



April 11, 2018

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

LITTLE ABITIBI RIVER BRIDGE REPLACEMENT - SITE NO. 39E-201
LAT. 49.341686; LONG. -80.485177
HIGHWAY 652, COCHRANE DISTRICT
TOWNSHIP OF SANGSTER
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5416-15-00, WP 5416-15-02

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REPORT





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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
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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 foundation engineering services for the replacement of the Little Abitibi River Bridge (Site No. 39E-201). The bridge is located on Highway 652, 65 km north of Highway 11 in the Township of Sangster, Ontario. The general location of this section of Highway 652 is shown on the Key Plan on Drawing 1.

2.0 SITE DESCRIPTION

It should be noted that the orientation (i.e., north, south, east, west) stated in the text of this report is referenced to project north and therefore may differ from magnetic north shown on Drawing 1. For the purposes of this report, Highway 652 is considered to be oriented in an east-west direction at this site.

In general, the topography in the area of the structure consists of rolling/valley terrain with densely forested areas immediately beyond the Highway 652 right-of-way and the vicinity of the river. The existing bridge consists of an approximately 42.7 m long by 4.6 m wide, three-span, single-lane Temporary Modular Bridge (TMB). Based on the previous General Arrangement (GA) drawing, the existing bridge abutments are supported by timber cribs founded on granular fill pads with the piers founded on driven steel H-piles (HP310x79). Based on the survey drawing provided by AECOM, the bridge deck is at approximately Elevation 263.2 m and 263.5 m at the east and west abutments, respectively. The existing embankment front slopes are about 5.5 m high relative to the river bottom and inclined at a profile ranging from about 1.5 Horizontal to 1 Vertical (1.5H:1V) to 2H:1V. The existing approach embankment side slopes are about 2 m to 3 m high and inclined at a profile ranging from about 1.5H:1V to 2H:1V. The ground surface conditions in the vicinity of the bridge are shown on Photographs 1 to 4. Based on the 2015 Ontario Structure Inspection Manual (OSIM) report, our July 2017 site review, and the available site photographs, the existing embankments appear to be performing satisfactorily.

3.0 INVESTIGATIONS

3.1 Previous Investigation

A previous foundation investigation was completed for the existing bridge in 1981 with the details of the investigation presented in the following report:

- Ministry of Transportation and Communications, 1981. Foundation Investigation Report for Detour Lake Rd. Line 'A', Little Abitibi River Structure, W.P. 7-81-10, Site 39E-201. Geocres No. 42H-015

Borehole 2 from the 1981 investigation, located at the west abutment, was considered suitable for supplementing the current investigation. The location of Borehole 2 has been converted from previous station and offset to approximate coordinates in MTM NAD83 (Zone 12). Further, we understand from AECOM that the elevation of the ground surface at Borehole 2 was originally surveyed to a local datum; for the purposes of this report, the elevation at Borehole 2 has been converted to the geodetic datum based on the 2017 survey provided by AECOM.



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Borehole	MTM NAD 83 Northing	MTM NAD 83 Easting	Approximate Ground Surface Elevation (m)	Borehole Depth (m)
2	5467200.1	342190.2	260.6	15.6

The approximate location of Borehole 2, along with the approximate locations of Boreholes 1 and 3 from the 1981 investigation (which have not been used to supplement the current investigation), are shown on Drawing 1. Records of Boreholes 1 to 3 are presented in Appendix A.

3.2 Current Investigation

The field work for the current subsurface investigation was carried out on July 27 and August 1, 2017, during which time a total of three boreholes (LA-1 to LA-3) were advanced at the approximate locations shown on Drawing 1. Boreholes LA-1 and LA-2 were advanced through the existing highway embankment at the east and west abutments, respectively. Borehole LA-3 was advanced at the north toe of the east approach embankment.

The boreholes were advanced using a track-mounted CME 55LC drill rig supplied and operated by George Downing Estate Drilling Ltd. of Grenville-sur-la-Rouge, Quebec. The boreholes were advanced using 108 mm inside diameter hollow-stem augers with Borehole LA-3 also using NW casing with wash boring, and NQ rock coring (as required to core the bedrock). Soil samples were obtained at depth intervals of 0.75 m and 1.5 m, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer, carried out in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). One in-situ field vane shear test was completed within a cohesive layer in Borehole LA-2 in accordance with ASTM D2573, using an MTO Standard 'N' size vane. All boreholes were backfilled with bentonite and cuttings upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The field work was supervised on a full-time basis by a member of our technical staff, who observed the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and took custody of the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury Laboratory where the samples underwent further visual examination and laboratory testing. Index and classification testing consisting of water content, Atterberg limits and grain size distribution were carried out on selected soil samples. The geotechnical laboratory testing was performed in accordance with MTO LS standards.

Two soil samples were obtained on July 27, 2017, from Boreholes LA-1 and LA-2, using appropriate sampling protocols and submitted to a specialist analytical laboratory under chain-of-custody procedures for testing for a suite of parameters including pH, resistivity, conductivity, sulphates and chlorides. The results of the analytical testing are presented in Table B1 in Appendix B.

The as-drilled borehole locations and ground surface elevations of the boreholes were measured and surveyed by a member of our technical staff, referenced to the highway centreline and existing bridge structure and converted to northings/eastings coordinates. The ground surface elevations were referenced to local benchmarks in the vicinity of the bridge and the benchmark elevations were obtained from the survey drawing provided by AECOM. The MTM NAD83 Zone12 northing and easting coordinates and geographical coordinates, ground



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surface elevations referenced to Geodetic datum, and borehole depths at each borehole location are presented on the Record of Borehole sheets in Appendix A, and summarized below.

Borehole	MTM NAD 83 Northing (Latitude)	MTM NAD 83 Easting (Longitude)	Ground Surface Elevation (m)	Borehole Depth (m)
LA-1	5467216.5 (49.3417584)	342229.5 (-80.4848852)	263.2	9.8
LA-2	5467200.3 (49.3416153)	342187.1 (-80.4854702)	263.5	8.2
LA-3	5467227.1 (49.3418541)	342222.7 (-80.4849777)	260.0	22.2

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain Study (NOEGTS)¹, the Little Abitibi River site is located within a glaciolacustrine plain consisting primarily of clays and silts.

Based on geological mapping by the Ontario Ministry of Northern Development and Mines (MNDM)², the site is underlain by massive to foliated granodiorite to granite and Matachewan and Hearst swarms of mafic and ultramafic bedrock.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the current boreholes and the results of current in-situ and laboratory testing are provided on the Record of Borehole sheets contained in Appendix A. The results of the geotechnical laboratory testing are contained in Appendix B. The results of the in-situ tests (i.e., SPT 'N'-values and field vane) as presented on the borehole records and described in Section 4 are uncorrected values. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 1 and in the section on Drawing 2 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

At the time of the previous 1981 foundation investigation (GEOCRE 42H-15), prior to construction of the existing embankments and bridge, the subsurface soil conditions at the site generally consisted of peat/mixed organics underlain by deposits of soft to firm, low plasticity silt to clayey silt overlying very dense sand and gravel (west abutment) and loose to very dense silty sand overlying bedrock (east abutment).

The subsoil conditions encountered during the current borehole investigation consist of granular embankment fill and/or organic soil underlain by silt to clayey silt, sand and silt to sand, and/or sand and gravel till, which is generally consistent

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

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



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with the results of the previous (1981) investigation. A more detailed description of the soil deposits and groundwater conditions encountered in the boreholes as part of the current investigation (including Borehole No. 2) is provided below.

Deposit/Layer Description	Boreholes	Deposit Thickness (m)	Deposit Surface Elevation (m)	SPT N-Values (blows/0.3 m)	Laboratory Testing
				Field Vane Results (kPa)	
				Consistency or Relative Density	
Asphalt	LA-1 & LA-2	80 mm	263.2 & 263.5	n/a	n/a
(FILL) Sand and Gravel; Sand	LA-1 to LA-3	0.6 – 6.4	263.4 – 260.0	N = 1 – 18 n/a Very loose to compact	w = 4% - 25% 3 – M (Fig. B1)
Peat / Organic Silt	No. 2 & LA-3	0.9 & 0.8	260.6 & 259.4	N = 4 n/a Soft	n/a
Silty Sand (Upper)	LA-3	0.9	258.6	N = 4 Very Loose to Loose	n/a
Silt to Clayey Silt ¹	No. 2 & LA-1 to LA-3	0.5 – 12.1	259.7 – 256.7	N = 3 – 29 S _u = 15 - 47 kPa, S = 1 - 3 Loose to Compact (with Firm clayey silt layers/seams)	w = 18% – 31% 4 - MH (Fig. B2) 5 - MH (Fig. No.1) 4 - AL (Fig. B3) w _L = 18% - 25% w _p = 13% - 18% I _p = 4% - 8% 3 – AL (NP)
Sand and Silt to Sand	LA-1 & LA-3	>2.8 – 8.1	256.2 – 254.4	N = 3 – 25 n/a Very loose to compact	w = 19% – 45% 3 – M/MH (Fig.B4)
TILL- Sand and Gravel	LA-3	5.3	246.3	N = 46-106 n/a Dense to very dense	w = 7% 1 – M (Fig. B5)

Where:

N = SPT 'N'-value; number of blows for 0.3 m of penetration
 s_u = Undrained shear strength from in situ field 'N'-vane (kPa)
 S = Calculated sensitivity
 w = Natural Moisture Content (%)
 M = Sieve Analysis for Particle Size
 MH = Combined Sieve and Hydrometer Analysis

AL = Atterberg limits test
 w_p = Plastic Limit (%)
 w_L = Liquid Limit (%)
 I_p = Plasticity Index (%)
 NP = Non-plastic test result in Atterberg limits

Notes:

1. Plasticity of silt stratum ranges from non-plastic (NP) to low plasticity. Clayey silt layers/seams encountered within this stratum below a depth of 7.1 m (Elevation 256.4 m) in Borehole LA-2.



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4.2.1 Bedrock/Refusal

Bedrock was cored in Borehole LA-3 and the depth/elevation of the bedrock surface is presented below.

Borehole No.	Location	Depth to Bedrock (m)	Bedrock Surface Elevation (m)	Bedrock Coring (m)
No. 2	Existing west abutment	13	247.6	2.6
LA-3	East approach at toe of north slope	19	241.0	3.2

The bedrock core retrieved from the borehole is described as fresh, strongly foliated, dark grey to black, fine grained, biotite rich, metasedimentary bedrock with quartz veining. Additional details of the bedrock core are presented on the Record of Drillhole LA-3 in Appendix A, including data on the discontinuity frequency and type. Photographs of the bedrock core samples from Borehole LA-3 are shown on Figure B7 in Appendix B. The bedrock properties, as encountered in the cored boreholes and/or tested on selected samples, are summarized below.

Borehole No.	Total Core Recovery (TCR)	Rock Quality Designation (RQD)	Quality Classification (Table 3.10 of CFEM 2006 ³)	UCS (MPa)	Strength Classification (Table 3.5 of CFEM 2006)
No. 2	97% - 100%	-	-	-	-
LA-3	100%	90% - 98%	Excellent	45	(R3) Medium Strong

4.3 Groundwater Conditions

The un-stabilized groundwater level measured in the open Borehole LA-3 upon completion of drilling was 0.3 m below ground surface (Elevation 259.7 m). As this borehole was advanced using NW casing and wash boring techniques, the water level may not be representative of stabilized groundwater conditions. The river water level was measured by others to be at Elevation 259.8 m. Groundwater and river water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

5.0 CLOSURE

The field drilling program was supervised by Mr. Mathew Riopelle. This Foundation Investigation Report was prepared by David Muldowney, P.Eng., and the technical aspects were reviewed by Mr. André Bom, P.Eng., a geotechnical engineer and Associate of Golder. Mr. Paul Dittrich, P.Eng., an MTO Foundations Designated Contact and Principal of Golder, conducted an independent quality control review and technical audit of this report.

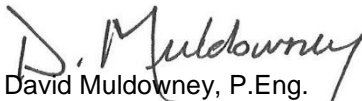
³ Canadian Geological Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.



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Report Signature Page

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https://golderassociates.sharepoint.com/sites/19476g/wo5_5_bridges_hwy_652/11_reporting/002_little_abitibi_river/final/1651997-002-r-rev0_aecom_mto_little_abitibi_river_fidr_11apr_2018.docx



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PART B

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the Little Abitibi River Bridge (Site 39E-201) located on Highway 652 northeast of Cochrane, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation as well as from Borehole No. 2 advanced during the previous (1981) investigation. The discussion and recommendations presented are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the foundations for the replacement structure. Further investigation and analyses will be required during detail design.

The Foundation Investigation Report, discussion and recommendations are intended for the use of MTO and their design team and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor.

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

6.1 General

The existing Little Abitibi River Bridge, which was constructed in 1984, consists of a single-lane, three-span TMB structure approximately 42.7 m long by 4.6 m wide. Based on the available GEOCRESS information, we understand that the existing abutments are supported on timber cribs and the existing piers are supported by steel HP 310x79 piles driven to bedrock. The existing embankment front slopes are about 5.5 m high relative to the river bottom and inclined at a profile ranging from about 1.5H:1V to 2H:1V. The existing approach embankment side slopes are about 2 m to 3 m high and inclined at a profile ranging from about 1.5H:1V to 2H:1V.

Based on the General Arrangement (GA) drawing provided by AECOM on December 20, 2017, we understand that the proposed replacement structure is to consist of a two lane, single-span TBM constructed on the same alignment as the existing bridge. The replacement bridge will be approximately 48.8 m long by 7.4 m wide with the new abutments located about 3 m back from (or behind) the existing abutments. The finished grade of Highway 652 will essentially remain the same.

Based on the results of Boreholes LA-1 and LA-2, and the information on the existing bridge GA drawing, the peat encountered in the previous (1981) investigation within the proposed roadway appears to have been sub-excavated during construction of the granular pads for the existing bridge abutments and prior to construction of the existing approach embankments.



6.2 Consequence and Site Understanding Classification

It is understood that the bridge is to be designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC). A “typical consequence level” is considered appropriate for the Little Abitibi River Bridge 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 Options

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

- **Shallow Foundations:** Shallow foundations perched within the existing granular fill embankment are considered feasible to support the proposed bridge abutments. However, depending on construction staging requirements and considering that the new TMB structure is only marginally longer than the existing bridge, the excavation and construction for new shallow foundations may conflict with the existing abutment foundations. In addition, given the very loose to compact nature of the existing embankment fill at this site, relatively low geotechnical axial resistances are available for shallow foundation design which may not be suitable for design of this structure.
- **Driven Steel H-piles:** Steel H-piles driven to bedrock are feasible for support of the abutments and are preferred for this site if the abutment loads cannot be accommodated by shallow foundations and/or if piles are considered to be preferable from a constructability and/or staging perspective.
- **Drilled steel casings (small diameter):** Drilled steel casings, which are typically between 305 mm and 750 mm in diameter, have the advantage over driven piles of being able to penetrate strata where frequent obstructions are present in overburden soil deposits and where steep bedrock surfaces are present; however, the cost premium for this type of foundation may not be warranted for a TMB replacement structure are not discussed further in this report.
- **Drilled shafts/caissons (large diameter):** Large diameter drilled shafts (caissons) terminating in the bedrock deposit are also considered to be feasible for a deep foundation option; however, caissons are not commonly constructed in Northern Ontario due to constructability issues associated with large-diameter drill holes through wet subgrade soils and into very strong bedrock. As such, drilled shafts/caissons for the replacement structure are not discussed further in this report.

The following sections provide preliminary recommendations for both shallow and deep (i.e., driven pile) foundation options. Shallow foundations may initially be perceived to be more economical than deep (pile)



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foundations, however, considering the relatively low axial geotechnical resistances available for shallow foundations and considering the potential for conflicts during construction with the existing abutment foundations (and the corresponding additional costs for support and/or shoring that may be required), driven steel piles have been identified as the preferred foundation alternative for this site.

6.4 Shallow Foundations

6.4.1 Founding Elevations

If shallow strip or spread footings are selected for support of the new abutments, the strip or spread footing should be founded within the existing granular embankment fill and be provided with a minimum 2.6 m of conventional soil cover (relative to the lowest surrounding grade) for frost protection purposes as further discussed below in Section 6.4.4.

6.4.2 Geotechnical Resistance

Strip or spread footings placed within the existing embankment fill and founded at about Elevation 260.9 m and Elevation 260.6 m (approximately 2.6 m depth) at the west and east abutments, respectively, should be designed based on the factored ultimate geotechnical resistances and factored serviceability geotechnical resistances given below.

Founding Stratum	Footing Width (m)	Factored Ultimate Geotechnical Resistance ¹ (kPa)	Factored Serviceability Geotechnical Resistance (for 25 mm settlement) (kPa)
Very Loose to compact sand fill	1.0	400	125
	1.5	425	85
	2.0	450	75

1. The factored ultimate geotechnical resistances assume that the footings are placed at least 6 m back from (i.e., behind) the crest of the front slope.

The factored geotechnical resistances and corresponding settlement are dependent on the footing size, depth of embedment, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differ from those given above. In addition, 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 preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during Detail Design. Further, the stability of the front slopes under the additional loading from the footings should be checked during Detail Design if the shallow foundation option is selected.



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6.4.3 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the concrete footings and the granular embankment fill (or a granular levelling course) should be calculated in accordance with Section 6.10.5 of the CHBDC (2014) applying the appropriate consequence and degree of site understanding factors as noted in Section 6.2. For cast-in-place concrete footings founded on the granular embankment fill / levelling course, the coefficient of friction ($\tan \delta$) should be taken as 0.5; for precast footings, the coefficient of friction ($\tan \delta$) should be taken as 0.4.

6.4.4 Frost Protection

In the Cochrane area, the frost penetration depth, as per Ontario Provincial Standard Drawing (OPSD) 3090.100 (Foundation Frost Penetration Depths for Northern Ontario) is estimated to be 2.6 m. Therefore, to minimize the potential for damage due to frost action, foundations (i.e., footings and/or pile caps) should be provided with at least 2.6 m of conventional soil cover or an equivalent combination of insulation and soil cover. As a guideline for design, 25 mm of rigid polystyrene foam insulation provides a 300 mm reduction in soil cover.

At this site, the footings would be constructed within the existing granular fill, which is considered to be a free-draining material with a relatively low frost susceptibility based on the classification systems provided in the MTO Pavement Design and Rehabilitation Manual (2013). As such, consideration could be given to placing the foundations at shallower depths and/or reducing the thickness/extent of insulation to address potential constructability issues related to the close proximity of the existing and proposed bridge abutments. These recommendations should be reviewed and/or further refined during detail design.

6.5 Driven Steel Piles

Deep foundations consisting of steel piles driven to bedrock are feasible and preferred for the support of the proposed structure abutments. For the installation of steel H-piles (or steel pipe piles), consideration must be given to the potential presence of cobbles and boulders within the glacially derived till deposit at this site. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to have a greater potential of “hanging up” or being deflected away from their vertical orientation or ‘batter’ during installation, if obstructions are encountered.

6.5.1 Founding Elevations and Axial Geotechnical Resistances

The following summarizes the proposed elevation of the underside of the pile cap, the pile tip elevation, pile length, as well as the factored geotechnical resistances for HP310x110 driven steel piles at the proposed abutments.



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Foundation Element (Boreholes)	Pile Size	Elevation of Underside of Pile Cap ¹	Pile Tip Elevation ²	Length of Pile from Underside of Pile Cap	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement) ²
West Abutment (BH-2)	HP 310x110 ³	260.9 m	247.0 m	13.0 m	2,000 kN	N/A
East Abutment (LA-3)		260.6 m	241.0 m	19.6 m		

1. Based on a minimum 2.6 m of frost protection as per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario).
2. The factored serviceability geotechnical resistance for 25 mm of settlement will be greater than or equal to the factored ultimate geotechnical resistance and therefore, at this site, the serviceability geotechnical resistance does not apply.
3. If consideration is given to installing steel HP360x132 to have consistent piles sizes with the other modular bridges for this project, a factored ultimate geotechnical resistance of 2,400 kN will apply.

6.5.2 Downdrag

Based on discussion and preliminary staging drawings from AECOM, we understand that embankment widening is required at this site to accommodate the wider bridge structure and the staging/laydown areas, which will result in minor settlements as discussed in Section 6.7.4.4. Although the anticipated settlement at the west and east abutments is relatively minor (i.e., 25 mm or less), there is still a potential for downdrag loads to occur along the new piles. Based on published literature (Poulos and Davis, 1980 and Fellenius and Broms, 1969), downdrag loads can be induced by relative pile-soil movements (i.e., settlement of the foundation stratum relative to the piles) as small as a few millimetres. As such, there is a risk of downdrag loads occurring on the new piles at both the west and east abutments and mitigation measures to reduce these drag loads should be considered as part of the design/construction sequencing if the structural capacity of the piles cannot tolerate the additional downdrag loads outlined below. At this site, given the relatively limited time required for primary consolidation to occur (i.e., about two weeks), it is recommended to mitigate the risk of downdrag loads on the piles by preloading the laydown/launch area embankment prior to installation the new abutment piles. If preload mitigation is not carried out, downdrag loads (negative skin friction) may be induced on the piles supporting the abutments due to settlement of the surrounding silt to clayey silt and friction/adhesion along the piles.

The structural design of the west and east abutment piles (HP310X110 or HP360X132) should be based on the following estimated unfactored downdrag loads:

Foundation Location	HP 310x110	HP 360x132
West Abutment	525 kN	600 kN
East Abutment	145 kN	165 kN



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The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.11.4.10 of the CHBDC (2014) and its Commentary for factored ultimate and serviceability conditions.

The preliminary factored geotechnical resistances and downdrag loads provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation at the foundation elements.

6.5.3 Set Criteria

Pile installation should be carried out in accordance with OPSS 903 (Deep Foundations). For end-bearing piles, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on or within the bedrock, and then to gradually increase the energy over a series of blows to seat the pile in the bedrock. A Non-Standard Special Provision (NSSP) that outlines criteria for seating the piles on bedrock should be prepared at Detail Design.

The piles should be fitted with rock points such as Titus Injector or Oslo Point as per Ontario Provincial Standard Drawing OPSD 3000.201 (HP310 Oslo Pont), or equivalent to assist in seating the pile on sloping bedrock. An NSSP should be prepared at Detail Design.

The pile driving note that should be added to the drawings is Note 2 in Clause 3.3.3 of the Structural Manual (MTO, 2014) as follows:

- “Piles to be driven to bedrock”

The piles should be tapped to confirm they are seated on the bedrock.

6.5.4 Resistance to Lateral Loads

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

Where ground conditions are generally competent and the lateral loads on piles are relatively small, such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory as outlined below. However, it should be noted that the response of a pile to lateral loads is highly nonlinear and methods that assume linear behavior (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1 percent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behavior of the soil should be considered by the use of P-y curves.



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The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory suggested in the CHBDC (2014) Commentary (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction, k_h , (kPa/m) is based on the equations given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (CFEM 1992).

For cohesionless soils:

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

and for cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{where} \quad \begin{array}{l} s_u \text{ is the undrained shear strength of the soil (kPa)} \\ B \text{ is the pile diameter or width (m)} \end{array}$$

The lateral resistance of the piles should be developed primarily from the passive resistance of the soil. The values of n_h and s_u to be used to calculate the coefficient of horizontal subgrade reaction (k_h) to be utilized in the structural analysis for the piles at this location are given below.

Foundation Element (Relevant Borehole)	Soil Unit	Elevation (m)	n_h (kPa/m)	s_u (kPa)
West Abutment (LA-2 and BH-2)	Sand Fill (loose to compact) (above water table)	260.9 to 259.8	4,400	-
	Sand Fill (loose to compact) (below water table)	259.8 to 259.7	1,300	-
	Silt to Clayey Silt (Loose to compact/Soft to Firm)	259.7 to 247.6	2,800 ¹	25 ¹
East Abutment (LA-1 and LA-3)	Sand Fill (loose to compact) (above water table)	260.6 to 259.8	4,400	-
	Sand Fill (loose to compact) (below water table)	259.8 to 256.7	1,300	-
	Silt to Sand (very loose to compact)	256.7 to 246.3	2,800	-
	Sand and Gravel Till (dense to very dense)	246.3 to 241.0	11,000	-

Note:

- Given the potential cohesive nature of the silt to clayey silt deposit, the lateral resistance should be evaluated for both n_h and s_u conditions and the more conservative value of k_h adopted for design.

It is recommended that both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case. For serviceability, the horizontal reaction should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (CHBDC (2014) Commentary Section 6.11.2.2).



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The upper zone of soil (down to a depth below the pile cap equal to about $1.5 \times B$ (where B = pile diameter)) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during installation.

Group action for lateral loading should also be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1982) in the direction of loading by a reduction factor, R , as follows:

Pile Spacing in Direction of Loading d = Pile Diameter	Subgrade Reaction Reduction Factor
8d	1.00
6d	0.70
4d	0.40
3d	0.25

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

6.5.5 Frost Protection

The pile cap at the abutment locations should be provided with a minimum of 2.6 m of conventional soil cover or an equivalent combination of insulation and soil cover for frost protection as discussed above in Section 6.4.4.

6.6 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the borehole investigations. Based on the anticipated foundation levels, the site may be classified as Site Class D "Stiff Soil" in accordance with Table 4.1 of the CHBDC (2014), in the absence of any geophysical testing. Geophysics testing (i.e., shear wave velocity measurements), if carried out, could potentially provide a more favourable Site Class C designation. Site Classes A and B, however, are not appropriate for this site.

Based on the information obtained from the NRCAN (2015) Hazard Calculator for this site located at latitude 49.341656° and longitude -80.48518° , 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 return period)
$S_a(0.2)$ (g)	0.152
$S_a(1.0)$ (g)	0.045



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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. If this structure is considered a lifeline structure, it should be considered to be within Seismic Performance Zone 2 and detailed seismic analysis may be required (but only if a multiple span option is considered for the structure).

6.7 Approach Embankments

Based on discussions with AECOM we understand that the finished grade of Highway 652 at this site is to be maintained (i.e., no grade raise); however, an approximately 10 m widening of the existing approach embankments will be required on the south side of the existing bridge to facilitate the proposed constructing staging (i.e., a laydown/launch area and temporary landing area).

6.7.1 Removal of Organics

It is recommended that all existing organics (i.e., peat, topsoil, organic silt and/or mixed organic soil) be removed below the footprint of the proposed embankment widenings within the limits of the approach embankments (i.e., up to about 20 m beyond the abutments) to mitigate settlements and maintain stability.

At the south side of the west approach embankment, sub-excavation is anticipated to be required to approximately 0.9 m below ground surface to remove the peat based on the results of Borehole 2. At the south side of the east approach embankment, sub-excavation is anticipated to be required to approximately 1.4 m below ground surface to remove the fill/organic silt based on Borehole LA-3 and Borehole 1. All excavations should be backfilled with appropriate granular material as discussion below in Section 6.7.2.

6.7.2 Subgrade Preparation and Embankment Construction

Fill for reconstruction of the highway embankment behind the new abutments and for the proposed widening(s) and shoulder(s) should consist of granular fill OPSS.PROV 1010 (Aggregates) Granular 'A', Granular 'B' (Type I or II) or rock fill. From a geotechnical/foundations perspective Granular 'B' Type I (i.e., sand fill) will provide good compatibility with the existing Highway 652 embankment fill materials remaining in place in the existing embankment side slopes. However, for the portions of backfilling required below the existing ground surface (and in particular, below the groundwater level) as part of the sub-excavation and replacement of organic soils, it is recommended that Granular 'B' Type II material be used. The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and OPSS.PROV 2016 (Grading). Granular fill embankment side slopes should be constructed no steeper than 2H:1V. Benching of the existing highway embankment should be carried out to "key in" the new fill materials for the widening, in accordance with OPSS 208.010 (*Benching of Earth Slopes*).

The approach embankment front slopes and side slopes adjacent to the river require erosion protection in accordance with OPSS 511 (Rip Rap, Rock Protection and Granular Sheet piling). Erosion protection should be



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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. The structural designer should address the potential for scour below the footings or pile caps in the design of the bridge foundations.

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 protection measures, such as covering the slopes with straw or gravel sheeting as per OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting) to prevent erosion, will be required to reduce the potential for remedial works on the side slopes in the Spring prior to topsoil dressing and seeding.

6.7.3 Approach Embankment Stability

Based on our review of the available GEOCREST report and the results of the current investigation, we understand that the peat/organic soils were previously sub-excavated prior to construction of the existing highway embankments. The analysis discussed below assumes that all organics at the toe of the embankment slope will be sub-excavated and replaced with granular fill prior to construction of the new embankment widenings.

6.7.3.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program Slope/W from GeoStudio 2007 (Version 7.23), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure 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 Factor of Safety (FoS) is defined as the ratio of forces tending to resist failure to the driving forces tending to cause the failure. For the 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. $FoS = 1/(\psi * \phi_{gu})$). Accordingly, a target minimum FoS of 1.3 has been used for design of the end-of-construction embankment side slopes, and a FoS of 1.5 for the design of the final embankment configuration as per Table 6.2 of CHBDC (2014) for the total stress (short-term undrained) and effective stress (long-term, drained) conditions, as applicable.

The stability analyses carried out for the preliminary design includes assessment of the existing west approach front slope as well as the proposed 10 m widening along the south side of the west approach embankment. The stability analyses were completed based on the subsurface conditions as encountered in Boreholes LA-1 (current investigation) and Boreholes 2 and 3 (previous investigations) and the geometries provided in the GA drawing and cross-sections provided by AECOM. For the proposed granular fill widening to be constructed on top of the existing granular embankment fill side slopes at this site, the target FoS will be achieved so long as: the new slopes are no steeper than 2H:1V; all organic soils are removed prior to construction; and the surface of the slopes are provided with erosion protection.



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6.7.3.2 Parameter Selection

For the new/existing granular fill and the non-cohesive silt, sand, and sand and gravel till deposit, effective stress parameters were employed in the analysis assuming drained conditions, and the parameters were estimated from empirical correlations using 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 native silt to clayey silt deposit, total stress parameters were employed in the embankment for the short-term, undrained conditions. The total stress parameters (i.e., average mobilized undrained shear strength - s_u) for the cohesive soils were assessed based on the results of in-situ field vane shear tests from the current and 1980 investigation (GEOCRE 42H-15) and also estimated from correlations with the SPT results from the current investigation. Effective friction angles have also been estimated for this cohesive deposit for analysis of the factor of safety in the long-term condition. Further, given the borderline cohesive nature of the silt to clayey silt deposit, both the short- and long-term conditions have been assessed using the undrained shear strength parameters and effective friction angle parameters.

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 (i.e. Granular A or B Type I or II)	21	35	-	35
Existing Granular Embankment Fill	20	32	-	32
Silt to Clayey Silt	18	-	25	27
Sand and Silt to Sand	20	30	-	30
Sand and Gravel (TILL)	20	32	-	32

6.7.3.3 Results of Analysis

The stability analyses indicate that the approximately 5 m high (to the river bottom) front slope of the west approach embankment inclined at approximately 2H:1V has a FoS against global instability in the short-term (undrained) condition and long-term (drained) conditions greater than 1.5, as shown on Figures 1 and 2, respectively. Similarly, the approximately 4 m high widened south side slope of the west approach embankment also meets/exceeds the minimum required FoS for short-term and long-term conditions as shown on Figures 3 and 4, respectively. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the final embankment geometries and incorporating any additional loadings (i.e. if shallow foundations are adopted and/or any additional loadings as part of the replacement bridge construction) or subsurface information obtained during detail design.



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6.7.4 Approach Embankment Settlement

6.7.4.1 Methodology

To estimate the magnitude of the settlement due to the embankment widenings, analyses were carried out on the critical section of the widened north approach embankment near the proposed abutment using the commercially available computer program *Settle-3D* (Version 3.020) from Rocscience Inc. as well as hand calculations. The sources of settlement were considered to include:

- Immediate settlement of the cohesionless deposits.

As a large portion of the silt to clayey silt deposit is considered non-plastic (cohesionless) with some samples tested indicating a borderline cohesive material (i.e. silt of slight plasticity), for the purposes of this settlement estimate, the silt to clayey silt deposit is considered to be a cohesionless deposit rather than a cohesive deposit.

It is recommended that all organic soils be removed from below the footprint of the proposed embankment widenings prior to construction and as such, the settlement analyses assume that these soils have been removed.

6.7.4.2 Settlement Criteria

Based on MTO's "*Embankment Settlement Criteria for Design*" (MTO, 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
Longitudinal Transitions (Non-Freeways)	25 (0 to 20 m from abutment) 50 (20 m to 50 m from abutment) 75 (50 m to 75 m from abutment)	n/a
Widened Embankments (Non-Freeways)	75	100:1

These criteria have been used for determining 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.

6.7.4.3 Parameter Selection

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

The immediate compression of the non-cohesive deposits was modelled by estimating an elastic modulus of deformation based on the SPT "N"-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990).



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Deposit	Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Existing Granular Fill (Very Loose to Compact)	20	5 to 10
Silt to Clayey Silt (Very Loose to Compact / Soft to Firm)	18	5
Sand and Silt to Sand (Very Loose to Compact)	20	15
Sand and Gravel Till (Very Dense)	20	50

6.7.4.4 Results of Analysis

A summary of the results of the settlement analysis for the approach embankment widening is presented below.

Critical Section	Relevant Borehole	Settlement During Construction		
		Existing Hwy 652 Centerline	Existing Hwy 652 Edge of Shoulder (South side)	Proposed West Crest of Laydown/Launch Area (i.e., near Existing West Toe of Slope)
West Approach	LA-2 and BH-2	10 mm	25 mm	120 mm
East Approach	LA-1 and LA-3	5 mm	15 mm	70 mm

It is anticipated that the majority of the settlement will be immediate and will occur primarily during embankment construction. If the schedule permits, we recommend placing the embankment fill for the lay down area at least 2 weeks prior to the opening of the lay down area, then re-grading the area as required, to mitigate embankment settlement and reduce the risk of downdrag loads on the new abutment piles, if required (as discussed in Section 6.5.2). The results of the settlement analysis indicate the settlement criteria of less than 20 mm for approach embankments adjacent to structures (within 20 years) will be achieved. 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 fill, 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 widening configuration



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including any additional loadings (i.e., if shallow foundations are adopted and/or any additional loadings as part of the replacement bridge construction).

6.8 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.

6.8.1 Excavation and Temporary Roadway Protection

The excavations for pile caps (or for spread footings) would extend approximately 2.6 m into the loose to compact granular embankment fill (unless rigid insulation is used to provide frost protection to foundation elements founded at a higher elevation). If space permits, (giving due consideration to the proximity of the existing abutment foundations and requirements for construction staging), open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing granular embankment fill should be classified as Type 3 soil, according to the OHSA. Temporary open-cut excavations in Type 3 soils should remain stable if side slopes are formed no steeper than 1H:1V.

Excavations are also anticipated for removal of the organics prior to embankment widening(s). The organic soils are classified as Type 3 soils above the ground water level and Type 4 soils below the groundwater level. Temporary open-cut excavations in Type 3 soils should remain stable if side slopes are formed no steeper than 1H:1V. In Type 4 soils, the side slopes should be formed no steeper than 3H:1V.

Given that the existing bridge is a single-lane structure, it is anticipated that a full road closure will be required for installation of the replacement bridge foundations and as such, temporary shoring support systems may not be required, depending on the type of new foundation selected and the proximity to the existing abutment foundations. However, if required, the temporary support systems could consist of either driven steel sheet-piling or soldier piles and lagging. Support to the system could be in the form of struts and wales and rakers or anchors. Depending on the required depth of temporary shoring system, installation of sheet-piles could be impeded by the potential presence of cobbles/boulders within the till deposit and/or by the very dense zones within the till deposit.

All temporary excavation support systems should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539. Design of the temporary support system should include an evaluation of base stability, soil squeezing stability and hydraulic uplift stability as defined in the Canadian Foundation Engineering Manual (CFEM 2006). The design of the temporary support systems, as may be required for the temporary staging, is the responsibility of the Contractor, and may be designed using the following parameters:



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Soil Type	Unit Weight	Internal Angle of Friction	Coefficient of Earth Pressure ⁽¹⁾		
	(γ , kN/m ³)	(ϕ , degrees)	Active, K_a	At Rest, K_o	Passive, K_p ⁽²⁾
New Granular Fill	21	35	0.27	0.43	3.69
Existing Granular Fill (Very Loose to Compact)	20	32	0.31	0.47	3.25
Silt to Clayey Silt (Very Loose to Compact / Soft to Firm)	18	27	0.38	0.55	2.66
Sand and Silt to Sand (Very Loose to Compact)	20	30	0.33	0.50	3.00
Sand and Gravel Till (Very Dense)	20	32	0.31	0.47	3.25

Notes:

1. 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.
2. The total passive resistance below the base of the excavation (i.e., 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.

6.8.2 Groundwater Control

Excavation and construction for pile cap(s) or footing(s) is anticipated to be carried in dry conditions, within the existing embankment fill since the underside of foundations (at Elevation 260.9 m to 260.6 m) are above the adjacent river level (at Elevation 259.8 m in August 2017) and are also above the groundwater encountered in Borehole LA-3 (at Elevation 259.7 m). However, depending on the time of year that construction is carried out, it is possible that groundwater levels could be higher and/or that perched groundwater could be present within the embankment. Dewatering, if required, could be handled in the form of pumping from properly filtered temporary sumps installed below the base of the excavation.

Excavations up to about 0.9 m and 1.4 m below the existing ground surface (based on available information) will be required at the toe(s) of the embankment slope(s) for removal and replacement of the organics below the footprint of the widened embankments. It is anticipated that portions of these excavations will be below the groundwater/river water level. However, the excavation and backfilling in these areas could be carried out in-the-wet, so long as the recommendations in Section 6.7.1 and Section 6.7.2 are followed along with the requirements of OPSS 209 and OPSS 206.

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



6.8.3 Obstructions

The soils at this site (in particular the till soils) may contain cobbles and boulders, which could affect the installation of temporary support systems (including cofferdams) and/or deep foundations. Records of the frequency of encountering cobbles and/or boulders are recommended in the next stage of investigation in support of the detail design. An NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils.

6.8.4 Vibration Monitoring During Pile Installation

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities (such as pile driving) will reach this threshold level.

6.8.5 Existing Structure Monitoring

We recommend that the piers and abutments of the existing structure be monitored for settlement and lateral movement during the new construction, especially during excavation for the new abutments and while pile driving through the till stratum for the following reasons:

- the age of the existing structure
- the close proximity of the existing and proposed abutments
- the requirements for relatively large embankment widenings (up to about 10 m) for staged construction
- the requirement for the existing structure to carry traffic at stages during construction

This monitoring could be carried out using survey points (lateral and vertical deformation) and/or settlement points. An NSSP should be included in the Contract Documents developed during the detail design stage.

6.8.6 Analytical Testing for Construction Materials

The results of analytical tests on two samples of soil taken from the abutment boreholes at about the anticipated foundation (i.e. footing or pile cap) elevation are presented in Table B1 in Appendix B. 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 reinforcing elements.

For potential sulphate attack on concrete, the results of the soil analysis were compared to Table 3 in CSA A23-1-09, and indicate that the relative degree of sulphate attack is low (less than the moderate range). However, given that the bridge is located on Highway 652 and will be exposed to de-icing salts, it is recommended that C-1 class exposure concrete be considered for the footings or pile caps and abutments. Further, the resistivity results indicate that the granular fill has a very low corrosiveness potential based on the Transportation Research Board Guidelines (Transportation Research Board, National Research Council, 1998 as referenced in the MTO Gravity Pipe Manual, 2014). It should be noted that the soil chemistry may vary due to precipitation events and



PRELIMINARY FOUNDATION REPORT LITTLE ABITIBI RIVER BRIDGE REPLACEMENT SITE 39E-201 HIGHWAY 652

variations in water chemistry. These recommendations are provided as guidance only; the structural designer should take the results of the laboratory testing, the potential for corrosion, and the ultimate selection of materials each into consideration.

6.8.7 Recommendations for Further Work During Detail Design

Based on conversations with AECOM and MTO, we understand that additional foundation investigation and analysis (i.e., detail foundation investigation and design) is not being considered for this project which may present some risk to the construction and performance of the structure and associated approach embankments. Foundation related risk could be further mitigated at this site by advancing additional boreholes at the foundation elements and within the approach embankment widening areas, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided herein, as follows:

- further assessment of the depth and extent of any organics, cohesive fill and granular fill (i.e., previous construction) within the footprint of the widened approach embankments to be removed as part of the new construction (particularly on the south side of the east and west embankments where no borehole information currently exists)
- further assessment of the estimated magnitude of settlement under the widened approach embankments
- further assessment of the stability of the embankment front slopes and side slopes based on the final embankment geometries and any additional loadings on the embankments as part of the staging and replacement bridge construction
- assessment of the requirements for any temporary foundations in the laydown/launch area and temporary landing area as part of the replacement bridge construction
- further analytical testing for soil/groundwater corrosivity

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Adam Core, P.Eng., and the technical aspects were reviewed by Mr. André Bom, P.Eng., a geotechnical engineer and Associate of Golder. Mr. Paul Dittrich, Ph.D., P.Eng., an MTO Foundations Designated Contact and Principal with Golder, conducted an independent quality control review of this report.



**PRELIMINARY FOUNDATION REPORT
LITTLE ABITIBI RIVER BRIDGE REPLACEMENT
SITE 39E-201 HIGHWAY 652**

Report Signature Page

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https://golderassociates.sharepoint.com/sites/19476g/wo5_5_bridges_hwy_652/11_reporting/002_little_abitibi_river/final/1651997-002-r-rev0_aecom_mto_little_abitibi_river_fidr_11apr_2018.docx



PRELIMINARY FOUNDATION REPORT LITTLE ABITIBI RIVER BRIDGE REPLACEMENT SITE 39E-201 HIGHWAY 652

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- Canadian Geotechnical Society, 1992. Canadian Foundation Engineering Manual, 3rd Edition.
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- Poulos, H.G. and Davis, E.H. 1980. Pile Foundation Analysis and Design. John Wiley and Sons.
- Terzaghi, K. and Peck, R.B. 1967. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York.
- Unified Facilities Criteria, U.S. Navy. 1986. NAVFAC Design Manuals 7.01 and 7.02. Soil Mechanics, Foundation and Earth Structures. Alexandria, Virginia.

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil

Commercial Software

- Slope/W - Geostudio (Version 7.23) by Geo-Slope International Ltd.
- Settle-3D (Version 3.020) by RocScience Inc.

Ontario Provincial Standard Drawings

OPSD 208.010	Benching of Earth Slopes
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PRELIMINARY FOUNDATION REPORT LITTLE ABITIBI RIVER BRIDGE REPLACEMENT SITE 39E-201 HIGHWAY 652

OPSD 3000.201	Foundation, Piles, Steel H-Pile Olso Point
OPSD 3090.100	Foundation, Frost Penetration Depths for Northern Ontario

Ontario Provincial Standard Specifications

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting
OPSS.PROV 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 903	Construction Specification for Deep Foundations
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Water Resources Act

Ontario Regulation 903/90	Wells: O.Reg. 468/10 Amendment to Ontario Regulation 903
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PRELIMINARY FOUNDATION REPORT LITTLE ABITIBI RIVER BRIDGE REPLACEMENT SITE 39E-201 HIGHWAY 652

Table 1: Evaluation of Abutment Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Driven Steel H-Piles	1	<ul style="list-style-type: none"> ■ Relatively straight forward construction. ■ Potentially smaller excavation required for pile cap construction (as compared with spread footing option) which may result in less conflict with existing abutment foundations thereby reducing the requirements for temporary protection/shoring. ■ Higher geotechnical axial resistances compared to spread footings founded on existing embankment fill. ■ Pile foundations have lower risk of being affected by adjacent excavations for sub-excavation and replacement of organic and clayey soils below the widened footprint. 	<ul style="list-style-type: none"> ■ Some potential for refusal or "hanging up" of piles on cobbles and boulders within till deposit, but likely easier to advance than pipe piles ■ Vibration monitoring recommended during pile driving adjacent to the existing structure. 	<ul style="list-style-type: none"> ■ Relative costs higher than shallow foundations due to requirements to mobilize pile driving rig. ■ Relative costs lower than other deep foundation options. 	<ul style="list-style-type: none"> ■ Some risk of vibrations during driving affecting existing bridge. ■ Vibration monitoring and settlement/lateral movement monitoring recommended to identify and control risks.
Spread Footings	2	<ul style="list-style-type: none"> ■ Straightforward construction ■ Use of pre-cast footing(s) could accelerate construction. 	<ul style="list-style-type: none"> ■ Low geotechnical axial resistances may not be suitable resistances for structure. ■ Potential for differential settlement across existing/widened embankment. ■ Footings have higher risk of being affected by adjacent excavations for sub-excavation of organic materials and/or cohesive fill below the widened footprint. ■ Depending on final bridge geometry and abutment location, geotechnical resistances may have to be reduced due to proximity to adjacent slope. ■ Larger excavation anticipated to be required for construction of footings which could result in conflicts with existing abutment foundations and may require temporary protection, support and/or shoring. 	<ul style="list-style-type: none"> ■ Shallow foundations typically have lower cost than deep foundations, however, additional costs associated with temporary protection, support and/or shoring may be required. 	<ul style="list-style-type: none"> ■ Higher risk of differential settlement due to variable in embankment fill relative density. ■ Depending on final bridge geometry, location of new footings could affect global embankment stability; this would need to be evaluated at detail design stage.



PRELIMINARY FOUNDATION REPORT LITTLE ABITIBI RIVER BRIDGE REPLACEMENT SITE 39E-201 HIGHWAY 652

Foundation Type	Rank	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Drilled Steel Casings (Small Diameter)	NR	<ul style="list-style-type: none"> Higher axial resistances compared to steel H-piles. Easier to penetrate obstructions (if encountered) compared to larger diameter caissons, or driven H-Piles. Drilled steel casings have lower risk of being affected by adjacent excavations for sub-excavation of organic materials and/or cohesive fill below the widened footprint. 	<ul style="list-style-type: none"> Base of casing must be cleaned and inspected prior to completing pile installation/placing concrete. Placement of tremie concrete below the water required to complete the DSC elements. 	<ul style="list-style-type: none"> Relative costs higher than for steel H-piles 	<ul style="list-style-type: none"> Lower risk of difficulties during installation through till deposits.
Drilled Shafts/Caissons (Large Diameter)	NR	<ul style="list-style-type: none"> Higher axial resistances compared to steel H-piles and smaller diameter DSCs. 	<ul style="list-style-type: none"> Temporary liners would be required to control groundwater inflow. Potential for difficulties penetrating through obstructions compared to piles or drilled steel casings. Base of caisson must be cleaned and inspected prior to completing caisson installation/placing concrete. Placement of tremie concrete below the water required to complete the caissons. 	<ul style="list-style-type: none"> Relative costs much higher than for steel H-piles. 	<ul style="list-style-type: none"> Potential for construction problems associated with groundwater inflow during caisson installation.

NR: Not Recommended



PHOTOGRAPHS



**Photograph 1: Little Abitibi River Bridge
North side of East embankment looking East (July 2017)**



**Photograph 2: Little Abitibi River Bridge
North side of East embankment looking West (July 2017)**



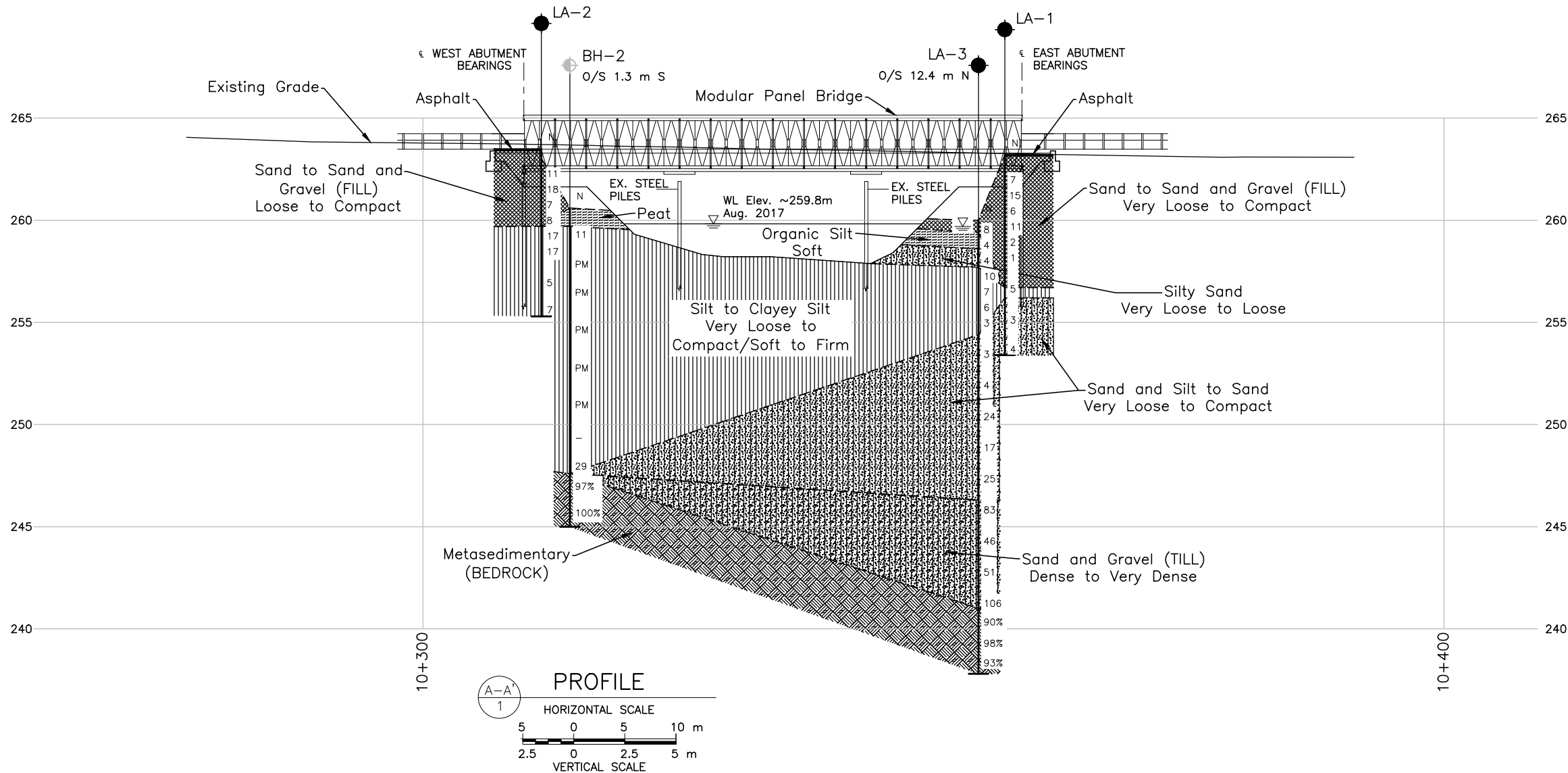
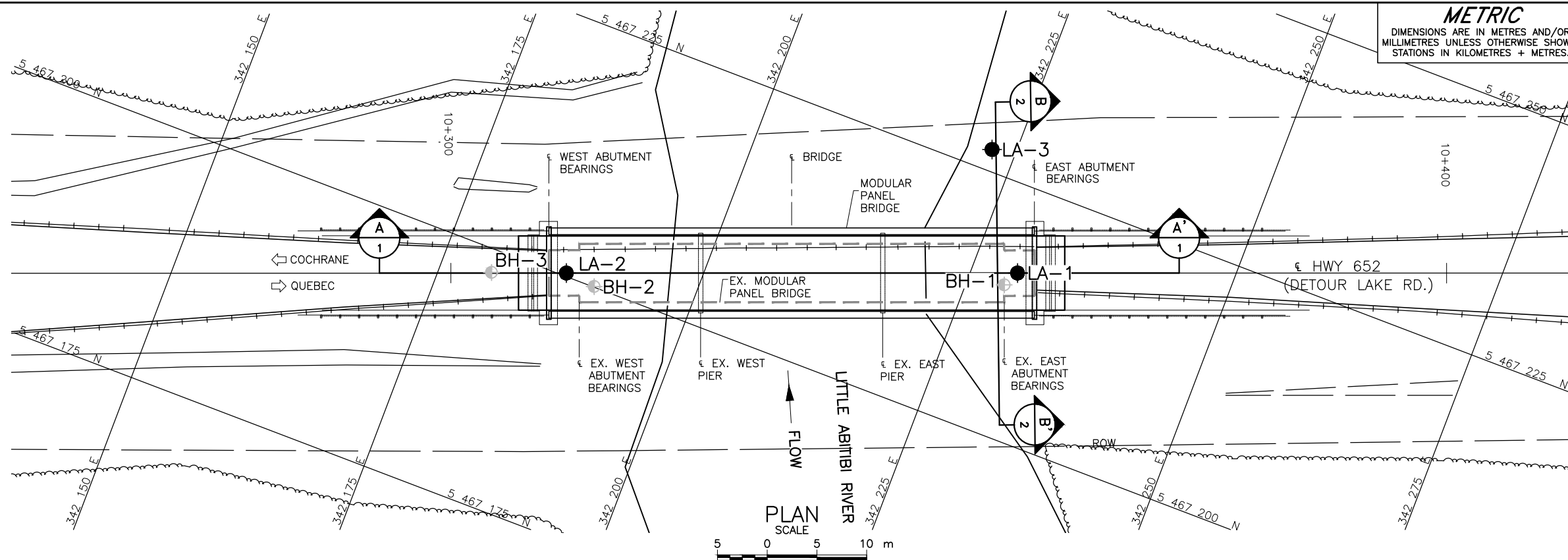
PHOTOGRAPHS



**Photograph 3: Little Abitibi River Bridge
West approach looking East (OSIM Report – November 2015)**

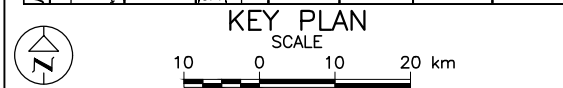
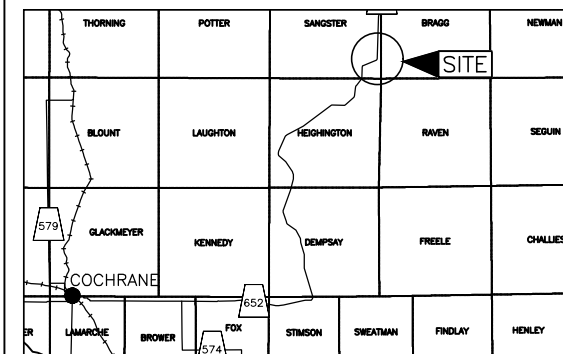


**Photograph 4: Little Abitibi River Bridge
North Elevation Looking South-West (OSIM Report – November 2015)**



CONT No. WP No. 5416-15-02

HIGHWAY 652
LITTLE ABITIBI RIVER BRIDGE
LAT. 49.341686, LONG. -80.485177
BOREHOLE LOCATIONS AND SOIL STRATA



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation (Geocres 42H-015)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- 100% Rock Quality Designation (RQD)

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
BH-2	260.6	5467200.1	342190.2
LA-1	263.2	5467216.5	342229.5
LA-2	263.5	5467200.3	342187.1
LA-3	260.0	5467227.1	342222.7

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, drawing file no. 60546679-P10.dwg, received DEC 20, 2017.

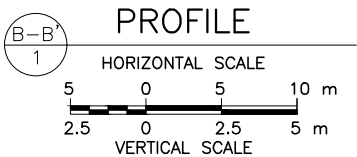
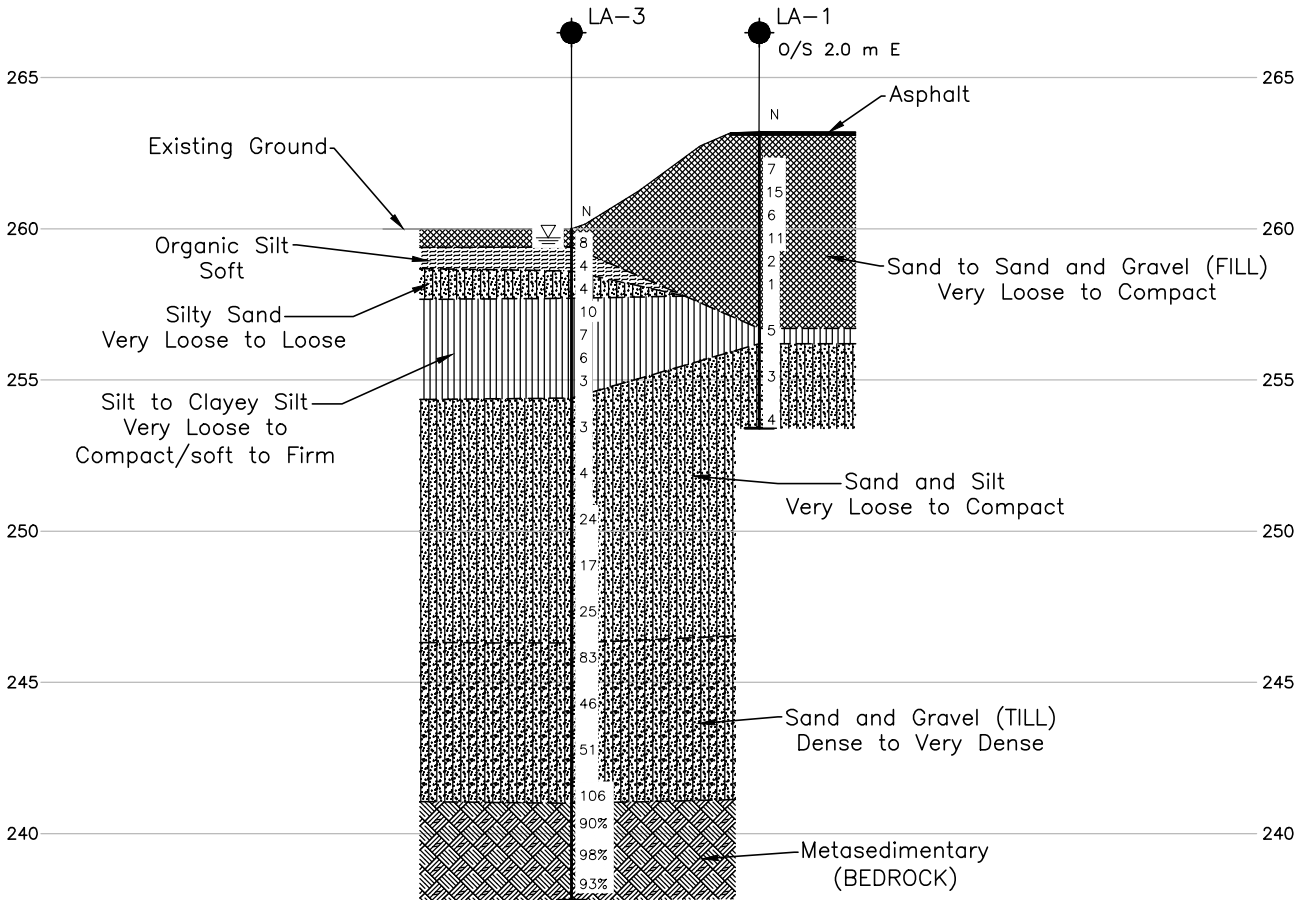


NO.	DATE	BY	REVISION
Geocres No. 42H-75			
HWY. 652	PROJECT NO. 1651997		DIST. .
SUBM'D. AD	CHKD. AC	DATE: 4/11/2018	SITE: 39E-201
DRAWN: JJJ/TB	CHKD. AB	APPD. JPD	DWG. 1

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

CONT No.
WP No. 5416-15-02

HIGHWAY 652
LITTLE ABITIBI RIVER BRIDGE
LAT. 49.341686, LONG. -80.485177
SOIL STRATA



LEGEND

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

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
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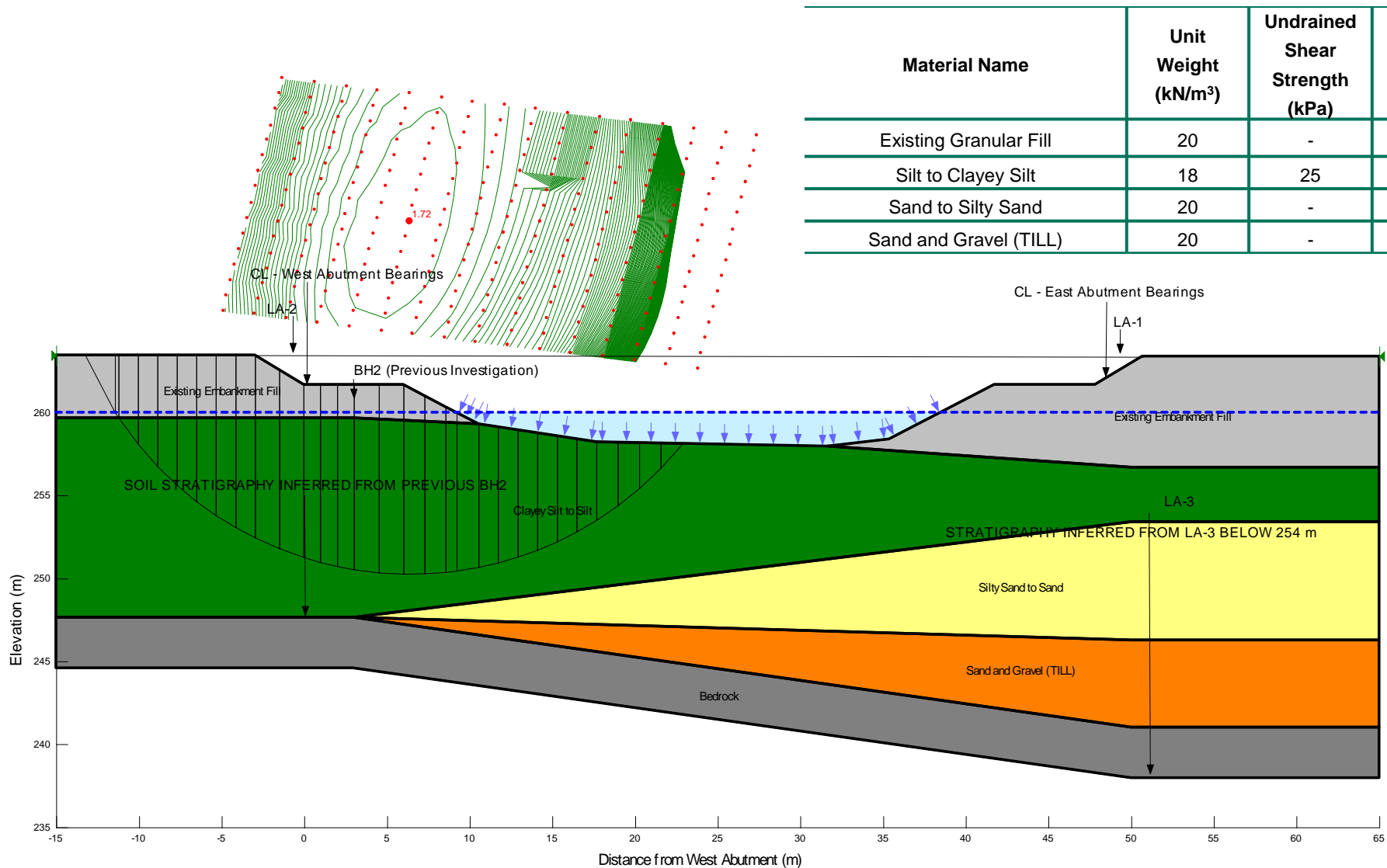


NO.	DATE	BY	REVISION
Geocres No. 42H-75			
HWY. 652	PROJECT NO. 1651997		DIST. .
SUBM'D.	CHKD. AC	DATE: 4/11/2018	SITE: 39E-201
DRAWN: TB	CHKD. AB	APPD. JPD	DWG. 2



Stability Analysis West Front Slope Short-Term (Undrained) Analysis

Figure 1



Material Name	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	32
Silt to Clayey Silt	18	25	-
Sand to Silty Sand	20	-	30
Sand and Gravel (TILL)	20	-	32

Date: January, 2018

Project No: 1651997 – Little Abitibi River Bridge

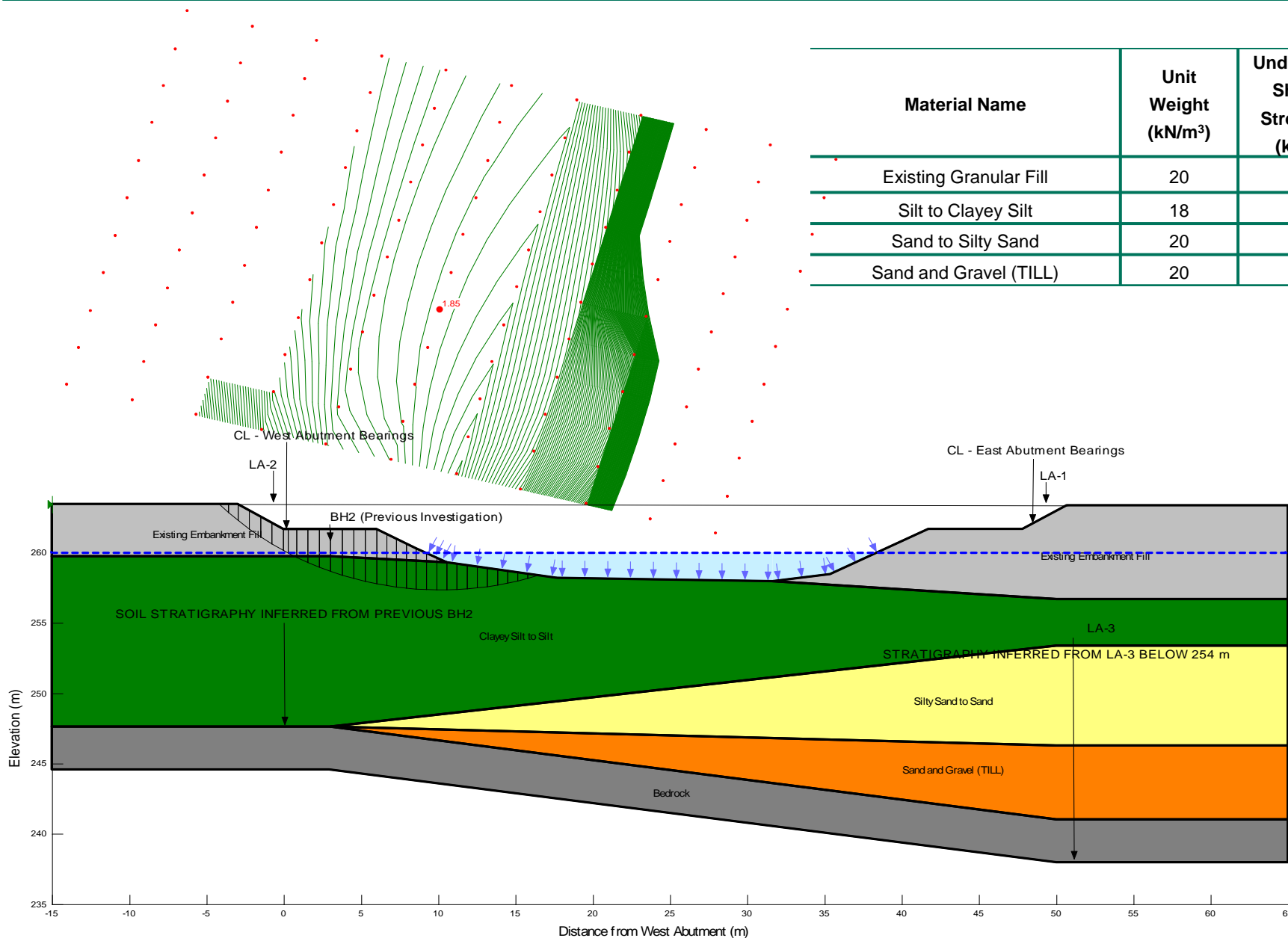
Analysis By: AC
Reviewed By: AB





Stability Analysis West Front Slope Long-Term (Drained) Analysis

Figure 2



Material Name	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	32
Silt to Clayey Silt	18	-	27
Sand to Silty Sand	20	-	30
Sand and Gravel (TILL)	20	-	32

Date: January, 2018

Project No: 1651997 – Little Abitibi River Bridge

Analysis By: AC
Reviewed By: AB

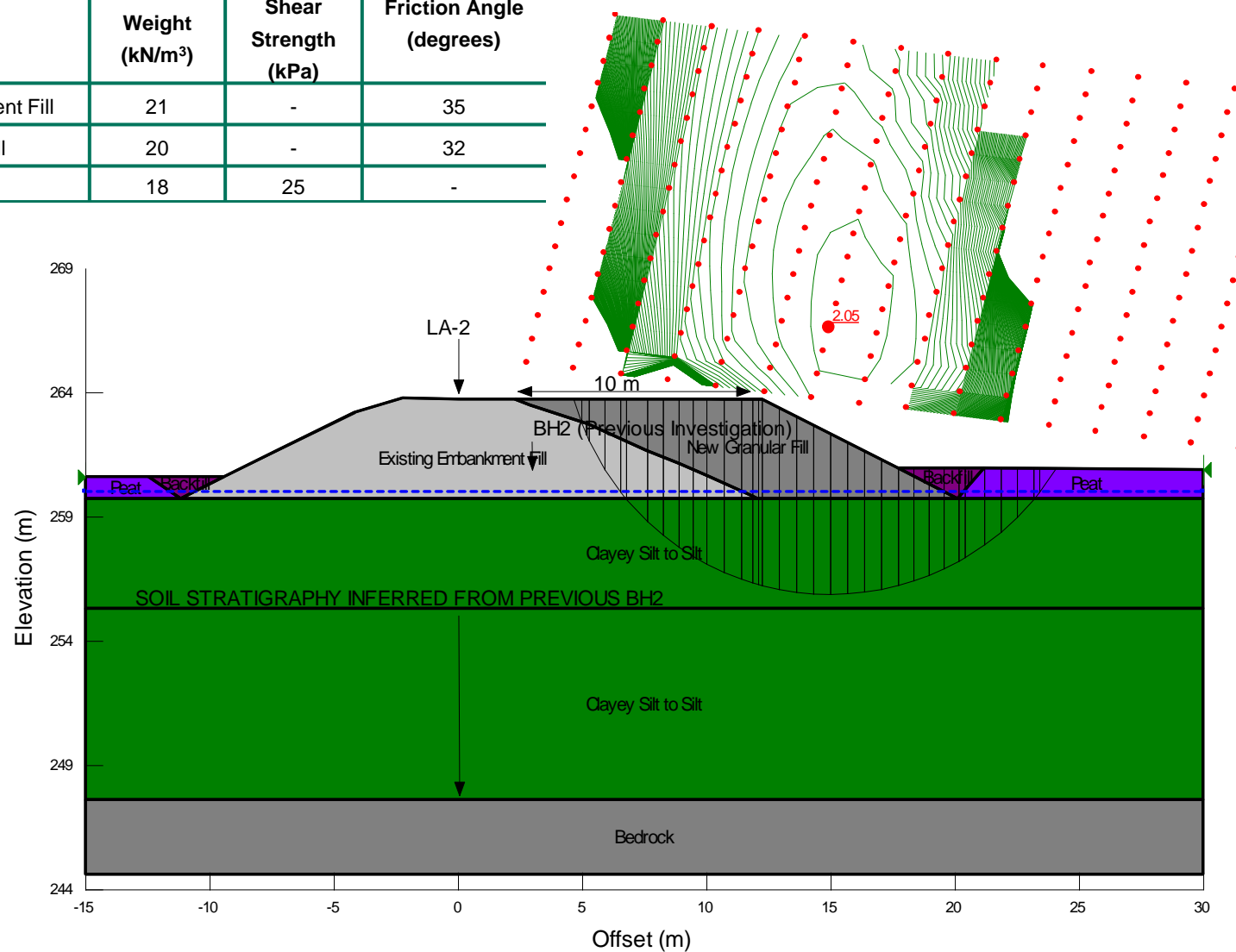




Stability Analysis West Approach, South Embankment Widening Short-Term (Undrained) Analysis

Figure 3

Material Name	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
New Granular Embankment Fill	21	-	35
Existing Granular Fill	20	-	32
Silt to Clayey Silt	18	25	-

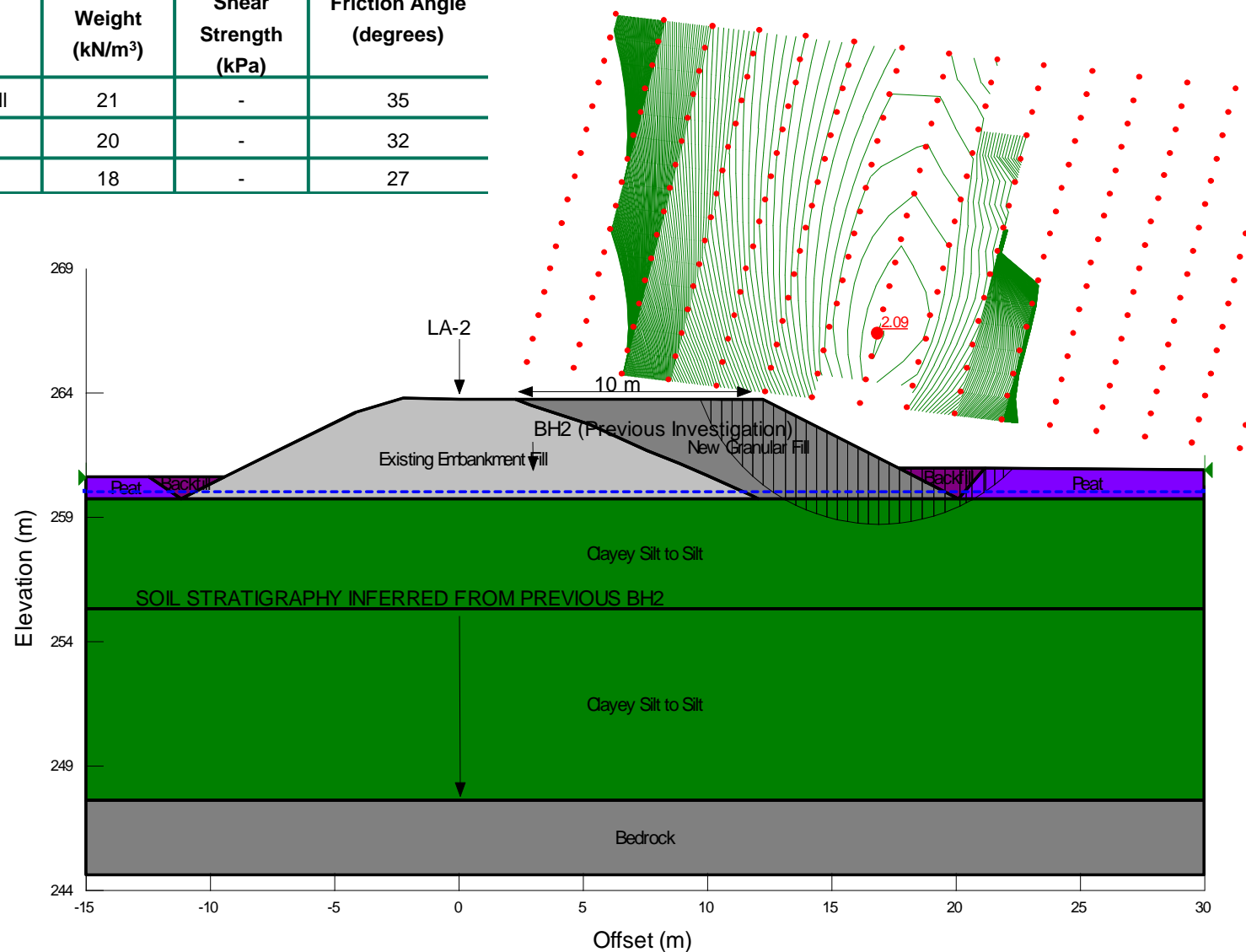




Stability Analysis West Approach, South Embankment Widening Long-Term (Drained) Analysis

Figure 4

Material Name	Unit Weight (kN/m ³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
New Granular Embankment Fill	21	-	35
Existing Granular Fill	20	-	32
Silt to Clayey Silt	18	-	27



Date: January, 2018

Project No: 1651997 – Little Abitibi River Bridge

Analysis By: AC
Reviewed By: AB





APPENDIX A

Record of Boreholes



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

(b) Cohesive Soils Consistency

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

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

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

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 90° angle is horizontal.

Description and Notes

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

Abbreviations

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

PROJECT 1651997-5000		RECORD OF BOREHOLE No LA-1				1 OF 1 METRIC								
W.P. 5416-15-02		LOCATION N 5467216.5; E 342229.5 MTM ZONE 12 (LAT. 49.3417584; LONG. -80.4848852)				ORIGINATED BY MR								
DIST _____ HWY 652		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers				COMPILED BY TB								
DATUM GEODETIC		DATE July 27, 2017				CHECKED BY AB								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
263.2	GROUND SURFACE													
0.0	ASPHALT (80 mm)													
	Sand and gravel (FILL)													
0.3	Sand, trace to some gravel, trace to some fines (FILL) Very loose to compact Brown Moist to wet		1	SS	7									
			2	SS	15									
			3	SS	6									
			4	SS	11									
			5	SS	2									
			6	SS	1									
			7A	SS	5									
256.7	CLAYEY SILT, trace to some sand Grey Wet		7B											
256.2	SAND, trace to some silt, trace clay Very loose to loose Grey Wet		8	SS	3									
253.4	END OF BOREHOLE		9	SS	4									

PROJECT 1651997-5000		RECORD OF BOREHOLE No LA-2				1 OF 1 METRIC								
W.P. 5416-15-02		LOCATION N 5467200.3; E 342187.1 MTM ZONE 12 (LAT. 49.3416153; LONG. -80.4854702)				ORIGINATED BY MR								
DIST _____ HWY 652		BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers				COMPILED BY TB								
DATUM GEODETIC		DATE July 27, 2017				CHECKED BY AB								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
263.5	GROUND SURFACE													
0.0	ASPHALT (80 mm)													
0.1	Sand and gravel (FILL)													
262.9														
0.6	Sand, trace to some gravel, trace to some fines (FILL) Loose to compact Brown to grey Moist to wet		1	SS	11									
			2	SS	18									
			3	SS	7									
			4	SS	8									
259.7														
3.8	SILT, trace to some clay Loose to compact Grey Wet		5	SS	17								NP	0 0 90 10
			6	SS	17									
			7	SS	5								NP	0 2 87 11
	Clayey silt layers/seams below 7.1 m depth													
255.3			8	SS	7									
8.2	END OF BOREHOLE													

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1651997 AECOM_5015-E-0045_NE RETAINER02_DATA\GINTV1651997.GPJ GAL-MISS.GDT 1/24/18 TB

PROJECT 1651997-5000			RECORD OF BOREHOLE No LA-3			1 OF 2 METRIC														
W.P. 5416-15-02			LOCATION N 5467227.1; E 342222.7 MTM ZONE 12 (LAT. 49.3418541; LONG. -80.4849777)			ORIGINATED BY MR														
DIST _____ HWY 652			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring and NQ Coring			COMPILED BY TB														
DATUM GEODETIC			DATE August 1, 2017			CHECKED BY AB														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ kN/m³	GR SA SI CL			
							20 40 60 80 100	20 40 60 80 100	20 40 60	20 40 60	20 40 60									
260.0	GROUND SURFACE																			
0.0	Sand, trace gravel (FILL) Loose Brown Moist		1	SS	8	▽														
259.4																				
0.6	ORGANIC SILT Soft Dark brown Wet		2	SS	4		259													
258.6																				
1.4	SILTY SAND, trace to some organics Very loose to loose Brown Wet		3	SS	4		258													
257.7																				
2.3	SILT, trace to some clay, trace to some sand Very loose to compact Grey Wet		4	SS	10		257													
			5	SS	7															
			6	SS	6		256													
			7	SS	3		255													
254.4																				
5.6	SAND and SILT to SAND, trace to some gravel, trace clay Very loose to compact Grey Wet		8	SS	3		254													
							253													
			9	SS	4		252													
							251													
			10	SS	24		250													
			11	SS	17		249													

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1651997 AECOM_5015-E-0045_NE RETAINER02_DATA\GINTV1651997.GPJ GAL-MISS.GDT 1/24/18 TB

PROJECT 1651997-5000			RECORD OF BOREHOLE No LA-3				2 OF 2 METRIC													
W.P. 5416-15-02			LOCATION N 5467227.1; E 342222.7 MTM ZONE 12 (LAT. 49.3418541; LONG. -80.4849777)				ORIGINATED BY MR													
DIST _____ HWY 652			BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring and NQ Coring				COMPILED BY TB													
DATUM GEODETIC			DATE August 1, 2017				CHECKED BY AB													
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20	40	60	80	100						20	40	60
	--- CONTINUED FROM PREVIOUS PAGE ---																			
246.3	SAND and SILT to SAND, trace to some gravel, trace clay Very loose to compact Grey Wet		12	SS	25															
13.7	SAND and GRAVEL, trace to some silt (TILL) Dense to very dense Grey Wet		13	SS	83															
			14	SS	46															
			15	SS	51															
			16	SS	106															
241.0	METASEDIMENTARY (BEDROCK)																			
19.0	Bedrock cored from 19.0 m depth to 22.2 m depth. For coring details see Record of Drillhole LA-3.		1	RC	REC 100%												RQD = 90%			
			2	RC	REC 100%												RQD = 98%			
			3	RC	REC 100%												RQD = 93%			
237.8	END OF BOREHOLE																			
22.2	Note: 1. Water level at a depth of 0.3 m below ground surface (Elev. 259.7 m) upon completion of drilling.																			

PROJECT: 1651997-5000
 LOCATION: N 5467227.1; E 342222.7
 MTM ZONE 12 (LAT. 49.3418541; LONG. -80.4849777)
 INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: LA-3

SHEET 1 OF 1
 DRILLING DATE: August 1, 2017
 DATUM: GEODETIC

DRILL RIG: CME 55LC
 DRILLING CONTRACTOR: Downing Drilling

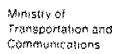
DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	FLUSH	RECOVERY			R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA						HYDRAULIC CONDUCTIVITY				Diametral Point Load Index (MPa)	RMC -Q AVG.																					
								TOTAL CORE %	SOLID CORE %	TYPE AND SURFACE DESCRIPTION			Jr	Ja	Jn	10°	10°	10°	10°																										
																				Jr	Ja	Jn			10°	10°	10°	10°																	
																													Jr	Ja	Jn	10°	10°	10°	10°										
JN - Joint	FLT - Fault	SHR - Shear	VN - Vein	CJ - Conjugate	BD - Bedding	FO - Foliation	CO - Contact	OR - Orthogonal	CL - Cleavage	PL - Planar	CJ - 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.																					
19	CME 55 NQ Coring	REFER TO PREVIOUS PAGE		241.0	1	Grey 100																																							
20		2		Grey 100																																									
21		3		Grey 100																																									
22		END OF DRILLHOLE	237.8	22.2																																									
23																																													
24																																													
25																																													
26																																													
27																																													
28																																													
29																																													
30																																													
31																																													

DEPTH SCALE
 1 : 60



LOGGED: MR
 CHECKED: AB

SUD-RCK MTM ZN INC LAT/LONG S:\CLIENTS\MTM\1651997 AECOM_5015-E-0045 NE RETAINER02_DATA\GINTV1651997.GPJ GAL-MISS GDT 1/24/18 TB



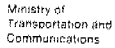
RECORD OF BOREHOLE No 1

W P 7-81-10 LOCATION Sta. 13+356.3 0.8 RT. @ Detour Lake Rd. Line 'A' ORIGINATED BY RM
DIST 16 HWY Detour Lk. Rd. BOREHOLE TYPE EX Casing & Wash COMPILED BY RM
DATUM Geodetic DATE 80 10 31 CHECKED BY _____

[illegible]

+3, x5: Numbers refer to Sensitivity

OFFICE REPORT ON SOIL EXPLORATION



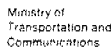
RECORD OF BOREHOLE No 2

W P 7-81-10 LOCATION Sta. 13+314.0 2.0 RT @ Detour Lake Rd. Line 'A' ORIGINATED BY RM
DIST 16 HWY Detour Lk. Rd. BOREHOLE TYPE BX Casing & Wash COMPILED BY RM
DATUM Geodetic DATE 80 11 18 CHECKED BY

[illegible]

+3, x⁵: Numbers refer to Sensitivity

20
15
10



RECORD OF BOREHOLE No 3

W P 7-81-10 LOCATION Sta. 13+306.8 @ Detour Lake Rd. Line 'A' ORIGINATED BY RM
DIST 16 HWY Detour Lk. Rd. BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY RM
DATUM Geodetic DATE 80 11 20 CHECKED BY _____

[illegible]

+3, x5: Numbers refer to Sensitivity

OFFICE REPORT ON SOIL EXPLORATION



APPENDIX B

Laboratory Testing



**PRELIMINARY FOUNDATION REPORT
LITTLE ABITIBI RIVER BRIDGE REPLACEMENT
SITE 39E-201 HIGHWAY 652**

Table B1: Summary of Analytical Testing of Little Abitibi River Soil Samples

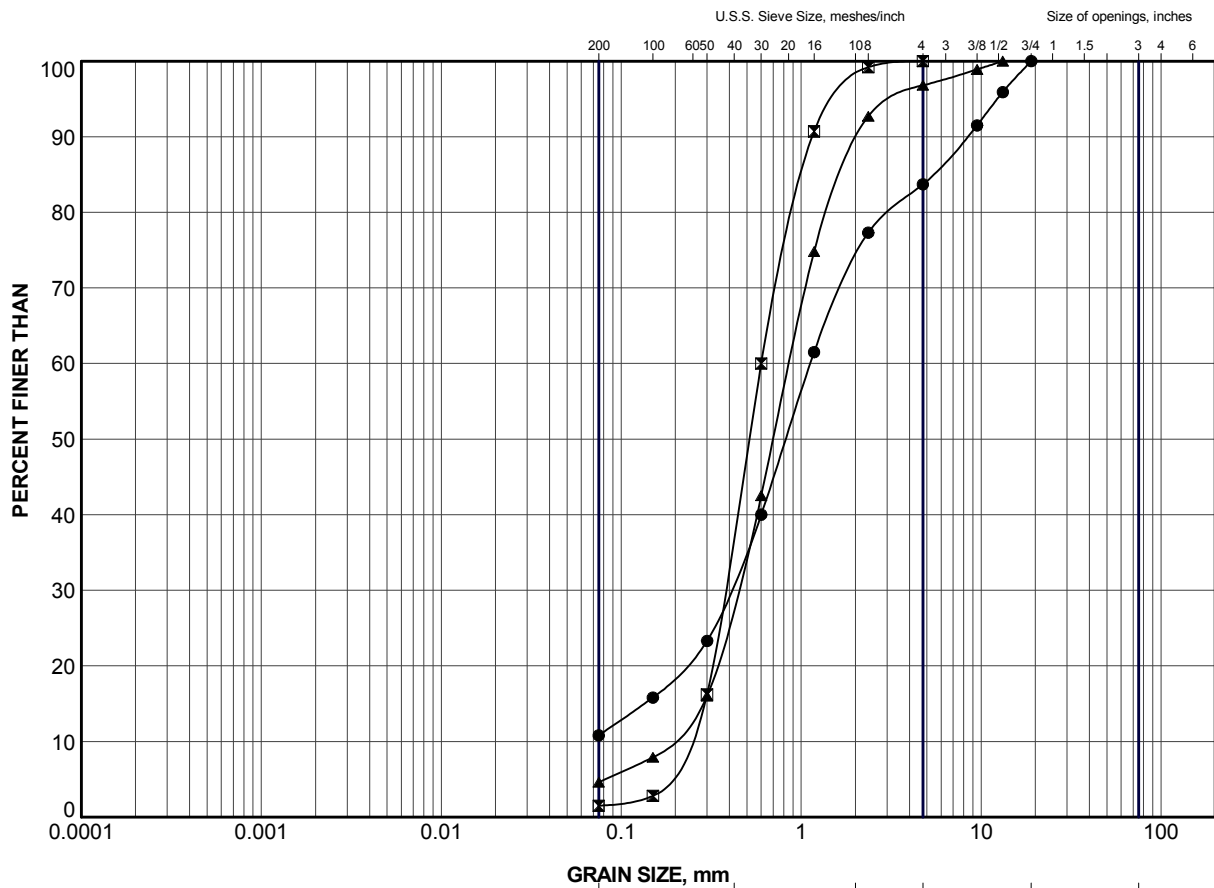
Location	Parameter	Units	Result
East Abutment (LA-1, Sa #5)	Chloride (CL)	ug/g	57
	Sulphate (SO4)	ug/g	ND
	Conductivity (EC)	umho/cm	159
	Resistivity	ohm-cm	6,300
	pH	n/a	7.82
West Abutment (LA-2, Sa #6)	Chloride (CL)	ug/g	32
	Sulphate (SO4)	ug/g	190
	Conductivity (EC)	umho/cm	331
	Resistivity	ohm-cm	3,000
	pH	n/a	7.84

Notes: 1. Samples from Boreholes LA-1 and LA-2 obtained on July 27, 2017, and submitted to Maxxam on November 22, 2017, which is beyond the standard hold time.

2. Analytical testing carried out by Maxxam.

Prepared by: AC


Checked by: AB

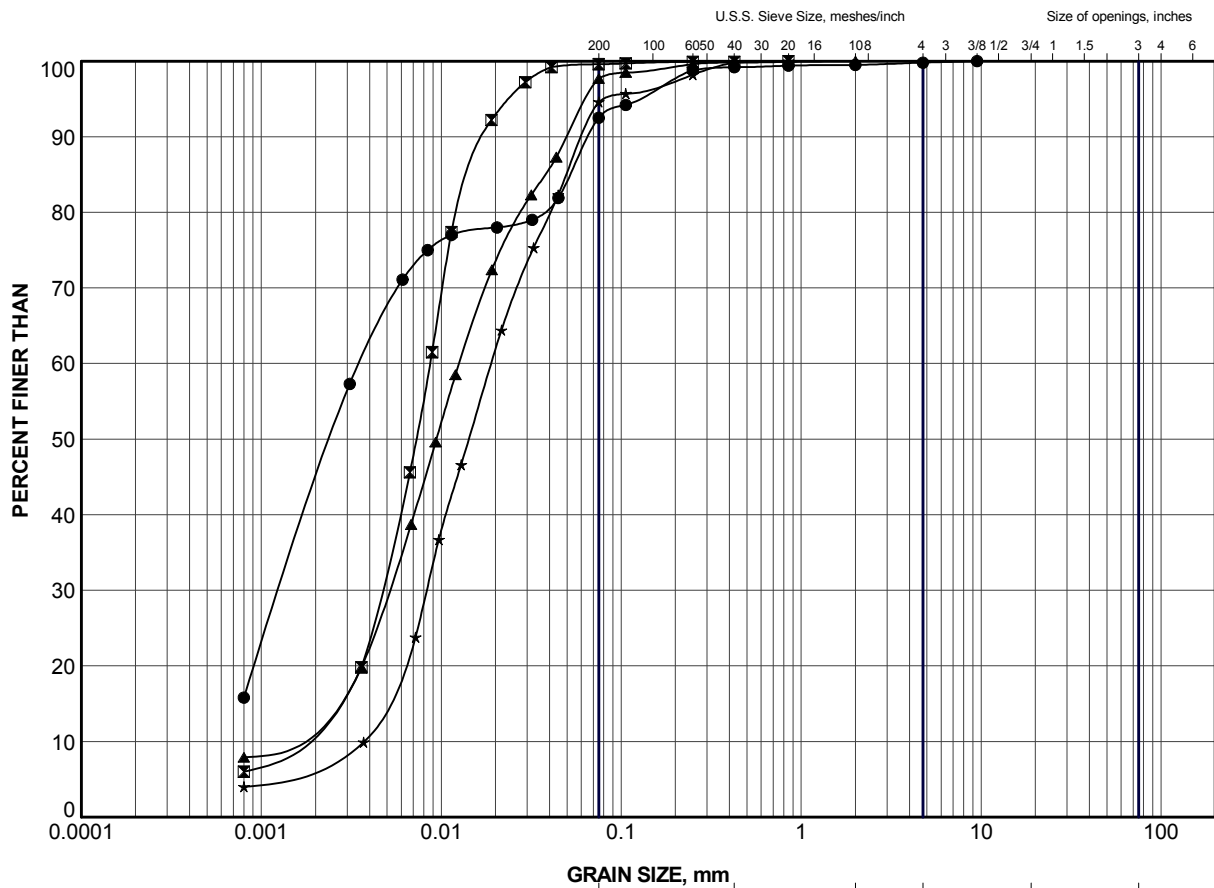


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	LA-1	2	261.4
⊠	LA-1	6	258.3
▲	LA-2	3	260.9


PROJECT					
HIGHWAY 652 LITTLE ABITIBI RIVER					
TITLE					
GRAIN SIZE DISTRIBUTION SAND (FILL)					
PROJECT No.				FILE No. 1651997.GPJ	
DRAWN	TB	Jan 2018	SCALE	N/A	REV.
CHECK	AB	Jan 2018			
APPR	JPD	Jan 2018			
 Golder Associates SUDBURY, ONTARIO			FIGURE B1		

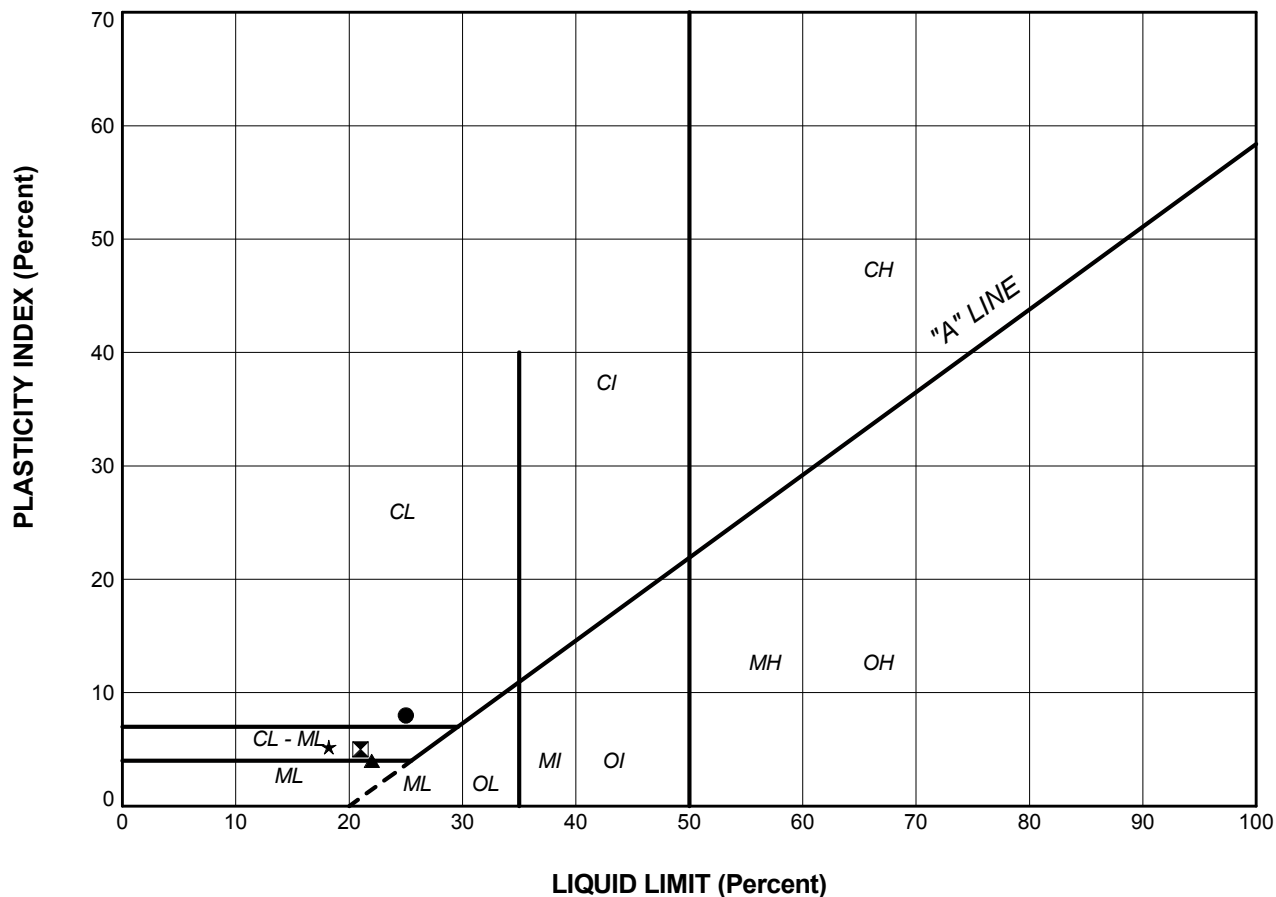


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	LA-1	7B	256.6
⊠	LA-2	5	259.4
▲	LA-2	7	257.1
★	LA-3	4	257.4

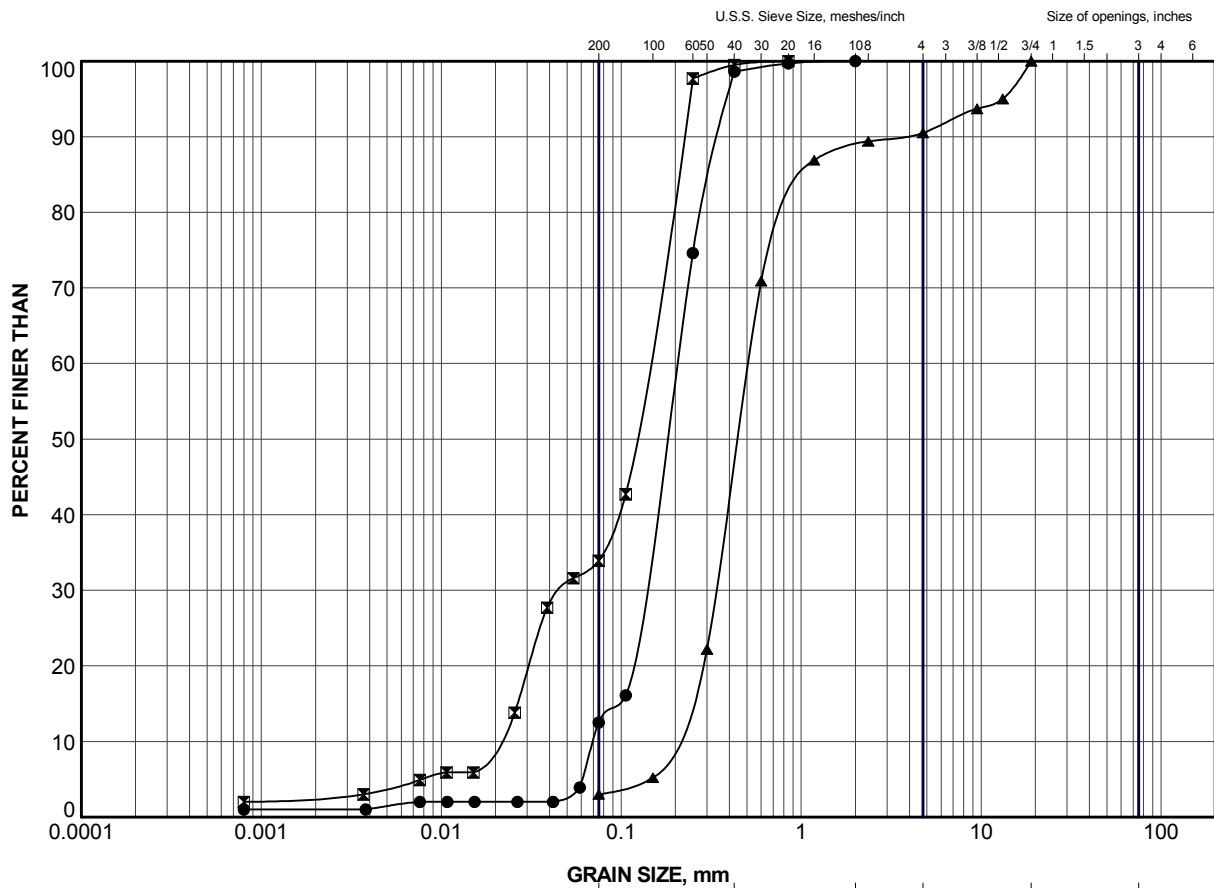
PROJECT						HIGHWAY 652 LITTLE ABITIBI RIVER BRIDGE					
TITLE						GRAIN SIZE DISTRIBUTION SILT to CLAYEY SILT					
PROJECT No.			1651997			FILE No.			1651997.GPJ		
DRAWN	TB	Apr 2018	SCALE	N/A	REV.						
CHECK	AB	Apr 2018									
APPR	JPD	Apr 2018									
 GOLDER SUDBURY, ONTARIO						FIGURE B2					



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	2	1	25.0	17.0	8.0
⊠	2	2	21.0	16.0	5.0
▲	2	8	22.0	18.0	4.0
★	LA-1	7B	18.2	13.0	5.2

PROJECT					
HIGHWAY 652 LITTLE ABITIBI RIVER BRIDGE					
TITLE					
PLASTICITY CHART CLAYEY SILT to SILT of Slight Plasticity					
PROJECT No. 1651997			FILE No. 1651997.GPJ		
DRAWN	TB	Apr 2018	SCALE	N/A	REV.
CHECK	AB	Apr 2018			
APPR	JPD	Apr 2018			
GOLDER			FIGURE B3		
SUDBURY, ONTARIO					

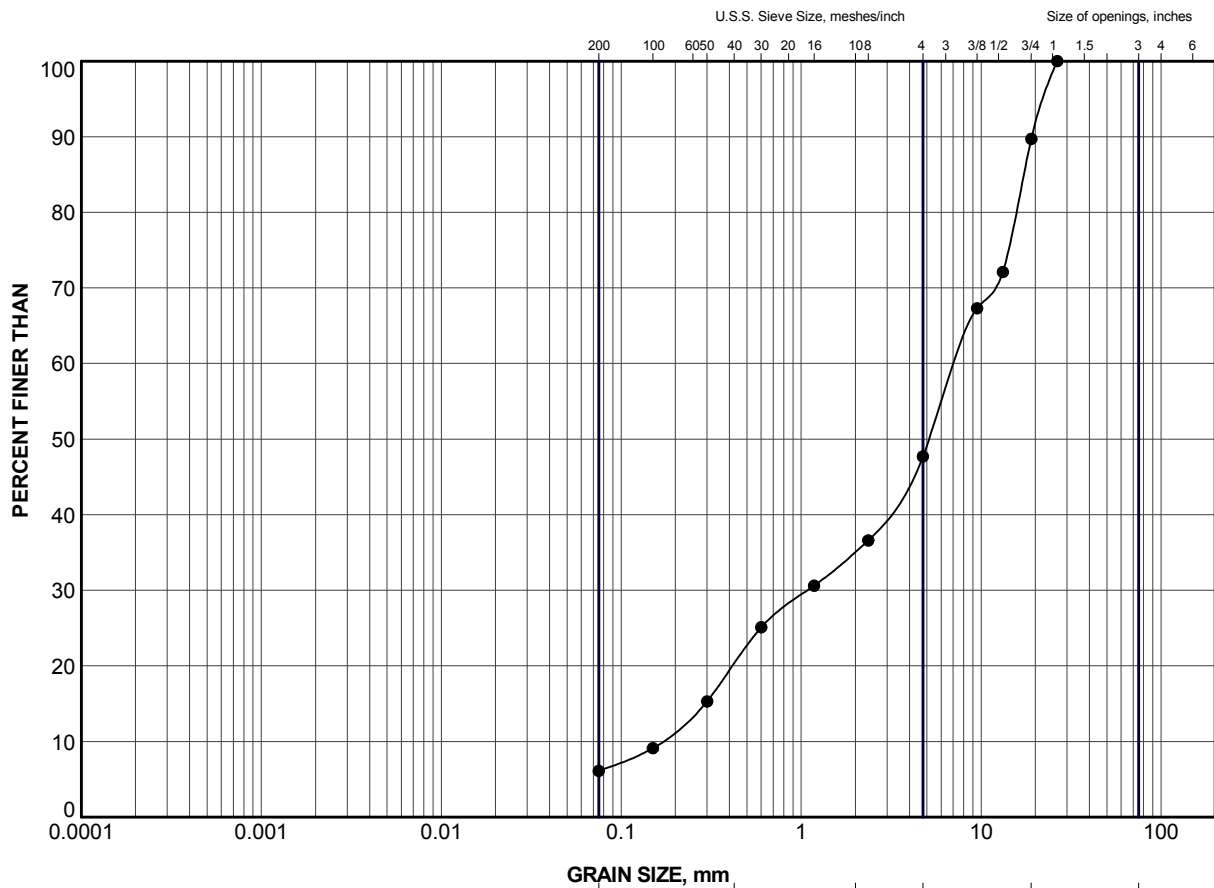


GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	LA-1	8	255.3
⊠	LA-3	8	253.6
▲	LA-3	11	249.0

PROJECT					
HIGHWAY 652 LITTLE ABITIBI RIVER BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION SAND and SILT to SAND					
PROJECT No.		1651997		FILE No. 1651997.GPJ	
DRAWN	TB	Apr 2018	SCALE	N/A	REV.
CHECK	AB	Apr 2018			
APPR	JPD	Apr 2018			
GOLDER SUDBURY, ONTARIO			FIGURE B4		



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

LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	LA-3	14	244.5

PROJECT					
HIGHWAY 652 LITTLE ABITIBI RIVER BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION SAND and GRAVEL (TILL)					
PROJECT No.		1651997		FILE No. 1651997.GPJ	
DRAWN	TB	Apr 2018	SCALE	N/A	REV.
CHECK	AB	Apr 2018	FIGURE B5		
APPR	JPD	Apr 2018			
SUDBURY, ONTARIO					

Borehole LA-3



Box 1: 19.0 m – 22.2 m


PROJECT		Highway 652 Little Abitibi Bridge			
TITLE		Bedrock Core Photographs			
		PROJECT No. 1651997		FILE No. ----	
		DESIGN	AD	DEC 2017	SCALE NTS
		CADD	--	---	REV.
		CHECK	AB	JAN 2018	FIGURE B6
		REVIEW			

Golder Associates Ltd.

33 Mackenzie Street
Sudbury, Ontario, Canada P3C 4Y1
Telephone: (705) 524-8861
Fax: (705) 524-1984


**SUMMARY OF ROCK CORE TEST DATA**

PROJECT NO.: **1651997 5300**
PROJECT NAME: **AECOM/5015-E-0045/NE Retainer**
TYPE OF UNIT: **Rock Core**
TESTED BY: **JP**
DATE TESTED: **August 31, 2017**

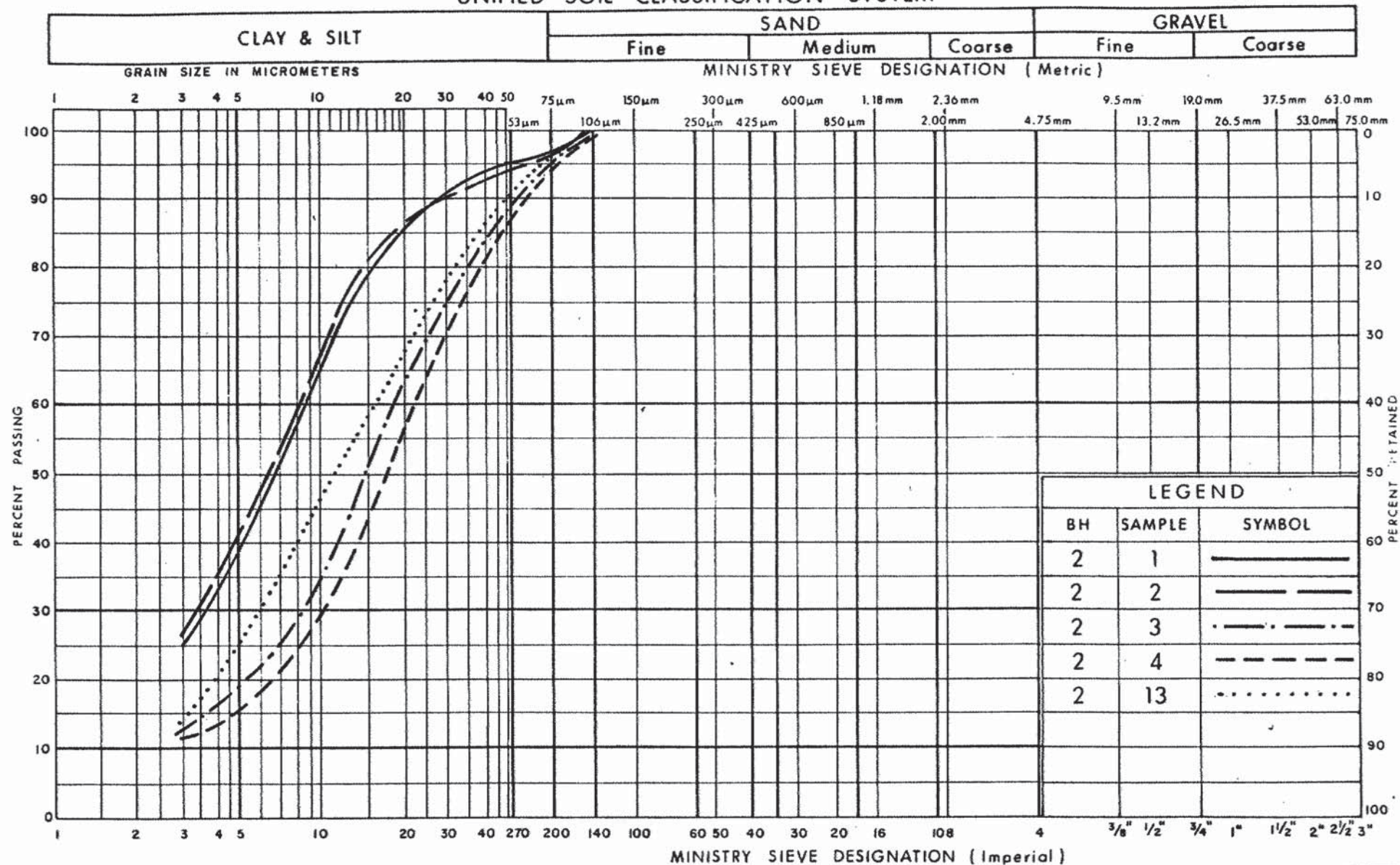
GOLDER LAB NUMBER	C1380				
BOREHOLE NUMBER:	LA-3				
SAMPLE NUMBER:	N/A				
DEPTH OF TESTED CORE (ft)	64.6				
LENGTH AS CUT (mm)	101.3				
DIAMETER (mm)	47.4				
DENSITY (kg/m3)	2745				
COMPRESSIVE STRENGTH (KN)	79.4				
CORRECTED STRENGTH (MPa)	45.0				
TYPE OF FRACTURE	3				
Type of Fracture  1 2 3 4 5 6					

COMMENTS:

Input by: SM
Reviewed by: [Signature]

PROJECT		Highway 652 Little Abitibi Bridge			
TITLE		Summary of Rock Core Test Data			
	PROJECT No.	1651997		FILE No.	----
	DESIGN	AD	DEC 2017	SCALE	NTS
	CADD	--			REV.
	CHECK	AC	APR 2018	FIGURE B7	
REVIEW					

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SILT TO SILTY CLAY
TRACE OF SAND

FIG No 1

WP 7-81-10

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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