



THURBER ENGINEERING LTD.



**FOUNDATION INVESTIGATION AND DESIGN REPORT
BOSTON CREEK BRIDGE REPLACEMENT
HIGHWAY 564
NEW LISKEARD DISTRICT, ONTARIO
G.W.P. 5130-06-00, SITE NO. 47-021**

GEOCRES No. 32D-20

Report

to

MMM Group Limited

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the existing Boston Creek Bridge along Highway 564, in the District of New Liskeard, 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 MMM Group Limited, under the Ministry of Transportation Ontario (MTO) Agreement Number 5014-E-0019.

2. SITE DESCRIPTION

The existing Boston Creek Bridge is located on Highway 564, approximately 5.8 km east of Highway 112. Built in 1954, the existing structure is a single-span steel bailey bridge with a span length of 18.3 m and a deck width of 4.3 m. The bridge superstructure is supported on rock-filled timber cribs at both east and west abutments. The approach embankment fill is retained by gabion baskets.

Boston Creek flows northerly at the bridge site. Natural ground at both approaches slope downwards towards the creek with the surrounding vegetation characterized by tall grass and shrubs interspersed with frequent trees. The local topography is of low relief with visible bedrock outcrops to the east of the site.

Photographs in Appendix C show the general nature of the site and the existing bridge.

The site lies within the physiographical area of Abitibi Subprovince. Surficial geology at the site is featured by bedrock outcrops and glacio-lacustrine deposits at low-lying areas. The bedrock consists typically of Precambrian mafic to intermediate metavolcanic rocks.

3. INVESTIGATION PROCEDURES

The site investigation and field testing for this project was carried out in three phases with the first from June 9 to June 10, 2015, the second from November 29 to December 10, 2015 and the third on February 19, 2016. A total of four boreholes, identified as BO-01 to BO-04, were drilled in conjunction with Standard Penetration Test (SPT) to depths ranging from 7.6 to 12.2 m below the ground surface. Boreholes BO-01 and BO-02 were drilled adjacent to the existing structure while Boreholes BO-03 and BO-04 were drilled along an alternate alignment to the south of the proposed replacement bridge. Boreholes BO-01, BO-02 and Borehole BO-04 were cored approximately 3.0 m into the bedrock. Borehole BO-03 was terminated upon Dynamic Cone Penetration Test (DCPT) refusal on probable bedrock.

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawings in Appendix D.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling operations. The borehole coordinates and ground surface elevations were estimated from a topographic plan provided to Thurber by MMM Group Limited.

A track-mounted CME 45 drill rig was used to advance Boreholes BO-01 and BO-04 in the overburden using solid stem augers and NW casing/wash boring techniques. A track-mounted D-25 drill rig was used to advance Borehole BO-02 in the overburden using a combination of solid stem augers and NQ coring methods to penetrate the rock fill. Borehole BO-03 was advanced using a portable tri-pod drilling rig in the overburden using NW casing/wash boring techniques. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). NQ coring methods were used to core 3 m into the bedrock in Boreholes BO-01, BO-02 and BO-04.

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

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Groundwater conditions observed after completion of drilling were not representative of site conditions where water was used during wash boring/coring operations. A standpipe piezometer

was installed in Borehole BO-01 to monitor the groundwater level after drilling. The boreholes were backfilled in general accordance with MOE Regulation 903. Completion details of the piezometer and boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Boreholes	Piezometer Installation		Completion Details
	Screen Depth/Elev. (m)	Monitored Stratum	
BO-01	6.1 - 9.1 / 242.4 - 239.4	Sand and Gravel	Cuttings from 12.2 m to 9.1 m, sand from 9.1 m to 5.5 m, bentonite holeplug from 5.5 m to surface.
BO-02	None Installed		Bentonite holeplug and cuttings from 12.2 m to surface.
BO-03	None Installed		Bentonite holeplug and cuttings from 7.6 m to surface.
BO-04	None Installed		Bentonite holeplug and cuttings from 10.7 m to surface.

4. LABORATORY TESTING

All recovered soil samples were subjected to visual identification and natural moisture content determination. Selected samples were subjected to grain size distribution analyses (sieve and hydrometer). The results of this testing program are summarized on the Record of Borehole sheets included in Appendix A and on the figures presented in Appendix B.

In order to assess the potential for sulphate attack on foundation concrete and the potential for corrosion, a sample of the native soil and a sample of the surface water from the creek upstream of the bridge were collected. The samples were submitted to AGAT Laboratories in Mississauga, Ontario for analytical testing of corrosivity parameters and sulphate content. The results of the analytical testing are summarized in Section 6 below and are presented 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” drawings included in Appendix D. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions. It must be recognized that soil conditions may vary between and beyond the borehole locations.

The soil stratigraphy below the embankment fill typically comprises a discontinuous surficial layer of silty clay underlain by a sand and gravel deposit overlying sandy silt to silt above granitic bedrock. The sand and gravel deposit extends to bedrock in BO-01 near east abutment of the existing bridge. More detailed descriptions of individual strata are presented below.

5.1 Topsoil

Topsoil was encountered in Boreholes BO-01 to BO-03. The thickness of the topsoil ranged between 40 and 50 mm. Topsoil thickness may vary between and beyond borehole locations.

5.2 Embankment Fill

Sand and gravel fill containing cobbles was encountered below the topsoil in Boreholes BO-01 and BO-02. Rock fill was encountered below the sand and gravel fill in BO-02. The fill thickness ranged between 2.1 and 2.7 m with the lower boundary at Elev. 246.3 and 244.4. The thickness of the rock fill encountered in BO-02 was 1.2 m and was penetrated by coring.

SPT-N values recorded in the sand and gravel fill ranged typically from 28 to 44 blows per 0.3 m of penetration with exception of one value of 11 blows, indicating a compact to dense relative density. Moisture contents of the sand and gravel fill ranged from 4 to 6%.

The results of grain size analyses conducted on a sample of the fill is provided on the Record of Borehole sheets in Appendix A and plotted in Figure B1 of Appendix B. The results are summarized below.

Soil Particle	Percentage (%)
Gravel	46
Sand	52
Silt & Clay	2

5.3 Silty Clay

A thin layer of brown to grey silty clay with some sand was encountered below the sand and gravel fill in BO-01 and below the topsoil in BO-03. The silty clay contains occasional cobbles and trace roots and rootlets. The thickness of the silty clay ranged between 0.5 and 1.5 m with the lower boundary at Elev. 245.3 and 244.8.

SPT-N values recorded in the silty clay ranged from 3 to 24 blows per 0.3 m of penetration, indicating a soft to very stiff consistency. Natural moisture contents of the silty clay ranged between 9 and 33%.

The results of grain size analyses conducted on a sample of the silty clay is provided on the Record of Borehole sheets in Appendix A and plotted in Figure B2 of Appendix B. The results are summarized below.

Soil Particle	Percentage (%)
Gravel	0
Sand	39
Silt	30
Clay	31

5.4 Sandy Gravel to Sand and Gravel

A native sandy gravel to sand and gravel deposit was encountered below the silty clay in Boreholes BO-01 and BO-03, underlying the fill in Borehole BO-02 and at the surface in Borehole BO-04. The deposit contains occasional cobbles and boulders and trace silt. The thickness of the layer ranged from 1.4 to 5.6 m with the lower boundary at Elev. 243.0 to 239.2.

SPT-N values recorded in the deposit ranged from 30 to over 100 blows per 0.3 m of penetration, indicating a dense to very dense relative density. Natural moisture contents of the cohesionless deposit ranged from 10 to 21%.

The results of grain size analyses conducted on samples of the deposit are provided on the Record of Borehole sheets in Appendix A and plotted in Figure B3 of Appendix B. The results are summarized below.

Soil Particle	Percentage (%)
Gravel	57 to 75
Sand	22 to 37
Silt & Clay	3 to 6

5.5 Sandy Silt to Silt

A compact sandy silt to silt deposit was encountered below the sandy gravel to sand and gravel in Boreholes BO-02, BO-03 and BO-04. This grey layer contained trace to some clay and gravel. Borehole BO-03 was terminated within the sandy silt upon refusal on probable bedrock at Elev. 238.3. The thickness of the deposit ranged between 4.7 and 5.0 m, with the lower boundaries at Elev. 238.2 and 238.0.

SPT-N values recorded in the deposit ranged from 4 to 120 blows and typically 4 to 23 blows per 0.3 m of penetration, indicating a loose to compact relative density. The SPT-N value of 120 blows per 0.3 m penetration was recorded immediately above the refusal in Borehole BO-03. Natural moisture contents ranged from 18 to 39%.

The results of grain size analyses conducted on samples of the deposit are provided on the Record of Borehole sheets in Appendix A and plotted in Figure B4 of Appendix B. The results are summarized below.

Soil Particle	Percentage (%)
Gravel	0 to 27
Sand	11 to 32
Silt	46 to 76
Clay	4 to 13

5.6 Bedrock

Granite bedrock was encountered in Borehole BO-01 below the sandy gravel to sand and gravel, and underlying the sandy silt to silt in Boreholes BO-02 and BO-04, and was proven by coring. Borehole BO-03 was terminated upon auger refusal on probable bedrock. Table 5.1 summarizes the depth to bedrock and the bedrock surface elevations determined in the boreholes.

Table 5.1 – Depth to Bedrock at Borehole Locations

Borehole	Depth to Bedrock (m)	Top of Bedrock Elevation (m)	Comment
BO-01	9.3	239.2	Core 3 m
BO-02	9.1	238.0	Core 3 m
BO-03	7.8 (Inferred)	238.1 (Inferred)	DCPT Refusal
BO-04	7.6	238.2	Core 3 m

The bedrock is generally described as moderately weathered to fresh and red in colour with trace grey and white. Total Core Recovery (TCR) in the bedrock ranged from 95 to 100% with solid core recovery (SCR) ranging from 33 to 98%. The Rock Quality Designation (RQD) determined from the recovered cores generally ranged from 31 to 93%, indicating poor to excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to greater than 25. Unconfined compressive strengths (UCS) of the rock ranged between 156 and 263 MPa based on correlations with the point load tests (PLT), indicating very strong to extremely strong rock strength.

5.7 Groundwater Conditions

Where possible, water levels were monitored in the open boreholes during drilling operations. Wash boring and/or coring methods were used to advance all boreholes and therefore water levels recorded during or upon completion of drilling may not reflect natural groundwater levels. A standpipe piezometer was installed in Borehole BO-01 to monitor the groundwater level after

completion. The water level measured in the piezometer and in open boreholes are shown in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
BO-01	June 15, 2015	2.6	245.9	In piezometer
	June 27, 2015	1.8	246.7	
BO-02	November 30, 2015	2.4	244.7	In Open Borehole

In the preliminary GA provided by MMM, the creek level at the bridge location was reported at Elev. 245.6 on June 3, 2015. The water levels measured in the piezometer are 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.

6. CORROSIVITY AND SULPHATE TEST RESULTS

A sample of the native sandy silt to silt and a sample of the surface water from the Boston creek were tested for corrosivity parameters and sulphate content. The results of the analytical tests are shown 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	
			BO-04 SS#5	Creek Water
Sulphide	%	mg/L	0.14	< 0.05
Chloride	µg/g	mg/L	83	1.94
Sulphate	µg/g	mg/L	44	14.5
pH	pH Units	pH Units	8.02	7.52
Electrical Conductivity	mS/cm	µS/cm	0.238	162
Resistivity	ohm.cm	ohm.cm	4200	6170
Redox Potential	mV	mV	312	316
Langlier Index	-	-	-	-0.55
Total Hardness (as CaCO ₃)	-	mg/L	-	80.3
Total Dissolved Solids	-	mg/L	-	104
Alkalinity (as CaCO ₃)	-	mg/L	-	63
Calcium	-	mg/L	-	23.2
Magnesium	-	mg/L	-	5.44

7. MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. The coordinates and the ground surface elevations for the boreholes were established based on topographic survey information provided by MMM Group Limited.

Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling of Hawkesbury, Ontario supplied a track-mounted CME-45 drill rig, a track-mounted D-25 drill rig and a tripod drill rig, and conducted the drilling, sampling and in-situ testing operations for the boreholes. The drilling operations were supervised by Mr. Amir Fereidouni and Mr. George Azzopardi of Thurber.

Overall supervision of the field program and interpretation of the data were carried out by Mr. Stephane Loranger, CET and Ms. Deanna Pizycki, EIT.

The report was prepared by Mr. Keli Shi, P.Eng., and reviewed by Mr. Alastair Gorman, M.Sc., P.Eng., and 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 geotechnical recommendations to assist the design team in selecting and designing a suitable foundation system for the proposed replacement structure for Boston Creek Bridge on Highway 564 located in the District of New Liskeard, 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. The 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.

At present, Highway 564 crosses the Boston Creek on a single-span steel bailey bridge with a span length of 18 m and a deck width of 4 m accommodating one lane of traffic. The bridge superstructure is supported on rock-filled timber cribs at both east and west abutments. The approach embankment fill is retained by gabion baskets. The existing road grade on the bridge is at approximate Elev. 249 m.

Three alternatives were developed for the bridge replacement during the preliminary design stage. A general description of each of the three alternatives is as follows:

- Alternative 1 alignment shifted the highway alignment approximately 8.5 m south of the existing structure. This offset provides sufficient room to construct the new bridge while maintaining traffic on the existing structure.

- Alternative 2 alignment relocated Highway 564 to fit within a new Right-of-Way that MTO had previously acquired but not designated. This alternative alignment is approximately 25 m south of the existing bridge and the realignment is roughly 400 m in length.
- Alternative 3 alignment maintains the existing highway alignment with a slight grade raise.

Alternative 2 was eliminated due to its high costs. Alternative 1 has slightly higher costs than Alternative 3 but does not require a full closure of the roadway. Due to the above reasons, Alternative 1 was selected for detailed design.

The proposed replacement structure will be a single-span modular bridge with a 27.4 m long span and 5.25 m wide deck. The preliminary GA drawing indicates that the abutments are proposed to be founded on shallow spread footings.

9. STRUCTURE FOUNDATIONS

In general, the foundation soil stratigraphy consists of a layer of soft to very stiff silty clay underlain by dense to very dense sand and gravel at the east abutment and a relatively thin layer of dense sand and gravel underlain by compact sandy silt at the west abutment. A 1.2 m thick layer of rock fill is present at the west abutment.

The creek level in the preliminary GA was shown at Elev. 245.6 m on June 3, 2015. The 2-year storm level is reported at Elev. 246.1 m.

Based on the subsurface conditions, initial consideration was given to supporting the replacement bridge on spread footings on native soil or engineered fill or on steel H-piles driven to bedrock.

Spread footings founded on engineered granular pad are not recommended at this site since the pads will be constructed close to the creek banks and during high water level events, there is a risk that the sand fraction and finer grain sizes may get washed out through the rock protections. Accordingly, a footing on engineered rock fill is recommended which will minimize this concern.

Recommendations for design of the feasible foundation alternatives are presented in the following sections along with the corresponding geotechnical design parameters. A preferred foundation scheme from a geotechnical perspective is recommended.

9.1 Spread Footings on Native Soil

The existing embankment fill is underlain by soft to very stiff silty clay at the east abutment and by dense to very dense sand and gravel at the west abutment. Given the significant difference in strength and compressibility between the native silty clay and sand and gravel, spread footings

placed directly on the native soils would potentially result in large differential settlement between the two abutments and are therefore not recommended.

Placement of spread footing on dense to very dense sand and gravel would require excavation extending 0.3 to 0.8 m below the normal creek level at the east abutment. Dewatering and temporary protection system would be required to construct the footing in the dry. Although technically feasible, this option is not considered to be cost effective, and therefore, not recommended.

9.2 Spread Footings on Engineered Rock Fill Pad

9.2.1 Founding Levels

The preliminary GA drawing indicates that the new spread footings will be founded at approximately Elevation 248 m at both west and east abutments. Placement of new fill next to the existing embankment will be required to establish the founding level across the footprint of the footings. Support of footings partially on new fill and partially on existing fill is not recommended considering unknown variability in the existing fill and the need to provide a uniform subgrade condition for the new footings.

A modular bridge supported on concrete spread footings placed on a minimum 2 m thick rock fill pad can be considered at this site. The preliminary GA drawing indicates the finished road grade at Elev. 249.3 m at the east abutment and Elev. 249.0 m at the west abutment. It shows the base of the engineered rock fill pad located at approximate Elev. 245.6 m or the normal creek level. At that elevation, the engineered rock fill pad will be constructed on the sand and gravel and the existing rock fill at the west abutment and on the firm to very stiff silty clay at the east abutment.

9.2.2 Engineered Rock Fill Construction

The engineered rock fill pads should consist of well graded and freshly produced rock fill having a maximum size of 250 mm. A sketch of the abutment footing placed on rock fill pad is presented on Figure 1 enclosed in Appendix F.

Excavations for the engineered rock fill pad construction will require the existing embankment fill and gabion baskets to be partially removed. Suggested wording for an NSSP on the construction of the engineered rock fill pad is included in Appendix E. The following construction sequence may be considered:

1. Excavate to remove all deleterious material from the footprint of the new foundation;

2. The minimum depth of excavation must accommodate the concrete foundation slab and the thickness of engineered rock fill pad below the slab;
3. The subgrade for the engineered rock fill pad should be inspected and all organics, soft/loose soils, and any deleterious materials should be removed from the footprint of the excavation. Dewatering measures should be provided, as required, to place the engineered rock fill in the dry;
4. The dimensions of the base of the excavation should be determined by assuming a pad 1.0 m wider than the footing at the level of the footing base and projecting outward and downward no steeper than 1.5H: 1V.

9.2.3 Factored Geotechnical Resistance and Geotechnical Reaction

Geotechnical resistances were estimated for 1.5 m and 2.0 m wide footings founded on 2 m thick engineered rock fill pads placed at various distances from the top of the header (creek facing) slope. The base of the footing is assumed to be embedded 0.5m below the top of rock fill erosion protection placed over the header slope.

The calculated geotechnical resistances at the factored ULS and the SLS reaction are provided in Table 9.1. The geotechnical resistances at the SLS correspond to footing settlement not exceeding 25 mm.

The geotechnical resistances are based on a spread footing subjected to vertical, concentric loading. In the case of eccentric and inclined loading, the geotechnical resistances must be calculated according to the CHBDC.

Table 9.1 – Geotechnical Resistances for Spread Footings

Footing Width (B)	Distance of Edge of Footing to Top of Rock Fill Slope (b)					
	b = 0.5 m		b = 0.75 m		b = 1.0 m	
	Factored ULS (kPa)	SLS (kPa)	Factored ULS (kPa)	SLS (kPa)	Factored ULS (kPa)	SLS (kPa)
B = 1.5 m	270	200	300	200	335	200
B = 2.0 m	300	170	330	170	360	170

The value of a Factored Geotechnical Resistance at ULS was assessed assuming a Consequence Factor of 1.0 (Typical), and a Resistance Factor of 0.5 (Typical), 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 provided above is for concentric, vertical loading conditions 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.

The lateral resistance of the footings founded on engineered rock fill pad may be computed using an unfactored friction coefficient of 0.45.

9.3 Driven H-Pile Foundations

The ground conditions at this site are considered to be suitable for the use of steel H-piles driven to refusal on bedrock.

For HP 310x110 piles driven to refusal on granitic bedrock, a factored geotechnical resistance at ULS of 2,000 kN per pile should be used for design. The geotechnical resistance at SLS does not govern for piles driven to bedrock.

The estimated tip elevations of piles driven to the bedrock surface, are presented in Table 9.2.

Table 9.2 – Estimated Pile Tip Elevation for Driven H-Piles

Foundation Element	Borehole	Estimated Pile Tip Elevation (m)
East Abutment	BO-01	239.2
West Abutment	BO-02	238.0

Pile tip protection is recommended for driven H-piles to prevent pile damage when setting the piles on bedrock or if cobbles or boulders are encountered. The tips of all driven H-piles must be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

Pile installation must be in accordance with OPSS.PROV 903. The appropriate pile driving note is “Piles to be driven to bedrock”.

Cobbles, boulders and/or rock fill may be encountered when driving piles through the existing fill and native soil. The Contract Documents should contain an NSSP alerting bidders to the presence of the cobbles, boulders and rock fill within the existing embankment and in the foundation soil. Suggested wording for an NSSP addressing this issue is included in Appendix E.

9.4 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, spread footings on engineered rock fill pads are considered suitable foundation options at this site.

9.5 Frost Cover

The depth of frost penetration at this site is approximately 2.3 m. It is recommended that the base of the footings or pile caps be provided with a minimum 2.3 m of earth cover as protection against frost action.

In the case of spread footings or sleeper slabs used under a modular bridge, it is considered acceptable to found these on granular pads with a minimum embedment of 0.5 m.

10. EXCAVATION AND DEWATERING

The creek level was reported at Elev. 245.6 on June 3, 2015. Groundwater levels measured in standpipe piezometer in BO-01 were at elevation 245.9 and 246.7 on June 15 and 27, 2015, respectively. Where excavations penetrate below the water level, the Contractor must implement effective dewatering procedures. Design of a suitable dewatering scheme is the responsibility of the Contractor.

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill may be classified as Type 3 soil above the water table and as Type 4 soil below the water table. Flatter slopes may be required at locations where water seepage affects surficial stability.

The excavation and backfilling for foundations must be carried out in accordance with OPSS 902.

The selection of the method of excavation is the responsibility of the Contractor and must be based on his equipment, experience and interpretation of the site conditions. It is anticipated that a hydraulic excavator will be suitable. Provision must be made for the handling of potential obstructions in the fill, cobbles and boulders, and rock fill.

11. TEMPORARY SUPPORT SYSTEM

If required, the temporary excavation support system must be designed and constructed to Performance Level 2 in accordance with OPSS.PROV 539. The Contractor should select the wall type and design taking into account the soil conditions encountered in the boreholes.

The following parameters apply for design of the temporary shoring system:

$$\begin{aligned}\gamma &= 21 \text{ kN/m}^3 && \text{(bulk unit weight for fill)} \\ &= 20 \text{ kN/m}^3 && \text{(bulk unit weight for native silty clay)}\end{aligned}$$

γ'	=	11 kN/m ³	(submerged unit weight for fill)
	=	10 kN/m ³	(submerged unit weight for native silty clay)
K_a	=	0.31	(active pressure coefficient for fill)
	=	0.37	(active earth pressure coefficient for native silty clay)
K_p	=	3.3	(passive pressure coefficient for fill)
	=	2.7	(passive earth pressure coefficient for native silty clay)

The short-term groundwater level to be used for design of the temporary shoring may be assumed at elevation 246.0 m.

The actual pressure distribution acting on the shoring system is a function of construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system.

The design of temporary shoring system should be the responsibility of the contractor. All shoring systems should be designed by a Professional Engineer experienced in such design.

12. LATERAL EARTH PRESSURES

Earth pressures acting on the structure may be assumed to be distributed triangularly and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K (\gamma h + q)$$

Where: p_h = horizontal pressure on the wall at depth h (kPa)

K = coefficient of lateral earth pressure (see Table 12.1)

γ = unit weight of retained soil (see Table 12.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 12.1.

Table 12.1 – Coefficients of Lateral Earth Pressure (K)

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 or Type III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active K_A (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At-rest K_0 (Restrained Wall)	0.43	-	0.47	-
Passive K_P	3.7	-	3.3	-

* For wing walls

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 acting on the wall.

The active and passive earth pressure coefficients in Table 12.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code (CHBDC).

In accordance with the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 1.7 m for Granular B Type I or Type III, or at a depth of 2.0 m for Granular A or Granular B Type II.

13. SEISMIC CONSIDERATIONS

According to Clause 4.4.4 of the CHBDC, an earthquake with a 2475-year return period or 2% probability of exceedance in 50 years should be used for seismic design. The peak ground acceleration (PGA) associated with the design earthquake is 0.102 g for Site Class C.

Based on the encountered soil conditions, this site is assessed to be Site Class C for seismic site response according to Table 4.1 of the CHBDC. The above PGA value should be modified by a site coefficient of 1.00 based on Table 4.8 of the CHBDC.

In accordance with Clause 4.6.5 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

For the design of retaining walls under seismic loading, the coefficients of horizontal earth pressure in Table 13.1 may be used:

Client: MMM Group Ltd.

Date: June 16, 2017

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Table 13.1 – Earth Pressure Coefficient for Earthquake Loading (K_E)

Loading Condition	Granular A or Granular B Type II $f' = 35^\circ$; $g = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I or Type III $f' = 32^\circ$; $g = 21.2 \text{ kN/m}^3$
Active (K_{AE}) *	0.31	0.35
At-rest (K_{0E}) **	0.56	0.61
Passive (K_{PE})	3.6	3.1

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods (1973).

14. APPROACH EMBANKMENTS

The preliminary GA drawings indicate fill placement will be required at the bridge approaches. The heights of the new approach embankments above the existing ground at the abutments vary from 0.9 to 3.1 m at the west abutment and from 0.8 to 3.1 m at the east abutment as the proposed replacement alignment is close to the existing approach embankments.

Based on the encountered soil conditions, the ground settlements at the bridge abutments are estimated to be less than 25 mm. The estimated settlements will be essentially complete at the end of construction.

Global stability of the approach embankments near the creek valley slopes is not considered to be an issue given the encountered ground conditions.

15. SCOUR AND EROSION PROTECTION

Erosion protection should be provided along any soil surfaces that may be in contact with the creek flow.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS.PROV 804.

16. CORROSION & SULPHATE ATTACK POTENTIAL

The results of the corrosivity and sulphate analytical tests conducted on the native sandy silt to silt and the creek water indicate the following:

- The potential for sulphate attack on concrete foundations from the surrounding soil or surface water is considered to be negligible due to the low concentration of sulphate in the samples tested.

- The potential for corrosion on metal structural elements is considered to be moderate to mild.
- Appropriate protection measures should be taken to address the moderate to mild potential for corrosion on metal structural elements.

17. CONSTRUCTION CONCERNS

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

- Occasional cobbles and boulders were noted in the existing embankment fill and native soils; some of the boulders required coring to advance the borehole. Cobbles and boulders may interfere with excavations, installation of temporary protection system and driving of piles.
- The Contractor's attention must be drawn to the presence of rock fill at the west abutment as this may impede the installation of piles unless it is first excavated. Alternatively, the Contractor may elect to drill through the rock fill.
- Seasonal fluctuations of the groundwater and creek 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, which may impact the construction.

18. CLOSURE

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

Thurber Engineering Ltd.



Keli Shi, M.Eng., P.Eng.
Senior Foundations Engineer

Alastair Gorman, M.Sc., P.Eng.
Senior Associate, Senior Foundations 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


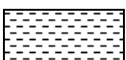

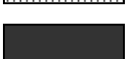

ROCK WEATHERING CLASSIFICATION

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 (MPa)	Approximate Uniaxial Compressive Strength (psi)	Field Estimation of Hardness*
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			

RECORD OF BOREHOLE No BO-01

1 OF 2

METRIC

GWP# 5130-06-00 LOCATION Boston Creek Bridge N 5 319 589.0 E 383 222.5 ORIGINATED BY AHF
 HWY 564 BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY MFA
 DATUM Geodetic DATE 2015.06.09 - 2015.06.10 CHECKED BY MEF

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 (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
248.5	GROUND SURFACE							20	40	60	80	100					
0.9	TOPSOIL: (50mm)							20	40	60	80	100					
	Silty SAND and GRAVEL , occasional cobbles, trace roots and rootlets Compact to Dense Brown Moist (FILL)		1	SS	11		248										
			2	SS	28												
	boulder fragments Reddish Grey		3	SS	35		247										
246.3																	
2.2	Silty CLAY , some sand, trace gravel, occasional cobbles, trace roots and rootlets Soft to Very Stiff Grey Wet		4	SS	3		246										0 39 30 31
			5	SS	24												
244.8							245										
3.7	SAND and GRAVEL , trace silt, occasional cobbles and boulders Dense to Very Dense Grey Wet		6	SS	68		244										
							243										
			7	SS	95/ 0.100		242										
							241										
			8	SS	30		240										
239.2			9	SS	82/ 0.075		239										
9.3	GRANITE strong to extremely strong, greyish red highly fractured zones at 9.32 to 9.53 and 10.01 to 10.16m																RUN #1 TCR=100% SCR=68% RQD=55%

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BO-01

2 OF 2

METRIC

GWP# 5130-06-00 LOCATION Boston Creek Bridge N 5 319 589.0 E 383 222.5 ORIGINATED BY AHF
 HWY 564 BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY MFA
 DATUM Geodetic DATE 2015.06.09 - 2015.06.10 CHECKED BY MEF

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 (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			
								20	40	60	80						100	20	40	60
	Continued From Previous Page		1	RUN											>25	RUN #2 TCR=100% SCR=48% RQD=31%				
	subvertical fractures at 9.53 to 9.55, 9.58 to 9.63 and 9.83 to 9.93m														0					
	horizontal fracture at 10.59m subhorizontal fractures at 10.72 to 10.74 and 10.77 to 10.80m subvertical fractures at 10.90 to 11.13, 11.23 to 11.25, 11.28 to 11.30, 11.33 to 11.35, 11.73 to 11.79, 11.91 to 11.94 and 11.96 to 12.09m highly fractured zones at 11.13 to 11.18 and 12.14 to 12.19m horizontal fractures at 11.58 and 11.73m		2	RUN										1						
	vertical fractures at 11.66 to 11.73 and 12.06 to 12.14m														5					
															6					
236.3															3					
12.2															4					
	END OF BOREHOLE AT 12.2m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) 2015.06.15 2.6 245.9 2015.06.27 1.8 246.7																			

RECORD OF BOREHOLE No BO-02

1 OF 2

METRIC

GWP# 5130-06-00 LOCATION Boston Creek Bridge N 5 319 587.0 E 383 193.5 ORIGINATED BY GA
 HWY 564 BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2015.11.29 - 2015.11.30 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
247.1	GROUND SURFACE						20	40	60	80	100					
0.0	TOPSOIL: (40mm)		1	SS	30											46 52 2 (SI+CL)
	SAND and GRAVEL, trace silt, occasional cobbles Dense Brown Moist (FILL)		2	SS	44											
245.6	ROCKFILL, mixed with sand															
1.5																
244.4	SAND and GRAVEL, trace silt Dense Grey Wet		3	SS	34											
2.7																
243.0	Sandy SILT, trace to some clay, occasional cobbles Compact Grey Wet		4	SS	11											0 27 64 9
4.1																
			5	SS	11											
			6	SS	23											
	Coring from 8.5m															
238.0	GRANITE fresh, coarse grained, strong to extremely strong, red/black/white		1	RUN												RUN #1 TCR=100% SCR=77% RQD=72% UCS=263MPa (Average)
9.1																

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BO-02

2 OF 2

METRIC

GWP# 5130-06-00 LOCATION Boston Creek Bridge N 5 319 587.0 E 383 193.5 ORIGINATED BY GA
 HWY 564 BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2015.11.29 - 2015.11.30 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W P	W	W L					
								SHEAR STRENGTH kPa			WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE								
							● QUICK TRIAXIAL × LAB VANE									
	Continued From Previous Page						237							0	RUN #2 TCR=100% SCR=98% RQD=93% UCS=156MPa (Average)	
	Highly broken zone from 9.1m to 9.5m													0		
	Horizontal joint at 9.4m, 9.5m and 9.7m													0		
	Horizontal joint at 11.0m						236							0		
	Sub-horizontal joint at 10.9m and 11.1m		2	RUN										0		
	Highly broken zone at 11.0m													0		
234.9							235							0		
12.2	END OF BOREHOLE AT 12.2m BOREHOLE OPEN TO 12.2m AND WATER LEVEL AT 2.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.															

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BO-03

1 OF 1

METRIC

GWP# 5130-06-00 LOCATION Boston Creek Bridge N 5 319 565.7 E 383 220.1 ORIGINATED BY AHF
 HWY 564 BOREHOLE TYPE Tripod COMPILED BY AN
 DATUM Geodetic DATE 2015.12.10 - 2015.12.10 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						
245.9	GROUND SURFACE													
0.0	TOPSOIL: (50mm)		1	SS	8									
245.3	Silty CLAY , some sand, trace gravel, trace roots and rootlets Firm Grey Wet		2	SS	76/ 0.250		245							
0.6	SAND and GRAVEL , trace silt, occasional cobbles Dense to Very Dense Brown to Grey Wet		3	SS	132		244						57 37 6 (SI+CL)	
			4	SS	45		243							
242.9			5	SS	15		242							
3.0	Sandy SILT , some gravel, trace clay, occasional cobbles Compact to Very Dense Grey Wet		6	SS	15		241							
			7	SS	10		240						16 32 48 4	
			8	SS	11		239							
			9	SS	120								27 22 46 5	
238.3														
7.6	END OF BOREHOLE AT 7.6m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE. DCPT REFUSAL AT 7.8m UPON PROBABLE BEDROCK.													

ONTMT4S 19-5161-251.GPJ 2015TEMPLATE(MTO).GDT 8/15/16

+³, ×³: Numbers refer to
Sensitivity




20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BO-04

1 OF 2

METRIC

GWP# 5130-06-00 LOCATION Boston Creek Bridge N 5 319 563.7 E 383 189.6 ORIGINATED BY AHF
 HWY 564 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2016.02.19 - 2016.02.19 CHECKED BY DJP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
245.8	GROUND SURFACE							20	40	60	80	100			
0.0	Sandy GRAVEL to SAND and GRAVEL , trace silt, occasional cobbles and boulders Dense to Very Dense Grey Wet Cobbles (225mm) at 1.2m		1	SS	31		245								75 22 3 (SI+CL)
			2	SS	50/ 0.075										
			1	NQ	-		244								
			3	SS	39		243								
242.9															
2.9	Sandy SILT to SILT , some sand, trace to some clay Loose to Compact Grey Wet		4	SS	11		242								
			5	SS	6		241								
			6	SS	5		240								
			7	SS	4		239								0 11 76 13
238.2															
7.6	GRANITE with trace quartz, very to extremely strong, red/grey/white Highly broken zone 150mm at 7.6m and 350mm at 8.0m Sub-vertical fracture at 50mm at 7.6m and 150mm at 8.4m 75mm vertical fracture at 8.2m Sub-horizontal fracture (25mm to 50mm) at 7.8m, 8.0m and 8.2m Horizontal fracture (25mm) at 7.9m, 8.0m, 8.1m 8.4m, 8.6m and 8.7m Sub-horizontal fracture (25mm to 75mm) at 9.5m, 9.6m and 9.9m		1	RUN			238								RUN #1 TCR=95% SCR=33% RQD=32% UCS=244MPa (Average)
			2	RUN			237								RUN #2 TCR=98% SCR=59% RQD=57% UCS=240MPa (Average)
							236								

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No BO-04

2 OF 2

METRIC

GWP# 5130-06-00 LOCATION Boston Creek Bridge N 5 319 563.7 E 383 189.6 ORIGINATED BY AHF
 HWY 564 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2016.02.19 - 2016.02.19 CHECKED BY DJP

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									
	Continued From Previous Page																
	75mm horizontal fracture at 10.2m																
	275mm highly broken zone at 10.4m																
235.1																	
10.7	END OF BOREHOLE AT 10.7m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.																

ONTMT4S 19-5161-251.GPJ 2015TEMPLATE(MTO).GDT 8/15/16



Appendix B

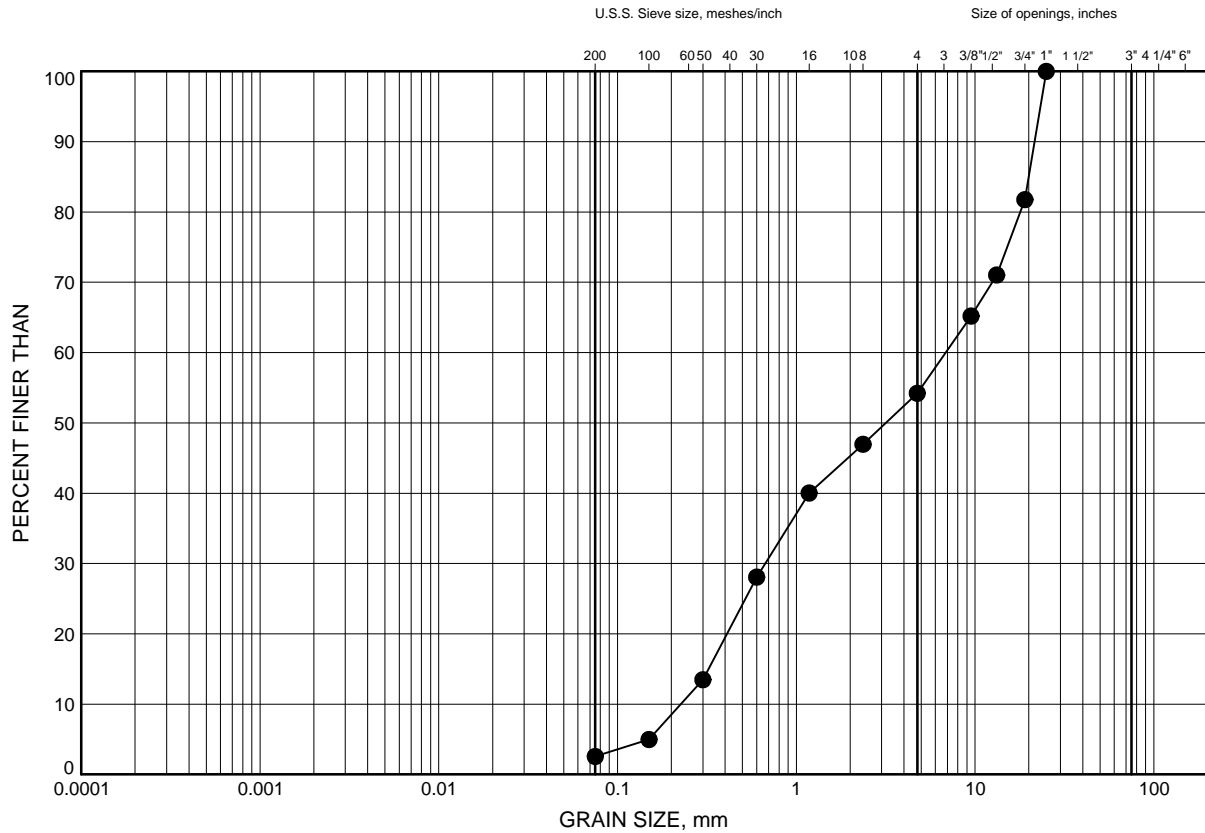
Laboratory Test Results

Boston Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND & GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BO-02	0.30	246.80

Date March 2016
GWP# 5130-06-00



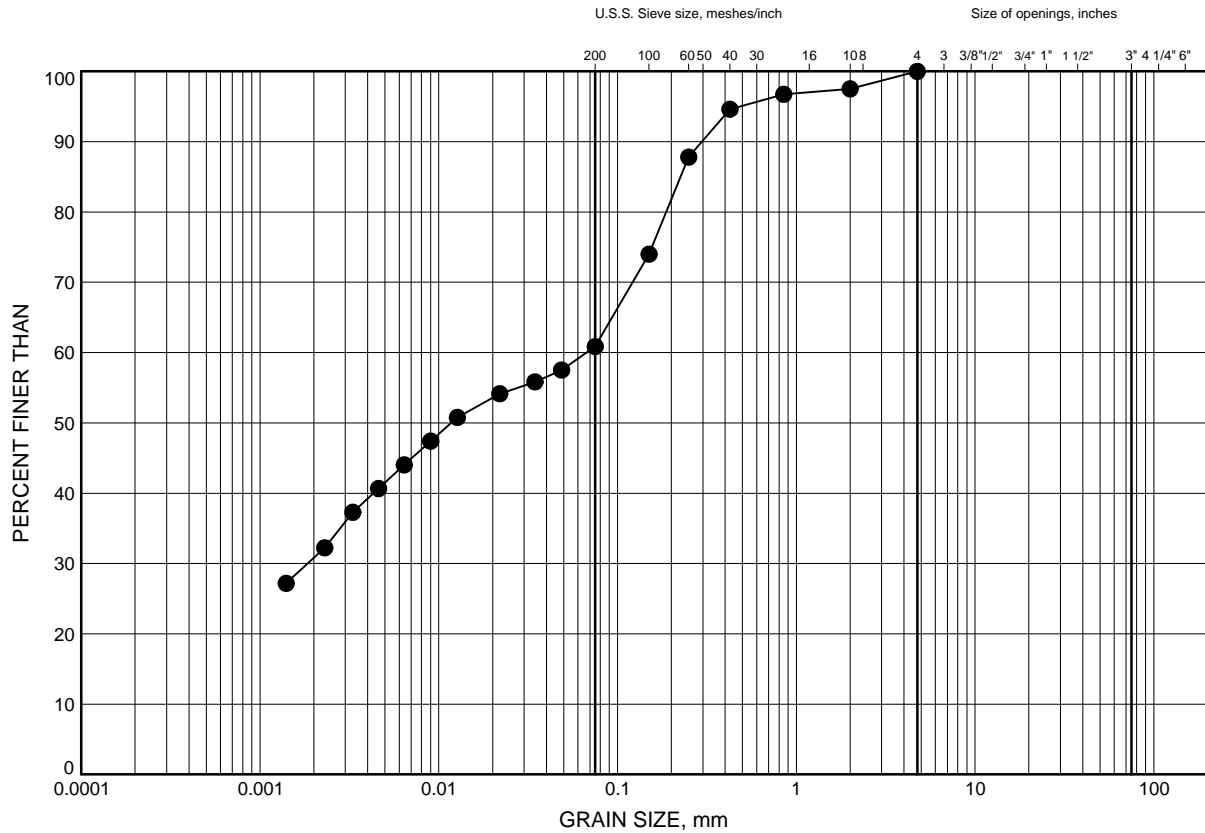
Prep'd AN
Chkd. DJP

Boston Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BO-01	2.59	245.91

Date March 2016
GWP# 5130-06-00



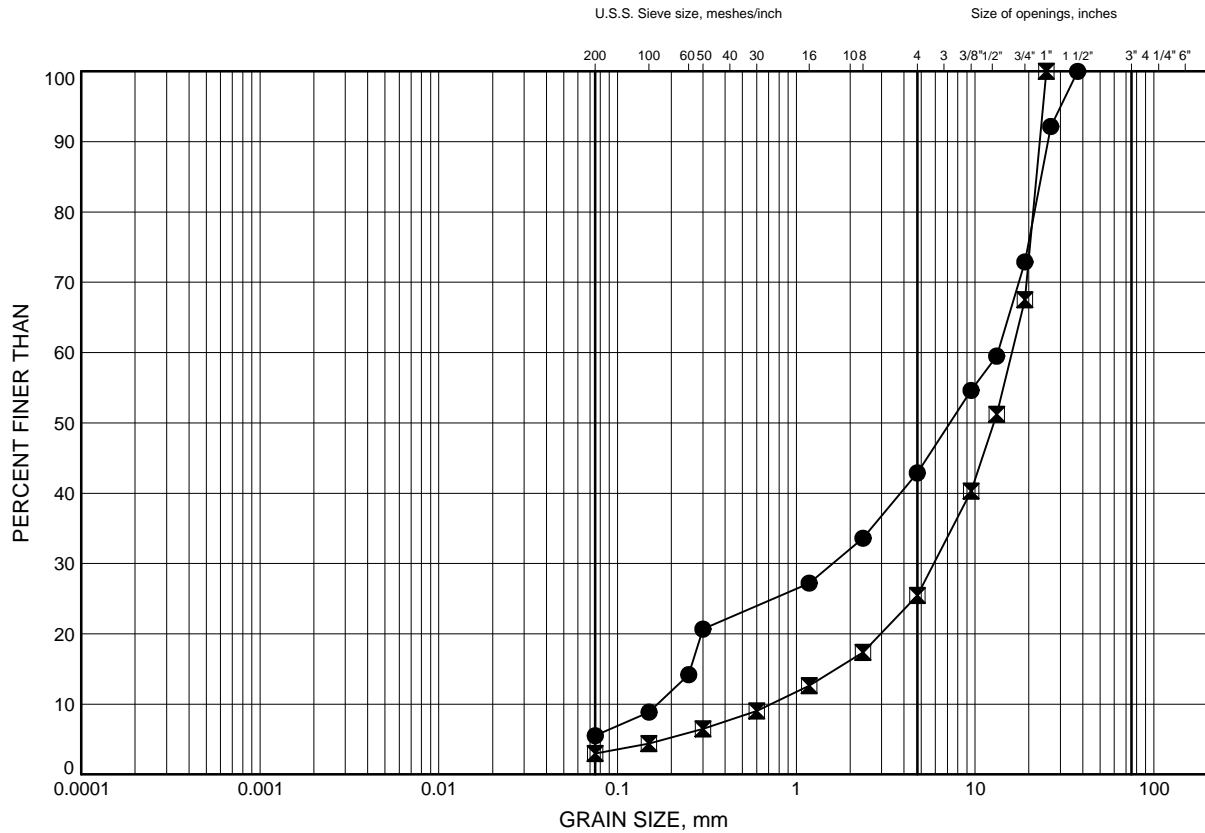
Prep'd AN
Chkd. DJP

Boston Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B3

Sandy GRAVEL to SAND & GRAVEL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BO-03	1.52	244.38
⊠	BO-04	0.30	245.50

Date March 2016
GWP# 5130-06-00



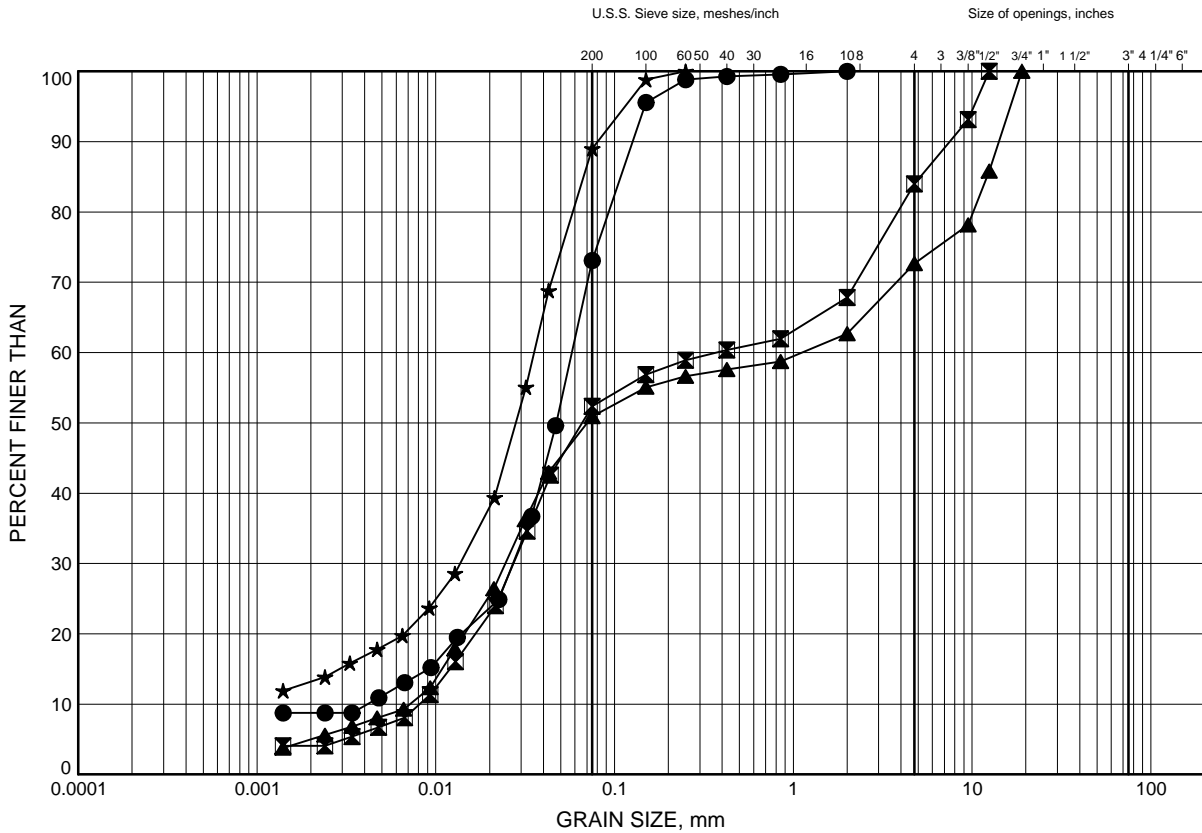
Prep'd AN
Chkd. DJP

Boston Creek Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B4

Sandy SILT to SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BO-02	4.88	242.22
⊠	BO-03	5.49	240.41
▲	BO-03	7.32	238.58
★	BO-04	6.40	239.40

Date March 2016
GWP# 5130-06-00



Prep'd AN
Chkd. DJP



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No : 19-5161-251

Client : MMM Group Limited

Date Drilled : 19-Feb-16

Project Name : Highway 564 Boston Creek Bridge Replacement

Date Tested : 01-Mar-16

Core Size : NQ BH No : BO-02

Tester : OA/BT

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	9.9	D	28.5	47.4	142.8	234.8	Granite	Very Strong
2	1	10.4	D	35.1	47.3	151.2	290.1	Granite	Extremely Strong
3	2	11.3	D	11.6	47.5	151.3	95.3	Granite	Strong
4	2	11.9	D	26.3	47.4	151.2	216.7	Granite	Very Strong
5									
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
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27									
28									
29									
30									
31									
32									
33									
34									
35									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

Job No : 19-5161-251

Client : MMM Group Limited

Date Drilled : 19-Feb-16

Project Name : Highway 564 Boston Creek Bridge Replacement

Date Tested : 01-Mar-16

Core Size : NQ BH No : BO-04

Tester : OA/BT

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	7.8	D	21.5	47.4	122.8	212.4	Granite	Very Strong
2	1	9.0	D	27.9	47.4	97.9	276.3	Granite	Extremely Strong
3	2	9.2	D	20.5	47.6	92.1	201.5	Granite	Very Strong
4	2	9.7	D	22.3	47.5	79.7	219.3	Granite	Very Strong
5	2	10.0	D	20.9	47.4	87.8	206.1	Granite	Very Strong
6	2	10.2	D	34.0	47.5	67.2	334.9	Granite	Extremely Strong
7									
8									
9									
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34									
35									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 16T078548

PROJECT: 19-5161-251

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: THURBER ENGINEERING LTD

SAMPLING SITE: Temiskaming Structures

ATTENTION TO: Deanna Pizycki

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2016-03-18

DATE REPORTED: 2016-03-30

		SAMPLE DESCRIPTION:		BO-04 SS5
		SAMPLE TYPE:		Soil
		DATE SAMPLED:		2/19/2016
Parameter	Unit	G / S	RDL	7450212
Sulphide*	%		0.05	0.14
Chloride (2:1)	µg/g		2	83
Sulphate (2:1)	µg/g		2	44
pH (2:1)	pH Units		NA	8.02
Electrical Conductivity (2:1)	mS/cm		0.005	0.238
Resistivity (2:1)	ohm.cm		1	4200
Redox Potential (2:1)	mV		5	312

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

7450212 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).

Certified By:

Amanjot Bhela



Quality Assurance

CLIENT NAME: THURBER ENGINEERING LTD

PROJECT: 19-5161-251

SAMPLING SITE: Temiskaming Structures

AGAT WORK ORDER: 16T078548

ATTENTION TO: Deanna Pizycki

SAMPLED BY:

Soil Analysis

RPT Date: Mar 30, 2016			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
Corrosivity Package															
Sulphide*	7444756		< 0.05	< 0.05	NA	< 0.05	95%	80%	120%	NA			NA		
Chloride (2:1)	7443948		70	69	1.4%	< 2	98%	80%	120%	100%	80%	120%	95%	70%	130%
Sulphate (2:1)	7443948		337	336	0.3%	< 2	97%	80%	120%	102%	80%	120%	96%	70%	130%
pH (2:1)	7449192		7.50	7.62	1.6%	NA	102%	90%	110%	NA			NA		
Electrical Conductivity (2:1)	7443948		0.531	0.536	0.9%	< 0.005	97%	90%	110%	NA			NA		
Redox Potential (2:1)	7449192		381	380	0.3%	< 5	109%	70%	130%	NA			NA		

Comments: NA signifies Not Applicable.

Duplicate Qualifier: As the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Certified By:

Amanjot Bhela

Method Summary

CLIENT NAME: THURBER ENGINEERING LTD

AGAT WORK ORDER: 16T078548

PROJECT: 19-5161-251

ATTENTION TO: Deanna Pizycki

SAMPLING SITE: Temiskaming Structures

SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Sulphide*	MIN-200-12025	ASTM E1915-09	GRAVIMETRIC
Chloride (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	McKeague 4.12 & SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	MSA part 3 & SM 4500-H+ B	PH METER
Electrical Conductivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B	EC METER
Resistivity (2:1)	INOR-93-6036	McKeague 4.12, SM 2510 B, SSA #5 Part 3	CALCULATION
Redox Potential (2:1)		McKeague 4.12 & SM 2510 B	REDOX POTENTIAL ELECTRODE

Certificate of Analysis

CLIENT NAME: THURBER ENGINEERING LTD

PROJECT: 19-5161-251

SAMPLING SITE:

AGAT WORK ORDER: 15T990315

ATTENTION TO: MARK FARRANT

SAMPLED BY:

Inorganic Chemistry (Water)							
SAMPLE TYPE: Water		SAMPLE ID: 6699728			DATE RECEIVED: Jun 29, 2015		
DATE SAMPLED: Jun 27, 2015				DATE REPORTED: Jul 07, 2015			
SAMPLE DESCRIPTION: Boston Creek Bridge							
PARAMETER	UNIT	RESULT	G / S	RDL	DATE ANALYZED	INITIAL	DATE PREPARED
Electrical Conductivity	uS/cm	162		2	Jul 03, 2015	JC	Jul 03, 2015
pH	pH Units	7.52		NA	Jul 03, 2015	JC	Jul 03, 2015
Langelier Index		-0.55			Jul 06, 2015	SYS	Jul 06, 2015
Total Hardness (as CaCO3)	mg/L	80.3		0.5	Jul 03, 2015	SYS	Jul 03, 2015
Total Dissolved Solids	mg/L	104		20	Jul 06, 2015	AP	Jul 03, 2015
Alkalinity (as CaCO3)	mg/L	63		5	Jul 03, 2015	JC	Jul 03, 2015
Chloride	mg/L	1.94		0.10	Jul 03, 2015	WZ	Jul 03, 2015
Sulphate	mg/L	14.5		0.10	Jul 03, 2015	WZ	Jul 03, 2015
Calcium	mg/L	23.2		0.05	Jul 03, 2015	PB	Jul 03, 2015
Magnesium	mg/L	5.44		0.05	Jul 03, 2015	PB	Jul 03, 2015
Resistivity	ohms.cm	6170			Jul 03, 2015	SYS	Jul 03, 2015
Sulphide	mg/L	<0.05		0.05	Jul 02, 2015	SN	Jul 02, 2015
Redox Potential	mV	316		5	Jul 06, 2015	BG	Jul 06, 2015

COMMENTS:

RDL - Reported Detection Limit; G / S - Guideline / Standard

Certified By:





Appendix C

Site Photographs



Photograph 1 – South Elevation Looking West



Photograph 2 – North Elevation Looking East



Photograph 3 – East Abutment

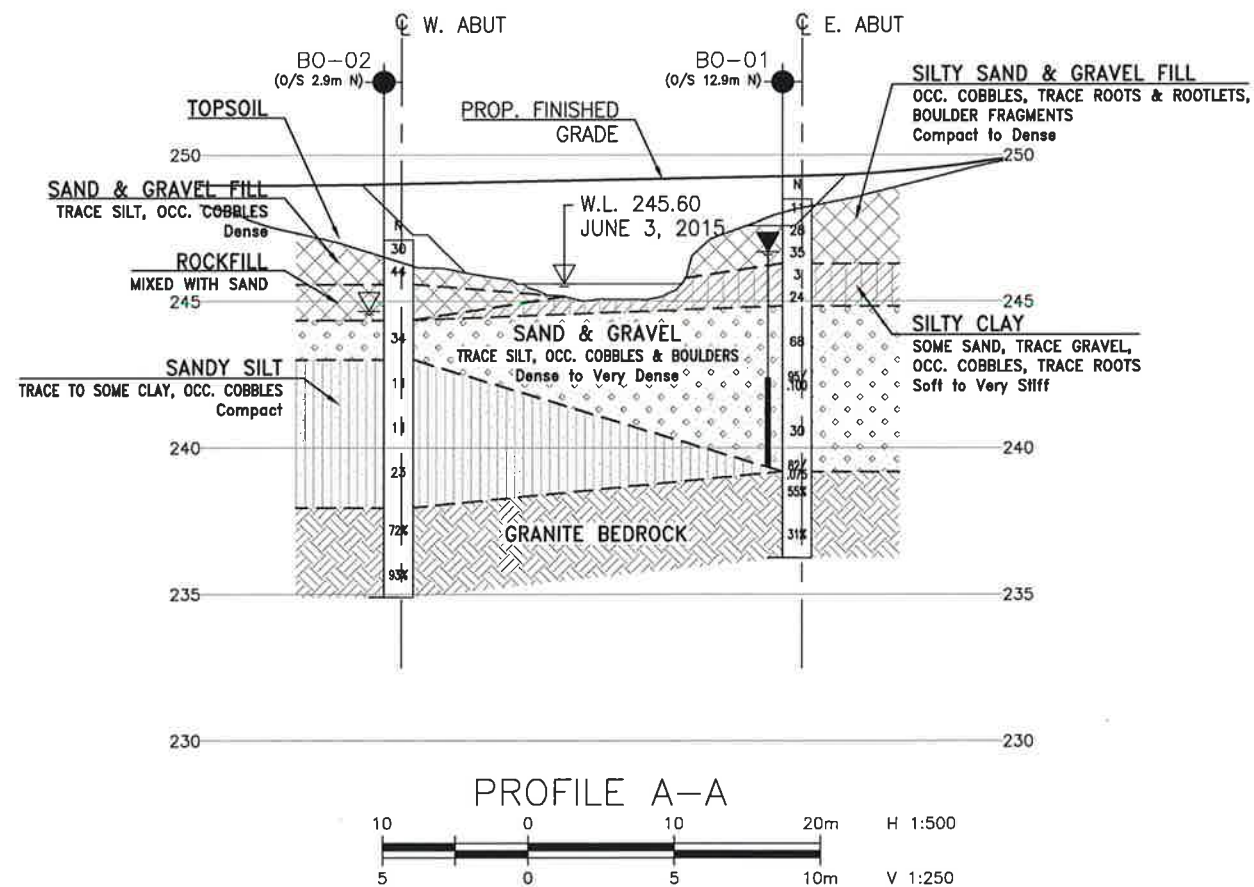
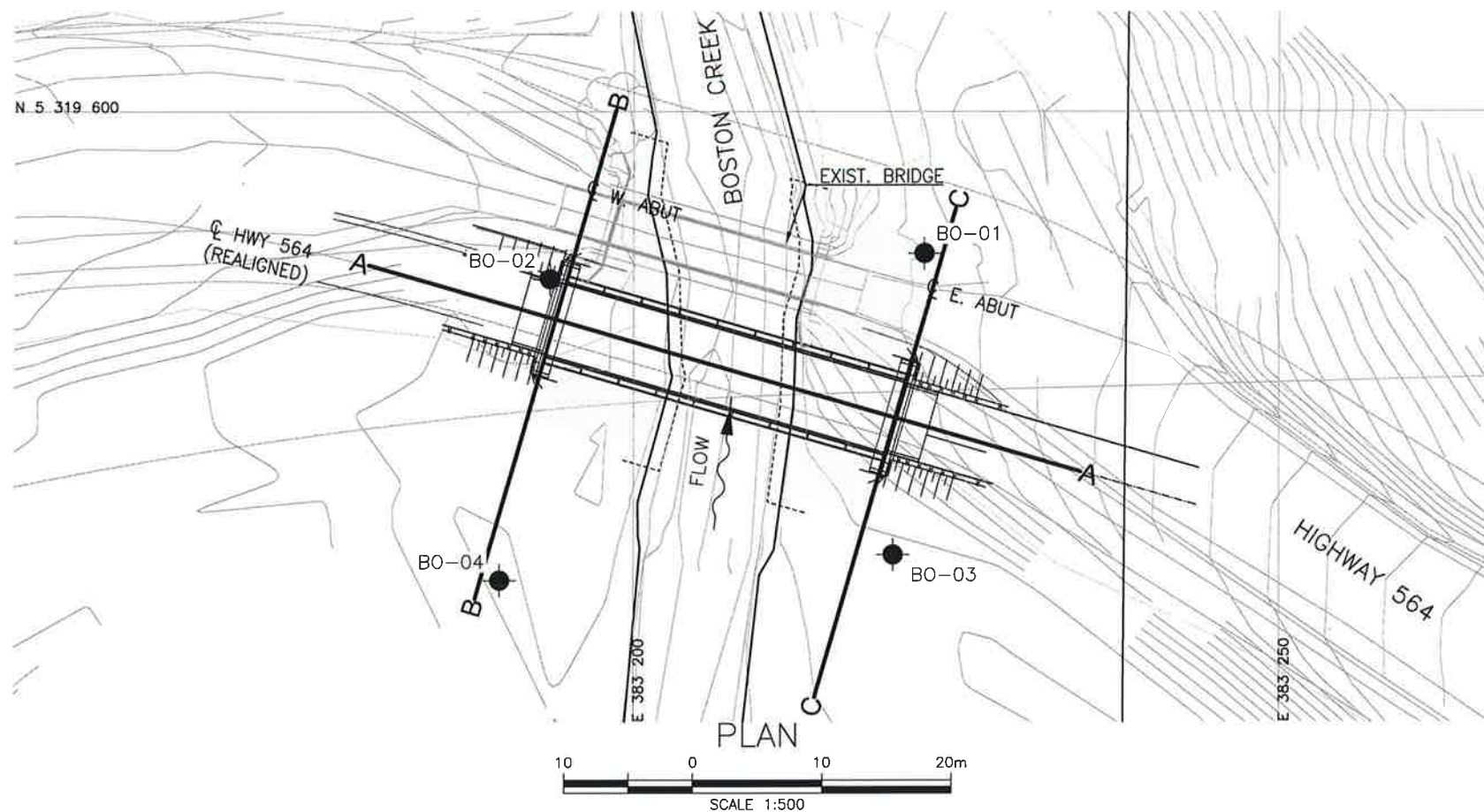


Photograph 4 – West Abutment



Appendix D

Borehole Locations and Soil Strata Drawings



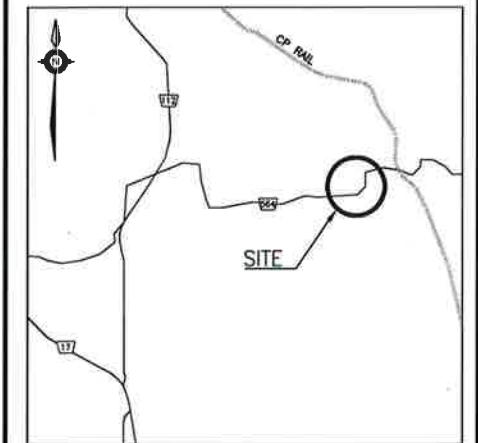
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 5130-06-00



HIGHWAY 564
BOSTON CREEK BRIDGE
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



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 (MTM)	EASTING (MTM)
BO-01	248.5	5 319 589.0	383 222.5
BO-02	247.1	5 319 587.0	383 193.5
BO-03	245.9	5 319 565.7	383 220.1
BO-04	245.8	5 319 563.7	383 189.6

NOTES

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 32D-20



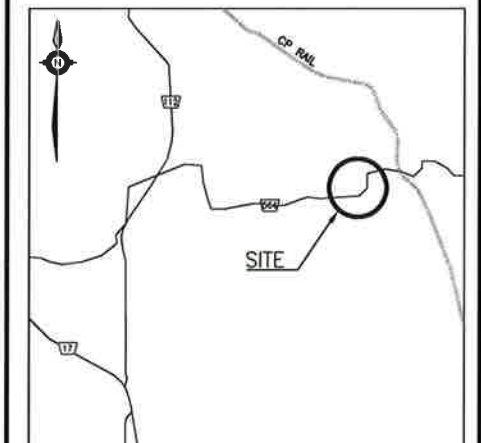
REVISIONS	DATE	BY	DESCRIPTION
DESIGN	KS	CHK PKC	CODE
DRAWN	MFA	CHK KS	SITE
			LOAD
			STRUCT
			DWG 1
			DATE AUG 2016

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

CONT No
GWP No 5130-06-00

HIGHWAY 564
BOSTON CREEK BRIDGE
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



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

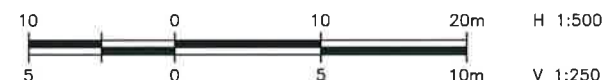
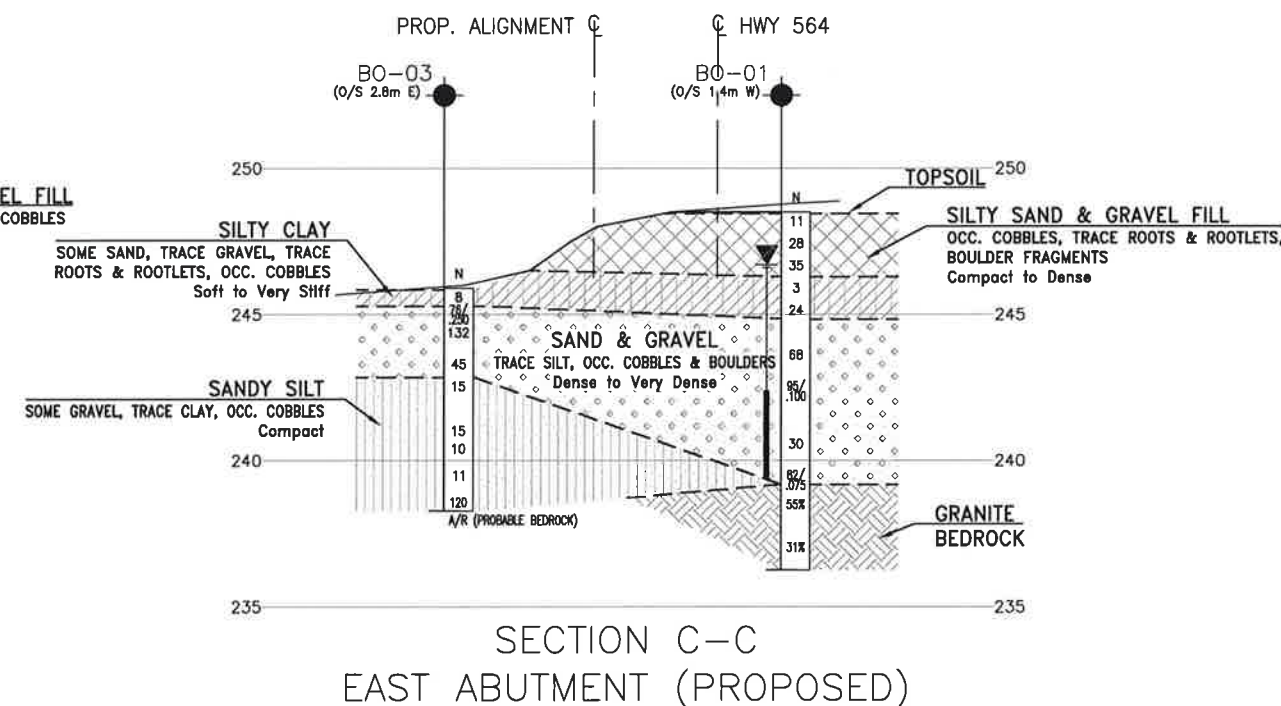
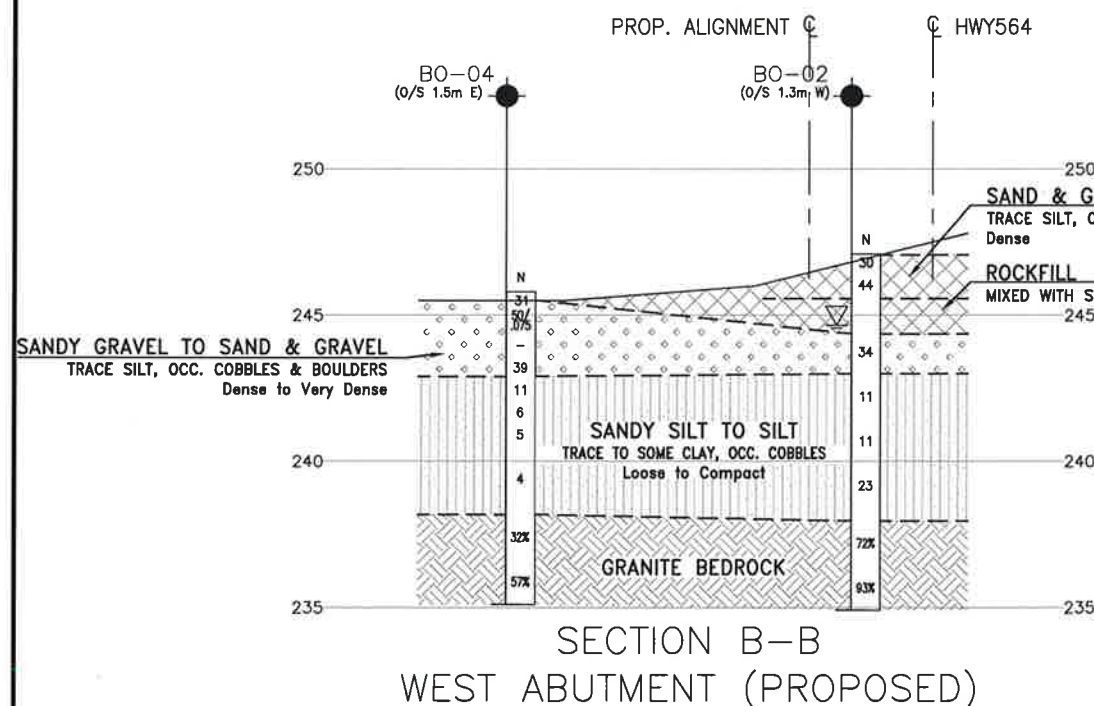
NO	ELEVATION	NORTHING (MTM)	EASTING (MTM)
BO-01	248.5	5 319 589.0	383 222.5
BO-02	247.1	5 319 587.0	383 193.5
BO-03	245.9	5 319 565.7	383 220.1
BO-04	245.8	5 319 563.7	383 189.6

NOTES

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
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GEOCRES No. 32D-20

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Appendix E

List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS.PROV 206
- OPSS.PROV 539
- OPSS.PROV 804
- OPSS 902
- OPSS.PROV 903

2. Suggested text for NSSP on “Obstructions”

Cobbles and boulders and rock fill are present within the existing embankment and underlying native soils at this site. These cobbles and boulders and rock fill may impede excavations, installation of piles and/or temporary support system. At some locations, the installation may not be able to penetrate the obstructions and reach the design elevations. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions to achieve the design depths.

3. Suggested text for NSSP on “Compacted Rock Fill Pads below Footings”

For rock fill pads below abutment footings, the rock fill shall be well graded, freshly produced in a quarry, and have a maximum size of 250 mm.

Rock fill pad construction must be carried out in the dry. The rock fill layers shall not exceed 500 mm in thickness prior to compaction. Material in each layer shall be fully compacted before the succeeding layer is placed. Each rock fill layer shall be compacted with a tractor bulldozer, crawler type as specified in the Tractor Bulldozer – Crawler Type for Rock Embankment Construction subsection of OPSS.PROV 206. The minimum number of complete passes shall be six and the maximum number of passes shall be eight. A complete pass shall be defined as 100% coverage of the layer surface.

For the rock fill pads, materials shall be placed in their final position by blading. End dumping or depositing of rock over the end of any layer by hauling equipment is not permitted. Each layer shall be levelled in place and compacted to minimize voids and bridging of large rock fragments within the rock fill pad.

The top surface of the rock fill pad shall be chinked with rock fragments and spalls to form the subgrade prior to the placement of the levelling pad in order to minimize voids and prevent migration of levelling pad material into the rock fill.

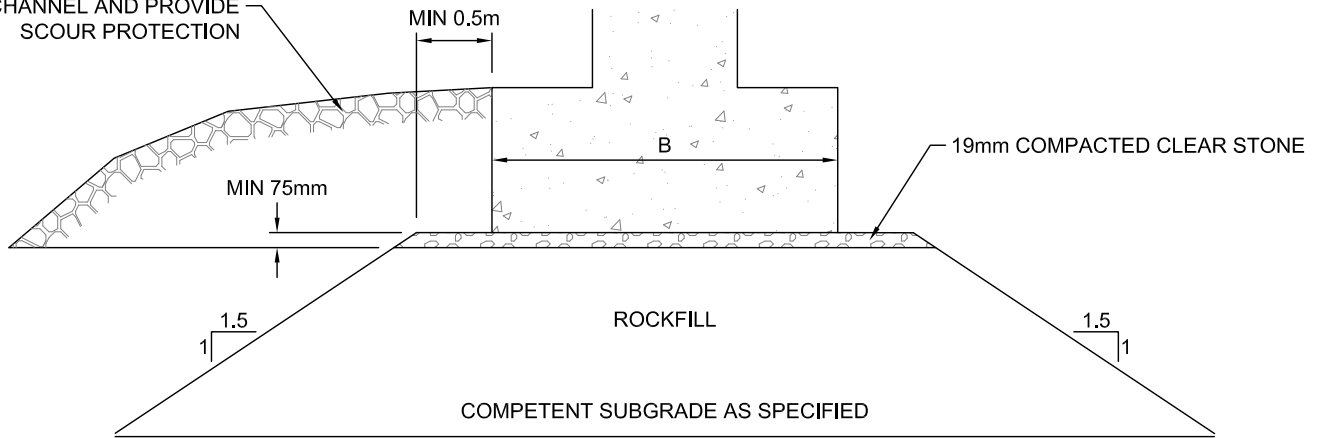
Care shall be taken to avoid large boulders and rock fragments protruding above the rock fill pad surface.

A minimum 75 mm thick layer of compacted 19 mm clear stone should be placed above the rock fill to provide an even founding surface for placement of the footings. Details of footing construction on rock fill are presented in Figure 1 of Appendix F.

Appendix F

Figure 1 – Abutment Footing on Rock Fill

RE-INSTATE GROUND SURFACE
IN CHANNEL AND PROVIDE
SCOUR PROTECTION



CROSS-SECTION

FOOTING ON ROCKFILL CORE



THURBER ENGINEERING LTD.

ENGINEER :	DRAWN :	APPROVED :
KS	MFA	PKC
DATE :	SCALE :	DRAWING No.
APRIL 2017	N.T.S.	FIGURE 1