



Preliminary Foundation Investigation and Design Report

North Driftwood River Bridge Replacement (Site 39E-0013/B0)

Highway 11, Driftwood, Ontario

G.W.P. 5282-14-00, W.P. 5282-14-02, MTO Agreement No. 5017-E-0022

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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
NORTH DRIFTWOOD RIVER BRIDGE REPLACEMENT (SITE 39E-0013/B0)
HIGHWAY 11, DRIFTWOOD, ONTARIO
G.W.P. 5282-14-00, W.P. 5282-14-02, MTO AGREEMENT NO. 5017-E-0022**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by D.M. Wills Associates Ltd. (Wills) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement of the North Driftwood River Bridge (Site 39E-0013-B0) on Highway 11 in Driftwood, Ontario (G.W.P 5282-14-00, W.P. 5282-14-02), as part of MTO's Agreement No. 5017-E-0022.

The purpose of this investigation is to obtain subsurface soil, bedrock, and groundwater information at the site by means of a limited number of boreholes and geotechnical laboratory testing. The investigation also included a review of available geotechnical information from a previous foundation investigation completed by others at the structure site, as presented in the following report:

- **MTO GEOCREs No. 42H-24:** "Foundation Investigation Report for Driftwood River Bridge, 4.2 km West of Cochrane, Highway 11, W.P. 55-79-03, Site 39E-13, District 16, Cochrane" by the Ontario Ministry of Transportation and Communications, dated 1985.

2.0 SITE DESCRIPTION

The existing bridge carries two lanes of traffic along Highway 11 over the North Driftwood River, approximately 11 kilometres (km) west of Highway 655 and 30 km northwest of Cochrane, Ontario, as shown on the Key Plan on Drawing 1. At the site, Highway 11 is oriented in the west-east direction and North Driftwood River flows in the south to north direction. The lands surrounding the bridge are relatively flat and consist of heavily treed areas, grassy areas and farmland. A residential property is located to the southeast of the existing bridge.

The existing five-span bridge was constructed in 1942 and is about 89 metres (m) long and about 10 m wide. The bridge abutments and piers were supported on 1.9 m (6 feet 3 inches) overall diameter caissons formed from twelve sections of driven sheet piling with bent flanges, filled with reinforced concrete. The original abutments failed due to excessive lateral earth pressures and in 1986/1987, the original abutments were reinforced with 16 HP 310X110 driven piles (8 piles at each abutment) each in the area of the original caissons and the pile cap reconstructed to re-incorporate the caissons/piles at the original design level.

Based on the 1986 General Arrangement (GA) drawing for the structure site (Site No. 39E-13), the original ground surface at the site is about Elevation 249 m and the Highway 11 grade ranges from about Elevation 252 m at the west end to Elevation 251 m at the east end, indicating the embankments are about 2 m to 3 m high. The existing embankment side slopes are included at about 2 Horizontal: 1 Vertical (2H:1V).

3.0 INVESTIGATION PROCEDURES

The field investigation was carried out on October 30 and 31, 2018, during which time two boreholes (designated as Boreholes 18-1 and 18-2) were advanced to depths of 12.0 m and 11.1 m below ground surface and included 3.4 m and 3.3 m of bedrock coring, respectively. Borehole 18-1 was advanced within the southwest quadrant of the existing bridge site and Borehole 18-2 was advanced within the northeast quadrant of the bridge site, as shown on Drawing 1.

The field investigation was carried out using a track-mounted CME-55 drill rig supplied and operated by Landcore Drilling of Chelmsford, Ontario. The boreholes were advanced through the overburden using wash boring methods and 'NW' casing. Soil samples were obtained in the boreholes at approximate 0.75 m and 1.5 m intervals of depth using a 50 millimetre (mm) outside diameter (O.D.) split-spoon sampler, driven by an automatic

hammer in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586-11)¹. Field vane shear tests using standard MTO 'N' size vanes (ASTM D2573)² were carried out in the cohesive stratum. The results of in-situ field tests (i.e., SPT "N"-values and shear strength tests) as presented on the Record of Boreholes and in Section 4.2 are uncorrected. Core samples of cobbles, boulders, and bedrock were obtained using an 'NQ' coring method.

Due to the use of wash boring methods (addition of water in the borehole), the groundwater conditions in the open boreholes could not be observed during drilling, but the depth of the water level was measured in one partially open borehole immediately following removal of the casing. A standpipe piezometer was installed in Borehole 18-1 to permit monitoring of the groundwater level at the site. The installed piezometer consists of a 50 mm diameter PVC pipe, with a 1.5 m slotted screen sealed within a sand filter pack. The borehole and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to near ground surface with a mixture of soil cuttings and topped with a mixture of bentonite pellets and cement. The surface zone was capped with sand fill. The open borehole (Borehole 18-2) was backfilled with bentonite upon completion, in accordance with Ontario Regulation 903 (as amended).

Field work was monitored on a full-time basis by a member of Golder's engineering and technical staff, who located the boreholes in the field, arranged for the clearance of underground services, observed the drilling, sampling and in-situ testing operations, and logged the subsurface conditions. The samples were identified in the field, placed in appropriate labelled containers and transported to Golder's geotechnical laboratory in Mississauga where the soil samples and rock core samples underwent further visual examination and select laboratory testing. Index and classification tests consisting of water content and organic content, grain size distributions and Atterberg limits were carried out on soil samples; and unconfined compression (UC) tests were carried out on selected bedrock core samples. All laboratory testing was carried out according to applicable MTO LS and ASTM standards. The geotechnical laboratory results are presented in Appendix B.

The borehole locations and ground surface elevations were obtained using a GPS Trimble R2 RTK, having an accuracy of +/- 4 cm in the vertical and horizontal. The borehole locations are positioned relative to MTM NAD 83 (Zone 10) coordinate system and the ground surface elevations are referenced to Geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole No.	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)	Bedrock Coring Length (m)
	Northing (Latitude)	Easting (Longitude)			
18-1	5450183.8 (49.188917)	272857.1 (-81.438254)	248.7	12.0	3.4
18-2	5450198.9 (49.189056)	272922.3 (-81.437362)	246.9	11.1	3.3

¹ ASTM D1586-08a Standard Test Method for Standard Penetration Test.

² ASTM D2573M-15 Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain Study (NOEGTS)³ mapping, the site is located within a ground moraine deposit consisting primarily of till and nearby areas of organic terrain. Based on geological mapping by the Ontario Ministry of Northern Development and Mines (MNDM)⁴, the site is underlain by massive to foliated granodiorite to granite bedrock.

4.2 Subsurface Conditions

Subsurface soil, bedrock and groundwater conditions as encountered in the boreholes are presented on the Record of Borehole and Drillhole sheets in Appendix A. Geotechnical laboratory test results and photographs of the recovered bedrock core samples are provided in Appendix B.

The stratigraphic boundaries shown on the borehole records and on the stratigraphic profile on Drawing 1 are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs and field vane shear strength tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations, however, the factual data presented on the record of borehole governs any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawing 1 is a simplification of the subsurface conditions.

In general, the stratigraphy encountered at the site consists of a surface layer of topsoil and fill underlain by sequential deposits of organic silt / silt, sand to sand and gravel with a silt and sand interlayer, and sandy silt at one location. The native soils are underlain by meta-diorite - granite bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

An approximately 125 mm to 250 mm thick layer of topsoil was encountered at ground surface in Boreholes 18-1 and 18-2, respectively.

4.2.2 Fill

Cohesive fill and granular fill were encountered underlying the topsoil to a depth of 1.5 m below ground surface to Elevations 247.2 and 245.4 m in Boreholes 18-1 and 18-2, respectively. The cohesive fill consists of sandy clayey silt, some gravel and the granular fill consists of sandy silt to silty sand, trace gravel.

The SPT "N"-value measured within the cohesive fill is 6 blows per 0.3 m of penetration, suggesting a firm consistency. The SPT "N"-value measured within the granular fill is 9 blows per 0.3 m of penetration, indicating a loose compactness condition.

The in-situ water content measured on one sample of the granular fill is about 16 per cent.

³ Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 42ANE.

⁴ Ontario Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey – Map 2543.

4.2.3 Organic Silt / Silt

A 0.7 m and 3.1 m thick deposit of organic silt to clayey organic silt and silt was encountered underlying the fill and extended to depths of 4.6 m and 2.2 m below ground surface (Elevation 244.1 m and 244.7 m) in Boreholes 18-1 and 18-2, respectively. The organic silt deposit in Borehole 18-1 contains some sand and gravel, trace to some clay and becomes clayey below a depth of 3.5 m below ground surface (Elevation 245.2 m). Trace rootlets and trace wood and shell fragments were also encountered within the organic silt deposit. In Borehole 18-2, the silt deposit contains some sand, trace to some clay and trace gravel.

The SPT “N”-values measured within the organic silt / silt deposit range between weight of hammer (WH) and 20 blows per 0.3 m of penetration, suggesting a very loose to compact state of compactness. Two in-situ shear vane tests carried out in the clayey portion of the organic silt deposit measured shear strengths of 33 kPa and 60 kPa with a calculated sensitivity between 3 and 4. The undrained shear strengths indicate that the clayey organic silt portion of the deposit has a firm to stiff consistency.

Grain size distribution testing was carried out on two samples of the organic silt / silt deposit and the results are presented on Figure B-1 in Appendix B. The natural water content measured on one sample of the silt deposit is about 20 per cent and the natural water content measured on three samples of the organic silt portion of the deposit range from about 90 per cent to 190 per cent. Organic content testing was carried out on two samples of the organic silt portion of the deposit from Borehole 18-1 and measured organic contents of about 8 per cent and 26 per cent.

4.2.4 Sand to Sand and Gravel

A 3.4 m and 5.6 m thick deposit of sand to sand and gravel containing trace to some silt, trace clay was encountered underlying the organic silt / silt deposit and extends to depths of 8.0 m and 7.8 m below ground surface (Elevations 240.7 m and 239.1 m), in Boreholes 18-1 and 18-2, respectively. A 400 mm boulder and 200 mm cobble were encountered within the deposit in Borehole 18-2 at depths of 3.4 m and 4.3 m below ground surface (Elevation 243.5 m and 242.6 m), respectively.

The SPT “N”-values measured within the sand to sand and gravel deposit range from 22 blows to 100 blows for 0.3 m of penetration, with N-values of 76 blows and 84 blows for 0.2 m and 0.1 m of penetration, respectively, indicating a compact to very dense compactness condition.

Grain size distribution testing was carried out on three samples of the sand to sand and gravel deposit and the results are shown on Figure B-2 in Appendix B. The natural water content measured on samples of the sand to sand and gravel deposit range from about 6 per cent to 13 per cent.

4.2.5 Sandy Silt to Silt and Sand

Pockets of sandy silt or silt and sand containing trace to some gravel and trace clay were encountered in both boreholes: in Borehole 18-1, the 0.6 m thick pocket of sandy silt was encountered underlying the sand to sand and gravel deposit at a depth of 8.0 m below ground surface (Elevation 240.7 m); and in Borehole 18-2, the 1.1 m thick interlayer of sandy silt was encountered within the sand to sand and gravel deposit at a depth of 4.5 m below ground surface (Elevation 242.4 m).

One SPT “N”-value measured within the silt and sand deposit in Borehole 18-2 is 35 blows per 0.3 m of penetration indicating a dense compactness condition.

One grain size distribution test was carried out on a sample of the silt and sand interlayer and the result is shown on Figure B-3 in Appendix B. The natural water content measured on one sample of the silt and sand interlayer is about 12 per cent.

4.2.6 Bedrock

Bedrock was encountered underlying the granular deposits and was cored for lengths of approximately 3.4 m and 3.3 m in Boreholes 18-1 and 18-2, respectively. Photographs of the respective borehole bedrock core samples are presented on Figures B-4 and B-5. The depths to bedrock below ground surface, as inferred from bedrock coring, and the corresponding bedrock surface elevations are summarized in the table below.

Borehole	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)	Comments
18-1	8.6	240.1	Cored for 3.4 m
18-2	7.8	239.1	Cored for 3.3 m

Based on a review of the bedrock core samples, the bedrock consists of meta-diorite with zones of pegmatite (granite). In general, the bedrock core samples are described as fresh, slightly foliated, grey and coarse grained.

The Rock Quality Designation (RQD) measured on the core samples ranges from about 94 per cent to 100 per cent, indicating a rock mass of excellent quality as per Table 3.10 of CFEM (2006)⁵. The Total Core Recovery (TCR) ranges from about 97 per cent to 100 per cent and the Solid Core Recovery (SCR) ranges from about 88 per cent to 97 per cent.

Unconfined compressive (UC) testing was carried on two bedrock core samples of the meta-diorite and indicates the uniaxial compressive strength (UCS) of the tested bedrock samples is about 39 MPa and 47 MPa, indicating the bedrock samples are classified as medium strong (R3, 25 MPa < UCS < 50 MPa), according to Table 3.5 of CFEM (2006)⁵. The laboratory UC tests results are presented on the drillhole records and on Figures B-6A/B and B-7A/B, in Appendix B.

4.2.7 Groundwater Conditions

A standpipe piezometer was installed in Borehole 18-1 and sealed within the organic silt / sand to sand and gravel / sandy silt soil strata to monitor the groundwater level at the site. The water level measured in the standpipe piezometer installed as part of the investigation is summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level* (m)	Groundwater Elevation (m)	Date
18-1	248.7	0.8	247.9	31-Oct-18
		1.0	247.7	29-Nov-18

⁵ Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual (CFEM), 4th Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.

The groundwater level in Borehole 18-2, although potentially influenced by the addition of water for coring, was estimated to be at a depth of 1.5 m below ground surface (Elevation 245.4 m) upon removal of the casing.

The water level of North Driftwood River was about Elevation 245.2 m on November 3, 1982, at about Elevation 245.5 m in November 2018 and about Elevation 244.1 m in January 2019, as measured by others.

It should be noted that the groundwater level is subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

5.0 CLOSURE

This report was prepared by Ms. Anastasia Poliacik, P.Eng. and the technical aspects were reviewed by Sarah Poot, P. Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge Costa, P.Eng., an MTO Foundations Designated Contact and Senior Consultant of Golder, conducted a quality control review of the report.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
NORTH DRIFTWOOD RIVER BRIDGE REPLACEMENT (SITE 39E-0013/B0)
HIGHWAY 11, DRIFTWOOD, ONTARIO
G.W.P. 5282-14-00, W.P. 5282-14-02, MTO AGREEMENT NO. 5017-E-0022**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary design foundation engineering recommendations for replacement of the North Driftwood River Bridge (Site 39E-0013/B0) on Highway 11 in Driftwood, Ontario (G.W.P 5282-14-00, W.P. 5282-14-0), as part of MTO's Agreement No. 5017-E-0022. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced at the site and the results of laboratory testing.

The discussion and recommendations presented are intended to provide the designers with sufficient information to assess preliminary design level feasible alternatives for the bridge replacement. The foundation investigation report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and their designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.1 General

The existing five-span bridge was constructed in 1942 and is about 89 m long and about 10 m wide. The bridge abutments and piers were supported on 1.9 m (6 feet 3 inches) overall diameter caissons formed from twelve sections of driven sheet piling with bent flanges, filled with reinforced concrete. The original abutments failed due to excessive lateral earth pressures and in 1986/1987, the original abutments were reinforced with 16 HP 310X110 driven piles (8 piles at each abutment) each in the area of the original caissons and the pile cap reconstructed to re-incorporate the caissons/piles at the original design level.

The original ground surface at the site is at about Elevation 249 m and the Highway 11 grade at the site ranges from Elevation 252 m at the west end to Elevation 251 at the east end of the bridge, indicating the embankments are about 2 m to 3 m high. The existing embankment side slopes are inclined at about 2 Horizontal: 1 Vertical (2H:1V). Based on observations of the embankment slopes and roadway platform at the time of the subsurface investigation, the side slopes appear to be performing adequately with no visual evidence of surficial sloughing or slope instability nor settlement of the roadway. Additional details of the existing foundations elements are summarized below.

Foundation Element	Road Grade (m)	Foundation Type	Estimated Pile/Caisson Length (m)	Estimated Founding/Tip Elevation (m)	Factored Geotechnical Resistances (kN)
Existing Abutments	251.7 (West), 250.9 (East)	Two 0.6 m diameter driven shafts (caissons) ¹	9.1	241 ²	Not Available
		Battered steel 310 X 110 steel H-Piles	>7.0	Below 244	1,200 at ULS 800 at SLS
Existing Piers	NA	Two 1.7 m diameter driven shafts (caissons) ¹	10.7	245 ²	Not Available

Notes:

1. Original caissons consist of driven interlocking steel sheet piles forming an overall 1.9 m cylindrical/star shaped enclosure, filled with reinforced concrete.

2. Elevations estimated from Sheet 1942 Sheet Pile Record.

Details of the proposed bridge replacement structure, including alignment location, structure width, structure, length, structure span, and grade raise, were not available at the time of preparation of this report. However, we understand that the structure will be in the general vicinity of the existing bridge (i.e. just to the north or south, or on/shifted from the existing alignment) and will be of similar length and at a similar grade, although the number of spans is not yet decided.

6.2 Consequence and Site Understanding Classification

The proposed structure has the potential to impact alternative transportation corridors. In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code and its Commentary (CHBDC 2014), the proposed bridge and its foundation system is classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree of site understanding in Section 6.5 of the 2014 CHBDC, the level of confidence for design is considered to be a “low degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ , from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Table 6.2 of the 2014 CHBDC have been used for design.

6.3 Foundation Options

The following shallow and deep foundations options may be considered for the abutments and piers of the bridge replacement structure. A summary comparison of the advantages and disadvantages, relative costs and risks/consequences associated with each option is provided below, and a comparison of the options is presented in Table 1.

- **Shallow foundations – spread/strip footings:** Shallow foundations comprised of spread or strip footings, founded on the compact to very dense sand to sand and gravel deposit or on a compacted granular pad, are feasible for support of the new abutments and piers; however, this foundation type would preclude the use of integral abutments. This option provides lower factored geotechnical resistances for design than that of deep foundations and would require large excavations and dewatering for construction.
- **Deep foundations – driven steel H-piles or pipe (tube) piles:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments and piers and this foundation type would permit design of

conventional abutments, semi-integral abutments (for H-piles and pipe piles) or integral abutments (for H-piles only). However, due to the presence of cobbles and boulders at the site, there is a risk of pile advancement obstructions. Should deep foundations be adopted, driving shoes should be considered a required element for either H-Piles or pipe piles.

- **Deep foundations – drilled shaft (caissons):** Drilled shafts are feasible for the support of the abutments and piers; however, this option would preclude integral abutment design. This option would be more expensive than shallow foundations or driven pile foundations, although fewer caisson elements would be required in comparison to the number of driven steel piles that would otherwise be required. If caissons are adopted for support of the abutments and piers, temporary liners would be required during construction to control potential ground losses and/or disturbance of the caisson side walls and base. Advancement difficulties would also result from the presence of cobbles/boulders at the site.

Based on the above considerations and as detailed in Table 1, both shallow and deep foundation options are considered feasible for the support of the new abutments and piers, although drilled shafts (caissons) are preferred and recommended from a foundations perspective for all foundation elements of the replacement structure. Should an integral abutment design be a requirement of the replacement structure, then steel H-piles placed within predrilled holes and socketed into the bedrock could be considered.

Recommendations for any potential piers could not be provided at this time as there were no boreholes advanced within the river but are anticipated to be similar to those at the abutments, described below.

6.4 Shallow Foundations

6.4.1 Founding Elevation

Spread footings may be considered for the support of the new abutments. It is recommended that the footings be founded on the compact to very dense sandy silt to sand and gravel deposit, or on compacted granular pads. The highest founding elevations recommended for preliminary design of footings are summarized below.

Foundation Element	Estimated Final Road Elevation ¹ (m)	Recommended Founding Elevation (m)	Founding Stratum
West Abutment	251.7	243.5	Compact Sand and Gravel
East Abutment	250.9	244.5	Dense Sand and Gravel

Note: 1. Estimated to be same as the existing road elevation.

Compacted granular pads may be considered to raise the founding level and minimize the height of the abutment walls, such that the abutment foundations could be “perched” in the new or existing approach embankments. In this case, the compacted granular pad should have a minimum thickness of 2 m. Any existing fill, organic soils and/or loose soils within the zone of influence below the compacted granular pad should be sub-excavated and replaced with engineered fill, such that the pad is founded on the native sand and gravel deposit at the elevations given above. The pad should consist of OPSS.PROV 1010 (*Aggregates*) Granular ‘A’ or Granular ‘B’ Type II material extending at least 1 m beyond the edges of the footing(s), then outward and downward at 1H:1V. The granular fill should be placed in accordance with OPSS.PROV 501 (*Compacting*).

All footings should be provided with a minimum 2.6 m thick layer of soil cover for frost protection as interpreted from OPSS 3090.100 (*Frost Penetration Depths for Northern Ontario*), as measured vertically from ground surface and perpendicular to the face of the abutment slope to the edge of the underside of the footing. If

adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.4.2 Geotechnical Resistances

Assuming a bridge width of 10 m, strip footings placed on the native soils at or below the design elevations given in Section 6.4.1, should be designed based on the factored ultimate geotechnical resistances and factored serviceability geotechnical resistances (for 25 mm of settlement) given below.

Foundation Element	Footing Width (m)	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
West Abutment	4 to 5	650	225
East Abutment	4 to 5	625	235

Alternatively, strip footings founded on compacted Granular 'A' or Granular 'B' Type II pads should be designed based on a factored ultimate geotechnical resistance of 650 kPa and a factored serviceability geotechnical resistance (for 25 mm of settlement) of 300 kPa.

The geotechnical resistances should be reviewed if the selected footing width or founding elevations differ from those given above. The preliminary factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be considered in accordance with Section 6.10.4 of the 2014 CHBDC.

The footing subgrade should be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS.PROV 902 (*Excavating and Backfilling Structures*) to check that all unsuitable materials have been removed. Where sub-excavation is required, the sub-excavated area should be backfilled with granular material meeting OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II, placed and compacted in accordance with OPSS.PROV 501 (*Compacting*), or the thickness of the footing may be increased to the full excavation depth.

Dewatering will be required during construction of the spread/strip footings, as discussed in Section 6.9.3.

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the new concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the 2014 CHBDC. For cast-in-place concrete footings constructed directly on native soils or on a compacted granular pad, the sliding resistance may be calculated based on the unfactored coefficient of friction, $\tan \Phi'$ or δ respectively, which can be taken as follows, as interpreted from NAVFAC (1982):

- Cast-in-place footing to native sand and gravel deposit: $\tan \delta = 0.5$
- Cast-in-place footing to compacted granular pad (Granular 'A' or Granular 'B' Type II): $\tan \delta = 0.6$

6.5 Steel H-Pile or Pipe Pile Foundations

6.5.1 Founding Elevations

Steel H-piles or pipe piles driven to bedrock may be considered for the support of the new abutments. The tip elevations recommended for preliminary design of driven piles are summarized below. Pile cap elevations equal to or higher than those provided below are acceptable for design from a foundations perspective as long as adequate soil cover is provided to the pile caps for frost protection.

Foundation Element	Estimated Underside of Pile Cap ¹ (m)	Estimated Design Pile Tip Elevation (m)	Estimated Pile Length (m)
West Abutment	249	240	9
East Abutment	248	239	9

Note: 1. Estimated to be about 2.6 m below the existing road elevation (for frost protection).

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the soil deposits, as encountered in Borehole 18-2. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a slightly higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes or flange plates for protection during driving in accordance with OPSD 3000.100 (*Foundation Piles – Steel H-Pile Driving Shoe*) and driven in accordance with OPSS.PROV 903 (*Deep Foundations*). Where cobbles/boulders may be encountered, driving shoes such as Titus Standard “H” Bearing Pile Points are preferred over flange plates. If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II (*Steel Tube Pile Driving Shoe*). It should be noted that at detail design it may be warranted to specify that the H-piles be fitted with Oslo Points (as per OPSD 3000.201) or equivalent Titus Pile Points to seat the piles on bedrock, especially if it is found that the bedrock surface is sloping. The pile driving note to be included in the foundation design drawing is Note 5 of Section 3.3.3 in MTO’s Structural Manual (2014):

- Piles to be driven to bedrock.

All pile caps should be provided with a minimum 2.6 m of soil cover for frost protection interpreted from OPSD 3090.100 (*Frost Penetration Depths for Northern Ontario*), as measured vertically from and perpendicular to the face of the abutment slope to the edge of the underside of the pile cap. If adequate soil cover cannot be provided for the pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.5.2 Geotechnical Resistances

Driven steel H-piles (HP 310x110) and closed-end, concrete-filled 324 mm outside-diameter (O.D.) steel pipe piles having a minimum wall thickness of 9.5 mm, advanced to the recommended pile tip elevations given in Section 6.5.1, should be designed based on the factored ultimate geotechnical resistances and factored serviceability geotechnical resistances (for 25 mm of settlement) given below.

Foundation Element	Pile Type	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance ¹ (kN) (for 25 mm of Settlement)
West Abutment	HP 310x110	2,100	-
	324 mm O.D. Pipe	2,000	-
East Abutment	HP 310x110	2,100	-
	324 mm O.D. Pipe	2,000	-

Note: 1. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance (at ULS) and as such, the SLS condition does not apply.

Pile installation should be in accordance with OPSS.PROV 903 (*Deep Foundations*). For piles driven to bedrock, the contractor should be alerted that the set criteria is to be "Refusal" – less than 12.5 mm penetration for 10 blows of the driving hammer at full energy.

6.6 Drilled Shafts (Caissons)

6.6.1 Founding Elevations

Drilled shaft (caissons) socketed a nominal length (1.0 m) into bedrock may be considered for support of the abutments. The founding elevations recommended for preliminary design of drilled shafts (caissons) are summarized below.

Foundation Element	Estimated Underside of Pile Cap ¹ (m)	Estimated Design Bottom of Caisson Elevation (m)	Estimated Caisson Length (m)*
West Abutment	249	239	10
East Abutment	248	238	10

Note: 1. Estimated to be about 2.6 m below the existing road elevation (for frost protection); but may be constructed at the underside of the superstructure.

If drilled shafts are adopted, appropriate equipment and procedures will be required to penetrate into the granite bedrock. If adopted, a temporary liner should be used to support the overburden soils during construction to minimize disturbance to the side walls due to groundwater pressure / seepage, as well as to permit inspection of the base via shaft inspection device. In addition, placement of concrete by tremie methods is recommended based on the groundwater level at the site (i.e. at about Elevation 248 m).

All buried caisson/pile caps should be provided with a minimum 2.6 m of soil cover for frost protection as interpreted from OPSP 3090.100 (*Frost Penetration Depths for Northern Ontario*), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the pile cap. If adequate soil cover cannot be provided for the pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration. Alternatively, the pile caps could be constructed at the underside of the superstructure.

6.6.2 Geotechnical Resistances

Drilled shafts (caissons) advanced to the recommended founding elevations given in Section 6.6.1, should be designed based on the factored ultimate geotechnical resistances and factored serviceability geotechnical resistances (for 25 mm of settlement) given below.

Foundation Element	Shaft Diameter (m)	Factored Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance ¹ (kN) (for 25 mm of Settlement)
West Abutment	0.9	9,000	-
	1.2	12,000	-
East Abutment	0.9	9,000	-
	1.2	12,000	-

Note: 1. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance (at ULS) and as such, the SLS condition does not apply.

The performance of the drilled shafts will depend to a large degree upon the final cleaning and verification of the condition of the base of the drilled shaft as these are considered end-bearing foundation units. As such, the base of each drilled shaft excavation must be cleaned to remove all loose cuttings or softened material to ensure that the concrete is in intimate contact with the bedrock subgrade, consistent with OPSS.PROV 903 (*Deep Foundations*). A qualified geotechnical engineer should be retained during construction to inspect the drilled shafts to verify that the conditions encountered are consistent with the information obtained from the boreholes and to confirm the base elevation of the drilled shaft and cleanliness. To allow for visual remote inspection of the base of the drilled shafts (via a shaft inspection device such as a video camera), the drilled shaft excavations must be lined.

6.7 Lateral Earth Pressures for Design of Abutment and Wing Walls

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

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

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II, should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*).
- 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 2014 CHBDC Section 6.12.3 and Figure 6.6. Hand-operated compaction equipment should be used to compact the backfill soils immediately behind the walls as per OPSS.PROV 501. Other surcharge loadings should be accounted for in the design, as required.

- For restrained walls, granular fill should be placed in a zone with the width equal to at least 2.6 m behind the back of the wall as shown on Figure C6.20(a) of the Commentary to the 2014 CHBDC. For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn flatter than as shown at 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing or pile cap on Figure C6.20(b) of the Commentary to the 2014 CHBDC.

6.7.1 Static Lateral Earth Pressures for Design

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

- For a restrained wall, the pressures are based on the embankment fill behind the granular backfill zone, and the following parameters (unfactored) may be used assuming the use of earth fill; values are also provided for the use of Granular B Type I, which is recommended for this site:

Material	Earth Fill	Granular B Type I or SSM
Soil Unit Weight:	20 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.33	0.31
At rest, K_o	0.50	0.47

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

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

- 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.
- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the 2014 CHBDC.

6.8 Approach Embankments

6.8.1 Subgrade Preparation and Embankment Construction

Prior to construction of new approach embankments all topsoil, loose/soft fill, and organic materials (such as the silt to organic silt at this site) must be removed from within the embankment footprint, as organic silt/clayey organic silt strata that are left in place will undergo long-term settlement due to decomposition/decay and, therefore, future maintenance of the embankments/roads would be required.

Fill for construction of the new embankment sections/widenings should consist of Select Subgrade Material (SSM) or Granular 'B' Type I or Type II meeting the specifications of OPSS.PROV 1010 (*Aggregates*). The embankment fill should be placed and compacted in accordance with OPSD 208.010 (*Benching of Earth Slopes*), OPSS.PROV 501 (*Compacting*) and OPSS.PROV 206 (*Grading*). Where adopted, embankment side slopes should be constructed no steeper than 2H:1V in granular fill to achieve the required factor of safety for global stability and should provide mitigation of surficial erosion.

6.8.2 Global Stability

A global slope stability analysis was carried out for new approach embankment side slopes, using the commercially available program SLIDE 2018, developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure.

The stability analysis assumes 5 m high embankments constructed at 2H:1V using SSM or Granular 'B' Type I, with a groundwater level at a depth of 1.0 m below existing ground surface. The results indicate that the approach embankment sections will have a Factor of Safety of 1.6, which is less than the required minimum Factor of Safety when a resistance factor for a "low degree of site understanding" is applied, as per Table 6.2 of the 2014 CHBDC and as such, mitigation measures may be required. However, upon further foundation investigation being carried out, a "typical degree of site understanding" would likely be applicable, such that a minimum Factor of Safety of 1.5 would be required, as per Table 6.2 of the 2014 CHBDC, and therefore stability mitigation measures would likely not be required. Once the preliminary embankment details are available, and upon completion of the detailed field investigation, a detailed stability analysis should be carried out to confirm that the required slope stability Factor of Safety is achieved.

6.8.3 Settlement

Settlement analyses were carried out for the construction of new sections or widened sections of the approach embankments (adjacent to the existing approach embankments) using the commercially available program SETTLE3D (Version 4.0), developed by Rocscience Inc.

The settlement associated with construction of new 5 m high new approach embankment section is estimated to be between about 80 mm to 150 mm. The source of settlement at this site is considered to include immediate settlement of the subgrade granular soil strata only and therefore the estimated settlement is expected to occur during and shortly after construction. However, in addition to the settlements induced by the embankment construction, if organic materials such as the organic silt/clayey organic silt strata are left/trapped below the new embankment fills, then long-term settlement of the organic deposit(s) would occur due to decomposition/decay/and consolidation of the organics within the deposit and therefore, future maintenance of the embankments/roads would be required.

6.9 Construction Considerations

6.9.1 Open-Cut Excavations

Excavations for spread/strip footings and/or pile caps will extend to depths of about 4.6 m below existing ground surface (or 7.6 m below the existing Highway 11 grade), depending on the final structure alignment. Open-cut excavations must be carried out in accordance with the guidelines outlined in the most recent version of the Occupational Health and Safety Act and Regulation for Construction Activities. The existing fill (above the water table) is classified as Type 3 soil and the silt to organic silt/clayey organic silt, sandy silt to sand and gravel (below the groundwater table) is classified as Type 4 soil. Temporary excavations (i.e. those that are open for a relatively short time period) within Type 3 soils should be made with side slopes no steeper than 1H:1V and temporary excavations within Type 4 soils should be made with side slopes no steeper than 3H:1V.

6.9.2 Temporary Protection Systems

Temporary protection systems may be required to facilitate construction of the new abutments (and piers, if applicable). Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the temporary protection systems should meet Performance Level 2 as specified in OPSS.PROV 539, provided that the existing structure and any adjacent utilities can tolerate this magnitude of deformation. The selection and design of the protection system will be the responsibility of the Contractor.

6.9.3 Groundwater and Surface Water Control

It is anticipated that the groundwater level at the site will not be any lower than the river water level, and likely it would be between about Elevations 247 m and 248 m.

Excavations for spread/strip footings will extend to depths of between about 3.2 m and 3.8 m below the groundwater level and will therefore require active dewatering to allow for construction in dry conditions. Similarly, excavations for embankment construction will extend to depths of about 3.2 m to 3.8 m below the groundwater level and will also require active dewatering to allow for construction in dry conditions; or settlement and potentially stability mitigation measures would be required if organic materials are left in place.

At the abutments, assuming the new pile caps will be constructed at/near the existing pile cap elevations, excavations for the pile caps will not extend below the groundwater level. However, it is anticipated that any water inflow into pile cap excavations can be handled by pumping from properly filtered sump pumps placed at the base of the excavation. If drilled shafts are adopted, temporary liners should be used to support the overburden soils. Balancing groundwater pressures during construction by utilizing a head of water or bentonite drilling slurry inside the temporary liner will be required.

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations.

6.9.4 Obstructions

As encountered during the field investigation at Borehole 18-2, cobbles and/or boulders should be expected within the sand to sand and gravel deposit, which may affect the installation of deep foundations as well as temporary protection systems. As discussed in Section 6.5.1, it is recommended that driving shoes or pile points be used on all steel H-piles, if this foundation type is adopted, to facilitate driving through the overburden soils and seat the pile on bedrock.

6.10 Recommendations for Future Work During Detailed Design

Once the preliminary details of the replacement structure are defined, it is recommended that a detailed field investigation be carried out such that boreholes are advanced within or as close as practical to each abutment and pier foundation element, consistent with MTO's Terms of Reference (TOR) requirements typically stated in Section 7.7 – Foundation Engineering and details specified in accompanying Tables 7.7a and 7.7b. The boreholes should be extended to bedrock and bedrock should be cored for a minimum depth of 3 m. It is also recommended that additional groundwater monitors be installed to confirm the groundwater level on either side of the river/bridge.

Based on the findings from the preliminary field investigation, the detailed foundation investigation should include requirements for:

- Assessing the extent and depth of the organic soil deposits within the footprint of the proposed embankment sections/widening;
- Obtaining additional bedrock core samples for geotechnical laboratory analysis, specifically uniaxial compressive strength, in order to optimize the geotechnical resistance values provided herein;
- Obtaining additional samples of the organic materials for organic content determination;
- Obtaining relatively undisturbed thin-walled Shelby tube samples of the organic materials and carry out laboratory consolidation testing to estimate magnitude and rate of consolidation, in the event that consideration is given to not fully sub-excavating/replacing the organic deposit;
- Carrying out additional stability and settlement analyses;
- Analyzing for parameters used to assess the soil corrosion potential to buried steel and deterioration of concrete;
- Evaluating the seismic Site Class and seismic hazard values;
- Recording the occurrence of grinding of the augers during advancement of the boreholes to assess for the presence of obstructions, such as cobbles and boulders and core the larger sized obstructions where possible, as such obstructions may affect excavations and the installation of driven steel H-piles and caisson foundations;
- Installing an additional standpipe piezometer at the one of the abutment locations to allow for groundwater level monitoring, together with measuring the groundwater level in the existing standpipe piezometer in Borehole 18-1; and,
- Abandoning the existing standpipe piezometer in Borehole 18-1 and any new piezometer, subsequent to taking water level readings a few weeks post-installation or confirm that the standpipe piezometer may be left in place to allow for water level monitoring immediately prior to construction and subsequent abandonment by the construction contractor.

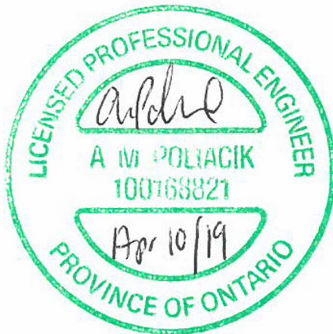
Further, the Detail Design of the approach embankments should include an assessment of alternative measures for mitigating settlement and instability, in the event that the organic deposit is left in place (not fully sub-excavated) under the embankment footprint.

7.0 CLOSURE

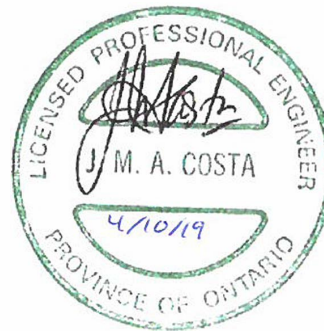
This report was prepared by Ms. Anastasia Poliacik, P.Eng., a geotechnical engineer. Mr. Jorge Costa, P.Eng., a MTO Designated Contact and Senior Consultant with Golder, conducted a technical and quality control review of the report.

Signature Page

Golder Associates Ltd.



Anastasia Poliacik, P.Eng.
Geotechnical Engineer



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MTO Foundations Designated Contact, Senior Consultant

AMP/SEMP/JMAC/cr;ljb

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Canadian Standards Association (CSA), 2006. *Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6 06. CSA Special Publication, S6.1 06*.

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Structural Manual 2014. Ministry of Transportation Provincial Highways Management Division, Highways Standards Branch, Bridge Office. August.

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Ontario Provincial Standard Specifications (OPSS) and Drawings (OPSD)

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation Piles, Steel H-Pile Driving Shoe
OPSD 3001.201	Foundation Piles, Steel HP 310 Oslo Point
OPSD 3001.100	Foundation Piles, Steel Tube Pile Driving Shoe
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario

ASTM International:

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT)
ASTM D2573M-15	Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)
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Ontario Occupational Health and Safety Act:

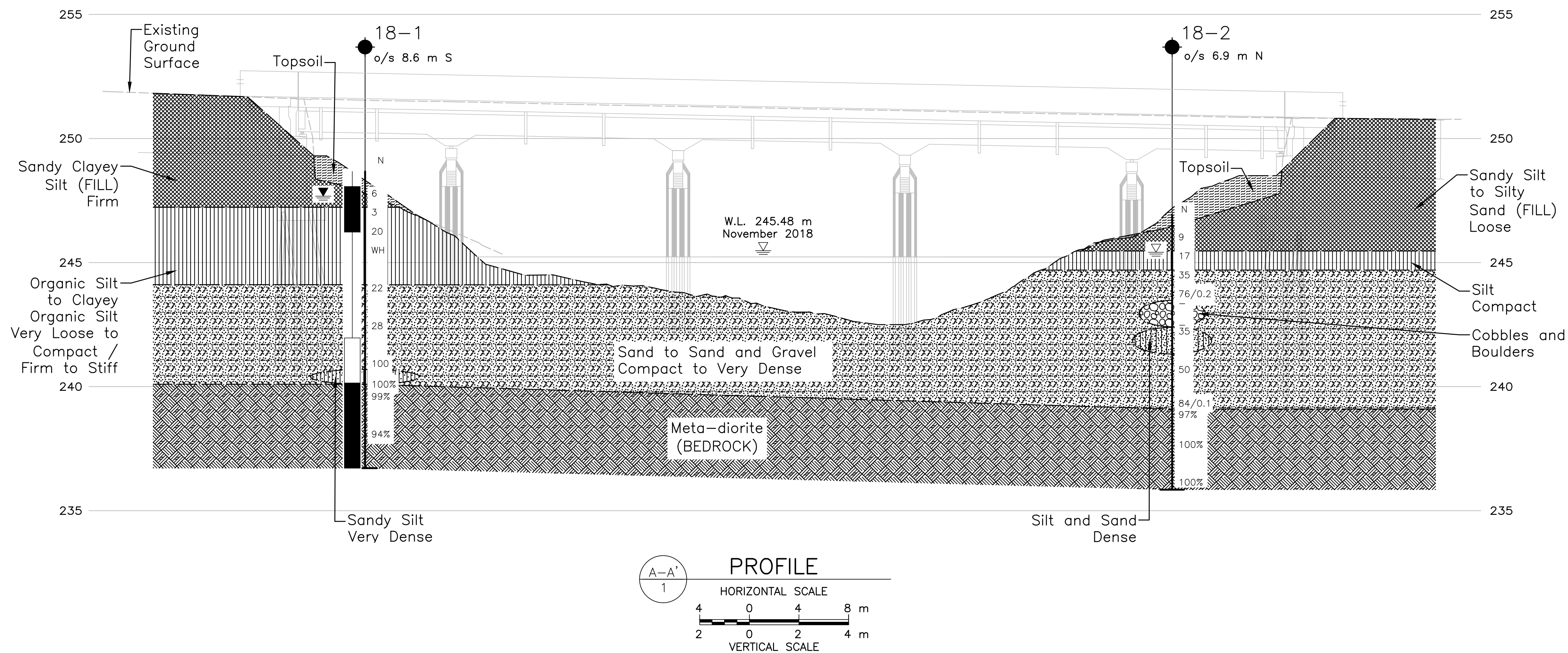
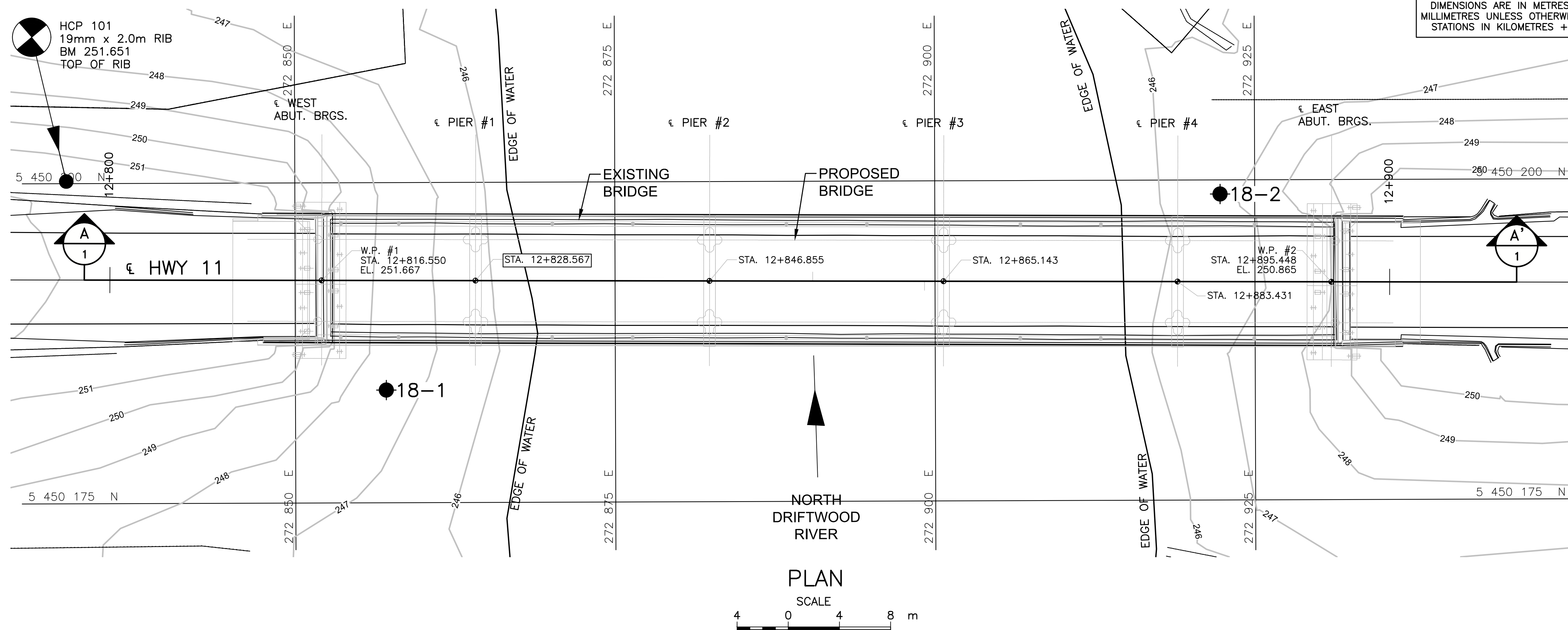
Ontario Regulation 213	Construction Projects (as amended)
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Proprietary Software:

Rocscience Inc.	SLIDE 2018
Rocscience Inc.	SETTLE3D (V.4.0)

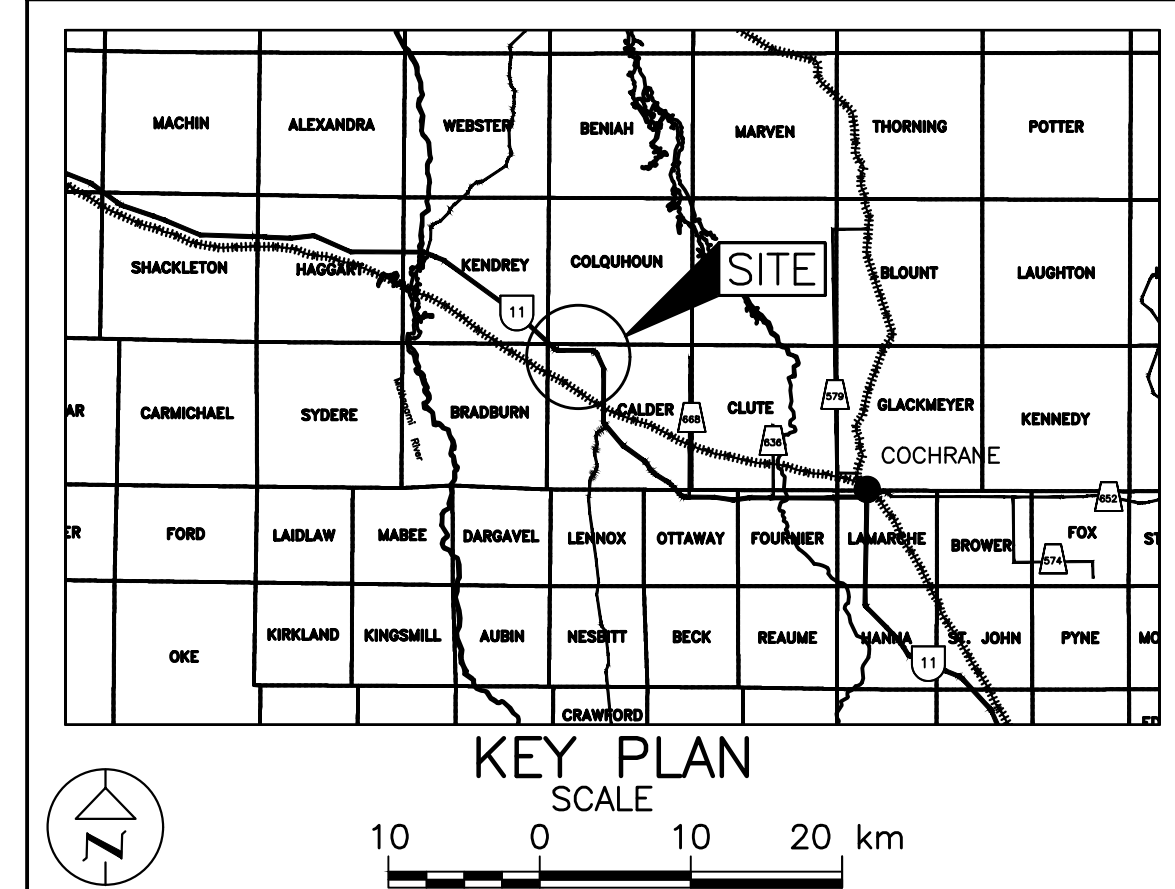
TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Shallow Foundations: Spread/strip footings founded on the compact to very dense sand to sand and gravel deposit or on a compacted granular pad	Feasible for support of the abutments and potential by the pier(s); may require temporary protection for staged construction depending on final bridge alignment relative to existing bridge.	<ul style="list-style-type: none">Permits semi-integral abutment configuration.	<ul style="list-style-type: none">Precluded the use of integral abutments.Provides lower geotechnical resistances compared to deep foundation options.Requires large excavations and dewatering for construction.	<ul style="list-style-type: none">Conventional excavation and construction techniques.	<ul style="list-style-type: none">Lower relative cost than deep foundations, although cost could be increased substantially due to dewatering requirements (as experienced by MTO on projects requiring groundwater control).
Driven Deep Foundations: Steel H-piles or Steel Pipe piles founded within bedrock	Feasible for support of the abutments and potential pier(s); may require temporary protection for staged construction depending on final bridge alignment relative to existing bridge.	<ul style="list-style-type: none">Permits design of conventional abutments, semi-integral abutments (for H-piles and pipe piles) or integral abutments (for H-piles only)Abutment pile caps could be maintained higher than shallow foundations, potentially reducing excavation depth and associated protection system requirements; however, this may not be feasible for connecting to existing substructure.Higher bearing resistances than for shallow foundations	<ul style="list-style-type: none">Larger/specialized equipment required for installation of piles than for construction of shallow foundations.Risk of advancement difficulties due to encountered obstructions (cobbles and boulders); driving shoes would be required.	<ul style="list-style-type: none">Conventional construction methods for driven piles; driving shoes would be required due to potential obstructions.Installation of steel H-piles into pre-drilled holes and socketed into bedrock may allow for design of integral abutments.	<ul style="list-style-type: none">Estimated cost is approximately \$250/m length for pile installation and \$600/m3 for pile cap constructionPotentially less costly maintenance over life of the structure for integral abutment design than semi-integral abutment structures; but additional costs would be incurred for predrilling holes/sockets into bedrock.
Drilled Deep Foundations: Drilled shafts founded on bedrock	Feasible for support of abutments and potential pier(s); may require temporary protection for staged construction depending on final bridge alignment relative to existing bridge.	<ul style="list-style-type: none">Abutment caisson caps could be maintained higher than steel H-pile pile caps or footings founded on native soils, potentially reducing excavation depth and associated protection system requirements; however, this may not be feasible for connecting to existing substructure.Higher bearing resistances than for driven deep foundations, requiring fewer elements.Smaller working area than for multiple rows of steel H-piles.	<ul style="list-style-type: none">Precludes integral abutment design.Temporary liners would be required during construction to control for ground and groundwater control.Risk of advancement difficulties due to encountered obstructions (cobbles and boulders).	<ul style="list-style-type: none">Conventional construction methods for drilled shaft foundations, plus specialized equipment and procedures for socketing into bedrock; temporary liners required for ground and groundwater control.	<ul style="list-style-type: none">Estimated cost is approximately \$1000/m length for caisson installation and \$600/m3 for pile cap construction (if pile caps are adopted at the pier); this cost expected to be higher to account for temporary liners.








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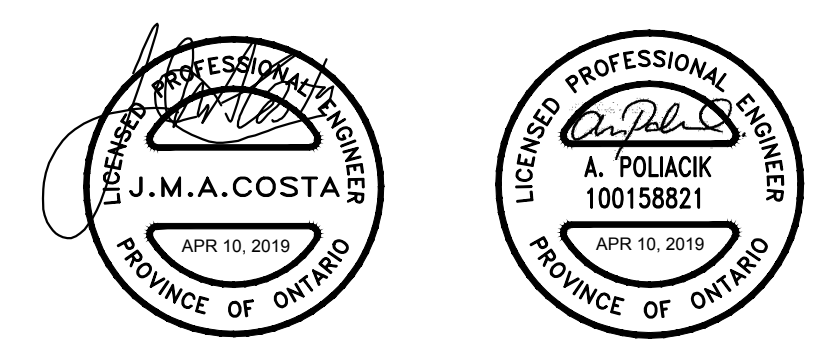
CONT No. WP No. 5282-14-02
NORTH DRIFTWOOD RIVER BRIDGE REPLACEMENT HIGHWAY 11 BOREHOLE LOCATIONS AND SOIL STRATA



LEGEND

	Borehole
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
	WL in piezometer, measured on NOV 29, 2018
	WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 12)			
No.	ELEVATION	NORTHING	EASTING
18-1	248.7	5450183.8	272857.1
18-2	246.9	5450198.9	272922.3



NOTES

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

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

REFERENCE
Base plans provided in digital format by CALLON DIETZ, drawing file nos.
b04690011001.dwg, received Feb 15, 2019.

[illegible]

APPENDIX A

**Record of Borehole and Drillhole
Sheets**

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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



1 OF 2 **METRIC**

CHECKED BY JMAC

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

S:\SUD-MTO\001 S:\CLIENTS\MT\HWY11\02 DATA\GINT\18104224-RUSSEL AND WABBLER CULVERTS\18104224.GPJ GAL-MISS.GDT 3-7-19 TR

PROJECT <u>18104224 / WO#4</u>				RECORD OF BOREHOLE No 18-1				2 OF 2 METRIC			
G.W.P. <u>5282-14-00</u>				LOCATION <u>N 5450183.8; E 272857.1 NAD83 MTM ZONE 12 (LAT. 49.188917; LONG. -81.438254)</u>				ORIGINATED BY <u>MA/SK</u>			
DIST <u>NE</u> HWY <u>11</u>				BOREHOLE TYPE <u>Wash Boring; NW Casing; NQ Rock Coring</u>				COMPILED BY <u>SB/AMP</u>			
DATUM <u>Geodetic</u>				DATE <u>October 31, 2018</u>				CHECKED BY <u>JMAC</u>			

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W			W _L								
12.0	END OF BOREHOLE NOTES: 1. Groundwater measured in standpipe piezometer as follows: <table style="margin-left: 40px;"> <tr> <td>Date</td> <td>Depth(m)</td> <td>Elev. (m)</td> </tr> <tr> <td>31-Oct-18</td> <td>0.8</td> <td>247.9</td> </tr> <tr> <td>29-Nov-18</td> <td>1.0</td> <td>247.7</td> </tr> </table>	Date	Depth(m)	Elev. (m)	31-Oct-18	0.8	247.9	29-Nov-18	1.0	247.7															
Date	Depth(m)	Elev. (m)																							
31-Oct-18	0.8	247.9																							
29-Nov-18	1.0	247.7																							

PROJECT: 18104224 / WO#4

LOCATION: N 5450183.8; E 272857.1

NAD83 MTM ZONE 12 (LAT. 49.188917; LONG. -81.438254)

INCLINATION: -90° AZIMUTH: —

RECORD OF DRILLHOLE: 18-1

SHEET 1 OF 1

DRILLING DATE: October 31, 2018

DATUM: Geodetic

DRILL RIG: CME 55 Coring, Wash Bore

DRILLING CONTRACTOR: Landcore Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate										BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage										PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular										PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break										BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX METRES	B Angle	DIP w.r.t CORE AXIS	DISCONTINUITY DATA						ROCK STRENGTH INDEX			WEATH- ERING INDEX				Q AVG.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
								TOTAL CORE %	SOLID CORE %					TYPE AND SURFACE DESCRIPTION			Jr	Ja	Jn	R4	R3	R2	R1	W1	W2	W3		W4																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
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DEPTH SCALE

1 : 60



GOLDER

LOGGED: MA/SK

CHECKED: JMAC

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

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: Landcore Drilling

DATUM: Geodetic

1 : 60



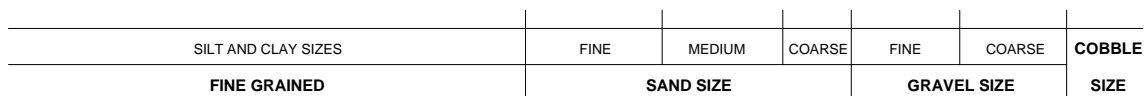
CHECKED: JMAC

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APPENDIX B

Laboratory Test Results and Bedrock Core Photographs

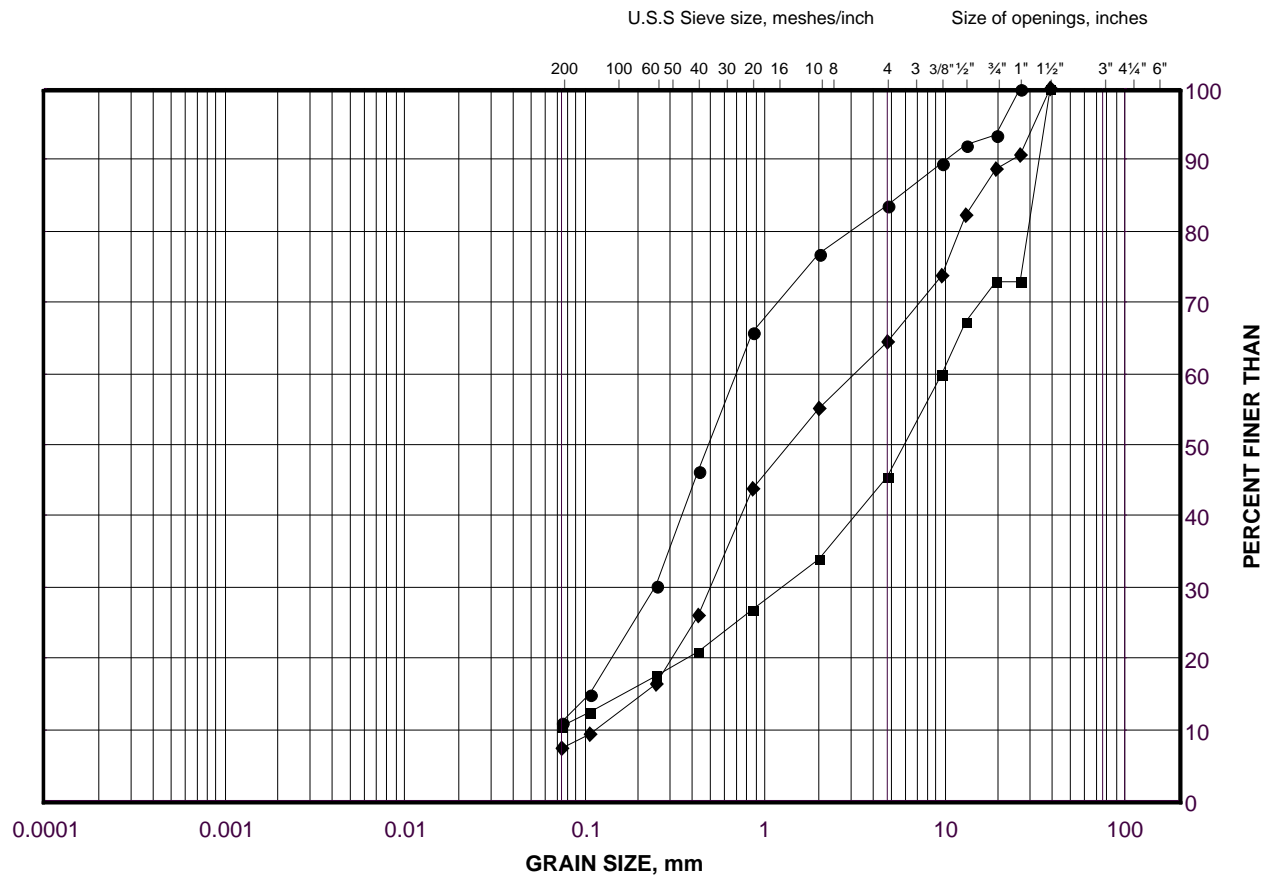
FIGURE B-1



SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	18-2	2	245.1
■	18-1	3	246.1

SAND to SAND and GRAVEL

FIGURE B-2



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	18-1	6	242.3
■	18-2	7	239.2
◆	18-1	7A	240.9

Project Number: 18104224 (WO#4)

Checked By: AMP

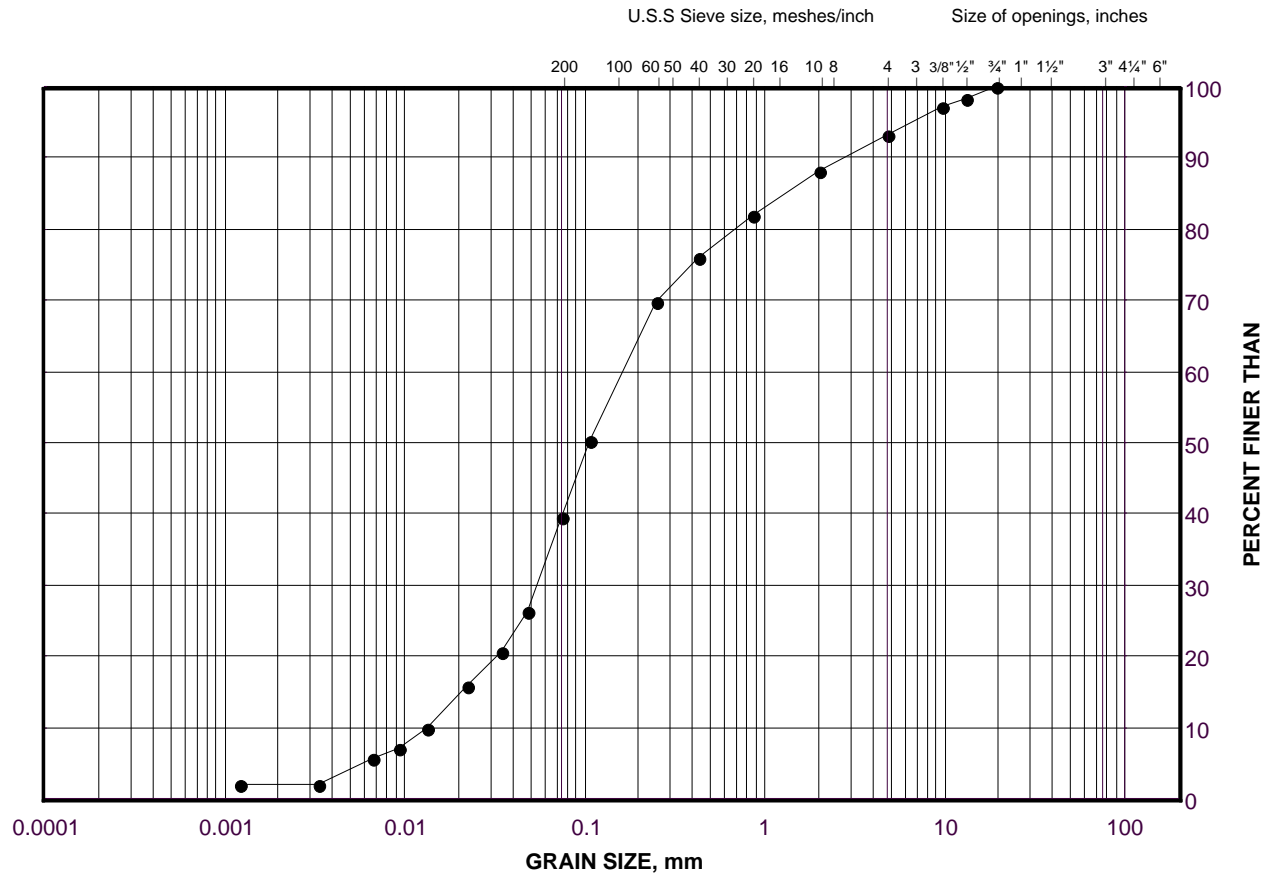
Golder Associates

Date: 28-Feb-19

GRAIN SIZE DISTRIBUTION

SILT and SAND

FIGURE B-3



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	18-2	5	242.1

Project Number: 18104224 (WO#4)

Checked By: AMP

Golder Associates

Date: 28-Feb-19

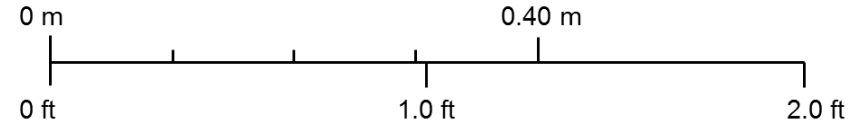
Start of Run No. 1 (8.6 m)

Start of Run No. 2 (9.1 m)




Start of Run No. 3 (10.6 m)

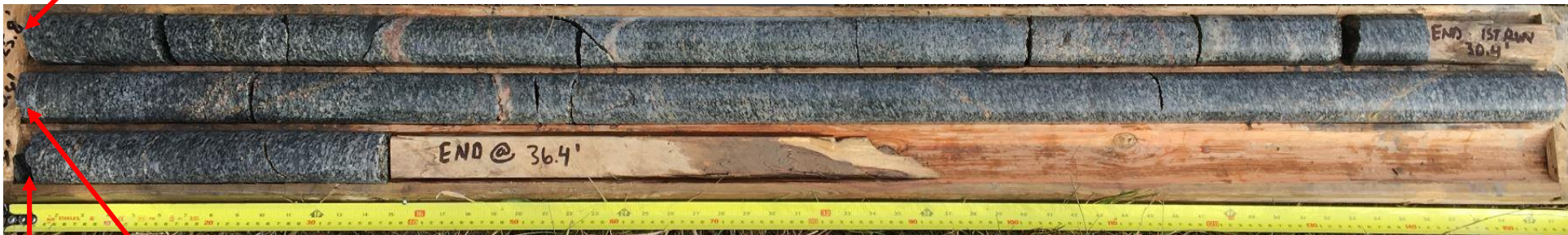
Box 1: 8.6 m to 12.0 m



Scale

PROJECT						
Driftwood River Bridge Replacement (Site 39E-0013/B0) Highway 11, Driftwood, Ontario						
TITLE						
Bedrock Core Photograph Borehole 18-1 (8.6 m to 12.0 m)						
 GOLDER	PROJECT No. 18104224			FILE No. WO#4		
	DRAFT	MJB	2019-02-12	SCALE	NTS	VER. 1.
	CADD	--		FIGURE B-4		
	CHECK	AMP	2019-02-12			
	REVIEW	JMAC	2019-03-04			

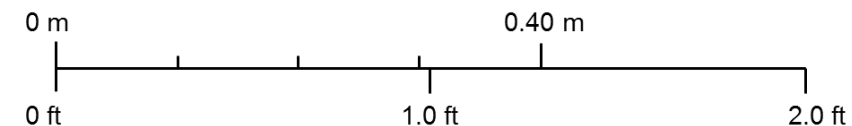
Start of Run No. 1 (7.8 m)




Start of Run No. 2 (9.3 m)

Start of Run No. 3 (10.8 m)

Box 1: 7.8 m to 11.1 m



Scale

PROJECT						
Driftwood River Bridge Replacement (Site 39E-0013/B0) Highway 11, Driftwood, Ontario						
TITLE						
Bedrock Core Photograph Borehole 18-2 (7.8 m to 11.1 m)						
 GOLDER	PROJECT No. 18104224			FILE No. WO#4		
	DRAFT	MJB	2019-02-12	SCALE	NTS	VER. 1.
	CADD	--		FIGURE B-5		
	CHECK	AMP	2019-02-12			
	REVIEW	JMAC	2019-03-04			

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

SAMPLE IDENTIFICATION

PROJECT NUMBER	18104224	SAMPLE NUMBER	(Run #1) 1
PROJECT NAME	MTO 5017-E-0021,22,23/NE LVR	SAMPLE DEPTH, m	8.71-8.93
BOREHOLE NUMBER	18-1	DATE:	November 19, 2018

TEST CONDITIONS

MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.21

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.44	WATER CONTENT, (specimen) %	0.30
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m ³	26.76
SAMPLE AREA, cm ²	17.54	DRY UNIT WT., kN/m ³	26.68
SAMPLE VOLUME, cm ³	183.07	SPECIFIC GRAVITY	-
WET WEIGHT, g	499.71	VOID RATIO	-
DRY WEIGHT, g	498.22		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

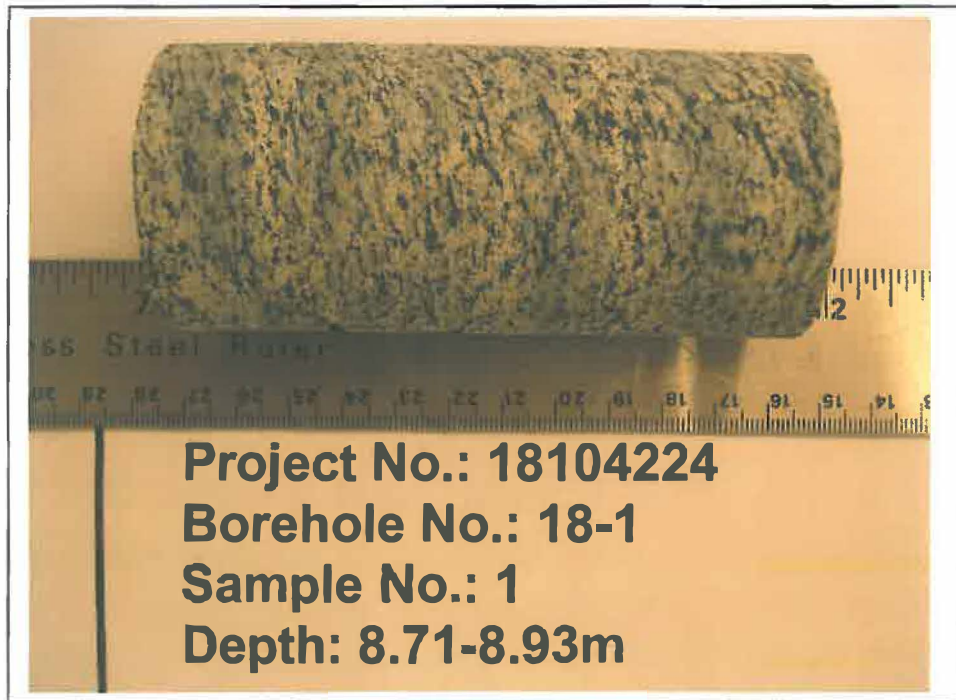
STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	46.9
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REMARKS:

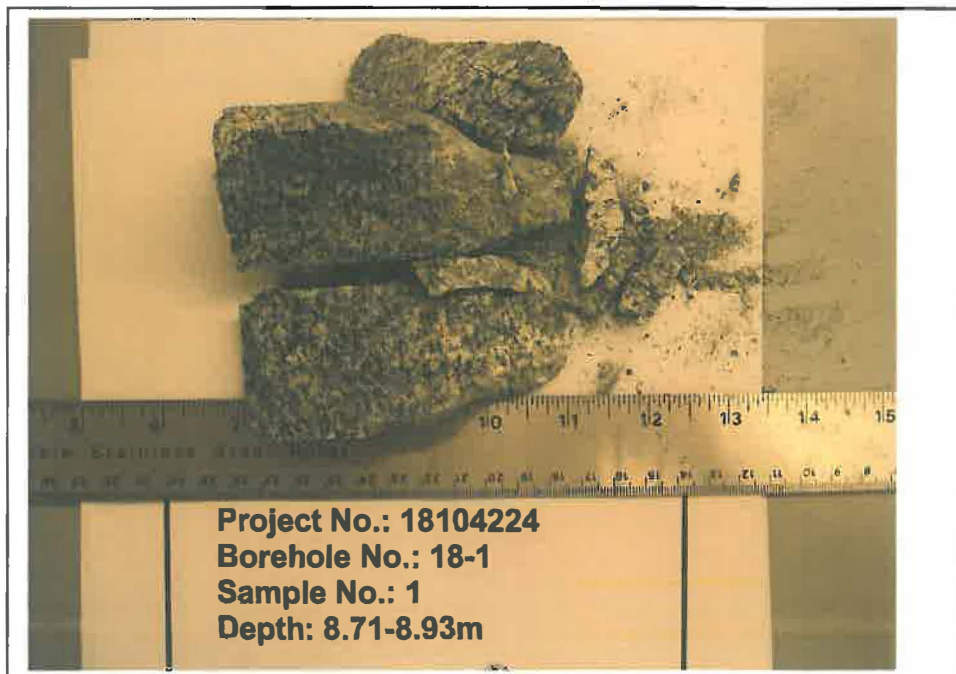
Checked By:



Golder Associates



BEFORE COMPRESSION



AFTER COMPRESSION

Date Nov. 19, 2018
Project 18104224

Golder Associates

Drawn Frank
Chkd. [Signature]

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

SAMPLE IDENTIFICATION

PROJECT NUMBER	18104224	SAMPLE NUMBER	(Run #2) 1
PROJECT NAME	MTO 5017-E-0021,22,23/NE LVR	SAMPLE DEPTH, m	9.25-9.47
BOREHOLE NUMBER	18-2	DATE:	November 19, 2018

TEST CONDITIONS

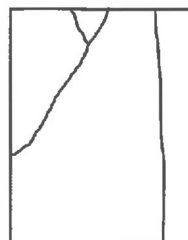
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.21

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.50	WATER CONTENT, (specimen) %	0.30
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m ³	26.69
SAMPLE AREA, cm ²	17.67	DRY UNIT WT., kN/m ³	26.61
SAMPLE VOLUME, cm ³	185.52	SPECIFIC GRAVITY	-
WET WEIGHT, g	505.16	VOID RATIO	-
DRY WEIGHT, g	503.65		

VISUAL INSPECTION

FAILURE SKETCH



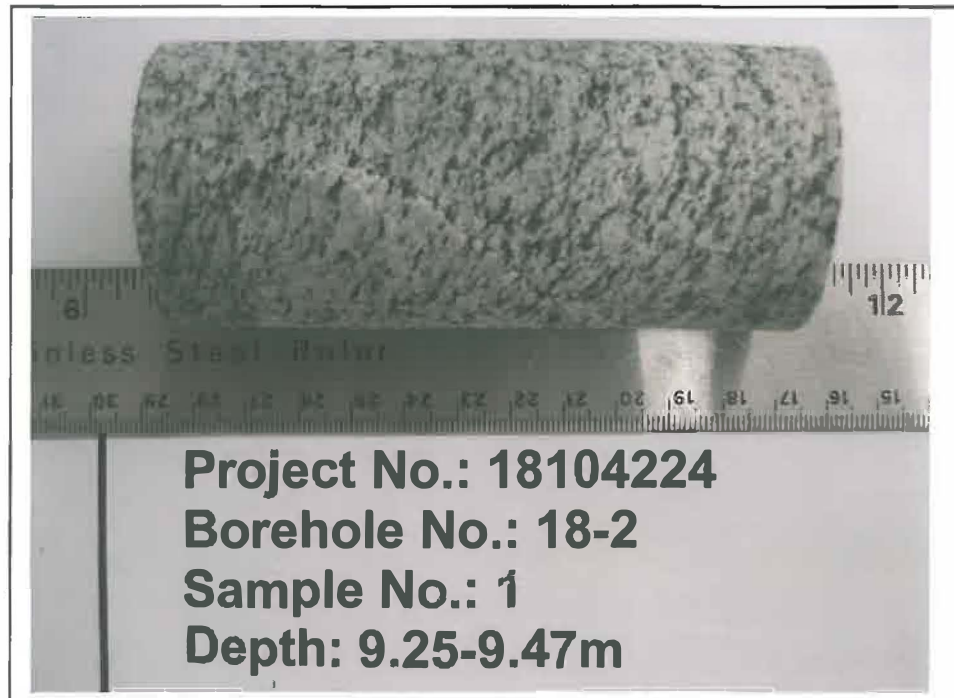
TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	39.0
----------------------	-----	---------------------------	------

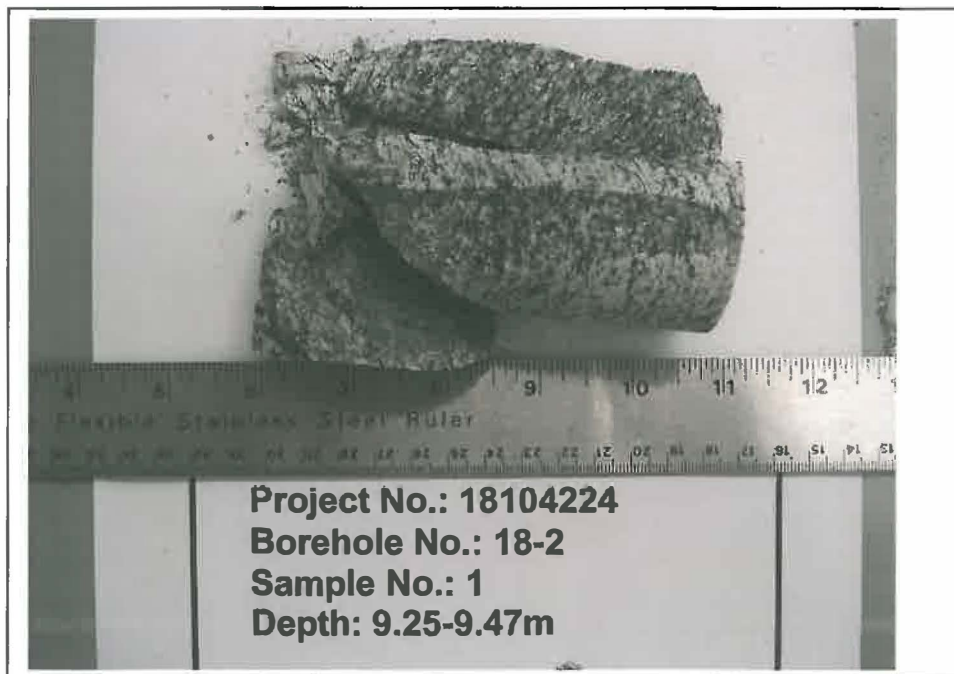
REMARKS:

Checked By: 

Golder Associates



BEFORE COMPRESSION



AFTER COMPRESSION

Date Nov. 19, 2018
Project 18104224

Golder Associates

Drawn Frank
Chkd. [Signature]



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