GOLDER

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Nat River Bridge Replacement (Site No. 46X-0011/B0) Highway 101, Reeves Township, District of Sudbury Ministry of Transportation, Ontario GWP 5180-16-00; WP 5180-16-01

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MINISTRY OF TRANSPORTATION, ONTARIO
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary design Foundation Engineering services for the replacement of the existing Nat River Bridge (Site No. 46X-0011/B0). The Nat River Bridge is located on Highway 101 at Station 18+080 in Reeves Township in the District of Sudbury, Ontario (i.e., approximately 48 km west of the Highway 144 junction). The general location of this section of Highway 101 and the location of the investigation area are presented on Drawing 1.

The Terms of Reference (TOR) for the Foundation Investigation are outlined in MTO's Request for Proposal (Assignment 5017-E-0018, dated October 2017), and the subsequent clarifications/addenda. Golder's originally proposed scope of work is outlined in our proposal dated January 26, 2018, which was included as Section 16.8 in AECOM's technical proposal for this assignment.

At time of the proposal submission, it was assumed that the bridge was to be replaced along the current alignment with a temporary detour bridge being utilized to carry traffic across the Nat River during construction as outlined in the RFP. Based on discussion with AECOM, we understand that a replacement bridge along a new alignment (i.e., approximately 18.6 m north of the existing alignment) has been selected by the AECOM and MTO design team as the preferred option to be carried forward for the Design-Build Ready package. Our revised Foundation Investigation scope of work is outlined in our Change Request 1 letter dated May 14, 2019.

This work has been carried out in accordance with Golder's Supplementary Specialty Plan for Foundation Engineering services for this project, dated April 11, 2018.

2.0 SITE DESCRIPTION

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 from magnetic north shown on the drawing. For the purpose of this report, Highway 101 is oriented in a west-east direction with the Nat River flowing in a south-north direction at the bridge structure.

In general, the topography in the area of the bridge structure is relatively flat with gently rolling/undulating terrain and dense tree cover beyond the highway right-of-way. Ground surface conditions at the existing and proposed bridge locations are shown on Photographs 1 to 4.

The existing Nat River Bridge consists of an approximately 32.3 m long by 10.4 m wide (overall) three span, reinforced concrete slab on steel girder structure, which was constructed in 1964. The three spans (from west to east) are 7.6 m, 17.1 m, and 7.6 m in length. Based on the information provided in the Request for Proposal (RFP), the General Arrangement and Staging Details drawing included in Contract 94-202, and the General Arrangement drawing included in Contract 2010-5115, we understand that the existing bridge abutments and piers are supported by shallow timber crib foundations; however, the foundation bearing stratum (i.e., native soil or bedrock) is not known.

Based on the Ontario Structure Inspection Manual (OSIM) report, dated July 23, 2015, the existing bridge is generally in good condition with minor deterioration of several elements including more significant deterioration of the deck wearing surface, exterior deck soffit, abutment walls and interior parapet walls. The existing highway embankments were also noted to be in good conditions with no deficiencies identified in the 2015 OSIM report.

Based on our site observations at the time of the field investigation and a review of the available site photographs/satellite images, the existing embankments in the area of the existing bridge structure generally appear to be performing satisfactorily with no evidence of soil movement, tilted vegetation, or tension cracks which could indicate instability.

3.0 INVESTIGATION PROCEDURES

Field work for this subsurface investigation was carried out on May 10 and 11, 2019, during which time six boreholes (NR-1 to NR-6) were advanced at the approximate locations shown on Drawing 1. Boreholes NR-1 and NR-6 were advanced at the west and east approaches along the proposed re-alignment. Boreholes NR-2 and NR-3 were advanced at the proposed west abutment and Boreholes NR-4 and NR-5 were advanced at the proposed east abutment.

The boreholes were advanced using a CME-55 LC track-mounted drilling rig supplied and operated by George Downing Estate Drilling of Grenville-Sur-La-Rouge, Quebec. An excavator was used to provide drilling rig access to the borehole locations, which was supplied and operated by Demora Construction Services Inc. (Demora) of New Liskeard, Ontario. Demora also provided traffic control, to facilitate loading/unloading the excavator and drilling rig, which was performed in accordance with the Ontario Traffic Control Manual Book 7 – Temporary Conditions.

The boreholes were advanced using 108 mm inside diameter hollow-stem augers, NW casing with wash boring techniques, and NQ coring. Water from the Nat River was used for the wash boring and coring operations. Soil samples were obtained in the boreholes at 0.75 m intervals of depth using 50 mm outer diameter split-spoon samplers driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedures (ASTM D1586). The groundwater conditions and water levels in the open boreholes were observed during the drilling operations. A standpipe piezometer, which was decommissioned prior to demobilizing from the site, was installed in Borehole NR-2 to obtain a stabilized groundwater level about one day following completion of drilling. The boreholes were backfilled and the standpipe piezometer was decommissioned in accordance with Ontario Regulation 903 Wells (as amended).

Field work was monitored on a full-time basis by a member of Golder’s technical staff who: located the boreholes in the field; arranged for the clearance of underground services; supervised the drilling and sampling operations; logged the boreholes and drillholes; and examined the soil and rock core samples. The soil and rock core samples were identified in the field, placed in labelled containers and transported to Golder’s geotechnical laboratory in Sudbury for further examination and laboratory testing. Index and classification testing consisting of water content determinations, grain size distributions and Atterberg limits were carried out on selected soil samples. In addition, four uniaxial compressive strength (UCS) tests were carried out on specimens of the retrieved bedrock core. The geotechnical laboratory testing was completed according to ASTM and MTO LS standards, as applicable.

The as-drilled borehole locations were measured by a member of our technical staff relative to the existing highway centreline and existing bridge structure using a measuring tape and converted into northing/easting coordinates on the plan drawing. Given the relatively moderate distances between the boreholes and the existing highway centerline / bridge structure, the measurements are considered to be accurate to within 0.5 m horizontally. The ground surface elevations at the borehole locations were obtained using a survey level and rod and the survey loop was closed to within 0.1 m vertically. The boreholes were surveyed relative to a nearby benchmark [horizontal control point (HCP) 103] and the Geodetic elevation of the benchmark was obtained from the survey drawing (101REEVES GWP 5017-E-0018.dwg) provided by AECOM. The NAD 83 MTM Zone 12 northing and easting coordinates, World Geodetic System 1984 (WGS 84) geographical coordinates, ground surface elevations referenced to Geodetic datum, and borehole depths at each borehole location are presented on the Record of Borehole Sheets presented in Appendix A and summarized below.

Borehole Number	Location (NAD 83, MTM Zone 12)		Location (WGS 84)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting	Latitude	Longitude		
NR-1	5342735.6	221903.6	48.218013	-82.115666	324.5	4.9
NR-2	5342732.3	221918.7	48.217985	-82.115462	323.5	4.5
NR-3	5342727.7	221919.8	48.217944	-82.115447	323.4	5.5
NR-4	5342719.5	221958.7	48.217876	-82.114922	323.5	4.5
NR-5	5342714.8	221954.0	48.217833	-82.114984	323.5	3.8
NR-6	5342711.5	221971.7	48.217805	-82.114745	325.2	5.6

Note: Borehole depths include 3.0 m to 3.3 m of bedrock coring.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain Study (NOEGTS) [1] mapping by the Ministry of Natural Resources, the Nat River Bridge site is located within an outwash plain, valley train deposit comprised of silt bordered by organic terrain deposits of peat/muck and ground moraine deposits of sand.

Based on geological mapping by the Ministry of Northern Development and Mines (MNDM) [2], the overburden deposits are underlain by mafic to intermediate metavolcanics rocks consisting of basaltic and andesitic flows, tuffs, breccias, chert, iron formations, minor sedimentary and intrusive rocks, and related migmatites.

4.2 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions encountered in the boreholes are presented on the Record of Borehole and Record of Drillhole Sheets in Appendix A. The detailed results of the geotechnical

[1] Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 42BSE

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

laboratory testing are contained in Appendix B. The results of the in-situ field tests (i.e., SPT ‘N’ values) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile and cross-sections on Drawings 1 and 2 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

In summary, the subsurface conditions encountered at the site consist of sand fill, topsoil/peat, clayey silt-silt to silt, gravelly silty sand to sand, and silty gravel and sand underlain by bedrock at relatively shallow depth. A detailed description of the soil deposits, bedrock, and groundwater conditions encountered in the boreholes is provided in the following sections.

4.2.1 Sand (SW) Fill

A 150 mm thick layer of brown, moist, sand fill, trace gravel, trace silt was encountered from ground surface in Borehole NR-6, which was advanced from the existing boat launch located about 15 m north of the existing east abutment.

4.2.2 Topsoil/Peat

A 25 mm to 150 mm thick layer of topsoil/peat was encountered from ground surface in Boreholes NR-1 to NR-4.

4.2.3 Clayey Silt-Silt (CL-ML) to Silt (ML)

A deposit of brown to grey, moist to wet, clayey silt-silt, trace sand, trace organics was encountered below the topsoil in Borehole NR-2 and a deposit of silt, some sand, some clay, trace gravel was encountered below the sand fill in Borehole NR-6. The surface of the deposit was encountered at Elevation 323.4 m and 325.0 m in Boreholes NR-2 and NR-6, respectively and was 1.0 m and 0.5 m thick, respectively.

The SPT ‘N’-value in the clayey silt-silt deposit is 2 blows per 0.3 m of penetration suggesting a soft consistency and in the silt deposit is 16 blows per 0.3 m of penetration indicating a compact state of compactness.

The water content measured on a sample of the clayey silt-silt deposit is 23 per cent and on a sample of the silt deposit is 13 per cent.

The results of grain size distribution tests completed on two samples of the clayey silt-silt to silt deposit are shown on Figure B1 in Appendix B.

Atterberg limits testing was carried out on two samples of the deposit, which measured liquid limits of about 20 per cent and 22 per cent, plastic limits of about 17 per cent and 18 per cent, and plasticity indices of about 3 per cent and 4 per cent. The Atterberg limit test results are shown on the plasticity chart on Figure B2 in Appendix B, which indicates the deposit ranges from a clayey silt-silt of low plasticity to a silt of slight plasticity.

4.2.4 Gravelly Silty Sand (SM) to Silty Sand (SM)

A deposit of brown to grey, moist to wet, gravelly silty sand to silty sand, trace to some gravel, trace clay, and some organics (in places), was encountered below the topsoil/peat in Boreholes NR-1, NR-3, and NR-4, from ground surface in Borehole NR-5, and below the clayey silt-silt to silt deposit in Boreholes NR-2 and NR-6. The surface of the deposit was encountered between Elevation 324.5 m and 322.4 m and the deposit was between 0.3 m and 2.3 m thick.

The SPT ‘N’-values measured within the silty sand deposit range from 2 blows to 32 blows per 0.3 m of penetration, indicating a very loose to dense state of compactness.

The water content measured on two samples of the silty sand deposit are 14 per cent and 16 per cent.

The results of grain size distribution tests completed on two samples of the silty sand deposit are shown on Figure B3 in Appendix B.

4.2.5 Silty Gravel (GM) and Sand

A deposit of brown, wet, silty gravel and sand was encountered below the silty sand deposit in Borehole NR-1. The surface of the deposit was encountered at Elevation 323.8 m and the deposit was 0.4 m thick.

The SPT ‘N’-value measured within the silty gravel and sand deposit was 26 blows per 0.3 m of penetration, indicating a compact state of compactness.

The water content measured on one sample of the deposit was 8 per cent.

The results of a grain size distribution test completed on one sample of the silty gravel and sand deposit is shown on Figure B4 in Appendix B.

4.2.6 Bedrock

Bedrock was encountered below the overburden soils in all boreholes advanced at the site. The upper portion of the bedrock in Boreholes NR-1 and NR-6 was completely to moderately weathered. The surface of the completely to moderately weathered zone of bedrock was encountered at Elevations 323.4 m and 323.1 m in Boreholes NR-1 and NR-6, respectively and was 0.8 m and 0.2 m thick, respectively at these locations.

The lower portion of the bedrock was slightly weathered to fresh. The surface of the slightly weathered to fresh zone was encountered between Elevation 322.9 m and 321.1 m.

Bedrock was cored in Boreholes NR-1 to NR-6. The bedrock surface elevations, as encountered in the cored boreholes, are presented below.

Borehole No.	Completely to Moderately Weathered Bedrock		Slightly Weathered to Fresh Bedrock	
	Elevation (m)	Thickness Sampled / Length Cored (m)	Elevation (m)	Length Cored (m)
NR-1	323.4	0.8	322.6	3.0
NR-2	-	-	322.1	3.1
NR-3	-	-	321.1	3.2
NR-4	-	-	322.1	3.1
NR-5	-	-	322.7	3.0
NR-6	323.1	0.2	322.9	3.3

The retrieved bedrock core in Boreholes NR-1 to NR-3 is described as a fine grained, strong, light grey, completely weathered to fresh, greywacke. In Boreholes NR-4 and NR-5, the bedrock is described as a fine grained, medium strong, light grey, slightly weathered to fresh, sericite-chlorite schist and in Borehole NR-6, the bedrock is described as a fine grained, dark to light grey, slightly weathered to fresh, talc schist.

More detailed descriptions and conditions of the bedrock core samples are presented on the Record of Drillhole sheets in Appendix A. Photographs of the bedrock core samples and the UCS test results are presented on Figures B5 and B6, respectively, which are included in Appendix B. The bedrock properties from core samples selected for laboratory testing are summarized below.

Borehole No.	Slightly Weathered to Fresh Bedrock					
	Total Core Recovery (%)	Solid Core Recovery (%)	Rock Quality Designation (%)	Quality Classification (Table 3.10 of CFEM 2006)	Uniaxial Compressive Strength (MPa)	Strength Classification (Table 3.5 of CFEM 2006)
NR-1	88 – 100	48 – 100	27 – 100	Poor to Excellent	n/a	-
NR-2	93 – 100	77 – 83	77 – 83	Good	85	(R4 – Strong)
NR-3	100	43 – 100	27 – 100	Poor to Excellent	77	(R4 – Strong)

Borehole No.	Slightly Weathered to Fresh Bedrock					
	Total Core Recovery (%)	Solid Core Recovery (%)	Rock Quality Designation (%)	Quality Classification (Table 3.10 of CFEM 2006)	Uniaxial Compressive Strength (MPa)	Strength Classification (Table 3.5 of CFEM 2006)
NR-4	100	73 – 100	70 – 100	Fair to Excellent	48	(R3 – Medium Strong)
NR-5	100	97 – 100	97 – 100	Excellent	41	(R3 – Medium Strong)
NR-6	95 – 98	71 – 90	71 – 88	Fair to Good	n/a	-

4.2.7 Groundwater Conditions

Unstabilized groundwater levels measured in the open boreholes upon or shortly after completion of drilling are summarized below. A standpipe piezometer was installed in Borehole NR-2 and the groundwater level was measured about one day following completion of drilling. The river water level, as surveyed by Golder on May 11, 2019, was at Elevation 323.3 m. Groundwater and river water levels in the area are subject to seasonal fluctuations and precipitation events.

Borehole No.	Depth to Groundwater Level (m)	Approximate Groundwater Elevation (m)
NR-1	1.1	323.4
NR-2	0.1 (piezometer)	323.4 (piezometer)
NR-3	0.1	323.3
NR-4	0.2	323.3
NR-5	0.2	323.3
NR-6	1.8	323.4

The water levels in Boreholes NR-1 to NR-6 could potentially have been affected by water introduced into the boreholes during wash boring for NW casing advancement and/or during NQ coring operations; however, the water levels in the open boreholes are generally consistent and consistent with the surveyed river water levels at the time of the investigation.

4.3 Analytical Test Results of Soil Samples

One soil sample was selected from Borehole NR-2 (west abutment) and from Borehole NR-5 (east abutment) and submitted to Maxxam Analytics for corrosivity testing, under chain-of-custody documentation. The analytical laboratory test results are provided on the Certificate of Analysis presented in Appendix C and are summarized below.

Parameter	Units	West Abutment	East Abutment
		Borehole NR-2, Sample 2B (Elev. 322.3 m)	Borehole NR-5, Sample 1 (Elev. 323.2 m)
Resistivity	ohm-cm	5800	8100
Conductivity	µmho/cm	174	123
pH	pH	6.96	7.31
Sulphate	µg/g	<20[1]	<20[1]
Chloride	µg/g	60	41

Note(s): 1. The sulphate concentrations are below the reportable detection limit of 20 µg/g.

5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. Shane Albert, under the overall direction of Mr. David Muldowney, P.Eng. This Foundation Investigation Report was prepared by Ms. Kirsten Janssen, EIT, and Mr. David Muldowney, P.Eng. provided a technical review of the report. Mr. Paul Dittrich, P.Eng., an MTO Foundations Designated Contact and Principal of Golder, conducted an independent quality control review of this report.

Signature Page

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT

NAT RIVER BRIDGE REPLACEMENT – SITE NO. 46X-0011/B0

HIGHWAY 101, REEVES TOWNSHIP, DISTRICT OF SUDBURY

MINISTRY OF TRANSPORTATION, ONTARIO

GWP 5180-16-00; WP 5180-16-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation engineering recommendations for the proposed replacement of the Nat River Bridge (Site No. 46X-0011/B0). The recommendations presented are based on an interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives, and to carry out conceptual level designs for the temporary dewatering systems. The foundation investigation report and the discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO), and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The contractors must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction 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 Nat River Bridge is located on Highway 101 at Station 18+080 in Reeves Township in the District of Sudbury, Ontario (i.e., approximately 48 km west of the Highway 144 junction). The existing bridge consists of an approximately 32.3 m long by 10.4 m wide (overall) three span, reinforced concrete slab on steel girder structure, which was constructed in 1964. The three spans (from west to east) are 7.6 m, 17.1 m, and 7.6 m in length.

Based on the General Arrangement (GA) drawing provided by AECOM, we understand that the proposed replacement structure is to consist of a two lane, single-span, structure constructed along a new alignment located 18.6 m north of the existing Highway 101 alignment. The replacement bridge will be approximately 35.0 m long by 12.6 m wide (overall), with the proposed west and east abutments located at about Station 18+071 and 18+106, respectively. The finished grade of the re-aligned Highway 101 will be at approximately Elevation 327.2 m and 327.1 m at the west and east abutments, respectively. The proposed front slopes of the approach embankments will be about 4.5 m to 5 m high and inclined at 2H:1V. The proposed side slopes will be about 4 m high and will also be inclined at 2H:1V.

6.2 Consequence and Site Understanding Classification

A “typical consequence level” is considered appropriate for the Nat River Bridge replacement as outlined in Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2014) and its Commentary. Further, given the scope of work of the foundation field investigation and laboratory testing program as outlined in Sections 3.0 and 4.0, a “typical degree of site and prediction model understanding” has been utilized. Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, and geotechnical resistance factors, Φ_{gu} and Φ_{gs}, from Tables 6.1 and 6.2 of the CHBDC (2014) have been used for design.

6.3 Foundation Recommendations

6.3.1 Foundation Options

Based on the proposed bridge geometry and the subsurface conditions at this site, shallow footings are considered to be the most practical and most economical foundation option for the proposed replacement structure. Give the shallow depth to bedrock, deep foundations are not considered to be practical at this site and therefore are not discussed further in this report.

6.3.2 Founding Level and Geotechnical Resistances

Based on the subsurface conditions at this site, we recommend that the footings for the replacement bridge be founded directly on the slightly weathered to fresh, fair quality [i.e., rock quality designation (RQD) > 50 per cent] bedrock at/or below Elevation 322.1 m and 322.7 m at the west and east abutments, respectively.

Based on discussions with AECOM, we understand that footings up to about 3 m wide could potentially be required for supporting the abutments of the proposed replacement bridge. For footings founded on the good to excellent quality, strong, greywacke bedrock encountered at the proposed west abutment, a factored ultimate geotechnical axial resistance of 16 MPa may be used for design. For footings founded on the fair to excellent quality, medium strong, sericite-chlorite schist encountered at the east abutment, a factored ultimate geotechnical axial resistance of 6 MPa may be used for design. At both abutments, the factored serviceability geotechnical resistances for 25 mm of settlement will be greater than the factored ultimate geotechnical resistances and as such, ULS conditions will govern the design.

The preliminary geotechnical resistances provided above are dependent on the footing size, depth of embedment, configuration, and applied loads; and will have to be re-evaluated and modified as necessary during detail design. These preliminary 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 6.10.4 and Section C6.10.4 of CHBDC (2014) and its Commentary. 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.

6.3.3 Subgrade Preparation

All existing overburden soils (i.e., fill, topsoil/peat, clayey silt-silt to silt, gravelly silty sand to silty sand, and silty gravel and sand), completely to moderately weathered bedrock (if encountered), and very poor to poor quality 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. The bedrock surface was noted to be sloping across the footprint of the proposed abutments with a top of bedrock elevation difference of about 1.0 m across the west abutment (estimated to be up to about 1.7 m following sub-excavation of the poor quality upper portion of bedrock in Borehole NR-3) and about 0.6 m across the east abutment. Given the sloping bedrock conditions, consideration should be given to developing a Notice to Contractor at Detail Design to alert the Contractor to the variability in the bedrock surface elevations at this site. Consideration may also need to be given to dowelling the footings and/or levelling the bedrock surface with mass concrete to create a horizontal bearing surface for the footings. Dowels connecting the footing/bedrock should be incorporated into the design where bedrock is found to be sloping at greater than 10 degrees and/or if additional horizontal resistance is required. 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. The bedrock is classified as medium strong to 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 excavation 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 excavation. 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. The effect of blasting on the existing roadway, existing bridge and temporary protection systems (if required) should be considered by the designer and by the blasting contractor.

The subgrade (excavated bedrock surface) should be inspected by a Foundation Engineering specialist following sub-excavation 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 OPSS 902 (Excavating and Backfilling Structures).

6.3.4 Frost Protection

The estimated frost penetration depth in the area of the Nat River Bridge is 2.4 m as interpreted from OPSD 3090.100 (Frost Protection Depths for Northern Ontario). However, for footings founded on bedrock or mass concrete over bedrock, soil cover for protection from frost penetration is not considered necessary.

6.3.5 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 6.10.5 of the CHBDC (2014) applying the appropriate consequence and degree of site understanding factor as noted in Section 6.2. For footings founded directly on clean, sound (i.e., slightly weathered to fresh) bedrock, the coefficient of friction (tan δ) may be taken as 0.70 for cast-in-place footings.

Dowels connecting the concrete footing to the bedrock should be incorporated into the design if additional horizontal resistance is required.

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.

6.4 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the borehole investigation. Considering the anticipated foundation levels, the site may be classified as Site Class B "rock" in accordance with Table 4.1 of the CHBDC (2014). Geophysics testing (i.e., shear wave velocity measurements), if carried out, could potentially provide a more favourable Site Class A designation.

Using the information obtained from the NRCan (2015) Hazard Calculator for the proposed replacement bridge located at latitude 48.217912°and longitude -82.115220°, the following values were obtained for the spectral acceleration for a return period of 2,475 years:

Seismic Hazard Values	2% Exceedance in 50 years (2,475 year return period)
Sa (0.2) (g)	0.089
Sa (1.0) (g)	0.037

Based on the values noted above and in accordance with Table 4.10 of the CHBDC (2014), this site should be considered to be located in Seismic Performance Zone 1 for major-route and other bridges. In accordance with Section 4.4.5.1 of the CHBDC (2014), no seismic analysis is required for structures located in Seismic Performance Zone 1.

6.5 Approach Embankments

6.5.1 Subgrade Preparation and Embankment Construction

All existing organics (i.e., peat, topsoil, and/or mixed organic soil) shall be removed below the footprint of the proposed embankments to mitigate settlement and maintain embankment stability. 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), rock protection or concrete slope paving.

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.

6.5.2 Approach Embankment Stability

The stability analysis discussed below assumes that all organics within the footprint of the new embankments will be sub-excavated and replaced with granular fill prior to placement of any new granular embankment fill material.

6.5.2.1 Methodology

Limit equilibrium slope stability analysis was carried out for the re-aligned highway embankments using the commercially available program GeoStudio 2019 (Version 9.0.3.15488), produced by Geo-Slope

International Ltd., 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 1.33 has been used for design of the temporary, end-of-construction embankment side slopes, and a FoS of 1.54 has been used for the design of the permanent, final embankment configuration as per Table 6.2 of CHBDC (2014) using total stress (short-term, undrained) and effective stress (long-term, drained) conditions, as applicable.

The stability analyses carried out for preliminary design includes assessment of the proposed west and east approach embankments at about Stations 18+071 and 18+106, respectively, which corresponds to the highest embankment heights. The stability analyses were completed based on the subsurface conditions as encountered in Boreholes NR-2 and NR-3 (west abutment) and NR-4 and NR-5 (east abutment) and using the embankment geometries in the cross-section drawing (Hwy 101 Nat River Opt 3.dwg) provided by AECOM.

6.5.2.2 Parameter Selection

For the new granular fill and the non-cohesive gravelly silty sand to silty sand, 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.

For the cohesive clayey silt-silt deposit, total stress parameters were employed for the short-term, undrained conditions. The total stress parameters (i.e., average mobilized undrained shear strength - s_u) for the cohesive soil were estimated from correlations with the SPT ‘N’-value. Effective friction angles have also been estimated for this deposit for analysis of the factor of safety in the long-term, drained condition.

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

Soil Deposit	Bulk Unit Weight (kN/m^3)	Short-Term Analysis		Long-Term Analysis
		Effective Friction Angle (°)	Undrained Shear Strength (kPa)	Effective Friction Angle (°)
New Granular Fill (compacted)	21	35	-	35
Clayey Silt-Silt (soft)	18	-	15	28
Gravelly Silty Sand to Silty Sand (very loose to dense)	19	30	-	30

6.5.2.3 Results of Analyses

The results of the analyses indicate that for an approximately 4 m high west approach embankment constructed using OPSS.PROV 1010 Granular ‘B’ (Type I or II) fill material with side slopes inclined at 2H:1V, the FoS for global stability is 1.71 and 1.73 for the short-term (temporary) and long-term (permanent) conditions, respectively, which satisfies the minimum target FoS for the respective conditions (see Figures 1 and 2).

For the proposed east approach embankment, given that the embankment fill and gravelly silty sand to silty sand deposit are non-cohesive in nature, the effective strength parameters will apply to both the short-term (temporary, undrained) and long-term (permanent, drained) cases and therefore the long-term scenario with the higher target minimum FoS governs for design. The results of the analyses indicate that for an approximately 4 m high east approach embankment constructed using OPSS.PROV Granular ‘B’ (Type I or II) fill material with side slopes inclined at 2H:1V, the FoS against global instability is 1.78 for both the short-term and long-term conditions, which satisfies the minimum target FoS for both conditions (see Figure 3). Based on the above, stability mitigation measures will not be required for the proposed approach embankment side slopes.

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

6.5.3 Approach Embankment Settlement

The settlement analysis discussed below assumes that all organics within the footprint of the new embankments will be sub-excavated and replaced with granular fill prior to placement of any new granular embankment fill material.

6.5.3.1 Methodology

To estimate the magnitude of the settlement due to the new embankment loading, analyses were carried out on the critical sections at Station 18+071 and 18+106 for the west and east approach embankments, respectively, using the commercially available computer program *Settle-3D* (Version 4) from Rocscience Inc. The sources of settlement were considered to include:

- time-dependent consolidation of the low plasticity clayey silt-silt deposit (west approach)
- immediate settlement of the cohesionless silt, gravelly silty sand to silty sand, and silty gravel and sand deposits (west and east approach)

6.5.3.2 Settlement Criteria

Based on MTO’s “Embankment Settlement Criteria for Design” (July 2010), the following post-construction settlement and differential settlement criteria are considered acceptable for settlements to occur within 20 years post-paving for the bridge approach embankments (including temporary widening) at this site.

Location	Maximum Limits During Pavement Design Life	
	Total (mm)	**Differential (mm)**
Longitudinal Transitions (Non-Freeways)	25 (0 to 20 m from abutment) 50 (20 m to 50 m from abutment) 100 (50 m to 75 m from abutment) 200 (≥ 75 m from abutment)	n/a

These criteria have been used for evaluating whether mitigation measures are required to limit post-construction settlement of the approach embankments. The total settlement and differential settlement are considered to be applicable over a 20-year period following completion of construction (i.e., final paving). These performance criteria form part of the overall design performance for the embankment in the vicinity of the bridge replacement.

6.5.3.3 Parameter Selection

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

The immediate compression of the non-cohesive silt, gravelly silty sand to silty sand, and silty gravel and sand deposits was calculated using estimated elastic moduli of deformation based on the SPT ‘N’-values and the empirical correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).

The consolidation settlement of the clayey silt-silt was calculated using an estimated constrained modulus based on an inferred undrained shear strength of 15 kPa (from the SPT ‘N’-value of 2 blows per 0.3 m of penetration) along with the results of the index testing (i.e., water content and Atterberg limits).

All values were tempered by engineering judgment based on precedent experience in similar soils.

Deposit	Bulk Unit Weight (kN/m^3)	Elastic Modulus, E’ (MPa)	Constrained Modulus, D’ = $1/m_v$ (MPa)
Clayey Silt-Silt (soft)	19	-	3
Silt	18	10	-
Gravely Silty Sand to Silty Sand to Silty Gravel and Sand	19	15	-

A coefficient of consolidation, c_v (cm^2/s), required in the time-rate of settlement analysis, was estimated from the empirical correlation with liquid limit (NAVFAC, 1986). A c_v equal to 5.7×10^{-3} cm^2/s is conservatively estimated for the clayey silt-silt soil at this site based on the limited information available.

6.5.3.4 Results of Analyses

A summary of the results of the settlement analysis at the west and east approaches is present below.

Critical Section	Relevant Borehole	Embankment Settlement at Highway 101 Centerline (re-aligned)		
		Immediate (mm)	Consolidation (mm)	Total (mm)
West Approach	NR-2	5	35	40
East Approach	NR-4	15	-	15

Based on the results of the analyses, a total of up to 40 mm of settlement is anticipated to occur at the proposed west approach, including: about 5 mm of settlement in the cohesionless gravelly silty sand to silty sand to silty gravel and sand deposits, which will occur immediately (i.e., during construction); and about 35 mm of consolidation settlement in the cohesive clayey silt-silt deposit, which will occur partially during and possibly post-construction. Based on the c_v value indicated above and assuming two-way drainage in the approximately 1.1 m thick clayey silt-silt deposit, it is estimated that about 90 per cent of the consolidation settlement will be completed in about one week. Given the relatively short duration for the anticipated settlement to occur, we recommend that an operational constraint be incorporated into the contract package to delay final paving of the roadway by one to two weeks to mitigate potential risks associated with post-construction settlement impacts to the final paved surface. Given the limited sampling and field/laboratory testing available for the clayey silt-silt deposit at this preliminary design stage, a two-week paving delay is considered prudent to mitigate the risk associated with post-construction settlement; alternatively, consideration could be given to sub-excavating the clayey silt-silt deposit where encountered, or carrying out more investigation, laboratory testing and analysis at the detail design stage to refine the settlement and time-rate estimate.

At the proposed east abutment, a total of about 15 mm of immediate settlement is anticipated within the cohesionless gravelly silty sand to silty sand, and silt deposits, which will occur during construction (i.e., no post-construction settlement). Although settlement mitigation is not strictly required at the east approach, given the variability of the subgrade soils at this site, consideration should be given to applying the same settlement mitigations measures as discussed for the west approach. Alternatively, consideration could also be given to conducting additional investigations along the east approach at detail design.

Given the cohesive clayey silt-silt deposit was only encountered in Borehole NR-2 at the north side of the proposed west abutment, it is anticipated that the subsurface soils adjacent to the existing highway embankment are generally comprised of cohesionless soils (i.e., gravelly silty sand to silty sand, silty gravel and sand, and silt). As such, settlements along the existing roadway platform, as a result of the new embankment construction, are anticipated to be relatively minor (i.e., less than about 5 mm to 15 mm) and settlement mitigation measures are not considered to be required for the existing highway embankment.

The above preliminary estimates do not include compression of the fill itself, which would occur during construction of the embankment depending on the type of material used. The magnitude of granular fill compression may range from 0.5 per cent to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In this case, settlement of the granular fill itself is expected to occur essentially during embankment construction. Non-granular earth fill materials are not recommended for embankment construction as they may exhibit some additional settlement over time depending on their gradation, plasticity and field compaction effort. Should rock fill be considered, long term settlement of the rock fill will need to be considered during detail design.

This preliminary assessment of the settlement(s) should be reviewed and confirmed based on additional subsoil conditions encountered during detail design and utilizing the finalized embankment geometry/configuration.

6.6 Construction Considerations

The following subsections identify construction issues that should be considered at this stage of the design as they may impact the planning for detail design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during detail design for incorporation into the Contract Documents.

6.6.1 Excavations and Temporary Shoring

Excavations for construction of the replacement bridge foundations are anticipated to extend through the existing overburden soils (i.e., clayey silt-silt, silt, gravelly silty sand to silty sand, and silty gravel and sand), the completely to moderately weathered bedrock (where encountered), and any very to poor to poor quality rock to expose the underlying clean, sound bedrock (i.e., slightly weathered to fresh with an RQD > 50 per cent) at depths between about 0.8 m and 3.0 m below grade (i.e., Elevation 322.7 m to 320.4 m).

Open-cut excavations are considered to be feasible at this site and where utilized, 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 1 Horizontal to 1 Vertical (1H:1V). In Type 4 soils, the excavation side slopes should be formed no steeper than 3H:1V. Excavations into the bedrock, where required, may be cut vertically or near vertically depending on the degree of weathering.

If required, temporary shoring support systems could consist of either driven steel sheet piling or soldier piles and lagging. The installation of sheet-piles for temporary shoring could potentially be impeded by the presence of cobbles and/or boulder obstructions and may require pre-drilling through the obstructions and/or the use of a heavier sheet pile section. It is recommended that a Notice to Contractor be developed during detail design to alert the Design-Build Contractor to the presence of obstructions. Given the shallow depth to bedrock at this site, sheet pile installations would 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 cobble and boulder 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 excavation support systems shall be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems).* The lateral movement of the temporary shoring system shall meet Performance Level 2 as specified in OPSS.PROV 539. Design of the temporary support system shall include an evaluation of base stability, soil squeezing stability and hydraulic uplift stability as defined in the Canadian Foundation Engineering Manual (CFEM 2006).

Although the Design-Build Contractor is responsible for the selection and complete detail design of the temporary support system, the following parameters are provided to enable the structural designer(s) to evaluate the alternative conceptual temporary support systems as part of the Preliminary Design. As noted in Section 4.2.7, groundwater conditions measured at the six boreholes (NR-1 to NR-6) ranged from Elevations 323.3 m to

332.4 m and the river water level at the time of the Foundation investigation was measured at Elevation 323.3 m. These groundwater and creek water levels should be considered when evaluating the alternative conceptual temporary support system(s).

Soil Type	Unit Weight	Undrained Shear Strength (1)	Internal Angle of Friction	Coefficient of Earth Pressure (2)		
	(γ, kN/m^3)	(s_u, kPa)	(ϕ, degrees)	Active, K_a	At Rest, K_o	Passive, K_p(3)
New Granular Fill (compacted)	21	-	35	0.27	0.43	3.69
Clayey Silt-Silt (very loose to compact)	18	15	28	0.36	0.53	2.77
Gravelly Silty Sand to Silty Sand to Silt (loose to compact)	19	-	30	0.33	0.50	3.00
Silty Gravel and Sand (compact)	20	-	32	0.31	0.47	3.25

1. The temporary shoring design should be assessed for both the drained (ϕ) and undrained (s_u) cases and the design should be based on the more conservative earth pressure conditions.
2. The lateral earth pressure coefficients presented above are based on a horizontal surface adjacent to the excavation. If sloped surfaces are expected, the coefficients should be corrected accordingly.
3. The total passive resistance below the base of the excavation adjacent to the temporary protection system may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

Consideration could be given to either partial or full removal of the temporary shoring system upon completion of construction. Where possible, full removal of the temporary shoring system is typically preferred to mitigate potential impediments to future rehabilitation/reconstruction work at the bridge site. Although there is some potential risk that full removal of the temporary shoring system could result in subgrade softening of the clayey silt-silt deposit and some risk of soil adhesion along the sheet pile (or H-pile) walls (CFEM 2006), which could create a void below the excavation, these risks are considered to be relatively low given the limited thickness/extent of the low plasticity clayey silt-silt deposit. As such, from a foundations perspective, it is recommended that the temporary shoring system be fully removed, if utilized.

6.6.2 Control of Groundwater and Surface Water

Temporary excavations along the proposed new alignment will be required to facilitate sub-excavation of the native soils, the completely to moderately weathered bedrock (where encountered), and the very poor to poor quality rock in advance of the footing construction. Groundwater seepage into the excavation should be expected from the relatively permeable native soils 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 6.6.1, 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 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 and allow unwatering and footing construction in the dry. To minimize groundwater seepage, consideration may need to be given to sub-excavating the native soils to allow placement of the sandbags or inflatable bladder 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 should also be given to the use of a tremie concrete plug to seal the base of excavation(s) prior to dewatering; however, additional effort would be required by the contractor 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. Given the subsurface conditions at this site, it is not considered necessary for the dewatering Design Engineer and/or Design Checking Engineer to have a minimum 5 years experience. As such, Note 1 of Table A of SP 517F01 should indicated “No”. Further, given the apparent lack of infrastructure in the vicinity of the bridge, a preconstruction survey is not considered to be required at this site. As such, the foundation designer fill-in for Table A of SP 517F01 should indicate that the preconstruction survey distance is not applicable.

Although the foundation stratum below the timber crib foundations of the existing bridge structure is unknown, it is anticipated that the timber cribs would have been founded directly on bedrock and/or engineering fill overlying bedrock to provide sufficient geotechnical axial resistances to support the existing bridge loads. As such, the risk of potential settlement impacts to the existing bridge as a result of a temporary groundwater lowering are considered to be relatively low. However, given that the existing timber crib foundations could potentially be underlain by a deposit of compressible clayey silt-silt, consideration should be given to monitoring the existing bridge during construction. Alternatively, additional investigations could be performed at detail design to confirm the foundation stratum below the existing timber crib foundations.

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

6.6.3 Obstructions

Cobble and/or boulders obstructions, as encountered in Borehole NR-5 and inferred to be present in Borehole NR-3, could affect the installation of temporary support systems and/or temporary cofferdams (if required). It is recommended that a Notice to Contractor be developed during detail design to alert the Design-Build Contractor to the presence of cobble/boulder obstructions. Note that the extent and depth of the cobble and boulder obstructions may vary beyond and between the borehole locations.

6.7 Analytical Testing for Construction Materials

The results of analytical testing on two samples taken from the Nat River Bridge site are summarized in Section 4.3 and the analytical laboratory test results are included in Appendix C. The suite of parameters tested is intended to allow the design engineer to assess the requirements for the appropriate type of cement to be used in construction and the need for corrosion protection of steel elements.

For potential sulphate attack on concrete, the results of the soil analysis were compared to Table 3 in CSA A23.1-14. The sulphate concentrations were less than 0.002 per cent (i.e., the minimum reportable detection limit) in the two samples, which is below the exposure class S-3 “Moderate”; and may be considered negligible according to Table 7.2 of the MTO Gravity Pipe Design Guidelines (2004). However, given that the bridge location is on Highway 101 and will be exposed to de-icing salts it is recommended that a C-1 class exposure concrete by considered.

The resistivity results indicate that the soil corrosiveness is very low (10,000>R>6,000) to low (6,000>R>4,500) as per Table 3.2 of the MTO Gravity Pipe Design Guidelines (2014). Further, the pH was measured at 6.96 and 7.31. According to the MTO Gravity Pipe Design Guidelines (2014), a pH of less than 8.0 is not considered detrimental to durability.

It should be noted that the river water levels in the area are subject to seasonal fluctuations and variations due to the precipitation events and the soil/water chemistry could also be variable. These recommendations are provided as guidance only; the structural designer should take the results of the laboratory testing and the potential for corrosion into consideration as part of the ultimate materials selection.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Kirsten Janssen, EIT, and the technical aspects were reviewed by Mr. David Muldowney, P.Eng. Mr. Paul Dittrich, P.Eng., an MTO Foundations Designated Contact and Principal of Golder, conducted an independent quality control review of this report.

Signature Page

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ASTM International

ASTM D1586 Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils

Commercial Software

Slope/W – Geostudio 2019 (Version 9.0.3.1548) by Geo-Slope International Ltd.

Settle-3D (Version 4) by RocScience Inc.

Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.100 Foundation, Frost Penetration Depths for Northern Ontario

Ontario Provincial Standard Specifications (OPSS)

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.PRVO 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 802	Construction Specification for Topsoil
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling – Structures
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resource Act

Regulation 903 Wells (as amended)

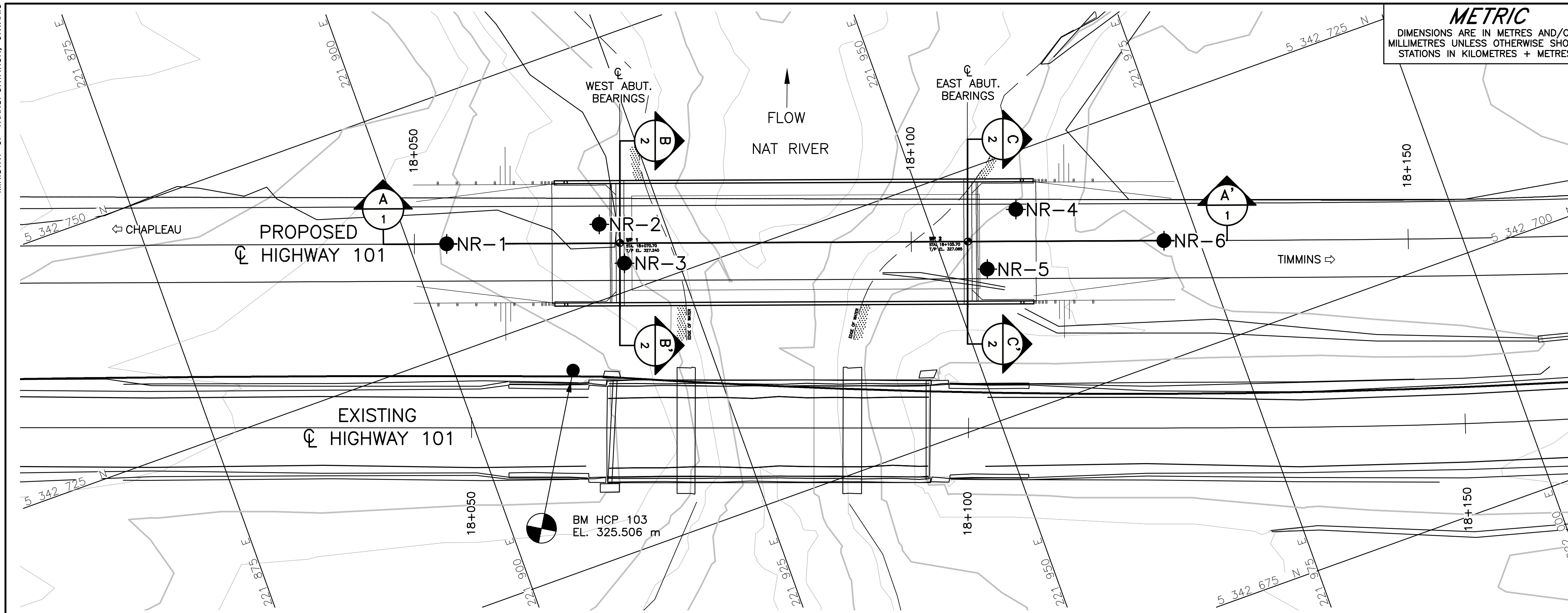
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5180-16-01

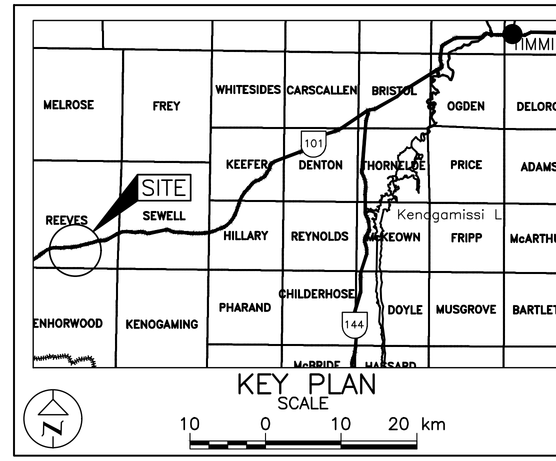
SHEET

HIGHWAY 101
NAT RIVER BRIDGE STA. 18+080
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER



PLAN
SCALE
5 0 5 10 m



KEY PLAN
SCALE
10 0 10 20 km

LEGEND

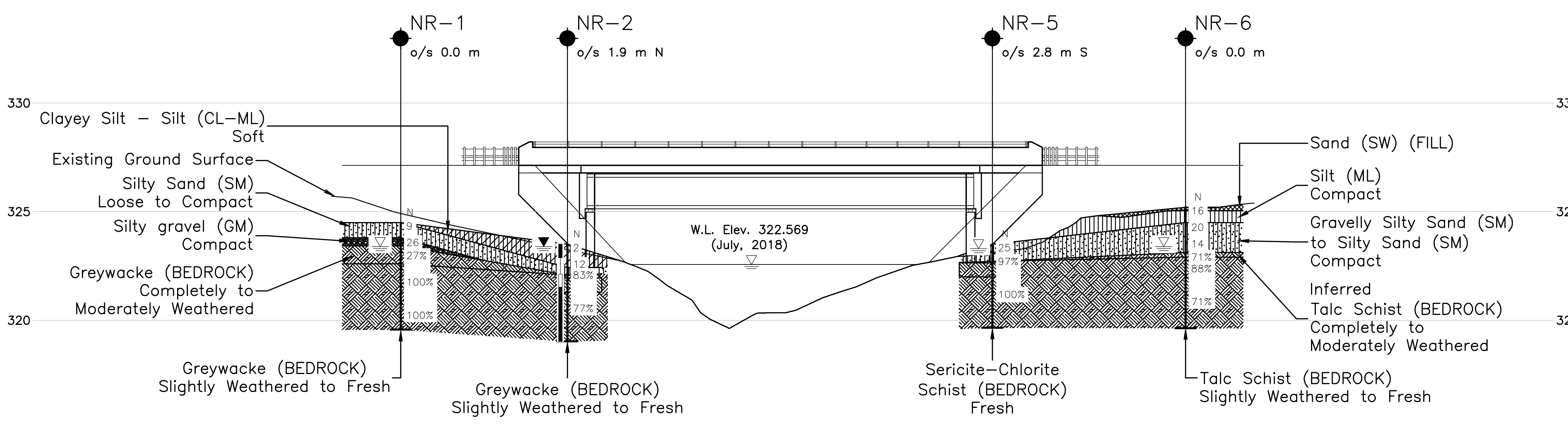
⬤	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▽	WL upon completion of drilling
▼	WL in piezometer, measured on May 12, 2019
	Seal
	Piezometer

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
NR-1	324.5	5342735.6	221903.6
NR-2	323.5	5342732.3	221918.7
NR-3	323.4	5342727.7	221919.8
NR-4	323.5	5342719.5	221958.7
NR-5	323.5	5342714.8	221954.0
NR-6	325.2	5342711.5	221971.7

LICENSED PROFESSIONAL ENGINEER J.P. DITTRICH DEC 10, 2019 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER D.A. MULDOWNEY 100164716 DEC 10, 2019 PROVINCE OF ONTARIO



A-A' 1 CENTERLINE PROFILE
HORIZONTAL SCALE
5 0 5 10 m
2.5 0 2.5 5 m
VERTICAL SCALE

NOTES

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

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

REFERENCE

Base plans provided in digital format by AECOM CANADA LTD. drawing file no. 101REEVES GWP 5017-E-0018, received JUNE 6, 2019.

NO.	DATE	BY	REVISION

Geocres No. 42B-14

HWY. 101		PROJECT NO. 1790414	DIST.
SUBM'D.	CHKD.	DATE: 12/10/2019	SITE: 46X-0011/B0
DRAWN: TR	CHKD. DAM	APPD. JPD	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

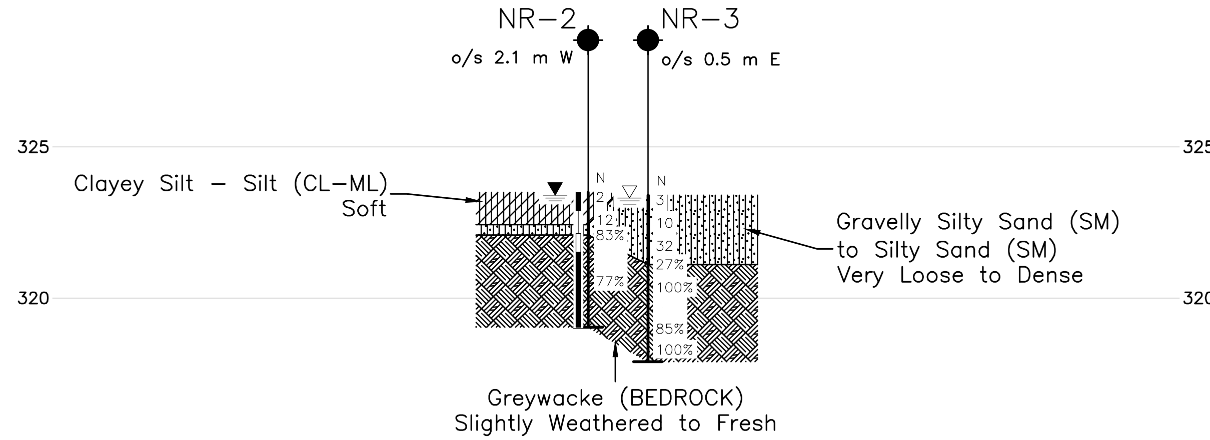
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5180-16-01

HIGHWAY 101
NAT RIVER BRIDGE STA. 18+080
SOIL STRATA

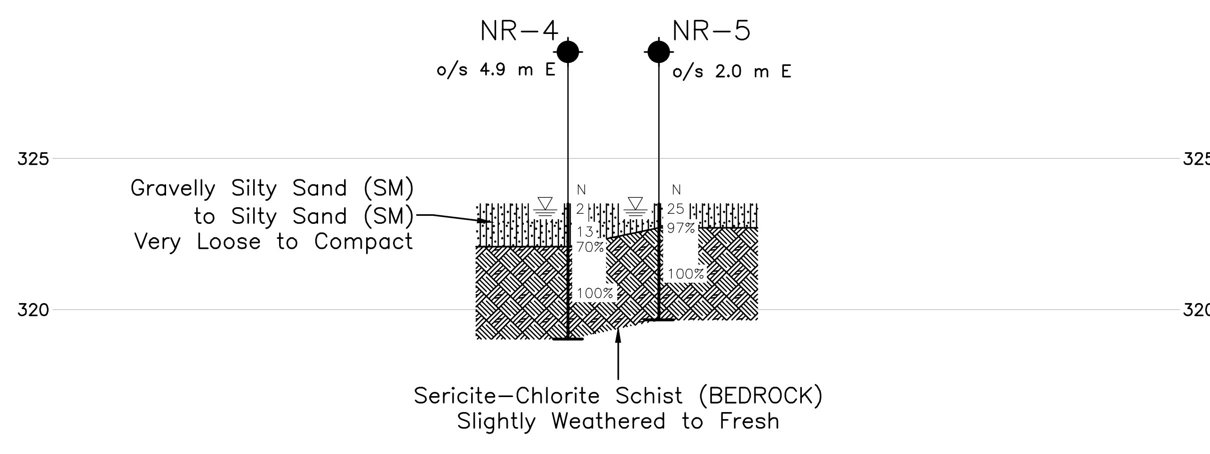
SHEET

GOLDER



B-B' / 1 WEST ABUTMENT CROSS-SECTION

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



C-C' / 1 EAST ABUTMENT CROSS-SECTION

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

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)
- WL upon completion of drilling
- WL in piezometer, measured on May 12, 2019
- Seal
- Piezometer

LICENSED PROFESSIONAL ENGINEER
J.P. DITTRICH
DEC 10, 2019
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
D.A. MULDOWNEY
100164716
DEC 10, 2019
PROVINCE OF ONTARIO

NOTES

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

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

REFERENCE

Base plans provided in digital format by AECOM CANADA LTD. drawing file no. 101REEVES GWP 5017-E-0018.dwg, received JUNE 6, 2019.

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 42B-14

HWY. 101		PROJECT NO. 1790414	DIST.
SUBM'D.	CHKD.	DATE: 12/10/2019	SITE: 46X-0011/B0
DRAWN: TR	CHKD. DAM	APPD. JPD	DWG. 2

PLOT DATE: December 9, 2019
FILENAME: S:\Clients\MTO\Hwy17&101\99_PROJ\1790414_Aecom\40_PROD\0001_Nat_River_Bridge\1790414-0001-BG-0002.dwg

Photograph 1: Nat River Bridge, Facing West (May 2019)
(looking towards location of proposed west abutment)

Photograph 2: Nat River Bridge, Facing Southeast (May 2019)
(from location of proposed west abutment)

Photograph 3: Nat River Bridge, Facing West (May 2019)
(from location of proposed east abutment)

Photograph 4: Nat River Bridge, Facing West (May 2019)
(from towards south side of existing bridge)

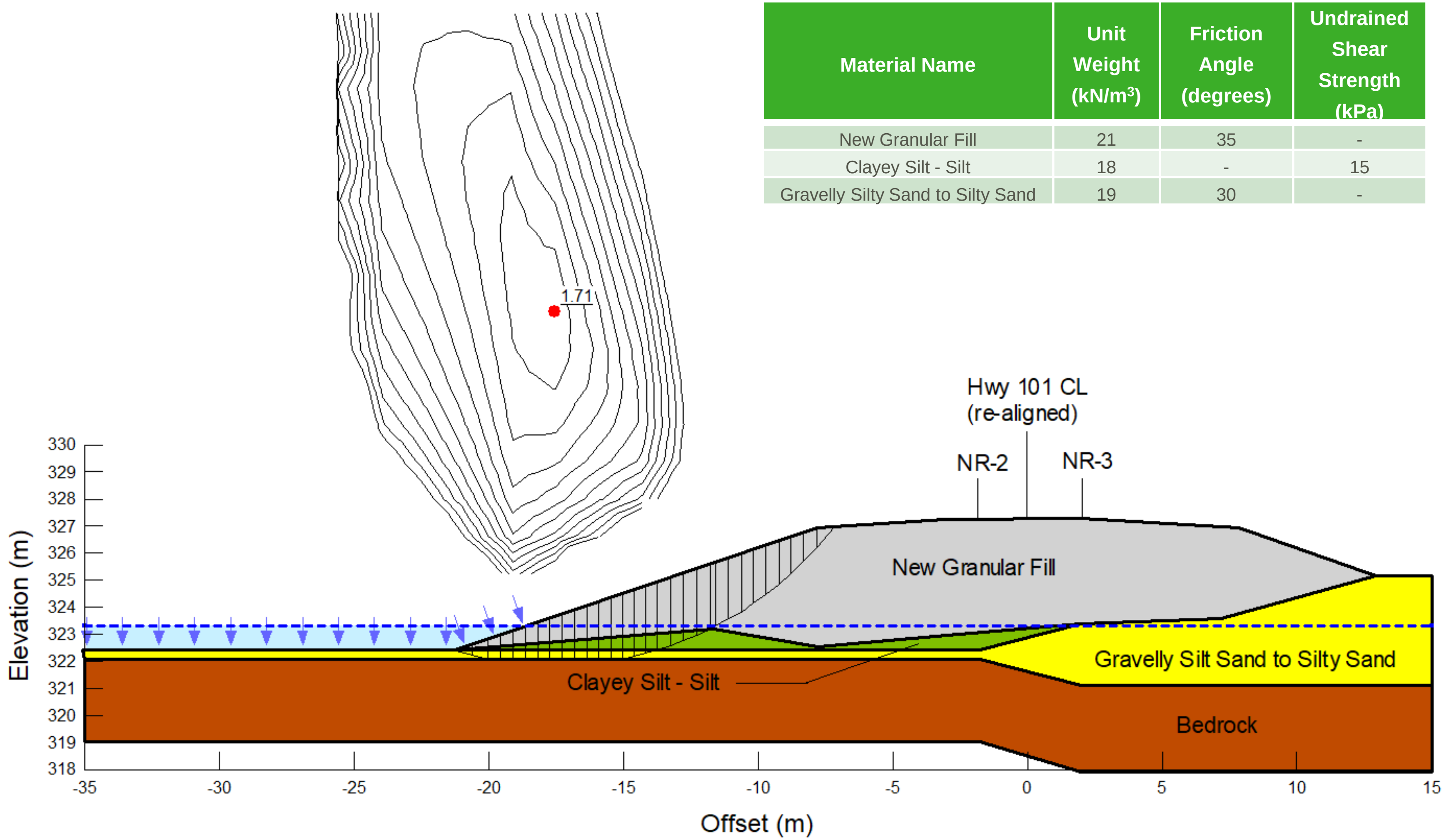
Global Stability Analysis

West Approach

Short-Term (Undrained) Analysis

Figure 1

Material Name	Unit Weight (kN/m³)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
New Granular Fill	21	35	-
Clayey Silt - Silt	18	-	15
Gravelly Silty Sand to Silty Sand	19	30	-



Date: September 2019
Project Number: 1790414 – Nat River Bridge

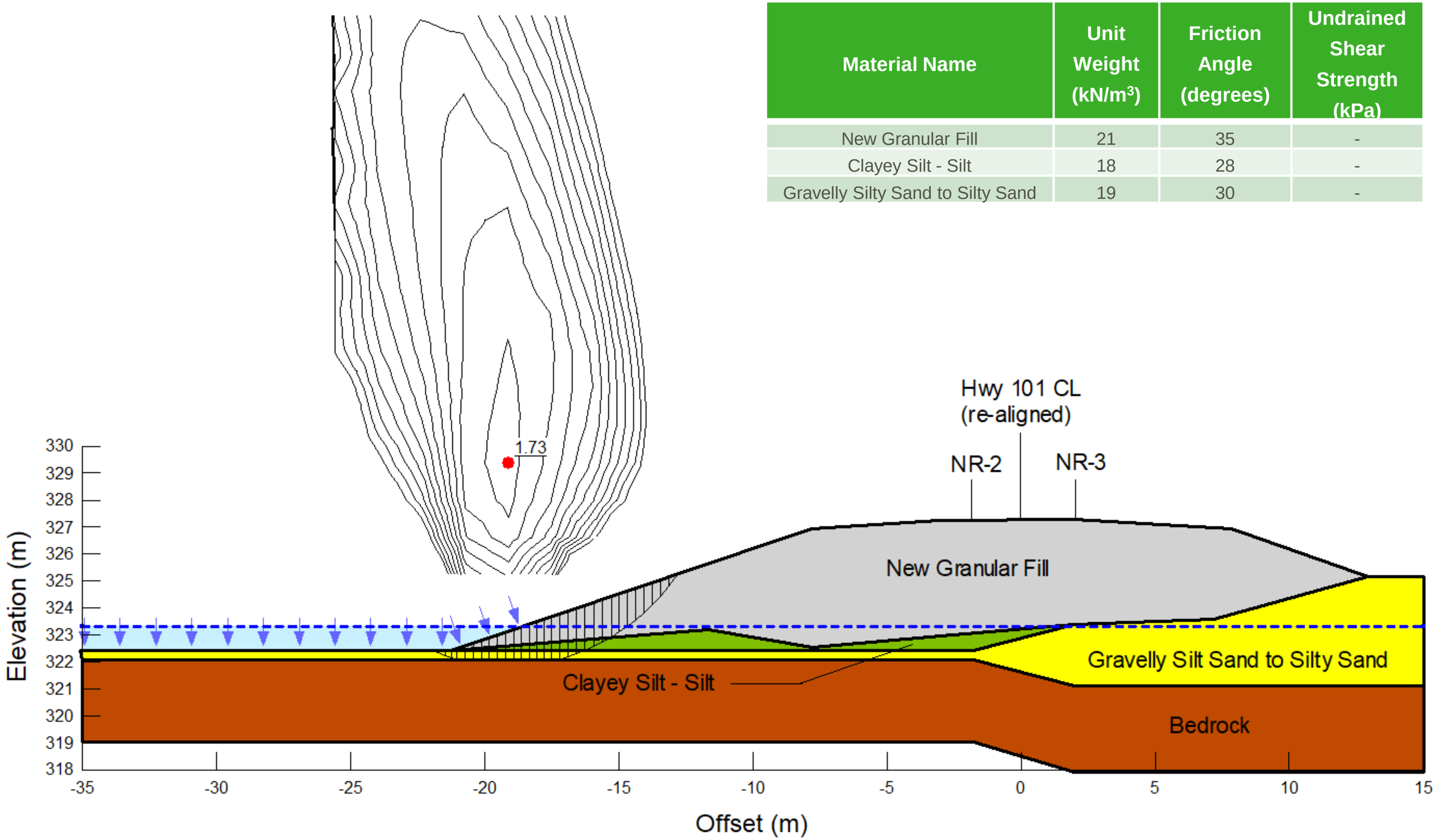
Analysis By: DAM
Reviewed By: JPD

Global Stability Analysis

Figure 2

West Approach
Long-Term (Drained) Analysis

Material Name	Unit Weight (kN/m³)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
New Granular Fill	21	35	-
Clayey Silt - Silt	18	28	-
Gravelly Silty Sand to Silty Sand	19	30	-



Date: September 2019
Project Number: 1790414 – Nat River Bridge

Analysis By: DAM
Reviewed By: JPD

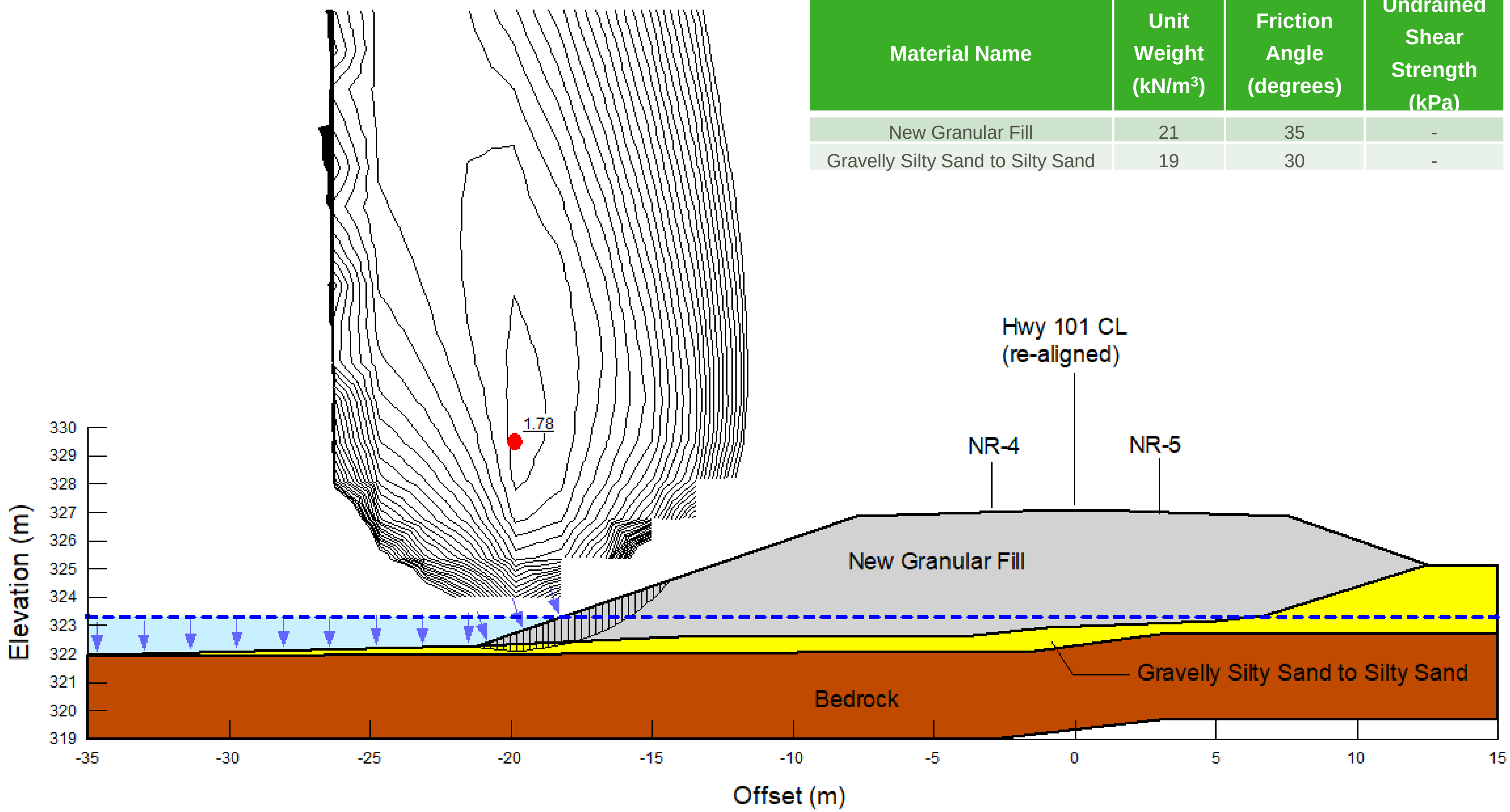
Global Stability Analysis

Figure 3

East Approach

Short-Term & Long-Term (Drained) Analysis

Material Name	Unit Weight (kN/m^3)	Friction Angle (degrees)	Undrained Shear Strength (kPa)
New Granular Fill	21	35	-
Gravelly Silty Sand to Silty Sand	19	30	-



APPENDIX A

Record of Borehole and Drillhole Sheets

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	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
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

3. 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.
4. 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.

LIST OF SYMBOLS
MINISTRY OF TRANSPORTATION, ONTARIO

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (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_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 δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

GOLDER

PROJECT 1790414

RECORD OF BOREHOLE No NR-1

METRIC

G.W.P. 5180-16-00

LOCATION N 5342735.6; E 221903.6 NAD83 MTM ZONE 12 (LAT. 48.218013; LONG. -82.115666)

ORIGINATED BY SA

DIST ______ HWY 101

BOREHOLE TYPE NW Casing with Wash Boring and NQ Coring

COMPILED BY TR

DATUM GEODETIC

DATE May 11, 2019

CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
324.5	GROUND SURFACE							
0.0	TOPSOIL (25 mm)							
	SILTY SAND (SM), trace gravel, some organics Loose Moist Brown	1	SS	9		324		
323.8 / 0.7	SILTY GRAVEL (GM) and sand Compact Wet Brown	2A / 2B	SS	26	∇		○	44 42 (14)
323.4 / 1.1	Completely to moderately weathered, brown to grey, GREYWACKE (BEDROCK)					323		
322.6 / 1.9	GREYWACKE (BEDROCK) Slightly weathered to fresh. For coring details see Record of Drillhole NR-1.	1	RC	REC 88%		322		RQD = 27%
		2	RC	REC 100%		321		RQD = 100%
		3	RC	REC 100%		320		RQD = 100%
319.6 / 4.9	END OF BOREHOLE							

NOTE:

1. Water level at a depth of 1.1 m below ground surface (Elev. 323.4 m) upon completion of drilling.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

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

RECORD OF DRILLHOLE: NR-1

PROJECT: 1790414
LOCATION: N 5342735.6; E 221903.6
NAD83 MTM ZONE 12 (LAT. 48.218013; LONG. -82.115666)
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: May 11, 2019
DRILL RIG: CME55 LC
DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

SHEET 1 OF 1
DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	COLOUR % RETURN	DISCONTINUITY TYPE AND SURFACE DESCRIPTION
		GROUND SURFACE	323.0			
	NW	Completely to moderately weathered, brown to grey, GREYWACKE	1.5			
2	NQ Coring, May 11, 2019	GREYWACKE Fine grained Light grey Slightly weathered to fresh - Broken rock between 1.9 m and 2.2 m depth	322.6 1.9	1	Grey 100	BR BR MB MB MB
3				2	Grey 100	MB MB MB MB MB MB MB MB MB
4		- Mechanically broken rock between 3.8 m and 4.1 m depth		3	Grey 100	JNIRRo MB MB JNIRRo
5		END OF DRILLHOLE	319.6 4.9			

DEPTH SCALE
1 : 60

GOLDER

LOGGED: SA
CHECKED: DAM

SUD-MTO-RCK S:\CLIENTS\MTO\HWY17&101\02_DATA\GINT\1790414.GPJ GAL-MISS.GDT 12-4-19 TR

GOLDER

RECORD OF BOREHOLE No NR-2

 METRIC

PROJECT 1790414
G.W.P. 5180-16-00
LOCATION N 5342732.3; E 221918.7 NAD83 MTM ZONE 12 (LAT. 48.217985; LONG. -82.115462)
ORIGINATED BY SA
DIST ____ HWY 101
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Wash Boring and NQ Coring
COMPILED BY TR
DATUM GEODETIC
DATE May 11, 2019
CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
323.5	GROUND SURFACE						
0.0	TOPSOIL (50 mm)						
0.1	CLAYEY SILT - SILT (CL-ML), trace sand, trace organics Soft Brown to grey Moist to wet	1	SS	2	323		
322.4 1.1	SILTY SAND (SM), some gravel Compact Brown Wet	2A 2B	SS	12		Wp–W plotted near 20	0 6 78 16
322.1 1.4	GREYWACKE (BEDROCK) Slightly weathered to fresh For coring details see Record of Drillhole NR-2.	1	RC	REC 93%	322 321		RQD = 83%
		2	RC	REC 100%	320		RQD = 77%
319.0 4.5	END OF BOREHOLE NOTES: 1. Water level at a depth of 0.2 m below ground surface (Elev. 323.3 m) upon completion of drilling. 2. Water level at a depth of 0.1 m below ground surface (Elev. 323.4 m) on May 12, 2019 in standpipe piezometer.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³

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

PROJECT: 1790414

RECORD OF DRILLHOLE: NR-2

SHEET 1 OF 1

LOCATION: N 5342732.3; E 221918.7
NAD83 MTM ZONE 12 (LAT. 48.217985; LONG. -82.115462)
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: May 11, 2019
DRILL RIG: CME55 LC
DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	DISCONTINUITY TYPE AND SURFACE DESCRIPTION	NOTES
		GROUND SURFACE	322.1				
1–2	NW	GREYWACKE Fine grained Strong Light grey Slightly weathered to fresh	1.4	1	Grey 100	MB MB MB BR	UCS = 85 MPa
3–4	NQ Coring, May 11, 2019	- Broken rock between 2.7 m and 3.1 m depth - Broken rock between 3.8 m and 4.1 m depth	319.0	2	Grey 100	BR MB MB MB MB BR BR JNIRRo MB MB MB	
		END OF DRILLHOLE	4.5				

Column headings: FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX METRES; DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION, Jr, Ja, Jn); HYDRAULIC CONDUCTIVITY k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.

Depth scale: 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13

DEPTH SCALE
1 : 60

GOLDER

LOGGED: SA
CHECKED: DAM

SUD-MTO-RCK S:\CLIENTS\MTO\HWY17&101\02_DATA\GINT\1790414.GPJ GAL-MISS.GDT 12-4-19 TR

GOLDER

RECORD OF BOREHOLE No NR-3

 METRIC

PROJECT 1790414

G.W.P. 5180-16-00

LOCATION N 5342727.7; E 221919.8 NAD83 MTM ZONE 12 (LAT. 48.217944; LONG. -82.115447)

ORIGINATED BY SA

DIST ____ HWY 101

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Wash Boring and NQ Coring

COMPILED BY TR

DATUM GEODETIC

DATE May 11, 2019

CHECKED BY DAM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
323.4	GROUND SURFACE										
0.0	TOPSOIL (25 mm)					▽					
	Gravelly SILTY SAND (SM) to SILTY SAND (SM), trace gravel Very loose to dense Brown to grey Moist to wet		1	SS	3		323				
	- 50 mm peat layer at 0.8 m depth		2	SS	10		322				
	- Auger refusal on inferred cobble / boulder at 1.5 m depth. Switched to NW Casing.		3	SS	32						
321.1 2.3	GREYWACKE (BEDROCK) Slightly weathered to fresh For coring details see Record of Drillhole NR-3.		1	RC	REC 100%		321				RQD = 27%
			2	RC	REC 100%		320				RQD = 100%
			3	RC	REC 100%		319				RQD = 85%
317.9			4	RC	REC 100%		318				RQD = 100%
5.5	END OF BOREHOLE NOTE: 1. Water level at a depth of 0.1 m below ground surface (Elev. 323.3 m) approximately 30 minutes after completion of drilling.										

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

PROJECT: 1790414

RECORD OF DRILLHOLE: NR-3

SHEET 1 OF 1

LOCATION: N 5342727.7; E 221919.8
NAD83 MTM ZONE 12 (LAT. 48.217944; LONG. -82.115447)
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: May 11, 2019
DRILL RIG: CME55 LC
DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN (FLUSH)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES
		GROUND SURFACE		321.1				
	NW	GREYWACKE Fine grained Strong Light grey Slightly weathered to fresh - Broken rock betwen 2.5 m and 2.8 m depth		2.3	1	Grey 100	BR BR	
3	NQ Coring May 11, 2019				2	Grey 100	JNIRRo JNIRRo MB MB MB	UCS = 77 MPa
4					3	Grey 100	MB MB JNIRRo JNIRRo MB MB MB MB	
5				317.9	4	Grey 100	MB	
		END OF DRILLHOLE		5.5				
6								
7								
8								
9								
10								
11								
12								
13								
14								

Column headings also include: RECOVERY (TOTAL CORE %, SOLID CORE %), R.Q.D. %, FRACT. INDEX PER 0.25 METRES, B Angle, DIP w.r.t. CORE AXIS, Jr, Ja, Jn, HYDRAULIC CONDUCTIVITY k, cm/s, Diametral Point Load Index (MPa), RMC -Q' AVG.

DEPTH SCALE
1 : 60

GOLDER

LOGGED: SA
CHECKED: DAM

RECORD OF BOREHOLE No NR-4

 METRIC

PROJECT 1790414
G.W.P. 5180-16-00
LOCATION N 5342719.5; E 221958.7 NAD83 MTM ZONE 12 (LAT. 48.217876; LONG. -82.114922)
ORIGINATED BY SA
DIST ___ HWY 101
BOREHOLE TYPE NW Casing with Wash Boring and NQ Coring
COMPILED BY TR
DATUM GEODETIC
DATE May 11, 2019
CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
323.5	GROUND SURFACE							
0.0	Fibrous PEAT (150 mm), trace sand, trace silt Soft Black Moist	1A / 1B	SS	2	∇	323		
0.2	Gravelly SILTY SAND (SM) to SILTY SAND (SM), trace gravel Very loose to compact Brown to grey Moist to wet - Trace organics in Sample 1B	2	SS	13			○ (approx. 13)	26 50 (24)
322.1 1.4	SERICITE-CHLORITE SCHIST (BEDROCK) Slightly weathered to fresh For coring details see Record of Drillhole NR-4.	1	RC	REC 100%		322 321		RQD = 70%
		2	RC	REC 100%		320		RQD = 100%
319.0 4.5	END OF BOREHOLE NOTE: 1. Water level at a depth of 0.2 m below ground surface (Elev. 323.3 m) upon completion of drilling.							

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; STRAT PLOT

RECORD OF DRILLHOLE: NR-4

PROJECT: 1790414
LOCATION: N 5342719.5; E 221958.7
NAD83 MTM ZONE 12 (LAT. 48.217876; LONG. -82.114922)
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: May 11, 2019
DRILL RIG: CME55 LC
DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	TYPE AND SURFACE DESCRIPTION	Diametral Point Load Index (MPa) / Notes
		GROUND SURFACE		322.1				
	NW; NQ Coring, May 11, 2019	SERICITE-CHLORITE SCHIST Fine grained Medium strong Light grey Slightly weathered to fresh - Mechnically broken rock between 1.4 m and 2.0 m depth		1.4	1	Grey 100	MB MB JNIRRo JNIRRo JNIRRo MB MB	UCS = 48 MPa
		- Mechnically broken rock between 4.3 m and 4.5 m depth		319.0 4.5	2	Grey 100	MB MB MB MB MB MB MB	
		END OF DRILLHOLE						

Column headers also include: RECOVERY (TOTAL CORE %, SOLID CORE %), R.Q.D. %, FRACT. INDEX METRES, DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION, Jr, Ja, Jn), HYDRAULIC CONDUCTIVITY k, cm/s, Diametral Point Load Index (MPa), RMC -Q' AVG.

DEPTH SCALE
1 : 60

GOLDER

LOGGED: SA
CHECKED: DAM

RECORD OF BOREHOLE No NR-5

 METRIC

PROJECT 1790414

G.W.P. 5180-16-00 LOCATION N 5342714.8; E 221954.0 NAD83 MTM ZONE 12 (LAT. 48.217833; LONG. -82.114984) ORIGINATED BY SA

DIST HWY 101 BOREHOLE TYPE NW Casing with Wash Boring and NQ Coring COMPILED BY TR

DATUM GEODETIC DATE May 11, 2019 CHECKED BY DAM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
323.5	GROUND SURFACE								
0.0	SILTY SAND (SM), trace gravel, trace clay Compact Brown Moist to wet	1	SS	25	▽	323	○		2 62 31 5
322.7 0.8	- 100 mm cobble at 0.7 m depth SERICITE-CHLORITE SCHIST (BEDROCK) Fresh For coring details see Record of Drillhole NR-5.	1	RC	REC 100%		322			RQD = 97%
		2	RC	REC 100%		321 320			RQD = 100%
319.7 3.8	END OF BOREHOLE NOTE: 1. Water level at a depth of 0.2 m below ground surface (Elev. 323.3 m) upon completion of drilling.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

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

PROJECT: 1790414

LOCATION: N 5342714.8; E 221954.0

NAD83 MTM ZONE 12 (LAT. 48.217833; LONG. -82.114984)

INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: NR-5

DRILLING DATE: May 11, 2019

DRILL RIG: CME55 LC

DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN (FLUSH)	DISCONTINUITY TYPE AND SURFACE DESCRIPTION	NOTES
		GROUND SURFACE	322.7				
0.8–3.8	NW (to ~1.1 m); NQ Coring, May 11, 2019	SERICITE-CHLORITE SCHIST Fine grained Medium strong Light grey Fresh - Mechnically broken rock between 0.9 m and 1.0 m depth - Mechnically broken rock between 1.7 m and 2.0 m depth - Mechnically broken rock between 3.3 m and 3.4 m depth	0.8	1	Grey 100	MB MB JNIRRo JNIRRo MB MB	UCS = 41 MPa
				2	Grey 100	MB JNIRRo MB MB MB	
			319.7 3.8				
		END OF DRILLHOLE					

DEPTH SCALE

1 : 60

GOLDER

LOGGED: SA

CHECKED: DAM

GOLDER

RECORD OF BOREHOLE No NR-6

 METRIC

PROJECT 1790414
G.W.P. 5180-16-00
DIST ____ HWY 101
DATUM GEODETIC

LOCATION N 5342711.5; E 221971.7 NAD83 MTM ZONE 12 (LAT. 48.217805; LONG. -82.114745)
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing with Wash Boring and NQ Coring
DATE May 10, 2019

ORIGINATED BY SA
COMPILED BY TR
CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
325.2	GROUND SURFACE						
0.0	Sand (SW), some gravel, trace silt (FILL) Brown Moist	1A	SS	16			
0.2	SILT (ML) of slight plasticity, some sand, some clay, trace gravel Compact Brown Moist	1B				○H	3 16 65 16
324.5 / 0.7	Gravelly SILTY SAND (SM) to SILTY SAND (SM), some gravel, trace clay Compact Brown to grey Moist to wet	2	SS	20			
		3	SS	14	▽		
323.1	Inferred completely to moderately weathered, Brown to grey, TALC SCHIST (BEDROCK)						
2.3	TALC SCHIST (BEDROCK) Slightly weathered to fresh For coring details see Record of Drillhole NR-6.	1	RC	REC 95%			RQD = 71%
		2	RC	REC 98%			RQD = 88%
		3	RC	REC 98%			RQD = 71%
319.6 / 5.6	END OF BOREHOLE NOTE: 1. Water level at a depth of 1.8 m below ground surface (Elev. 323.4 m) approximately 30 minutes after completion of drilling.						

Elevation scale: 325, 324, 323, 322, 321, 320

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³

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

PROJECT: 1790414

RECORD OF DRILLHOLE: NR-6

SHEET 1 OF 1

LOCATION: N 5342711.5; E 221971.7
NAD83 MTM ZONE 12 (LAT. 48.217805; LONG. -82.114745)
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: May 10, 2019
DRILL RIG: CME55 LC
DRILLING CONTRACTOR: George Downing Estate Drilling Ltd.

DATUM: GEODETIC

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	RECOVERY: TOTAL CORE %	RECOVERY: SOLID CORE %	R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA: B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		GROUND SURFACE		322.9																
	NW	TALC SCHIST Fine grained Dark to light grey Slightly weathered to fresh		2.3	1	Grey 100								JNIRRo MB MB MB						
3	NQ Coring May 10, 2019				2	Grey 100								JNIRRo JNIRRo JNIRRo JNIRRo MB MB						
4														JNIRRo						
5				319.6 5.6	3	Grey 100								JNIRRo JNIRRo MB MB MB						
		END OF DRILLHOLE																		
6																				
7																				
8																				
9																				
10																				
11																				
12																				
13																				
14																				

Scales: TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20; R.Q.D. % 80 60 40 20; FRACT. INDEX 5 10 15 20; B Angle 0 90 180 270; DIP w.r.t. CORE AXIS 0 30 60 90; HYDRAULIC CONDUCTIVITY 10^{-6} 10^{-5} 10^{-4} 10^{-3}; Point Load Index 2 4 6

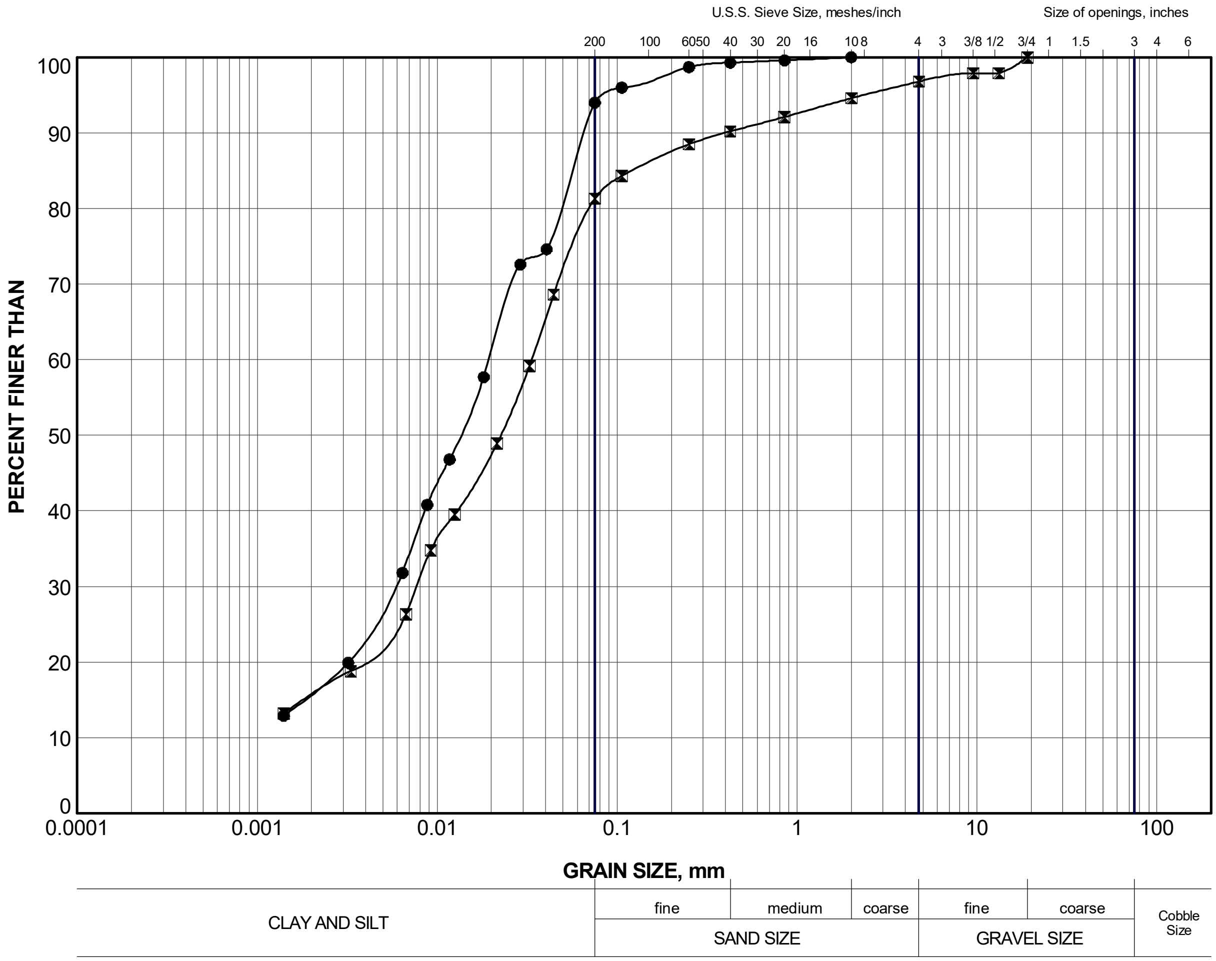
DEPTH SCALE
1 : 60

GOLDER

LOGGED: SA
CHECKED: DAM

APPENDIX B

Laboratory Test Results



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NR-2	2A	322.6
⧗	NR-6	1B	324.8

PROJECT

HIGHWAY 101
NAT RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

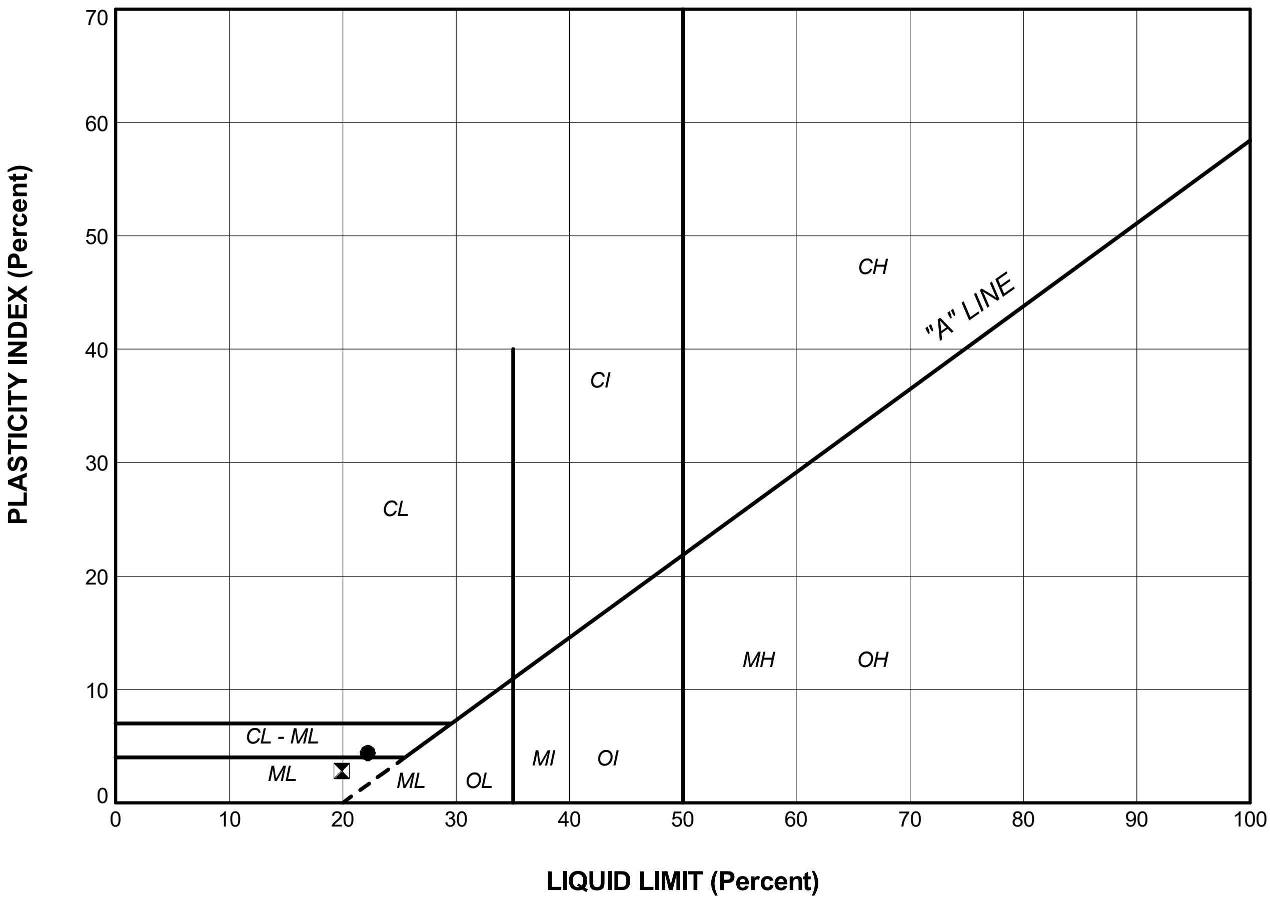
CLAYEY SILT - SILT (CL-ML) to SILT (ML)

GOLDER

SUDBURY, ONTARIO

PROJECT No.		1790414	FILE No.	1790414.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	DAM	Sep 2019	**FIGURE B1**	
APPR	JPD	Sep 2019		

SUD-MTO GSD GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	NR-2	2A	22.2	17.8	4.4
⧗	NR-6	1B	19.9	17.1	2.8

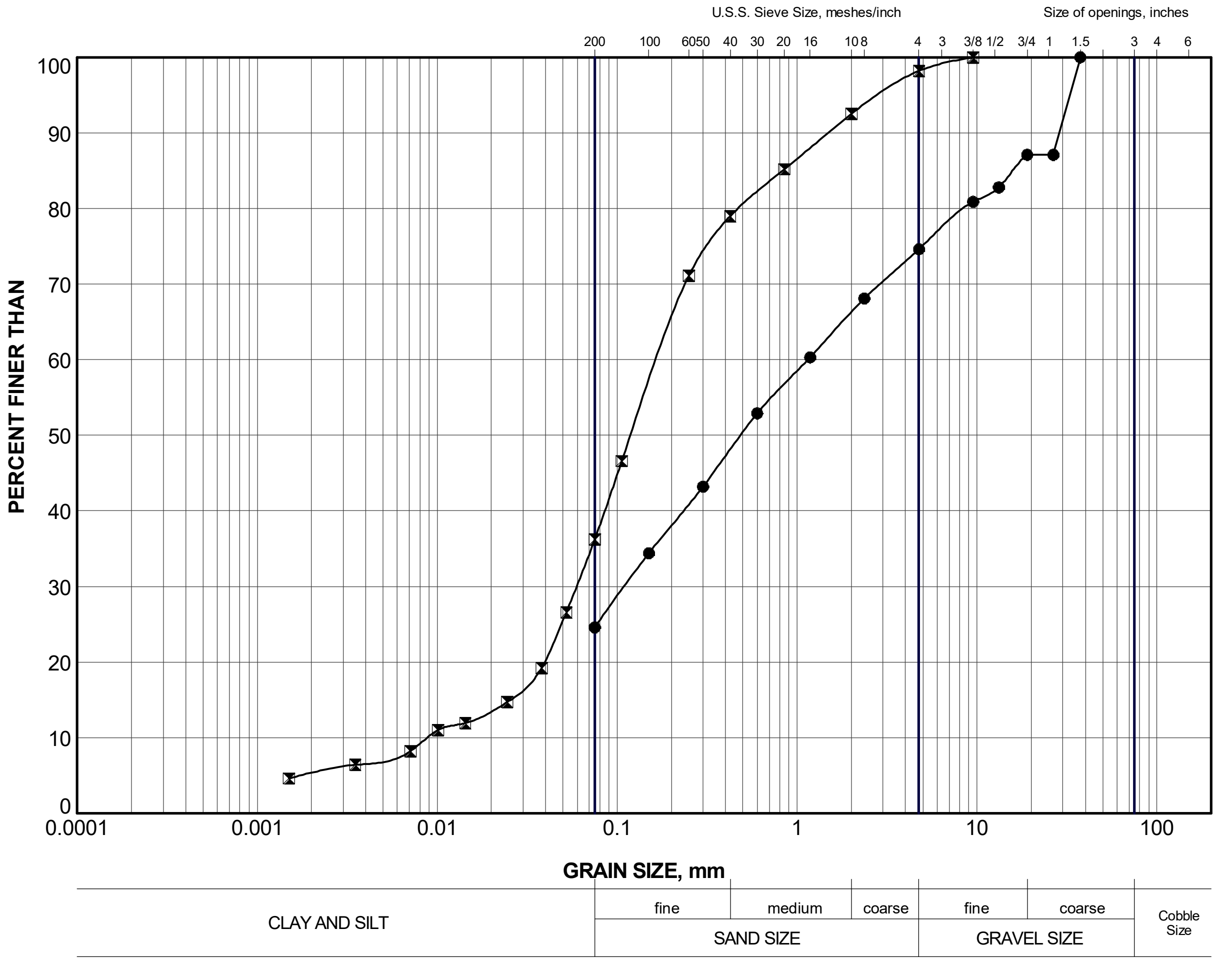
PROJECT
HIGHWAY 101
NAT RIVER BRIDGE

TITLE
PLASTICITY CHART
CLAYEY SILT - SILT (CL-ML) TO SILT (ML)

GOLDER
SUDBURY, ONTARIO

PROJECT No.		1790414	FILE No.	1790414.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	DAM	Sep 2019		
APPR	JPD	Sep 2019	FIGURE B2	

SUD-MTO PL GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NR-4	2	322.4
⧗	NR-5	1	323.2

PROJECT

HIGHWAY 101
NAT RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

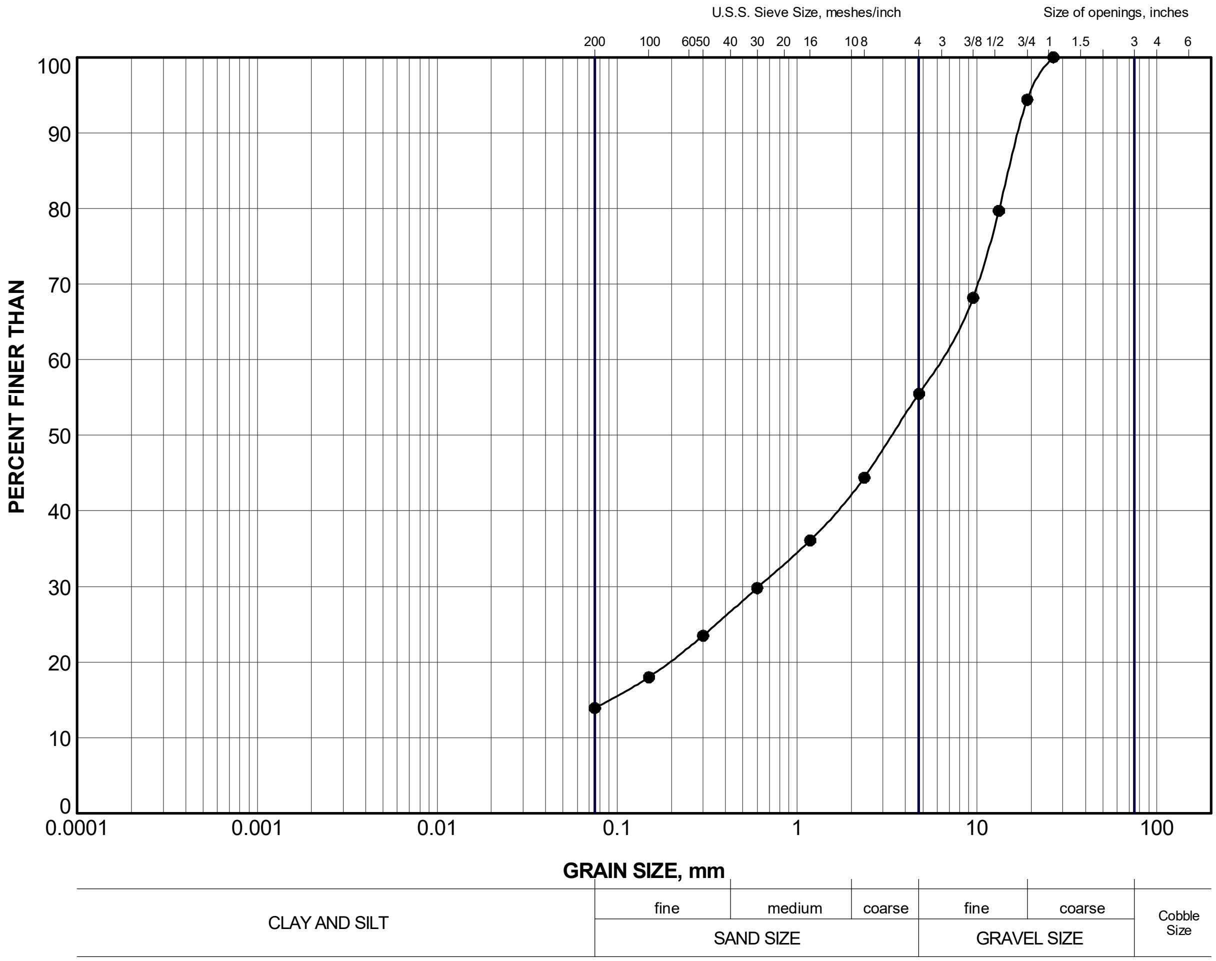
GRAVELLY SILTY SAND (SM) TO SILTY SAND (SM)

GOLDER

SUDBURY, ONTARIO

PROJECT No.		1790414	FILE No.	1790414.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	DAM	Sep 2019	FIGURE	B3
APPR	JPD	Sep 2019		

SUD-MTO GSD GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	NR-1	2	323.6

PROJECT

HIGHWAY 101
NAT RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION
SILTY GRAVEL (GM) AND SAND

GOLDER
SUDBURY, ONTARIO

PROJECT No.		1790414	FILE No.	1790414.GPJ
DRAWN	TR	Sep 2019	SCALE N/A	REV.
CHECK	DAM	Sep 2019	**FIGURE B4**	
APPR	JPD	Sep 2019		

SUD-MTO GSD GLDR_LDN.GDT

GOLDER

Bedrock Core Photographs

Nat River Bridge (Site 46X-011/B0)

Highway 101

323.0 m

Borehole NR-1

Elevation 323.0 m to 319.6 m

322.1 m

319.0 m

Borehole NR-2

Elevation 322.1 m to 319.0 m

0 m 0.3 m 0.6 m

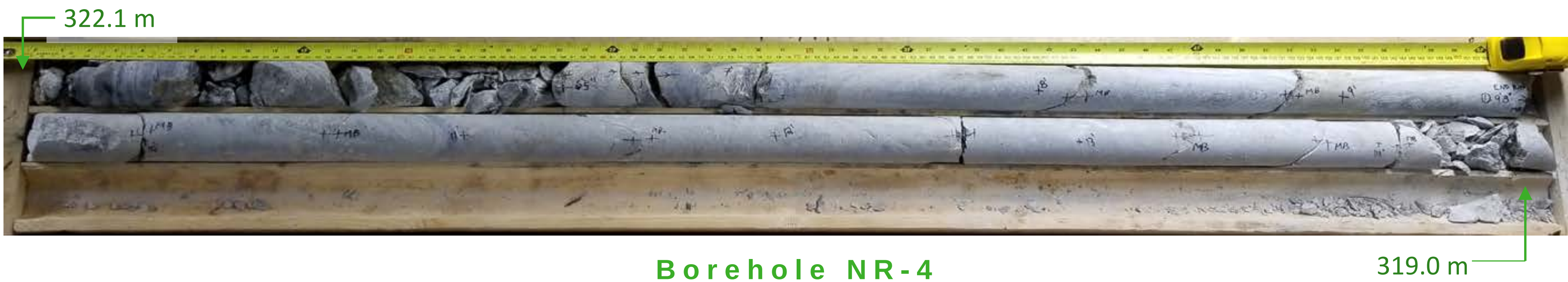
Bedrock Core Photographs

Nat River Bridge (Site 46X-011/B0)

Highway 101

Borehole NR-3

Elevation 321.1 m to 317.9 m



Borehole NR-4

Elevation 322.1 m to 319.0 m

0 m 0.3 m 0.6 m

Bedrock Core Photographs

Nat River Bridge (Site 46X-011/B0)

Highway 101

Borehole NR-5

Elevation 322.7 m to 319.7 m

Borehole NR-6

Elevation 322.9 m to 319.6 m

0 m 0.3 m 0.6 m

GOLDER

Date: August 2019
Project Number: 1790414

Prepared By: KJ
Reviewed By: DAM

Rock Core Test Data

Figure B6

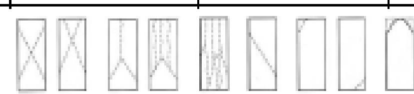
Nat River Bridge (Site 46X-011/B0)
Highway 101

SUMMARY OF ROCK CORE TEST DATA

PROJECT NO.: 1790414-1000-1300
PROJECT NAME: AECOM-Nat River Bridge, Hwy 17/101
TYPE OF UNIT: Rock Core
TESTED BY: JP
DATE TESTED: June 4, 2019

GOLDER LAB NUMBER	T723	T724	T725	T726	
BOREHOLE NUMBER:	NR-2	NR-3	NR-4	NR-5	
SAMPLE NUMBER:	Run 1	Run 2	Run 1	Run 1	
DEPTH OF TESTED CORE	7'6"	12'9"	7'9"	7'0"	
LENGTH AS CUT (mm)	103.5	101.7	101.0	102.1	
DIAMETER (mm)	47.2	47.1	47.1	45.0	
DENSITY (kg/m3)	2764	2821	2841	2775	
COMPRESSIVE STRENGTH (KN)	147.9	133.8	83.8	65.4	
CORRECTED STRENGTH (MPa)	84.6	76.8	48.1	41.2	
TYPE OF FRACTURE	3	3	3	3	

Type of Fracture



1 2 3 4 5 6

COMMENTS:

Input by: SM
Reviewed by: JM

APPENDIX C

Analytical Test Results

Maxxam
A Bureau Veritas Group Company

Your Project #: 1790414
Your C.O.C. #: 712313-01-01

Attention: David Muldowney

Golder Associates Ltd
33 Mackenzie Street
Suite 100
Sudbury, ON
Canada P3C 4Y1

Report Date: 2019/05/29
Report #: R5729598
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B9C8163
Received: 2019/05/13, 15:20

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	3	2019/05/15	2019/05/16	CAM SOP-00463	SM 4500-Cl E m
Conductivity	3	2019/05/17	2019/05/17	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 3)	3	2019/05/21	2019/05/21	BBY8SOP-00017	BCMOE BCLM Dec2000 m
Sulphide in Soil (1)	3	2019/05/21	2019/05/23	BBY6SOP-00052 BBY6SOP-00006	EPA-821-R-91-100 m
pH CaCl2 EXTRACT	3	2019/05/15	2019/05/15	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2019/05/14	2019/05/17	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	3	2019/05/15	2019/05/16	CAM SOP-00464	EPA 375.4 m
Redox Potential (2, 4)	3	N/A	N/A		

Remarks:

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

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam’s profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

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

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by Maxxam, results relate to the supplied samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
(1) This test was performed by Campo to Burnaby - Offsite
(2) This test was performed by Sub from Campo to Env. Testing Canada (Eurofins)
(3) Offsite analysis requires that subcontracted moisture be reported.

Your Project #: 1790414
Your C.O.C. #: 712313-01-01

Attention: David Muldowney

Golder Associates Ltd
33 Mackenzie Street
Suite 100
Sudbury, ON
Canada P3C 4Y1

Report Date: 2019/05/29
Report #: R5729598
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B9C8163
Received: 2019/05/13, 15:20
(4) Oxidation-Reduction Potential (ORP) values are determined using a Ag/AgCl reference electrode.

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Alisha Williamson, Project Manager
Email: AWilliamson@maxxam.ca
Phone# (613)274-0573
==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam
A Bureau Veritas Group Company

Maxxam Job #: B9C8163
Report Date: 2019/05/29

Golder Associates Ltd
Client Project #: 1790414
Sampler Initials: SA

RESULTS OF ANALYSES OF SOIL

Maxxam ID		JSA209		JSA210			JSA210		
Sampling Date		2019/05/09 11:00		2019/05/11 10:30			2019/05/11 10:30		
COC Number		712313-01-01		712313-01-01			712313-01-01		
	UNITS	**AB-1**	**QC Batch**	**NR-2**	**RDL**	**QC Batch**	**NR-2 Lab-Dup**	**RDL**	**QC Batch**
CONVENTIONALS									
Sulphide	ug/g	<0.30	6139910	<0.30	0.30	6139910			
Calculated Parameters									
Resistivity	ohm-cm	24000	6120894	5800		6120894			
Inorganics									
Soluble (20:1) Chloride (Cl-)	ug/g	<20	6122300	60	20	6122300			
Conductivity	umho/cm	42	6127141	174	2	6127141	177	2	6127141
Available (CaCl2) pH	pH	5.34	6122353	6.96		6122355			
Soluble (20:1) Sulphate (SO4)	ug/g	<20	6122302	<20	20	6122302			
Physical Testing									
Moisture-Subcontracted	%	16	6139909	9.9	0.30	6139909			

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

Maxxam ID		JSA211			JSA211		
Sampling Date		2019/05/11 15:00			2019/05/11 15:00		
COC Number		712313-01-01			712313-01-01		
	UNITS	**NR-5**	**RDL**	**QC Batch**	**NR-5 Lab-Dup**	**RDL**	**QC Batch**
CONVENTIONALS							
Sulphide	ug/g	<0.50	0.50	6139910	<0.50	0.50	6139910
Calculated Parameters							
Resistivity	ohm-cm	8100		6120894			
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g	41	20	6122300			
Conductivity	umho/cm	123	2	6127141			
Available (CaCl2) pH	pH	7.31		6122353			
Soluble (20:1) Sulphate (SO4)	ug/g	<20	20	6122302			
Physical Testing							
Moisture-Subcontracted	%	18	0.30	6139909			

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

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Maxxam Job #: B9C8163
Report Date: 2019/05/29

Golder Associates Ltd
Client Project #: 1790414
Sampler Initials: SA

TEST SUMMARY

Maxxam ID: JSA209
Sample ID: AB-1
Matrix: Soil

Collected: 2019/05/09
Shipped:
Received: 2019/05/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6122300	2019/05/15	2019/05/16	Deonarine Ramnarine
Conductivity	AT	6127141	2019/05/17	2019/05/17	Kazzandra Adeva
Moisture (Subcontracted)	BAL	6139909	2019/05/21	2019/05/21	William Zou
Sulphide in Soil	SPEC/UVVS	6139910	2019/05/21	2019/05/23	David Huang
pH CaCl2 EXTRACT	AT	6122353	2019/05/15	2019/05/15	Gnana Thomas
Resistivity of Soil		6120894	2019/05/17	2019/05/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6122302	2019/05/15	2019/05/16	Deonarine Ramnarine
Redox Potential	COND	6146214	2019/05/29		Katherine Szozda

Maxxam ID: JSA210
Sample ID: NR-2
Matrix: Soil

Collected: 2019/05/11
Shipped:
Received: 2019/05/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6122300	2019/05/15	2019/05/16	Deonarine Ramnarine
Conductivity	AT	6127141	2019/05/17	2019/05/17	Kazzandra Adeva
Moisture (Subcontracted)	BAL	6139909	2019/05/21	2019/05/21	William Zou
Sulphide in Soil	SPEC/UVVS	6139910	2019/05/21	2019/05/23	David Huang
pH CaCl2 EXTRACT	AT	6122355	2019/05/15	2019/05/15	Gnana Thomas
Resistivity of Soil		6120894	2019/05/17	2019/05/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6122302	2019/05/15	2019/05/16	Deonarine Ramnarine
Redox Potential	COND	6146214	2019/05/29		Katherine Szozda

Maxxam ID: JSA210 Dup
Sample ID: NR-2
Matrix: Soil

Collected: 2019/05/11
Shipped:
Received: 2019/05/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	6127141	2019/05/17	2019/05/17	Kazzandra Adeva

Maxxam ID: JSA211
Sample ID: NR-5
Matrix: Soil

Collected: 2019/05/11
Shipped:
Received: 2019/05/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	6122300	2019/05/15	2019/05/16	Deonarine Ramnarine
Conductivity	AT	6127141	2019/05/17	2019/05/17	Kazzandra Adeva
Moisture (Subcontracted)	BAL	6139909	2019/05/21	2019/05/21	William Zou
Sulphide in Soil	SPEC/UVVS	6139910	2019/05/21	2019/05/23	David Huang
pH CaCl2 EXTRACT	AT	6122353	2019/05/15	2019/05/15	Gnana Thomas
Resistivity of Soil		6120894	2019/05/17	2019/05/17	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	6122302	2019/05/15	2019/05/16	Deonarine Ramnarine
Redox Potential	COND	6146214	2019/05/29		Katherine Szozda

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TEST SUMMARY

Maxxam ID: JSA211 Dup
Sample ID: NR-5
Matrix: Soil

Collected: 2019/05/11
Shipped:
Received: 2019/05/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphide in Soil	SPEC/UVVS	6139910	2019/05/21	2019/05/23	David Huang

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GENERAL COMMENTS

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

Package 1	5.0°C

Results relate only to the items tested.

Maxxam
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Maxxam Job #: B9C8163
Report Date: 2019/05/29

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1790414
Sampler Initials: SA

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
6122300	Soluble (20:1) Chloride (Cl-)	2019/05/16	114	70 - 130	106	70 - 130	<20	ug/g	NC	35
6122302	Soluble (20:1) Sulphate (SO4)	2019/05/16	101	70 - 130	101	70 - 130	<20	ug/g	20	35
6122353	Available (CaCl2) pH	2019/05/15			100	97 - 103			0.68	N/A
6122355	Available (CaCl2) pH	2019/05/15			100	97 - 103			0.025	N/A
6127141	Conductivity	2019/05/17			104	90 - 110	<2	umho/cm	1.8	10
6139909	Moisture-Subcontracted	2019/05/21					<0.30	%		
6139910	Sulphide	2019/05/23	89	75 - 125	107	75 - 125	<0.50	ug/g	NC	30

N/A = Not Applicable

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

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

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

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

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

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VALIDATION SIGNATURE PAGE

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

Anastassia Hamanov, Scientific Specialist

David Huang, BBY Scientific Specialist

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

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CHAIN OF CUSTODY RECORD

Page of

INVOICE TO:	REPORT TO:	PROJECT INFORMATION:	Laboratory Use Only:
Company Name: #7575 Golder Associates Ltd	Company Name: Golder Associates	Quotation #: B80683	Maxxam Job #: / Bottle Order #: 712313
Attention: Accounts Payable	Attention: David Muldowney	P.O. #:	
Address: 33 Mackenzie Street Suite 100, Sudbury ON P3C 4Y1	Address: 33 MacKenzie St. Suite 100, Sudbury ON P3C 4Y1	Project: 1790414	COC #: C#712313-01-01
Tel: (705) 524-6861 Fax: (705) 524-1984	Tel: 705 524-6861 Fax:	Project Name: / Site #:	Project Manager: Alisha Williamson
Email: AP_CustomerService@golder.com	Email: D_Muldowney@Golder.com	Sampled By:	

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

Regulation 153 (2011): ☐ Table 1 ☐ Res/Park ☐ Medium/Fine; ☐ Table 2 ☐ Ind/Comm ☐ Coarse; ☐ Table 3 ☐ Agri/Other ☐ For RSC; ☐ Table ___

Other Regulations: ☐ CCME ☐ Sanitary Sewer Bylaw; ☐ Reg 558. ☐ Storm Sewer Bylaw; ☐ MISA Municipality ___; ☐ PWQO; ☐ Other ___

Special Instructions

Include Criteria on Certificate of Analysis (Y/N)? ___

ANALYSIS REQUESTED (PLEASE BE SPECIFIC): Field Filtered (please circle): Metals / Hg / Cr VI; Soil Corrosivity Package

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects

Regular (Standard) TAT: ☒
(will be applied if Rush TAT is not specified):
Standard TAT = 5-7 Working days for most tests.
Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.

Job Specific Rush TAT (if applies to entire submission)
Date Required: ___ Time Required: ___
Rush Confirmation Number: ___ *(call lab for #)*

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered / Metals / Hg / Cr VI	Soil Corrosivity Package	# of Bottles	Comments
1	AB-1	Alona Bay CK HWY17 Culvert	May09/19	11:00	Soil		X		
2	NR-2	NAT RIVER Bridge HWY101	May11/19	10:30	Soil		X		
3	NR-5	NAT RIVER Bridge HWY 101	May11/19	15:00	Soil		X		
4									
5									
6									
7									
8									
9									
10									

13-May-19 15:20
Alisha Williamson
B9C8163
URE ENV-1367

Received in Sudbury

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
[signature]	19/05/13	15:20	Bradley Frappier	2019/05/13	15:20	
			Kyle Gilliham Idriz	2019/05/14	08:56	

Laboratory Use Only

Time Sensitive	Temperature (°C) on Recei	Custody Seal	Yes	No
	5, 5, 5 °C	Present		✓
		Intact		

White: Maxxa Yellow: Client

* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO MAXXAM'S STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.MAXXAM.CA/TERMS.

* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

3|3|3

** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT HTTP://MAXXAM.CA/WP-CONTENT/UPLOADS/ONTARIO-COC.PDF.

SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM

Maxxam Analytics International Corporation o/a Maxxam Analytics

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