April 10, 2018

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

FLOODWOOD RIVER BRIDGE REPLACEMENT - SITE NO. 39E-203
LAT 49.490886; LONG. -80.312558
HIGHWAY 652, COCHRANE DISTRICT
TOWNSHIP OF TWEED
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5416-15-00, WP 5416-15-04

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REPORT

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
FLOODWOOD RIVER BRIDGE REPLACEMENT, SITE NO. 39E-203
HIGHWAY 652, COCHRANE DISTRICT
TOWNSHIP OF TWEED
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5416-15-00; WP 5416-15-04

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 Floodwood River Bridge (Site No. 39E-203). The existing Floodwood River bridge is located on Highway 652 north of Cochrane, Ontario in the Township of Tweed at about Sta. 10+330 (approximately 66 km north of Translimit Road). The key plan showing the general location of this section of Highway 652 and the location of the investigated area is shown 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 Drawing 1. Highway 652 is considered to be oriented in a north-south direction at this site.

In general, the topography surrounding the Floodwood River bridge site consists of undulating to rolling terrain with densely forested areas immediately beyond the Highway 652 Right-of-Way. The existing Floodwood River bridge consist of an approximately 42.6 m long by 5.4 m wide, three-span, single lane Temporary Modular Bridge (TMB). Based on information presented in the previous 1981 bridge General Arrangement (GA) drawings (Contract 81-456, WP 7-81-12 and GEOCRES 42H-019) we understand that the existing north and south abutments are founded on shallow foundations constructed on granular pads while the piers are founded on driven steel piles. Based on the survey drawing provided by AECOM, the existing bridge deck is at Elevation 284.8 m at the south abutment and Elevation 284.7 m at the north abutment.

The front slopes of the existing approach embankments are approximately 5 m to 6 m high relative to the river bottom and are inclined at profiles ranging from 1.3 horizontal to one vertical (1.3H:1V) to 2H:1V. The side slopes of the existing embankments are about 2.5 m and 3.5 m high, based on the approximate ground surface elevations of the toe of slope boreholes at the north and south approaches, respectively, and are inclinded at profiles of about 2.5H: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 INVESTIGATION PROCEDURE

The field work for the subsurface investigation was carried out from July 23 to 26 and on July 30, 2017, during which time a total of four boreholes (FR-1 to FR-4) were advanced at the locations shown on Drawing 1. Boreholes FR-1 and FR-3 were advanced through the existing embankments immediately behind the existing abutments. Boreholes FR-2 and FR-4 were advanced at the east toes of the north and south approach embankments, respectively.

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. Boreholes FR-1 and FR-3 were advanced using 108 mm inside diameter hollow-stem augers and Boreholes FR-2 and FR-4 employed the use of NW casing and wash boring techniques. 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 slopes were backfilled with bentonite pellets and soil cuttings upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The fieldwork was supervised 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 samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury Geotechnical Laboratory where the samples underwent further visual examination and laboratory testing. Index and classification testing consisting of water content, grain size distribution and Atterberg limits was carried out on selected samples. The geotechnical laboratory testing was performed in accordance with MTO LS standards.

Select soil samples were obtained on July 29 and 30, 2017, from Boreholes FR-1 and FR-3 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.

The as-drilled borehole locations and ground surface elevations at the boreholes were measured and surveyed by a member of our technical staff, referenced to the highway centerline and existing bridge structure and converted to northings/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 survey drawing [Feature_B652TWE2 (Floodwood River).dwg] provided by AECOM on September 26, 2017. 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	Location (MTM NAD 83, Zone12)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude)	Easting (Longitude)		
FR-1	5483918.4 (49.4910509)	354607.3 (-80.3124551)	284.7	6.7
FR-2	5483911.3 (49.4909861)	354618.5 (-80.3123014)	282.0	37.2
FR-3	5483875.4 (49.4906658)	354588.5 (-80.31272)	285.0	6.7
FR-4	5483872.0 (49.4906341)	354601.8 (-80.3125369)	281.2	21.8

4.0 SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain (NOEGTS)[1] mapping, the Floodwood River Bridge site is located within an esker complex, crevasse filling plain deposit consisting primarily of clay till bordered by a clay till ground moraine deposit immediately east of the site.

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 bordered by mafic to intermediate metavolcanic rocks comprising of basaltic and andesitic flows, tuffs and breccias, chert, iron formation, minor metasedimentary and intrusive rocks, related migamites.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory testing carried out on selected soil samples, are presented on the borehole records in Appendix A and the laboratory test sheets in Appendix B. The results of the in situ tests (i.e., SPT 'N'-values) as presented on the borehole records and described in Section 4 are uncorrected. 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 (GEOCRES 42H-19), prior to construction of the existing embankments and bridge, the subsurface soil conditions encountered at this site are described as generally consisting of a 1.7 m to 6.9 m thick deposit of slightly plastic, soft to firm silty clay to silt underlain by deposits of compact to very dense silty sand and/or compact to very dense granular till. The subsoil conditions encountered during the current borehole investigation consist of granular embankment fill overlying deposits of compact silt and/or loose to very dense sandy gravel to gravelly silty sand to sand. 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)	N Values (blows/0.3 m) Consistency or Relative Density	Laboratory Testing
Asphalt	FR-1, FR-3	0.1	285.0 - 284.7	n/a	n/a
Topsoil	FR-2 and FR-4	0.1 – 0.6	282.0 - 281.2	2 **Very loose**	n/a
		0.5 – 5.5	284.9 - 281.9	N = 1 – 19	w = 2% – 21%

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

[2] Ontario 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)	N Values (blows/0.3 m)	Consistency or Relative Density	Laboratory Testing
(FILL) Gravelly Sand to Sand trace to some gravel, trace to some silt; brown; moist to wet	FR-1 to FR-3				**Very loose to compact**	4 – M (Fig. B1)
PEAT (Amorphous); trace sand; black; wet	FR-2	0.9	281.4	N = 5	**Loose**	w = 52 %
Silt, trace sand, trace to some clay, silty clay laminations; grey; wet	FR-1 and FR-2	>0.9 – 7.0	279.1 - 280.5	N = 11 – 20	**Compact**	w = 21% - 24% 3 - MH (Fig. B2) 2 – AL (NP)
Sandy Gravel to Gravelly Silty Sand to Sand[1,2], trace to some silt, trace clay; grey; wet	FR-2 to FR-4	>1.1 – >28.7 (not fully penetrated)	280.6 - 273.5	N = 7 – 197	**Loose to Very Dense**	w = 8% – 15% 1 - M (Fig. B3) 10 MH (Fig. B3)

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
NP = Non-Plastic test result

Notes:

1. Silt and Sand and Sandy Silt layers were noted within the Sandy Gravel to Gravelly Silty Sand to Sand deposit as noted on the Records of Boreholes.
2. A 500 mm diameter boulder was encountered in Borehole FR-2 at 22.3 m depth (Elevation 259.7 m).

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 approximately 5.2 m below the existing structure grade, corresponding to Elevation 279.6 m in August 2017. Groundwater and river water levels in the area are subject to seasonal fluctuations and variations due to precipitation events.

Borehole	Depth to Unstabilized Groundwater Level (m)	Approximate Groundwater Elevation (m)
FR-1	dry	n/a
FR-2	0.5	281.5
FR-3	dry	n/a
FR-4	0.2	281.0

Boreholes FR-2 and FR-4 were advanced using NW casing and wash boring techniques. As such, the water levels may not be representative of stabilized groundwater conditions.

5.0 CLOSURE

The field drilling program was supervised by Mr. Mathew Riopelle. This Foundation Investigation Report was prepared by Ms. Aronne-Kay De Souza, EIT, 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.

Report Signature Page

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
FLOODWOOD RIVER BRIDGE REPLACEMENT, SITE NO.39E-203
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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 Floodwood River Bridge (Site 39E-203) 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 the 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 Floodwood River Bridge, which was constructed in 1984, consists of a single-lane, three-span TMB bridge structure approximately 42.7 m long by 5.7 m wide. Based on the available GEOCRES information and the structure drawing dated April 1981, we understand that existing abutments are supported on timber cribs and the existing piers are supported by steel HP 310x79 piles driven into the very dense sand/gravel deposit. The front slopes of the existing approach embankments are about 5 m to 6 m high relative to the river bottom and inclined at profiles ranging from 1.3 horizontal to one vertical (1.3H:1V) to 2H:1V. The side slopes of the existing embankments are about 2.5 m and 3.5 m high, based on the approximate ground surface elevations of the toe of slope boreholes at the north and south approaches, respectively, and are inclinded at profiles of about 2.5H:1V. We further understand that prior to the original embankment construction, the surficial organics were sub-excavated and replaced with granular fill.

Based on the General Arrangement (GA) drawing provided by AECOM on December 20, 2017, (dated December 2017) we understand that the proposed replacement structure is to consist of a two lane, single-span TMB constructed on the same alignment as the existing bridge. The replacement bridge will be 48.8 m long by 7.4 m wide with new abutments located about 6 m back from (or behind) the existing abutments. 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 Floodwood 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 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 at 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.

- **Driven Steel H-piles:** Driven steel H-piles terminating in the very dense portion of the sand/gravel deposit are feasible for support of the abutments and would be preferred if higher loads are required that 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 (i.e., cobbles and boulders) are present in overburden soil deposits; however, the cost premium for this type of foundation may not be warranted for a TMB replacement structure and are not discussed further in this report.

- **Drilled shafts/caissons (large diameter):** Drilled shafts (caissons) terminating in the very dense sand/gravel deposit are also considered to be feasible for a deep foundation option at this site. 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. Shallow foundations may initially be perceived to be more economical than deep (pile) foundations, however, 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 frost cover (relative to the lowest surrounding grade) as further discussed in Section 6.4.4.

6.4.2 Geotechnical Resistance

Strip or spread footings placed within the existing embankment fill founded at about Elevation 282.1 m (approximately 2.6 m depth), could 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)
Abutment footings on compact gravelly sand to sand embankment 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., behind) the crest of the front slope.

The factored geotechnical resistances and corresponding settlements 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 would have to be re-evaluated and modified as necessary during Detail Design once the footing size(s) and locations and approach embankment geometry has been finalized. Further, the stability of the front slopes under the additional loading from the footings would have to 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 preliminary design, 25 mm of rigid polystyrene 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 depth(s) 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 into the very dense sand/gravel deposit 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 deposit(s) 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 Capacity

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 (Boreholes)	Pile Size	Elevation of Underside of Pile Cap[1] (m)	Pile Tip Elevation (m)	Length of Pile from Underside of Pile Cap (m)	Factored Geotechnical Axial Resistance at ULS[2]	Geotechnical Reaction at SLS for 25 mm of Settlement[3]
North Abutment (FR-2)	HP 310x110	282.2	250	32.2	1,200 kN	N/A
	HP 360x132				1,400 kN	
South Abutment (FR-4)	HP 310x110	282.1	264	18.1	1,100 kN	N/A
	HP 360x132				1,300 kN	

(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 indicated 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 geotechnical reaction at SLS for 25 mm of settlement will be greater than or equal to the factored geotechnical axial resistance at ULS and therefore, the SLS condition does not apply.

At the north abutment, a 500 mm diameter boulder was encountered at 22.3 m depth in Borehole FR-2 in the sand/gravel deposit. It is anticipated that both HP310x110 and HP360x132 piles could potentially hang up on boulders of this size, if present below the proposed abutments. Given the potential for cobbles/boulders within the 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 additional subsurface investigations.

The preliminary factored geotechnical resistances 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.

6.5.2 Downdrag

Downdrag loads are currently not anticipated at the abutments because no grade raises are proposed as part of the bridge replacement and since (based on the current investigation) the organic deposits appears to have been fully sub-excavated prior to construction of the existing approach embankments.

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 very dense portion of the sand/gravel 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 2014 CHBDC Commentary (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction, k_h, (kPa/m) is based on the equation 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 = constant of horizontal subgrade reaction (kPa/m), as given below;
z = depth (m)
B = pile diameter or width (m)

The following values of n_h may be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) for structural analysis for a single vertical pile.

Foundation Element (Relevant Boreholes)	**Soil Unit**	**Elevation (m)**	n_h **(kPa/m)**
North Abutment (FR-1 & FR-2)	Sand Fill (very loose to compact) (above the water table)	282.2 to 281.5	6,600
	Sand Fill (very loose to compact) (below the water table)	281.5 to 279.1	4,400
	Silt (compact)	279.1 to 273.5	2,800
	Sandy Gravel to Gravelly Silty Sand to Sand (loose to very dense)	273.5 to 250.0	11,000
South Abutment (FR-3 & FR-4)	Sand Fill (very loose to compact) (above the water table)	282.1 to 281.0	6,600
	Sand Fill (very loose to compact) (below the water table)	281.0 to 279.4	4,400
	Sandy Gravel to Gravelly Silty Sand to Sand (loose to very dense)	279.4 to 264.0	11,000

It is recommended that both the structural and geotechnical resistances of the piles 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, R
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 provided in the above summary. 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, if carried out, could potentially provide a more favourable Site Class C designation. Site Classes A and B, however, would not be considered appropriate for this site.

Based on the information obtained from the NRCan (2015) Hazard Calculator for this site located at latitude 49.4908° and longitude -80.3131°, 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)	0.120
Sa (1.0) (g)	0.042

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, 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 (only if multiple span option is being considered).

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) at about Elevation 284.8 m; however, an approximately 10 m widening of the existing approach embankments will be required on the west 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 and/or mixed organic soil) be removed from 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. All excavation and backfilling should be carried out simultaneously in accordance with OPSS.PROV 209 (Embankments over Swamps).

Sub-excavation is anticipated to be required to up to approximately 1.5 m below ground surface to remove the organic soils on the west side of the embankment based on the boreholes advanced at the east toes of embankment slopes. All 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 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 OPSD 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 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 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 GEOCRES report and the results of the current investigation, we understand that the peat/organics soils were previously sub-excavated prior to construction of the existing highway embankments. The analysis discussed below assumes that the existing organics at the toe of the embankment slope are sub-excavated and replaced prior to the new embankment widenings.

6.7.3.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program 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 failure. For the purpose of the stability analysis, the target minimum 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 FoS of 1.5 for the design of the final embankment configuration as per Table 6.2 of CHBDC (2014) for the effective stress (long-term, drained) conditions.

The stability analysis carried out for the preliminary design includes the evaluation of the existing front slope of the north approach as well as the proposed 10 m widening (west side) of the north approach, which relates to the highest embankment widening when compared to the south approach. The stability analyses were completed based on the subsurface conditions as encountered in Boreholes FR-1 and FR-2 and the geometries provide in the GA drawing and cross-sections provided by AECOM.

6.7.3.2 Parameter Selection

For the new granular fill, the existing granular fill, and the non-cohesive native soil deposits, effective stress parameters were employed in the analysis assuming drained conditions, and the parameters were estimated from empirical correlations using the 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.

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

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (ϕ’)	Cohesion (kPa)
New Granular Fill (i.e., Granular A or B Type I or II)	21	35	-
Existing Granular Embankment Fill	20	32	-
Peat	12	27	1
Silt (Loose to Compact)	19	28	-
Sandy Gravel to Gravelly Silty Sand to Sand (Compact)	20	32	-

6.7.3.3 Results of Analysis

The stability analyses indicates that the approximately 5 m to 6 m high existing north front slope (height relative to river bottom) inclined at approximately 2H:1V has a FoS greater than 1.5 against global instability in the long-term (drained) conditions as shown on Figure 1. Similarly, the approximately 3 m high widened northwest approach embankment also meets/exceeds the minimum required FoS for long-term conditions (i.e., FoS>1.5) as shown on Figure 2. This preliminary assessment of the stability of the 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 expected settlements due to the embankment widenings, analyses were carried out on the critical sections of the proposed approach embankments using the commercially available computer software *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.

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 widened 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 stiffness (moduli) and unit weights employed for 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).

Soil Deposit	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)
Existing Granular Embankment Fill (Very Loose to Compact)	20	10
Silt (Compact)	19	15
Sandy Gravel to Gravelly Silty Sand to Sand (Compact to Very Dense)	20	50

6.7.4.4 Results of Analysis

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

Critical Section	Relevant Borehole	Settlement during Construction		
		Existing Hwy 652 Centerline (mm)	Existing Hwy 652 Edge of Shoulder (west side) (mm)	Proposed West Crest of Laydown/Launch Area (i.e., near Existing West Toe of Slope) (mm)
North Approach	FR-1 and FR-2	5	10	25

We anticipate that the above noted settlements will be immediate and will occur primarily during embankment construction. 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.

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 a 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 constructing the 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 the temporary shoring system, installation of sheet-piles could be impeded by the presence of cobbles/boulders within the gravelly sand deposit and/or by the very dense zones within the gravelly sand 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:

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 (Loose to Compact)	20	32	0.31	0.47	3.25
Silt (Compact)	19	28	0.36	0.53	2.77
Sandy Gravel to Gravelly Silty Sand to Sand (Loose to Very Dense)	20	32	0.31	0.47	3.25

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., within the sheet pile cofferdam and/or 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 out in dry conditions, within the existing embankment fill since the underside of foundation (at Elevation 282.2 m) is well above the adjacent river level (at Elevation 279.6 m) and is also above the groundwater encountered in the boreholes (at Elevation 281.5 m). Perched groundwater may be present within the embankment fill depending on the time of year that the construction is carried out, however, if encountered, dewatering could be handled in the form of pumping from properly filtered temporary sumps installed below the base of the excavation.

Excavations up to about 1.5 m below the existing ground surface will be required at the toe(s) of the embankment slope(s) for removal and replacement of the organics material 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 6.7.2 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 are expected to contain coarse gravel, cobbles and boulders, which could affect the installation of temporary support systems (including cofferdams) or deep foundations. Cobbles/boulders were noted in the previous foundation investigation at this site along with a 500 mm boulder encountered in Borehole FR-2. Additional records on the frequency of encountering cobbles and/or boulders are recommended during 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 cobbles and boulders for the following reasons:

- the age of the existing structure
- the existing piers may be 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) as part of the 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 testing on samples of soil taken from the abutment boreholes at about the anticipated foundation (i.e., pile cap or footing) 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 pile caps (or footings) and abutments. Further, the resistivity results indicate that that the granular fill has a low to 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 when 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 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 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 (particularly on the west side of the 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 assessment and potential to shorten the pile lengths (particularly at the north abutment) and provide lower axial geotechnical resistance for the pile foundations (if required)
- 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.

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ASTM International

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

Commercial Software

GeoStudio 2007 (Version 7.23) by Geo-Slope International Ltd.

Settle 3D (Version 4.0) by Rocscience

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 Specifications for Grading.
OPSS.PROV 209	Construction Specifications for Embankments Over Swamps and Compressible Soils
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 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**

Table 1: Evaluation of Abutment Foundation Alternatives

Foundation Type	Rank	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Driven Steel H-Piles	1	■ 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 soils below the widened footprint.	■ Potential for refusal or "hanging up" of piles on cobbles and boulders within the gravelly sand deposit, but likely easier to advance than pipe piles. ■ Vibration monitoring recommended during pile driving adjacent to the existing structure.	■ 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 and settlement/lateral movement monitoring recommended to identify and control risks.
Spread Footings	2	■ Straightforward construction ■ Adequate geotechnical axial resistances available given size of structure. ■ Use of pre-cast footing(s) could accelerate construction.	■ Potential for differential settlement across existing/widened embankment. ■ Footings have higher risk of being affected by adjacent excavations for sub-excavation of organic materials 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. ■ 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
Drilled Steel Casings (Small Diameter)	NR	■ Higher axial resistances compared to steel H-piles. ■ Easier to penetrate obstructions 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 below the widened footprint.	■ 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.	■ Relative costs higher than for steel H-piles	■ Lower risk of difficulties during installation through very dense gravelly sand deposits containing cobbles/boulders. ■ Potential for disturbance at the base of the DSC (terminated in gravelly sand, not bedrock) could affect the geotechnical resistance(s).
Drilled Shafts/Caissons (Large Diameter)	NR	■ Higher axial resistances compared to steel H-piles and smaller diameter DSCs.	■ 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.	■ Relative costs much higher than for steel H-piles.	■ 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: Floodwood River Bridge
South approach, West side of bridge, looking North (July 2017)

Photograph 2: Floodwood River Bridge
South approach embankment, East side of bridge, looking South (July 2017)

Photograph 3: Floodwood River Bridge
Borehole FR-4, South approach, East side of bridge, looking North (July 2017)

Photograph 4: Floodwood River Bridge
West elevation looking South-East (OSIM Report June 2015)

MINISTRY OF TRANSPORTATION, ONTARIO

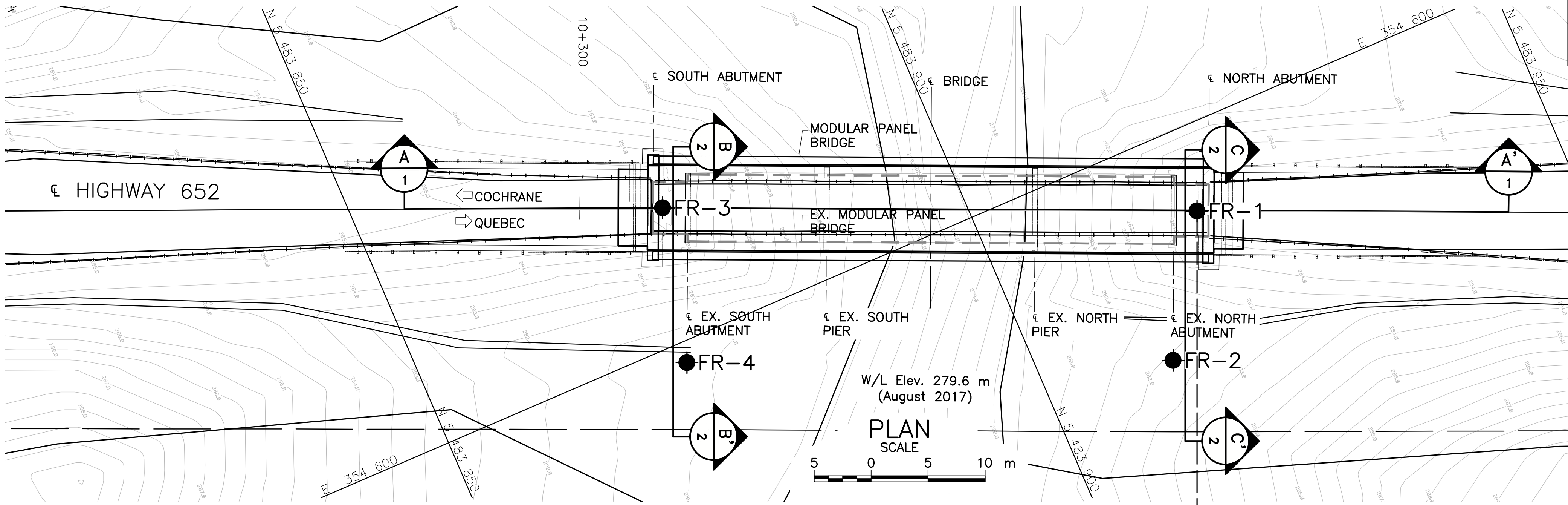
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5416–15–00

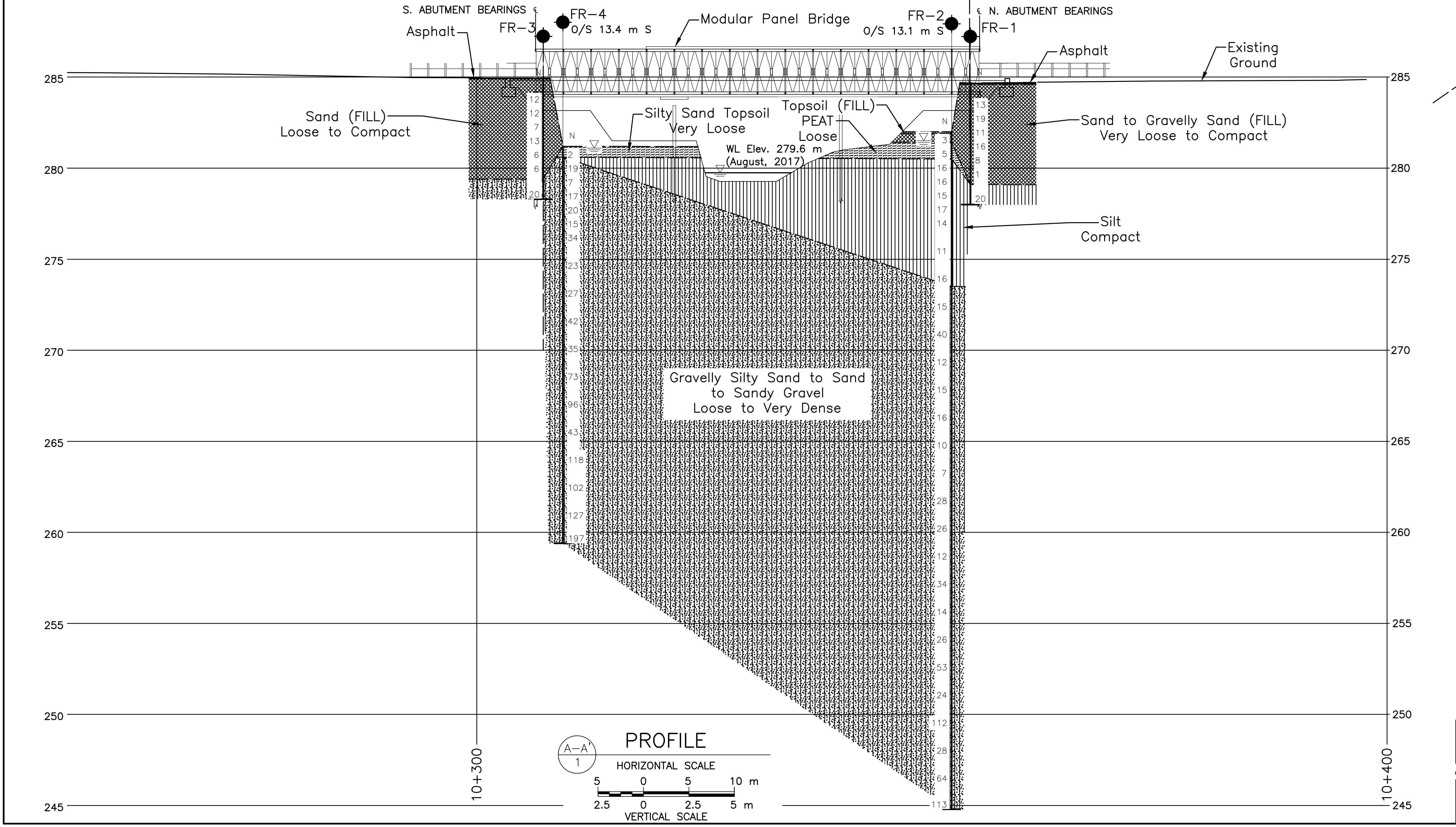
HIGHWAY 652
FLOODWOOD RIVER BRIDGE
LAT. 49.490886, LONG. –80.312558
BORHOLE LOCATIONS AND SOIL STRATA

SHEET

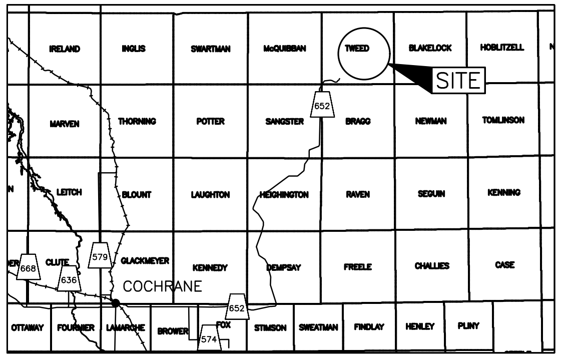
Golder Associates



PLAN
SCALE
5 0 5 10 m



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



KEY PLAN
SCALE
15 0 15 30 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 (NAD 83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
FR–1	284.7	5483918.4	354607.3
FR–2	282.0	5483911.3	354618.5
FR–3	285.0	5483875.4	354588.5
FR–4	281.2	5483872.0	354601.8

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. Floodwood.dwg, received SEPT 26, 2017 and Features_B652TWE2 (Floodwood River).dwg, received OCT 26, 2017. General Arrangement provided by AECOM, drawing file no. 60547656–P30.dwg, received DEC 20, 2017.

D.A. MULDOWNEY
100164716
APR 10, 2018

J.P. DITTRICH
April 10, 2018

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 42H–77

HWY. 652		PROJECT NO. 1651997	DIST. .
SUBM'D. AC	CHKD. .	DATE: 4/10/2018	SITE: 39E–203
DRAWN: TB	CHKD. AB	APPD. JPD	DWG. 1

PLOT DATE: April 10, 2018
FILENAME: \\golder.gds\gal\Sudbury\CAD–GIS\SIM\Clients\MTO\1651997 AECOM_5015–E–0045_NE Retainer\99_PROJ\1651997_Floodwood\40_PROD\1651997–0001–BG–0001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: April 10, 2018
FILENAME: \\golder.gds\gal\Sudbury\CAD-GIS\SIM\Clients\MTO\1651997 AECOM_5015-E-0045_NE Retainer\99_PROJ\1651997_Floodwood\40_PROD\1651997-0001-BG-0001.dwg

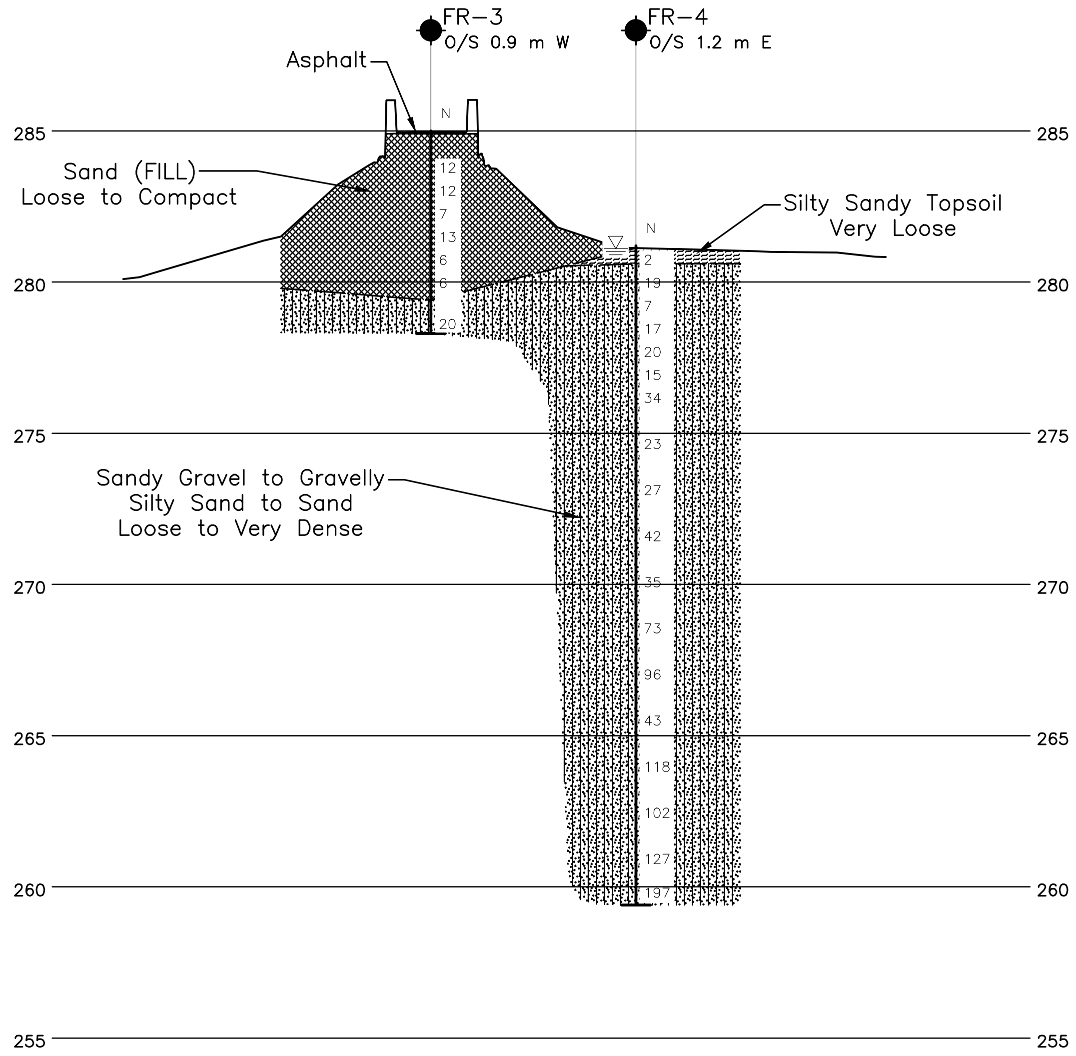
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 5416-15-00

HIGHWAY 652
FLOODWOOD RIVER BRIDGE
LAT. 49.490886, LONG. -80.312558
SOIL STRATA

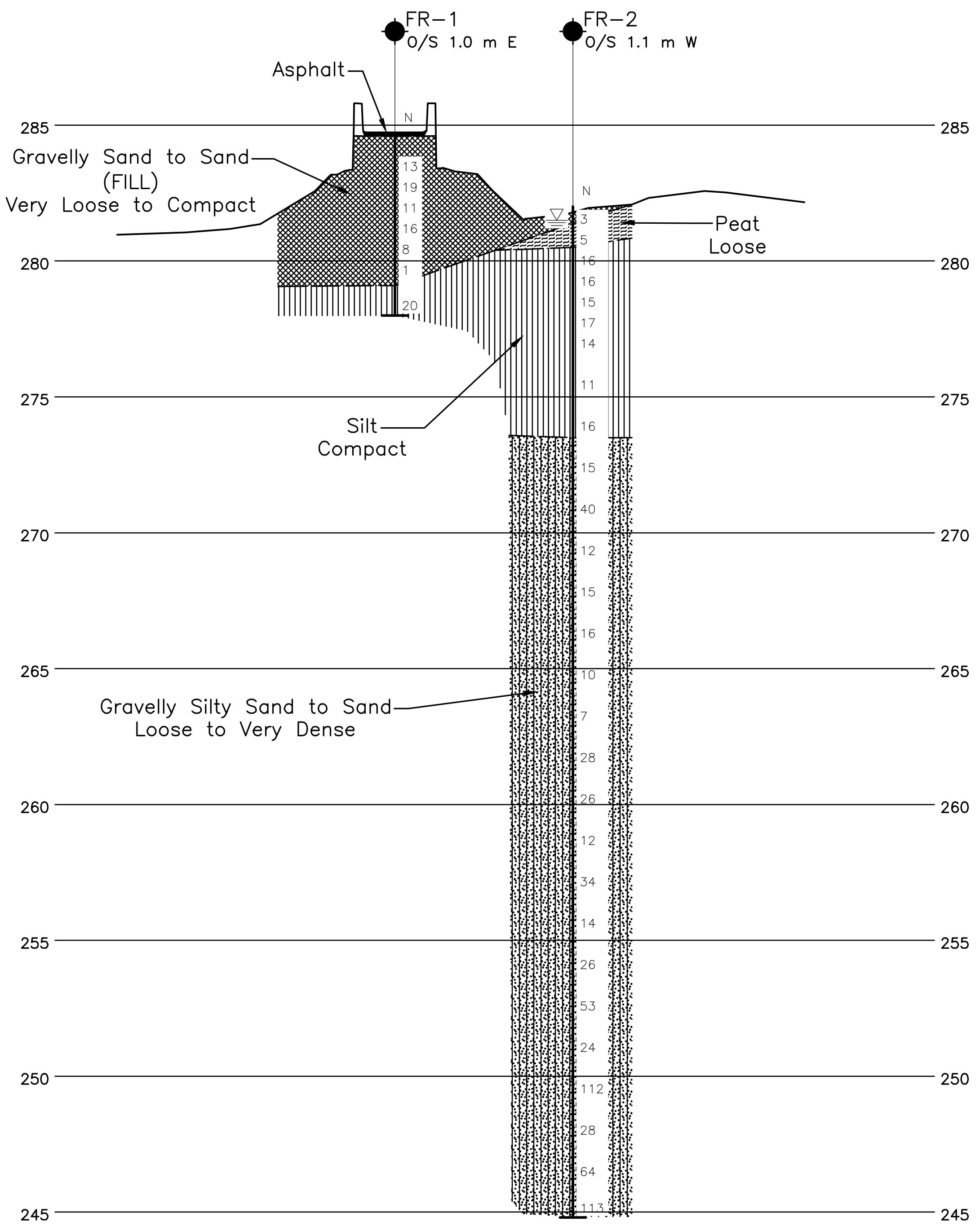
SHEET
.

Golder Associates



B-B' 1 CROSS-SECTION
STA 10+308

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



C-C' 1 CROSS-SECTION
STA 10+353

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

LEGEND

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

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 12)

No.	ELEVATION	NORTHING	EASTING
FR-1	284.7	5483918.4	354607.3
FR-2	282.0	5483911.3	354618.5
FR-3	285.0	5483875.4	354588.5
FR-4	281.2	5483872.0	354601.8

NOTES

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REFERENCE

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D.A. MULDOWNEY
100164716
APR 10, 2018

J.P. DITTRICH
April 10, 2018

NO.	DATE	BY	REVISION
.	.	.	.

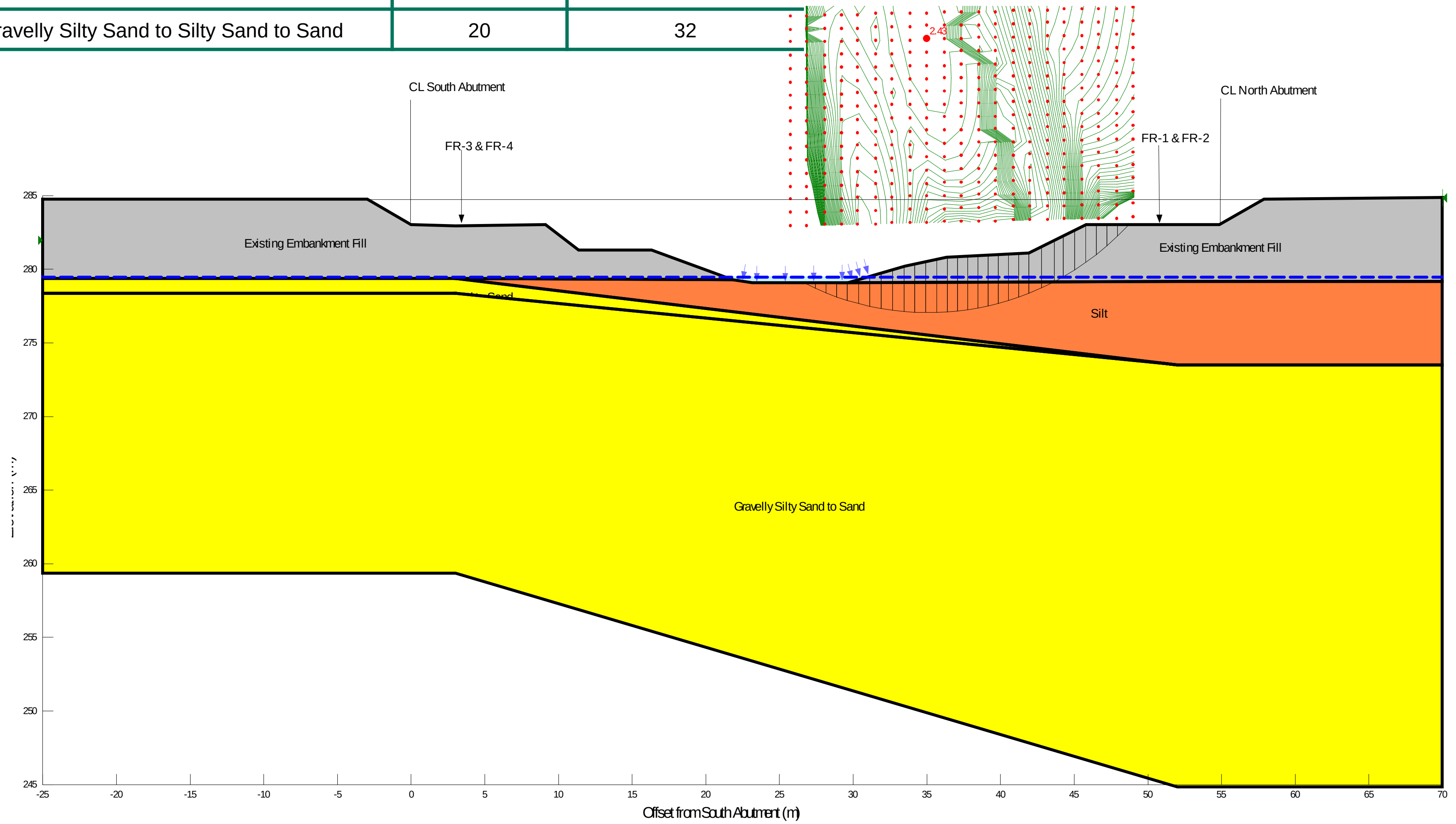
Geocres No. 42H-77

HWY. 652		PROJECT NO. 1651997	DIST. .
SUBM'D. AC	CHKD. .	DATE: 4/10/2018	SITE: 39E-203
DRAWN: TB	CHKD. AB	APPD. JPD	DWG. 2

Stability Analysis
North Approach Front Slope
Long-Term (Drained) Condition

Figure 1

Material Name	Unit Weight (kN/m^3)	Friction Angle (degrees)
Existing Granular Embankment Fill	20	32
Silt	19	28
Gravelly Silty Sand to Silty Sand to Sand	20	32



Date: December, 2017

Project No: 1651997 – Floodwood River Bridge

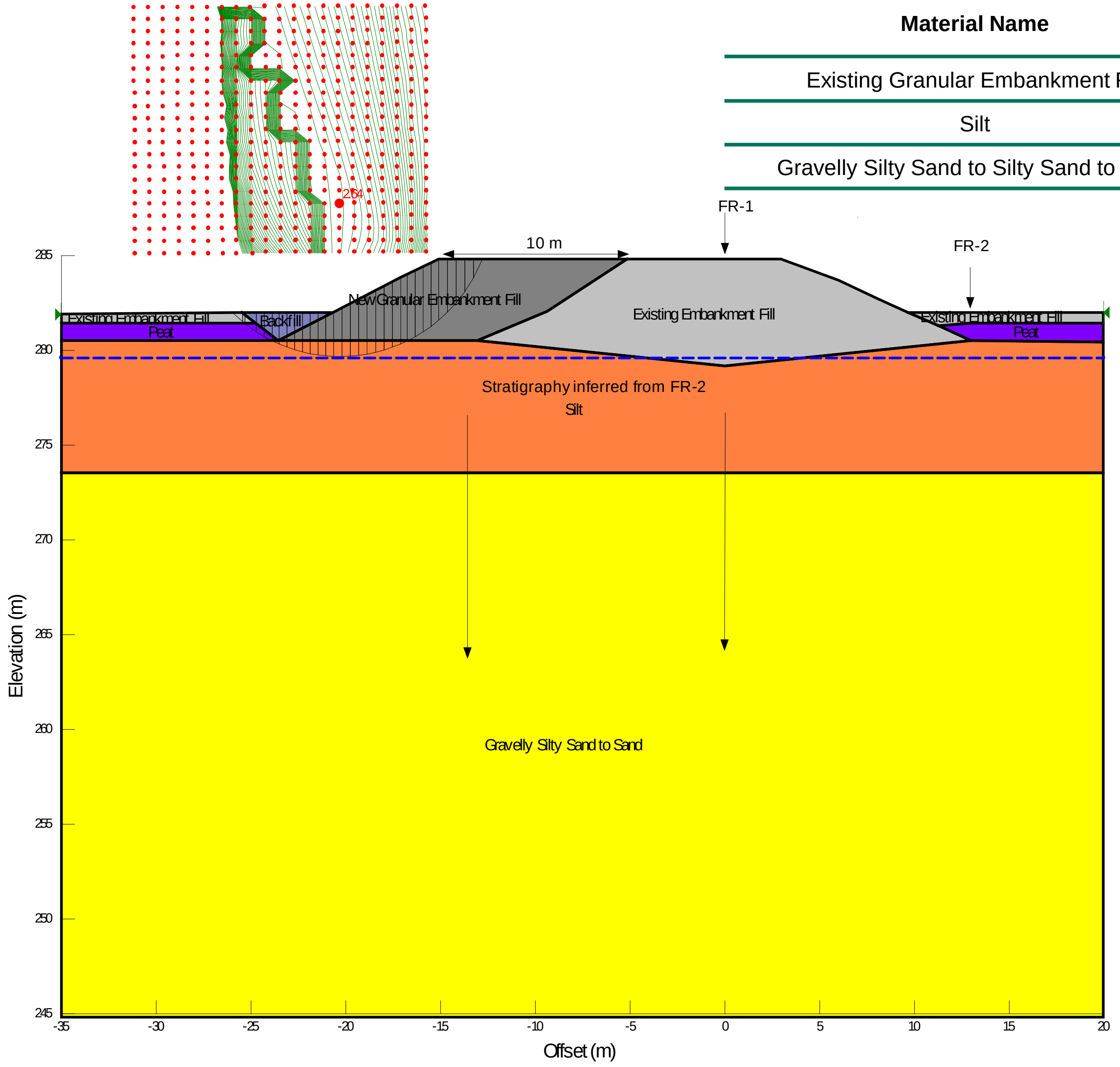
Analysis By: AC

Reviewed By: AB

Stability Analysis
North Approach West Embankment Widening
Long-Term (Drained) Condition

Figure 2

Material Name	Unit Weight (kN/m^3)	Friction Angle (degrees)
Existing Granular Embankment Fill	20	32
Silt	19	28
Gravelly Silty Sand to Silty Sand to Sand	20	32



Date: December, 2017

Project No: 1651997 – Floodwood 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
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 = $\tan\delta$
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

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 (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, 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

Density Index Relative Density	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	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	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

RECORD OF BOREHOLE No FR-1

1 OF 1 **METRIC**

PROJECT 16519971651997-WO5
G.W.P. 5416-15-00
LOCATION N 5483918.4; E 354607.3 NAD83 MTM ZONE 12 (LAT. 49.4910509; LONG. -80.3124551)
ORIGINATED BY MR
DIST
HWY 652
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing
COMPILED BY AD
DATUM GEODETIC
DATE July 30, 2017
CHECKED BY AB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
284.7	GROUND SURFACE									
0.0	ASPHALT (100 mm)									
0.1	Gravelly sand (FILL) Brown Moist				284					
284.1 0.6	Sand, trace to some gravel, trace to some silt (FILL) Very loose to compact Brown Moist to wet	1	SS	13						
		2	SS	19	283					
		3	SS	11	282		15	77	(8)	
		4	SS	16	281					
		5	SS	8						
		6	SS	1	280		1	98	(1)	
279.1 5.6	SILT, trace clay Compact Grey Wet				279					
		7	SS	20		NP	0	0	95	5
278.0 6.7	END OF BOREHOLE Note: 1. Borehole dry upon completion of drilling.				278					

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

RECORD OF BOREHOLE No FR-2

1 OF 4 **METRIC**

PROJECT 16519971651997-WO5

G.W.P. 5416-15-00

LOCATION N 5483911.3; E 354618.5 NAD83 MTM ZONE 12 (LAT. 49.4909861; LONG. -80.3123014)

ORIGINATED BY MR

DIST

HWY 652

BOREHOLE TYPE NW Casing and Wash Boring

COMPILED BY AD

DATUM GEODETIC

DATE July 23 to 25, 2017

CHECKED BY AB

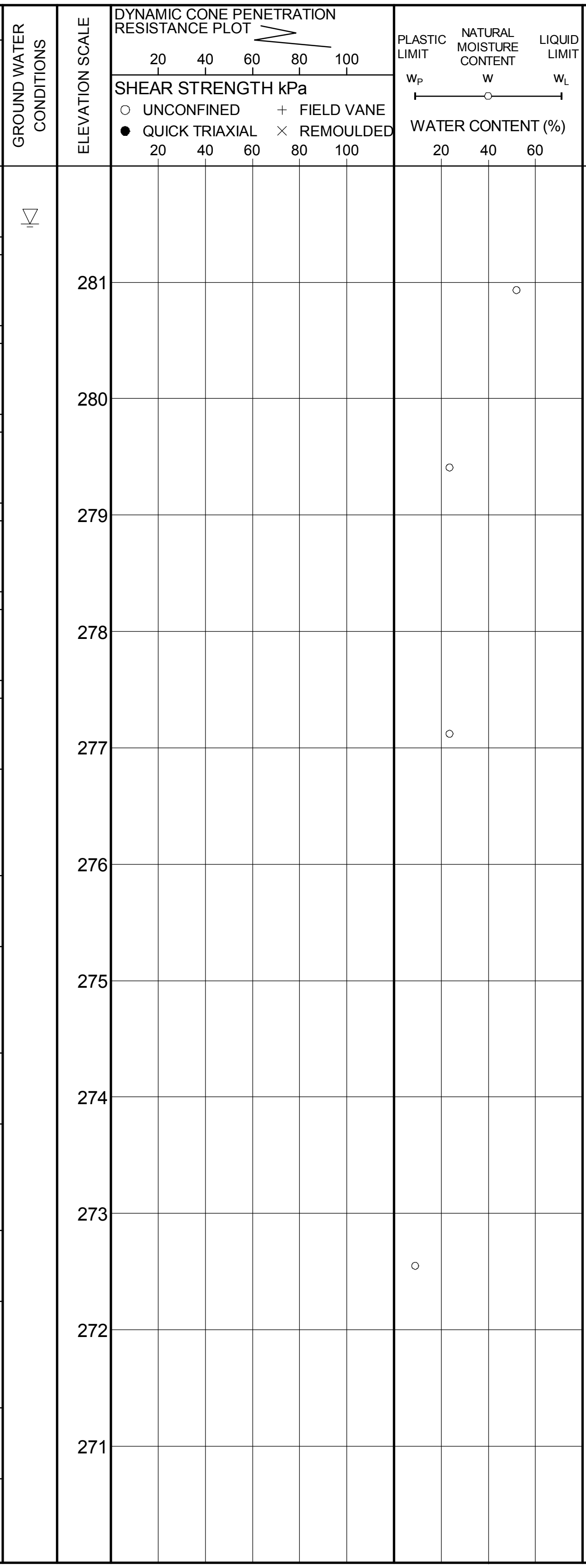
SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
282.0	GROUND SURFACE
0.0	TOPSOIL
0.1	Sand, trace gravel (FILL) Very loose Brown Moist
281.4 / 0.6	Amorphous PEAT, trace sand Loose Black Wet
280.5 / 1.5	SILT, trace sand, trace to some clay, silty clay laminations Compact Grey Wet
273.5 / 8.5	Gravelly SILTY SAND to SAND, some gravel, trace to some silt, trace clay Loose to very dense Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	SS	3					
2	SS	5					
3	SS	16					
4	SS	16	NP	0	0	92	8
5	SS	15					
6	SS	17					
7	SS	14		0	3	89	8
8	SS	11					
9	SS	16					
10	SS	15		23	62	15	0
11	SS	40					

REMARKS & GRAIN SIZE DISTRIBUTION (%)



Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No FR-2

2 OF 4 **METRIC**

PROJECT 16519971651997-WO5			
G.W.P. 5416-15-00	LOCATION N 5483911.3; E 354618.5 NAD83 MTM ZONE 12 (LAT. 49.4909861; LONG. -80.3123014)	ORIGINATED BY MR	
DIST	HWY 652	BOREHOLE TYPE NW Casing and Wash Boring	COMPILED BY AD
DATUM GEODETIC		DATE July 23 to 25, 2017	CHECKED BY AB

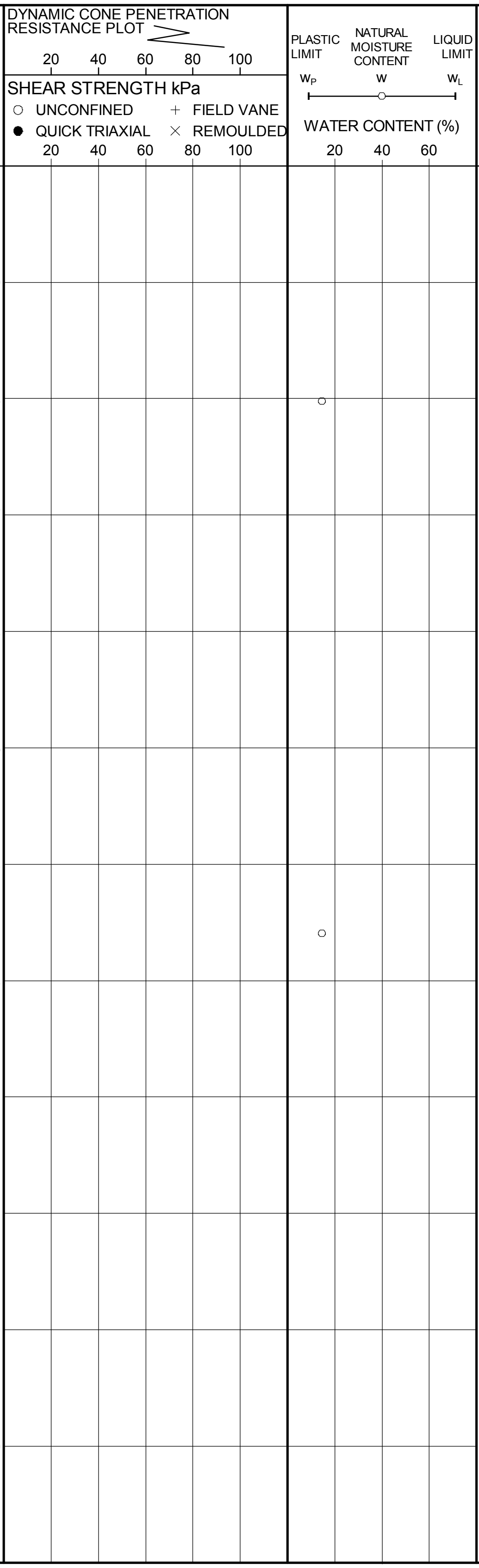
SOIL PROFILE — ELEV DEPTH — DESCRIPTION — STRAT PLOT

--- CONTINUED FROM PREVIOUS PAGE ---

Gravelly SILTY SAND to SAND, some gravel, trace to some silt, trace clay
Loose to very dense
Grey
Wet

A 500 mm boulder was encountered at 22.3 m depth.

SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
12	SS	12						
				269				
13	SS	15		268	3	89	(8)	
				267				
14	SS	16						
				266				
15	SS	10		265				
				264				
16	SS	7			22	48	28	2
				263				
17	SS	28		262				
				261				
18	SS	26						
				260				
19	SS	12		259				



UNIT WEIGHT γ kN/m³

Continued Next Page

+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 4/9/18 TB

RECORD OF BOREHOLE No FR-2

3 OF 4 **METRIC**

PROJECT 16519971651997-WO5
G.W.P. 5416-15-00
LOCATION N 5483911.3; E 354618.5 NAD83 MTM ZONE 12 (LAT. 49.4909861; LONG. -80.3123014)
ORIGINATED BY MR
DIST HWY 652
BOREHOLE TYPE NW Casing and Wash Boring
COMPILED BY AD
DATUM GEODETIC
DATE July 23 to 25, 2017
CHECKED BY AB

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---											
	Gravelly SILTY SAND to SAND, some gravel, trace to some silt, trace clay Loose to very dense Grey Wet	20	SS	34								
						257						
	Silt and sand layer at 25.9 m depth.					256						
		21	SS	14			○		12	41	46	1
						255						
		22	SS	26								
						254						
						253						
		23	SS	53								
						252						
		24	SS	24								
						251						
						250						
		25	SS	112			○		12	76	11	1
						249						
		26	SS	28								
						248						
						247						
		27	SS	64								

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

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

RECORD OF BOREHOLE No FR-2

4 OF 4 **METRIC**

PROJECT 16519971651997-WO5

G.W.P. 5416-15-00

LOCATION N 5483911.3; E 354618.5 NAD83 MTM ZONE 12 (LAT. 49.4909861; LONG. -80.3123014)

ORIGINATED BY MR

DIST

HWY 652

BOREHOLE TYPE NW Casing and Wash Boring

COMPILED BY AD

DATUM GEODETIC

DATE July 23 to 25, 2017

CHECKED BY AB

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	Gravelly SILTY SAND to SAND, some gravel, trace to some silt, trace clay Loose to very dense Grey Wet										
244.8			28	SS	113		245				
37.2	END OF BOREHOLE Note: 1. Water level at a depth of 0.5 m below ground surface (Elev. 281.5 m) upon completion of drilling.										

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+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No FR-3

1 OF 1 **METRIC**

PROJECT 16519971651997-WO5

G.W.P. 5416-15-00

DIST ___________ HWY 652

DATUM GEODETIC

LOCATION N 5483875.4; E 354588.5 NAD83 MTM ZONE 12 (LAT. 49.4906658; LONG. -80.31272)

BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers

DATE July 30, 2017

ORIGINATED BY MR

COMPILED BY AD

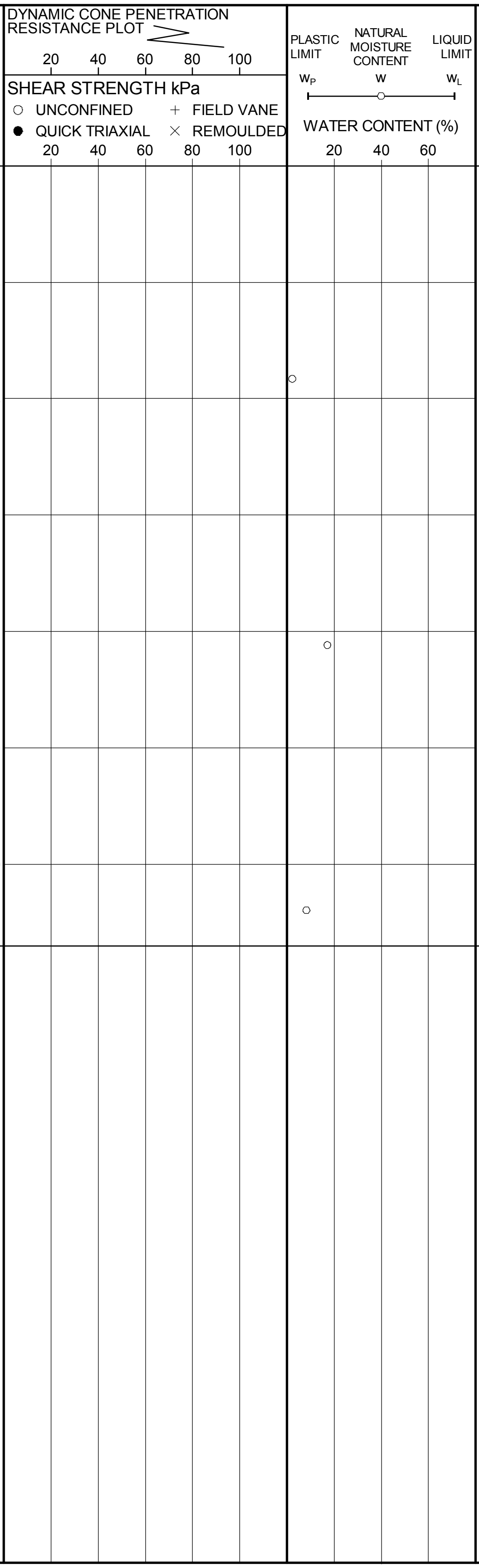
CHECKED BY AB

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
285.0	GROUND SURFACE
0.0	ASPHALT (100 mm)
0.1	Sand, trace to some gravel, trace to some silt (FILL) Loose to compact Brown Moist to wet
279.4 5.6	SAND, some silt, trace gravel, trace clay Compact Grey Wet
278.3 6.7	END OF BOREHOLE Note: 1. Borehole dry upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	12				
2	SS	12	20	73	(7)	
3	SS	7				
4	SS	13				
5	SS	6	0	83	(17)	
6	SS	6				
7	SS	20	3	79	16	2



GROUND WATER CONDITIONS

UNIT WEIGHT γ kN/m³

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

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RECORD OF BOREHOLE No FR-4

1 OF 2 **METRIC**

PROJECT 16519971651997-WO5

G.W.P. 5416-15-00

LOCATION N 5483872.0; E 354601.8 NAD83 MTM ZONE 12 (LAT. 49.4906341; LONG. -80.3125369)

ORIGINATED BY MR

DIST

HWY 652

BOREHOLE TYPE NW Casing and Wash Boring

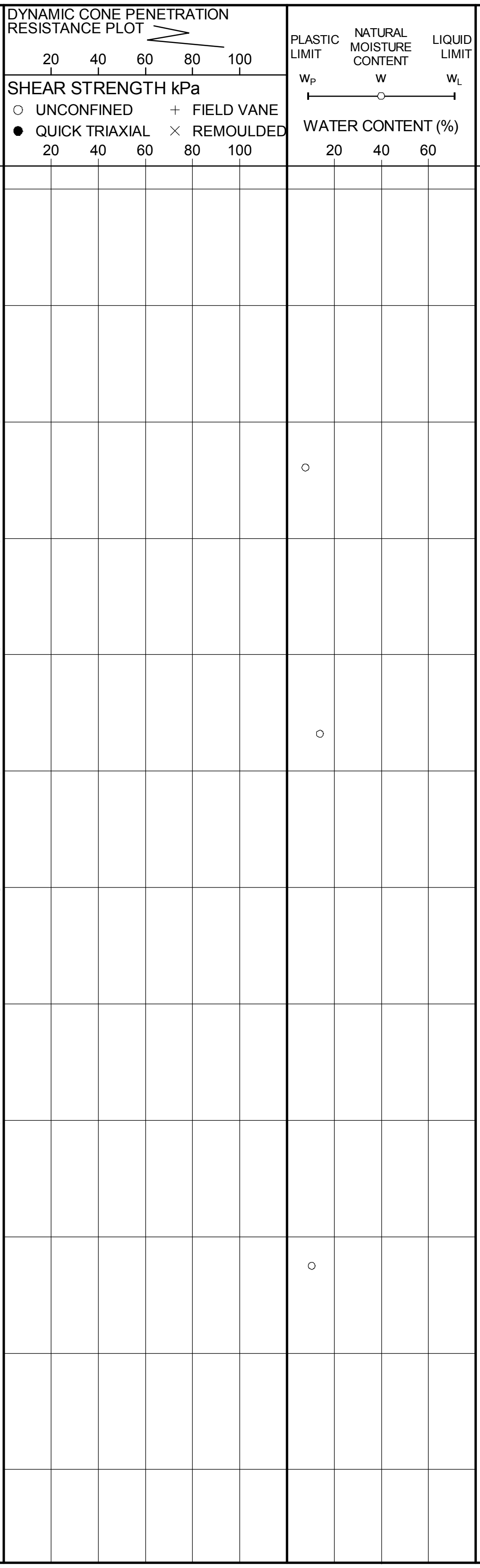
COMPILED BY AD

DATUM GEODETIC

DATE July 25 to 26, 2017

CHECKED BY AB

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
281.2	GROUND SURFACE										
0.0	Silty Sandy TOPSOIL, trace gravel Very loose Black to dark brown Wet	1	SS	2	∇	281					
280.6 0.6	Sandy GRAVEL, some silt to Gravelly Silty SAND to SAND, trace clay Loose to very dense Grey Wet	2	SS	19		280					
	No recovery in Sample 3.	3	SS	7		279					
		4	SS	17				56	30	13	1
		5	SS	20		278					
		6	SS	15		277					
	Sandy silt layer at 4.6 m depth.	7	SS	34		276		5	25	68	2
		8	SS	23		275					
		9	SS	27		274 273					
		10	SS	42		272		22	45	28	5
		11	SS	35		271 270					



REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

GOLDER

Foundation Design

PROJECT 16519971651997-WO5	**RECORD OF BOREHOLE No FR-4**	2 OF 2	**METRIC**
G.W.P. 5416-15-00	LOCATION N 5483872.0; E 354601.8 NAD83 MTM ZONE 12 (LAT. 49.4906341; LONG. -80.3125369)	ORIGINATED BY MR	
DIST HWY 652	BOREHOLE TYPE NW Casing and Wash Boring	COMPILED BY AD	
DATUM GEODETIC	DATE July 25 to 26, 2017	CHECKED BY AB	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Sandy GRAVEL, some silt to Gravelly Silty SAND to SAND, trace clay Loose to very dense Grey Wet						269				
			12	SS	73						
							268				
			13	SS	96		267		○		28 49 21 2
							266				
			14	SS	43		265				
			15	SS	118		264				
	Silt and sand layer at 18.3 m depth.						263				
			16	SS	102		262		○		1 60 38 1
			17	SS	127		261				
							260				
259.4			18	SS	197						
21.8	END OF BOREHOLE Note: 1. Water level at a depth of 0.2 m below ground surface (Elev. 281.0 m) upon completion of drilling.										

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

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APPENDIX B

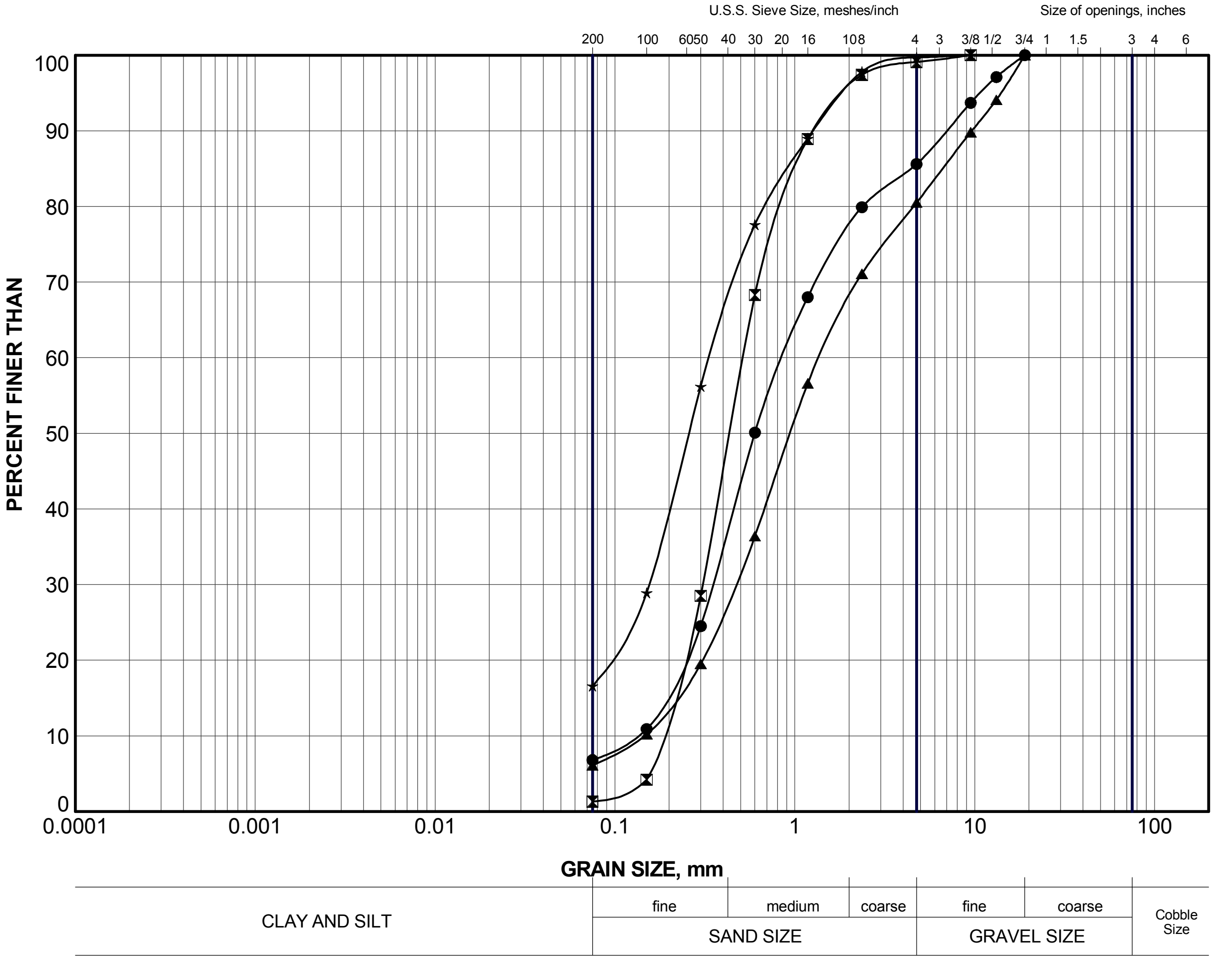
Laboratory Test Results

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

Location	***Parameter***	***Units***	***Result***
North Abutment (Borehole FR-1, Sample 5)	*Chloride (CL)*	*ug/g*	*ND*
	Sulphate (SO4)	*ug/g*	*ND*
	Conductivity (EC)	*umho/cm*	98
	Resistivity	*ohm-cm*	10,000
	pH	*n/a*	7.81
South Abutment (Borehole FR-3, Sample 6)	*Chloride (CL)*	*ug/g*	*ND*
	Sulphate (SO4)	*ug/g*	90
	Conductivity (EC)	*umho/cm*	185
	Resistivity	*ohm-cm*	5,400
	pH	*n/a*	7.70

Notes:
1. Samples from Boreholes FR-1 and FR-3 obtained on July 30, 2017, respectively 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



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	FR-1	3	282.1
⧗	FR-1	6	279.8
▲	FR-3	2	283.2
★	FR-3	5	280.9

PROJECT

HIGHWAY 652
FLOODWOOD RIVER BRIDGE

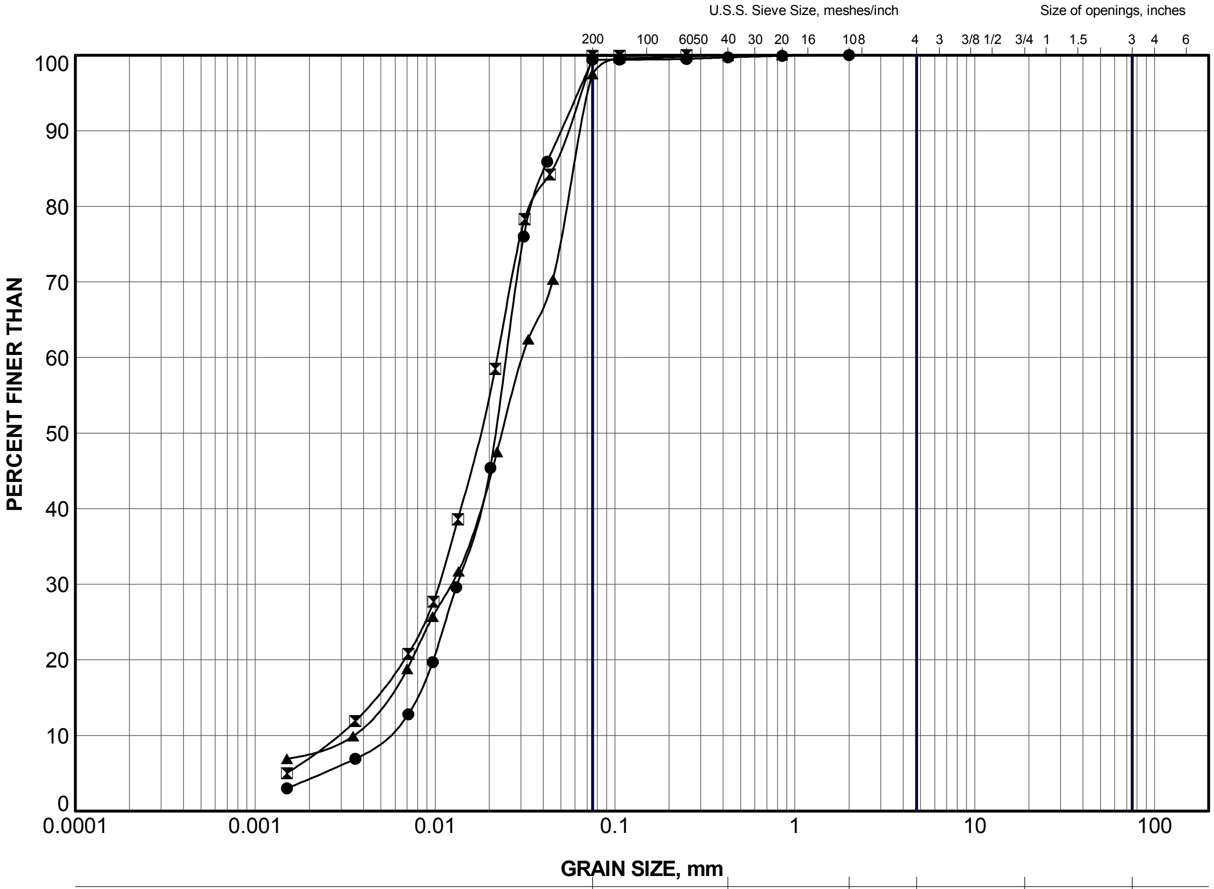
TITLE

GRAIN SIZE DISTRIBUTION
SAND (FILL)

Golder Associates
SUDBURY, ONTARIO

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

SUD-MTO GSD (2016) GLDR_LDN.GDT



CLAY AND SILT	SAND SIZE			GRAVEL SIZE		Cobble Size
	fine	medium	coarse	fine	coarse	

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	FR-1	7	278.3
⧗	FR-2	4	279.4
▲	FR-2	7	277.1

PROJECT

HIGHWAY 652
FLOODWOOD RIVER BRIDGE

TITLE

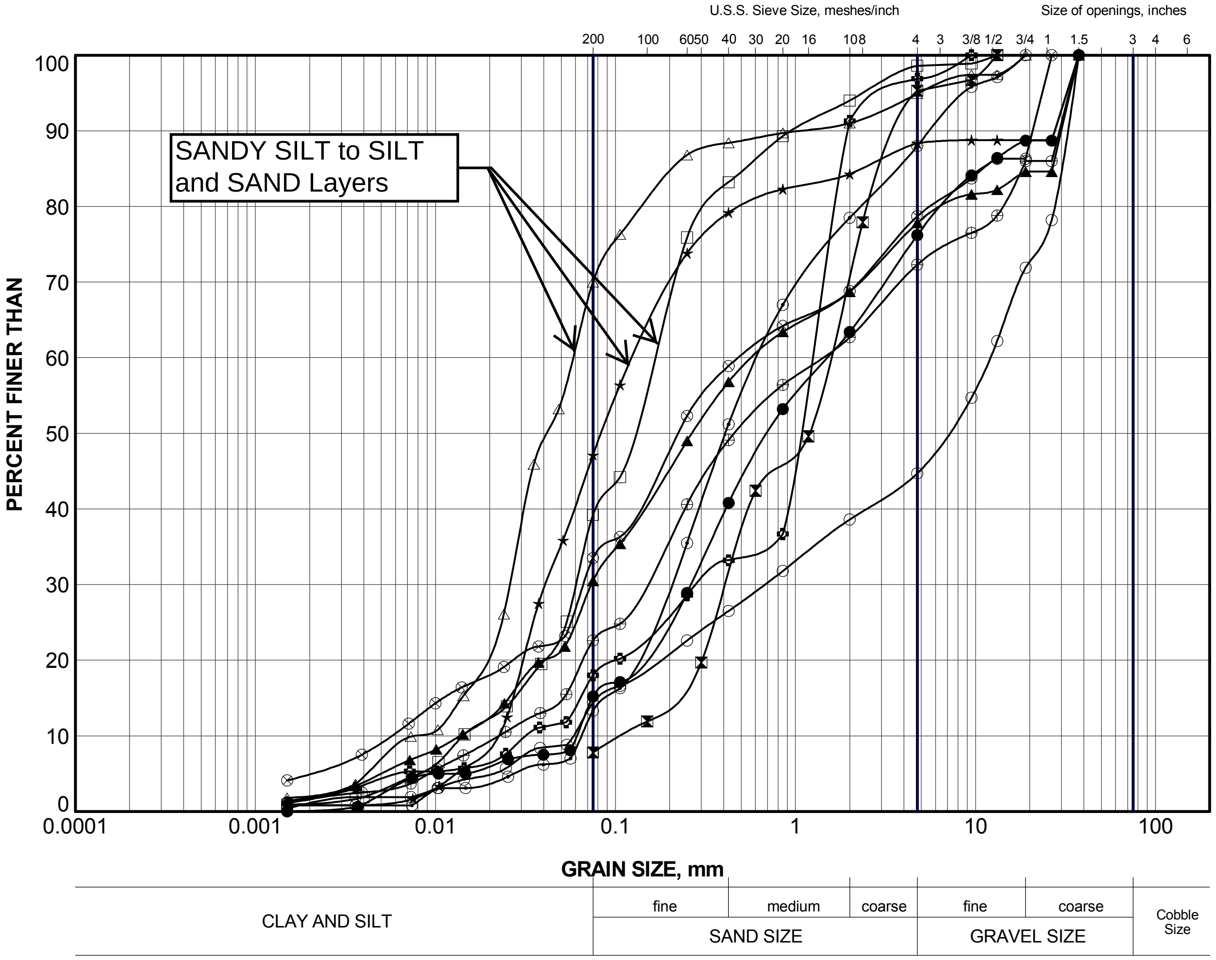
GRAIN SIZE DISTRIBUTION

SILT

Golder Associates
SUDBURY, ONTARIO

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

SUD-MTO GSD (2016) GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	FR-2	10	272.6
⧗	FR-2	13	268.0
▲	FR-2	16	263.4
★	FR-2	21	255.8
⊙	FR-2	25	249.7
✤	FR-3	7	278.6
○	FR-4	4	278.6
△	FR-4	7	276.3
⊗	FR-4	10	271.8
⊕	FR-4	13	267.2
□	FR-4	16	262.6

PROJECT

HIGHWAY 652
FLOODWOOD RIVER BRIDGE

TITLE

GRAIN SIZE DISTRIBUTION

SANDY GRAVEL to GRAVELLY SILTY SAND to SAND

Golder Associates
SUDBURY, ONTARIO

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

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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