



THURBER ENGINEERING LTD.



**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
BADESDAWA RIVER BRIDGE REPLACEMENT
NORT ROAD (FORMELY HIGHWAY 808),
DISTRICT OF KENORA, ONTARIO
G.W.P 6623-17-00, SITE NO. 41S-102**

GEOCRES No.: 52P-4

Report

to

Hatch

Date: May 3, 2017
File: 15697

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation conducted at the site of the existing bridge carrying Nort Road over Badesdawa River, also known as Mud River, in the Unsurveyed Territory, District of Kenora, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to Hatch, under the Ministry of Transportation Ontario (MTO) Agreement Number 6018-E-0018-010.

2. SITE DESCRIPTION

The existing bridge is located on Nort Road (formerly Highway 808), approximately 54.7 km north of Pickle Lake Road, in the Unsurveyed Territory, District of Kenora. At the structure site, Nort Road is a gravel surface road running generally in the southwest-northeast direction.

The existing bridge is a 33.8 m in length and 6 m in width single span, single lane steel Bailey bridge. The superstructure is supported by timber crib /gabion abutments. Badesdawa River flows from southeast to northwest at the bridge location and joins Otokwin River some distance northwest of the bridge. In the vicinity of the existing bridge, the river banks are between 4 m and 5 m in height. The land on both sides of the river is densely vegetated with trees, shrubs and grass.

In the river channel, five rows of timber piles (bents) cut-off above the river water level indicate presence of an old bridge that existed on the same alignment. Erosion of the river banks, including oversteepening of the front slopes at the abutments was evident. Loss of ground with part of timber cribs exposed was observed.

Based on published geological information, the site is located in the Beren River Subprovince of the Superior Province. The general area of the project is covered by glaciolacustrine and glaciomarine deep water deposits of silts and clays. The silts and clays are underlain by the Neo- to Mesozoic massive to foliated granodiorite and granite bedrock.

3. INVESTIGATION PROCEDURES

The field investigation program for this project was carried out between October 18 and 20, 2016. The program consisted of drilling and sampling two (2) boreholes numbered 16-01 and 16-02, which were drilled at the east and west abutments to depths of 19.2 m and 19.5 m, respectively. The approximate locations of all completed boreholes are shown on the attached Borehole Locations and Soil Strata Drawing enclosed in Appendix D.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling operations. The ground surface elevations for the boreholes were derived from the Survey Plan B-514893-Nort-2, dated November 2016, provided to Thurber by Hatch.

Track-mounted CME 750 drill rig was used to drill the boreholes. The boreholes were advanced using hollow stem augers and NW casing. An NQ core barrel was used to obtain 3 m of rock core in each borehole. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) procedures, as per ASTM D-1586-99.

The drilling and sampling operations were supervised on a full time basis by members of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed throughout the drilling operations and in open boreholes after completion of drilling. The groundwater level observations may not be representative of the site conditions, as water was used during bedrock coring operations. The boreholes were backfilled in general accordance with MOE Regulation 903 (amended by Ontario Reg. 372). Completion details of the boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Unit	Borehole Number	Borehole Depth/ Base Elev. (m)	Completion Details
East Abutment	16-01	19.2 / 314.9	Bentonite holeplug to surface.
West Abutment	16-02	19.5 / 315.0	Bentonite holeplug to surface.

4. LABORATORY TESTING

All recovered soil samples were subjected to visual identification (VI) and natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer). All laboratory tests were carried out to MTO and / or ASTM Standards, as appropriate. The results of the geotechnical laboratory program are summarized on the Record of Borehole sheets included in Appendix A and on figures presented in Appendix B.

In order to assess the potential for sulphate attack on concrete foundations, as well as the potential for corrosion associated with the below ground position of the steel structure, a sample of the native silty clay and a sample of surface water from the river upstream of the bridge, were collected. The samples were submitted to SGS Laboratories in Lakefield, Ontario, for analytical testing of corrosivity parameters and sulphate. The results of the analytical testing are summarized in Section 6 below and the Certificates of the Analysis are enclosed in Appendix B.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing included in Appendix D. An overall description of the stratigraphy is given in the following paragraphs. The factual data presented on the Record of Borehole sheets takes precedence over this general description and should be used for interpretation of the site conditions. It should be recognized and expected that soil conditions may vary between and beyond borehole locations.

In summary, the embankment fill at this site overlies a layer of silty clay, which in turn is underlain by a cohesionless deposit grading with depth from sand to sand and silt. The overburden at this site is underlain by a granite bedrock at a depth of 16.2 to 16.5 m (Elev. 318.0).

Water level in Badesdawa River was indicated on the Survey Plan drawing at Elev. 330.73 in September 2016.

5.1 Embankment Fill

Embankment fill material consisting of brown sand and gravel with trace to some silt was encountered extending from ground surface in both boreholes. The fill thickness was 0.8 m and 2.2 m in Borehole 16-01 and 16-02 with the underside of the fill between Elev. 333.4 and Elev. 332.3, respectively.

SPT 'N' values obtained within the fill in Borehole 16-01 was 37 blows per 0.3 m of penetration, indicating a dense relative density and, the values ranged from 5 to 7 blows per 0.3 m of penetration in Borehole 16-02, indicating a loose relative density. Moisture contents between 3% and 8% were measured in the fill. Field vane shear tests measured undrained shear strength ranging from 45 kPa to 58 kPa. Vane tests measured that the sensitivity of the silty clay ranged from 1.4 to 2.4 indicating that the silty clay has low sensitivity.

The results of grain size distribution analysis carried out on a fill sample are presented on the Record of Borehole sheets included in Appendix A and on Figure B1 in Appendix B. The results of the grain size distribution analysis are summarized below:

Soil Particle	Percentage (%)
Gravel	46
Sand	43
Silt & Clay	11

5.2 Silty Clay

A deposit of brown to grey silty clay was encountered below the fill in both boreholes. The silty clay contained trace to some sand, trace gravel and occasional silt seams. The deposit was 4.8 m and 6.5 m thick and extended to depths of 5.6 m (Elev. 328.5) in Borehole 16-01 and 8.7 m (Elev. 325.8) in Borehole 18-02.

SPT 'N' values obtained within the silty clay ranged from 2 to 11 blows per 0.3 m of penetration, indicating a soft to stiff consistency, however, the SPT 'N' values typically ranged from 4 to 7 blows per 0.3 m of penetration, indicating a firm consistency. Field vane shear tests measured undrained shear strength ranging from 45 kPa to 58 kPa, indicating a firm to stiff consistency. Vane tests measured that the sensitivity of the silty clay ranged from 1.4 to 2.4 indicating that the silty clay has low sensitivity.

The results of grain distributions analyses carried out on representative samples of the deposit are presented on the Record of Borehole sheets in Appendix A and on Figure B2 in Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	0
Sand	0 to 18
Silt	65 to 77
Clay	17 to 33

Moisture contents between 12% and 50% were measured in the silty clay.

5.3 Sand

Underlying the silty clay is a layer of sand with trace silt and trace gravel. Traces of organic matter, namely fragments of decayed wood, were noted in the upper 0.5 m zone of sand in Borehole 16-01. The sand layer varied in thickness from 6.3 m in Borehole 16-01 to 1.5 m in Borehole 16-02. The underside of the sand was encountered between depths of 11.9 m (Elev. 322.2) and 10.2 m (Elev.324.3) in Boreholes 16-01 and 16-02, respectively.

SPT N-values measured in the sand typically ranged from 7 to 24 blows per 0.3 m penetration, indicating a loose compact relative density. The sand was very loose with a SPT N-value of 3 in the upper zone of Borehole 16-01, immediately beneath the silty clay. The results of grain size analysis conducted on a sample of the sand are provided on the Record of Borehole sheets in Appendix A, and illustrated in Figures B3 of Appendix B. The results are summarized in the following table.

Soil Particle	Percentage (%)
Gravel	1
Sand	96
Silt and Clay	3

The measured water contents of sand samples ranged from 12% to 17%.

5.4 Sand and Silt

Underlying the sand was a layer of sand and silt with trace clay and trace gravel. The layer was 4.3 m and 6.3 m in thickness with the underside at depths of 16.2 m and 16.5 m, both at Elev.318.0.

SPT 'N' values obtained in the sand and silt ranged from 11 to 24 blows per 0.3 m penetration, indicating a compact relative density.

Samples of the sand and silt were subjected to the grain size distribution testing; the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A. The grain size distribution curves for these samples are shown on Figure B4 of Appendix B.

Soil Particles	%
Gravel	0 to 3
Sand	46 to 63
Silt	35 to 50
Clay	2 to 4

Measured moisture contents ranged from 11% to 18%.

5.5 Bedrock

Granite bedrock was encountered below the sand and silt in both boreholes and proved by coring. Depths and elevations to the top of bedrock at the borehole locations are summarized in Table below. The top of bedrock may vary across the site and between and beyond the borehole locations.

Table 5.1 – Depths and Elevations of Top of Bedrock

Foundation Element	Borehole	Top of Bedrock	
		Depth (m)	Elevation (m)
East Abutment	16-01	16.2	318.0
West Abutment	16-02	16.5	318.0

The bedrock was described as slightly weathered to fresh, grey and strong.

Total Core Recovery (TCR) in the bedrock ranged from 96% to 100%. The RQD values ranged from 70% to 98%, which indicated a fair to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as number of fractures per 0.3 m of core run, varied from 0 to 4. A 300 mm clay seam was noted at 16.8 m depth (Elev. 317.7) in Borehole 16-02.

5.6 Groundwater Conditions

Where possible, water levels were monitored in the open boreholes during drilling operations. Water was introduced into boreholes during bedrock coring, and therefore water levels recorded

upon completion of drilling may not reflect natural groundwater levels. The water levels observed upon completion of drilling are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
16-01	October 20, 2016	Dry	-	Open Borehole
16-02	October 19, 2016	3.7	330.8	Open Borehole upon completion of drilling

The recorded levels are very short-term readings and seasonal fluctuations of the groundwater and river level are to be expected. In particular, the water level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

The Survey Plan provided by Hatch indicated water level in Badesdawa River at Elev. 330.73 in September 2016.

6. CORROSIVITY AND SULPHATE TEST RESULTS

A sample of the native silty clay and a sample of surface water from Badesdawa River were submitted to SGS Canada for analytical testing of corrosivity parameters and sulphate. The results of the analytical tests are summarized in Table 6.1. The laboratory certificates of analysis are presented in Appendix B.

Table 6.1 – Analytical Test Results

Parameter	Units (Soil)	Units (Water)	Test Results	
			BH 16-01, SS4 (7.5'-9.5')	Badesdawa River Water
pH	-	-	6.68 – 7.49	7.89
Redox Potential	mV	mV	91	235
Sulphide	%	mg/L	< 0.02	0.006
Chloride	µg/g	mg/L	2.5	0.30
Sulphate	µg/g	mg/L	11	0.37
Electrical Conductivity	µS/cm	µS/cm	32	91
Resistivity	Ohms.cm	Ohms.cm	31000	11000

7. MISCELLANEOUS

Borehole locations were established in the field by Thurber Engineering Ltd. The northing and easting coordinates and ground surface elevations were obtained from measurements taken in the field relative to the Survey Plans provided by Hatch.

Thurber obtained subsurface utility clearances prior to drilling. RPM Drilling Inc. of Thunder Bay, Ontario supplied and operated the drilling, sampling and in-situ testing equipment for the field investigation. The field investigation was supervised on a full time basis by Mr. Troy MacKinnon of Thurber. Overall supervision of the field program was provided by Mr. Mark Farrant, P.Eng. of Thurber.

Geotechnical laboratory testing was carried out at Thurber's geotechnical laboratory. Analytical laboratory testing was carried out by SGS Canada Inc.

Interpretation of the field data and preparation of this report were carried out by Ms. Anna Piascik, P.Eng. and Mr. Mark Farrant, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

8. GENERAL

This report presents interpretation of the geotechnical data in the factual report and provides preliminary geotechnical recommendations for the proposed replacement of the existing Badesdawa River Bridge located on Nort Road, in the Unsurveyed Territory, District of Kenora, Ontario.

This foundation investigation and design report with the interpretations and recommendations is intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractors. Contractors must make their own interpretations based on the factual data in part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

The existing bridge is a 33.8 m single span, single lane modular bridge with steel deck. The superstructure is supported by timber crib abutments. In the vicinity of the existing bridge, the river banks are between 4 and 5 m in height. The Nort Road is a gravel, un-paved road running at the bridge site generally in the southwest-northeast direction. Badesdawa River flows from the southeast to northwest to join Otokwin River some distance to the northwest of the bridge.

The preliminary General Arrangement drawing indicates that the replacement bridge will be a 35.1 m long single span, two lane structure with a deck width of 9.9 m. The replacement bridge will be located on the same alignment. The proposed grade raise for the replacement bridge will be approximately 330 mm at the west abutment and 260 mm at the east abutment.

The traffic on Nort Road will be diverted through a proposed temporary detour bridge to the north of the existing alignment during construction of the replacement bridge. The proposed detour bridge will be located at a centreline-to-centreline distance of approximately 7.8 m from the existing alignment. The foundation piles for the detour bridge will be installed as close as 1 m from the existing bridge abutment footings. The height of the temporary approach embankment fill will be approximately 2.6 m at the west approach and 2.0 m at the east approach.

The discussions and preliminary recommendations presented in this report are based on the information provided in the Terms of Reference, Survey Plan (Plan No. B-514893-Nort-2) dated November 2016 supplied by Hatch, and on the factual data obtained during the course of this investigation.

9. STRUCTURE FOUNDATIONS

In summary, the soil stratigraphy below the existing embankment fill consisted of a layer of silty clay underlain by a cohesionless deposit grading with depth from sand to sand and silt. The overburden at this site was underlain by a granite bedrock at a depth of 16.2 to 16.5 m (Elev. 318.0). Water level in Badesdawa River was indicated on the Survey Plan at Elev. 330.73 on September 2016.

The following foundation options could be considered for preliminary design of the bridge replacement:

- spread footings placed on native silty clay
- spread footings placed on engineered fill, and
- Steel H-piles driven to bedrock, including the integral abutment option.

Recommendations for preliminary design of the feasible foundation options are presented in the following sections together with the corresponding geotechnical design parameters, as applicable. A preferred foundation option from a geotechnical perspective is indicated.

9.1 Spread Footings on Native Silty Clay

Underlying the embankment fill is the silty clay of soft to stiff consistency. The layer of silty clay varies in thickness from 4.8 m at the east abutment to 6.5 m at the west abutment. Given the relatively low strength, high compressibility and variable thickness of the silty clay deposit, spread footings placed directly on the native silty clay are not recommended due to low bearing resistances available, and the potential for post-construction settlements for an extended period of time induced by the foundation loads.

9.2 Spread Footings on Engineered Fill Pads

9.2.1 Founding Level

The replacement bridge can be supported on concrete spread footings placed on granular engineered fill pads. The Survey Plan indicates the existing Nort Road grade at approximate Elev. 334.1 at the east abutment and Elev. 334.5 at the west abutment.

The existing embankment fill materials extend to depths of 0.8 m (Elev. 333.4) and 2.2 m (Elev. 332.3) at the borehole locations near the east and west abutments, respectively. The fill is loose to dense and consists of sand and gravel with trace to some silt. Underlying the fill is a native deposit of silty clay extending to depths of 5.6 m (Elev. 328.5) and 8.7 m (Elev. 325.8) at the east and west abutments, respectively.

The engineered granular fill pads at least 2 m thick may be placed on the native silty clay deposit at Elev. 332.0 or lower, preferably, above the water level in the river.

9.2.2 Engineered Fill Construction

The engineered fill pads should consist of OPSS Granular "A" or Granular B Type II placed in 150 mm lifts and compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content. A sketch of the abutment placed on compacted fill is enclosed in Appendix E, following the text of this report.

Excavations for the engineered fill construction will most likely require the existing timber cribs and gabion baskets to be removed or partially removed, depending on the adopted alignment and length of the replacement structure. For construction of the engineered fill pad, the following construction sequence may be considered:

1. The minimum depth of excavation should accommodate the concrete foundation slab and the thickness of engineered fill pad below the slab;
2. The subgrade for the engineered fill pad should be inspected and all organics, soft/loose soils, and any deleterious materials should be removed from the footprint of the excavation and replaced with compacted Granular A or Granular B Type II;
3. All timber and other deleterious material from the footprint of the new foundation should be removed. If the footprints of the engineered pads overlap the footprints of the existing timber crib abutments, the timber cribs should be removed, however no deeper than the level of the river;
4. Dewatering measures should be provided, as required, to place the engineered fill in the dry;

5. The dimensions of the base of the excavation should be determined by assuming a granular pad 1.0 m wider than the spread footing at the level of the footing base and projecting outward and downward at 1H:1V.

It will be beneficial to place/locate the new abutments/spread footings some distance behind the existing crib abutments to take advantage of the potential slope stabilizing effect of the existing crib foundations.

If the engineered fill pads are located close to the river channel, the forward slope of the foundation pads should be embedded at least 1.0 m below a 2H:1V face of the forward slope. Provision of properly design erosion protection works will be critical to ensure adequate performance of the foundations/engineered fill pads.

9.2.3 Axial Geotechnical Resistance and Geotechnical Reaction

The following values of factored Geotechnical Resistance at ULS and Geotechnical Reaction at SLS may be used for preliminary design of a minimum 1.5 m wide spread footings placed on engineered fill pads prepared as outlined above with the underside at or below Elev. 332.0:

Factored Geotechnical Resistance at ULS (kPa)	200
Geotechnical Reaction at SLS (kPa)	150

The value of the Geotechnical Reaction at SLS given corresponds to 25 to 35 mm of settlement.

The value of Factored Geotechnical Resistance at ULS was assessed assuming a Consequence Factor equal to 1 (Typical), and a Resistance Factor equal to 0.5 (Typical degree of understanding of the subsurface conditions), as per CHBDC 2014. The Geotechnical Reaction at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of the subsurface conditions.

The geotechnical resistance quoted above is for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance should be calculated as indicated in the CHBDC 2014 Clause 6.10.3 and Clause 6.10.4.

9.2.4 Lateral Resistance

The lateral resistance of the concrete footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.5. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

9.3 Driven H-Pile Foundations

The subsurface conditions at the site are considered to be suitable for the use of driven steel H-pile foundations to support the bridge abutments. The piles may be driven to refusal on bedrock, which was encountered at depths of 16.2 m (Elev. 318.0) and 16.5 m (Elev. 318.0) in the vicinity of the east and west abutments, respectively.

9.3.1 Axial Geotechnical Resistances

For preliminary design, a factored geotechnical resistance and geotechnical reaction as well as estimated tip elevations for HP 310x110 piles driven to the bedrock surface are presented in Table 9.1. The pile lengths were estimated assuming that the undersides of the pile caps were located at a depth of 2.8 m below the existing ground surface (depth of frost penetration at this site).

Table 9.1 – Axial Geotechnical Resistances for HP310x110 Piles Driven to Bedrock

Abutment Location / Reference Borehole	Estimated Pile Tip Elevation/Bedrock Surface (m)	Approximate Pile Length (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
East / 16-01	318.0	15	2,000	Does not govern.
West / 16-02	318.0	15		

9.3.2 Pile Installation

Pile installation should be in accordance with OPSS.PROV 903.

9.3.3 Pile Tips

To prevent pile damage when setting the piles on bedrock, piles should be equipped with tip protections. Pile tip protection supplied by an approved manufacturer such as Titus Steel (Standard H-point), Skyline Steel or approved equivalent could be used at this site.

9.3.4 Downdrag Load

Driven H-piles will encounter practical refusal on bedrock. The weight of the new approach fill placed to achieve the final design grade will induce consolidation settlements of the underlying silty clay layer. Downdrag load will develop along the length of abutment piles embedded in the silty clay.

The magnitude of downdrag load and its impact on the abutment piles should be evaluated (as per CHBDC Commentary Clause C6.11) during the final design stage of the project.

9.4 Abutment Type

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The subsurface conditions at this site are considered suitable for integral, semi-integral or conventional abutment design. The use of H-piles at the abutments would allow for the design of integral abutments, as envisioned in the Terms of Reference.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. To provide the required flexibility for piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design manual. After the pile is installed, the space between the pile and the CSP should be filled with loose uniform sand. The grain size distribution of the sand as listed in Table 9.2 can be used for backfilling of the CSPs.

Table 9.2 - Grading of Sand for Integral Abutment Backfill

MTO Sieve Designation	Percentage Passing
2 mm (#10)	100%
600 µm (#30)	80%-100%
425 µm (#40)	40%-80%
250 µm (#60)	5%-25%
150 µm (#100)	0%-6%

9.5 Lateral Pile Resistance

The geotechnical lateral resistance acting on a pile in cohesionless soils may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \gamma' z K_p \quad (\text{kPa})$$

Where

- z = depth of embedment of pile (m)
- D = pile width or diameter (m)
- n_h = coefficient related to soil relative density (kN/m^3)
- γ' = effective unit weight (kN/m^3)
- K_p = passive earth pressure coefficient

The geotechnical lateral resistance acting on a pile in cohesive soils may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = 67 S_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 S_u \quad (\text{kPa})$$

Where S_u = undrained shear strength (kPa)

D = pile width or diameter (m)

For preliminary analysis of the interaction between a pile and the surrounding soil, the above equations and parameters recommended in Table 9.3 below, may be used. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

Table 9.3 – Soil Parameters for Lateral Pile Resistance

Soil Unit	Elevation (m)		γ' (kN/m ³)	n_h (kN/m ³)	K_p	S_u (kPa)
	Top	Bottom				
East Abutment (Borehole 16-01)						
Silty Clay (above water level)	333.4*	330.7	19	-	-	40
Silty Clay (below water level)	330.7	328.5	9			35
Sand	328.5	322.2	11	2,500	3.0	-
Sand and Silt	322.2	318.0 (Bedrock)	10	3,000	3.0	-
West Abutment (Borehole 16-02)						
Silty Clay (above water level)	332.3*	330.7	19	-	-	45
Silty Clay (below water level)	330.7	325.8	9	-	-	45
Sand	325.8	324.3	11	4,000	3.3	-
Sand and Silt	324.3	318.0 (Bedrock)	11	4,000	3.3	-

Note: * Indicated top of silty clay layer; the design level of underside of pile caps at abutments and depth of frost penetration should be considered.

The spring constant, K_s , for analysis may be obtained by the expression, $K_s = k_s L D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} L D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

The modulus of subgrade reaction and ultimate lateral resistance may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 9.4. Intermediate values may be obtained by linear interpolation.

Table 9.4 – Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing (Centre to Centre)	Reduction Factor
Pile group oriented perpendicular to direction of loading	4D	1.0
	1D	0.5
Pile group oriented parallel to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

In the case of conventional abutments, i.e. not integral type, horizontal loads may be resisted by means of battered piles.

9.6 Frost Cover

The depth of frost penetration at this site is approximately 2.8 m, as per OPSD 3090.100.

If steel H-piles are adopted, the base of pile caps should be provided with a minimum of 2.8 m of earth cover as protection against frost action. If it is not practical to provide 2.8 m of earth cover, consideration can be given to using of expanded polystyrene insulation (EPS). Typically, 25 mm of EPS can be considered as an equivalent to 600 mm of earth cover. If EPS is used, it should be provided with a long term protection against erosion, environmental degradation and spills.

Concrete bearing slab foundations for modular bridge founded on a non-frost susceptible, free draining engineered fill pad above the river water level should be provided with a minimum embedment of 0.5 m.

9.7 Recommended Foundation

Steel H-piles driven to bedrock are preferred at this site due to ease and expeditiousness in construction. Spread footings placed on engineered fill pads are considered feasible foundation option at this site.

10. SHEET PILE WALLS

The preliminary design proposes the use of steel sheet pile walls adjacent to the pile foundations in lieu of conventional abutment walls. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill.

Lateral stability of the sheet pile walls should be checked by the wall designer and the depths of penetration or sheet pile tip elevations determined using the geotechnical design parameters presented in Table 10.1.

The coefficients of passive earth pressure (K_p) are provided for horizontal ground surface in front of the sheet pile wall. For sloping ground in front of the sheet pile wall, the recommended values for the coefficients of passive earth pressure (K_p) should be reduced.

Table 10.1 – Soil Parameters for Sheet Pile Analysis

Foundation Element (Reference Borehole)	Soil Unit	Elevation (m)		γ' (kN/m ³)	K_a	K_p
		Top	Bottom			
East Abutment (16-01)	Sand & Gravel Fill	334.9*	333.4	20	0.33	3.0
	Silty Clay (above water level)	333.4	330.7	19	0.41	2.5
	Silty Clay (below water level)	330.7	328.5	9	0.41	2.5
	Sand	328.5	322.2	11	0.33	3.0
	Sand & Silt	322.2	318.0 (bedrock)	10	0.32	3.1
West Abutment (16-02)	Sand & Gravel Fill	334.6*	332.3	20	0.33	3.0
	Silty Clay (above water level)	332.3	330.7	19	0.41	2.5
	Silty Clay (below water level)	330.7	325.8	9	0.41	2.5
	Sand	325.8	324.3	11	0.31	3.3
	Sand & Silt	324.3	318.0 (bedrock)	10	0.32	3.1

Note: * Elevation of top of sheet pile varies.

In general, backfill to the sheet pile walls should be in accordance with OPSS 902 and should consist of Granular A, Granular B Type II or III material. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS.PROV 501.

Given the presence of the sensitive sand/silt deposit underlying the silty clay deposit, vibratory methods must not be used at this site to install sheet piles.

Design of the permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause corrosion and reduce the service life of the structure. The native soils in front of the sheet piles should be protected from river erosion so that the sheet piles do not lose lateral support.

11. LATERAL EARTH PRESSURE

If abutment walls are required, the lateral earth pressures acting on the walls may be assumed to be triangularly distributed and governed by the characteristics of the backfill and existing fill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC 2014 but generally are given by the following equation and in the table below:

$$p_h = K (\gamma h + q) \quad (\text{kN/m}^3)$$

Where:

- p_h = horizontal pressure on the wall at depth h (kPa)
- K = coefficient of lateral earth pressure (see table below)
- γ = unit weight of retained soil (see table below)
- h = depth below top of fill where pressure is computed (m)
- q = value of any surcharge (kPa)

Table 11.1 – Coefficients of Lateral Earth Pressures

Loading Condition	OPSS Granular A or Granular B Type II $\phi = 35^\circ$; $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I (modified) $\phi = 32^\circ$; $\gamma = 21.2 \text{ kN/m}^3$		Existing Fill $\phi = 30^\circ$; $\gamma = 20 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*	0.33	0.52*
At-rest (Restrained Wall)	0.43	-	0.47	-	0.50	-
Passive	3.7	-	3.3	-	3.0	-

Note: * For wingwalls, if required

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is preferred as it results in lower earth pressures on the retaining structure.

A compaction surcharge should be added in accordance with Clause 6.12.3 of the CHBDC 2014.

12. SEISMIC CONSIDERATIONS

In accordance with the CHBDC, the selection of the seismic site class is based on the soil conditions encountered in the upper 30 m of the stratigraphy.

The stratigraphy at this site corresponds to a Seismic Site Class D in accordance with Table 4.1, Clause 4.4.3.2 of the CHBDC. The peak ground acceleration, PGA, for a 2,475-year return period seismic event at this site is 0.040 g as per the National Building Code of Canada (NBCC).

Given the firm to stiff silty clay and compact sands and silts underlain by bedrock at this site and relatively low value of PGA, the potential for liquefaction at this site is assessed to be low.

13. SCOUR AND EROSION CONTROL

The forward slopes appear to be experiencing erosion. Adequate scour and erosion protection measures should be provided for the forward slopes at the bridge and the river bank slopes on both sides of the bridge. Design of the scour and erosion protection works should be undertaken by a specialist in this field.

Protection of the river banks is important to avoid undermining of the bridge foundations. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS.PROV 804.

14. EXCAVATION AND GROUNDWATER CONTROL

Excavation for works associated with the construction of the new abutments will extend through the existing fill and into the native silty clay. For the footing on engineered fill pad, it is anticipated that the base of the excavation will be above the river level. Excavation below river level must not be undertaken without prior dewatering. Removal of the existing timber cribs and gabion baskets will be required for construction of engineered fill pad or pile caps/abutments.

All excavations should be carried out in accordance with OPSS 902 and the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the approach embankment fill within the depth of excavation may be classed as Type 3 soil above the water table and Type 4 below the water level. The native silty clay may be classed as Type 3 soil.

Selection of the method of excavation is the responsibility of the Contractor and should be based on the Contractor's experience, equipment and interpretation of the site conditions. The existing timber cribs and gabions may contain rock fill material. Provision should be made for handling of potential obstructions, such as cobbles and boulders.

Depending on the required depth of excavation, seepage into the excavation may be handled by pumping from filtered sumps. The use of sandbagged cofferdams may be considered if excavation below river level is planned. The design of groundwater control system is the responsibility of the Contractor.

15. ROADWAY PROTECTION SYSTEM

Given relatively shallow excavation required, it is unlikely that installation of roadway protection systems will be required. However, if a temporary protection system is used, the design of the protection system is typically the responsibility of the Contractor, as per OPSS.PROV 539. The protection system should be designed for Performance Level 2 (maximum 25 mm horizontal deflection). Temporary protection system may consist of interlocking sheet pile or steel soldier pile and timber lagging walls, if required.

Full hydrostatic pressure should be considered assuming a water level equal to the design river water level. The actual pressure distribution acting on the protection system is a function of the construction sequence and the relative flexibility of the wall.

All temporary protection systems should be designed by a Professional Engineer experienced in such designs.

16. APPROACH EMBANKMENTS

The existing approach embankments can be as high as 4 m to 5 m near the abutments. No evidence of instability of the existing approach embankments were noted during the time of the foundation investigation, although settlements at the abutments were evident. These settlements could be related to the river bank erosion leading to undermining of the timber cribs and loss/washout of the abutment fill.

The temporary approach embankment fill for the detour alignment should be constructed using earth fill or granular material conforming to OPSS.PROV 1010 and compacted in accordance with OPSS.PROV 501. The temporary embankment with side slopes no steeper than 2H:1V is assessed to be stable from a short-term perspective.

Foundation settlement under the 260 to 330 mm grade raise on the existing approach embankments is estimated to range between 15 mm and 20 mm. These settlements will be essentially complete within 1 to 2 months of completion of fill placement. In addition to the grade raise, the existing approach embankments will be widened by approximately 2 to 2.5 m to the north. Foundation settlement associated with the widening of the approach embankments is estimated to range between 40 mm and 50 mm.

Settlement of the temporary embankments up to 2.6 m high on the detour alignment is estimated to be in the order of 50 mm immediately behind the temporary abutments. About 50% of the estimated settlement will occur within approximately 2 months following the fill placement. The settlement will be essentially complete in 4 to 5 months after the completion of fill placement.

17. CORROSION AND SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on the samples of the native silty clay and river water indicate the following conditions at the locations tested:

- The potential for corrosion or sulphate attack on concrete foundations from the surrounding native soil is considered negligible due to low concentrations of sulphate and chloride in the samples tested.
- The potential for soil corrosion on metal is considered to be very mild.
- Appropriate protection measures are recommended if metal structural elements are used.

18. CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Seasonal fluctuations of the groundwater and river water levels are to be expected. In particular, the water level may be at a higher elevation after periods of heavy rainfall, which may impact the construction.
- Rock fill may be present as fill in the timber cribs, as well as in the gabions installed at the existing bridge. These obstructions may interfere with excavations or installation of piles and temporary protection system, should it be required.
- Excavation/removal of the existing timber crib abutments should be completed to the river water level prior to construction of new foundations.

19. CLOSURE

Engineering analysis and preparation of this report were carried out by Mr. Mark Farrant, P.Eng. and Mr. Keli Shi, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.



Mark Farrant, P.Eng.
Geotechnical Engineer



Keli Shi, P.Eng.
Senior Geotechnical Engineer



Dr. P.K. Chatterji, P.Eng.
Designated MTO Principal Contact



Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

EXPLANATION OF ROCK LOGGING TERMS


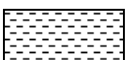

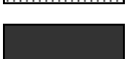

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

DISCONTINUITY SPACING

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

SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
	(MPa)	(psi)	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length
Solid Core Recovery:(SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run
Rock Quality Designation:(RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a % of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index:(FI)	Frequency of natural fractures per 0.3m of core run.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

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RECORD OF BOREHOLE No 16-01

2 OF 3

METRIC

GWP# 6623-17-00 LOCATION Badesdawa River Bridge Replacement N 5 738 870.1 E 331 412.5 ORIGINATED BY TM
HWY Nort Road (formerly Hwy 808) BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AM
DATUM Geodetic DATE 2016.10.20 - 2016.10.20 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
								WATER CONTENT (%)						
	Continued From Previous Page						20 40 60 80 100		20 40 60					
322.2	SAND and SILT , trace clay Compact Grey Moist		10	SS	7								0 63 35 2	
11.9														
			11	SS	11									
			12	SS	14									
			13	SS	14									
318.0	GRANITE , slightly weathered to fresh, strong, grey (BEDROCK) Vertical fracture at 17.1m												RUN #1 TCR=96% SCR=88% RQD=77%	
16.2			1	RUN										
			2	RUN									RUN #2 TCR=98% SCR=98% RQD=98%	
314.9	END OF BOREHOLE AT 19.2m. NO FREE STANDING WATER IN BOREHOLE. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO													
19.2														

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Sensitivity

20
15
10

(%) STRAIN AT FAILURE

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METRIC

[illegible]

RECORD OF BOREHOLE No 16-02

1 OF 3

METRIC

GWP# 6623-17-00 LOCATION Badesdawa River Bridge Replacement N 5 738 844.6 E 331 371.6 ORIGINATED BY TM
 HWY Nort Road (formerly Hwy 808) BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AM
 DATUM Geodetic DATE 2016.10.18 - 2016.10.19 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100					
334.5	GROUND SURFACE												
0.0	SAND and GRAVEL , trace silt Loose Brown Moist (FILL)		1	SS	7								
			2	SS	7								
			3	SS	5								
332.3													
2.2	Silty CLAY , trace to some sand, trace gravel Firm to Stiff Brown Moist		4	SS	7								
			5	SS	5								
			6	SS	6								
			7	SS	6								
			8	SS	11								
325.8													
8.7	SAND , trace silt, trace gravel Compact Brown Moist		9	SS	24								

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 16-02

2 OF 3

METRIC

GWP# 6623-17-00 LOCATION Badesdawa River Bridge Replacement N 5 738 844.6 E 331 371.6 ORIGINATED BY TM
 HWY Nort Road (formerly Hwy 808) BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AM
 DATUM Geodetic DATE 2016.10.18 - 2016.10.19 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			
								20 40 60 80 100						20 40 60			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			
Continued From Previous Page																	
324.3																	
10.2	SAND and SILT , trace clay, trace gravel Compact Grey Moist						324								0 46 50 4		
		10	SS	24													
							323										
		11	SS	14													
							321										
		12	SS	14											3 49 44 4		
							320										
							319										
318.0							318						FI				
16.5	GRANITE , slightly weathered to fresh, strong, grey (BEDROCK) Clay seam (300mm) at 16.8m		1	RUN									2	RUN #1 TCR=100% SCR=92% RQD=70%			
							317						1				
													0	RUN #2 TCR=100% SCR=92% RQD=85%			
													3				
													0				
			2	RUN			316						0				
													4				
													4				
315.0	Vertical break at 19.2m						315						0				
19.5	END OF BOREHOLE AT 19.5m. BOREHOLE OPEN AND FREE WATER AT 3.7m DEPTH.																

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 16-02

3 OF 3

METRIC

GWP# 6623-17-00 LOCATION Badesdawa River Bridge Replacement N 5 738 844.6 E 331 371.6 ORIGINATED BY TM
 HWY Nort Road (formerly Hwy 808) BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AM
 DATUM Geodetic DATE 2016.10.18 - 2016.10.19 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page													
	BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.													

ONTMT4S MTO-15697.GPJ 2015TEMPLATE(MTO).GDT 4/26/17



Appendix B

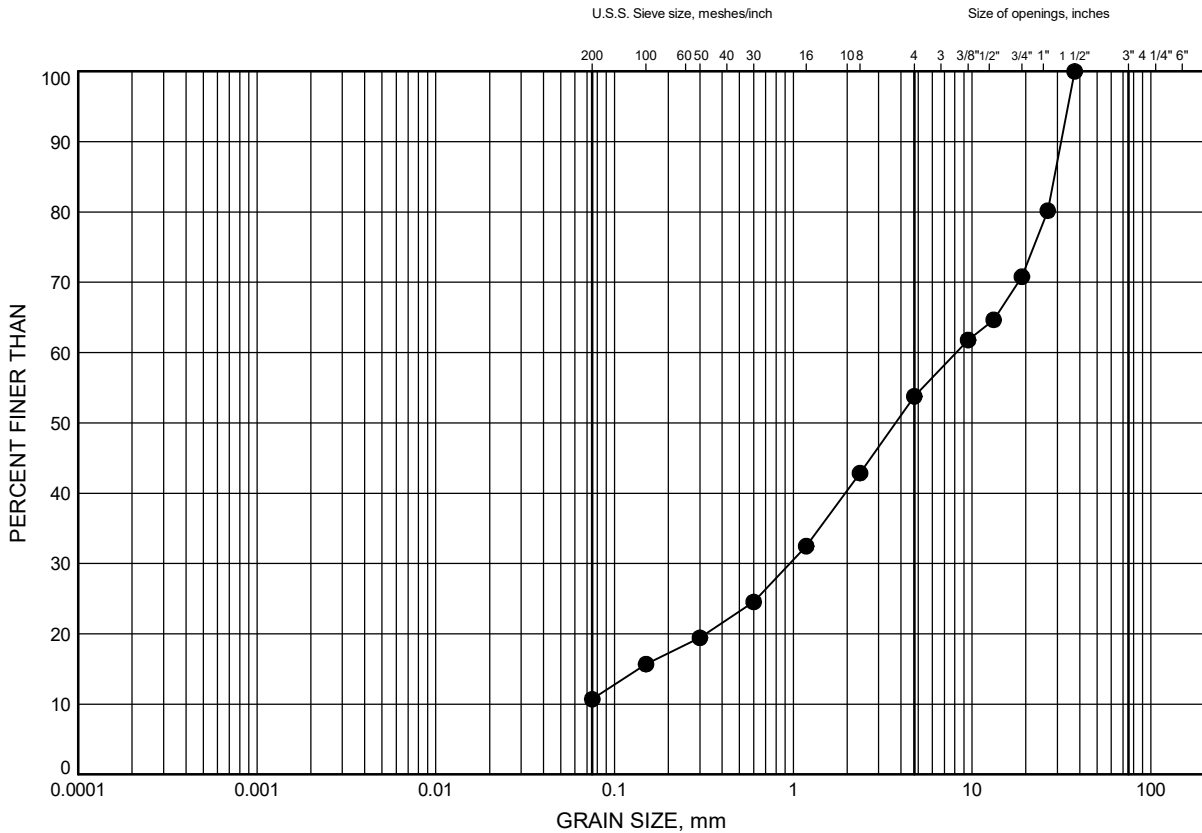
Geotechnical and Analytical Laboratory Test Results

Badesdawa River Bridge Replacement

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND and GRAVEL FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	0.30	333.83

Date April 2017
GWP# 6623-17-00

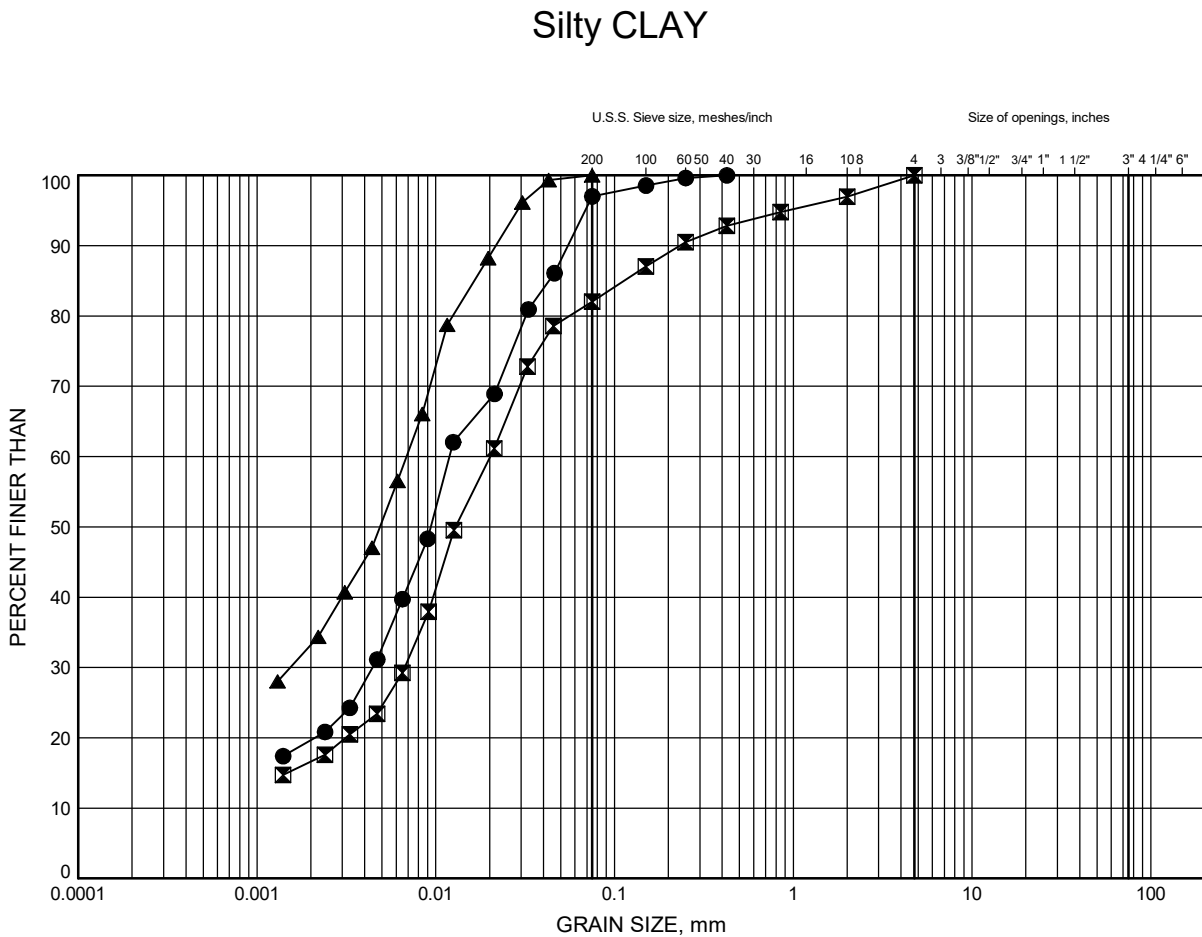


Prep'd MFA
Chkd. KS

Badesdawa River Bridge Replacement

GRAIN SIZE DISTRIBUTION

FIGURE B2



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	2.59	331.54
⊠	16-02	2.59	331.90
▲	16-02	6.40	328.09

Date April 2017
GWP# 6623-17-00

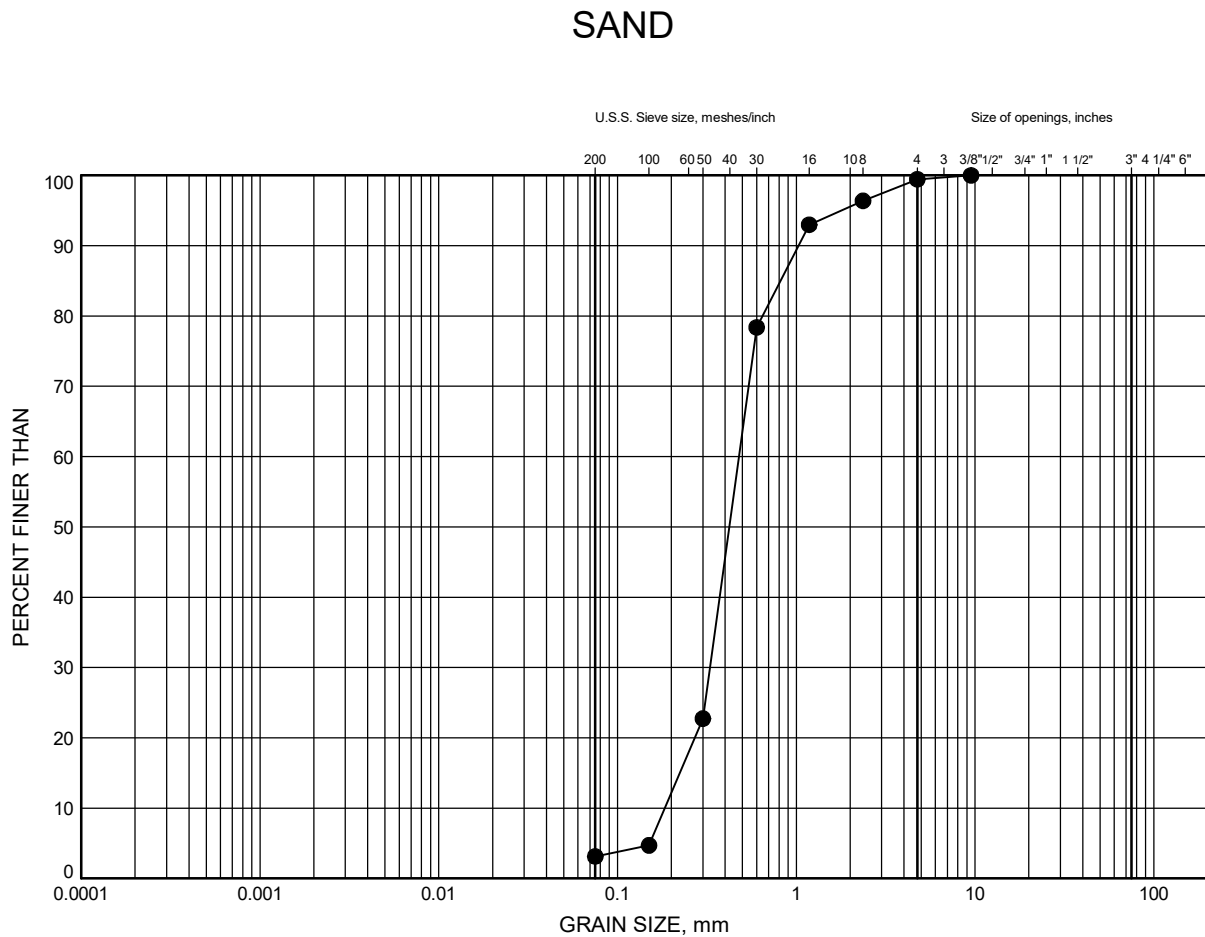


Prep'd MFA
Chkd. KS

Badesdawa River Bridge Replacement

GRAIN SIZE DISTRIBUTION

FIGURE B3



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	7.92	326.21

Date April 2017
GWP# 6623-17-00



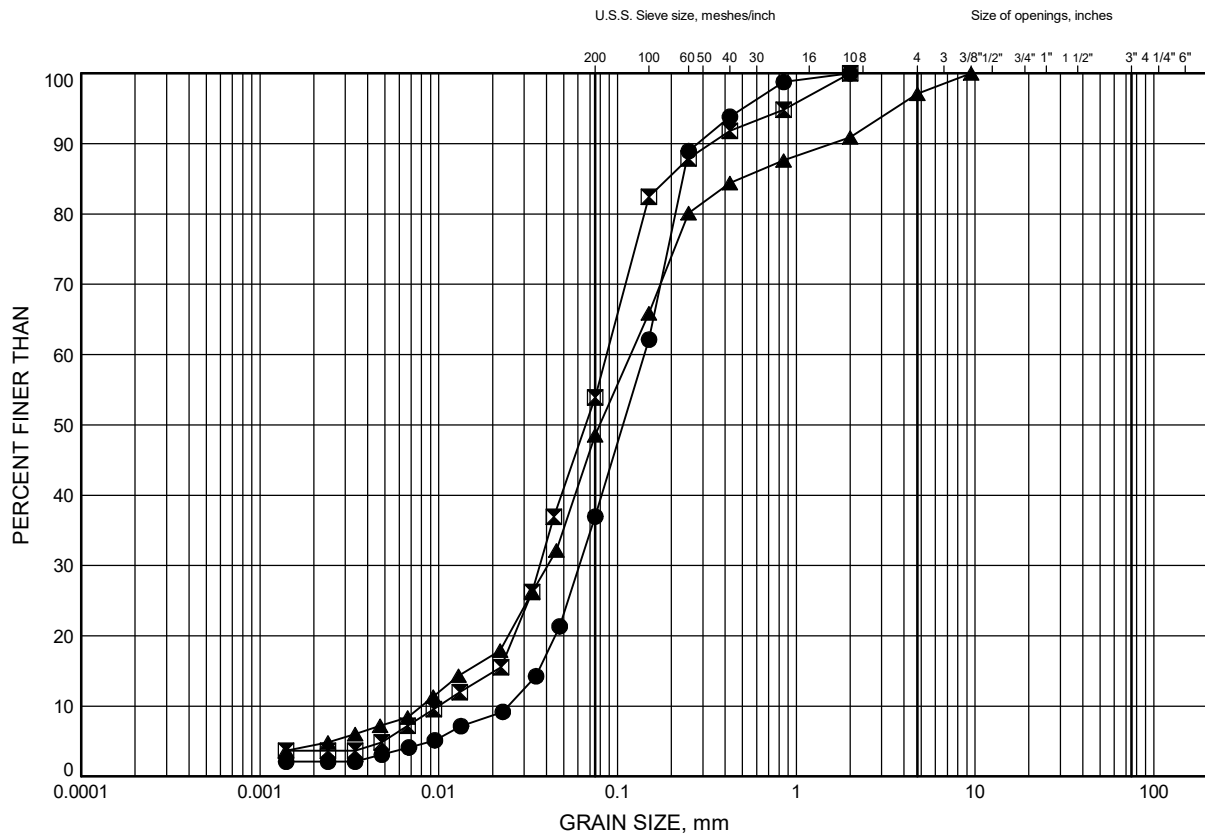
Prep'd MFA
Chkd. KS

Badesdawa River Bridge Replacement

GRAIN SIZE DISTRIBUTION

FIGURE B4

SAND and SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	16-01	12.50	321.64
⊠	16-02	10.97	323.51
▲	16-02	14.02	320.47

Date April 2017
GWP# 6623-17-00



Prep'd MFA
Chkd. KS

**SGS Canada Inc.**

P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - K0L 2H0
Phone: 705-652-2000 FAX: 705-652-6365

Project : 15697**12-April-2017****Thurber Engineering Ltd.****Attn : Mark Farrant**

103, 2010 Winston Park Drive
Oakville, ON
L6H 5R7,

Phone: 905-829-8666 x 228
Fax:

Date Rec. : 08 November 2016
LR Report: CA14154-NOV16
Reference: 15697 Mark Farrant

Copy: #1

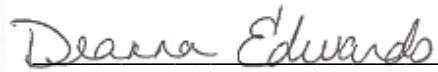
CERTIFICATE OF ANALYSIS

Final Report

Analysis	1: Analysis Start Date	2: Analysis Start Time	3: Analysis Approval Date	4: Analysis Approval Time	5: BH 16-01, SS4, 7'6"-9'-6"
Sample Date & Time					07-Nov-16
Corrosivity Index [none]	14-Nov-16	15:51	14-Nov-16	15:51	3.5
pH [no unit]	09-Nov-16	07:57	09-Nov-16	14:38	6.68
Soil Redox Potential [mV]	09-Nov-16	17:17	10-Nov-16	11:05	91
Sulphide [%]	14-Nov-16	14:06	14-Nov-16	15:47	< 0.02
% Moisture (wet wt) [%]	09-Nov-16	12:51	10-Nov-16	15:59	25.0
pH [no unit]	10-Nov-16	09:02	11-Nov-16	11:51	7.49
Chloride [µg/g]	11-Nov-16	20:14	14-Nov-16	08:21	2.5
Sulphate [µg/g]	11-Nov-16	20:14	14-Nov-16	08:21	11
Conductivity [µS/cm]	10-Nov-16	09:02	11-Nov-16	11:51	32
Resistivity (calculated) [Ohms.cm]	14-Nov-16	15:49	14-Nov-16	15:49	31000

Temperature of Samples upon receipt 15 degrees C
No cooling agent present

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.


Deanna Edwards, B.Sc, C.Chem
Project Specialist
Environmental Services, Analytical

**SGS Canada Inc.**

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Lakefield - Ontario - K0L 2H0

Phone: 705-652-2000 FAX: 705-652-6365

Project : 15697**LR Report :** CA14154-NOV16

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
Anions by IC	ME-CA-[ENV]IC-LAK-AN-001	EPA300/MA300-Ions1.3
Carbon/Sulphur	ME-CA-[ENV]ARD-LAK-AN-020	ASTM E1915-07A
Conductivity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2510
Metals Prep	ME-CA-[ENV]ARD-LAK-AN-013	
pH	ME-CA-[ENV]EWL-LAK-AN-001	SM 4500



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Phone: 705-652-2000 FAX: 705-652-6365

Project : 15697

LR Report : CA14154-NOV16

Quality Control Report

Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank				LCS / Spike Blank			Matrix Spike / Reference Material		
							Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
					RPD	Acceptance Criteria		Low	High		Low	High
Anions by IC - QCBatchID: DIO0187-NOV16												
Chloride	0.4	µg/g	<0.4		2	20	100	80	120	103	75	125
Sulphate	0.4	µg/g	<0.4		0	20	99	80	120	95	75	125
Carbon/Sulphur - QCBatchID: ECS0018-NOV16												
Sulphide	0.02	%	<0.02		NV	20	119	80	120			
Conductivity - QCBatchID: EWL0172-NOV16												
Conductivity	2	uS/cm	< 2		ND	10	99	90	110	NA		
pH - QCBatchID: ARD0035-NOV16												
pH	0.05	no unit			0	20	100	80	120			

**SGS Canada Inc.**

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Project : 15697

12-April-2017

Thurber Engineering Ltd.

Attn : Mark Farrant

103, 2010 Winston Park Drive
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L6H 5R7,

Phone: 905-829-8666 x 228
Fax:

Date Rec. : 27 October 2016
LR Report: CA15613-OCT16
Reference: Project: 15697 Mark Farrant


Copy: #1

CERTIFICATE OF ANALYSIS

Final Report

Analysis	1: Analysis Start Date	2: Analysis Start Time	3: Analysis Approval Date	4: Analysis Approval Time	5: MDL	6: Mud River Bridge
Sample Date & Time						20-Oct-16 08:20
Temperature Upon Receipt [°C]	---	---	--	--	---	20.0
pH [no unit]	28-Oct-16	07:54	31-Oct-16	08:53	0.05	7.89
Conductivity [µS/cm]	28-Oct-16	07:54	31-Oct-16	08:53	2	91
Resistivity (calculated) [Ohms.cm]	02-Nov-16	16:22	02-Nov-16	16:23	---	11000
Redox Potential [mV]	27-Oct-16	16:41	28-Oct-16	09:22	---	235
Chloride [mg/L]	01-Nov-16	01:42	02-Nov-16	11:15	0.04	0.30
Sulphate [mg/L]	01-Nov-16	01:42	02-Nov-16	11:15	0.04	0.37
Sulphide [mg/L]	28-Oct-16	15:13	28-Oct-16	16:45	0.006	0.006
Corrosivity Index [none]	02-Nov-16	16:25	02-Nov-16	16:25		2

Temperature of Sample upon receipt 20 degrees C
Cooling agent present
Custody Seal not Present


Deanna Edwards, B.Sc, C.Chem
Project Specialist
Environmental Services, Analytical

**SGS Canada Inc.**

P.O. Box 4300 - 185 Concession St.
Lakefield - Ontario - K0L 2H0
Phone: 705-652-2000 FAX: 705-652-6365

Project : 15697**LR Report :** CA15613-OCT16

Method Descriptions

Parameter	SGS Method Code	Reference Method Code
Anions by IC	ME-CA-[ENV]IC-LAK-AN-001	EPA300/MA300-Ions1.3
Conductivity	ME-CA-[ENV]EWL-LAK-AN-006	SM 2510
pH	ME-CA-[ENV]EWL-LAK-AN-006	SM 4500
Redox Potential		SM 2580
Sulphide by SFA	ME-CA-[ENV]SFA-LAK-AN-008	SM 4500



SGS Canada Inc.

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Phone: 705-652-2000 FAX: 705-652-6365

Project : 15697

LR Report : CA15613-OCT16

Quality Control Report

Inorganic Analysis												
Parameter	Reporting Limit	Unit	Method Blank				LCS / Spike Blank			Matrix Spike / Reference Material		
					RPD	Acceptance Criteria	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
						%		Low	High		Low	High
Anions by IC - QCBatchID: DIO0011-NOV16												
Chloride	0.04	mg/L	<0.04		1	20	104	80	120	107	75	125
Sulphate	0.04	mg/L	<0.04		3	20	99	80	120	95	75	125
Conductivity - QCBatchID: EWL0454-OCT16												
Conductivity	2	µS/cm	2		0	10	98	90	110	NA		
pH - QCBatchID: EWL0454-OCT16												
pH	0.05	no unit	NA		1		100			NA		
Redox Potential - QCBatchID: EWL0448-OCT16												
Redox Potential	no	mV	NA		5	20	101	80	120	NA		
Sulphide by SFA - QCBatchID: SKA0251-OCT16												
Sulphide	0.006	mg/L	<0.006		ND	20	94	80	120	90	75	125



Photograph B1 – Rock core sample from Borehole 16-01



Photograph B2 – Rock core sample from Borehole 16-02



Appendix C

Selected Site Photographs



Photograph 1 – Badesdawa River Bridge; Looking Southwest



Photograph 2 – Badesdawa River Bridge; Looking Northeast

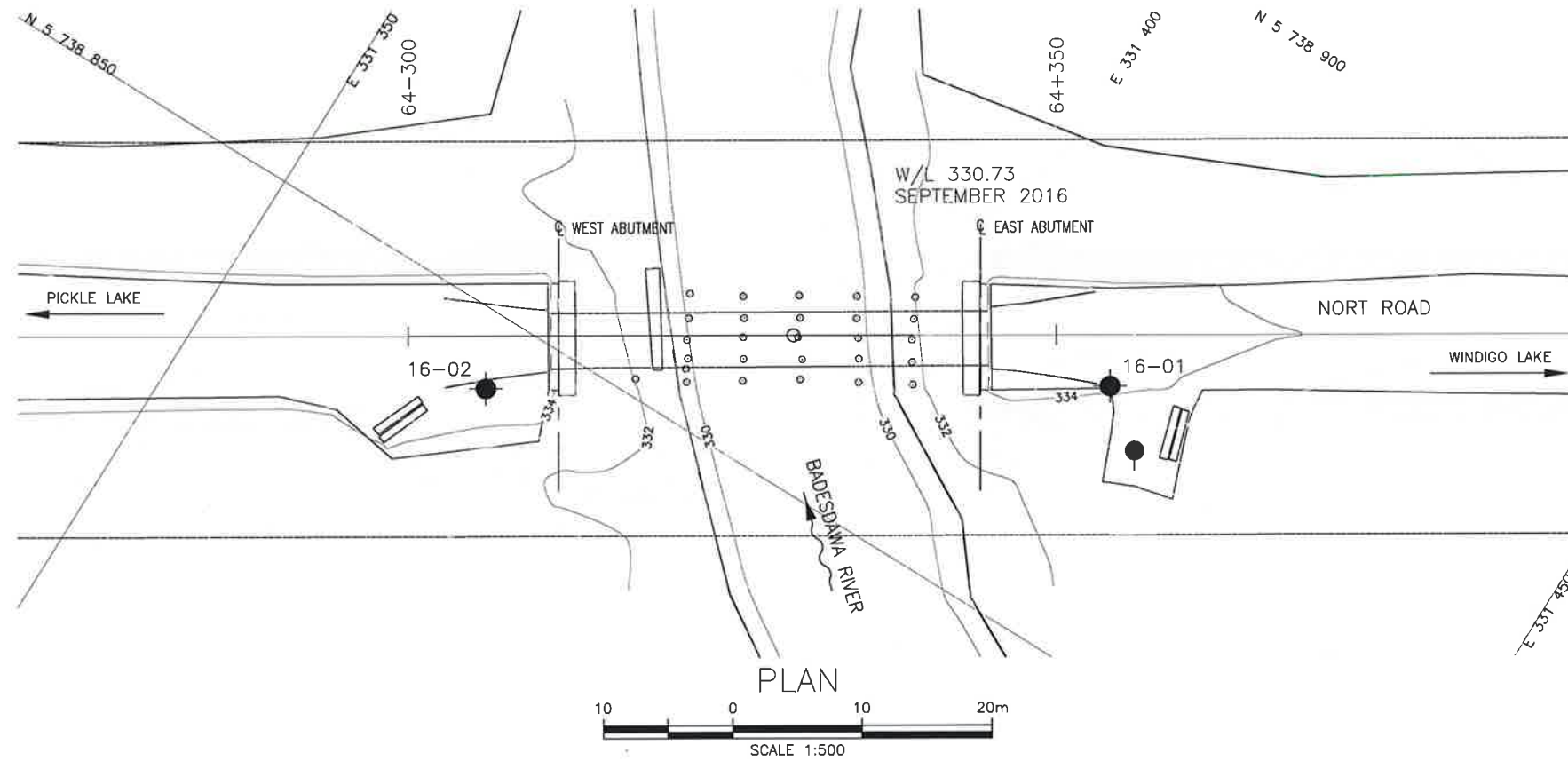


Photograph 3 – Badesdawa River Bridge; Looking East



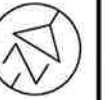
Appendix D

Borehole Locations and Soil Strata Drawing



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 6623-17-00



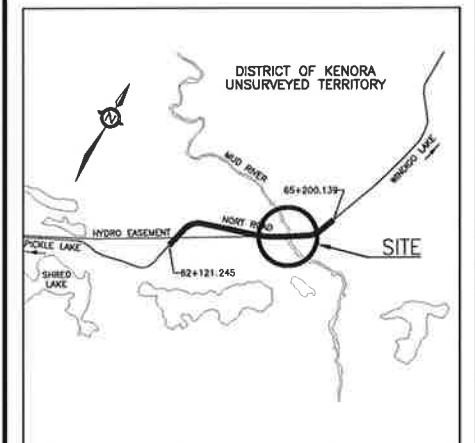
NORT ROAD
BADESDAWA RIVER
BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET |

HATCH








THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

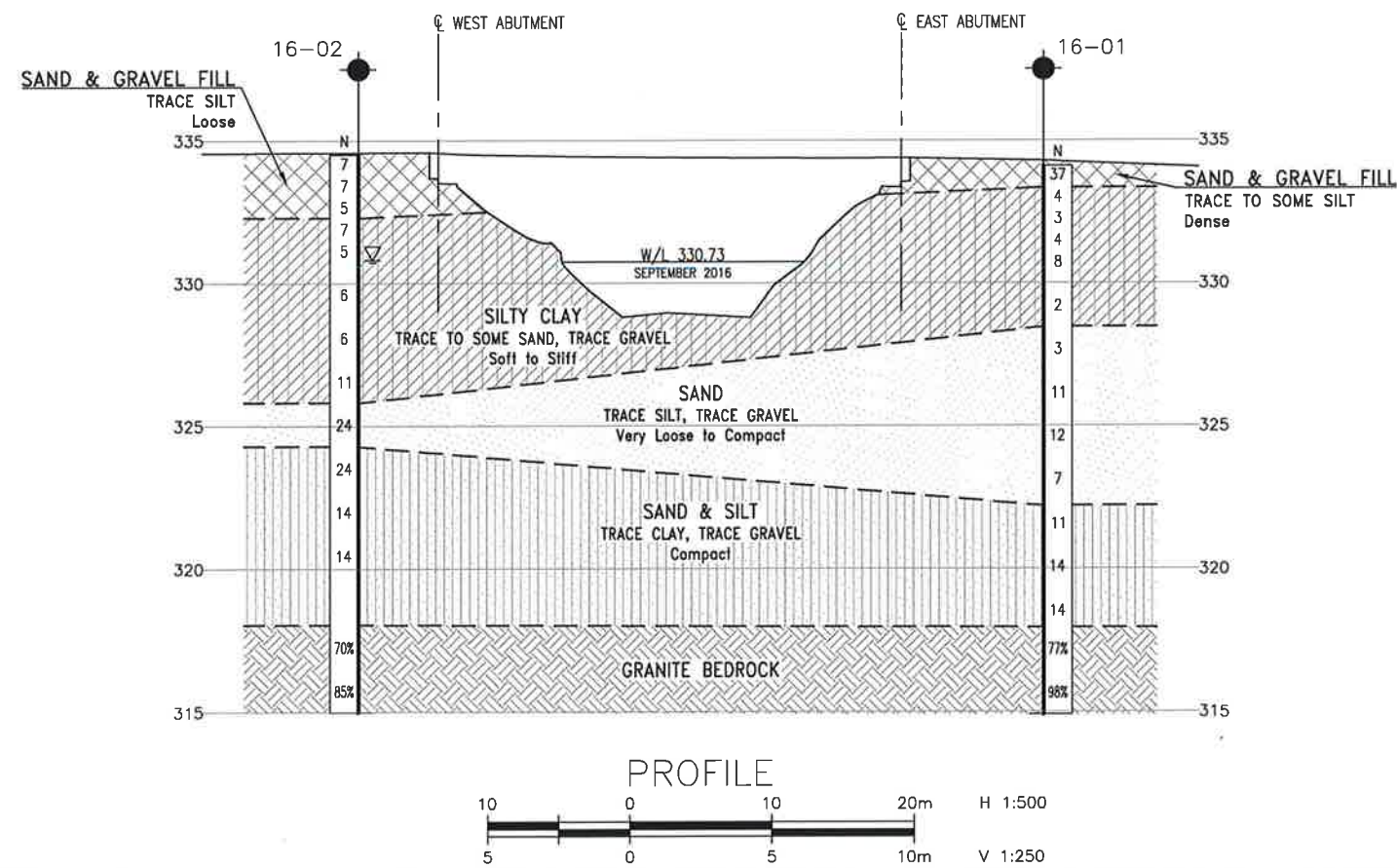
- | | |
|---|---------------------------------------|
|  | Borehole |
|  | Borehole and Cone |
| N | Blows /0.3m (Std Pen Test, 475J/blow) |
| CONE | Blows /0.3m (60° Cone, 475J/blow) |
| PH | Pressure, Hydraulic |
|  | Water Level |
|  | Head Artesian Water |
|  | Piezometer |
| 90% | Rock Quality Designation (RQD) |
| A/R | Auger Refusal |

NO	ELEVATION	NORTHING	EASTING
16-01	334.1	5 738 870.1	331 412.5
16-02	334.5	5 738 844.6	331 371.6

-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- 3) MTM Zone 15 coordinate system used to obtain Borehole Northings and Eastings.

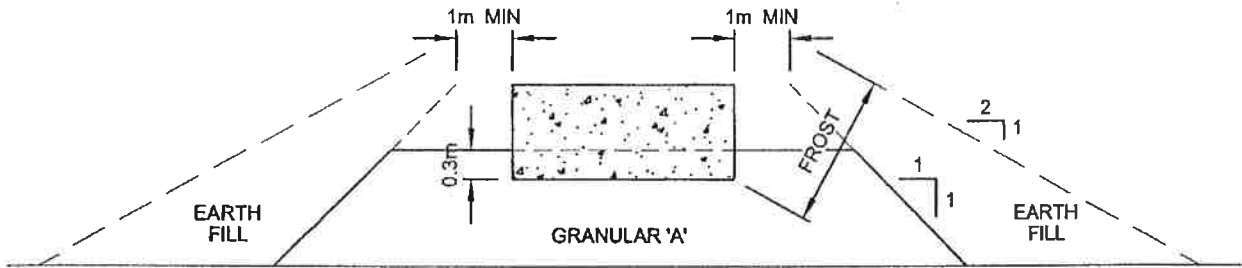
GEOCRES No. 52P-4

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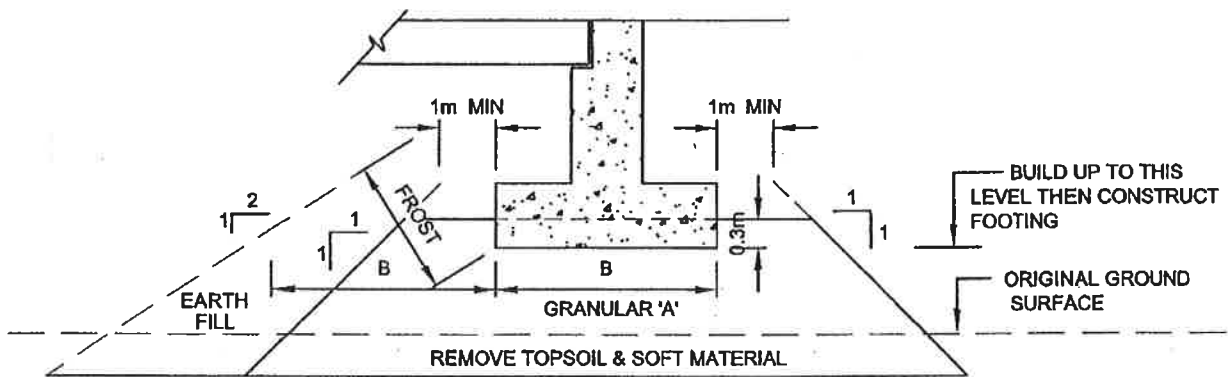


Appendix E

Figure 1 – Abutment on Compacted Fill



CROSS-SECTION



LONGITUDINAL SECTION

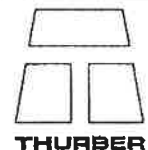
NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ENGINEER	AEG
DRAWN	SS
DATE	April , 2004
APPROVED	PKC
SCALE	NTS

ABUTMENT ON COMPACTED FILL SHOWING
GRANULAR A CORE



DWG. NO.

FIGURE 1