April 06, 2018

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

CHIN RIVER BRIDGE REPLACEMENT - SITE NO. 39E-197
LAT. 49.251387; LONG. -80.646589
HIGHWAY 652, COCHRANE DISTRICT
TOWNSHIP OF HEIGHINGTON
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5416-15-00; WP 5416-15-01

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GEOCRES NO.: 42H-74

Report Number: 1651997-WO5-001

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REPORT

Golder Associates

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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 Chin River Bridge (Site No. 39E-197). The Chin River Bridge is located in the Cochrane District in the Township of Heighington, Ontario at about Sta. 10+340 (approximately 24 km north of Translimit Road). The general location of this section of Highway 652 is shown on the Key Plan on Drawing 1.

2.0 SITE DESCRIPTION AND BACKGROUND INFORMATION

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

In general, the topography in the area of the Chin River Bridge consists of undulating to rolling terrain with densely forested areas immediately beyond the Highway 652 right-of-way and in the vicinity of the river. The existing Chin River Bridge consists of an approximately 48.8 m long by 4.6 m wide, three-span, single-lane Temporary Modular Bridge (TMB). Based on the previous General Arrangement (GA) drawing (GEOCRES 42H-23), the existing bridge is supported by driven steel friction piles (HP310x79) at both the abutments and piers. Based on the survey drawing provided by AECOM, the bridge deck is at Elevation 284.3 m at both the north and south abutments. The existing front slopes are about 6.5 m to 7.5 m high and the approach embankments side slopes are about 3 m to 4 m high. The existing embankment front slopes and side slopes are inclined at a profile of about 2 Horizontal to 1 Vertical (2H:1V). The ground surface conditions at the bridge abutments are shown on Photographs 1 to 3. Based on the 2016 Ontario Structure Inspection Manual (OSIM) report, our July 2017 site review, and the available site photographs, the existing embankments appear to be performing satisfactorily. However, as noted in the OSIM report, there is some slight erosion of the exposed granular front slope at the north abutment.

3.0 INVESTIGATION PROCEDURE

The field work for this subsurface investigation was carried out on July 19, 20 and 28, 2017, during which time a total of four boreholes (CR-1 to CR-4) were advanced at the approximate locations shown on Drawing 1. Boreholes CR-1 and CR-3 were advanced through the existing highway embankment at the existing north and south abutments, respectively. Boreholes CR-2 and CR-4 were advanced at the east toe of the north and south embankment slopes, respectively. Boreholes CR-1 to CR-4 were advanced using a track-mounted CME 55LC drill rig equipped with 108 mm inside diameter hollow-stem augers. The drill rig was supplied and operated by George Downing Estate Drilling Ltd. of Grenville-sur-la-Rouge Quebec.

Soil samples were obtained at depth intervals of 0.75 m and 1.5 m, using 50 mm outer diameter split-spoon samplers driven by an automatic hammer, carried out in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). The groundwater level in the open boreholes was observed during the drilling operations as described on the Record of Borehole sheets in Appendix A. The boreholes advanced at the existing bridge abutments were backfilled with a full column of bentonite grout. The boreholes advanced at the toe of the embankment slope were backfilled with bentonite pellets and soil cuttings upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The field work was supervised on a full-time basis by members of Golder's technical staff who: located the boreholes in the field; arranged for the clearance of underground services; supervised the drilling and sampling operations; logged the boreholes; and examined and cared for the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder's geotechnical laboratory in Sudbury for further examination and laboratory testing. Index and classification testing consisting of water content determinations, grain size distributions and Atterberg limits were carried out on selected soil samples. The geotechnical laboratory testing was performed in accordance with MTO LS standards.

Soil samples were obtained on July 28, 2017, from Boreholes CR-1 and CR-3 at the north and south abutments, respectively, 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. It should be noted that the samples were submitted beyond the standard hold times and as such some parameters may not be reliable.

The as-drilled borehole locations were measured and surveyed by members of our technical staff. The borehole locations were referenced to the highway centerline and existing bridge and converted to northing/easting coordinates on the plan drawing. The ground surface elevations were referenced to local benchmarks in the vicinity of the bridge and the benchmark elevations were obtained from the General Arrangement (GA) drawing provided by AECOM (drawing 60547656-P1.dwg). The MTM NAD83 Zone 12 northing and easting coordinates and geographical coordinates, ground 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 Number	MTM NAD83 Northing (Latitude)	MTM NAD83 Easting (Longitude)	Ground Surface Elevation	Borehole Depth
CR-1	5457122.5 m (49.2515939)	330572.2 m (-80.6459628)	284.3 m	9.8 m
CR-2	5457115.4 m (49.2515295)	330585.4 m (-80.645782)	280.0 m	20.4 m
CR-3	5457073.3 m (49.2511523)	330552.5 m (-80.6462366)	284.3 m	9.8 m
CR-4	5457069.0 m (49.2511131)	330566.5 m (-80.6460446)	280.2 m	21.9 m

4.0 SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain (NOEGTS)[1] Mapping, the Chin River Bridge site is located within a glaciolacustrine plain deposit consisting primarily of clay and silt soils bordered by organic terrain deposits of peat/muck and ground moraine deposits of clayey till.

[1] Digital Northern Ontario Engineering Geology Terrain Study (NOEGTS). Ontario Geological Survey, Miscellaneous Release – Date 160, Map 42HSE.

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 bedrock.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of 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) as presented on the Record of Borehole sheets and described in Section 4 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile on Drawing 1 and in the sections 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 (GEOCRES 42H-43), prior to construction of the existing embankments and bridge, the subsurface soil conditions at the site generally consisted of peat overlying thin deposits of firm to stiff silty clay underlain by an extensive deposit of compact to very dense silty sand. The subsoil conditions encountered during the current borehole investigation consist of granular embankment fill and clayey silt to silty clay fill overlying deposits of firm clayey silt and loose to very dense sand and silt to silty sand (till) containing cobbles and boulders, which is generally consistent with the previous findings. A more detailed description of the soil deposits and groundwater conditions encountered in the boreholes as part of the current investigation is provided below.

Deposit/Layer Description	Boreholes	Deposit Thickness (m)	Deposit Surface Elevation (m)	SPT N Values (blows/0.3 m) Relative Density	Laboratory Testing
Asphalt	CR-1 & CR-3	0.050	284.3	n/a	n/a
(FILL) Gravelly Sand to Sand, trace to some silt, trace clay, brown; moist to wet	CR-1 to CR-4	3.5 – 8.6	284.2 – 278.6	N = 0 (WH) to 25 **Very Loose to Compact**	w = 4% – 6% 5 – M/MH (Fig. B1)
(FILL) Clayey Silt to Silty Clay some sand, some organics, trace wood, dark brown; wet	CR-2 & CR-4	1.4 & 1.5	280.0	N = 9 **Stiff**	W = 29% 1 – AL (Fig. B2) w_l = 36% w_p = 16% I_p = 20%
Clayey Silt, trace gravel, trace sand; brown; wet	CR-1 & CR-2	>1.6 and 0.4 (not fully penetrated in Borehole CR-1)	276.1 & 275.1	N = 5 **Firm**	w = 20% & 28% 2 – AL (Fig. B3) w_l = 24% & 28% w_p = 14% I_p = 9% & 13%

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

Deposit/Layer Description	Boreholes	Deposit Thickness (m)	Deposit Surface Elevation (m)	SPT N Values (blows/0.3 m) / Relative Density	Laboratory Testing
Sand and Silt to Silty Sand (TILL), trace to some gravel, trace to some clay; grey; wet (presence of cobbles and boulders within TILL inferred from auger grinding)	CR-2 to CR-4	>1.3 – >16.6 (Boreholes terminated in this deposit)	275.6 – 274.7	N = 6 to 129 **Loose to Very Dense**	w = 8% – 10% 7 – MH (Fig. C4) 7 – AL (NP)

Where:
N = SPT 'N'-value; number of blows for 0.3 m of penetration (uncorrected)
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

Clayey Silt to Silty Clay Fill

The surficial clayey silt fill in Borehole CR-4 was placed by Golder during the field investigation to provide a level drilling platform and as such has not been included as part of the subsoil conditions summary provided above. A 0.2 m thick layer of silty clay fill was encountered in Borehole CR-4 at 276.4 m within the existing sand fill layer.

Topsoil

A 0.1 m thick layer of silty topsoil was encountered at ground surface in Borehole CR-4 prior to constructing the clayey silt fill drilling platform.

4.3 Groundwater Conditions

The unstabilized groundwater levels measured in the open boreholes upon completion of drilling are summarized below. The river water level was measured by others at Elevation 276.8 m in August 2017. Groundwater and river water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

Borehole	Ground Surface Elevation (m)	Depth to Groundwater (mbgs)	Groundwater Elevation (m)
CR-1	284.3	Dry (samples wet below 3.8 m)	Possible perched water at about 280.5
CR-2	280.0	2.0	278.0
CR-3	284.3	Dry (samples wet below 7.6 m)	Possible perched water at about 276.7
CR-4	280.2	2.2	278.0

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Adam Core, P.Eng., and the technical aspects were reviewed by Mr. David Muldowney, P.Eng. Mr. Paul Dittrich, Ph.D., P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.

Report Signature Page

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
CHIN RIVER BRIDGE - SITE NO. 39E-197
HIGHWAY 652, COCHRANE DISTRICT
TOWNSHIP OF HEIGHINGTON
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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 Chin River Bridge (Site 39E-197) 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 subsurface 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 structure foundations. 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 Chin River Bridge, which was constructed in 1984, consists of a single-lane, three-span TMB structure approximately 48 m long by 4.6 m wide. Based on the available GEOCRES information, we understand that the existing structure is supported by friction piles (driven steel HP 310x79 piles) at both the abutments and piers. The existing front slopes are about 6.5 m to 7.5 m high and the approach embankment side slopes are about 3 m to 4 m high. The existing embankment front slopes and side slopes are inclined at a profile of about 2H:1V. We further understand that prior to the original embankment construction, the surficial deposits of peat and silty clay were generally sub-excavated and replaced with granular fill.

Based on the General Arrangement (GA) drawing provided by AECOM, we understand that the proposed replacement structure is to consist of a two lane, single-span TMB bridge constructed on the same alignment as the existing bridge. The replacement bridge will be approximately 60 m long by 7.3 m wide with the new abutments located about 6 m back from (or behind) the existing south and north abutments, which are currently at about Sta. 10+314 and Sta. 10+363, respectively. The finished grade of Highway 652 will essentially remain the same.

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 Chin 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 embankments are considered feasible to support the proposed bridge abutments.
- **Driven Steel H-piles:** Driven steel H-piles terminating in the very dense portion of the till deposit are feasible for support of the abutments in particular if higher loads are required that cannot be accommodated by shallow foundations 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; 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 till 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. 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. From a foundations perspective, shallow foundations are considered to be more practical and economical in terms of initial construction costs; however, based on discussions with AECOM, we understand that driven steel piles have been identified as the preferred option from MTO's perspective.

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 281.7 m (approximately 2.6 m depth), 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)
Loose to compact sand fill	1.0	650	250
	1.5	675	165
	2.0	700	125

(1) The factored ultimate geotechnical resistances assume that the footings are placed at least 6 m back from (i.e., beyond) the edge 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.

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 δ) should be taken as 0.5; for precast footings, the coefficient of friction (tan δ) 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 friction piles driven into the generally compact to very dense sand and silt to silty sand till are also considered feasible for the support of the proposed structure. 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.

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 and HP360x132 driven steel piles at the proposed abutments.

Foundation Element	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)(3)
North Abutment	HP 310x110	281.7 m	263.5 m	18.2 m	750 kN	N/A
	HP 360x132				850 kN	N/A
South Abutment	HP 310x110	281.7 m	262.5 m	19.2 m	750 kN	N/A
	HP 360x132				850 kN	N/A

(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 piles may need to be driven to deeper depths to achieve the designed axial geotechnical resistances depending on the relatively density of the deposit at the pile tip, which based on the available information is variable.

(3) 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.

At the south and north abutments, zones of cobbles/boulders have been inferred to be present within the sand and silt to silty sand till deposit based on split-spoon refusal and/or observations of auger grinding during borehole advancement at the depths/elevations indicated below.

Borehole (Location)	Inferred Zones of Cobbles/Boulders	
	Depths (m)	Elevations (m)
CR-4 (South Abutment)	9.4 to 10.7	270.8 to 269.5
CR-2 (North Abutment)	11.4 to 18.3	268.6 to 261.7

It is anticipated that both HP310x110 and HP360x132 piles should be able to penetrate these zones of inferred cobbles/boulders. Given the potential for cobbles/boulders within the glacially derived till deposit, consideration may need to be given to the use of the heavier HP360x132 piles to minimize the chance of hang-up and/or damage to the piles during installation. The pile selection should be re-evaluated during detail design based on future additional subsurface investigations.

6.5.2 Downdrag

Based on discussion with AECOM, we understand that embankment widening is required at this site, which will result in minor settlements as discussed in Section 6.7.4.4. Although the anticipated settlement is relatively minor (i.e. about 5 mm), there is still a potential for downdrag loads to occur along the new piles at the north abutment due to settlement of the silty clay/clayey silt deposit. Downdrag loads are currently not anticipated at the south abutment since (based on the current investigation) the silty clay/clayey silt deposit appears to have been fully sub-excavated prior to construction of the existing south approach embankment.

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 the north abutment 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 one to 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 north abutment piles. If preload mitigation is not carried out, downdrag loads (negative skin friction) may be induced on the piles supporting the north abutment due to settlement of the surrounding clayey silt and friction/adhesion along the piles.

The structural design of the north abutment piles should be based on an estimated unfactored downdrag load of 385 kN per pile for HP310X110 piles and 450 kN per pile for HP360x132 piles. 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 in accordance with OPSS 903 *(Deep Foundations).* The pile termination or set criteria will be dependent on the pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods including empirical correlations, such as the use of the Hiley Formula, and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria need to be set to allow for founding of the piles into the dense to very dense portion of the sand and silt to silty sand till deposit and to also avoid overdriving and possibly damaging the piles.

For friction piles, the pile capacity must be verified in the field by the use of the Hiley Formula (Standard Structural Drawing SS 103-11) during the final stages of driving for the ultimate capacity at the elevations provided above.

The ultimate geotechnical axial resistance predicted from the Hiley Formula should then be multiplied by a consequence factor, Ψ of 1.0 and a geotechnical resistance factors, Φ_{gu} of 0.5 as per Tables 6.1 and 6.2 of the CHBDC (2014) to verify the factored ultimate geotechnical resistance design value.

The piles should be reinforced at the tip with driving shoes or flange plates (reinforced tips) in accordance with OPSD 3000.100 (Steel H-Driving Shoe) to reduce the potential for damage to the pile tips during driving. In very dense and/or cobbly/bouldery soils, as encountered at this site, driving shoes (such as Titus Standard "H" Bearing Pile Points) are preferred over flange plates.

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.

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:

$$k_h = \frac{n_h z}{B}$$

where

n_h is the constant of horizontal subgrade reaction (kPa/m)

z is the depth (m)

B is the pile diameter or width (m)

and for cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

where

s_u is the undrained shear strength of the soil (kPa)

B is the pile diameter or width (m)

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)
North Abutment (CR-1 and CR-2)	Sand Fill (loose to compact) (above the water table)	281.7 to 278.0	6,600	-
	Sand Fill (loose to compact) (below the water table)	278.0 to 276.1	4,400	-
	Clayey Silt (firm)	276.1 to 274.5	-	35
	Sand and Silty to Silty Sand Till (Compact to Very Dense)	274.5 to 268.0	4,400	-
		268.0 to 260.0	11,000	-
South Abutment (CR-3 and CR-4)	Sand Fill (loose to compact) (above the water table)	281.7 to 278.0	4,400	-
	Sand Fill (loose to compact) (below the water table)	278.0 to 275.6	3,000	-
	Sand and Silty to Silty Sand Till (Compact to Very Dense)	275.6 to 262.0	4,400	-
		262.0 to 260.0	11,000	-

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).

The upper zone of soil (down to a depth below the pile cap equal to about 1.5 x 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 49.251378° and longitude -80.646547°, 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)
Sa (0.2) (g)	0.187
Sa (1.0) (g)	0.049

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 is to be maintained (i.e., no grade raise); however, an approximately 10 m widening of the existing approach embankments (to the east) will be required to facilitate the proposed constructing staging (i.e., a laydown/launch area and temporary landing area).

6.7.1 Removal of Organics and/or Cohesive Fill

It is recommended that any existing organics (i.e., peat, topsoil and/or mixed organic soil) and/or existing clayey silt to silty clay fill be removed below the footprint of the proposed embankment widenings within the limits of the approach embankments (i.e., up to about 20m beyond the abutments). Approximately 1.4 m to 1.5 m of silty clay to clayey silt fill (and topsoil) was encountered at the east toes of the embankment slopes. The excavations should be backfilled with appropriate granular material as discussed 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 slope. 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 steep 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 OPSD 208.010 *(Benching of Earth Slopes).*

The abutment front slopes and side slopes adjacent to the river require erosion protection in accordance with OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting). Erosion protection should be placed on the slopes up to at least 0.5 m above the design high water level. Erosion protection could consist of a minimum 0.6 m thick layer of R-10 Rip Rap (300 mm diameter as per OPSS.PROV 1004, Aggregates), rock protection or concrete slope paving. The designer should address the potential for scour below the 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 slope 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 GEOCRES report and the results of the current investigation, we understand that a majority of the in-situ peat and native silty clay to clayey silt was previously sub-excavated prior to construction of the existing highway embankments; however, there is an approximately 1.6 m thick layer of clayey silt remaining below the embankment fill at the north abutment as noted in Borehole CR-1. It also appears that a portion of the previously sub-excavated silty clay to clayey silt was used as backfill at the toe of slope. The analysis discussed below assumes that the existing clayey silt to silty clay fill at the toe of the embankment slope will be sub-excavated and replaced prior to the construction of the new embankment widening(s).

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 analysis carried out for the preliminary design includes the existing north front slope as well as the proposed 10 m widening along the east side of the north approach embankment. The stability analyses were completed based on the subsurface conditions as encountered in Boreholes CR-1 and CR-2 and the geometries provide in the GA drawing and cross sections provided by AECOM. Stability analysis has not been carried out for the south approach (front slope and widening) because no cohesive soils were encountered at this location. For the proposed granular fill widening to be constructed on top of the existing granular embankment fill and overlying the sand and silt to silty sand foundation soil, the target FoS will be achieved so long as the new slopes are no steeper than 2H:1V and the surface of the slopes are provided with erosion protection.

6.7.3.2 Parameter Selection

For the new/existing granular fill and the non-cohesive sand and silt to silty sand 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 existing clayey silt to silt clay fill and native clayey silt deposits, 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 1981 investigation (GEOCRES 42H-23) and estimated from correlations with the SPT results from the current investigation. Effective friction angles have also been estimated for these cohesive deposits for analysis of the factor of safety in long-term conditions.

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
Existing Clayey Silt to Silty Clay Fill	17	-	40	28
Native Backfill	17	27	-	27
Clayey Silt	17	-	35	28
Silt and Sand to Silty Sand (TILL)	19	35	-	35

6.7.3.3 Results of Analysis

The stability analyses indicates that the approximately 6.5 m high north front slope inclined at approximately 2H:1V has a FoS greater than 1.3 and 1.5 against global instability in the short-term (undrained) and long-term (drained) conditions, respectively, as shown on Figures 1 and 2. Similarly, the approximately 3 m high widened north approach embankment (east side) 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 abutment/approach embankments should be reviewed and confirmed at the detail design stage based on the final embankment geometries and incorporating any additional loadings (i.e. if shallow foundations are adopted and/or any additional loadings on the embankments as part of the staging and replacement bridge construction) or subsurface information obtained during detail design.

6.7.4 Approach Embankment Settlement

6.7.4.1 Methodology

To estimate the magnitude of the settlement as a result of the proposed embankment widening(s), 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 rate of settlement of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidating theory. The sources of settlement were considered to include:

- immediate settlement of the cohesionless deposits
- time-dependent consolidation of the cohesive deposit

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)	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).

Deposit	Unit Weight (kN/m^3)	Elastic Modulus (MPa)
Existing Granular Fill (Very Loose to Compact)	20	15
Clayey Silt (Firm)	17	See below
Silt and Sand to Silty Sand Till (Loose to Compact)	19	25
Silt and Sand to Silty Sand Till (Dense to Very Dense)	19	50

The following correlation relating in-situ undrained shear strength to pre-consolidation stress (Mesri, 1975) was employed:

$$\sigma_p' = s_{u(mob)} / 0.22$$

where: σ_p' = pre-consolidation stress (kPa)

$s_{u(mob)}$ = average mobilized undrained shear strength (kPa)

The undrained shear strength was estimated based on SPT 'N'-values from the current investigation as well as the field vane shear strengths from the original 1981 investigation (GEOCRES 42H-23) prior to construction the existing bridge. The consolidation settlement of the cohesive deposits (clayey silt) was assessed using the results of the laboratory index testing from the both the current and original 1981 investigation to estimate the deformation parameters (i.e., recompression and compression indices) using empirical correlations proposed in literature by Koppula (1986). The estimated settlement parameters for the clayey silt strata are summarized below.

Deposit	σ_{vo}'	σ_p'	OCR	e_o	C_r	C_c
Clayey Silt (Firm)	65 kPa	160kPa	2.5	0.8	0.05	0.5

where: σ_{vo}' effective overburden pressure in kPa
σ_p' preconsolidation pressure in kPa
OCR overconsolidation ratio
e_o initial void ratio
C_c compression index (based on void ratio)
C_r recompression index (based on void ratio)

The coefficient of consolidation, c_v (cm^2/s), required in the time-rate of settlement analysis was estimate from the correlation with liquid limit (NAVFAC, 1986) assuming over-consolidated clays. A c_v equal to 4.0×10^{-2} cm^2/s is considered appropriate for the over-consolidated (re-compression) range.

6.7.4.4 Results of Analysis

A summary of the results of the settlement analysis at the north approach is present below.

Critical Section	Relevant Borehole	Post Construction Settlement		
		Hwy 652 Centerline	Hwy 652 Edge of Shoulder	Hwy 652 Toe of Slope (Laydown/Launch Area)
North Approach	CR-1 and CR-2	5 mm	5 mm	10 mm

Based on the Cv value of the $4.0x10^{-2}$ cm^2/s, it is estimate that about 90 per cent of the primary consolidation settlement will be completed in about one to two weeks (i.e., essentially during construction). Therefore settlement mitigation will not be required (other than the requirements for preloading to reduce the risk of dragloads on the new abutment piles if required (as discussed in Section 6.5.2); however, we recommend delaying paving as long as possible.

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

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

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 foundation excavations for spread footings or pile caps would extend approximately 2.6 m into the loose to compact granular embankment fill. If space permits, 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 and/or existing cohesive fill materials prior to embankment widening(s). The organic and/or the existing clayey silt to silty clay fill 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 are not anticipated. 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 presence of cobbles/boulders within the till deposit and/or by the very dense zones within the till deposit.

The 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:

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
Existing Clayey Silt to Silty Clay Fill (Stiff)	17	28	0.36	0.53	2.77
Clayey Silt (Firm)	17	28	0.36	0.53	2.77
Sand and Silt to Silty Sand Till (Loose to Very Dense)	19	35	0.27	0.43	3.69

(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

Although footing/pile cap construction is anticipated to be carried out in the dry conditions, excavations up to about 1.5 m below the existing ground surface are anticipated at the toe of the embankment slope (i.e., Elevation 278.5 m) for removal and replacement of the organics and/or existing cohesive fill material below the footprint of the widened embankments. However, depending on the time of year that construction is carried out, this may be below the groundwater/river water level. Dewatering could be in the form of temporary pumping from properly filtered sumps below the base of the excavation pumping from a temporary cofferdam. 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 are glacially derived and as such should be expected to contain coarse gravel, cobbles and boulders, which could affect the installation of temporary support systems (including cofferdams) or deep foundations. Further observations for the presence of cobbles and/or boulders are recommended in the next stage of investigation in support of the detail design. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the 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 cobbles and boulders for the following reasons:

- the age of the existing structure
- the existing piers are founded on friction piles
- 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 an analytical test on 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. It should be noted that the soil samples were submitted beyond the standard hold times and the test results may not be valid/reliable. As such, additional corrosivity testing should be performed at detail design.

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 that the soil 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 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 and deeper boreholes (at the foundation elements and within the approach embankment areas) and carrying out additional foundation analysis as the requirements for detail design are finalized. Such additional work would further assess and/or confirm the subsurface conditions and refine the preliminary foundation 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
- 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 assessment and potential to provide higher axial geotechnical resistance for the pile foundations (if required) by extending boreholes to a deeper depth
- 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. David Muldowney, P.Eng. Mr. Paul Dittrich, Ph.D., P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.

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ASTM International

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

Commercial Software

Slope/W - Geostudio (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
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe

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

Table 1: Evaluation of Abutment Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Spread Footings	1	■ Straightforward construction ■ Adequately geotechnical axial resistances. ■ Use of pre-cast footing could accelerate construction.	■ Relatively 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.	■ Shallow foundations typically have lower cost than deep foundations, however, additional costs associated with temporary protection, support and/or shoring may be required.	■ Higher risk of differential settlement due to variability in state of compactness of embankment fill subgrade. ■ Depending on final bridge geometry, location of new footings could affect global embankment stability; this would need to be evaluated at detail design stage.

Foundation Type	Rank	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Driven Steel H-Piles	2	■ 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 of organic materials and/or cohesive fill below the widened footprint.	■ 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. ■ Potential for downdrag on the piles related to embankment widening.	■ Relative costs higher than shallow foundations due to requirements to mobilize pile driving rig. ■ Relative costs lower than other deep foundation options	■ Some risk of vibrations during driving affecting existing bridge. ■ Vibration monitoring may be required to identify and control risks.
Drilled Steel Casings (Small Diameter)	NR	■ Higher axial resistances compared to steel H-piles. ■ Easier to penetrate compared to larger diameter caissons, or 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.	■ Placement of tremie concrete below the water required to complete the DSC elements.	■ Relative costs higher than for steel H-piles	■ Lower risk of difficulties during installation through till deposits ■ Potential for disturbance at the base of the DSC (terminated in till, not bedrock) could affect the geotechnical resistance(s).

Foundation Type	Rank	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Drilled Shafts/Caissons (Large Diameter)	NR	■ Higher axial resistances compared to steel H-piles and smaller diameter DSCs.	■ Temporary liners may be required to control groundwater inflow. ■ Potential for difficulties penetrating through obstructions compared to piles or drilled steel casings. ■ Placement of tremie concrete below the water required to complete the caissons.	■ Relative costs much higher than for steel H-piles.	■ Potential risk for not achieving design resistance at design pile tip elevation due to the presence of cobbles and boulders. ■ Potential for construction problems associated with groundwater inflow during caisson installation. ■ Potential for disturbance at the base of the caisson (terminated in till) could affect the geotechnical resistance(s).

NR: Not Recommended

Photograph 1: Chin River Bridge
North Approach Facing South

Photograph 2: Chin River Bridge
South Approach facing South

Photograph 3: Chin River Bridge
North Approach Facing South

MINISTRY OF TRANSPORTATION, ONTARIO

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

CONT No.
WP No.5416-15-01

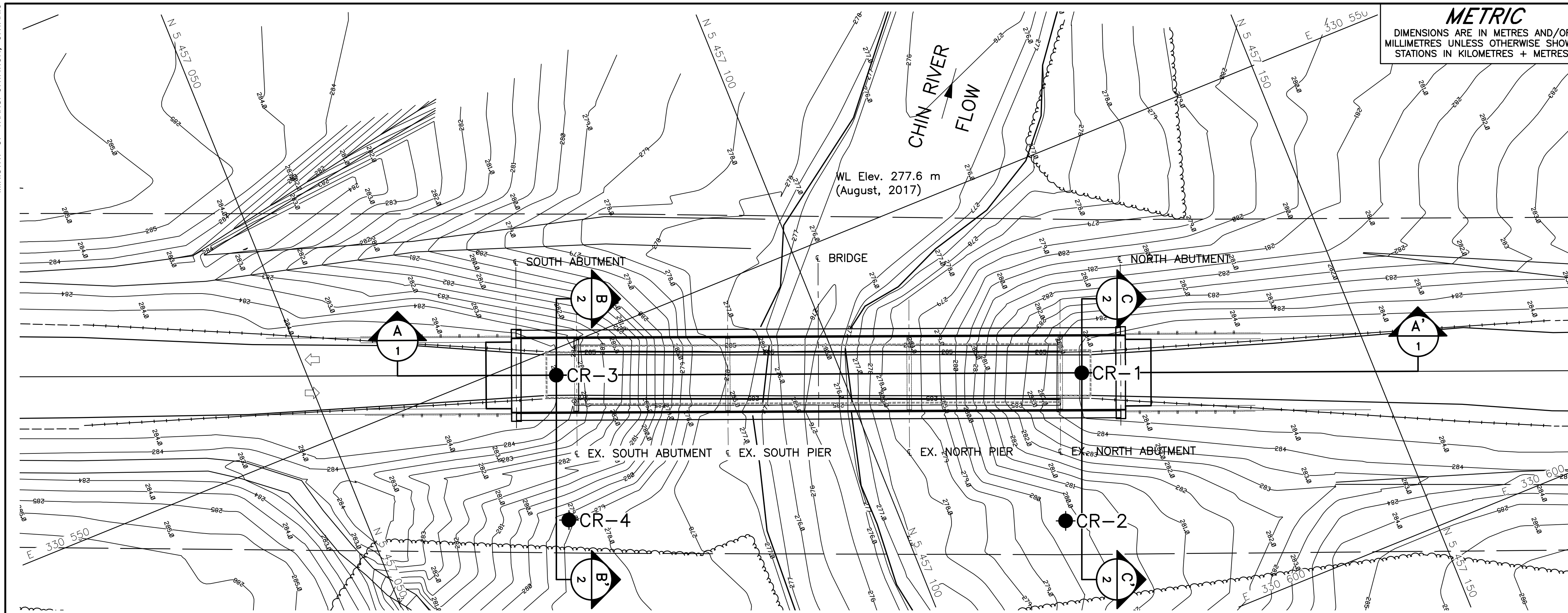
SHEET

HIGHWAY 652

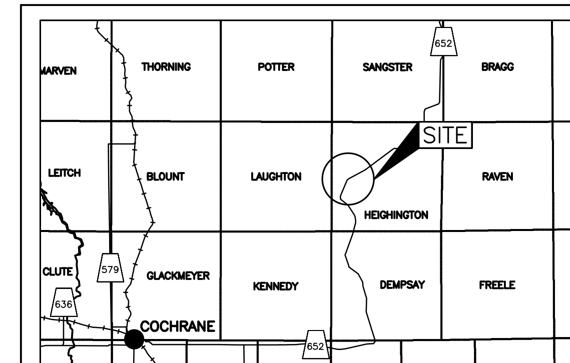
CHIN RIVER BRIDGE
LAT. 49.251387; LONG. -80.646589

BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER



PLAN
SCALE
5 0 5 10 m



KEY PLAN
SCALE
10 0 10 20 km

LEGEND

●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES (NAD83 MTM ZONE12)

No.	ELEVATION	NORTHING	EASTING
CR-1	284.3	5457122.5	330572.2
CR-2	280.0	5457115.4	330585.4
CR-3	284.3	5457073.3	330552.5
CR-4	280.2	5457069.0	330566.5

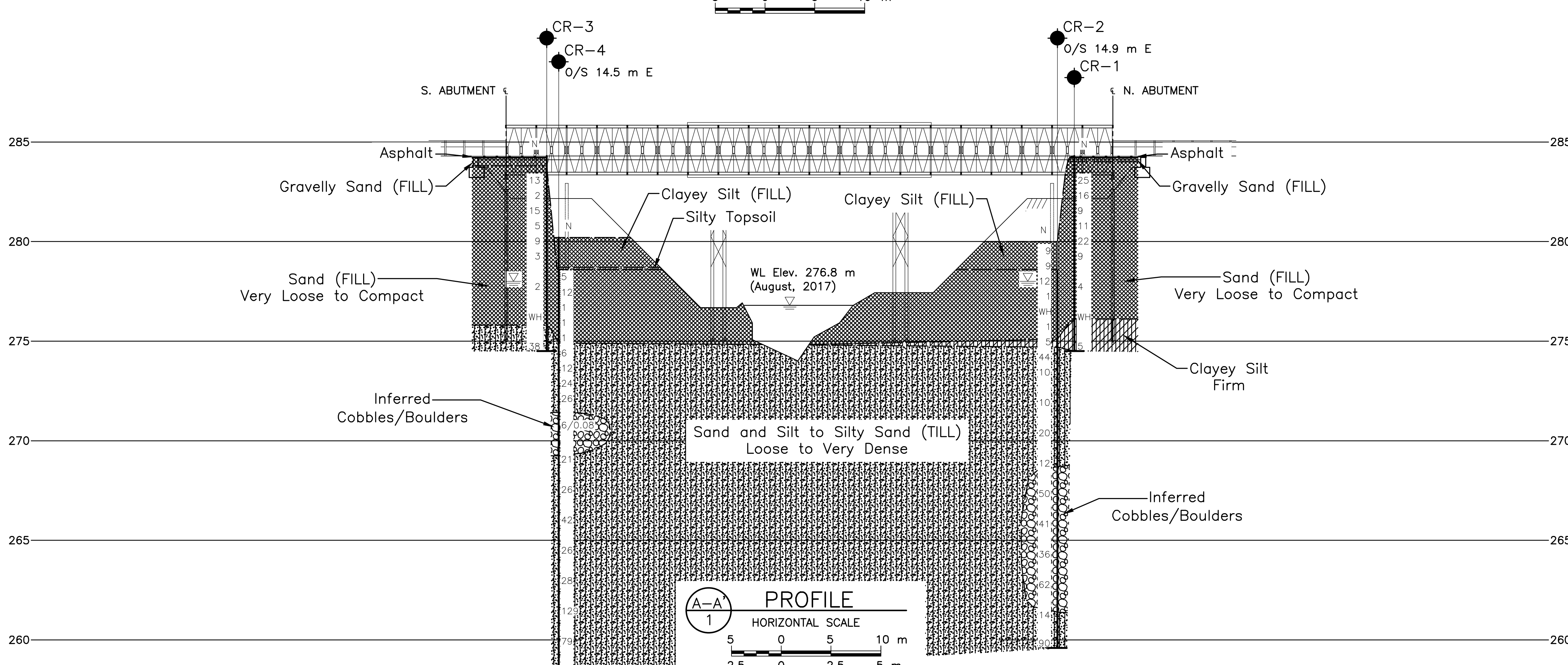
NOTES

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 nos. Chin.dwg, received SEPT 26, 2017. General Arrangement provided by AECOM, drawing file nos. 60547656-P1.dwg, received October 6, 2017.



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

D.A.MULDOWNEY 100164716 APR 06, 2018 LICENSED PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

J. P. DITTRICH LICENSED PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 42H-74

HWY. 652	PROJECT NO. 1651997		DIST.
SUBM'D. AC	CHKD.	DATE: 4/6/2018	SITE: 39E-197
DRAWN: JJL/TB	CHKD. DAM	APPD. JPD	DWG. 1

PLOT DATE: April 6, 2018
FILENAME: S:\Clients\MTO\1651997_AECOM_5015-E-0045_NE Retainer\99_PROJ\1651997_Chin_River\40_PROD\0001\1651997-0001-BG-0001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

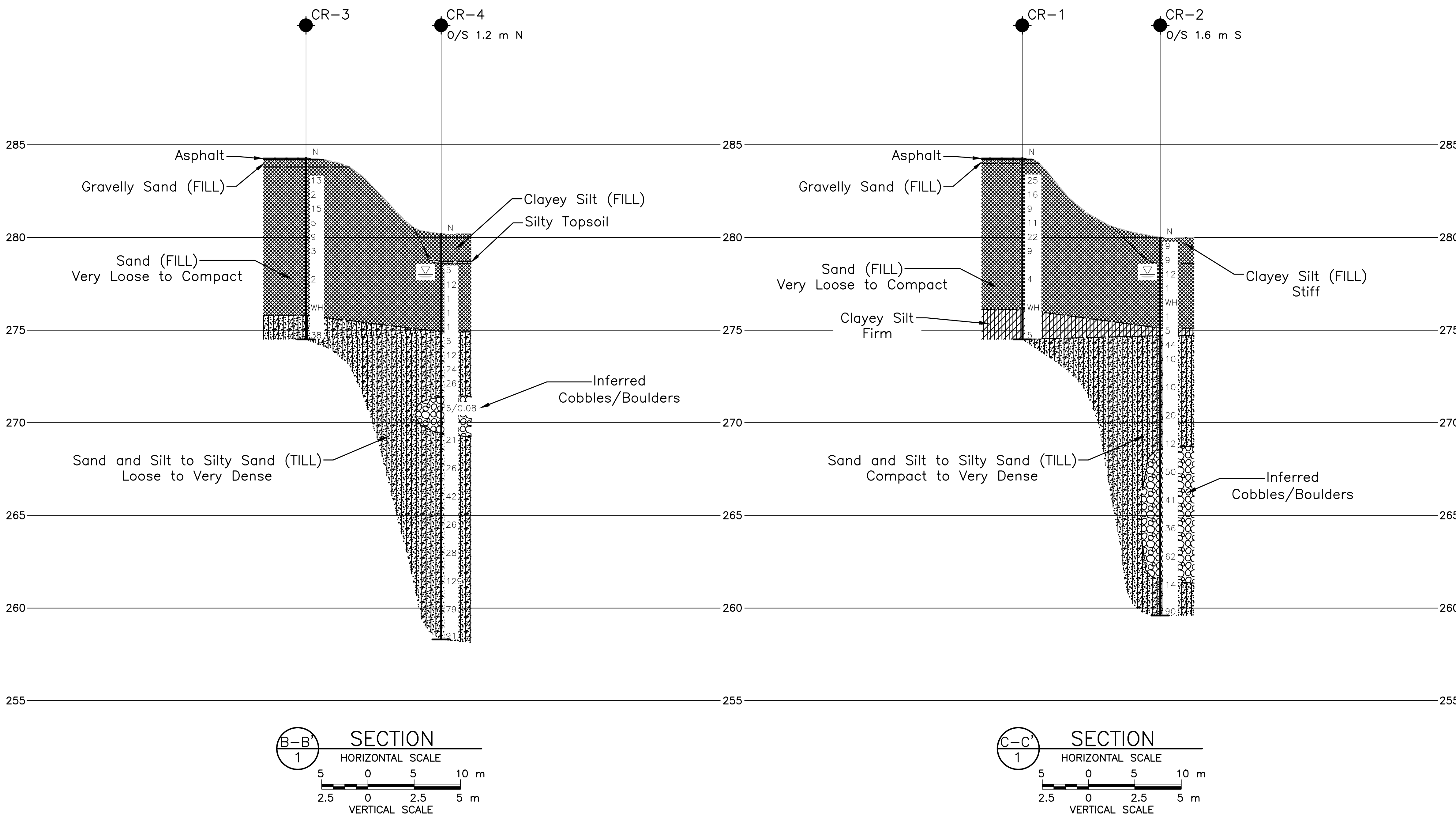
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No.5416–15–01

HIGHWAY 652
CHIN RIVER BRIDGE
LAT. 49.251387; LONG. –80.646589
SOIL STRATA

SHEET

GOLDER



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

BOREHOLE CO–ORDINATES (NAD83 MTM ZONE12)

No.	ELEVATION	NORTHING	EASTING
CR–1	284.3	5457122.5	330572.2
CR–2	280.0	5457115.4	330585.4
CR–3	284.3	5457073.3	330552.5
CR–4	280.2	5457069.0	330566.5

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 nos. Chin.dwg, received SEPT 26, 2017. General Arrangement provided by AECOM, drawing file nos. 60547656–P1.dwg, received October 6, 2017.

LICENSED PROFESSIONAL ENGINEER
D.A. MULDOWNEY
100164716
APR 06, 2018
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J. P. DITTRICH
PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 42H–74

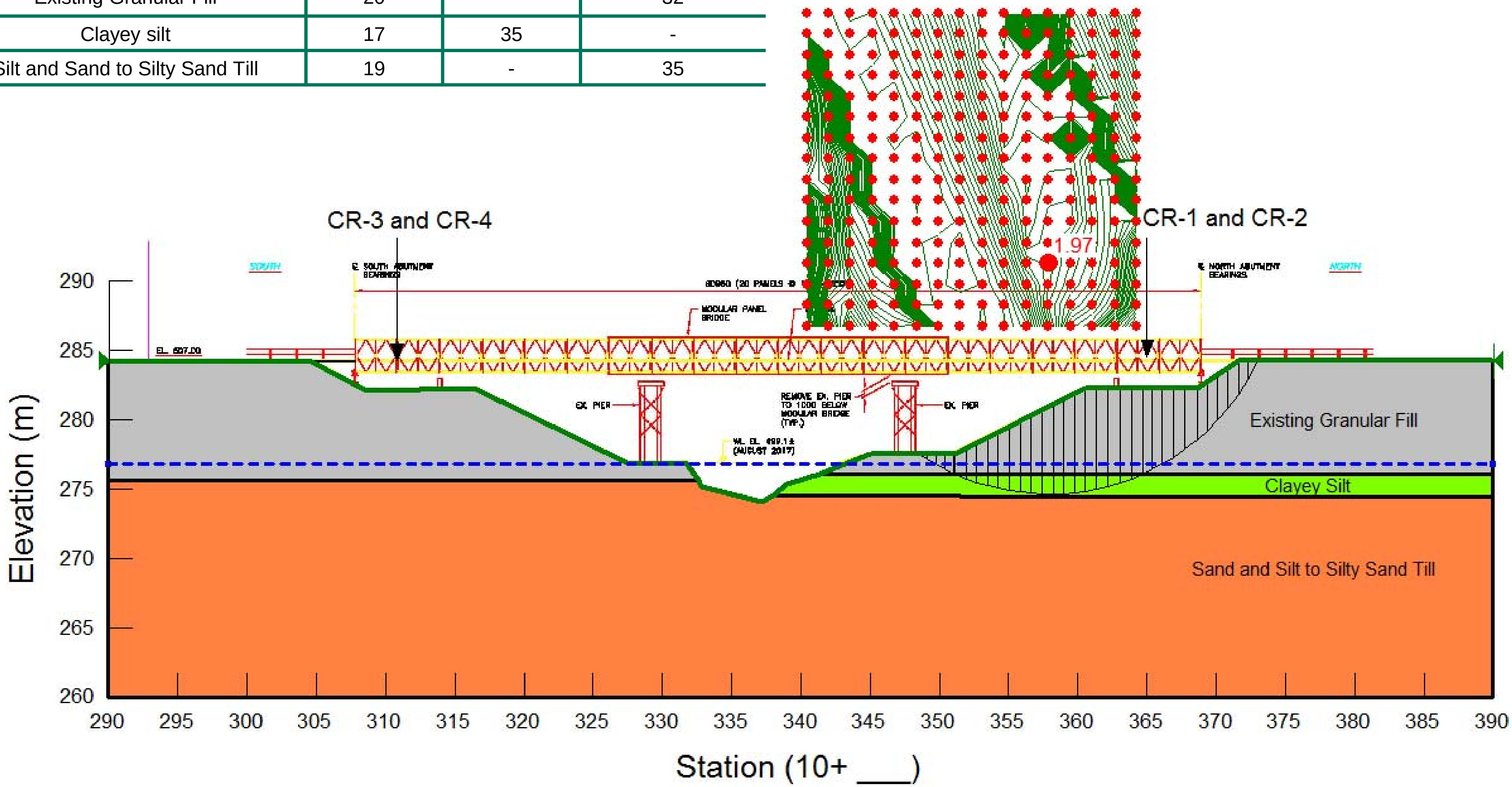
HWY. 652		PROJECT NO. 1651997	DIST. .
SUBM'D. AC	CHKD. .	DATE: 4/6/2018	SITE: 39E–197
DRAWN: JJL/TB	CHKD. DAM	APPD. JPD	DWG. 2

PLOT DATE: April 6, 2018

Global Stability Analysis
North Front Slope
Short-Term (Undrained) Analysis

Figure 1

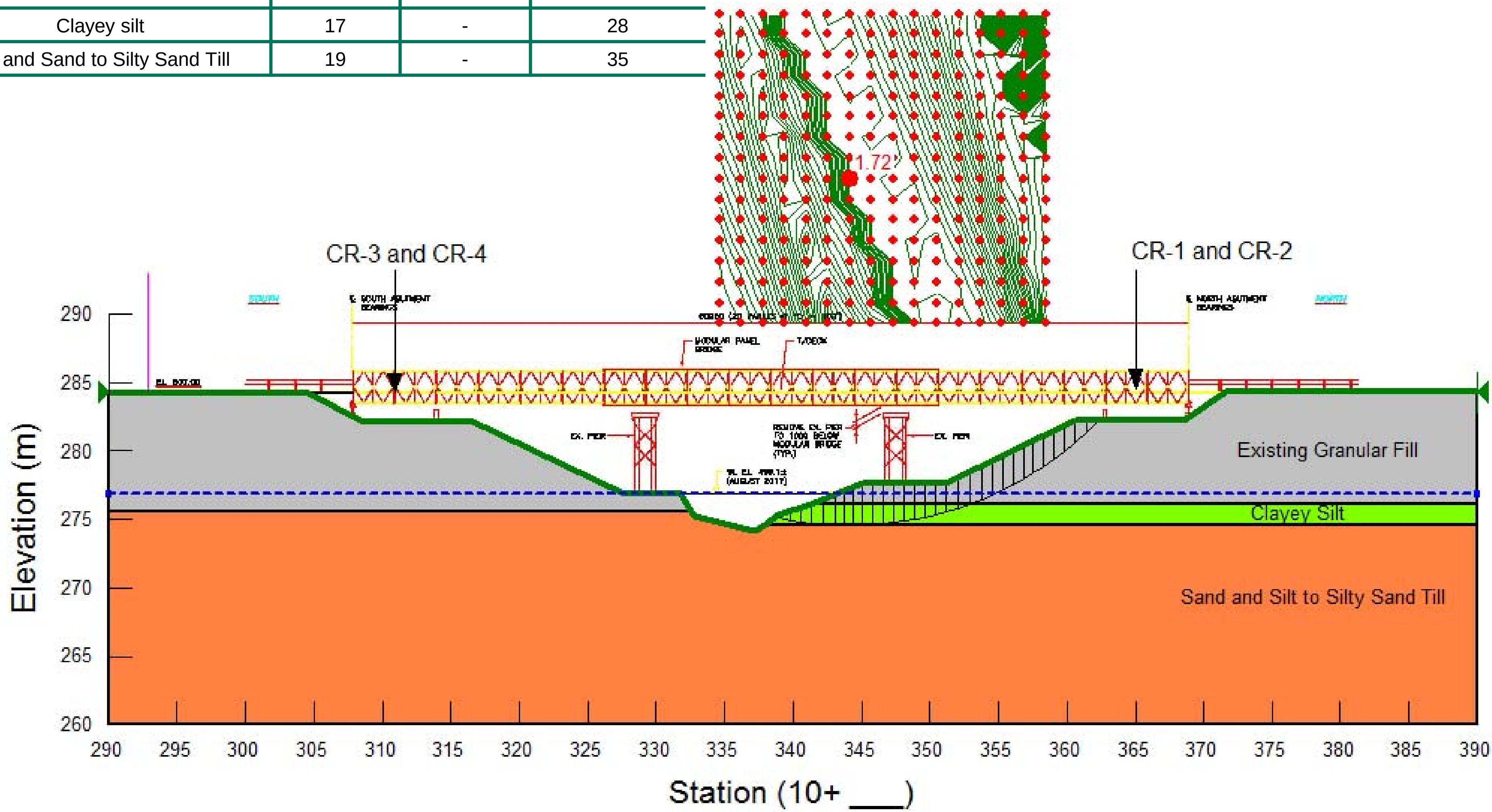
Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	32
Clayey silt	17	35	-
Silt and Sand to Silty Sand Till	19	-	35



Date: December, 2017
Project No: 1651997 – Chin River Bridge

Analysis By: AC
Reviewed By: DAM

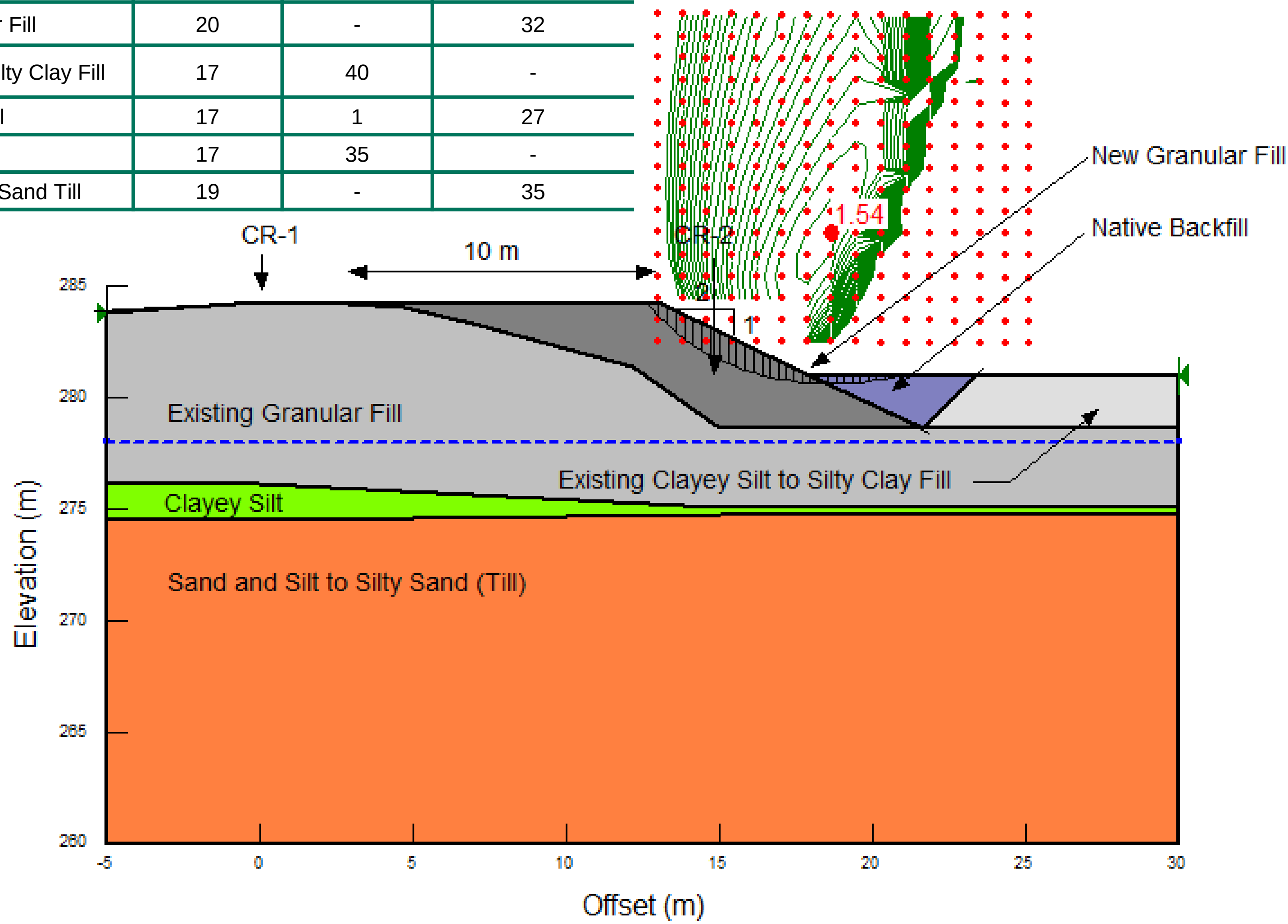
Material Name	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
Existing Granular Fill	20	-	32
Clayey silt	17	-	28
Silt and Sand to Silty Sand Till	19	-	35



Global Stability Analysis
Northeast Embankment Widening
Short-Term (Undrained) Analysis

Figure 3

Material Name	Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
New Granular Fill	21	-	35
Existing Granular Fill	20	-	32
Existing Clayey Silt to Silty Clay Fill	17	40	-
Native Backfill	17	1	27
Clayey Silt	17	35	-
Silt and Sand to Silty Sand Till	19	-	35



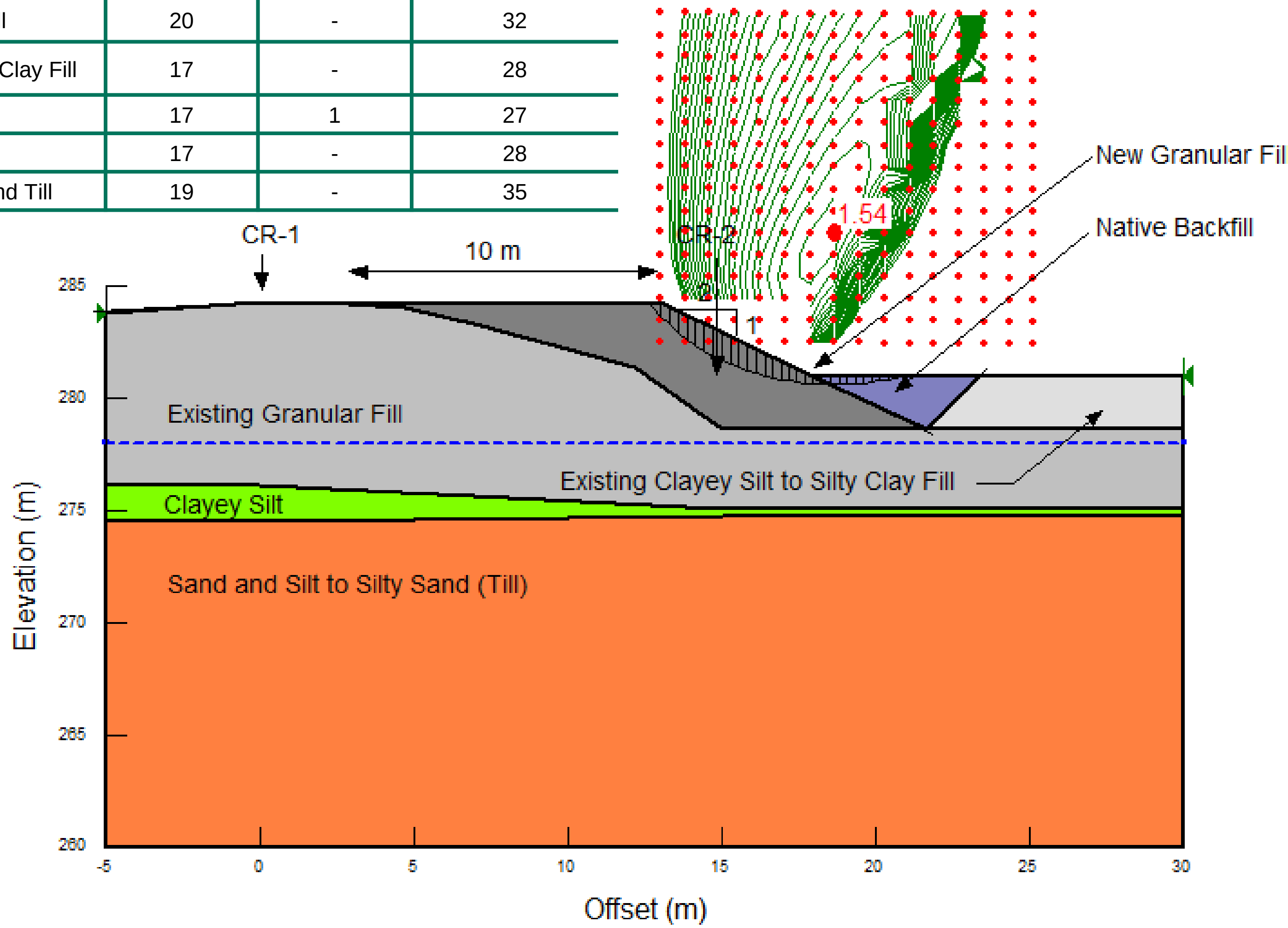
Date: December, 2017
Project No: 1651997 – Chin River Bridge

Analysis By: AC
Reviewed By: DAM

Global Stability Analysis
Northeast Embankment Widening
Long-Term (Drained) Analysis

Figure 4

Material Name	Unit Weight (kN/m³)	Undrained Shear Strength (kPa)	Friction Angle (degrees)
New Granular Fill	21	-	35
Existing Granular Fill	20	-	32
Existing Clayey Silt to Silty Clay Fill	17	-	28
Native Backfill	17	1	27
Clayey Silt	17	-	28
Silt and Sand to Silty Sand Till	19	-	35



Date: December, 2017
Project No: 1651997 – Chin River Bridge

Analysis By: AC
Reviewed By: DAM

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
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

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

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils
Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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

RECORD OF BOREHOLE No CR-1

 METRIC

PROJECT 16519971651997-WO5
W.P. 5416-15-01
LOCATION N 5457122.5; E 330572.2 NAD83 MTM ZONE 12 (LAT. 49.251594; LONG. -80.645963)
ORIGINATED BY MR
DIST
HWY 652
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY AC
DATUM GEODETIC
DATE July 28, 2017
CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
284.3	GROUND SURFACE						
0.0	ASPHALT (50 mm)				284		
0.3	Gravelly sand (FILL) Brown Moist						
	Sand, trace to some gravel, trace silt, trace clay (FILL) Very loose to compact Brown Moist to wet	1	SS	25	283		
		2	SS	16	282		
		3	SS	9		○	1 93 2 4
		4	SS	11	281		
	Samples wet below 3.8 m depth.	5	SS	22	280		
		6	SS	9	279		
		7	SS	4	278	○	13 83 (4)
		8	SS	WH	277		
276.1 8.2	CLAYEY SILT, trace sand, trace gravel Firm Grey Wet				276		
		9	SS	5	275	w_P–w–w_L	
274.5 9.8	END OF BOREHOLE						

Note:

1. Borehole dry upon completion of drilling inside augers.

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

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

RECORD OF BOREHOLE No CR-2

 METRIC

PROJECT 16519971651997-WO5

W.P. 5416-15-01

LOCATION N 5457115.4; E 330585.4 NAD83 MTM ZONE 12 (LAT. 49.251529; LONG. -80.645782)

ORIGINATED BY MR

DIST HWY 652

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

COMPILED BY AC

DATUM GEODETIC

DATE July 19, 2017

CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
280.0	GROUND SURFACE											
0.0	Clayey silt, some organics, trace wood (FILL) Stiff Dark brown Wet	1	SS	9								
		2	SS	9		279						
278.6 / 1.4	Sand, trace to some gravel, trace to some silt (FILL) Very loose to compact Brown Moist to wet	3	SS	12	∇	278	○		9	82	(9)	
		4	SS	1		277						
		5	SS	WH								
		6	SS	1		276						
275.1 / 4.9	CLAYEY SILT Firm Grey Wet	7A / 7B	SS	5		275	wP–w					
274.7 / 5.3	SAND and SILT to Silty SAND, trace to some gravel, trace to some clay (TILL) Compact to very dense Grey Wet	8	SS	44		274						
		9	SS	10		273						
		10	SS	10		272						
		11	SS	20		271	○	NP	8	46	40	6
		12	SS	12		270, 269						
	Augers grinding on inferred cobbles/boulders between 11.4 m and 18.3 m depth.											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

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

GOLDER

RECORD OF BOREHOLE No CR-2

 METRIC

PROJECT 16519971651997-WO5
W.P. 5416-15-01
DIST ____ HWY 652
DATUM GEODETIC
LOCATION N 5457115.4; E 330585.4 NAD83 MTM ZONE 12 (LAT. 49.251529; LONG. -80.645782)
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
DATE July 19, 2017
ORIGINATED BY MR
COMPILED BY AC
CHECKED BY DAM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
	SAND and SILT to Silty SAND, trace to some gravel, trace to some clay (TILL) Compact to very dense Grey Wet	13	SS	50							
					267						
		14	SS	41	266	○	NP	19	49	24	8
					265						
	Augers grinding on inferred cobbles/boulders between 11.4 m and 18.3 m depth.	15	SS	36							
					264						
		16	SS	62	263						
					262						
		17	SS	14			NP	10	54	27	9
					261						
		18	SS	90	260						
259.6 20.4	END OF BOREHOLE										

Note:

1. Water level at a depth of 2.0 m below ground surface (Elev. 278.0 m) upon completion of drilling.

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No CR-3

 METRIC

PROJECT 16519971651997-WO5

W.P. 5416-15-01 LOCATION N 5457073.3; E 330552.5 NAD83 MTM ZONE 12 (LAT. 49.251152; LONG. -80.646237) ORIGINATED BY MR

DIST HWY 652 BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers COMPILED BY AC

DATUM GEODETIC DATE July 28, 2017 CHECKED BY DAM

ELEV / DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
284.3	GROUND SURFACE										
0.0	ASPHALT (50 mm)										
0.1	Gravelly sand (FILL) Brown Moist				284						
283.8 / 0.5	Sand, trace to some silt, trace gravel, trace clay (FILL) Very loose to compact Brown Moist	1	SS	13	283						
		2	SS	2	282						
		3	SS	15	281						
		4	SS	5		○		2	86	11	1
		5	SS	9	280						
		6	SS	3	279						
		7	SS	2	278						
	Samples wet below 7.6 m depth.	8	SS	WH	277, 276						
275.6 / 8.7	Gravelly Silty SAND, trace to some clay (TILL) Dense Grey Wet	9	SS	38	275	○	NP	21	48	26	5
274.5 / 9.8	END OF BOREHOLE										

Note:

1. Borehole dry upon completion of drilling inside augers.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

GROUND WATER CONDITIONS; STRAT PLOT; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

GOLDER

RECORD OF BOREHOLE No CR-4

 METRIC

PROJECT 16519971651997-WO5
W.P. 5416-15-01
LOCATION N 5457069.0; E 330566.5 NAD83 MTM ZONE 12 (LAT. 49.251113; LONG. -80.646045)
ORIGINATED BY MR
DIST HWY 652
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers
COMPILED BY AC
DATUM GEODETIC
DATE July 20, 2017
CHECKED BY DAM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
280.2	GROUND SURFACE											
0.0	Clayey silt, some sand, some organics (FILL) Brown to black Moist					280, 279						
278.7	Silty TOPSOIL Black Wet	1	SS	5	∇							
1.6	Sand, trace to some silt (FILL) Very loose to compact Brown Wet	2	SS	12		278	○		0	94	(6)	
		3	SS	1		277						
	A 0.2 m thick silty clay (fill) layer encountered at 3.8 m depth.	4A / 4B	SS	WH		276	⊢○⊣					
		5	SS	1								
274.9						275						
5.3	SAND and SILT to Silty SAND, trace to some gravel, trace to some clay (TILL) Loose to very dense Grey Wet	6	SS	6		274						
		7	SS	12			○	NP	2	58	33	7
		8	SS	24		273						
		9	SS	26		272						
		10	SS	6/0.08		271						
	Split-spoon refusal (hammer bouncing) at 9.4 m depth. Augers grinding on inferred cobbles/boulders between 9.4 m and 10.7 m depth.					270						
		11	SS	21		269	○	NP	9	53	29	9

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60
REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

GOLDER

RECORD OF BOREHOLE No CR-4

 METRIC

PROJECT 16519971651997-WO5

W.P. 5416-15-01

LOCATION N 5457069.0; E 330566.5 NAD83 MTM ZONE 12 (LAT. 49.251113; LONG. -80.646045)

ORIGINATED BY MR

DIST HWY 652

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

COMPILED BY AC

DATUM GEODETIC

DATE July 20, 2017

CHECKED BY DAM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SAND and SILT to Silty SAND, trace to some gravel, trace to some clay (TILL) Loose to very dense Grey Wet				268						
		12	SS	26							
					267						
		13	SS	42							
					266						
					265						
		14	SS	26							
					264						
		15	SS	28							
					263						
					262						
		16	SS	129		w ≈ 20	NP	12	58	22	8
					261						
		17	SS	79							
					260						
					259						
258.3		18	SS	91							
21.9	END OF BOREHOLE Note: 1. The surficial clayey silt fill to 1.5 m depth was placed by Golder to provide a level drilling platform. 2. Water level at a depth of 2.2 m below ground surface (Elev. 278.0 m) upon completion of drilling.										

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

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

SUD-MTO 001 MTM ZN INC LAT/LONG S:\CLIENTS\MTO\1651997 AECOM_5015-E-0045_NE RETAINER\02_DATA\GINT\1651997.GPJ GAL-MISS.GDT 3/29/18 TB

APPENDIX B

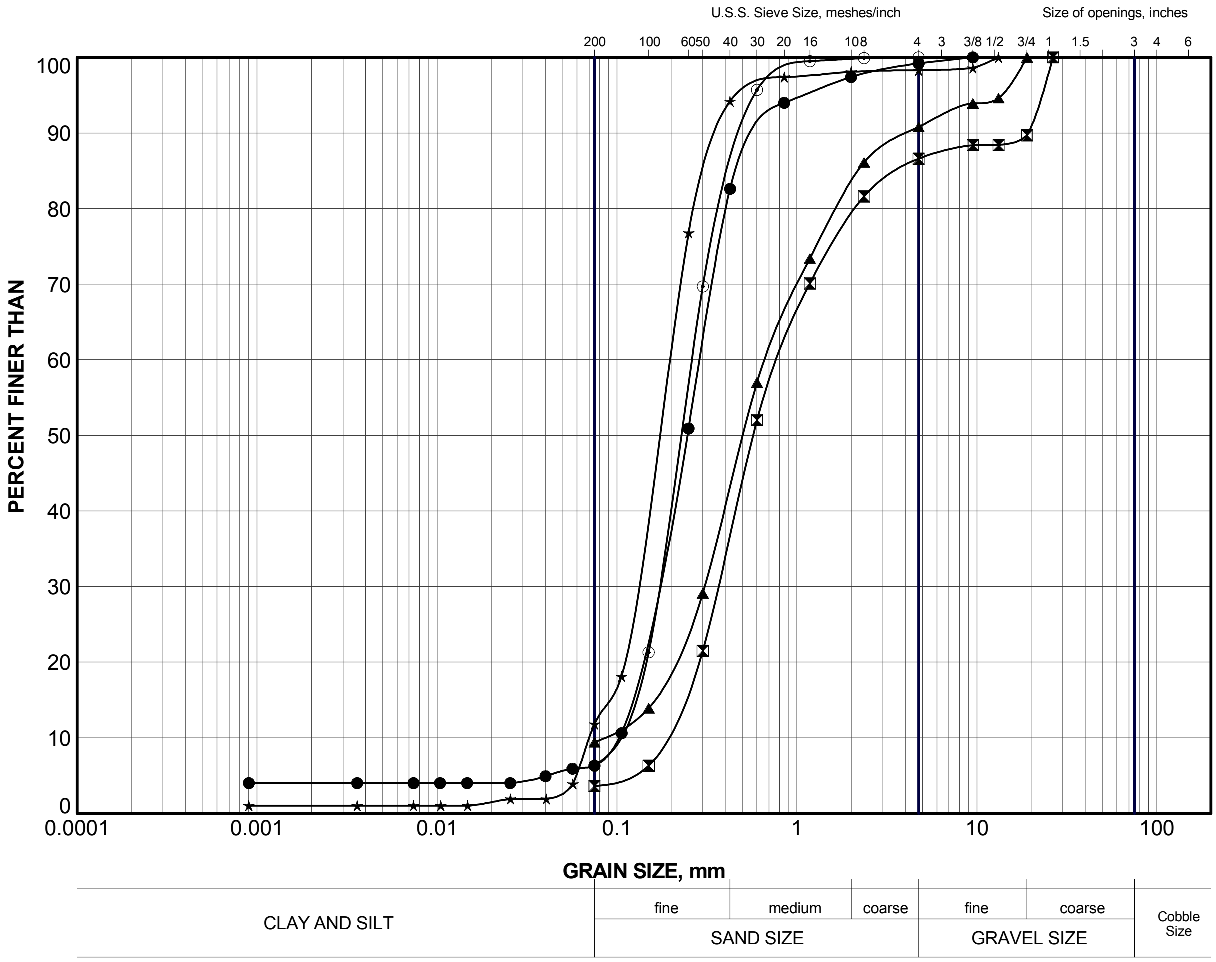
Laboratory Test Results

Table B1: Summary of Analytical Testing of Chin River Soil Samples

Location	Parameter	Units	Result
North Abutment (CR-1 SA4)	Chloride (CL)	ug/g	Not Detected (ND) (i.e., less than 20 ug/g)
	Sulphate (SO4)	ug/g	ND
	Conductivity (EC)	umho/cm	105
	Resistivity	ohm-cm	9,600
	pH	n/a	7.98
South Abutment (CR-3 SA5)	Chloride (CL)	ug/g	ND
	Sulphate (SO4)	ug/g	ND
	Conductivity (EC)	umho/cm	98
	Resistivity	ohm-cm	10,000
	pH	n/a	8.11

Notes:
1. Samples obtained on July 26, 2017 and submitted November 22, 2017
2. Analytical testing carried out by Maxxam.

Prepared by: AC
Checked by: DAM
Reviewed by: JPD



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	CR-1	3	281.7
⧗	CR-1	7	277.9
▲	CR-2	3	278.2
★	CR-3	4	280.9
⊙	CR-4	2	277.6

PROJECT

HIGHWAY 652
CHIN RIVER BRIDGE

TITLE

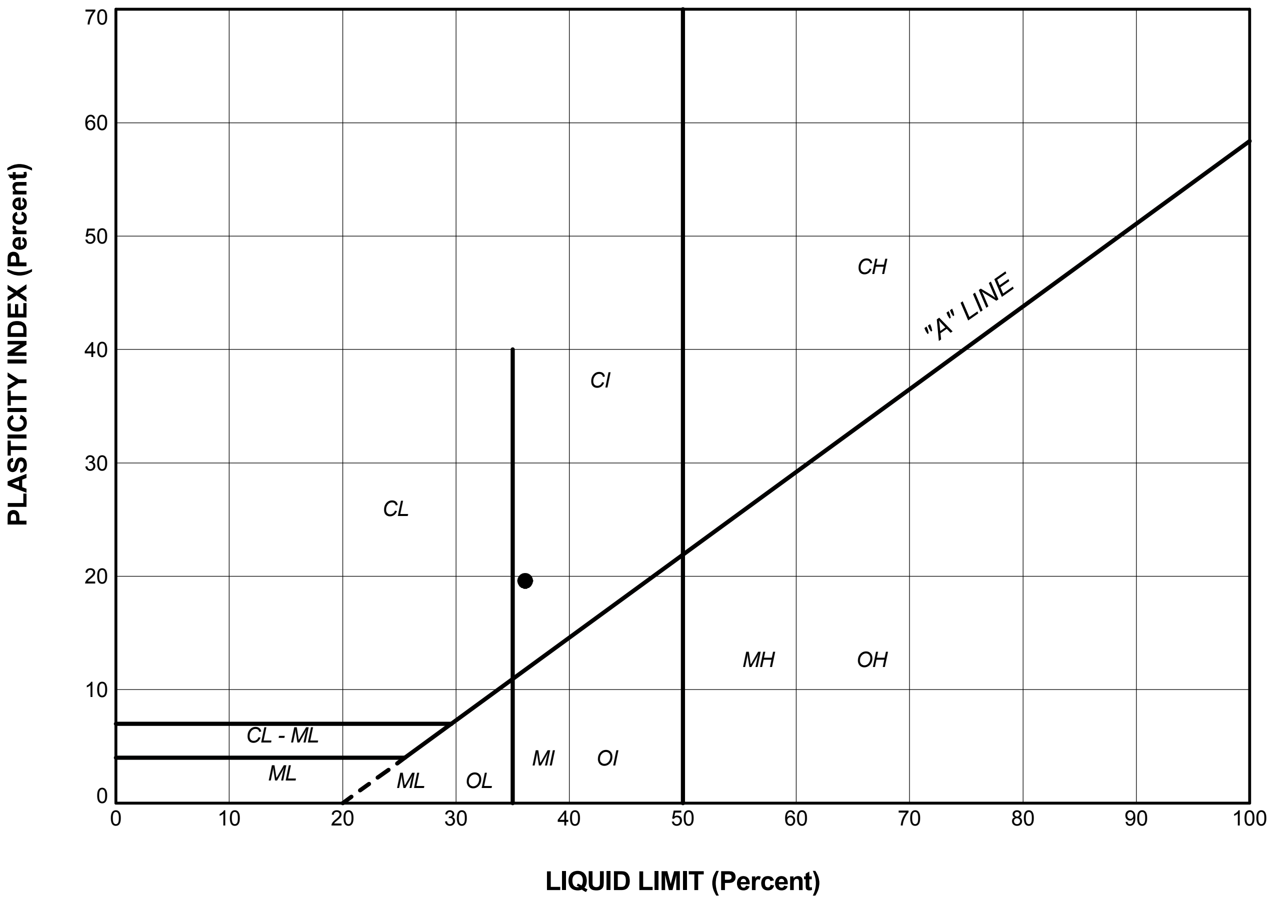
GRAIN SIZE DISTRIBUTION

SAND (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.			FILE No.	1651997.GPJ
DRAWN	JJL	Dec 2017	SCALE N/A	REV.
CHECK	DAM	Dec 2017		
APPR	JPD	Dec 2017	**FIGURE B1**	

SUD-MTO GSD (2016) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	CR-4	4A	36.1	16.5	19.6

PROJECT

HIGHWAY 652
CHIN RIVER BRIDGE

TITLE

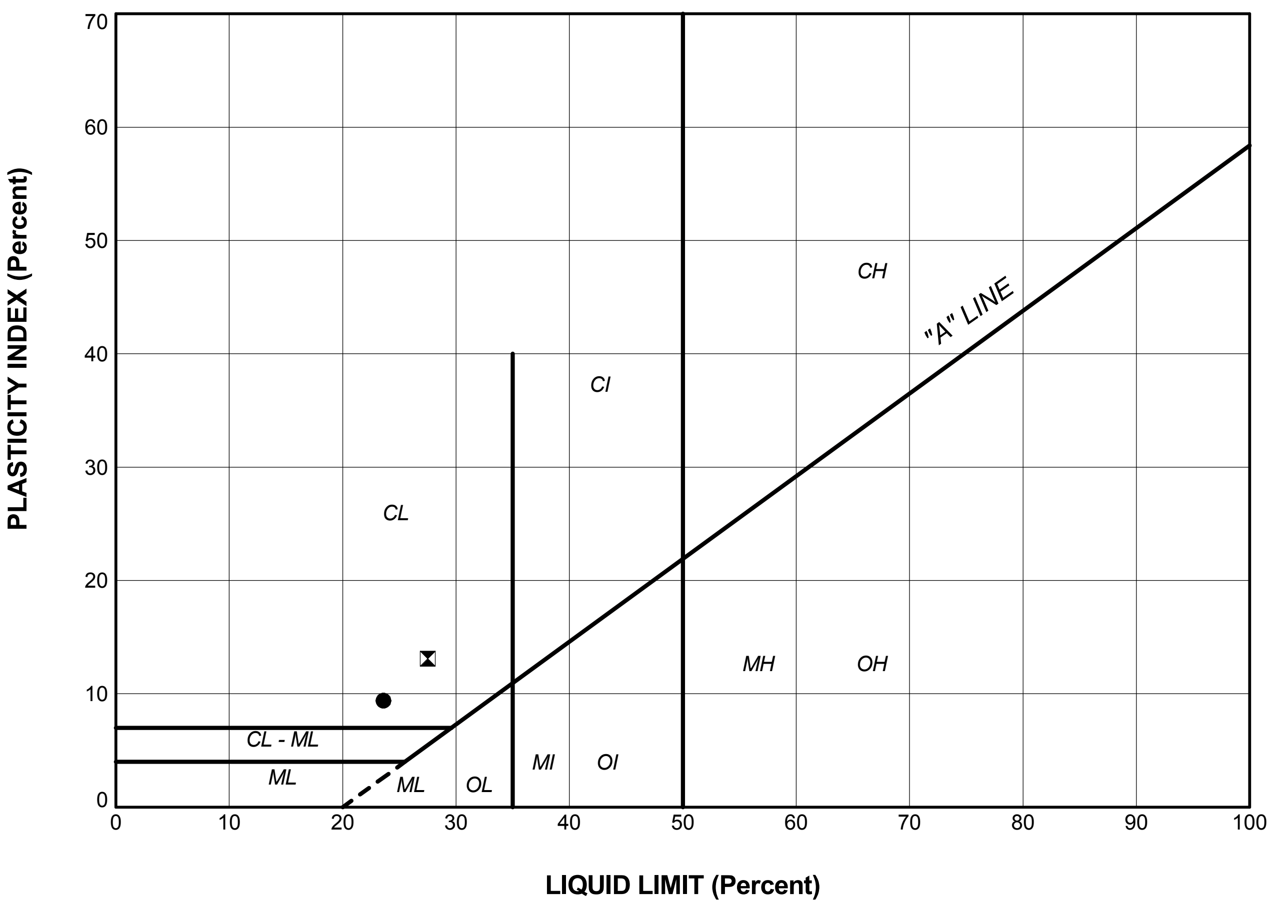
PLASTICITY CHART

SILTY CLAY (FILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.			FILE No. 1651997.GPJ	
DRAWN	JJL	Dec 2017	SCALE N/A	REV.
CHECK	DAM	Dec 2017		
APPR	JPD	Dec 2017	**FIGURE B2**	

SUD-MTO PL (2016) GLDR_LDN.GDT



SOIL TYPE
C = Clay
M = Silt
O = Organic

PLASTICITY
L = Low
I = Intermediate
H = High

LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	CR-1	9	23.6	14.2	9.4
⧗	CR-2	7B	27.5	14.4	13.1

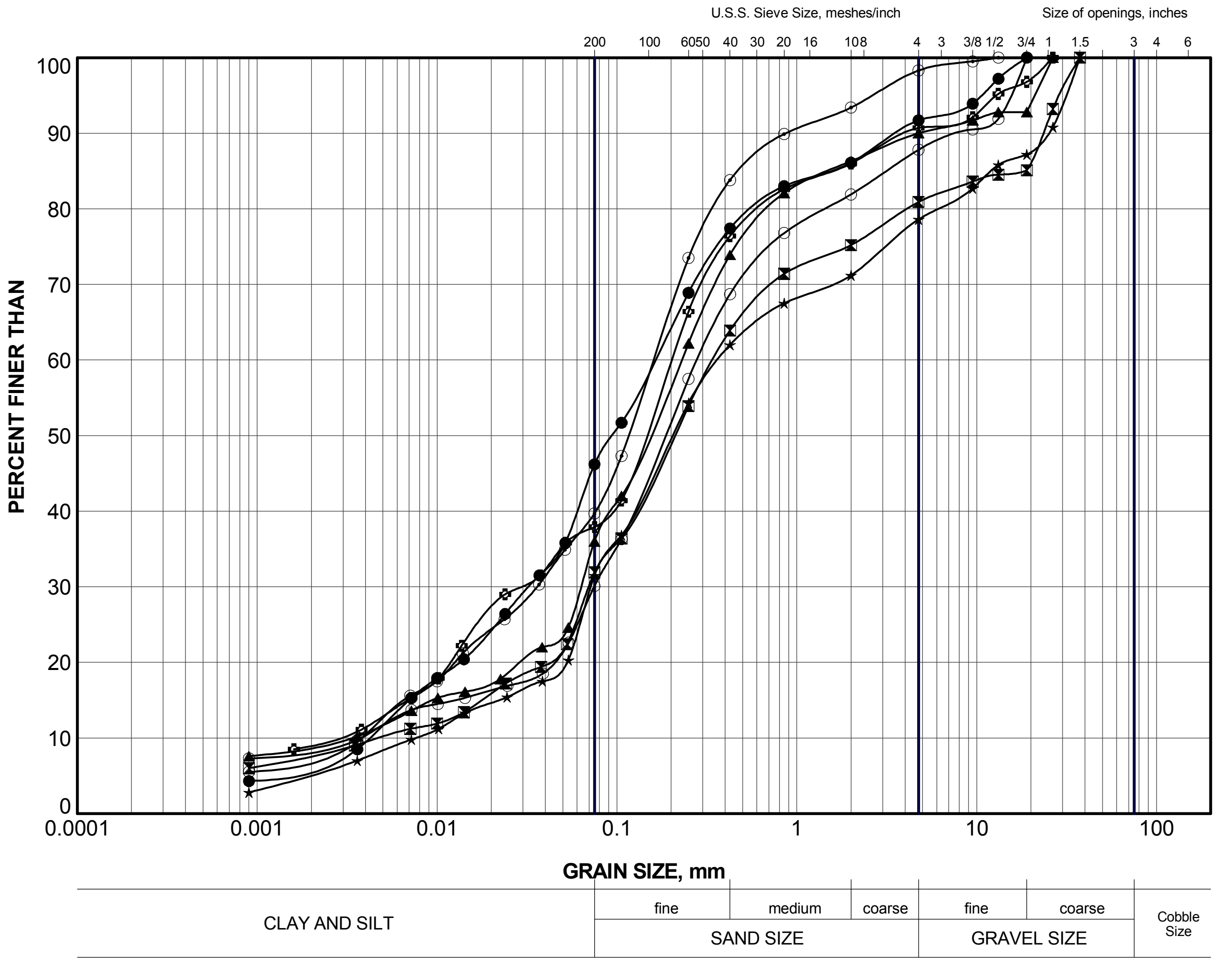
PROJECT
HIGHWAY 652
CHIN RIVER BRIDGE

TITLE
PLASTICITY CHART
CLAYEY SILT

Golder Associates
SUDBURY, ONTARIO

PROJECT No.			FILE No. 1651997.GPJ	
DRAWN	JJL	Dec 2017	SCALE N/A	REV.
CHECK	DAM	Dec 2017	**FIGURE B3**	
APPR	JPD	Dec 2017		

SUD-MTO PL (2016) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	CR-2	11	270.6
⧗	CR-2	14	266.0
▲	CR-2	17	261.4
★	CR-3	9	274.8
⊙	CR-4	7	273.8
✣	CR-4	11	269.2
○	CR-4	16	261.6

PROJECT

HIGHWAY 652
CHIN RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

SAND and SILT to SILTY SAND (TILL)

Golder Associates
SUDBURY, ONTARIO

PROJECT No.			FILE No. 1651997.GPJ
DRAWN	JJL	Dec 2017	SCALE N/A REV.
CHECK	DAM	Dec 2017	
APPR	JPD	Dec 2017	**FIGURE B4**

SUD-MTO GSD (2016) GLDR_LDN.GDT

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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