

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
BOSTON CREEK BRIDGE REHABILITATION  
NEW LISKEARD DISTRICT, ONTARIO  
G.W.P. 5027-14-00, SITE NO. 47-098  
Geocres Number: 31M-112**

**Report to:**

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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 Pacaud Concession Road 2 in the Township of Pacaud, New Liskeard District, 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-0024.

**2 SITE DESCRIPTION**

The existing Boston Creek Bridge is located on Pacaud Concession Road 2, approximately 7.5 km east of Highway 11.

The existing bridge is a steel bailey bridge with a timber deck supported on timber sills. The approximate length and width of the structure is 21.6 and 3.4 m, respectively. The bridge was constructed in 1957. The structure is in fair condition; deterioration of the abutment walls and structural elements is evident.

Boston Creek flows from north to south at the bridge location. The creek banks are well vegetated with tall grass and shrubs and frequent trees. The local topography is of low relief with no visible bedrock outcrops. Photographs in Appendix C show the general nature of the site and the existing bridge.

Based on published geological information, the general area of the project is covered by glaciolacustrine sediments of clays and silts deposited during the Pleisocene period. These deposits

are often varved clays, but massive clays are also present in the area. The clays are underlain by bedrock comprising Precambrian intrusive massive granodiorite to granite.

### **3 SITE INVESTIGATION AND FIELD TESTING**

The site investigation and field testing program for this project was carried out between July 26 and July 28, 2015. The program consisted of drilling and sampling two (2) boreholes numbered BC-01 to BC-02 to depths of 20.2 m and 29.6 m respectively. Borehole BC-01 was drilled to the south west of the west abutment and BC-02 was drilled to the north east of the east abutment. Coring with an NQ size core barrel was used below the overburden portion on Borehole BR-01 to a depth of 23.3 m. A Dynamic Cone Penetration Test (DCPT) was conducted below the sampled portion of Borehole BR-02 from 29.6 m to 34.7 m depth, where the SPT N value of 100 blows per 0.3 m penetration was encountered.

Prior to the start of drilling, the borehole locations were marked in the field and utility clearances were obtained. The coordinates and ground surface elevations for the boreholes were derived from topographic plans provided to Thurber by MMM Group Limited. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix D.

A truck-mounted CME 75 hi-torque drill rig was used to advance the Boreholes BC-01 and BC-02 to the target depth using NW casing and wash boring technique. A granular pad was constructed to facilitate the drilling of BC-02. Soil samples were obtained at selected intervals using a 50 mm diameter split spoon sampler in conjunction with Standard Penetration Testing (SPT). Field vane shear testing using an MTO “N” size vane were carried out in very soft to soft cohesive soils.

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 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 immediately after completion of drilling were not representative of site conditions as water was used during wash boring operations. A standpipe piezometer was installed in Borehole BC-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 borehole are summarized in Table 3.1.

**Table 3.1 – Borehole Completion Details**

<b>Foundation Unit</b>	<b>Boreholes</b>	<b>Piezometer Tip Depth/ Elevation (m)</b>	<b>Completion Details</b>
West Abutment	BC-01	20.1 / 204.7	Sand filter from 20.1 m to 16.5 m, bentonite holeplug from 16.5 m to 15.5 m, bentonite and cuttings to 1.0 m and bentonite holeplug to surface.
East Abutment	BC-02	None	Bentonite holeplug and cuttings from 34.7 m to surface.

The results of the field drilling and sampling are presented on the Record of Borehole sheets in Appendix A.

#### **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) and plasticity testing (Atterberg Limits). The results of the geotechnical laboratory 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 concrete, as well as the potential for corrosion associated with the structure, a sample of the existing native soil, and a sample of surface water from the creek upstream of the bridge were collected. The samples were submitted to AGAT Laboratories of Mississauga, Ontario for analytical testing of corrosivity parameters and sulphate. 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” drawing 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. The subsurface conditions may vary between and beyond the borehole locations.

In general, the site is underlain by an extensive cohesive deposit comprising a layer of brown silty clay that grades to sandy silty clay, which overlies a grey silty clay. Bedrock was encountered at a depth of 20.2 m in the borehole drilled on the west side of the creek. Water level in Boston Creek was inferred to be at Elev. 222.5 m. More detailed description of the individual strata are presented below.

## 5.1 Topsoil

A layer of topsoil was encountered at the surface of Borehole BC-01. The topsoil layer had a thickness of 250 mm at the borehole location. The topsoil thickness may vary in other areas of the site and this limited data is not intended for estimating purposes.

## 5.2 Fill

A silty clay fill was encountered in Borehole BC-01 underlying the topsoil. Some gravel, trace cobbles, and organic inclusions (roots and rootlets) were noted at shallow depths in the fill. This layer was 1.4 m thick and extended to a depth of 1.4 m or to Elevation 223.4 m. SPT 'N' values of 13 and 15 blows per 0.3 m penetration were recorded in this layer, indicating a stiff to very stiff consistency. A moisture content of 8% was recorded in this fill layer. It is probable that this fill layer is the native silty clay reworked during road/bridge construction.

A layer of brown sand and gravel fill with trace silt was encountered in Borehole BC-02. This fill was placed to accommodate access of the drilling equipment to the borehole location. The fill extended from the ground surface to 0.6 m depth (Elev. 224.5).

## 5.3 Silty Clay to Sandy Silty Clay

A layer of silty clay grading to sandy silty clay was encountered beneath the fill in Borehole BC-01 and underlying the granular fill in Borehole BC-02. The deposit was brown in colour with trace to some gravel and trace organic inclusions at shallow depths. This layer ranged in thickness from 2.7 to 3.5 m with the base at a depth of 4.1 m or at Elevations 221.0 to 220.7 m.

SPT 'N' values recorded in this layer ranged from 0 to 12 blows per 0.3 m penetration, indicating a very soft to stiff consistency. A very soft layer of this deposit was encountered at approximate Elevation of 222.8, which seems to coincide with the creek water level. Moisture contents in this layer ranged from 20 to 46%. The grain size analyses conducted on samples of the silty clay/sandy silty clay are presented in Figure B1, and Atterberg Limits test results are presented in Figure B3 in Appendix B. The results are summarized in the following table.

Soil Particles	%
Gravel	0 to 16
Sand	5 to 35
Silt	30 to 41
Clay	31 to 46
Soil Property	%
Liquid Limit	35 to 41
Plasticity Index	16 to 21

The results of the Atterberg Limits tests indicate that this silty clay is of low plasticity (CL) to intermediate plasticity (CI).

The results of laboratory testing are included in the Record of Borehole sheets in Appendix A.

#### 5.4 Silty Clay

Underlying the brown silty clay/sandy silty clay in both boreholes was a deposit of grey silty clay with trace sand. Where fully penetrated in Borehole BC-01, the thickness of the silty clay was 16.1 m and the base of the deposit was encountered at a depth of 20.2 m or at Elevation of 204.6 m. Borehole BC-02 was terminated in the silty clay at a depth of 29.6 m or at Elevation of 195.5 m.

SPT 'N' values recorded in this layer ranged from 0 to 6 blows per 0.3 m penetration. In conjunction with measured field vane shear strength ranging from 19 to 53 kPa, the clayey silt was described to have a soft to stiff consistency. Moisture contents in this layer varied between 34 and 69% with most values ranging from 40 to 60%. The grain size analyses conducted on samples are presented in Figure B2, and Atterberg Limits test results are presented in Figure B4 in Appendix B. The results are summarized in the following table.

Soil Particles	%
Gravel	0
Sand	0
Silt	33 to 45
Clay	55 to 67
Soil Property	%
Liquid Limit	32 to 48
Plasticity Index	13 to 23

The results of the Atterberg Limits tests indicate that the silty clay is typically of low plasticity (CL) to intermediate plasticity (CI).

Below the sampled depth of Borehole BC-02, a DCPT was carried out from a depth of 29.6 m to practical refusal (100 blows per 0.3 m) at 34.7 m (Elev. 190.4).

#### 5.5 Bedrock

Granite bedrock was encountered below the silty clay in Borehole BC-01 at a depth of 20.2 m (Elev. 204.6). The bedrock was cored for 3.1 m length to a depth so 23.3 m (Elev. 201.5).

Total Core Recovery (TCR) in the bedrock ranged from 83% to 100%. The measured RQD of the rock cores ranged from 11% to 26%, indicating a very poor to poor quality of rock. The



Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 5 to greater than 25.

## 5.6 Water Levels

Water levels were observed in the open boreholes during drilling operations. Wash boring technique was used to advance the boreholes and therefore water levels recorded during or upon completion of drilling may not reflect natural groundwater levels. A standpipe piezometer was installed in one borehole to monitor the groundwater level after completion. The water level measured in the piezometer is shown in Table 5.1.

**Table 5.1 – Water Level Measurements**

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
BC-01	July 27, 2015	1.2	223.6	In piezometer
	July 28, 2015	1.3	223.5	
BC-02	July 28, 2015	3.0	222.1	Open borehole

Based on the General Arrangement (GA) Drawing dated February 2016, the water level in Boston Creek was noted at Elev. 221.7 on May 14, 2015.

The above levels are short-term readings and seasonal fluctuations of the groundwater and creek levels 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 soil below the embankment fill and a sample of the surface water from the creek were submitted for analytical testing of corrosivity parameters and sulphate. 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	
			Borehole BC-02 Soil Sample	Boston Creek Water
			(Soil, 0.76-2.1 m deep)	(Creek Water)
Sulphide	%	mg/L	0.02	<0.05
Chloride	µg/g	mg/L	3	2.96
Sulphate	µg/g	mg/L	11	17.5
pH	pH Units	pH Units	7.7	7.96
Electrical Conductivity	mS/cm	µS/cm	0.162	-
Resistivity	ohm.cm	ohm.cm	6170	3820
Redox Potential	mV	mV	302	312
Langlier Index	-	-	-	0.34
Total Hardness (as CaCO <sub>3</sub> )	-	mg/L	-	127
Total Dissolved Solids	-	mg/L	-	138
Alkalinity (as CaCO <sub>3</sub> )	-	mg/L	-	112

## 7 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber obtained the northing and easting coordinates and ground surface elevations from measurements taken in the field relative to the topographic plans provided by MMM Group Limited.

Eastern Ontario Diamond Drilling of Hawkesbury, Ontario supplied a track-mounted CME-45 hi-torque drill rig and conducted the drilling, sampling and in-situ testing operations for the boreholes. The drilling operations were supervised by Mr. Amir Fereidouni of Thurber. Geotechnical laboratory testing was carried out by Thurber in its MTO-approved laboratory.

Overall supervision of the field program was carried out by Mr. Stephane Loranger, CET. The report was prepared by Ms. Deanna Pizycki, EIT and Ms. Anna Piascik, 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.

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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 for the proposed rehabilitation of the existing Boston Creek Bridge on Pacaud Concession Road 2, in the District of New Liskeard, Ontario.

The Terms of Reference for Foundation Engineering specify the design of the rehabilitation works for the existing bridge structure including assessment of settlements and stability of the abutments and approach embankments. As no details of the existing bridge (timber cribs dimensions, founding level, existing loading, etc.) are available, the report provides general comments on these aspects of the project. Given the deficiency of the present foundations, complete foundation replacement is required.

The engineering discussions and recommendations presented in this report are based on the factual data obtained during the course of this investigation and on a preliminary General Arrangement (GA) drawing dated February 2016 provided by MMM Group Limited.

The foundation-related aspects of the proposed rehabilitation works indicated on the GA drawing include:

- installation of new bearing slabs (foundations), and
- construction of new approach slabs

The discussion presented in the report is intended to provide the designer with information to assess feasible foundation alternatives and to carry out the design on the foundation aspects of the rehabilitation works.

**9 ASSESSMENT OF EXISTING CONDITIONS**

The summary of the MTO Report of Structure Inspection dated July 31, 2013, indicated the following “Performance Deficiencies: load carrying capacity and continuing settlement of the abutment walls,

continuing settlement of the abutment bearings, undermining of the foundations, flooding/channel blockage at the streams and waterways.” However, there is little known about the existing foundations. From what can be seen, it appears that the ends of the bridge rest on timber sills with no visible evidence of any other foundation, crib work, etc.

The creek banks below the bridge appear to consist of pit-run gravel, including cobbles and boulders. This is not consistent with the soils encountered in the boreholes and it has been concluded that this material is the approach fills of an earlier bridge, a conclusion supported by the presence of abandoned timber piles at the water’s edge. Thick vegetation covered the slopes of the approach embankments, hindering visual inspection. However, the top surface of the approach embankments did not show signs of global slope instability.

As shown on the photographs enclosed in Appendix C, rows of old timber piles cut-off above the creek level are present at the toe of the forward slopes, which may suggest that the existing bridge is not the original bridge at this location.

## **10 STRUCTURE FOUNDATIONS**

### **10.1 General**

In summary, the subsurface stratigraphy encountered below the embankment fill or topsoil generally consists of a layer of very soft to stiff brown silty clay/sandy silty clay underlain by a deposit of grey, soft to stiff silty clay. As the access to the abutments was difficult (narrow/single lane local road and steep embankment slopes), the boreholes were located some distance from the existing abutments and outside of the approach embankments, the type and thickness of embankment fill in the immediate vicinity of the abutments were not investigated. However, based on the exposures under the bridge, it is anticipated that the embankment fill could be in the order of 1 to 2 m in thickness in the abutment areas. Outside of the embankments, the upper brown silty clay/sandy silty clay deposit was encountered to 4.1 m depth (Elev. 221.0 to 220.7). On the west side of the creek at Borehole BC-01, some 1.4 m of silty clay fill overlies a native grey silty clay extending to 20.2 m depth (Elev. 204.6) to meet bedrock. On the east side of the creek, at Borehole BC-02, the silty clay was encountered to a depth of 29.6 m (Elev. 195.5), and then the DCPT was conducted and terminated at 34.7 m depth (Elev. 190.4) when reaching the SPT N-value of 100 blows per 0.3 m penetration (“practical refusal”). The refusal was not proved by coring, however, given the bedrock surface at 20.1 m on the west side, encountering bedrock at 34.7 m on the east side is probable.

The existing supports for the Boston Creek Bridge are reported to experience settlements under the existing bridge loading conditions.

Given the soil stratigraphy encountered and the requirements of modular bridge design, the following options could be considered for the new bridge foundations:

- spread footing (precast concrete slab) placed on engineered fill, and
- driven H-piles

Recommendations for design of the feasible foundation options are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation option is recommended.

## **10.2 Spread Footings on Engineered Fill Pads**

### **10.2.1 Spread Footing Installation**

As part of the rehabilitation works, the preliminary General Arrangement drawing dated February 2016 shows new precast concrete slabs supporting the abutments. Since the superstructure length remains unchanged, the new footings will be located within the footprints of the existing timber sills.

Since the details of the existing supports for the bridge are not known (beyond the presence of timber sills), the following construction sequence is recommended:

1. Excavate to remove all timber and other deleterious material from the footprint of the new foundation.
2. The minimum depth of excavation must accommodate the concrete foundation slab and at least 1.0 m of engineered fill below the slab, as described below.
3. Sand and gravel fill may be left in the base of the excavation, provided it is compacted to its 100% of SPMDD, otherwise the excavation must continue down to undisturbed, native soil.
4. The dimensions of the base of the excavation must be determined by assuming a granular pad 1.0 m wider than the footing at the level of the footing base and projecting outward at 2H:1V.

The new footings must be placed on the engineered fill pad consisting 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. The top of the engineered fill pad should be at least 1 m wider than the footprint of the spread footing at the underside of the footing and must be at least 1.0 m thick.

Excavations for the engineered fill pad construction and footing placement will require the superstructure to be removed or temporarily supported during construction.

### 10.2.2 Axial Resistance

A foundation slab constructed as described above may be designed using the following values:

Factored Geotechnical Resistance at ULS (kPa) -	125 kPa
Geotechnical Reaction at SLS (kPa) -	90 kPa.

The value of the Geotechnical Reaction at SLS given above is for 25 mm of settlement.

The values of a 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 illustrated in the CHBDC 2014 Clause 6.10.3 and Clause 6.10.4.

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.6 for cast-in-place concrete and 0.5 for pre-cast concrete. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

## 10.3 Driven H-Piles

### 10.3.1 Axial Pile Resistance

If a piled foundation is considered, driven steel H-piles, designed as friction piles can be used at the east abutment. In order to develop an adequate resistance, the steel H-piles will have to be driven to significant depths. The geotechnical resistances recommended for HP 310x110 piles are presented in Table 10.1.

**Table 10.1 – Geotechnical Resistance and Reaction for Driven HP310x110 Piles**

Foundation Element	Pile Tip Depth/Elevation (m)	Factored Geotechnical Resistance at ULS (kN) per pile	Geotechnical Reaction at SLS (kN) per pile (for 25 mm settlement)
East Abutment	20 / 203.7	300	200
	30 / 193.7	450	300

For the west abutment, piles could be driven to bedrock at a depth of approximately 20 m and designed for a factored ULS resistance of 2,000 kN. The SLS condition would not govern.

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used for any new fill through which the piles will be driven.

The actual pile tip elevations may vary during installation and pile length should be controlled as described in Section 10.3.2.

### 10.3.2 Driven Pile Installation

Pile installation should be in accordance with OPSS 903.

### 10.3.3 Pile Lateral Resistance

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 in metres

The above equations and recommended parameters in Table 10.2 below may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

**Table 10.2 – Soil Parameters for Lateral Pile Resistance**

Soil Unit	Elevation (m)		$\gamma'$ (kN/m <sup>3</sup> )	$n_h$ (kN/m <sup>3</sup> )	$K_p$	$S_u$ (kPa)
	Top	Bottom				
West Abutment (Borehole BC-01)						
Silty Clay Fill	GS	223.4	19	-	2.8	40
Silty Clay/Sandy Silty Clay	223.4	220.7	8*	-	-	30
Silty Clay	220.7	204.6/Bedrock	8*	-	-	30
East Abutment (Borehole BC-02)						
Sand and Gravel Fill	GS	224.5	20	2,000	1.3*	-
Silty Clay/Sandy Silty Clay	224.5	221.0	8*	-	-	30
Silty Clay	221.0	195.5	8*	-	-	30



- \* Effective unit weight to be used in the evaluation of the lateral resistance of piles below the water table.

For analysis, the spring constant,  $K_s$ , may be obtained from the expression:

$$K_s = k_s L D \text{ (kN/m),}$$

where  $k_s$  = coefficient of horizontal subgrade reaction ( $\text{kN/m}^3$ ),

$D$  = pile width (m), and

$L$  = 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 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 10.3. Intermediate values may be obtained by linear interpolation.

**Table 10.3 – Subgrade Reaction Reduction Factors for Pile Spacing**

Condition	Pile Spacing, Centre to Centre	Reduction Factor
Pile group oriented <i>perpendicular</i> to direction of loading	4D	1.0
	1D	0.5
Pile group oriented <i>parallel</i> 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.

#### 10.4 Recommended Foundation

Taking account of the soil stratigraphy and the requirements of modular bridge design, the recommended foundation is a concrete slab on 1.0 m (minimum) thick engineered fill pad.

## 10.5 Frost Cover

The depth of frost penetration at this site is approximately 2.4 m.

If piles are used, the base of pile caps must be provided with a minimum of 2.4 m of earth cover as protection against frost action. If it is not practical to provide 2.4 m of earth cover, consideration can be given to augmenting the frost protection by using expanded polystyrene insulation (EPS). Typically, 25 mm of EPS can be considered equivalent to 600 mm of earth cover. If EPS is used, it must be provided with long term protection against erosion, environmental degradation and spills.

Concrete slab foundations for modular bridges may be founded on an engineered fill pad with a minimum embedment of 0.5 m.

## 10.6 Impact of Existing Foundations

The existing foundation materials must be removed prior to constructing new foundations.

# 11 ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES

Any new backfill behind the modified abutment should be placed in accordance with OPSS 902. All backfill material should consist of Granular A, Granular B Type II or Granular B Type III material meeting the specifications of OPSS.PROV 1010. Fine rock fill should be used for the timber cribs backfill. Compaction equipment to be used adjacent to the walls should be restricted in accordance with OPSS.PROV 501.

Lateral earth pressures acting on the abutment walls may be assumed to be distributed triangularly and to be governed by the characteristics of the wall 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)

$K$  = earth pressure coefficient

$\gamma$  = unit weight of retained soil

$h$  = 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 and the existing material adjacent to the wall. Typical values are given in Table 11.1.

**Table 11.1 – Earth Pressure Coefficients (K)**

Condition	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I, Type III or Existing Granular Fill $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At Rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	-	3.3	-

\* For wing walls.

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

The coefficients provided in Table 10.1 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 Clause 6.12.3 of the CHBDC, 2014, 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 2.0 m for Granular B Type I, or at a depth of 1.7 m for Granular A or Granular B Type II.

## 12 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 1
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Ground Acceleration 0.05

The soil profile type at this site has been classified as Type III. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.5 could be used in seismic design.

## 13 SCOUR AND EROSION CONTROL

Erosion protection should be designed to protect the forward slopes and the slopes adjacent to the bridge abutments. Input from specialist in this field is required.

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

## 14 EXCAVATION AND GROUNDWATER CONTROL

It is anticipated that only comparatively shallow excavation will be required, 2 to 3 m maximum depending on what foundations are found under the existing structure.

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 fill within the depth of excavation may be classified as Type 3 soil above the water table.

The 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. It is anticipated that a hydraulic excavator will be suitable. Provision should be made for the handling of granular materials, potential obstructions in the fill, and cobbles and boulders.

Depending on the depth of excavation, a temporary protection system may be required. The system may consist of steel soldier pile and timber lagging walls or continuous sheet pile walls. The design of the temporary protection system, if required, is typically the responsibility of the Contractor and should be provided in accordance with OPSS PROV 539. The protection system should be design for Performance Level 2 (maximum 25 mm horizontal deflection).

The following parameters may be used in the design of the temporary protection system:

$\gamma$	=	20 kN/m <sup>3</sup> (embankment fill)
	=	17 kN/m <sup>3</sup> (silty clay)
$\gamma_w$	=	10 kN/m <sup>3</sup>
$K_a$	=	0.33 (embankment fill)
	=	0.41 (silty clay)
$K_p$	=	2.5 (silty clay)

Full hydrostatic pressure should be considered assuming a water level equal to the design creek 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.

The existing embankment fill and underlying native soils may contain occasional cobbles and boulders, which may interfere with installation of soldier piles or sheet piles. The Contractor should be advised of potential obstructions in the fill and native soils during installation.

Some groundwater may enter the excavation from the existing fill and minor groundwater seepage may occur. The volumes will depend on the depth of excavation, which cannot be accurately determined at this time. It is recommended that the contract contain a provisional item for dewatering.

Design of dewatering system that may be required is the responsibility of the Contractor.

## **15 CORROSION & SULPHATE ATTACK POTENTIAL**

The results of the corrosivity and sulphate analytical tests conducted on the embankment fill soil 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 soil or water corrosion on metal structural elements is considered to be moderate.
- Appropriate protection measures are recommended to address the moderate potential for corrosion on metal structure elements.

## **16 APPROACH EMBANKMENTS**

It is assumed no embankment grade raise or widening is proposed at this bridge site. No global slope instability of the existing approach embankments were noted during the time of the site investigation.

An inclination of the side and front slopes of the approach embankments should not be greater than 2H:1V. Erosion protection should be designed to protect the forward slopes and the creek bank slopes adjacent to the bridge abutments.

In view of the relatively low approach embankments and shallow creek banks, the global instability issues are not anticipated at this site.

## **17 CONSTRUCTION CONCERNS**

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

- No boreholes were drilled through the road directly behind the abutment due to traffic constraints. Rock fill including cobbles and boulders may be present in the abutment/timber crib fill. Occasional cobbles were noted in the existing embankment fill which was composed of reworked silty clay. Cobbles and boulders may be present and may interfere with excavations, driving piles, or installation of temporary protection system.

- 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.
- Confirmation that the backfills and approach fills are adequately placed and compacted to specifications will be required.

## 18 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. Anna Piascik, P.Eng. The report was reviewed by Alastair Gorman, P.Eng and P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

### Thurber Engineering Ltd.

Anna Piascik, P.Eng.  
Senior Geotechnical Engineer



Alastair Gorman, P.Eng.  
Senior Geotechnical Engineer/Senior Associate



P. K. Chatterji, P.Eng.  
Review Principal, Designated MTO 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 <sup>(1)</sup> '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


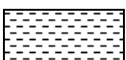

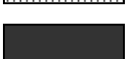

### ROCK WEATHERING CLASSIFICATION

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

### DISCONTINUITY SPACING

<b>Bedding</b>	<b>Bedding Plane Spacing</b>
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

<b>Rock Strength</b>	<b>Approximate Uniaxial Compressive Strength (MPa)</b>	<b>Approximate Uniaxial Compressive Strength (psi)</b>	<b>Field Estimation of Hardness*</b>
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 BC-01

1 OF 3

METRIC

GWP# 5027-14-00 LOCATION Boston Creek Bridge N 5 314 683.1 E 385 125.3 ORIGINATED BY AHF  
 HWY Local Rd. BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MFA  
 DATUM Geodetic DATE 2015.07.26 - 2015.07.27 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
224.8	GROUND SURFACE							20 40 60 80 100					
0.0	TOPSOIL: (250mm)							20 40 60 80 100					
224.5								20 40 60 80 100					
0.3	Silty <b>CLAY</b> , some gravel, trace cobbles, trace organics (roots and rootlets) Stiff to Very Stiff Brown Moist (FILL)		1	SS	15		224						
			2	SS	13								
223.4													
1.4	Silty <b>CLAY</b> , trace to some gravel, trace sand, grading to sandy Very Soft to Stiff Brown Moist		3	SS	4		223					16	5 41 38
			4	SS	0		222					0	35 34 31
			5	SS	12		221						
220.7													
4.1	Silty <b>CLAY</b> , trace sand Soft to Firm Grey Wet		6	SS	0		220						
							219	8.0 +					
			7	SS	0		218					0	0 33 67
							217	9.5 +					
			8	SS	0		216						
							215	14.0 +					
			9	SS	1								

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity 20  
15 10 5 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BC-01

2 OF 3

METRIC

GWP# 5027-14-00 LOCATION Boston Creek Bridge N 5 314 683.1 E 385 125.3 ORIGINATED BY AHF  
 HWY Local Rd. BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MFA  
 DATUM Geodetic DATE 2015.07.26 - 2015.07.27 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL LIMIT      MOISTURE      LIQUID CONTENT      LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE						
	Continued From Previous Page							20   40   60   80   100	20   40   60					
								4.4 +						
			10	SS	0		214				○			
							213	3.0 +						
			11	SS	1		212				○			
							211	8.5 +						
			12	SS	2						○			
							210	4.9 +						
			13	SS	1		209				○			
								4.9 +						
							208				○		0   0   34   66	
			14	SS	3									
							207	2.5 +						
			15	SS	0		206				○			
								2.0 +						
							205							

0 0 34 66

Continued Next Page


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

# RECORD OF BOREHOLE No BC-01

3 OF 3

METRIC

GWP# 5027-14-00 LOCATION Boston Creek Bridge N 5 314 683.1 E 385 125.3 ORIGINATED BY AHF  
 HWY Local Rd. BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY MFA  
 DATUM Geodetic DATE 2015.07.26 - 2015.07.27 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)		GR	SA	SI	CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
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204.6			16	SS	100/ 125																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
20.2	<b>GRANITE</b> , moderately weathered, medium strong to strong, pink Subvertical fractures at 20.37 to 20.52 and 20.65 to 20.73m Horizontal fractures at 20.42 and 20.50m  Subvertical fractures at 21.13 to 21.21, 21.24 to 21.36, 21.39 to 21.44 and 21.51 to 21.56m Horizontal fractures at 21.13 and 21.26m  Horizontal fractures at 21.92, 21.97, 22.17 and 22.23m Subvertical fractures at 22.20 to 22.27, 22.40 to 22.53, 22.53 to 22.58, 22.61 to 22.68, 22.71 to 22.76, 22.76 to 22.86, and 22.91 to 23.01m		1	RUN			204																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											</

# RECORD OF BOREHOLE No BC-02

1 OF 4

METRIC

GWP# 5027-14-00 LOCATION Boston Creek Bridge N 5 314 677.4 E 385 165.9 ORIGINATED BY AHF  
 HWY Local Rd. BOREHOLE TYPE NW Casing COMPILED BY MFA  
 DATUM Geodetic DATE 2015.07.27 - 2015.07.28 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
225.1	GROUND SURFACE							20 40 60 80 100										
0.0	<b>SAND</b> and <b>GRAVEL</b> , trace silt Compact Brown Moist (FILL)		1	SS	11		225											
224.5																		
0.6	Silty <b>CLAY</b> , some sand, trace gravel, trace organics (roots and rootlets) in the upper 0.5m zone, grading to sandy Very Soft to Firm Brown Moist to Wet		2	SS	6													
								224										
			3	SS	3													
								223										
			4	SS	0													
								222										
			5	SS	2													
221.0							221											
4.1	Silty <b>CLAY</b> , trace sand Firm to Stiff Grey Moist to Wet		6	SS	0													
							220											
							219											
			7	SS	1													
							218											
			8	SS	2													
							217											
							216											
			9	SS	0													

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BC-02

2 OF 4

METRIC

GWP# 5027-14-00 LOCATION Boston Creek Bridge N 5 314 677.4 E 385 165.9 ORIGINATED BY AHF  
 HWY Local Rd. BOREHOLE TYPE NW Casing COMPILED BY MFA  
 DATUM Geodetic DATE 2015.07.27 - 2015.07.28 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT      NATURAL LIMIT      MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL      × LAB VANE							
	Continued From Previous Page						215	14.0 +							
			10	SS	1		214					○			
								6.0 +							
			11	SS	1		213					○			
							212	8.0 +							
			12	SS	1		211					○			
								7.2 +							
			13	SS	1		210					○			
							209								
								7.6 +							
			14	SS	4		208					○			
								4.5 +							
							207								
			15	SS	4							○			
							206								
								4.4 +				○			

Continued Next Page

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

# RECORD OF BOREHOLE No BC-02

3 OF 4

METRIC

GWP# 5027-14-00 LOCATION Boston Creek Bridge N 5 314 677.4 E 385 165.9 ORIGINATED BY AHF  
 HWY Local Rd. BOREHOLE TYPE NW Casing COMPILED BY MFA  
 DATUM Geodetic DATE 2015.07.27 - 2015.07.28 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL LIMIT      MOISTURE      LIQUID CONTENT      LIMIT			UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED      + FIELD VANE						
							● QUICK TRIAXIAL      × LAB VANE							
							20   40   60   80   100				20   40   60			
	Continued From Previous Page		16	SS	5		205							
							204							
							203							
			17	SS	5		202					○		0   0   45   55
							201							
							200							
			18	SS	6		199					○		
							198							
							197							
			19	SS	4		196					○		
195.5														
29.6	End of sampling at 29.6m and start of DCPT.													

Continued Next Page

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



# RECORD OF BOREHOLE No BC-02

4 OF 4

METRIC

GWP# 5027-14-00 LOCATION Boston Creek Bridge N 5 314 677.4 E 385 165.9 ORIGINATED BY AHF  
 HWY Local Rd. BOREHOLE TYPE NW Casing COMPILED BY MFA  
 DATUM Geodetic DATE 2015.07.27 - 2015.07.28 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	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						195							
							194							
							193							
							192							
							191							
190.4 34.7	END OF BOREHOLE. WATER LEVEL AT 3.0m UPON COMPLETION. BOREHOLE BACKFILLED WITH CUTTINGS AND BENTONITE TO SURFACE.													

ONTMT4S 19-5161-252.GPJ 2015TEMPLATE(MTO).GDT 2/5/16

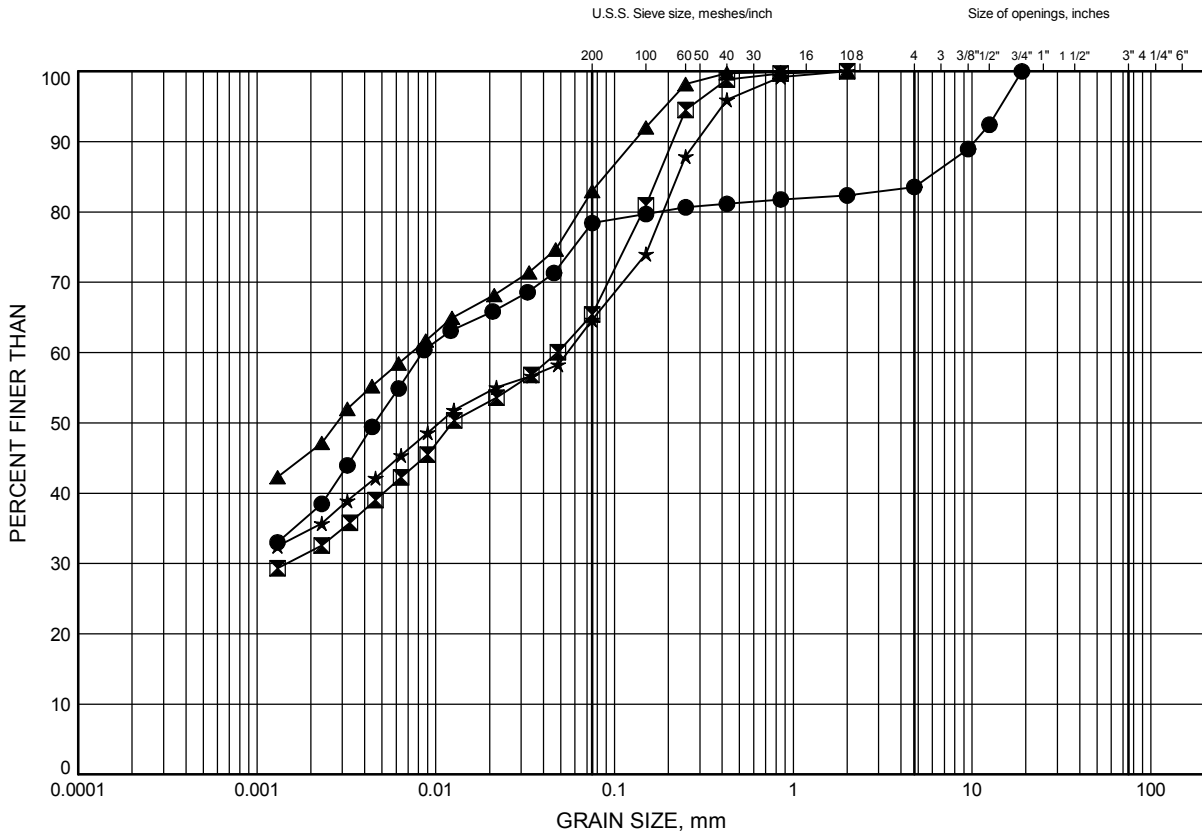
**Appendix B**  
**Geotechnical and Analytical**  
**Laboratory Test Results**

# Boston Creek Bridge

## GRAIN SIZE DISTRIBUTION

FIGURE B1

### Silty CLAY to Silty CLAY and SAND



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BC-01	1.83	222.97
⊠	BC-01	2.59	222.21
▲	BC-02	1.83	223.27
★	BC-02	3.35	221.75

Date February 2016

GWP# 5027-14-00



Prep'd MFA

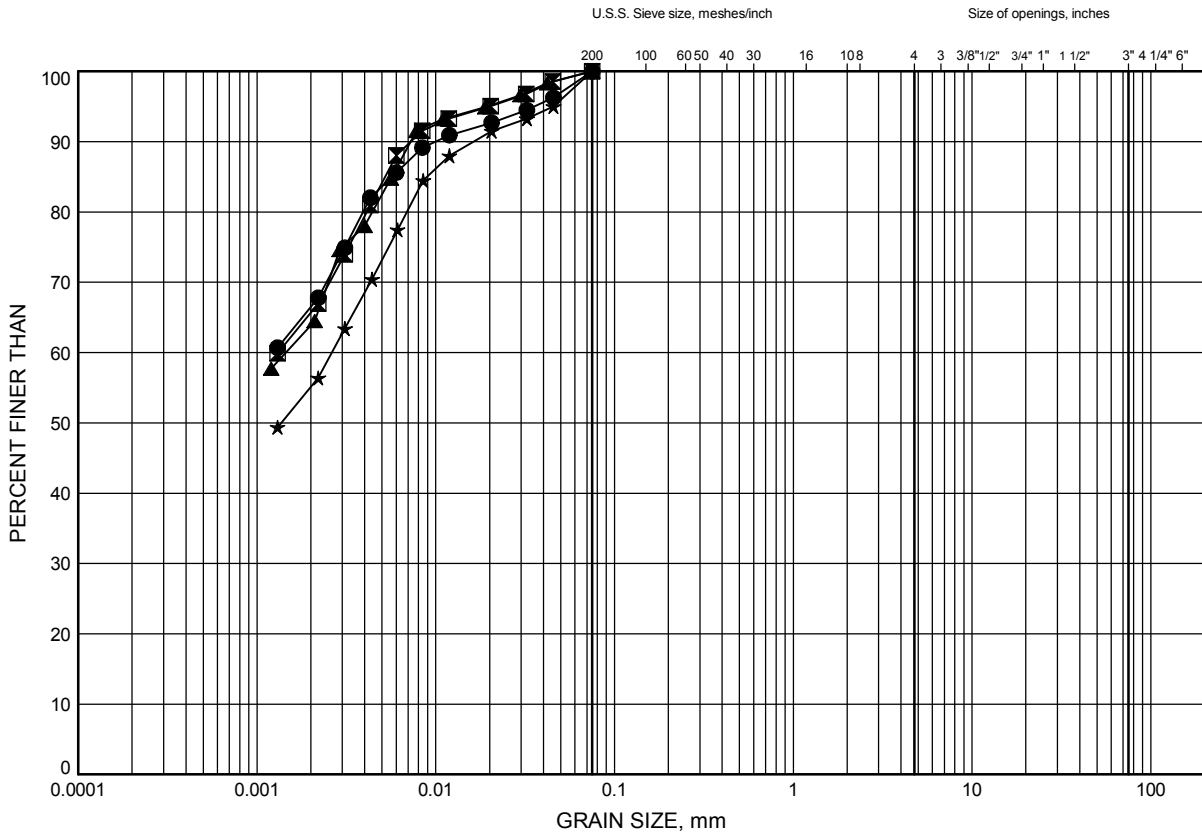
Chkd. DJP

# 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)
●	BC-01	6.40	218.40
◻	BC-01	17.07	207.73
▲	BC-02	4.88	220.22
★	BC-02	23.16	201.94

Date February 2016  
GWP# 5027-14-00

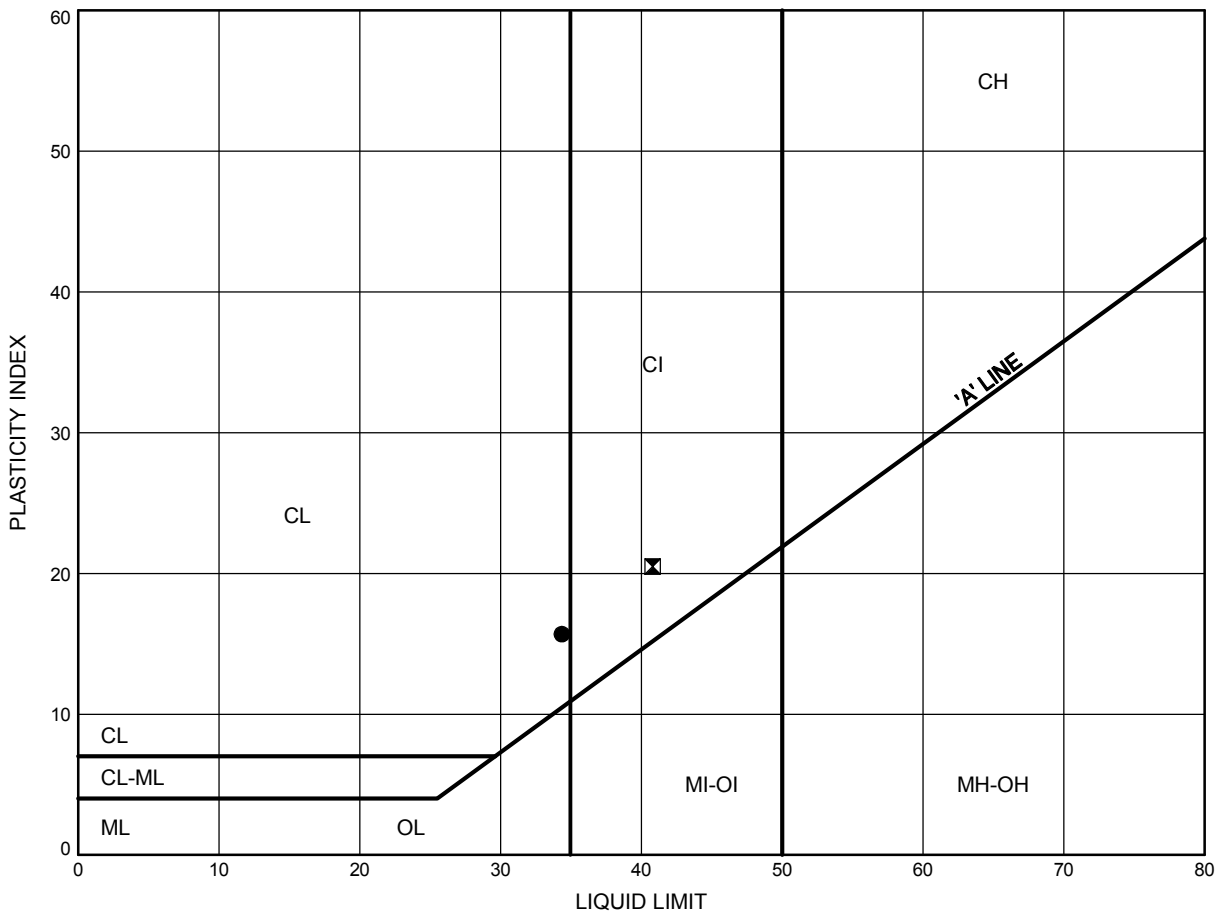


Prep'd MFA  
Chkd. DJP

Boston Creek Bridge  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B3

Silty CLAY to Silty CLAY and SAND



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BC-01	2.59	222.21
⊠	BC-02	1.83	223.27

Date February 2016  
 GWP# 5027-14-00

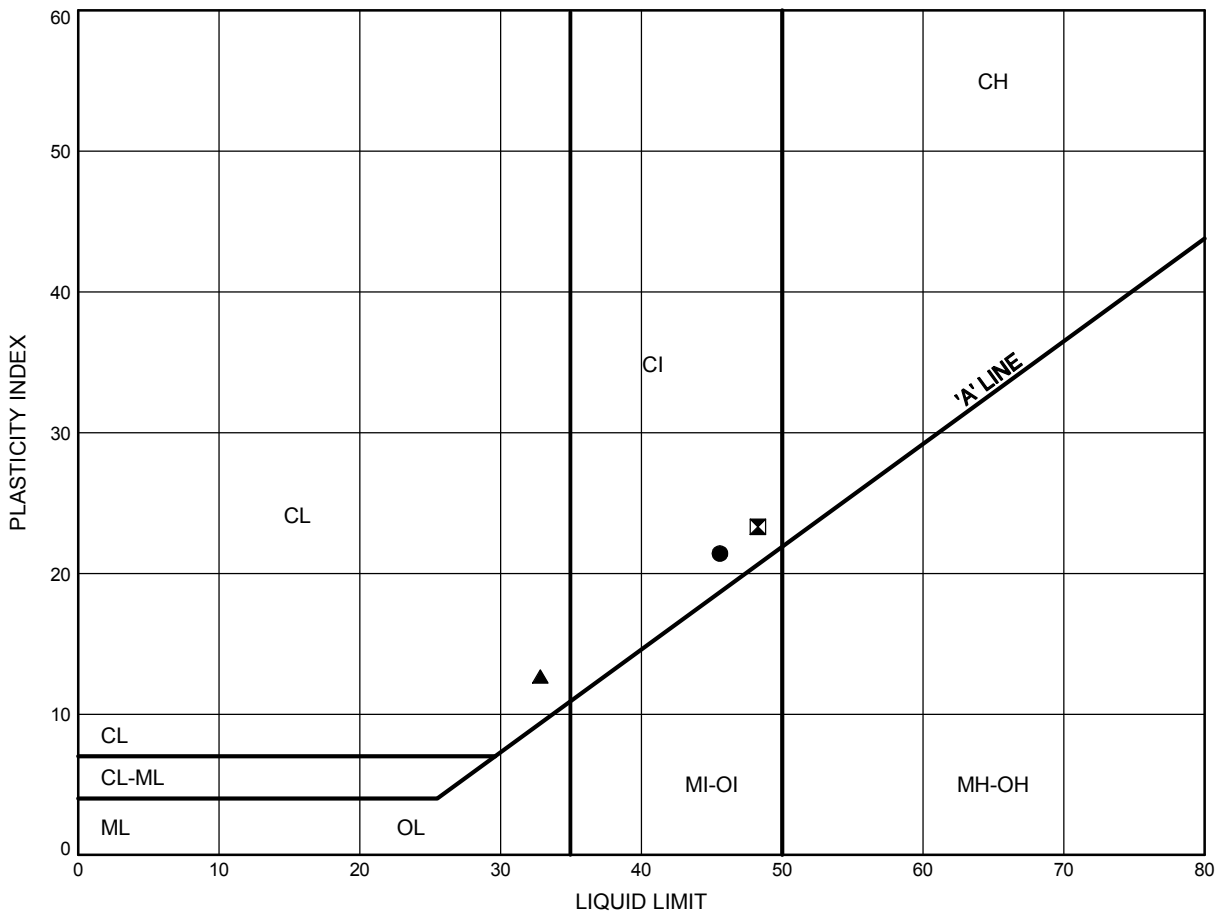


Prep'd MFA  
 Chkd. DJP

Boston Creek Bridge  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B4

Silty CLAY



**LEGEND**

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BC-01	6.40	218.40
⊠	BC-02	4.88	220.22
▲	BC-02	14.02	211.08

Date February 2016  
 GWP# 5027-14-00



Prep'd MFA  
 Chkd. DJP



## Certificate of Analysis

CLIENT NAME: THURBER ENGINEERING LTD

PROJECT: 19-5161-252

SAMPLING SITE:

AGAT WORK ORDER: 15T004093

ATTENTION TO: Deanna Pizycki

SAMPLED BY: DP

### Corrosivity Package

SAMPLE TYPE: Soil

SAMPLE ID: 6816995

DATE RECEIVED: Aug 06, 2015

DATE SAMPLED: Aug 05, 2015

DATE REPORTED: Aug 12, 2015

SAMPLE DESCRIPTION: BC-2 SS2 2'6" -4'6"

PARAMETER	UNIT	RESULT	G / S	RDL	DATE ANALYZED	INITIAL	DATE PREPARED
Sulfide	%	0.02		0.01	Aug 11, 2015	ME	Aug 10, 2015
Chloride (2:1)	µg/g	3		2	Aug 11, 2015	JC	Aug 11, 2015
Sulphate (2:1)	µg/g	11		2	Aug 11, 2015	JC	Aug 11, 2015
pH (2:1)	pH Units	7.70		NA		TM	
Electrical Conductivity (2:1)	mS/cm	0.162		0.005	Aug 11, 2015	TM	Aug 11, 2015
Resistivity (2:1)	ohm.cm	6170		1	Aug 11, 2015	SYS	Aug 11, 2015
Redox Potential (2:1)	mV	302		5	Aug 11, 2015	TM	Aug 11, 2015

#### COMMENTS:

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

\* Sulphide analysis was performed at AGAT Laboratories Vancouver.

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:



## Certificate of Analysis

CLIENT NAME: THURBER ENGINEERING LTD

PROJECT: 19-5161-252

SAMPLING SITE:

AGAT WORK ORDER: 15T004153

ATTENTION TO: Deanna Pizycki

SAMPLED BY:

### Inorganic Chemistry (Water)

SAMPLE TYPE: Water

SAMPLE ID: 6825043

DATE RECEIVED: Aug 06, 2015

DATE SAMPLED: Jul 28, 2015

DATE REPORTED: Aug 13, 2015

SAMPLE DESCRIPTION: Boston Creek

PARAMETER	UNIT	RESULT	G / S	RDL	DATE ANALYZED	INITIAL	DATE PREPARED
pH	pH Units	7.96		NA	Aug 10, 2015	BP	Aug 10, 2015
Langelier Index		0.34			Aug 11, 2015	SYS	Aug 11, 2015
Total Dissolved Solids	mg/L	138		20		AP	Aug 10, 2015
Alkalinity (as CaCO <sub>3</sub> )	mg/L	112		5	Aug 10, 2015	BP	Aug 10, 2015
Total Hardness (as CaCO <sub>3</sub> )	mg/L	127		0.5	Aug 11, 2015	SYS	Aug 11, 2015
Chloride	mg/L	2.96		0.10	Aug 07, 2015	JC	Aug 07, 2015
Sulphate	mg/L	17.5		0.10	Aug 07, 2015	JC	Aug 07, 2015
Sulphide	mg/L	<0.05		0.05	Aug 11, 2015	SN	Aug 11, 2015
Resistivity	ohms.cm	3820			Aug 10, 2015	SYS	Aug 10, 2015
Redox Potential	mV	312		5	Aug 12, 2015	BG	Aug 12, 2015

**COMMENTS:**

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

Certified By:



**Appendix C**  
**Site Photographs**



**Photograph 1 – Bridge Approach Looking East**



**Photograph 2 – Bridge Approach Looking West**





**Photograph 3 – South Elevation Looking East**



**Photograph 4 – North Elevation Looking East**





**Photograph 5 – East Front Slope, July 26, 2015**



**Photograph 6 - East Abutment, July 26, 2015**





**Photograph 7 – Looking South (Downstream)**



**Photograph 8 – Looking North (Upstream)**

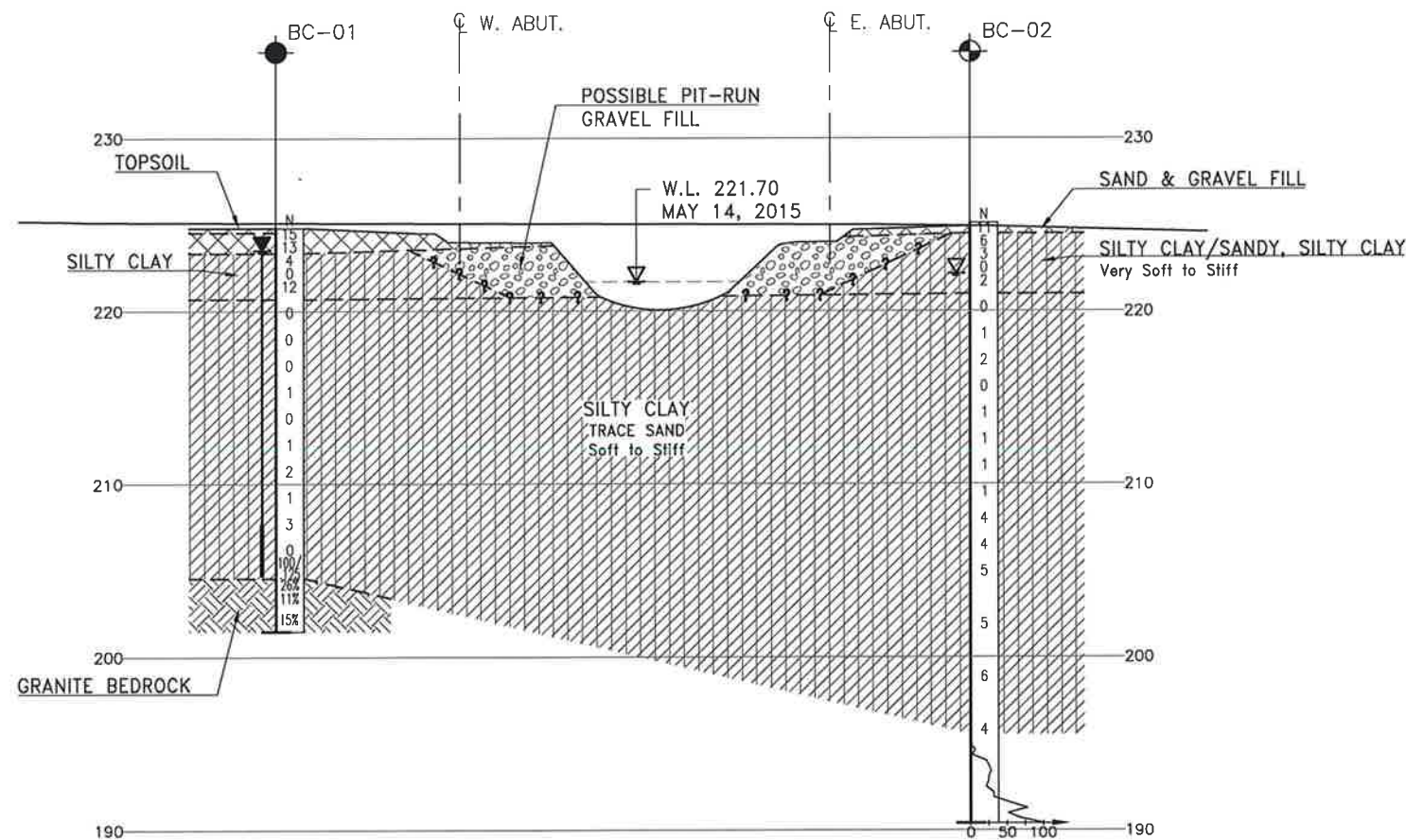
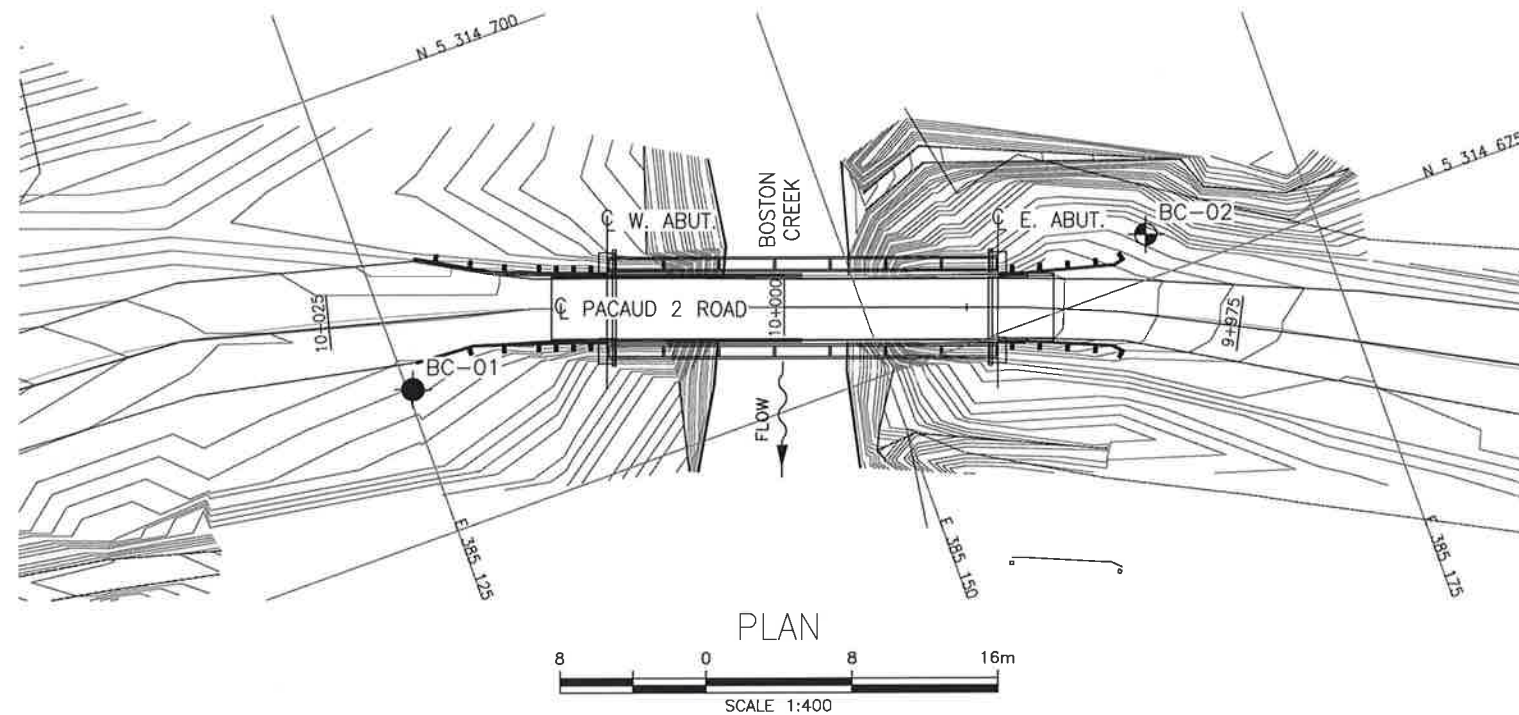




**Photograph 9 – East Front Slope – close-up**

**Appendix D**  
**Borehole Locations and Soil Strata Drawing**



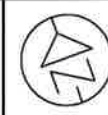


PROFILE ALONG C PACAUD 2 ROAD

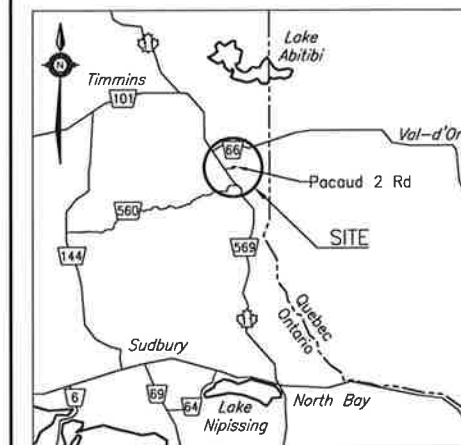
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No  
GWP No 5027-14-00

BOSTON CREEK  
BRIDGE  
REHABILITATION  
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	EASTING
BC-01	224.8	5 314 683.1	385 125.3
BC-02	225.1	5 314 677.4	385 165.9

-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.

GEOCRE No. 31M-112



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	DJP	CHK MEF	CODE
DRAWN	MFA	CHK DJP	SITE 47-098
			STRUCT
			OWG 1



**Appendix E**  
**List of SPs and OPSS**

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

- OPSS.PROV 501
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS 903
- OPSS.PROV 1010
- SS103-11