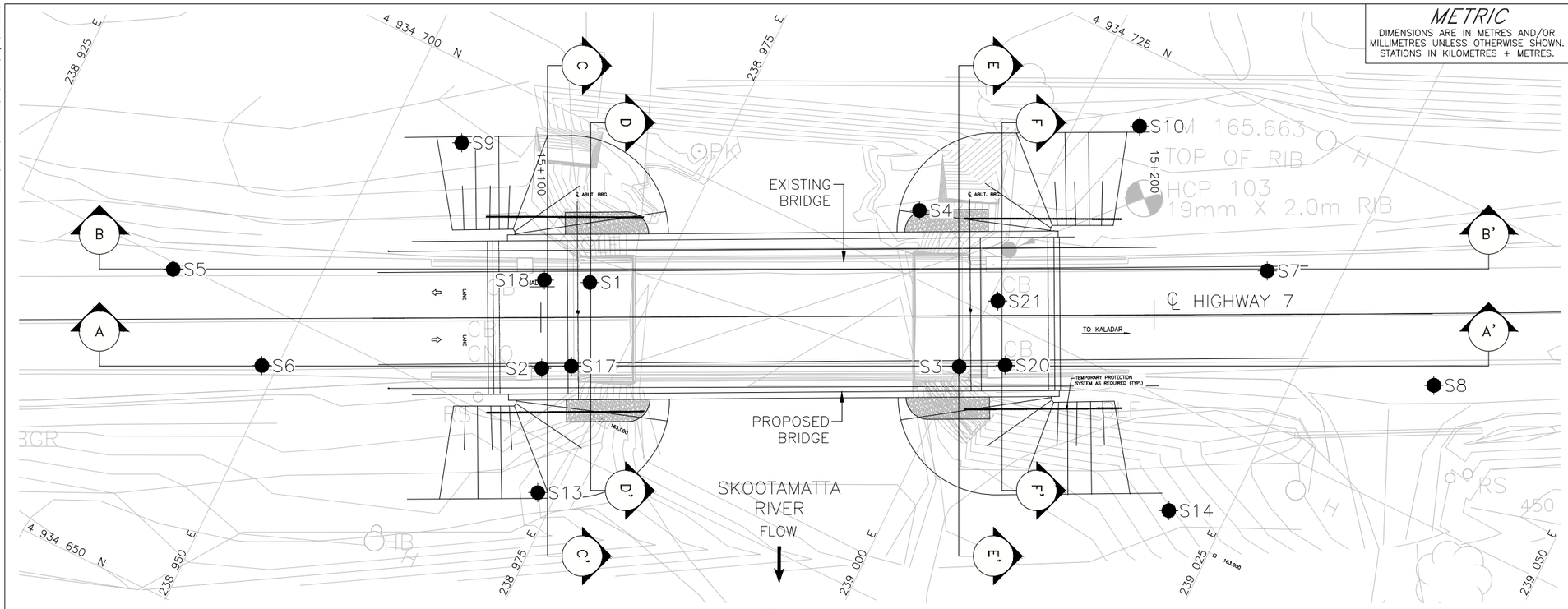
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 4091-14-01

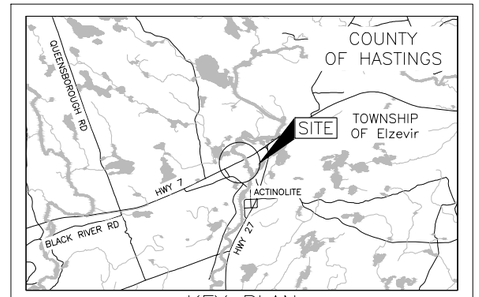
SHEET

SKOOTAMATTA RIVER BRIDGE
HIGHWAY 7
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER
MEMBER OF WSP



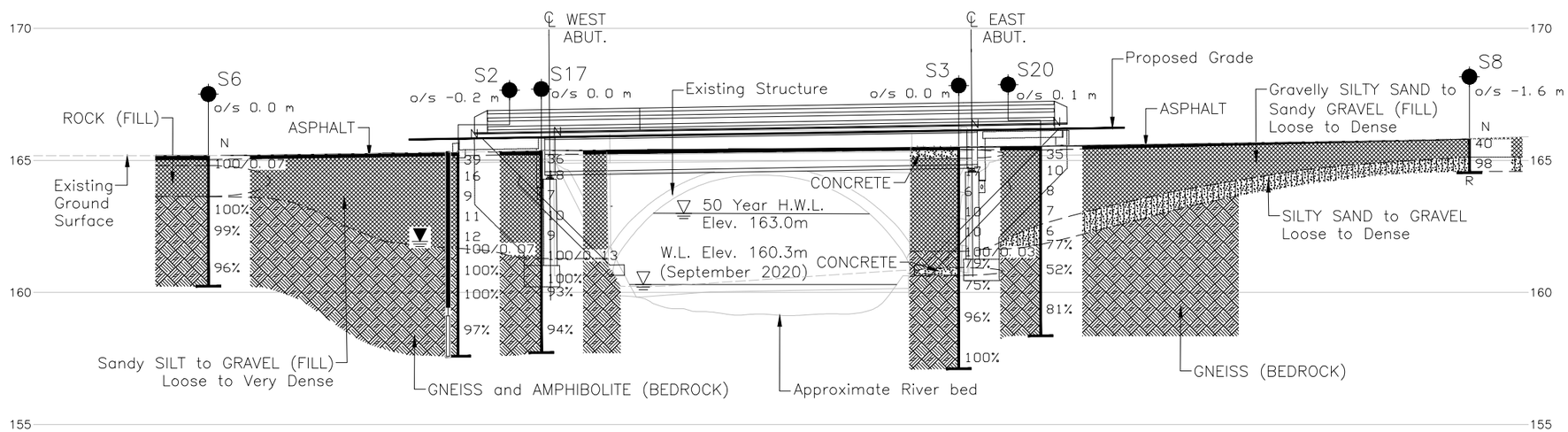
PLAN
SCALE
4 0 4 8 m



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Piezometer
- R Refusal
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on November 11, 2020



PROFILE A–A'

VERTICAL SCALE
2 0 2 4 m

HORIZONTAL SCALE
4 0 4 8 m

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE 9)			
No.	ELEVATION	NORTHING	EASTING
S1	165.4	4934688.2	238970.0
S2	165.3	4934680.2	238969.5
S3	165.5	4934695.1	239000.1
S4	160.5	4934705.2	238991.7
S5	165.2	4934674.5	238938.9
S6	165.2	4934670.5	238948.8
S7	165.7	4934713.0	239019.4
S8	165.8	4934710.5	239035.6
S9	164.0	4934693.9	238955.6
S10	164.7	4934719.1	239004.9
S13	163.1	4934670.9	238973.6
S14	163.0	4934691.9	239020.6
S17	165.4	4934681.4	238971.6
S18	165.4	4934686.8	238966.6
S20	165.5	4934696.8	239003.4
S21	165.6	4934701.3	239000.6

REFERENCE

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LICENSED PROFESSIONAL ENGINEER L.C. COYNE APR. 12, 2021 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.E.M. POOT 90471921 APR. 12, 2021 PROVINCE OF ONTARIO

NOTES

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NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 31C–304

HWY. 7		PROJECT NO. 1786659–17	DIST.
SUBM'D. MN	CHKD. MN	DATE: 4/12/2021	SITE:11X–0076/B0
DRAWN: DD/TR	CHKD. SEMP	APPD. LCC	DWG. 1

PLOT DATE: April 12, 2021
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MINISTRY OF TRANSPORTATION, ONTARIO

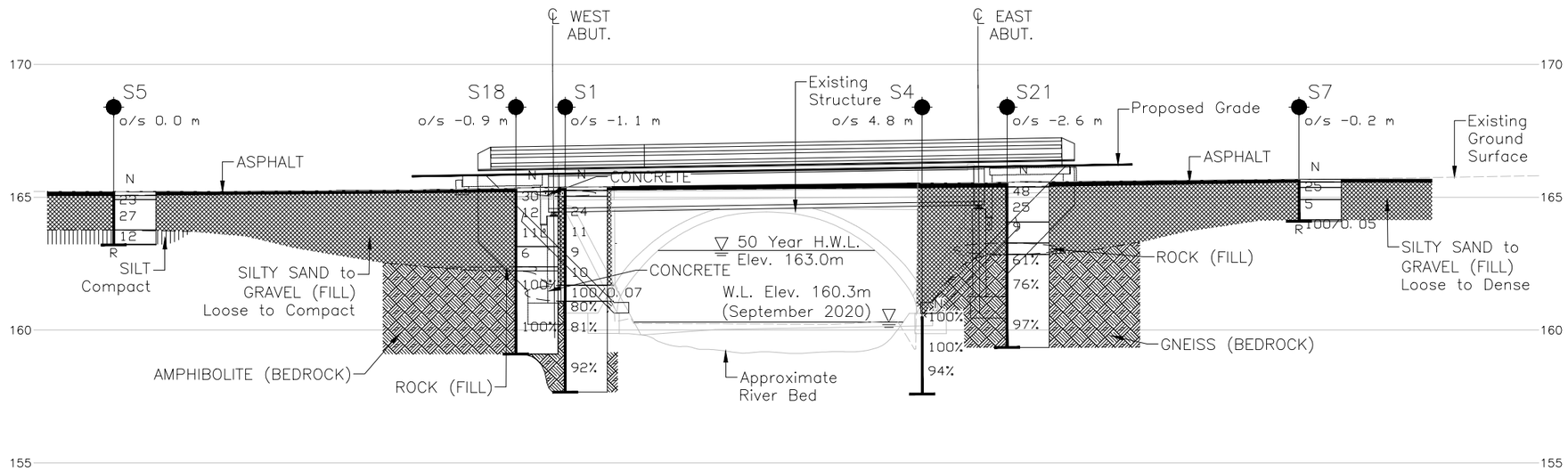
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 4091-14-01

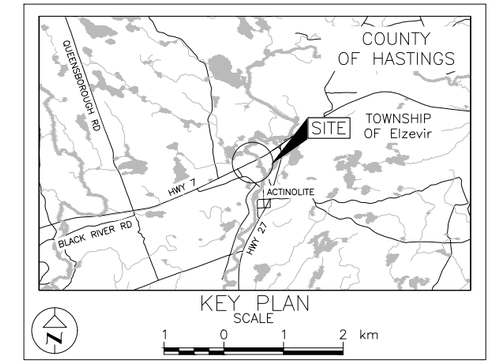
SKOOTAMATTA RIVER BRIDGE
HIGHWAY 7
SOIL STRATA

SHEET

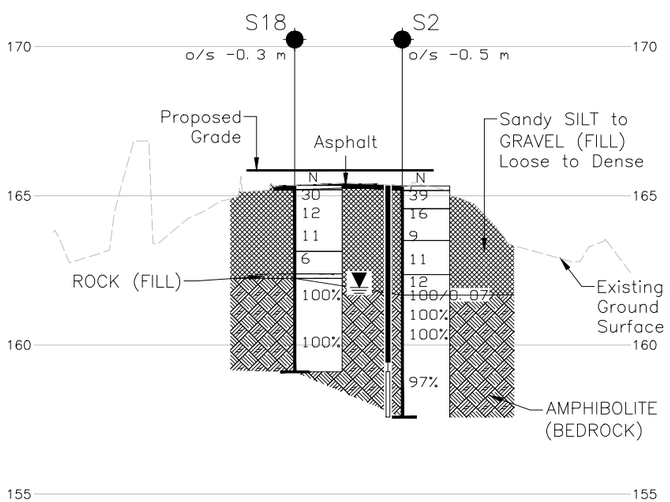
GOLDER
MEMBER OF WSP



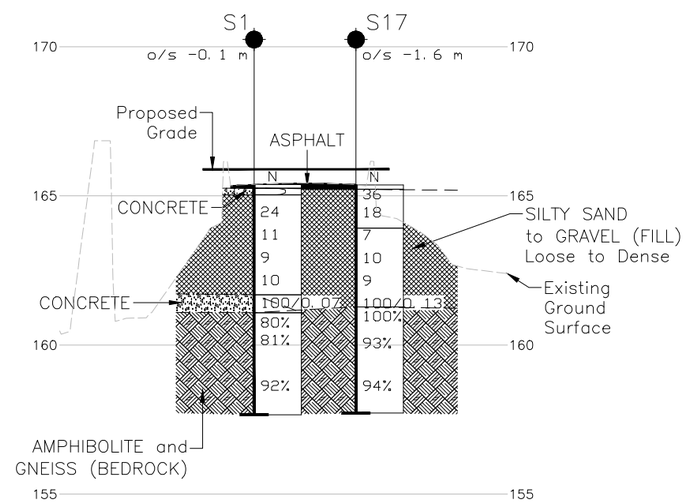
PROFILE B–B'



KEY PLAN
SCALE
1 0 1 2 km



CROSS–SECTION C–C'
(WEST ABUTMENT)



CROSS–SECTION D–D'
(WEST ABUTMENT)

VERTICAL SCALE
2 0 2 4 m

HORIZONTAL SCALE
4 0 4 8 m

LEGEND

- Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Piezometer
- R Refusal
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on November 11, 2020

BOREHOLE CO–ORDINATES (NAD83 MTM ZONE 9)

No.	ELEVATION	NORTHING	EASTING
S1	165.4	4934688.2	238970.0
S2	165.3	4934680.2	238969.5
S4	160.5	4934705.2	238991.7
S5	165.2	4934674.5	238938.9
S7	165.7	4934713.0	239019.4
S17	165.4	4934681.4	238971.6
S18	165.4	4934686.8	238966.6
S21	165.6	4934701.3	239000.6

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LICENSED PROFESSIONAL ENGINEER L.C. COYNER APR. 12, 2021 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.E.M. POOT 90471921 APR. 12, 2021 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 31C–304

HWY. 7		PROJECT NO. 1786659–17	DIST.
SUBM'D. MN	CHKD. MN	DATE: 4/12/2021	SITE:11X–0076/B0
DRAWN: DD/TR	CHKD. SEMP	APPD. LCC	DWG. 2

PLOT DATE: April 12, 2021
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MINISTRY OF TRANSPORTATION, ONTARIO

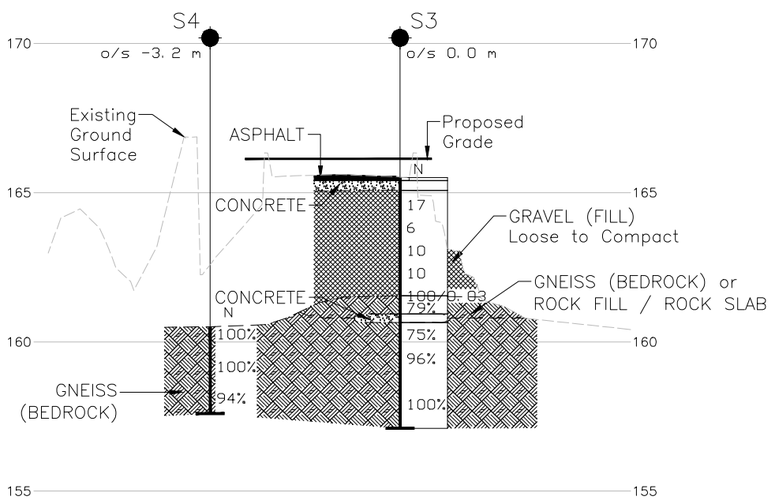
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 4091-14-01

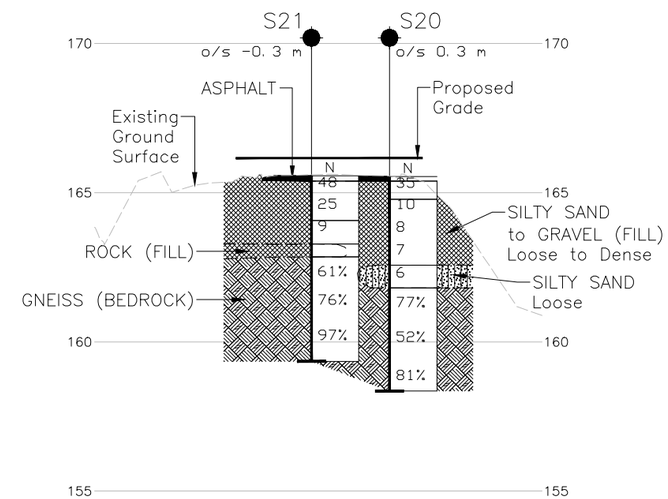
SKOOTAMATTA RIVER BRIDGE
HIGHWAY 7
SOIL STRATA

SHEET

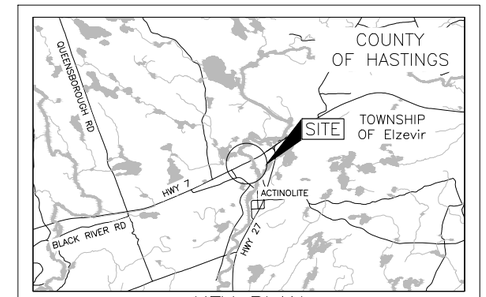
GOLDER
MEMBER OF WSP



CROSS-SECTION E-E'
(EAST ABUTMENT)



CROSS-SECTION F-F'
(EAST ABUTMENT)



LEGEND

●	Borehole – 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)

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE 9)

No.	ELEVATION	NORTHING	EASTING
S3	165.5	4934695.1	239000.1
S4	160.5	4934705.2	238991.7
S20	165.5	4934696.8	239003.4
S21	165.6	4934701.3	239000.6

NOTES

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VERTICAL SCALE
2 0 2 4 m

HORIZONTAL SCALE
4 0 4 8 m

LICENSED PROFESSIONAL ENGINEER
L.C. COYNE
APR. 12, 2021
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
S.E.M. POOT
90471921
APR. 12, 2021
PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 31C-304

HWY. 7	PROJECT NO. 1786659-17		DIST.
SUBM'D. MN	CHKD. MN	DATE: 4/12/2021	SITE:11X-0076/B0
DRAWN: DD/TR	CHKD. SEMP	APPD. LCC	DWG. 3

PLOT DATE: April 12, 2021
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MINISTRY OF TRANSPORTATION, ONTARIO

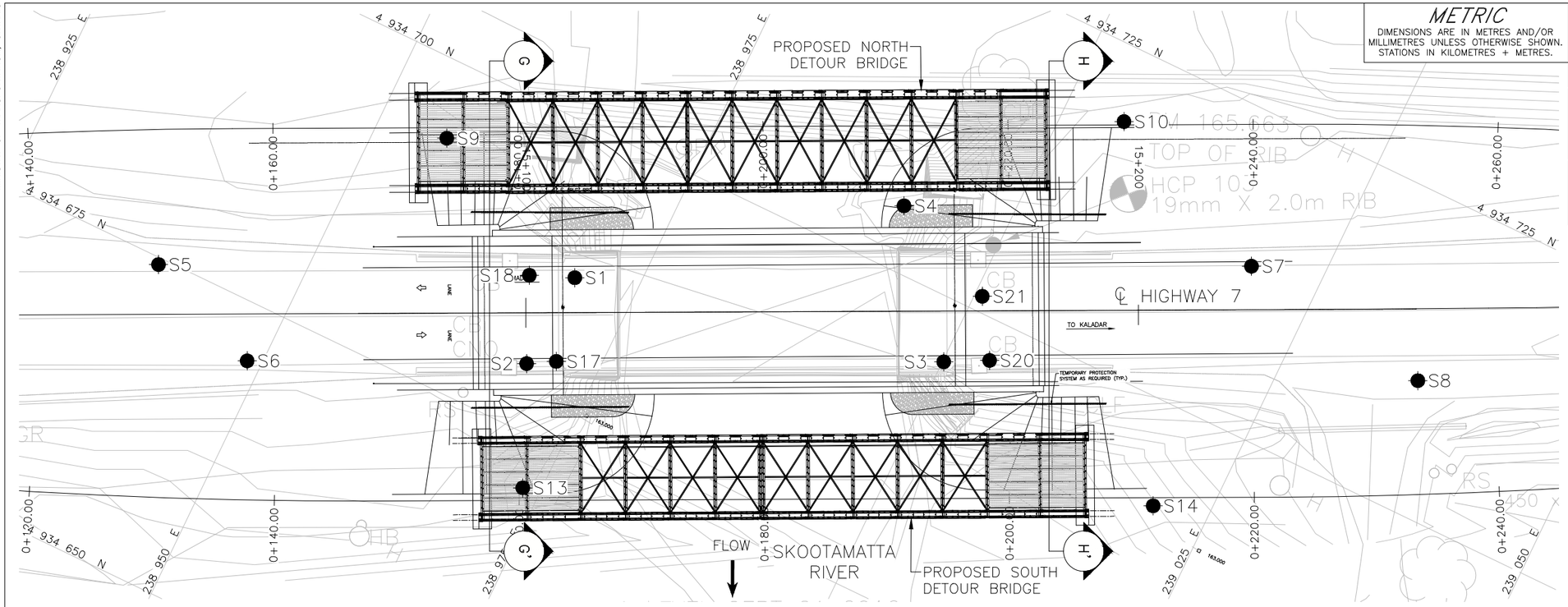
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 4091-14-01

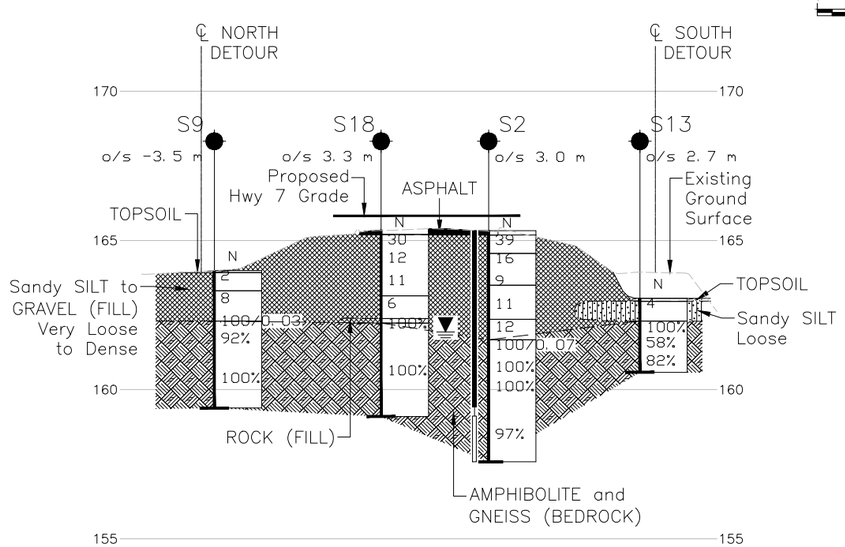
SHEET

SKOOTAMATTA RIVER BRIDGE DETOURS
HIGHWAY 7
BOREHOLE LOCATIONS AND SOIL STRATA

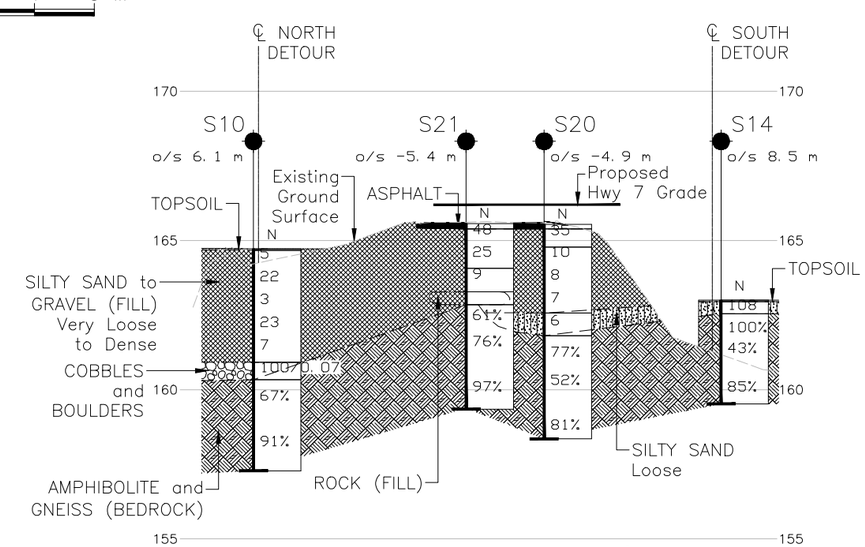
GOLDER
MEMBER OF WSP



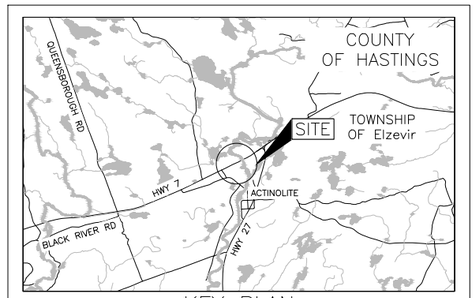
PLAN
SCALE
4 0 4 8 m



CROSS-SECTION G-G'
(WEST ABUTMENT)



CROSS-SECTION H-H'
(EAST ABUTMENT)



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Piezometer
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on November 11, 2020

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE 9)

No.	ELEVATION	NORTHING	EASTING
S1	165.4	4934688.2	238970.0
S2	165.3	4934680.2	238969.5
S3	165.5	4934695.1	239000.1
S4	160.5	4934705.2	238991.7
S5	165.2	4934674.5	238938.9
S6	165.2	4934670.5	238948.8
S7	165.7	4934713.0	239019.4
S8	165.8	4934710.5	239035.6
S9	164.0	4934693.9	238955.6
S10	164.7	4934719.1	239004.9
S13	163.1	4934670.9	238973.6
S14	163.0	4934691.9	239020.6
S17	165.4	4934681.4	238971.6
S18	165.4	4934686.8	238966.6
S20	165.5	4934696.8	239003.4
S21	165.6	4934701.3	239000.6

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Temporary Bridges provided in digital format by Parsons, drawing file no. 476977-TMB-Skootamatta River- South-1 Lane_Option 1 (meter).dwg and 476977-TMB-Skootamatta River-NORTH-2 Lane_Option4 (meter).dwg received November 12, 2020.

VERTICAL SCALE
2 0 2 4 m

HORIZONTAL SCALE
4 0 4 8 m

LICENSED PROFESSIONAL ENGINEER L.C. COYNE APR. 12, 2021 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.E.M. POOT 90471921 APR. 12, 2021 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 31C-304

HWY. 7		PROJECT NO. 1786659-17	DIST.
SUBM'D. MN	CHKD. MN	DATE: 4/12/2021	SITE:11X-0076/B0
DRAWN: DD/TR	CHKD. SEMP	APPD. LCC	DWG. 4

PLOT DATE: April 12, 2021
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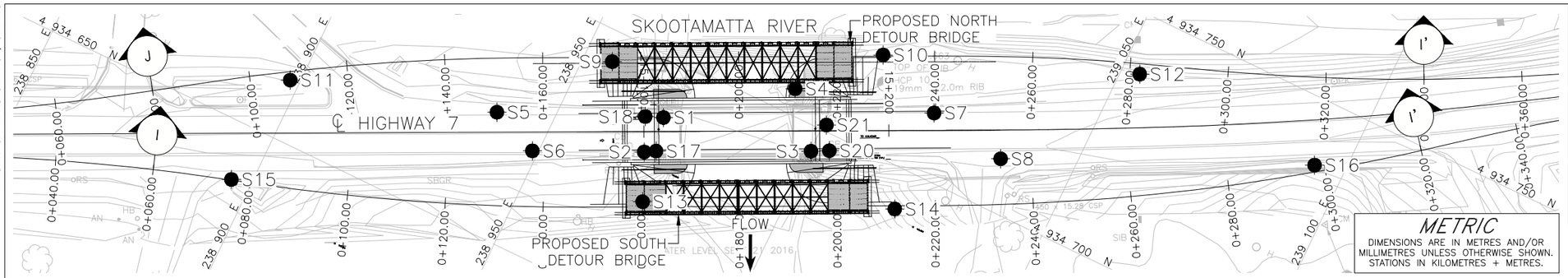
MINISTRY OF TRANSPORTATION, ONTARIO

CONT No.
WP No. 4091-14-01

SHEET

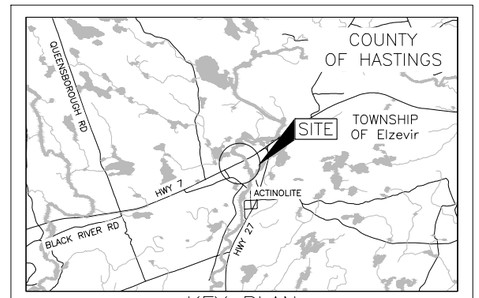
SKOOTAMATTA RIVER BRIDGE DETOURS
HIGHWAY 7
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER
MEMBER OF WSP

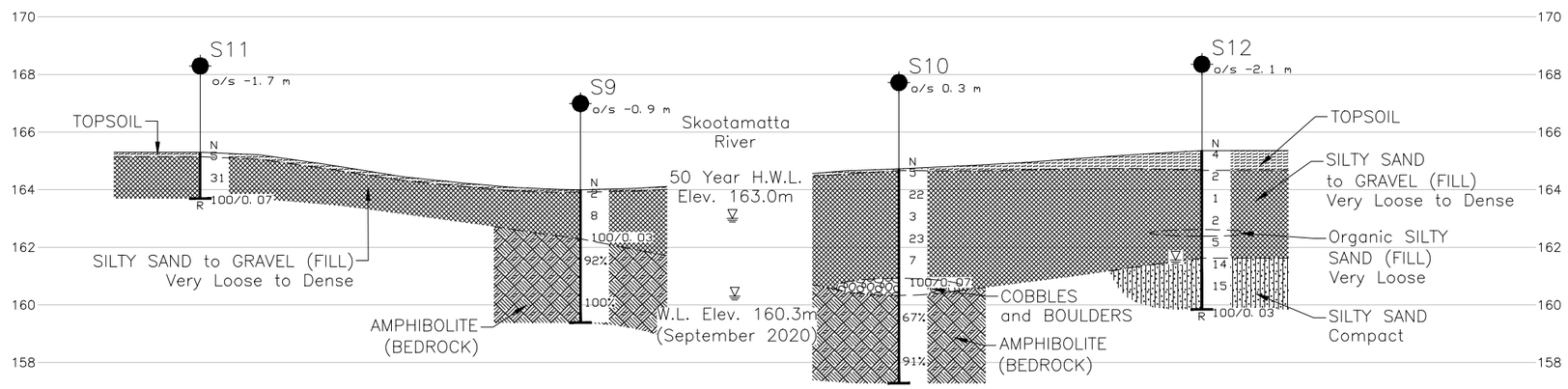


METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

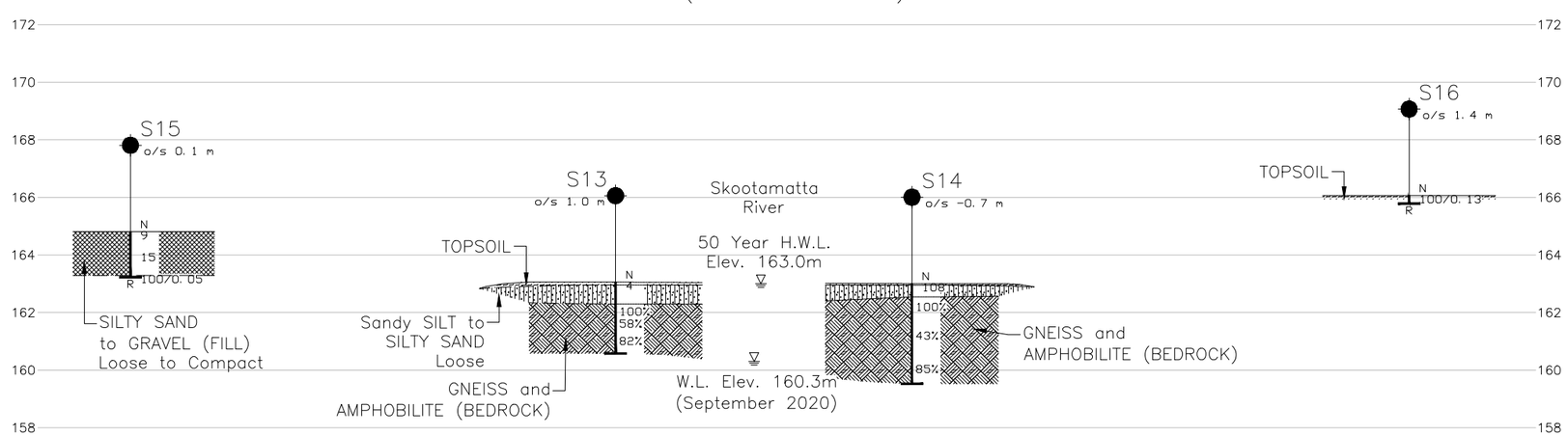
PLAN
SCALE
10 0 10 20 m



KEY PLAN
SCALE
1 0 1 2 km



PROFILE J–J'
(NORTH DETOUR)



PROFILE I–I'
(SOUTH DETOUR)

VERTICAL SCALE
2 0 2 4 m

HORIZONTAL SCALE
10 0 10 20 m

LEGEND

- Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
- 100% Rock Quality Designation (RQD)

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE 9)

No.	ELEVATION	NORTHING	EASTING
S1	165.4	4934688.2	238970.0
S2	165.3	4934680.2	238969.5
S3	165.5	4934695.1	239000.1
S4	160.5	4934705.2	238991.7
S5	165.2	4934674.5	238938.9
S6	165.2	4934670.5	238948.8
S7	165.7	4934713.0	239019.4
S8	165.8	4934710.5	239035.6
S9	164.0	4934693.9	238955.6
S10	164.7	4934719.1	239004.9
S11	165.3	4934662.1	238898.1
S12	165.4	4934738.3	239053.8
S13	163.1	4934670.9	238973.6
S14	163.0	4934691.9	239020.6
S15	164.8	4934638.6	238896.0
S16	166.1	4934737.1	239093.9
S17	165.4	4934681.4	238971.6
S18	165.4	4934686.8	238966.6
S20	165.5	4934696.8	239003.4
S21	165.6	4934701.3	239000.6

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LICENSED PROFESSIONAL ENGINEER L.C. COYNE APR. 12, 2021 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER S.E.M. POOT 90471921 APR. 12, 2021 PROVINCE OF ONTARIO

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NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 31C–304

HWY. 7		PROJECT NO. 1786659–17	DIST.
SUBM'D. MN	CHKD. MN	DATE: 4/12/2021	SITE:11X–0076/B0
DRAWN: DD/TR	CHKD. SEMP	APPD. LCC	DWG. 5

PLOT DATE: April 12, 2021
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Photograph 1: Skootamatta River Bridge, Facing North West (Oct 2020)
(looking towards location of proposed west abutment)

Photograph 2: Skootamatta River Bridge, Facing South West (Oct 2020)
(looking towards location of proposed west abutment)

Photograph 3: Skootamatta River Bridge, Facing North East (Oct 2020)
(looking towards location of proposed east abutment)

Photograph 4: Skootamatta River Bridge, Facing South East (Oct 2020)
(looking towards location of proposed east abutment)

Photograph 5: Existing Old Abutments at Skootamatta River Bridge (Oct 2020) (looking south west)

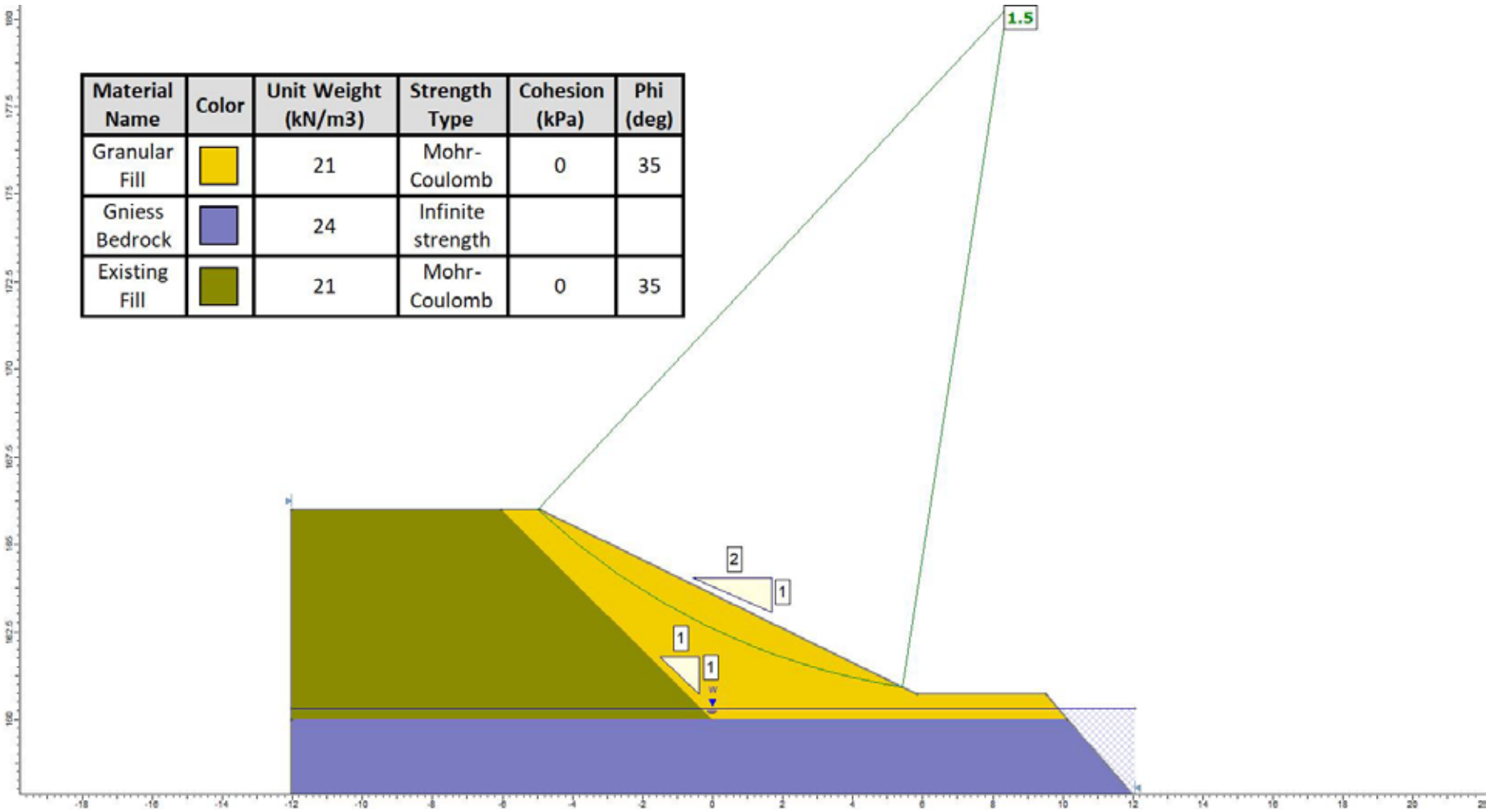
Photograph 6: Existing Old Abutments at Skootamatta River Bridge, Facing North East (image captured from Google Earth)

Global Stability Analysis

Figure 1

Skootamatta River Bridge Replacement – Highway 7
Granular Fill – Embankment Side Slope
Long-Term (Drained) Analysis

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Fill		21	Mohr-Coulomb	0	35
Gniess Bedrock		24	Infinite strength		
Existing Fill		21	Mohr-Coulomb	0	35



Date: April 2021
Project Number: 1786659-17

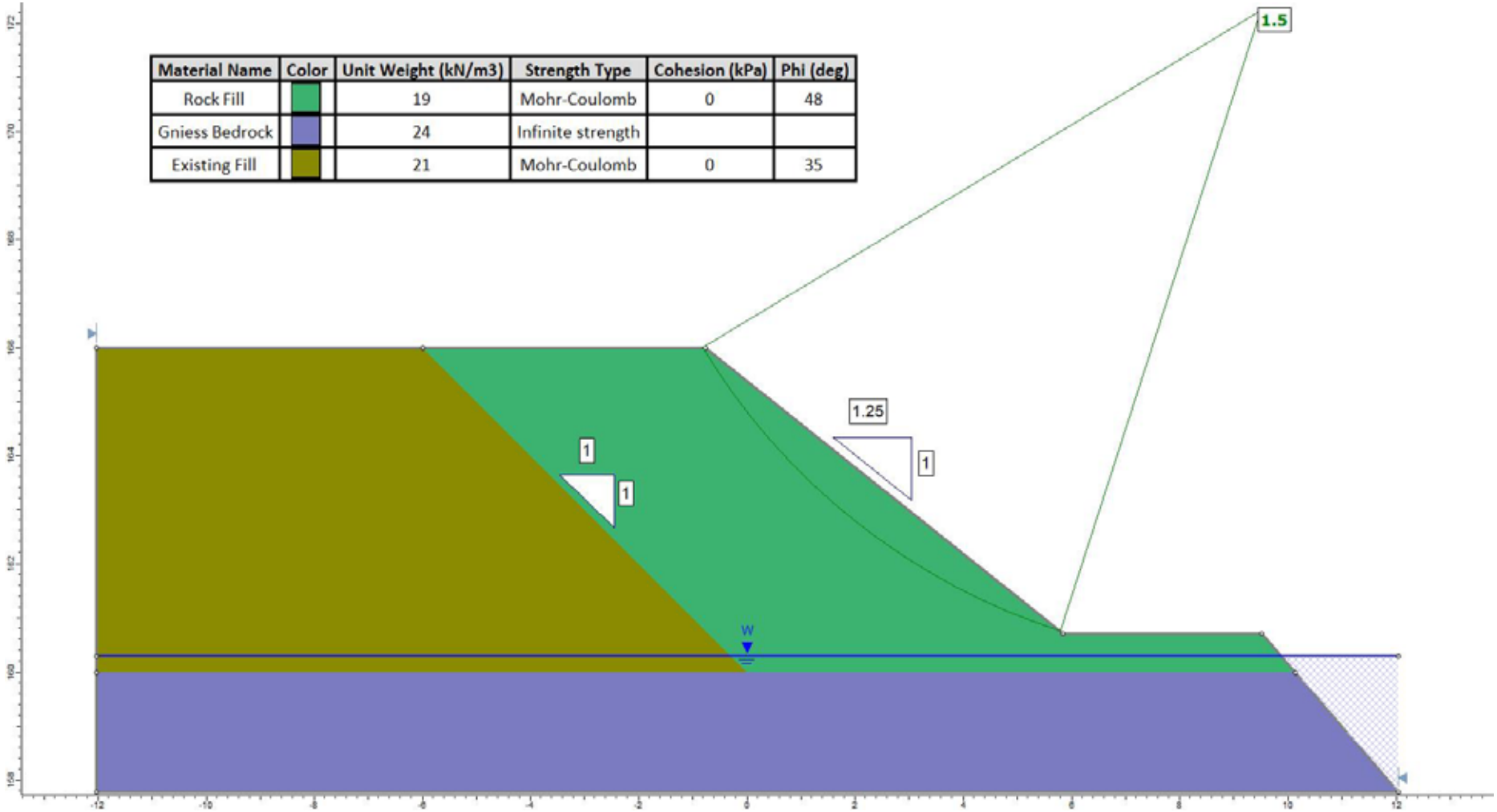
Analysis By: YS
Reviewed By: SEMP

Global Stability Analysis

Figure 2

Skootamatta River Bridge Replacement – Highway 7
Rock Fill – Embankment Side Slope
Long Term (Drained) Analysis

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Rock Fill		19	Mohr-Coulomb	0	48
Gniess Bedrock		24	Infinite strength		
Existing Fill		21	Mohr-Coulomb	0	35



Date: April 2021
Project Number: 1786659-17

Analysis By: YS
Reviewed By: SEMP

APPENDIX A

Borehole/Drillhole Records

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS
MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY COMPONENTS[1,2]

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

1. Only applicable to components not described by Primary Group Name.
2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

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

Dynamic Cone Penetration Resistance (DCPT); 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

SAMPLES

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

SOIL TESTS

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

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

COARSE-GRAINED SOILS

Compactness[1]

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

1. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
2. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

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

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

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

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

I.	**GENERAL**
π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety
II.	**STRESS AND STRAIN**
γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility
III.	**SOIL PROPERTIES**
(a)	**Index Properties**
$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

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

(a)	**Index Properties (continued)**
w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index = $(w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)
(b)	**Hydraulic Properties**
h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume
(c)	**Consolidation (one-dimensional)**
C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{\alpha(e)}$	secondary compression index
C_α	rate of secondary compression
$C_{\alpha(\varepsilon)}$	modified secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}
(d)	**Shear Strength**
τ_p, τ_r	peak and residual shear strength
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING CLASSIFICATION

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

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

ISRM Intact Rock Material Strength Classification

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

RECORD OF BOREHOLE No S1

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934688.2; E 238970.0 MTM NAD 83 ZONE 9 (LAT. 44.549206; LONG. -77.328525)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE September 23, 2020

CHECKED BY MN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
165.4	GROUND SURFACE										
0.0	ASPHALT (125 mm)										
	CONCRETE (225 mm)										
0.4	Gravelly SILTY SAND (SM) (FILL) Loose to compact Brown Moist to wet		1	SS	24		165		○		
			2	SS	11		164		○		
			3	SS	9		163		○		30 56 12 2 Non-Plastic
			4	SS	10		162		○		
161.7											
3.7	CONCRETE Cored from 3.7 m to 4.3 m		1	RC	-						
161.1											
4.3	AMPHIBOLITE (BEDROCK)		2	RC	REC 100%		161				RQD = 80%
	Bedrock cored from 4.3 m to 7.7 m. For rock coring details refer to Record of Drillhole S1.		3	RC	REC 100%		160				RQD = 81%
			4	RC	REC 100%		159				RQD = 92%
157.7							158				
7.7	END OF BOREHOLE										

NOTES:

1. Water level not recorded due to addition of water for wash boring.

2. Concrete encountered at a depth of 3.7 m below ground surface (Elev. 161.7 m) and is likely associated with existing footings.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S1

SHEET 1 OF 1

LOCATION: N 4934688.22 ;E 238969.99

DRILLING DATE: September 24, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S1.		161.70															
4	HQ Casing / NQ Coring	CONCRETE		3.69 / 161.09	1														
		Foliated AMPHIBOLITE with calcite veins; fine-grained, dark grey, very strong, fresh		4.30	2							JN,PL,RO							
5					3							JN,PL,RO JN,PL,SM JN,UN,RO						Broken Core Broken Core	UCS=106.9 MPa
												JN,ST,SM							
6												JN,UN,RO JN,IR,RO JN,UN,RO							
7				157.67	4							JN,ST,SM,SA JN,UN,SM,SA JN,UN,RO JN,UN,RO,CI, SA							
		END OF DRILLHOLE		7.72															
8		NOTE: 1. Concrete encountered at a depth of 3.7 m below ground surface (Elev. 161.7 m) and is likely associated with existing footings.																	
9																			
10																			
11																			
12																			
13																			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: IL

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RECORD OF BOREHOLE No S2

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934680.2; E 238969.5 MTM NAD 83 ZONE 9 (LAT. 44.549133; LONG. -77.328530)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE September 23, 2020

CHECKED BY MN

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
165.3	GROUND SURFACE				
0.0	ASPHALT (150 mm)				
0.2	Gravelly SILTY SAND (SM) (FILL) Dense Brown Moist	1	SS	39	
164.5					
0.8	GRAVEL (GP), some sand, trace fines (FILL) Loose to compact Brown Moist	2	SS	16	87 11 (12)
163.5					
1.8	Sandy SILT (ML) (FILL) Loose to compact Dark brown, organic inclusions Wet	3	SS	9	
162.3		4	SS	11	Non-Plastic
3.0	SILTY SAND (SM), some gravel (FILL) Compact Brown Moist	5	SS	12	13 65 17 5
161.7		6	SS	100/0.0[illegible]	
3.6	AMPHIBOLITE (BEDROCK) Amphibolite bedrock fragments recovered in split spoon 6 Bedrock cored from 3.6 m to 7.8 m. For rock coring details refer to Record of Drillhole S2.	1	RC	REC 100%	RQD = 100%
		2	RC	REC 100%	RQD = 100%
		3	RC	REC 100%	RQD = 97%
157.5					
7.8	END OF BOREHOLE				

NOTES:

1. Water level not recorded due to addition of water for wash boring.

2. Water level measured in standpipe piezometer at a depth of 4.8 m below ground surface (Elev. 160.5 m) on October 13, 2020 and at 3.4 m below ground surface (Elev. 161.9 m) on November 11, 2020.

SOIL PROFILE | STRAT PLOT | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 165, 164, 163, 162, 161, 160, 159, 158

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S2

SHEET 1 OF 1

LOCATION: N 4934680.20 ;E 238969.48

DRILLING DATE: September 22, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S2.		161.67															
	HQ Casing / NQ Coring	Foliated AMPHIBOLITE with calcite veins; fine-grained, dark grey, very strong, fresh		3.63														Broken Core	
												JN,UN,VR						UCS = 173.4 MPa	
4					1							JN,UN,RO,Ca, PC							
												JN,PL,RO,Qz, IN							
												JN,PL,K,He, PC							Bentonite
5					2							JN,PL,SM,He, SA							
6																			Sand
7					3														Screen
												JN,PL,RO							
				157.54								JN,UN,SM,PC,He							
		END OF DRILLHOLE		7.76															
8																			
9																			
10																			
11																			
12																			
13																			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: IL

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RECORD OF BOREHOLE No S3

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934695.1; E 239000.1 MTM NAD 83 ZONE 9 (LAT. 44.549270; LONG. -77.328147)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE September 22, 2020

CHECKED BY MN

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
165.5	GROUND SURFACE										
0.0	ASPHALT (100 mm)										
165.1	CONCRETE (330 mm)										
0.4	Sandy GRAVEL (GP) (FILL) Compact to loose Brown Moist						165				
			1	SS	17				○		60 29 (11)
							164				
			2	SS	6						
			3	SS	10		163				
			4	SS	10		162				
161.6			5	SS	100/0.03						
3.9	GNEISS (BEDROCK) or possible rock fill/rock slab		1	RC	REC 100%						RQD = 73%
161.0							161				
160.7	CONCRETE (300 mm)										
4.8	GNEISS (BEDROCK) Bedrock cored from 4.0 m to 8.4 m. For rock coring details refer to Record of Drillhole S3.		2	RC	REC 100%		160				RQD = 76%
			3	RC	REC 100%		159				RQD = 96%
			4	RC	REC 100%		158				RQD = 100%
157.1											
8.4	END OF BOREHOLE										

NOTES:

1. Water level not recorded due to addition of water for wash boring.

2. Concrete encountered at a depth of 4.6 m below ground surface (Elev. 160.9 m) and is likely associated with existing footings.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S3

SHEET 1 OF 1

LOCATION: N 4934695.09 ;E 239000.10

DRILLING DATE: September 21, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S3.		161.56															
4	HQ Casing / NQ Coring	Foliated Biotite-Hornblende GNEISS with quartz-calcite veins; medium-grained, dark grey, very strong, slightly weathered (possible rock fill/rock slab)		3.94	1						90	JN,UN,RO,He, ST JN,IR,VR,He, ST JN,PL,RO JN,UN,RO,Sa, PC				W1			
				160.96															
		CONCRETE		4.54								JN,UN,RO,Ca, SA							
				160.66															
5		Foliated Biotite-Hornblende GNEISS with quartz-calcite veins; medium-grained, dark grey, strong, slightly weathered to fresh		4.84	2						90	JN,UN,RO,Ca, SA JN,UN,RO JN,PL,SM JN,PL,RO JN,PL,RO JN,UN,SM JN,UN,SM JN,UN,SM JN,UN,VR							
6											90	JN,PL,RO,He, CC JN,UN,RO,Sa, IN JN,UN,SM JN,UN,SM,He, CC JN,UN,RO							
7					3						90	JN,UN,RO JN,PL,SM,PC,He							UCS=85.9 MPa
8					4														
				157.10															
		END OF DRILLHOLE NOTE: 1. Concrete encountered at a depth of 4.6 m below ground surface (Elev. 160.9 m) and is likely associated with existing footings.		8.40															
9																			
10																			
11																			
12																			
13																			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: IL

GTA-RCK 046 R:\MISSISSAUGA\SIM\CLIENTS\MTO\HWY_7_AND_HWY_38\02_DATA\GINT\1786659\HWY_7_AND_HWY_38.GPJ GAL-MISS.GDT 4/7/21

RECORD OF BOREHOLE No S4

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934705.2; E 238991.7 MTM NAD 83 ZONE 9 (LAT. 44.549360; LONG. -77.328254)

ORIGINATED BY

DIST Eastern HWY 7

BOREHOLE TYPE

COMPILED BY

DATUM Geodetic

DATE

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
160.5	GROUND SURFACE										
0.0	GNEISS (BEDROCK) Bedrock cored from 0.0 m to 2.9 m. For rock coring details refer to Record of Drillhole S4.		1	RC	REC 100%		160				RQD = 100%
			2	RC	REC 100%		159				RQD = 100%
			3	RC	REC 100%		158				RQD = 94%
157.6											
2.9	END OF BOREHOLE										

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

GTA-MTO 001 R:\MISSISSAUGA\SIM\CLIENTS\MTO\HWY_7_AND_HWY_38\02_DATA\GINT\1786659\HWY_7_AND_HWY_38.GPJ GAL-GTA.GDT 4/7/21

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S4

SHEET 1 OF 1

LOCATION: N 4934705.16 ;E 238991.66

DRILLING DATE: October 14, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Tripod

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		GROUND SURFACE		160.51															
0	HQ Casing	Foliated Biotite-Hornblende-Pyrite GNEISS with quartz-calcite veins; medium-grained, dark grey, very strong, fresh		0.00	1														
1	NQ Coring				2						90	JN,UN,SM							
											90	JN,UN,VR							
											80	JN,ST,SM,Ca,PC							
2					3						90	JN,UN,RO							UCS=124.4 MPa
											90	JN,UN,SM,Chl,SA							
				157.60								JN,IR,SM,He,SA							
3		END OF DRILLHOLE		2.91															
		NOTE:																	
		1. Bedrock exposed at ground surface, therefore, there is no borehole log for S4.																	
4																			
5																			
6																			
7																			
8																			
9																			
10																			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: IL

GTA-RCK 046 R:\MISSISSAUGA\SIM\CLIENTS\MTO\HWY_7_AND_HWY_38\02_DATA\GINT\1786659\HWY_7_AND_HWY_38.GPJ GAL-MISS.GDT 4/7/21

GOLDER

RECORD OF BOREHOLE No S5

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934674.5; E 238938.9 MTM NAD 83 ZONE 9 (LAT. 44.549079; LONG. -77.328914)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring

COMPILED BY SA

DATUM Geodetic

DATE September 24, 2020

CHECKED BY MN

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
165.2	GROUND SURFACE										
0.0	ASPHALT (150 mm)										
0.3	GRAVEL (GP), some sand (FILL) Compact Brown Moist		1	SS	23		165		○		
	SAND (SP) (FILL) Compact Brown Moist		2	SS	27		164				
163.7 / 1.5	SILT (ML), some sand Compact		3A 3B	SS	12				42.6 ○		Non-Plastic
163.2 / 2.0	Dark brown, organic inclusions Wet										
	Rock fragments in tip of spoon										
	END OF BOREHOLE CASING REFUSAL										

NOTE:

1. Water level not recorded due to addition of water for wash boring.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No S6

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934670.5; E 238948.8 MTM NAD 83 ZONE 9 (LAT. 44.549044; LONG. -77.328789)

ORIGINATED BY

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE September 23, 2020

CHECKED BY MN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
165.2	GROUND SURFACE										
0.0	ASPHALT (150 mm)										
	GRAVEL (GP), some sand (FILL)		1	SS	100/0.07		165		○		
0.4	Very dense Brown Moist										
	ROCK (FILL)						164				
163.6	70 mm to 150 mm cobbles recovered in casing										
1.6	GNEISS (BEDROCK)		1	RC	REC 100%		163				RQD = 100%
	Bedrock cored from 1.6 m to 5.0 m. For rock coring details refer to Record of Drillhole S6.		2	RC	REC 100%		162				RQD = 99%
			3	RC	REC 100%		161				RQD = 95%
160.2											
5.0	END OF BOREHOLE										

NOTE:

1. Water level not recorded due to addition of water for wash boring.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S6

SHEET 1 OF 1

LOCATION: N 4934670.51 ;E 238948.83

DRILLING DATE: September 23, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S6.		163.62															
	HQ Casing	Foliated Biotite-Hornblende GNEISS with quartz-calcite veins and hematite layers; medium-grained, dark grey, very strong, slightly weathered		1.56															
2	NQ Coring				1						90	JN,UN,SM							
3					2						90	JN,UN,SM,He, SA							
											90	JN,UN,SM,He, SA							
											90	JN,UN,RO,He, SA							
											90	JN,PL,RO,He, SA							
											90	JN,PL,RO						Broken Core	
4					3						90	JN,UN,RO							
											90	JN,UN,RO							
				160.22							90	JN,UN,RO							
5		END OF DRILLHOLE		4.96															
6																			
7																			
8																			
9																			
10																			
11																			

DEPTH SCALE

1 : 50

GOLDER

LOGGED: YS

CHECKED: IL

GTA-RCK 046 R:\MISSISSAUGA\SIM\CLIENTS\MTO\HWY_7_AND_HWY_38\02_DATA\GINT\1786659\HWY_7_AND_HWY_38.GPJ GAL-MISS.GDT 4/7/21

GOLDER

RECORD OF BOREHOLE No S7

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934713.0; E 239019.4 MTM NAD 83 ZONE 9 (LAT. 44.549433; LONG. -77.327907)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring

COMPILED BY SA

DATUM Geodetic

DATE September 24, 2020

CHECKED BY MN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR SA SI CL
165.7	GROUND SURFACE										
0.0	ASPHALT (100 mm)										
0.3	Gravelly SAND (SP) (FILL) Compact Brown Moist		1	SS	25		165		○		0 90 10 0
164.9											
0.8	SAND (SP), trace fines (FILL) Compact Brown Moist		2	SS	5						
164.2											
1.6	SILTY SAND (SM) (FILL) Loose Brown, organic inclusions, wood fragments Moist - Rock chips and organic inclusions at a depth of 1.5 m		3	SS	100/0.05						
	Rock fragments recovered in spoon										
	END OF BOREHOLE CASING REFUSAL										

NOTE:

1. Water level not recorded due to addition of water for wash boring.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

GOLDER

RECORD OF BOREHOLE No S8

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934710.5; E 239035.6 MTM NAD 83 ZONE 9 (LAT. 44.549412; LONG. -77.327701)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring

COMPILED BY SA

DATUM Geodetic

DATE September 25, 2020

CHECKED BY MN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
165.8	GROUND SURFACE										
0.0	Gravelly SILTY SAND (SM) (FILL) Dense Reddish brown Moist		1	SS	40						
165.1 / 0.7	Gravelly SILTY SAND (SM) Very dense Reddish brown Moist		2	SS	98		165				Non-Plastic
164.6 / 1.3	Rock fragments in tip of spoon										
	END OF BOREHOLE CASING REFUSAL										

NOTE:

1. Water level not recorded due to addition of water for wash boring.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No S9

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934693.9; E 238955.6 MTM NAD 83 ZONE 9 (LAT. 44.549256; LONG. -77.328706)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE October 9, 2020

CHECKED BY MN

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
164.0	GROUND SURFACE										
0.0 / 0.1	TOPSOIL (175 mm)										
163.3 / 0.7	SAND (SP), trace gravel (FILL) Very loose Brown Moist		1	SS	2				○		
162.3	GRAVEL (GP) (FILL) Loose Brown Moist		2	SS	8		163				no recovery
			3	SS	100/0.03						
1.7	AMPHIBOLITE (BEDROCK)						162				
	Bedrock cored from 1.7 m to 4.6 m. For rock coring details refer to Record of Drillhole S9.		1	RC	REC 100%		161				RQD = 92%
159.4			2	RC	REC 100%		160				RQD = 100%
4.6	END OF BOREHOLE										

NOTES:

1. Water level not recorded due to addition of water for wash boring.

2. Rock fragments recovered in split spoon sample 3.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S9

SHEET 1 OF 1

LOCATION: N 4934693.95 ;E 238955.64

DRILLING DATE: October 9, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S9.		162.29															
2	HQ Casing	Foliated AMPHIBOLITE with calcite veins; fine-grained, dark grey, very strong, fresh		1.70	1						0–30	JN,IR,VR JN,PL,SM,Ch, SA JN,IR,RO JN,UN,SM							UCS=166.9 MPa
3	NQ Coring																		
4					2						60–90	JN,IR,RO JN,UN,SM,Ca, IN							
				159.38															
		END OF DRILLHOLE		4.61															
5																			
6																			
7																			
8																			
9																			
10																			
11																			

DEPTH SCALE

1 : 50

GOLDER

LOGGED: YS

CHECKED: IL

GTA-RCK 046 R:\MISSISSAUGA\SIM\CLIENTS\MTO\HWY_7_AND_HWY_38\02_DATA\GINT\1786659\HWY_7_AND_HWY_38.GPJ GAL-MISS.GDT 4/7/21

GOLDER

RECORD OF BOREHOLE No S10

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934719.1; E 239004.9 MTM NAD 83 ZONE 9 (LAT. 44.549487; LONG. -77.328090)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE October 1, 2020

CHECKED BY MN

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
164.7	GROUND SURFACE										
0.0	TOPSOIL (50 mm)										
0.1	SILTY SAND (SM) (FILL) Very loose to compact Brown, organic inclusions Moist		1	SS	5						
			2	SS	22		164		○		
			3	SS	3		163				
			4	SS	23		162		○		0 71 26 3
			5	SS	7				○		
160.9							161				
3.8	COBBLES and BOULDERS		6	SS	100/0.07						
160.3	Bedrock fragments recovered in casing										
4.4	AMPHIBOLITE (BEDROCK) Bedrock cored from 4.4 m to 7.4 m. For rock coring details refer to Record of Drillhole S10.		1	RC	REC 100%		160				RQD = 67%
							159				
			2	RC	REC 100%		158				RQD = 91%
157.3											
7.4	END OF BOREHOLE										

NOTES:

1. Water level not recorded due to addition of water for wash boring.

2. Broken rock fragments recovered in SS 6.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S10

SHEET 1 OF 1

LOCATION: N 4934719.13 ;E 239004.86

DRILLING DATE: October 1, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S10.		160.33															
	HQ Casing, NQ Coring	Foliated AMPHIBOLITE with calcite veins; fine-grained, dark grey, very strong, slightly weathered to fresh		4.39															
5					1							JN,PL,SM,Ca, IN JN,PL,SM,Ca, IN JN,UN,SM						Broken Core Broken Core	
												JN,UN,RO							UCS=186.5 MPa
6												JN,UN,RO,Cl, SA JN,UN,RO,He, PC							
7					2							JN,UN,RO							
				157.28								JN,UN,RO,Ca, IN JN,UN,RO,Ca, IN							
		END OF DRILLHOLE		7.44															
8																			
9																			
10																			
11																			
12																			
13																			
14																			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: IL

GTA-RCK 046 R:\MISSISSAUGA\SIM\CLIENTS\MTO\HWY_7_AND_HWY_38\02_DATA\GINT\1786659\HWY_7_AND_HWY_38.GPJ GAL-MISS.GDT 4/7/21

GOLDER

RECORD OF BOREHOLE No S11

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934662.1; E 238898.1 MTM NAD 83 ZONE 9 (LAT. 44.548964; LONG. -77.329427)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring

COMPILED BY SA

DATUM Geodetic

DATE September 24, 2020

CHECKED BY MN

SOIL PROFILE			SAMPLES														
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
165.3	GROUND SURFACE													
0.0	TOPSOIL (150 mm)													
0.2	Gravelly SILTY SAND (SM) (FILL) Loose Brown Moist		1	SS	5		165		○		30	55	14	1
164.6 / 0.7	Sandy GRAVEL (GP), trace silt (FILL) Dense Black, organic inclusions Moist		2	SS	31		164							
163.8 / 1.6	Rock fragments recorded in spoon		3	SS	100/0.07									

END OF BOREHOLE
CASING REFUSAL

NOTE:

1. Water level not recorded due to addition of water for wash boring.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

GOLDER

RECORD OF BOREHOLE No S12

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934738.3; E 239053.8 MTM NAD 83 ZONE 9 (LAT. 44.549664; LONG. -77.327477)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers

COMPILED BY SA

DATUM Geodetic

DATE September 25, 2020

CHECKED BY MN

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
165.4	GROUND SURFACE										
0.0	TOPSOIL (680 mm)		1	SS	4		165				
164.7											
0.7	SAND (SP) (FILL) Very loose Brown Moist		2	SS	2		164		○		
			3	SS	1						
162.7			4A 4B	SS	2		163		94 ○	OC=23.6%	3 85 (12)
3.0	Organic SILTY SAND, trace gravel (SM) (FILL) Very loose Black Moist		5	SS	5		162		○		
161.7	SILTY SAND (SM) (FILL) Loose Brown, organic inclusions Moist		6	SS	14	▽	161				
3.7	SILTY SAND (SM) Compact Brown Wet		7	SS	15				○		Non-Plastic 0 85 15 0
159.9			8	SS	100/0.03		160				
5.5	Rock fragments recorded in spoon END OF BOREHOLE AUGER AND SPLIT-SPOON REFUSAL										

NOTE:

1. Water level encountered at a depth of 3.8 m below ground surface (Elev. 161.6 m) upon completion of drilling.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

GOLDER

RECORD OF BOREHOLE No S13

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934670.9; E 238973.6 MTM NAD 83 ZONE 9 (LAT. 44.549050; LONG. -77.328477)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ2 Coring

COMPILED BY SA

DATUM Geodetic

DATE October 21, 2020

CHECKED BY MN

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
163.1	GROUND SURFACE										
0.0	TOPSOIL (100 mm)										
0.1	Sandy SILT (ML), some gravel Loose Brown, organic inclusions Moist		1	SS	4						Non-Plastic
162.3 0.8	GNEISS (BEDROCK)		1	RC	REC 100%		162				RQD = 100%
	Bedrock cored from 0.8 m to 2.5 m.		2	RC	REC 100%						RQD = 58%
	For rock coring details refer to Record of Drillhole S13.		3	RC	REC 100%		161				RQD = 82%
160.6 2.5	END OF BOREHOLE										

NOTE:

1. Water level not recorded due to addition of water for wash boring.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S13

SHEET 1 OF 1

LOCATION: N 4934670.93 ;E 238973.59

DRILLING DATE: October 21, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S13.		162.30															
1	HQ Casing	Foliated Biotite-Hornblende-Pyrite GNEISS with brecciated quartz veins and calcite-hematite veins; medium-grained, dark grey, very strong, slightly weathered		0.76	1														
	NQ2 Coring				2						0	JN,UN,SM,He, SA							
											30	JN,UN,SM,He, SA							
												JN,PL,SM,He, SA							
											0	JN,UN,RO,He, SA							
											30	JN,PL,SM,He, SA							
2					3													Broken Core UCS=103.9 MPa	
				160.58							90	JN,UN,RO							
		END OF DRILLHOLE		2.48															
3																			
4																			
5																			
6																			
7																			
8																			
9																			
10																			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: IL

GTA-RCK 046 R:\MISSISSAUGA\SIM\CLIENTS\MTO\HWY_7_AND_HWY_38\02_DATA\GINT\1786659\HWY_7_AND_HWY_38.GPJ GAL-MISS.GDT 4/7/21

GOLDER

RECORD OF BOREHOLE No S14

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934691.9; E 239020.6 MTM NAD 83 ZONE 9 (LAT. 44.549243; LONG. -77.327888)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE Portable Tripod - HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE October 16, 2020

CHECKED BY MN

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
163.0	GROUND SURFACE										
0.0	TOPSOIL (50 mm)										
162.6	SILTY SAND (SM), trace gravel Loose Brown, organic inclusions Moist		1	SS	108						
0.4	AMPHIBOLITE (BEDROCK) Bedrock cored from 0.4 m to 3.5 m. For rock coring details refer to Record of Drillhole S14.		1	RC	REC 100%		162				RQD = 100%
			2	RC	REC 49%		161				RQD = 43%
			3	RC	REC 100%		160				RQD = 85%
159.5											
3.5	END OF BOREHOLE										

NOTES:

1. Spoon bouncing at 0.4 m depth. SPT N value not representative of the compactness condition of the silty sand layer as a result of the underlying bedrock.

2. Water level not recorded due to addition of water for wash boring.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S14

SHEET 1 OF 1

LOCATION: N 4934691.91 ;E 239020.60

DRILLING DATE: October 16, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Tripod

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S14.		162.56															
1	HQ Casing; NQ/NQ2 Coring	Foliated AMPHIBOLITE with calcite veins; medium-grained, dark grey, very strong, slightly weathered		0.44	1														UCS=136.7 MPa
2					2						30	JN,IR,SM,He,SA JN,ST,RO,He,SA							Lost Core
3				159.52	3						30	FLT,PL,K,Cl,IN							
		END OF DRILLHOLE		3.48															
4																			
5																			
6																			
7																			
8																			
9																			
10																			

DEPTH SCALE

1 : 50

GOLDER

LOGGED: YS

CHECKED: IL

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GOLDER

RECORD OF BOREHOLE No S15

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934638.6; E 238896.0 MTM NAD 83 ZONE 9 (LAT. 44.548753; LONG. -77.329449)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers

COMPILED BY SA

DATUM Geodetic

DATE September 25, 2020

CHECKED BY MN

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
164.8	GROUND SURFACE													
0.0	Gravelly SILTY SAND (SM) (FILL) Loose Brown Moist		1	SS	9									
164.1 / 0.7	Sandy GRAVEL (GP), trace silt (FILL) Compact Grey Moist		2	SS	15		164		○		66	25	9	1
163.3 / 1.6	Rock fragments recovered in split spoon		3	SS	100/0.05									

END OF BOREHOLE
AUGER AND SPLIT-SPOON REFUSAL

NOTE:

1. Borehole dry upon completion of drilling.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

GOLDER

RECORD OF BOREHOLE No S16

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934737.1; E 239093.9 MTM NAD 83 ZONE 9 (LAT. 44.549657; LONG. -77.326971)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers

COMPILED BY SA

DATUM Geodetic

DATE September 25, 2020

CHECKED BY MN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
166.1	GROUND SURFACE										
0.0	TOPSOIL (125 mm)		1	SS	100/0.13						
0.3	Rock fragments recovered from lower part of spoon										
	END OF BOREHOLE AUGER AND SPLIT-SPOON REFUSAL										

NOTE:

1. Borehole dry upon completion of drilling.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

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RECORD OF BOREHOLE No S17

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934681.4; E 238971.6 MTM NAD 83 ZONE 9 (LAT. 44.549144; LONG. -77.328504)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE October 5, 2020

CHECKED BY MN

SOIL PROFILE ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
165.4	GROUND SURFACE										
0.0	ASPHALT (152 mm)										
0.2	Gravelly SILTY SAND (SM) (FILL) Compact to dense Brown Moist		1	SS	36		165		○		32 53 13 3 Non-Plastic
			2	SS	18				○		
163.9							164				
1.5	GRAVEL (GP), trace sand (FILL) Loose Brown Wet		3	SS	7						no recovery
			4	SS	10		163				
			5	SS	9		162				no recovery
161.3			6	SS	100/0.13						
4.1	GNEISS AND AMPHIBOLITE (BEDROCK)		1	RC	REC 100%		161				RQD = 100%
	Gneiss and amphibolite bedrock fragments recovered in lower portion of split spoon 6. Bedrock cored from 4.2 m to 7.7 m. For rock coring details refer to Record of Drillhole S17.		2	RC	REC 100%		160				RQD = 93%
			3	RC	REC 100%		159				RQD = 94%
157.8							158				
7.7	END OF BOREHOLE										

NOTE:

1. Water level not recorded due to addition of water for wash boring.

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

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S17

SHEET 1 OF 1

LOCATION: N 4934681.41 ;E 238971.58

DRILLING DATE: October 14, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S17.		161.28															
	HQ Casing	Foliated Biotite-Hornblende-Pyrite GNEISS with quartz veins; medium-grained, dark grey, very strong, moderately weathered		4.09	1							JN,UN,RO							Broken Core
				160.60								JN,UN,RO,He,SA							
5	NQ Coring	Foliated AMPHIBOLITE with quartz-calcite veins; fine-grained, dark grey, very strong, slightly weathered to fresh		4.77	2							JN,UN,RO,He,PC JN,UN,RO,He,SA							
												JN,UN,RO,Cl,SA							
												JN,PL,SM,Cl,SA							
6												JN,IR,VR,Sa,PC							
												JN,PL,SM							
7					3							JN,PL,RO							
												JN,PL,RO							UCS=220.1 MPa
				157.72								JN,UN,RO JN,UN,RO							
		END OF DRILLHOLE		7.65															
8																			
9																			
10																			
11																			
12																			
13																			
14																			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: IL

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GOLDER

RECORD OF BOREHOLE No S18

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934686.8; E 238966.6 MTM NAD 83 ZONE 9 (LAT. 44.549192; LONG. -77.328568)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE October 5, 2020

CHECKED BY MN

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
165.4	GROUND SURFACE										
0.0	ASPHALT (150 mm)										
0.2	Gravelly SILTY SAND (SM) (FILL) Compact Brown Moist		1	SS	30		165				
			2	SS	12		164		○		35 49 13 3
			3	SS	11				○		
163.2											
2.2	GRAVEL (GP) (FILL) Loose Black Wet		4	SS	6		163				
162.4											
	ROCK (FILL)										
3.1	50 mm cobbles recovered in casing										
	AMPHIBOLITE (BEDROCK) Bedrock cored from 3.1 m to 6.3 m. For rock coring details refer to Record of Drillhole S18.		1	RC	REC 100%		162 / 161				RQD = 100%
			2	RC	REC 100%		160				RQD = 100%
159.1											
6.3	END OF BOREHOLE										

NOTE:

1. Water level not recorded due to addition of water for wash boring.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S18

SHEET 1 OF 1

LOCATION: N 4934686.79 ;E 238966.56

DRILLING DATE: October 5, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S18.		162.28															
4	HQ Casing / NQ Coring	Foliated AMPHIBOLITE with calcite veins, fine-grained, dark grey, very strong, fresh		3.12	1							JN,UN,RO							UCS=188.7 MPa
												JN,IR,RO,He,SA							
												JN,PL,RO,He,SA							
5					2							JN,UN,SM							
												JN,ST,RO,He,SA							
6												JN,ST,RO,He,SA							
				159.14								JN,PL,K,Ca,IN							
		END OF DRILLHOLE		6.26															
7																			
8																			
9																			
10																			
11																			
12																			
13																			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: IL

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GOLDER

RECORD OF BOREHOLE No S20

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934696.8; E 239003.4 MTM NAD 83 ZONE 9 (LAT. 44.549286; LONG. -77.328105)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE October 13, 2020

CHECKED BY MN

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL	REMARKS & GRAIN SIZE DISTRIBUTION (%)
165.5	GROUND SURFACE								
0.0	ASPHALT (152 mm)								
0.2	Gravelly SILTY SAND (SM) (FILL) Dense Brown Moist	1	SS	35					
164.7 / 0.8	Sandy GRAVEL (GP), trace fines (FILL) Loose Brown Wet	2	SS	10	61	34		(5)	
		3	SS	8					
		4	SS	7					
162.5 / 3.0	SILTY SAND (SM), trace gravel Loose Brown Moist	5	SS	6	9	57	31	3	
161.8 / 3.7	GNEISS (BEDROCK) Bedrock cored from 3.7 m to 7.2 m. For rock coring details refer to Record of Drillhole S20.	1	RC	REC 100%					RQD = 77%
		2	RC	REC 78%					RQD = 52%
		3	RC	REC 100%					RQD = 81%
158.3 / 7.2	END OF BOREHOLE								

NOTE:

1. Water level not recorded due to addition of water for wash boring.

ELEVATION SCALE: 165, 164, 163, 162, 161, 160, 159

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

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

○ 3% STRAIN AT FAILURE

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PROJECT: 1786659-17

RECORD OF DRILLHOLE: S20

SHEET 1 OF 1

LOCATION: N 4934696.82 ;E 239003.42

DRILLING DATE: October 13, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S20.		161.81															
4	HQ Casing / NQ Coring	Foliated Biotite-Hornblende GNEISS with calcite veins; medium-grained, dark grey, very strong, moderately weathered to fresh		3.73	1							JN,UN,SM,He,SA JN,UN,RO,He+Mu,IN JN,PL,RO,Cl,SA JN,PL,RO,He,SA JN,UN,RO,He,SA							Broken Core
5					2							JN,UN,RO,He,SA JN,UN,RO,He,SA JN,UN,RO,He,SA JN,UN,RO,He,SA JN,UN,RO,He,SA JN,UN,RO,Ca,PC JN,UN,RO,He,SA JN,UN,RO,He,SA JN,UN,RO,He,SA JN,UN,RO,He,SA JN,UN,RO,Cl,PC							Lost Core Broken Core	
6					3							JN,IR,RO JN,PL,RO,He,SA JN,PL,SM,Ca,PC JN,PL,RO								UCS=145.2 MPa
7				158.35								JN,ST,RO,He,PC JN,UN,RO,He,PC							
		END OF DRILLHOLE		7.19															
8																			
9																			
10																			
11																			
12																			
13																			

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: IL

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RECORD OF BOREHOLE No S21

SHEET 1 OF 1

METRIC

PROJECT 1786659-17

4077-14-00

LOCATION N 4934701.3; E 239000.6 MTM NAD 83 ZONE 9 (LAT. 44.549326; LONG. -77.328141)

ORIGINATED BY YS

DIST Eastern HWY 7

BOREHOLE TYPE HQ Casing with Wash Boring and NQ Coring

COMPILED BY SA

DATUM Geodetic

DATE October 5, 2020

CHECKED BY MN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
165.6	GROUND SURFACE										
0.0	ASPHALT (178 mm)										
0.2	SILTY SAND (SM), some gravel (FILL) Compact to dense Brown Moist		1	SS	48		165				
			2	SS	25				○		18 66 14 2
164.1											
1.5	Sandy GRAVEL (GP) (FILL) Loose Brown Moist		3	SS	9		164				
163.3											
2.3	ROCK (FILL)										
162.9							163				
2.7	50 mm cobbles recovered in casing		1	RC	REC 70%						RQD = 61%
	GNEISS (BEDROCK)										
	Bedrock cored from 2.7 m to 6.2 m. For rock coring details refer to Record of Drillhole S21.		2	RC	REC 100%		162				RQD = 76%
							161				
			3	RC	REC 100%		160				RQD = 97%
159.4											
6.2	END OF BOREHOLE										

NOTE:

1. Water level not recorded due to addition of water for wash boring.

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 1786659-17

RECORD OF DRILLHOLE: S21

SHEET 1 OF 1

LOCATION: N 4934701.27 ;E 239000.62

DRILLING DATE: October 5, 2020

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-45

DRILLING CONTRACTOR: Marathon

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jcon	WEATHERING INDEX	Diametral Point Load Index (MPa)	FEATURES	PIEZOMETER
		Continued from Record of Borehole S21.		162.85															
3	HQ Casing	Foliated Biotite-Hornblende GNEISS with quartz-calcite veins and hematite layers; medium-grained, dark grey, very strong, slightly weathered to fresh		2.72	1							JN,UN,RO						Lost Core Broken Core	
												JN,PL,SM,He,SA JN,PL,RO,He,IN							UCS=248.6 MPa
												JN,UN,RO							
4	NQ Coring				2							JN,UN,RO							
												JN,UN,RO,HE,PC							
												JN,UN,RO,CI,PC							
												JN,PL,K,Ca,IN							
5												JN,PL,K,Ca,IN							
					3							JN,PL,SM JN,UN,RO JN,UN,RO							
6				159.35															
		END OF DRILLHOLE		6.22															
7																			
8																			
9																			
10																			
11																			
12																			

DEPTH SCALE

1 : 50

GOLDER

LOGGED: YS

CHECKED: IL

GTA-RCK 046 R:\MISSISSAUGA\SIM\CLIENTS\MTO\HWY_7_AND_HWY_38\02_DATA\GINT\1786659\HWY_7_AND_HWY_38.GPJ GAL-MISS.GDT 4/7/21

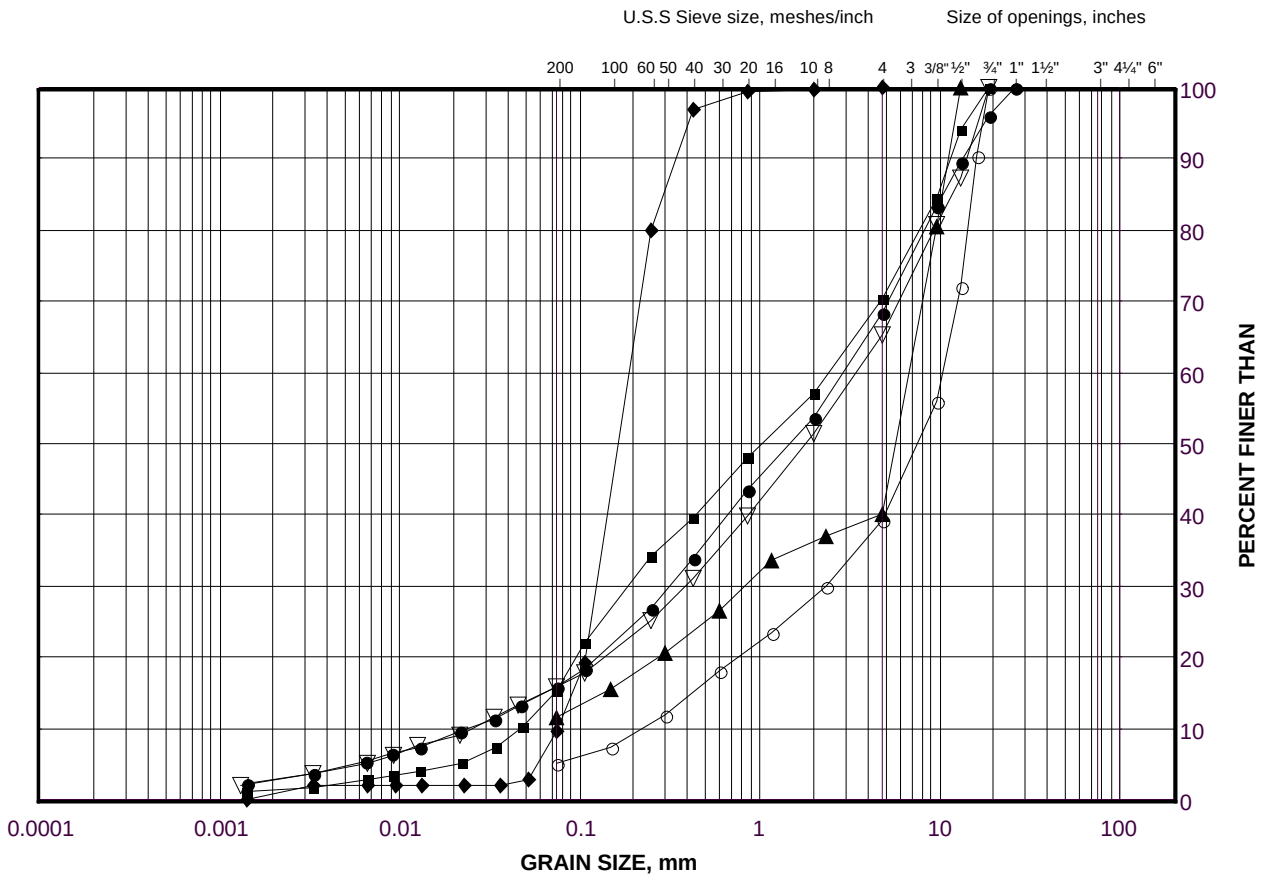
APPENDIX B

Laboratory Test Results and Bedrock Core Photographs

GRAIN SIZE DISTRIBUTION

SILTY SAND (SM) to SAND (SP) to GRAVEL (GP) (FILL)

FIGURE B1a



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S17	1	164.9
■	S11	1	165.0
◆	S7	1	165.2
▲	S3	1	164.5
▽	S18	2	164.4
❍	S20	2	164.5

Project Number: 1786659-17

Checked By: MN

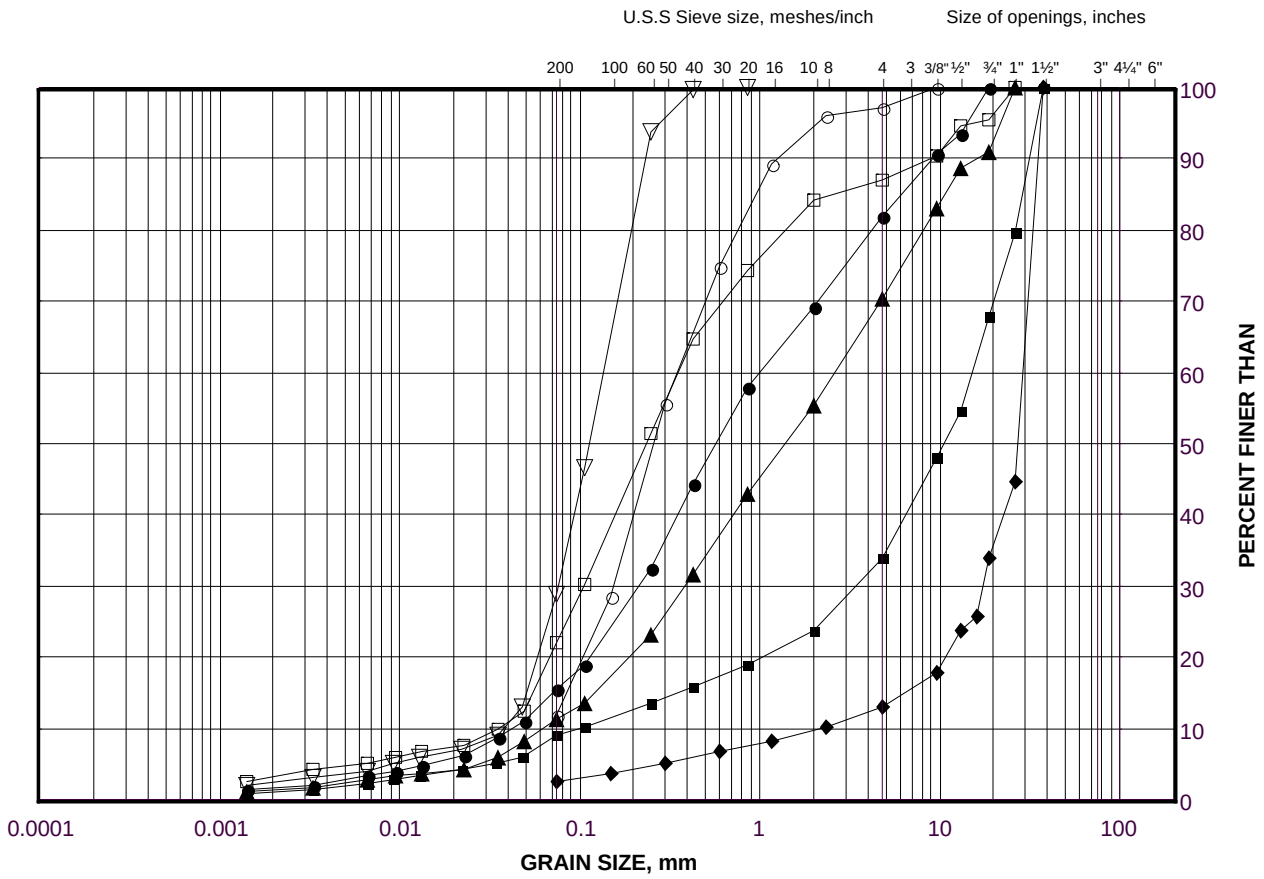
Golder Associates

Date: 08-Apr-21

GRAIN SIZE DISTRIBUTION

SILTY SAND (SM) to SAND (SP) to GRAVEL (GP) (FILL)

FIGURE B1b



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S21	2	164.5
■	S15	2	163.7
◆	S2	2	164.2
▲	S1	3	162.8
▽	S10	4	162.1
○	S12	4B	162.5
□	S2	5	162.0

Project Number: 1786659-17

Checked By: MN

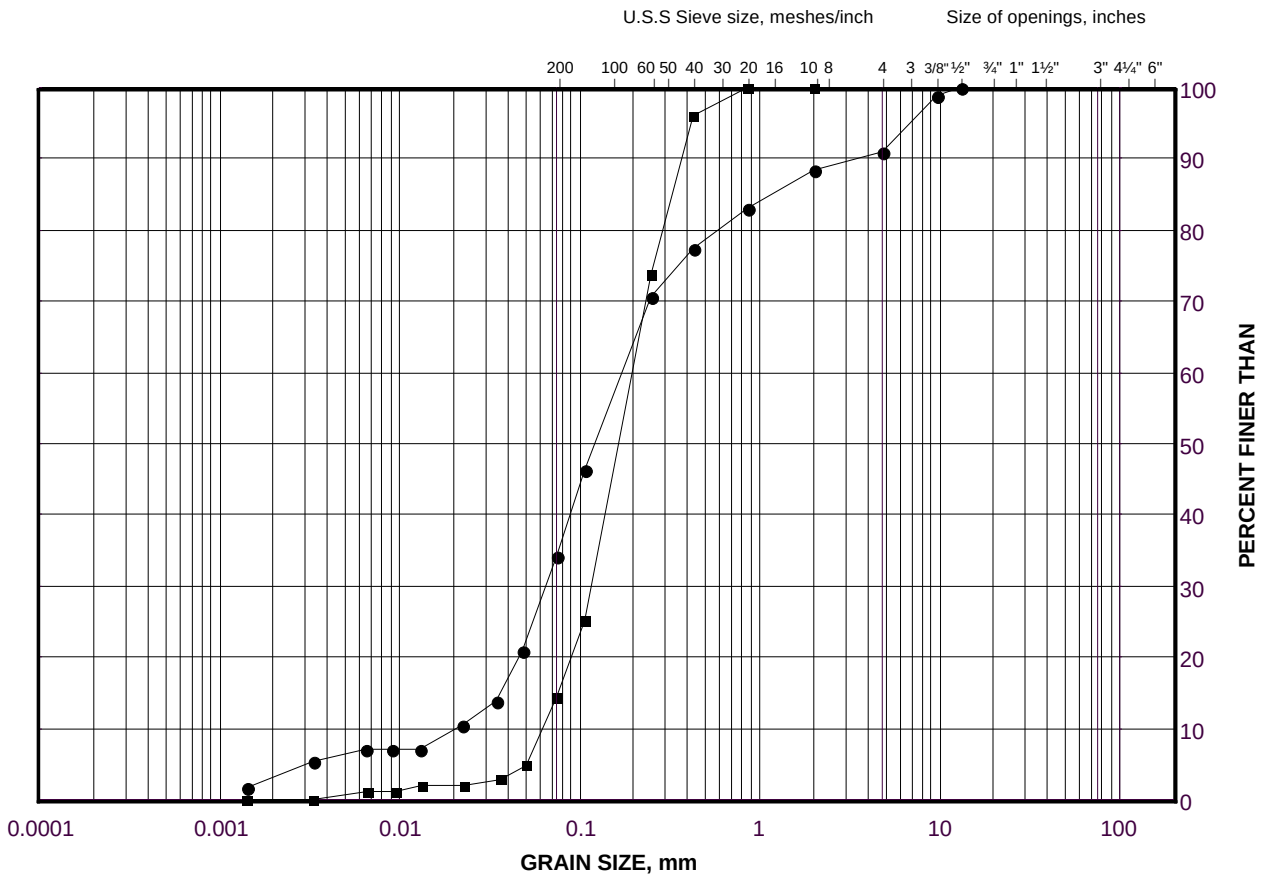
Golder Associates

Date: 08-Apr-21

GRAIN SIZE DISTRIBUTION

SILTY SAND (SM)

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	S20	5	162.1
■	S12	7	160.5

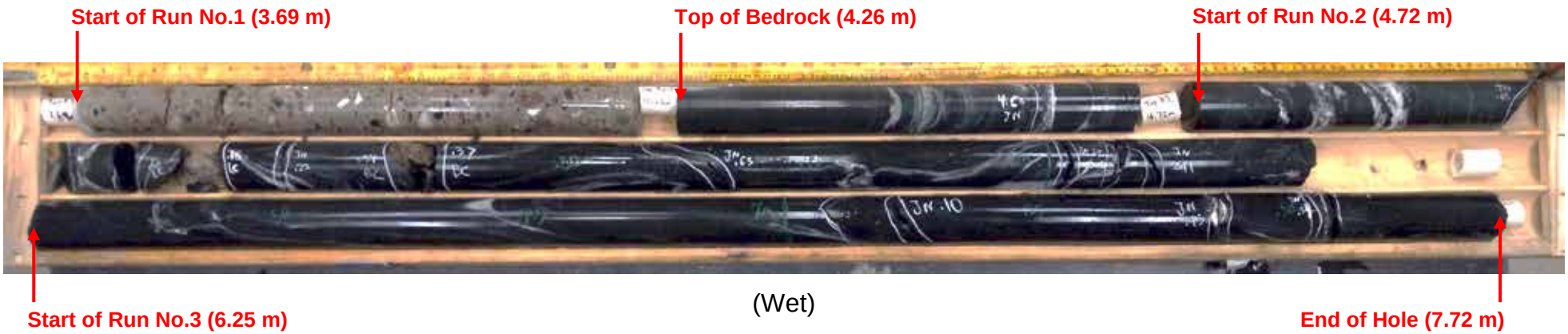
Project Number: 1786659-17

Checked By: MN

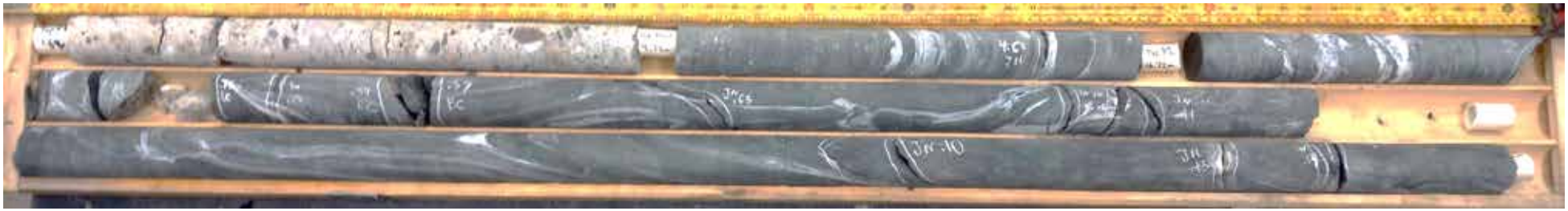
Golder Associates

Date: 08-Apr-21

Drillhole S1



(Wet)



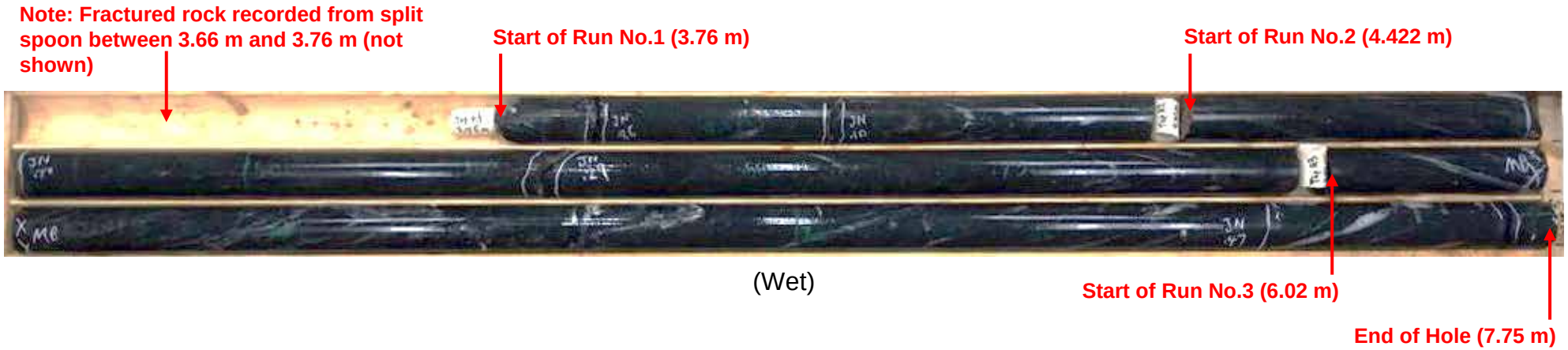
(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S1 (3.69 m – 7.72 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B3**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S2



(Wet)

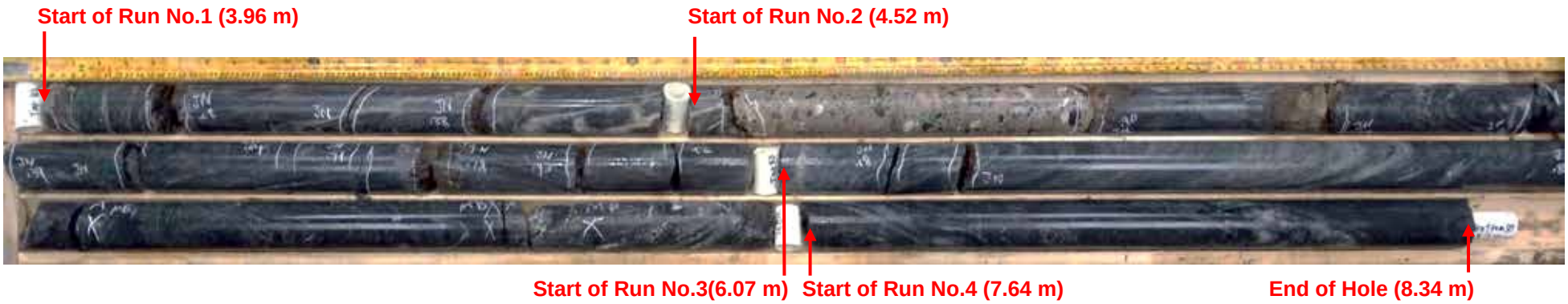
(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

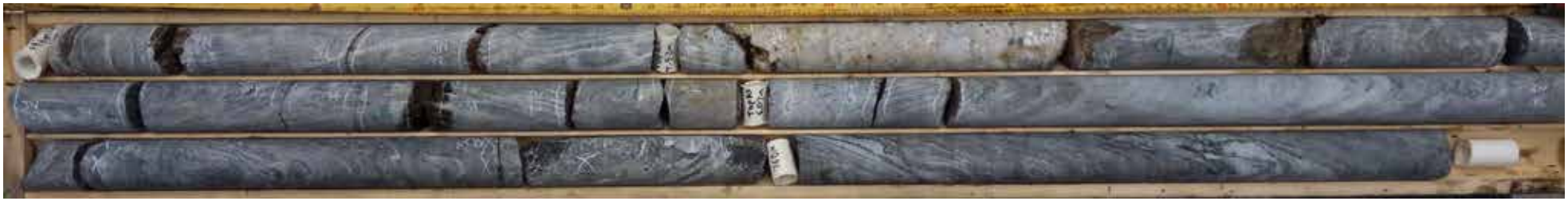
Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S2 (3.76 m – 7.75 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B4**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S3



(Wet)



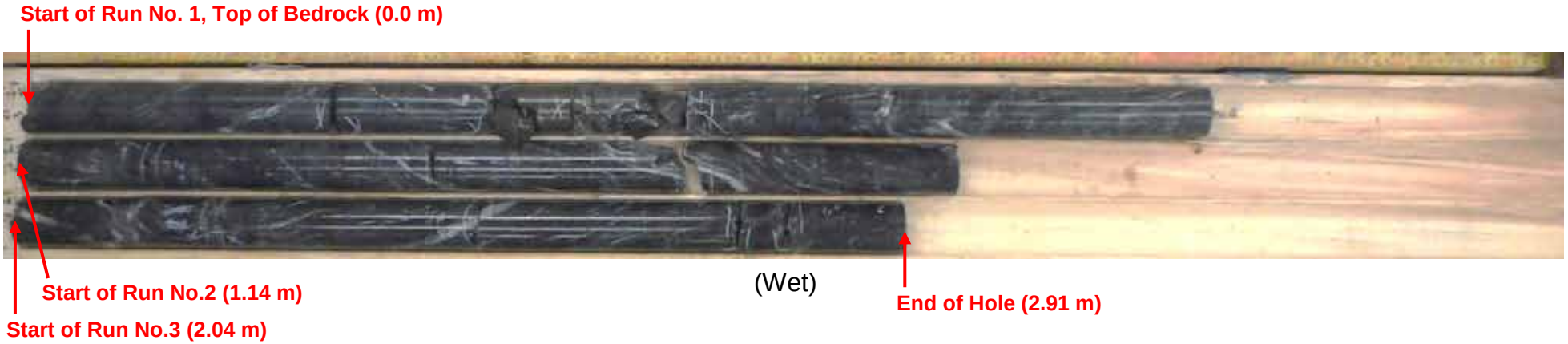
(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

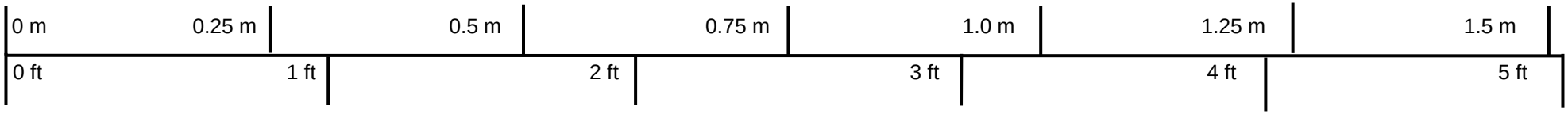
PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S3 (3.96 m – 8.34 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B5**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S4



(Wet)

(Dry)



Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S4 (0.0 m – 2.91 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B6**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S6

(Wet)

(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S6 (1.55 m – 4.95 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B7**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S9

Start of Run 1 - Top of Bedrock (1.70m)

Start of Run No.2 (3.20 m)

(Wet)

End of Hole (4.61 m)

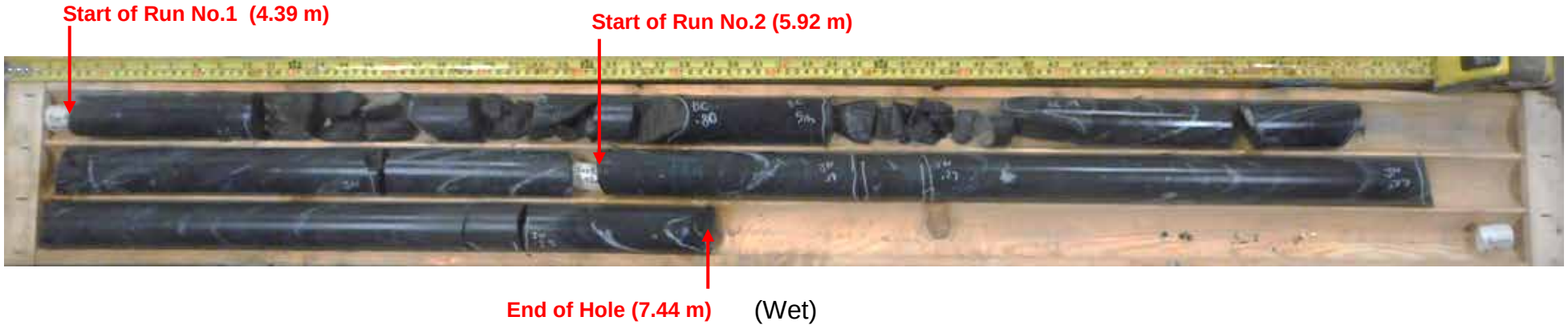
(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S9 (1.70 m – 4.61 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B8**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S10



(Wet)

(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S10 (4.39 m – 7.44 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B9**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Borehole S13

(Wet)

(Dry)

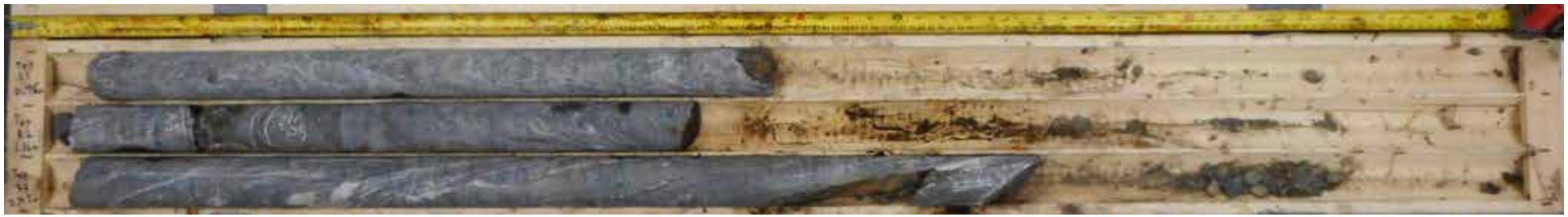
0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S13 (0.76 m – 2.48 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B10**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S14

(Wet)



(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S14 (0.46 m – 3.45 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B11**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S17

(Wet)

(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S17 (3.96 m – 7.65 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B12**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S18

(Wet)

(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S18 (3.12 m – 6.25 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B13**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S20

(Wet)

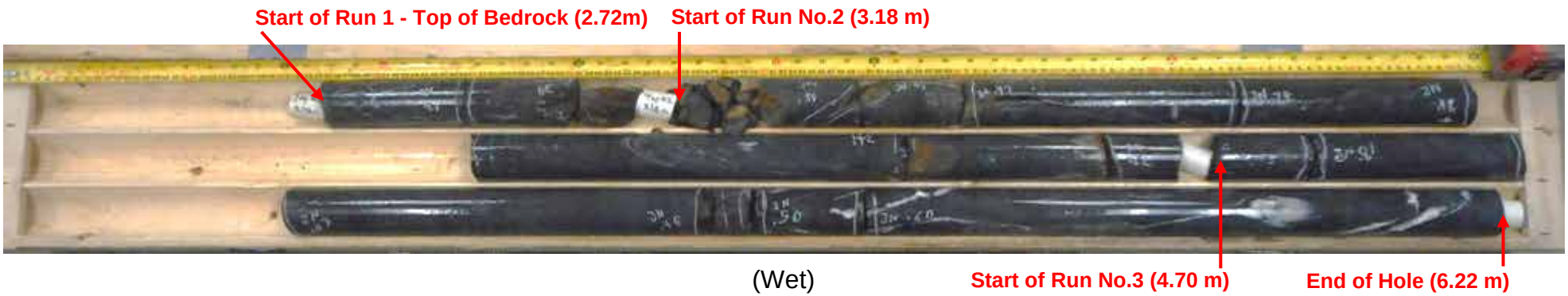
(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S20 (3.73 m – 7.19 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B14**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

Drillhole S21



(Wet)

(Dry)

0 m	0.25 m	0.5 m	0.75 m	1.0 m	1.25 m	1.5 m
0 ft	1 ft	2 ft	3 ft	4 ft	5 ft	

Scale

PROJECT	**Skootamatta River Bridge GWP 4077-14-00, WP 4091-14-01 Site No. 11X-0076/B0**				
TITLE	**Bedrock Core Photographs Drillhole S21 (2.72 m – 6.22 m)**				
GOLDER	PROJECT No. 1786659 (17)			FILE No.	
	DRAFT	IR	APR 2021	SCALE AS SHOWN	VER. 1.
	CADD	--		**FIGURE B15**	
	CHECK	MN	APR 2021		
	REVIEW	SEMP	APR 2021		

GOLDER

POINT LOAD TEST ON ROCK SAMPLES

ASTM D5731

PROJECT NO. 1786659

TITLE Parsons/LVR 4017-E-0023/Mega

DATE November 2, 2020

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core [2] Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm)** (MPa)	**Approx. [1] UCS** (MPa)
S1	02	6.41-6.61	D	54.93	34.22	-	30,920.00	29.31	-	25.032	21.105	485
S1	02	6.41-6.61	A	24.55	47.33	38.46	10,300.00	9.76	6.600	-	5.865	123
S3	02	6.25-6.44	D	61.57	43.48		22,120.00	20.97		11.092	10.416	240
S3	02	6.25-6.44	A	24.10	47.28	38.09	7,040.00	6.67	4.600	-	4.070	85
S4	02	1.80-2.05	D	66.96	43.44		28,500.00	27.02		14.318	13.440	309
S4	02	1.80-2.05	A	25.02	47.12	38.74	11,260.00	10.67	7.111	-	6.340	133
S17	02	6.30-6.50	D	61.29	43.37		32,980.00	31.27		16.622	15.591	359
S17	02	6.30-6.50	A	23.85	47.20	37.86	12,360.00	11.72	8.175	-	7.213	151
B2	02	5.95-6.37	D	46.25	43.22		16,640.00	15.77		8.445	7.909	182
B2	02	5.95-6.37	A	19.34	46.62	33.88	13,460.00	12.76	11.116	-	9.330	177
B3	02	4.72-4.92	D	51.41	42.63		9,420.00	8.93		4.914	4.574	105
B3	02	4.72-4.92	A	24.65	47.23	38.50	3,780.00	3.58	2.418	-	2.149	45
B4	02	5.91-6.12	D	61.71	42.63		19,820.00	18.79		10.339	9.623	221
B4	02	5.91-6.12	A	19.37	47.27	34.14	6,980.00	6.62	5.676	-	4.781	91
B18	02	6.15-6.33	D	64.95	44.06		14,100.00	13.37		6.886	6.505	150
B18	02	6.15-6.33	A	22.85	47.36	37.12	6,240.00	5.92	4.293	-	3.755	79

[1] Is_{50} x 19&21 from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

[2] Actual distance between point load cones at time of failure.

Checked By: AH

Golder Associates

Rock Laboratory Testing Results

A report submitted to:

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January 5, 2021
Project number: 1786659
(S-series)

Abstract

This document summarizes the results of rock laboratory testing, including 12 Uniaxial Compressive Strength (UCS) tests. The UCS and Young's modulus values along with photographs of samples before and after testing are presented herein.

In this document:

1 Uniaxial Compressive Strength Tests 1
Appendices 5

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1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing. 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 displacement rate of approximately 0.08 mm/min (Figure 1). The preparation and testing procedure for each UCS specimen included the following:

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

Figure 1: Forney loading frame setup for UCS 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 D4543-19. 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 note:

- Testing 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 UCS testing are summarized in Table 1. The corresponding stress-strain curves are presented in Figure 2 to Figure 3. The Young's modulus is the tangent modulus calculated as the slope of the best-fit line through 300 data points on either side of the point representing 50% of the UCS strength. Additional specimen and testing details are included in the summary spreadsheet that accompanies this report.

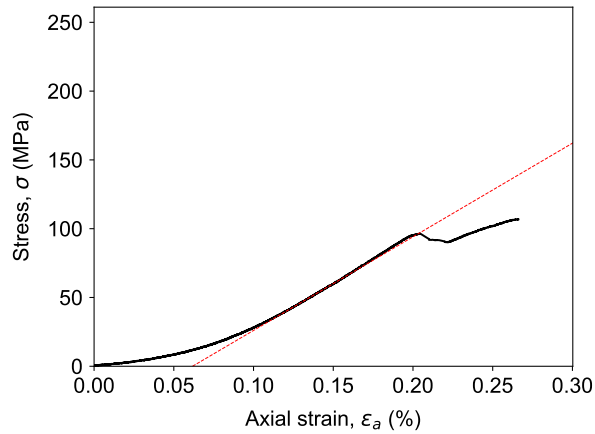
Table 1: Summary of Uniaxial Compression test results.

Sample	Depth (m)	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's modulus E (GPa)	Lithology	Failure description
SA-01, S1	4.75 - 5.05	3.056	106.9	68.1	Amphibolite	1
SA-01, S2	3.86 - 4.10	3.025	173.4	100.2	Amphibolite	1, 2
SA-01, S3	6.94 - 7.32	2.896	85.9	61.8	Bt-Hbl Gneiss	1, 2
SA-01, S10	5.43 - 5.73	3.033	186.5	109.2	Amphibolite	1, 2
S9, SA-01	2.25 - 2.51	3.075	166.9	102.8	Amphibolite	1, 2
S18, SA-01	4.43 - 4.70	3.049	188.7	109.1	Amphibolite	3, 2
S21, SA-01	3.74 - 4.18	3.006	248.6	106.1	Gneiss	4, 2
S4, SA-01	2.50 - 2.75	2.926	124.4	74.8	Gneiss	5, 2
S17, SA-01	7.27 - 7.56	3.012	220.1	101.5	Amphibolite	1, 2
S20, SA-01	6.70 - 6.98	2.821	145.2	67.2	Gneiss	3, 2
S13, SA-01	2.07 - 2.30	2.933	103.9	68.9	Gneiss	5, 2, 6
S14, SA-01	0.46 -0.70	2.871	136.7	64.2	Amphibolite	1, 2

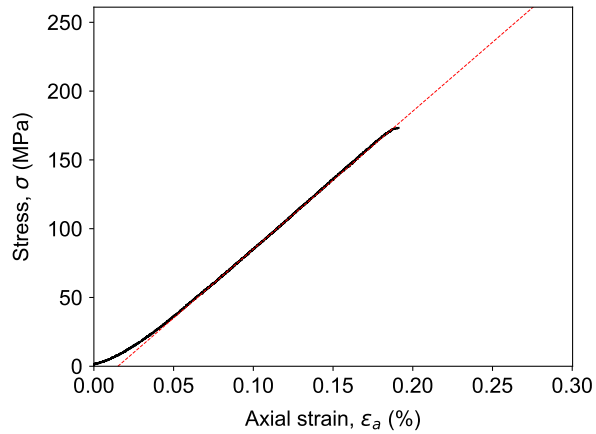
[1] Axial splitting failure
[2] Failure partly along pre-existing structure
[3] Inclined shear fracture and axial splitting failure
[4] Partial hourglass failure
[5] Inclined shear failure
[6] Length:Diameter ratio less than 2

1.3 Specimen photographs

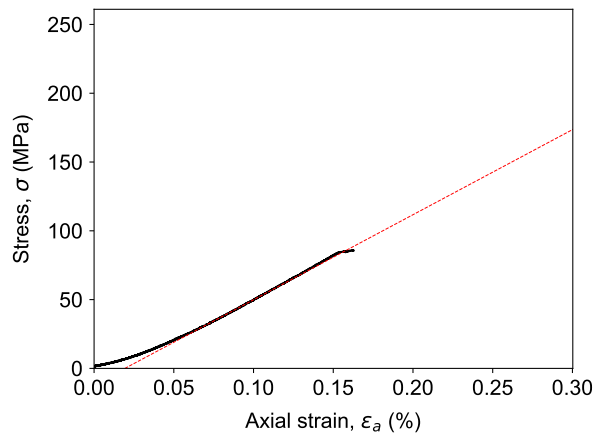
Photographs of the specimens before and after testing are presented in the Appendix of this report.



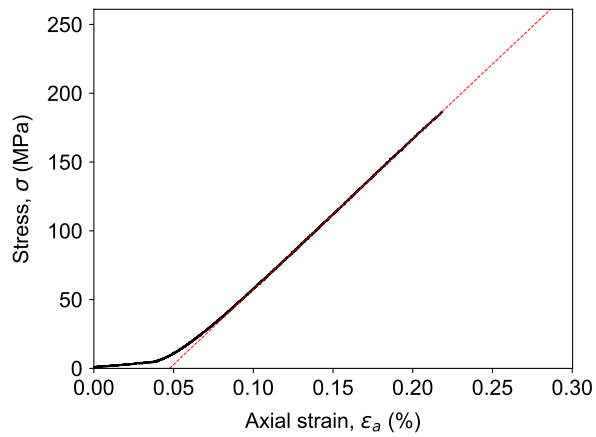
(a) SA-01-S1



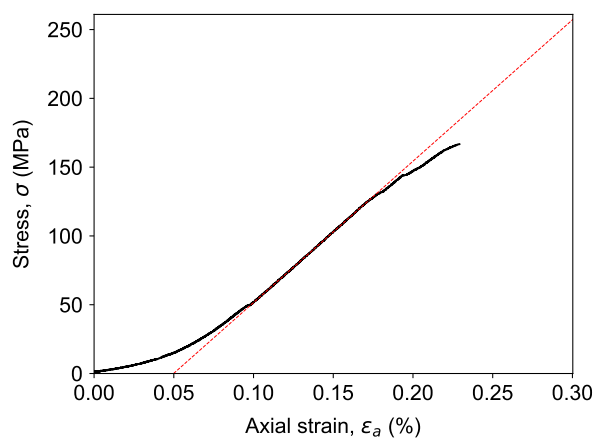
(b) SA-01-S2



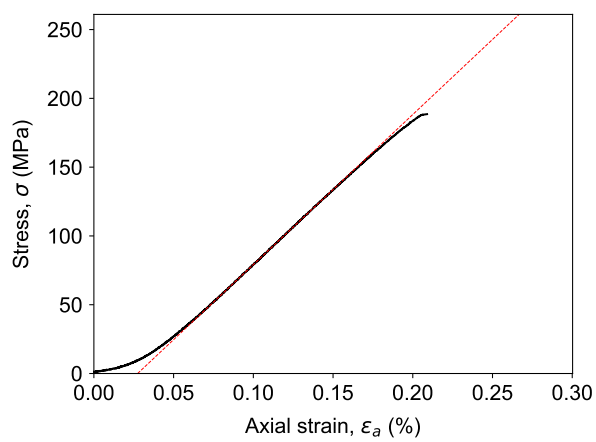
(c) SA-01-S3



(d) SA-01-S10

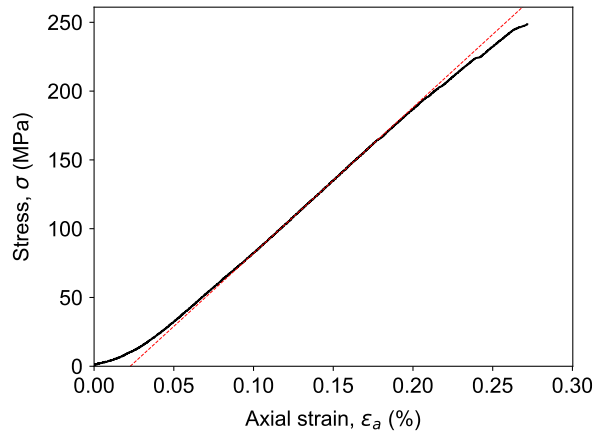


(e) S9-SA-01

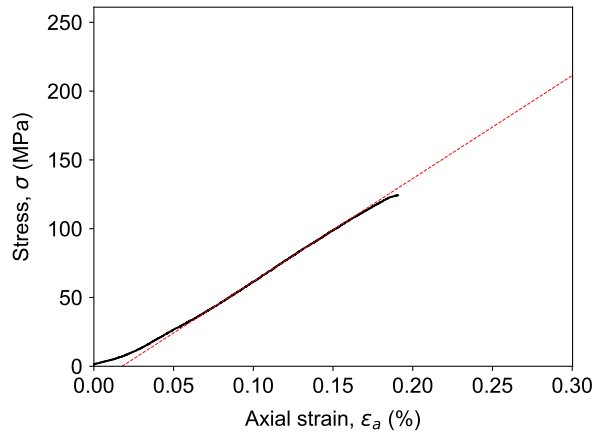


(f) S18-SA-01

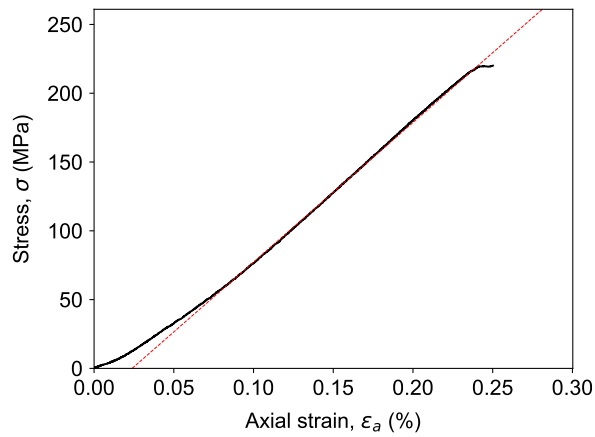
Figure 2: Measured stress-strain curves.



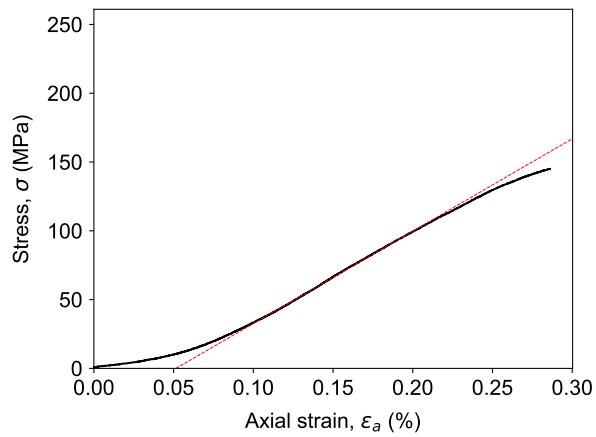
(a) S21-SA-01



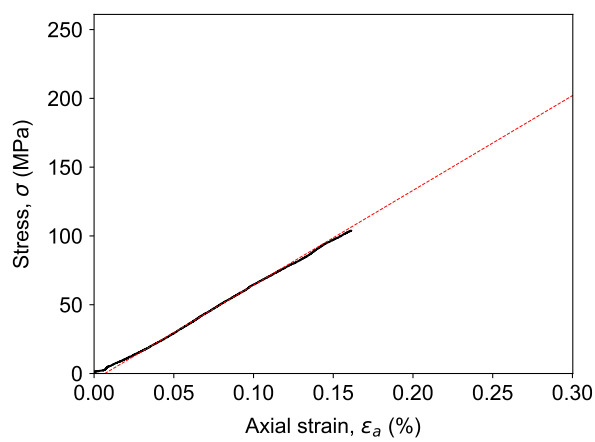
(b) S4-SA-01



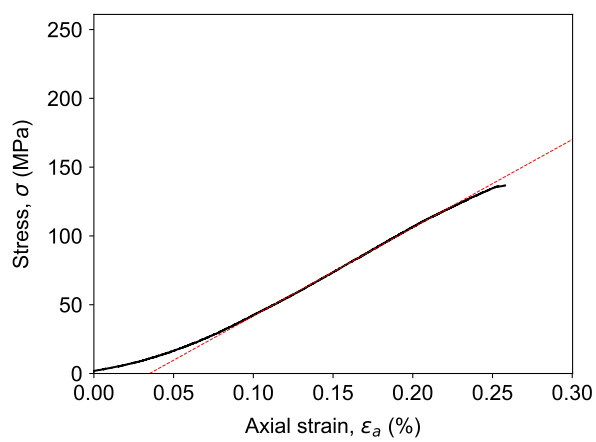
(c) S17-SA-01



(d) S20-SA-01



(e) S13-SA-01



(f) S14-SA-01

Figure 3: Measured stress-strain curves.

Appendices

Specimen sheets

- SA-01, S1
- SA-01, S2
- SA-01, S3
- SA-01, S10
- S9, SA-01
- S18, SA-01
- S21, SA-01
- S4, SA-01
- S17, SA-01
- S20, SA-01
- S13, SA-01
- S14, SA-01

Uniaxial Compression Test

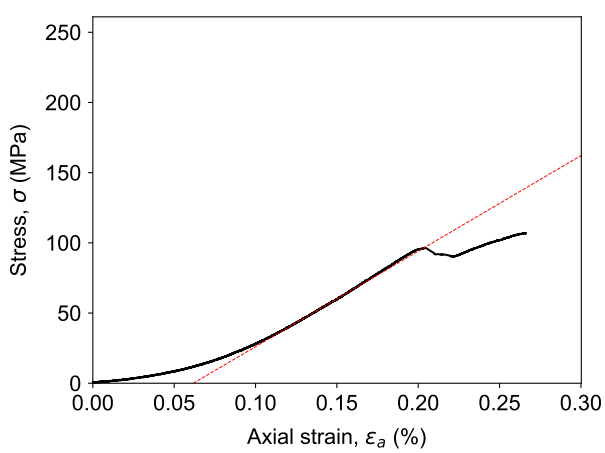
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	SA-01, S1	**Depth**	4.75 - 5.05

Specimen parameters	
Diameter (mm) [a]	47.32
Length (mm) [a]	98.45
Bulk density ρ (g/cm^3)	3.056
UCS (MPa)	106.9
Young's modulus E (GPa) [b]	68.1
Lithology	Amphibolite
Failure description [c]	1

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] 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.
[c] Failure description: [1] Axial splitting failure;

Prior to testing

After testing



Remarks:

Performed by	SL	**Date**	2020-10-16

Uniaxial Compression Test

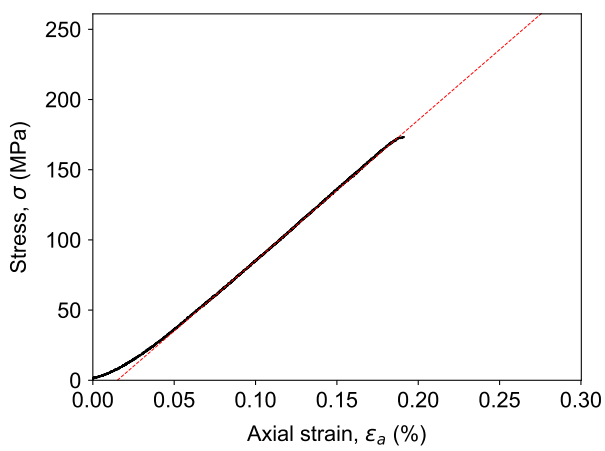
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	SA-01, S2	**Depth**	3.86 - 4.10

Specimen parameters	
Diameter (mm) [a]	47.19
Length (mm) [a]	98.84
Bulk density ρ (g/cm^3)	3.025
UCS (MPa)	173.4
Young's modulus E (GPa) [b]	100.2
Lithology	Amphibolite
Failure description [c]	1, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] 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.
[c] Failure description: [1] Axial splitting failure; [2] Failure partly along pre-existing structure;

Prior to testing

After testing



Remarks:

Performed by	SL	**Date**	2020-10-16

Uniaxial Compression Test

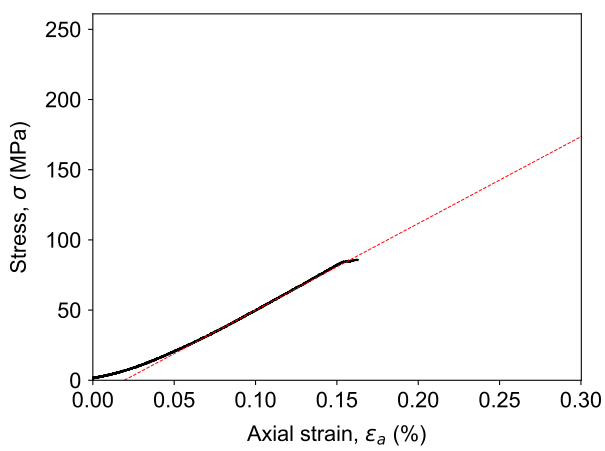
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	SA-01, S3	**Depth**	6.94 - 7.32

Specimen parameters	
Diameter (mm) [a]	47.36
Length (mm) [a]	99.24
Bulk density ρ (g/cm^3)	2.896
UCS (MPa)	85.9
Young's modulus E (GPa) [b]	61.8
Lithology	Bt-Hbl Gneiss
Failure description [c]	1, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] 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.
[c] Failure description: [1] Axial splitting failure; [2] Failure partly along pre-existing structure;

Prior to testing

After testing



Remarks:

Performed by	SL	**Date**	2020-10-16

Uniaxial Compression Test

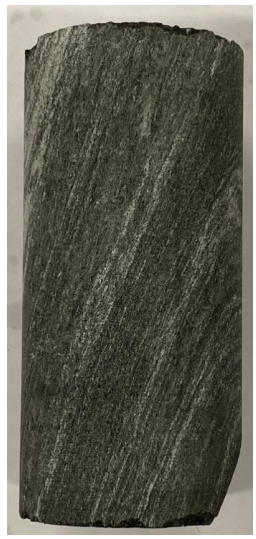
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	SA-01, S10	**Depth**	5.43 - 5.73

Specimen parameters

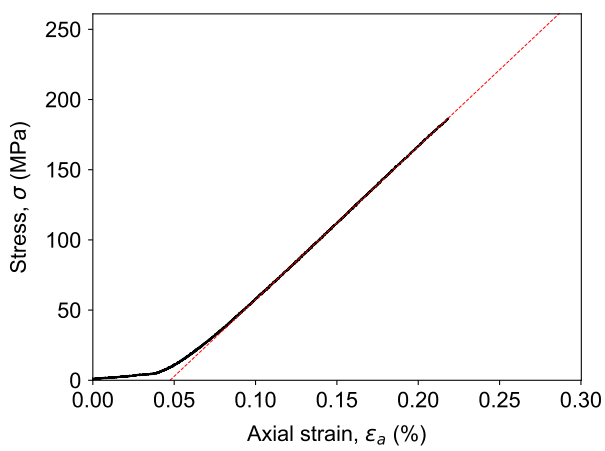
Diameter (mm) [a]	46.67
Length (mm) [a]	98.04
Bulk density ρ (g/cm^3)	3.033
UCS (MPa)	186.5
Young's modulus E (GPa) [b]	109.2
Lithology	Amphibolite
Failure description [c]	1, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] 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.
[c] Failure description: [1] Axial splitting failure; [2] Failure partly along pre-existing structure;

Prior to testing



After testing



Remarks:

Performed by	SL	**Date**	2020-10-16

Uniaxial Compression Test

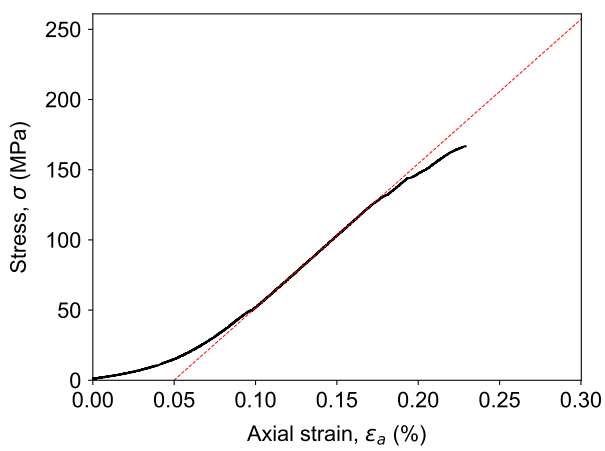
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	S9, SA-01	**Depth**	2.25 - 2.51

Specimen parameters	
Diameter (mm) [a]	47.35
Length (mm) [a]	97.90
Bulk density ρ (g/cm^3)	3.075
UCS (MPa)	166.9
Young's modulus E (GPa) [b]	102.8
Lithology	Amphibolite
Failure description [c]	1, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] 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.
[c] Failure description: [1] Axial splitting failure; [2] Failure partly along pre-existing structure;

Prior to testing

After testing



Remarks:

Performed by	SL	**Date**	2020-11-19

Uniaxial Compression Test

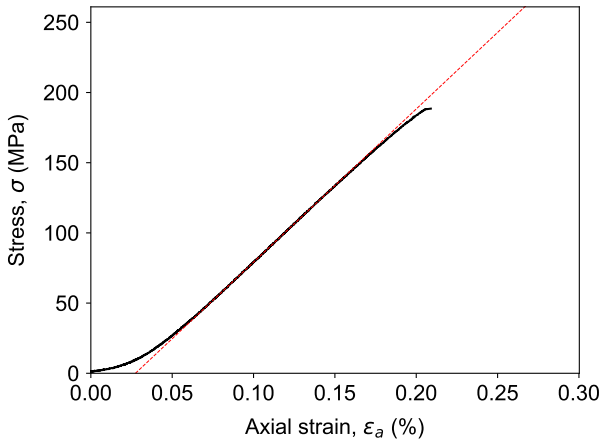
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	S18, SA-01	**Depth**	4.43 - 4.70

Specimen parameters	
Diameter (mm) [a]	47.30
Length (mm) [a]	98.27
Bulk density ρ (g/cm^3)	3.049
UCS (MPa)	188.7
Young's modulus E (GPa) [b]	109.1
Lithology	Amphibolite
Failure description [c]	3, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] 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.
[c] Failure description: [3] Inclined shear fracture and axial splitting failure; [2] Failure partly along pre-existing structure;

Prior to testing

After testing



Remarks:

Performed by	SL	**Date**	2020-11-19

Uniaxial Compression Test

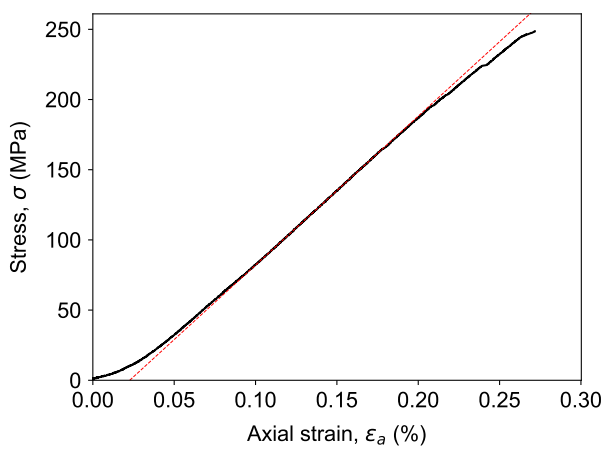
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	S21, SA-01	**Depth**	3.74 - 4.18

Specimen parameters	
Diameter (mm) [a]	47.35
Length (mm) [a]	98.06
Bulk density ρ (g/cm^3)	3.006
UCS (MPa)	248.6
Young's modulus E (GPa) [b]	106.1
Lithology	Gneiss
Failure description [c]	4, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] 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.
[c] Failure description: [4] Partial hourglass failure; [2] Failure partly along pre-existing structure;

Prior to testing

After testing



Remarks:

Performed by	SL	**Date**	2020-11-19

Uniaxial Compression Test

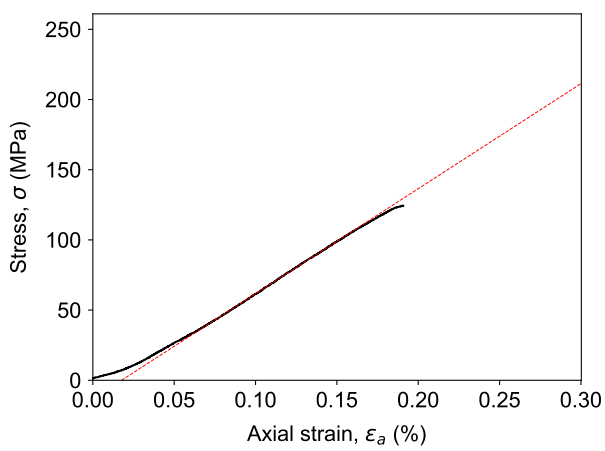
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	S4, SA-01	**Depth**	2.50 - 2.75

Specimen parameters	
Diameter (mm) [a]	47.21
Length (mm) [a]	97.78
Bulk density ρ (g/cm^3)	2.926
UCS (MPa)	124.4
Young's modulus E (GPa) [b]	74.8
Lithology	Gneiss
Failure description [c]	5, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Tangent modulus, calculated as the slope of the best fit line through ±235 data points on either side of the point representing 50.0% of the peak strength.
[c] Failure description: [5] Inclined shear failure; [2] Failure partly along pre-existing structure;

Prior to testing

After testing



Remarks: Failure in less than 3 minutes

Performed by	SL	**Date**	2020-11-19

Uniaxial Compression Test

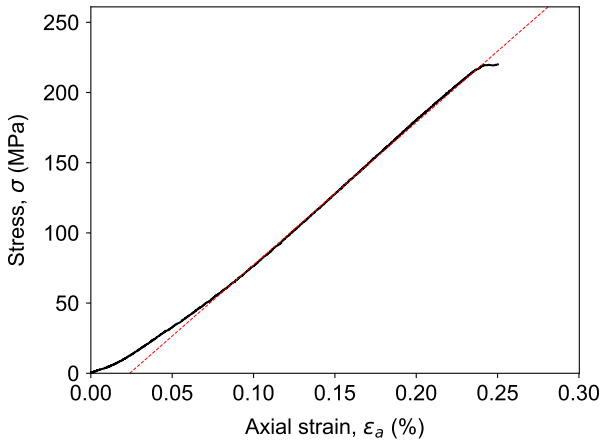
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	S17, SA-01	**Depth**	7.27 - 7.56

Specimen parameters	
Diameter (mm) [a]	47.24
Length (mm) [a]	98.57
Bulk density ρ (g/cm^3)	3.012
UCS (MPa)	220.1
Young's modulus E (GPa) [b]	101.5
Lithology	Amphibolite
Failure description [c]	1, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] 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.
[c] Failure description: [1] Axial splitting failure; [2] Failure partly along pre-existing structure;

Prior to testing

After testing



Remarks:

Performed by	SL	**Date**	2020-11-19

Uniaxial Compression Test

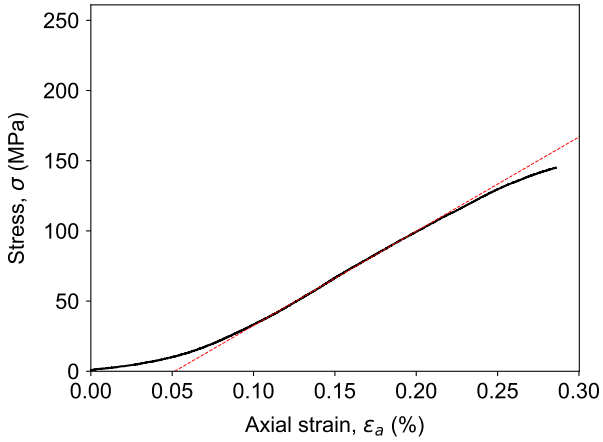
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	S20, SA-01	**Depth**	6.70 - 6.98

Specimen parameters	
Diameter (mm) [a]	47.35
Length (mm) [a]	97.68
Bulk density ρ (g/cm^3)	2.821
UCS (MPa)	145.2
Young's modulus E (GPa) [b]	67.2
Lithology	Gneiss
Failure description [c]	3, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] 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.
[c] Failure description: [3] Inclined shear fracture and axial splitting failure; [2] Failure partly along pre-existing structure;

Prior to testing

After testing



Remarks:

Performed by	SL	**Date**	2020-11-19

Uniaxial Compression Test

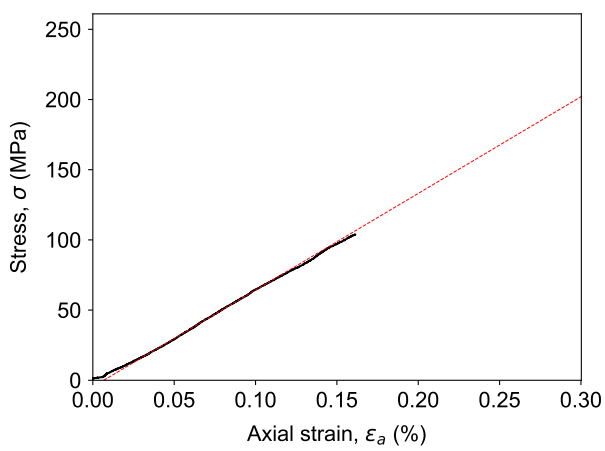
Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	S13, SA-01	**Depth**	2.07 - 2.30

Specimen parameters	
Diameter (mm) [a]	49.54
Length (mm) [a]	96.17
Bulk density ρ (g/cm^3)	2.933
UCS (MPa)	103.9
Young's modulus E (GPa) [b]	68.9
Lithology	Gneiss
Failure description [c]	5, 2, 6

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Tangent modulus, calculated as the slope of the best fit line through ±211 data points on either side of the point representing 50.0% of the peak strength.
[c] Failure description: [5] Inclined shear failure; [2] Failure partly along pre-existing structure; [6] Length:Diameter ratio less than 2;

Prior to testing

After testing



Remarks: Failure in less than 3 minutes

Performed by	SL	**Date**	2020-11-19

Uniaxial Compression Test

Client	Golder Associates Ltd.	**Project**	1786659 (S-series)
Sample	S14, SA-01	**Depth**	0.46 -0.70

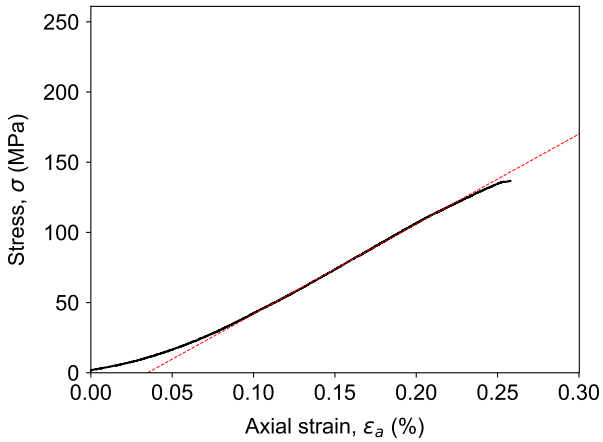
Specimen parameters

Diameter (mm) [a]	47.14
Length (mm) [a]	98.69
Bulk density ρ (g/cm^3)	2.871
UCS (MPa)	136.7
Young's modulus E (GPa) [b]	64.2
Lithology	Amphibolite
Failure description [c]	1, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] 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.
[c] Failure description: [1] Axial splitting failure; [2] Failure partly along pre-existing structure;

Prior to testing

After testing



Remarks:

Performed by	SL	**Date**	2020-11-19

APPENDIX C

Non-Standard Special Provisions

REQUIREMENTS FOR THE SUPPLY, INSTALLATION AND TESTING OF DOWELS INTO ROCK FOR PIER FOOTINGS

1.0 SCOPE

The work for the above noted tender item shall be in accordance with OPSS 904, including all Special Provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the pier footing.

2.0 REFERENCES

This specification refers to the following standards, specifications, or publications:

ASTM International

D1143M Standard Test Methods for Deep Foundations Under Static Axial Compressive Load

3.0 DEFINITIONS

For the purpose of this specification, the following definitions apply:

Dowels into Rock means reinforcing steel bar and non-shrink grout.

Design Engineer means an Engineer who has a minimum of five (5) years experience in all aspects associated with the underwater installation of Dowels into Rock, including drilling, underwater grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.

Quality Verification Engineer means an Engineer who has a minimum of five (5) years experience in all aspects associated with the underwater installation of Dowels into Rock, including drilling, underwater grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Working Drawings

Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.

The Contractor shall submit Working Drawings to the Contract Administrator as follows:

a) All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.

b) All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.

Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.

Working Drawings consisting of testing an installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.

Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:

a) Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.

b) Test results verifying the 28 day strength of non-shrink grout.

c) The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.

d) The procedures to verify hole length. Records of measurements that verify the hole length.

e) Records of all drilling procedures, rock conditions encountered, and installation times.

f) Test procedures for Dowels into Rock.

g) Drawings and design calculations for a suitable reaction system for the applied test loads.

h) Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.

i) Drawings and details for reference system arrangement.

j) Current calibration curves shall be provided for all gauges.

k) Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.

l) Remedial measures for unacceptable stressing results.

5.0 MATERIALS

5.01 Non-Shrink Grout

The non-shrink grout shall be an approved product from the MTO's Pre-Qualified Products List.

5.02 Anti-Washout Agent

The anti-washout agent shall be used with the non-shrink grout for the Dowels into Rock. The anti-washout agent shall be one of the following proprietary products:

1) Sikament 100 SC Anti-Washout Admixture
   Sika Canada Inc.
   6915 Davand Drive
   Mississauga, ON, L5T 1L5
   Toll Free Phone: 800-933-7452

2) Rheomac UW 450 Anti-Washout Admixture
   BASF Construction Chemicals Canada Ltd (Master Builders)
   1800 Clark Blvd
   Brampton, ON, L6T 4M7
   Toll Free Phone: 416-520-1392

5.03 Manufacturer Information

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

a) Data sheets for the non-shrink grout and anti-washout agent,

b) technical information that proves that the non-shrink grout and anti-washout agent are compatible, and

c) installation procedures.

6.0 EQUIPMENT

All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

The equipment shall not cause damage to the reinforcing steel bars.

7.0 CONSTRUCTION

7.01 Instructions to Contractor

These instructions are to be read in conjunction with the Contract Drawings.

A total of 2 test Dowels into Rock are required for the Dowels into Rock at the pier.

Dowels into rock at the pier shall be installed prior to unwatering the structure excavation. Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

7.02 Responsibilities of the Contractor

The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.

The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.

The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 4.0.

7.03 Subsurface Conditions

Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

7.04 Construction of Holes

The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.

At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

7.05 Installation of Reinforcing Steel Bar

Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

Dowels into Rock at the pier shall be installed prior to unwatering the structure excavation. Dowels shall extend through the tremie concrete for the pier footing and into sound bedrock.

Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

7.06 Grout and Anti-Washout Agent

The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.

The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

Anti-washout agent shall be used in accordance with the specifications of the manufacturer.

Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

8.0 QUALITY ASSURANCE

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.01 Qualifications

8.01.01 Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock

All work shall be performed under the direction of personnel experienced with all aspects associated with the underwater installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.

8.01.02 Qualifications of the Quality Verification Engineer

A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.

8.01.03 Qualifications of the Design Engineer

A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

8.02 Testing Requirements

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.02.01 General Testing Requirements

Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.

The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.

The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the pier. The Dowels into Rock for testing shall be 55M dowels grouted into 140 mm diameter holes filled with an approved non-shrink grout with a minimum 4,000 mm embedment into sound bedrock.

The Contractor shall submit Working Drawings that include proposed procedures for testing of the Dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.

The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.

The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.

The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

8.02.02 Testing Location

The Contractor shall remove all loose rock down to sound bedrock at the test location.

The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator. The water depth at the location of the test shall be at least 0.5 m deep.

If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

8.02.03 Testing Equipment

The dowels into rock will be carried out generally in accordance with the prevailing requirements of ASTM International D1143M superseded where applicable by the procedures specified in this document.

The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.

The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:

The beams shall be independently supported with the support firmly embedded in the ground.

The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.

Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

8.02.04 Testing for Dowels Into Rock, and Report

At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Jacks used for reinforcing steel bars shall have a minimum ram dimension of 152.6 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

8.02.05 Testing Loading

The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the test load of 1,150 kN. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

8.03 Acceptance Criteria

The following acceptance criteria apply:

a) The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at the pier footing.

b) Tests for Dowels into Rock shall have a capacity of at least 1035 kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

9.0 MEASUREMENT FOR PAYMENT

For measurement purposes, a count shall be made of the number of dowels installed.

10.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.

WARRANT: Use only in consultation with Regional Structural Section with this non-standard tender item.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN0003

Amendment to OPSS 902, November 2019

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 902.03 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.

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 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 [** Designer Fill-In, See Notes to Designer] 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.

902.04.02.02 Working Drawings

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

902.07 CONSTRUCTION

902.07.04 Dewatering Structure Excavation

Subsection 902.07.04 of OPSS 902 is amended by the addition of the following clauses:

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:

* 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.

<u>ROCK EXCAVATION FOR STRUCTURE</u> – Item No.

Non-Standard Special Provision

Amendment to OPSS 902, November 2010

902.07 Construction

902.07.05.02 Excavation for Foundations

Section 902.07.05.02 of OPSS 902 is amended by the deletion of the fifth paragraph and the addition of the following:

Excavations for foundations for the Skootamatta River replacement bridge and for a temporary modular bridge (TMB), will extend into the strong to very strong gneiss and amphibolite bedrock. As such, appropriate equipment and construction procedures will be required to penetrate the overburden and excavate the bedrock to reach the design founding level.

Depending on the detour alignment and replacement approach adopted by the Design-Builder, where bedrock excavation is required adjacent to and/or below an existing foundation that is supporting the existing bridge, such excavation shall be carried out using saw-cutting or line drilling techniques adjacent to such existing foundations. During construction, if removal of bedrock extends to 0.3 m or deeper below the base of the adjacent existing foundation, then immediate protection shall be required to support the existing foundation. Any over-excavation of the bedrock below the design foundation level must be replaced immediately with mass concrete, having a minimum 28-day compressive strength of 20 MPa.

DEWATERING STRUCTURE EXCAVATIONS (COFFERDAMS) – Item No.

Non-Standard Special Provision

1.0 SCOPE

As part of the work under this item, the Contractor shall design, supply, and install cofferdams to construct the foundations for the new abutments at the Skootamatta River Bridge Site.

2.0 REFERENCES – Not Used

3.0 DEFINITIONS

Stamped means drawings or details that have been reviewed and stamped "Conforms With Contract Documents". The stamp shall include the date and signature of the Contractor's Engineer.

Contractor's Engineer means an Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of cofferdams. The Contractor shall retain the Contractor's Engineer to ensure conformance with the contract document.

Cofferdam Design Engineer means an Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of cofferdams. In addition, the Cofferdam Design Engineer shall have had responsible experience in the design of at least 5 other cofferdams. The Contractor shall retain the Cofferdam Design Engineer to ensure conformance with the contract documents and issue certificate(s) of conformance for the design.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

The design of concrete cofferdams shall be in accordance with the Canadian Highway Bridge Design Code CAN/CSA-S6-19.

Submission of Shop Drawings

All shop drawings submissions shall bear the seal and signature of the Cofferdam Design Engineer.

At least two weeks prior to the commencement of cofferdam construction, the Contractor shall submit to the Contract Administrator, for information purposes only, a set of stamped drawings and calculations of the cofferdam system, which shall include the following:

- the cofferdam design;
- the location, type and dimensions of each cofferdam to be used;
- a schematic showing the configuration of all cofferdams; and
- details as applicable.

The Contractor's Engineer shall review all calculations, construction details, shop drawings and procedures.

All submissions shall bear the seal and signature of the Cofferdam Design Engineer and Contractor's Engineer.

5.0 MATERIALS – Not Used

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

The strong to very strong gneiss and amphibolite bedrock is found at relatively shallow depth below ground surface at the abutment locations, and the surface of the bedrock will vary and undulate across each foundation element. Appropriate equipment and procedures will be required to seat the cofferdams onto or into the surface of the bedrock and mitigate groundwater seepage at the overburden-bedrock interface at the abutment locations. In addition, obstructions may be present within the existing fill materials (i.e., rock fill) at the site. Appropriate equipment and procedures will be required to penetrate these obstructions to allow for installation of the cofferdam and for construction of the foundations in dewatered conditions.

The Contractor shall cut the cofferdam at the limits indicated on the Contract Drawings at the completion of the construction of the footings.

8.0 QUALITY ASSURANCE

Certificates of Conformance

The Cofferdam Design Engineer shall inspect the installation of each cofferdam. After the installation of each of the cofferdam has been completed, the Contractor shall submit a Certificate of Conformance for each cofferdam to the Contract Administrator, sealed and signed by the Cofferdam Design Engineer. The Certificates of Conformance shall state that the cofferdam is in place, and has been installed in conformance with the stamped shop drawings and the Contract Drawings.

9.0 MEASUREMENT FOR PAYMENT

Measurement for cofferdams shall be by length in metres of cofferdam installed.

10.0 BASIS OF PAYMENT

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Materials to carry out the work.

OBSTRUCTIONS – Item No.

Non-Standard Special Provision

The Contactor shall be alerted to the presence of cobbles, boulders and/or rock fill obstructions at this site as encountered in Boreholes S6, S10, S18 and S21; such obstructions should be expected between and beyond the borehole locations. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavations, and installation of temporary protection systems/cofferdams.

VIBRATION MONITORING - Item No.

Non-Standard Special Provision

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1.0 SCOPE

This special provision describes requirements for vibration monitoring during bedrock excavation, installation of temporary protection systems/cofferdams, and operation of vibratory compaction equipment for the construction of the Skootamatta River Bridge and approach embankments.

2.0 DEFINITIONS

For the purposes of this specification, the following definitions apply:

Peak Particle Velocity (PPV) means the maximum component velocity in millimetres per second (mm/sec) that ground particles move as a result of energy released from vibratory construction operations.

Pre-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, prior to the commencement of vibratory or vibration-inducing construction operations.

Post-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, after completion of vibratory or vibration-inducing construction operations.

3.0 DESIGN AND SUBMISSION REQUIREMENTS

3.1 Submission Requirements

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring plan to the Contract Administrator for information purposes. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

a) Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
b) Qualifications of vibration monitoring specialist.
c) Details regarding proposed instrumentation.
d) Proposed location of instruments adjacent to the residences, utilities, wells, or other potentially vibration-sensitive structures within a 150 m or 250 m radius from the Skootamatta River Bridge replacement site. The 150 m radius shall apply if relatively limited bedrock excavation will be completed using line drilling and hoe-ramming techniques, and the 250 m radius shall apply if blasting is used consistent with the requirements of OPSS 120.
e) Proposed frequency of readings.
f) Action plan to be taken to adjust bedrock excavation, protection system installation methods or vibratory compaction methods if readings show vibrations exceeding tolerable levels.

4.0 EQUIPMENT

4.1 Vibration Monitoring Equipment

All vibration monitoring equipment shall be capable of measuring and recording ground vibration PPV up to 200 mm/s in the vertical, transverse, and radial directions. The equipment shall have been calibrated within the last 12 months either by the manufacturer or other qualified agent. Proof of calibration shall be submitted to the Contract Administrator prior to commencement of any monitoring operations.

5.0 CONSTRUCTION

5.1 Pre- and Post-Construction Condition Surveys

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, and facilities within a 150 m or 250 m radius from the Skootamatta River Bridge replacement; the 150 m radius shall apply where rock excavation is completed using line drilling and hoe-ramming techniques, and the 250 m radius shall apply where blasting is employed.

5.1.1 Pre-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

The Pre-Construction Condition Survey, at each structure/well within the above-specified radius from the proposed Skootamatta River Bridge, shall be completed a minimum of two (2) weeks prior to commencement of bedrock blasting/excavation and installation of cofferdams and/or temporary protection systems. Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of bedrock excavation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

The Pre-Construction Condition Survey shall include, as a minimum, the following information:

a) Type of structure, including type of construction and if possible, the date when built.
b) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
c) Digital photographs or digital video or both, as necessary, to record areas of significant concern.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Pre-Construction Construction Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request.

5.1.2 Post-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

A Post-Construction Condition Survey at each structure within the above-specified radius from the proposed Skootamatta River Bridge, is required within two (2) months of completion of rock excavation, installation of cofferdams and/or temporary protection systems.

The Post-Construction Condition Survey shall include, as a minimum, the following information:

a) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
b) Digital photographs or digital video or both, as necessary, to record areas of significant concern.
c) Comparison between pre-condition survey documented concerns and post-condition concerns.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Post-Construction Condition Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request. The report shall confirm that there have been no changes to the property between the Pre-Construction Condition Survey and the Post-Construction Condition Survey as a result of the excavation bedrock and installation of spread/strip footings and temporary protection systems.

5.2 Monitoring

The vibration monitoring equipment shall be placed on the ground surface at radial distances of 25 m, 50 m, and 100 m (or otherwise adjusted to suit available space) from the foundation elements/protection system locations within the project toward the receptors (e.g., buildings, sensitive utilities). The Contractor shall take readings during excavation and installation of cofferdams and temporary protection systems.

The vibrations measured on private structures, wells, etc. shall not exceed 20 mm/s. Those measured on utilities, if applicable, shall not exceed 10 mm/s.

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations at the various locations are within acceptable levels.

5.3 Records

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring to the Contract Administrator as follows:

a) The time/duration of each reading.
b) Construction operations (i.e. installation of sheet piling) and timing of such relative to the readings.

c) Details of exceedances and modifications to operations.
d) Final report containing all relevant data including vibration monitoring and Pre- and Post-Construction Condition Surveys.

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