



**THURBER** ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
HEWITSON CREEK BRIDGE REPLACEMENT  
HIGHWAY 7047 (LAKESHORE ROAD)  
THUNDER BAY DISTRICT, ONTARIO  
G.W.P. 6905-10-00, SITE NO. 48C-129  
Geocres Number: 42D-41**

**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 bridge carrying Highway 7047 (Lakeshore Road) over Hewitson Creek, located in the District of Thunder Bay (Unorganized), 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 and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to Hatch Mott MacDonald (HMM), under the Ministry of Transportation Ontario (MTO) Agreement Number 6013-E-0027, Work Item 5.

**2 SITE DESCRIPTION**

The Hewitson Creek Bridge is located on Highway 7047 (Lakeshore Road) in Selim, approximately 1.8 km east from the intersection of Lakeshore Road and Highway 11, and about 8 km east of Rossport. The existing bridge is a single span modular structure with a length of 30.7 m and width of 3.5 m, supported on timber crib abutments. The west approach to the bridge is relatively wide and low, and merges into a driveway on the south side. The east approach embankment is narrow and approximately 1.5 to 2.0 m in height above the flood plain of the creek.

Hewitson Creek flows from the north to south, from Whitesand Lake to Lake Superior about 150 m to the south of the bridge. The creek was approximately 15 m wide and up to 1 m deep at the bridge site during the time of the field investigation. The creek channel is lined with cobbles and boulders. The

surrounding lands are generally heavily wooded and undeveloped, with the exception of recreational and residential properties fronting on Lake Superior.

Photographs in Appendix C show the general nature of the site.

The site lies within the physiographic region known as the Wawa Subprovince of the Superior Province of the Canadian Shield. The region is characterized by massive to foliated granodiorite and granite rocks. Overlying bedrock are nearshore and beach deposits consisting of predominantly glacio-lacustrine silt, sand, gravelly sand and gravel.

### 3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between November 3 and 6, 2015. The field testing consisted of drilling and sampling five boreholes, identified as Boreholes BH-01 to BH-05. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

Boreholes BH-01, BH-02 and BH-04 were drilled near the proposed abutments and were terminated at depths ranging from 14.9 to 19.4 m, including coring 3.2 to 4.6 m into bedrock. Dynamic cone penetration tests were advanced at a distance of approximately 1.5 m west and 2 m east of Boreholes BH-02 and BH-04, respectively, to supplement the sampled borehole information. Boreholes BH-03 and BH-05 were drilled to a depth of 5.2 m at the west and east approaches, respectively.

Details of the drilling program, including borehole locations, drilling depths, and completion details are summarized in Table 3.1 below.

**Table 3.1 – Details of Boreholes**

Location	Boreholes	Borehole Depth/ Base of Hole Elevation (m)	Completion Details
West Approach	BH-03	5.2 / 181.7	Borehole backfilled with bentonite holeplug and cuttings to surface.
West Abutment	BH-01	17.9 / 169.1	Borehole backfilled with bentonite holeplug and cuttings to surface.
	BH-02 DCPT	19.4 / 167.4 15.8 / 171.0	Boreholes backfilled with bentonite holeplug and cuttings to surface.
East Abutment	BH-04 DCPT	14.9 / 172.1 12.1 / 174.9	Borehole backfilled with bentonite holeplug and cuttings to surface.
East Approach	BH-05	5.2 / 181.8	Borehole backfilled with bentonite holeplug and cuttings to surface.

All boreholes were advanced using a CME55 truck-mounted drill rig in combination with hollow stem augers and NW casing/coring methods. Samples of the encountered soils were obtained from the boreholes at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Field vane shear tests were conducted in cohesive soils for determination of undrained shear strengths using MTO Standard “N” size vane and a calibrated torque wrench.

Core samples of the underlying bedrock were recovered from three boreholes using NQ rock coring equipment. All rock cores were logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

A member of Thurber’s technical staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber’s laboratory for further examination and testing. The ground surface elevations at the borehole locations were obtained from the drawings provided by HMM.

Groundwater conditions in the open boreholes were observed during the drilling operations.

#### **4 LABORATORY TESTING**

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets included in Appendix A. Selected samples were also subjected to grain size analysis and Atterberg Limits testing, and the results of this testing program are summarized on the Record of Borehole sheets in Appendix A, and are shown on the figures included in Appendix B.

Point load tests (PLT) were performed on selected intact rock core samples. Unconfined compressive strengths (UCS) of the rock cores correlated from the PLT results are shown on the Record of Borehole sheets in Appendix A.

#### **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 in Appendix F. 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 borehole locations.

In summary, the subsurface stratigraphy encountered below the existing embankment fill at the site consists of cohesionless glaciofluvial deposits of sand and gravel, overlying silty sand to sandy silt, underlain by a layer of silty clay, which in turn is underlain by bedrock. Descriptions of the individual strata are presented below.

## 5.1 Sand Fill

Fill forming the existing roadway embankment was encountered in all boreholes. The fill consists of sand with trace to some gravel, trace silt, and occasional to frequent cobbles. A 30 mm thick asphaltic surface treatment was noted in Borehole BH-04. The sand fill extended to depths of 0.8 to 2.0 m (Elev. 186.1 to 185.0).

SPT 'N' values recorded in the embankment fill at the west abutment ranged from 3 to 7 blows per 0.3 m penetration, indicating a very loose to loose relative density. At the east abutment and west approach, N-values of 10 to 21 blows per 0.3 m were recorded, indicating a compact condition.

Moisture contents of the sand fill ranged from 1% to 8%, locally 11% and 16% near the water level at the east abutment.

The results of grain size analyses conducted on fill samples are provided on the Record of Borehole sheets in Appendix A, and are illustrated in Figure B1 of Appendix B. The results are also summarized below:

Gravel %	2 to 5
Sand %	90 to 94
Silt and Clay %	4 to 5

## 5.2 Sand and Gravel

A cohesionless deposit of sand and gravel was encountered underlying the sand fill in all boreholes. The sand and gravel contained trace to some silt and occasional to frequent cobbles. In this regard, it was noted that the creek channel and banks are lined with cobbles and boulders. The deposit was 1.5 to 2.3 m thick, with a lower boundary encountered between 2.3 and 3.8 m depth (Elev. 184.6 and 183.1).

SPT 'N' values recorded in the sand and gravel typically varied from 15 to 45 blows per 0.3 m penetration, indicating a compact to dense relative density. Two values of 100 blows for less than 0.3 m penetration were also obtained, indicative of the presence of cobbles in the deposit.

Moisture contents of the sand and gravel ranged from 1% to 15%, typically less than 7%.

The results of grain size analyses conducted on three samples of this deposit are provided on the Record of Borehole sheets in Appendix A, and are illustrated in Figure B2 of Appendix B. The results are also summarized below:

Gravel %	39 to 54
Sand %	43 to 47
Silt and Clay %	3 to 14

### 5.3 Silty Sand to Sandy Silt

A cohesionless deposit consisting of various proportions of sand and silt underlies the sand and gravel in all boreholes. The deposit was classified as silty sand to sandy silt with trace gravel and trace clay. Occasional sand layers were encountered in this deposit.

Where fully penetrated in Boreholes BH-01, BH-02 and BH-04, the thickness of the deposit ranged from 6.0 to 9.2 m, with the lower boundary sloping down from 9.7 m depth (Elev. 177.3) in Borehole BH-04 on the east side of the creek, to approximately 13.0 m depth (Elev. 174.0) in Boreholes BH-01 and BH-02 on the west side of the creek. Boreholes BH-03 and BH-05 located at the approaches were terminated in this deposit at a depth of 5.2 m (Elev. 181.7 and 181.8).

SPT 'N' values recorded in the silty sand to sandy silt varied from zero (Weight of Hammer) to 29 blows per 0.3 m of penetration, indicating a very loose to compact relative density. Natural moisture contents ranged from 18% to 25%.

The results of grain size analyses conducted on samples of this deposit are provided on the Record of Borehole sheets in Appendix A, and are illustrated in Figures B3 to B5 of Appendix B. The results of the grain size analyses are summarized below:

	<u>Sandy Silt to Silty Sand</u>	<u>Sand Layer</u>
Gravel %	0 to 2	0
Sand %	24 to 67	94
Silt %	28 to 71	6
Clay %	3 to 6	

### 5.4 Silty Clay

A layer of grey silty clay with trace sand underlies the sand/silt deposit in Boreholes BH-01, BH-02 and BH-04. The clay deposit was 1.5 to 2.0 m in thickness and extended to the bedrock surface at depths of 11.3 to 14.8 m (Elev. 175.7 to 172.0). Occasional silty sand seams were noted in the deposit, and a 200 mm thick layer of very dense silty sand was noted immediately above the bedrock surface in Borehole BH-01.

SPT 'N' values of zero (Weight of Hammer) and 3 blows per 0.3 m of penetration were recorded in the silty clay. Field vane shear tests (VST) conducted in Borehole BH-02 measured undrained shear strengths of 80 to 85 kPa. Based on the SPT and VST data, the consistency of the deposit varied from soft to stiff.

The results of grain size analyses conducted on samples of the silty clay are provided on the Record of Borehole sheets in Appendix A, and are illustrated in Figure B6 of Appendix B. The results are summarized as follows:



Gravel %	0
Sand %	0
Silt %	53 to 55
Clay %	45 to 47

The results of Atterberg Limits testing are provided on the Record of Borehole sheets in Appendix A and are illustrated on the Plasticity Chart (Figure B7). Liquid limits ranged from 31 to 35 and the plasticity index ranged from 14 to 18, indicating low to medium plasticity.

Natural moisture contents of the silty clay ranged from 21% to 35%.

## 5.5 Bedrock

Bedrock was encountered below the silty clay in Boreholes BH-01, BH-02 and BH-04 at depths of 11.3 to 14.8 m (Elev. 175.7 to 172.0). The rock was proved by coring and details of rock coring are presented on the Record of Borehole sheets in Appendix A. The bedrock was described as slightly weathered to fresh, grey to reddish brown granite to granodiorite with occasional light grey quartz inclusions.

Table 5.1, below, summarizes the depth to bedrock and bedrock surface elevations determined at the borehole locations.

**Table 5.1 - Depths and Elevations of Bedrock Surface**

Nearest Foundation Unit	Borehole Number	Top of Bedrock	
		Depth (m)	Elevation (m)
West Abutment	BH-01	14.7	172.3
	BH-02	14.8	172.0
East Abutment	BH-04	11.3	175.7

Variations in the bedrock surface within short distances from the borehole locations should be anticipated.

Total Core Recovery (TCR) in the bedrock ranged from 97% to 100%, locally 68% in the initial run in Borehole BH-02. The Rock Quality Designation (RQD) determined from the recovered cores was typically 80 to 100%, indicating a good to excellent rock quality. The upper 1.5 m of bedrock in Borehole BH-02 was of fair quality with fractures and a measured RQD of 52%. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 4.

The unconfined compressive strength (UCS) of the rock, estimated from the results of point load tests conducted on the rock core samples, ranges from 175 to 233 MPa, indicating a very

strong intact rock. The point load test results (average values) carried out for rock samples are included on the Record of Borehole sheets in Appendix A.

## 5.6 Water Levels

Water levels in the boreholes were observed during drilling operations and upon completion of drilling. No standpipe piezometers were installed at this site. Water was used during the drilling and coring operations, and therefore the water levels measured on completion may not reflect prevailing groundwater levels at the site.

The water levels measured in the open boreholes are summarized in Table 5.2.

**Table 5.2 - Water Level Measurements**

Borehole Number	Date	Water Level (m)		Comments
		Depth	Elev.	
BH-01	Nov. 4, 2015	2.1	184.9	Water level in borehole during drilling.
BH-02	Nov. 5, 2015	2.1	184.7	Water level in borehole during drilling.
BH-03	Nov. 6, 2015	3.6	183.3	Water level on completion of drilling.
BH-04	Nov. 3, 2015	2.1	184.9	Water level on completion of drilling.
BH-05	Nov. 4, 2015	1.4	185.6	Water level on completion of drilling.

In light of cohesionless deposits overlying the site, the groundwater level will be governed by the water level in the Hewitson Creek. The preliminary General Arrangement drawing indicates the water level in Hewitson Creek was at Elev. 184.7 on Aug. 16, 2012.

The water level in the creek and groundwater levels are expected to fluctuate seasonally and subject to precipitation patterns, and therefore may vary from the levels presented herein.

## 6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. in consultation with HMM. The coordinates and the ground surface elevations for the boreholes were established based on topographic survey information provided by HMM. Thurber obtained utility clearances for the borehole locations prior to drilling.

RPM Drilling Inc. of Thunder Bay, Ontario supplied a track-mounted CME-55 drill rig and conducted the drilling, sampling and in-situ testing operations for the boreholes.

The drilling operations were supervised by Mr. Chris Murray of Thurber. Overall supervision of the field program was carried out by Mr. Stephane Loranger of Thurber.

The report was prepared by Ms. Anna Piascik, P.Eng and reviewed by Mr. Murray Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

### THURBER ENGINEERING LTD.

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**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**7 GENERAL**

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for design of a new bridge to replace the existing Hewitson Creek Bridge on Highway 7047 (Lakeshore Road) in Selim, Ontario.

The existing bridge is a single-span timber deck modular structure supported on timber crib abutments. The bridge span is approximately 30.7 m long and the width is 3.5 m. The west approach to the bridge is relatively wide and merges into an adjoining driveway. The east approach embankment is narrow and approximately 1.5 to 2.0 m in height above the flood plain of the creek.

It is understood that the current plan is to install a new modular bridge adjacent to the existing bridge for use as a detour structure while new bridge foundations are constructed on the existing alignment. The temporary detour bridge will then be moved sideways onto the new foundations on the existing/permanent alignment.

Based on the information provided by HMM, the proposed replacement bridge will be a single-span modular structure with a length of 30.3 m and width of 4.2 m. It is anticipated that a grade raise will not be required for the approach embankments.

The discussions and recommendations presented in this report are based on the information provided by HMM and on the factual data obtained in the course of the investigation.

**8 STRUCTURE FOUNDATIONS**

The soil stratigraphy at the bridge site consists of existing sand embankment fill underlain by a layer of sand and gravel ranging in thickness from 1.5 to 2.3 m, overlying a deposit of silty sand to sandy silt extending to depths of 9.7 to 13.0 m. Cobbles were noted within the fill and native sand and gravel.

A 1.5 to 2.0 m thick layer of silty clay underlies the cohesionless deposits and overlies granite to granodiorite bedrock at depths ranging from 11.3 to 14.8 m.

Groundwater levels are expected to reflect the water level in the creek, shown on the preliminary GA drawing to be at Elev. 184.7 on August 16, 2012.

Based on the subsurface conditions, consideration was given to supporting the bridge on the following foundation types:

- Spread footings on native soils and/or engineered granular fill;
- Driven steel H-piles;
- Augered caissons (drilled shafts).

A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix D.

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

### **8.1 Spread Footings on Native Soils and/or Engineered Fill**

Based on the subsurface conditions encountered at this site, the use of spread footings to support the bridge is considered feasible from a geotechnical perspective. However, spread footings are not the recommended option in consideration of the following:

- Excavation to the native sand and gravel for preparation of the founding surface will extend approximately to the reported creek water level, and potentially below water levels during periods of higher flow.
- The depth of the existing timber cribs is unknown, and removal of these structures may result in disturbance to the native subgrade soils below the water level and design founding level.
- The potential exists for scour of the creek banks and founding soils, undermining the footings.

Spread footings should be founded on the compact to dense native sand and gravel encountered at depths of 1.5 to 2.0 m (Elev. 185.0 to 185.5) at the abutments, or on engineered granular fill placed over the native sand and gravel to establish the design founding level.

The base of the engineered fill pad must be placed on the native sand and gravel below the level of all existing sand fill and timber cribs. The engineered fill must consist of OPSS Granular "A" placed in 150 mm lifts and compacted to 100% of its SPMDD at  $\pm 2\%$  of optimum moisture content. The fill should extend laterally beyond the edge of footing a distance of at least 1.0 m plus the thickness of the engineered fill.

Provided a minimum footing width of 1.5 m is maintained, footings bearing on the native compact to dense soil or engineered fill may be designed for the following resistance values:

Factored Geotechnical Resistance at ULS	350 kPa
Geotechnical Reaction at SLS	200 kPa

The width of footing should be designed based on the load demand for the bridge structure.

The geotechnical resistances are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC 2014, Clause 6.10.3 and Clause 6.10.4.

The geotechnical reaction at SLS for footings is based on an estimated total settlement not exceeding 25 mm. This settlement is expected to be essentially complete by the end of construction.

The lateral resistance of the footings may be computed using an unfactored friction coefficient of 0.6 between the concrete footing and underlying sand and gravel or engineered fill.

Scour and erosion protection must be provided for the footings.

## **8.2 Steel H-Pile Foundations**

### **8.2.1 Axial Resistance**

The ground conditions at the site are considered suitable for the use of driven steel H-pile foundations at the bridge abutments. To develop the required capacity, the piles should be driven to bedrock. The anticipated pile tip elevations for piles driven to bedrock are as follows:

**Table 8.1 – Anticipated Pile Tip Elevations on Bedrock**

<b>Foundation Unit</b>	<b>Borehole Number</b>	<b>Anticipated Pile Tip Elevation on Bedrock (m)</b>
West Abutment	BH-01	172.3
	BH-02	172.0
East Abutment	BH-04	175.7

The pile tip elevations may vary from those indicated in the table.

For an HP 310 x 110 pile driven to granite/granodiorite bedrock, the factored geotechnical resistance at ULS is estimated to be 9,000 kN per pile. This value is expected to exceed the structural capacity of the pile, and therefore the factored axial pile resistance should be based on the structural capacity of the pile as determined by the structural designer. The SLS condition will not govern the design of piles founded on bedrock.

Piles should be located to avoid encountering the existing timber cribs during installation.

### 8.2.2 Pile Tips

Pile tip protection is recommended for driven H-piles to prevent pile damage when setting the piles on bedrock or if cobbles and boulders are encountered in the fill and sand and gravel deposit. 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.

### 8.2.3 Pile Installation

Pile installation should be in accordance with OPSS 903.

The appropriate pile driving note is “Piles to be driven to bedrock”.

If the proposed bridge design requires that the deviation at the top of the pile be limited to tight tolerances, a driving template or other means may be required to achieve the specified maximum deviation.

The possibility exists that cobbles and boulders may be encountered while driving piles through the existing embankment fill or the underlying sand and gravel. The Contract Documents should contain a NSSP alerting Bidders to:

- the presence of the cobbles and possible boulders within the embankment fill and underlying sand and gravel deposit;
- the need to remove, dislodge or otherwise penetrate these obstructions to advance the piles to bedrock while meeting the specified deflection tolerances;
- the possibility of piles meeting refusal at different depths on the bedrock surface.

An NSSP addressing these issues is included in Appendix E.

### 8.2.4 Pile Lateral Resistance

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

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

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

where  $z$  = depth of embedment of pile (m)

$D$  = pile width or diameter (m)

$n_h$  = coefficient related to soil relative density ( $\text{kN/m}^3$ )

$\gamma'$  = effective unit weight ( $\text{kN/m}^3$ )

$K_p$  = passive earth pressure coefficient

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

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

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

where  $S_u$  = undrained shear strength (kPa)  
 $D$  = pile width or diameter (m)

The above equations and recommended parameters in Table 8.2 below may be used to analyse the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

**Table 8.2 - Soil Parameters for Lateral Pile Resistance**

Location	Soil Strata	Elevation (m)		$\gamma$ (kN/m <sup>3</sup> )	$n_h$ (kN/m <sup>3</sup> )	$K_p$	$S_u$ (kPa)
		Top	Bottom				
West Abutment	Sand Fill	GS	185.5	20	3,000	3.0	-
	Sand and Gravel	185.5	183.2	11*	5,000	3.5	-
	Sand / Silt	183.2	174.0	10*	3,000	3.0	-
	Silty Clay	174.0	172.2	9*	-	-	80
East Abutment	Sand Fill	GS	185.0	20	3,000	3.0	-
	Sand and Gravel	185.0	183.5	11*	5,000	3.5	-
	Sand / Silt	183.5	177.3	10*	3,000	3.0	-
	Silty Clay	177.3	175.7	9*	-	-	80

\*  $\gamma$  = submerged unit weight to be used for soils below the groundwater level.

The spring constant,  $K_s$ , for analysis may be obtained from the expression,  $K_s = k_s L D$  (kN/m), where  $k_s$  is the coefficient of horizontal subgrade reaction (kN/m<sup>3</sup>),  $D$  is the pile width (m) and  $L$  is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance,  $P_{ult}$ , may be obtained from the expression,  $P_{ult} = p_{ult} L D$ . This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements.

The modulus of subgrade reaction 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 8.3. Intermediate values may be obtained by linear interpolation.



**Table 8.3 - 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

Horizontal loads may be resisted by means of battered piles (i.e. for H-pile case) if load requirements exceed the available lateral pile resistances.

#### **8.2.5 Downdrag**

Road grades will not be raised and no additional placement of fill is anticipated. Considering the presence of predominantly cohesionless deposits, downdrag on the piles is not considered to be an issue at this site.

#### **8.3 Augered Caissons (Drilled Shafts)**

Caisson installation at this site would extend through cohesionless soils below the groundwater table and require the use of a temporary liner and/or drilling mud to support the caisson sidewalls. Sealing of the caisson liner into bedrock to prevent inflow of water and cohesionless soils from the base will be problematic. The presence of cobbles and boulders in the native soils may also significantly impact caisson excavation. The use of caissons is therefore not recommended and design recommendations have not been developed for this foundation option.

#### **8.4 Recommended Foundation**

Spread footings founded on native sand and gravel / engineered fill and steel H-piles driven to bedrock are both considered to be feasible foundation options for this bridge. Considering the potential scour and constructability issues with spread footings, driven H-piles are the preferred foundation type from a geotechnical viewpoint.

Spread footings may be preferred for support of the temporary bridge based on cost considerations and lower performance requirements for a temporary structure.

#### **8.5 Scour and Erosion Protection**

Erosion protection should be provided along any soil surfaces that may be in contact with the creek flow. In particular, erosion protection must be provided along the creek banks to prevent undermining of spread footings, if employed.

Typically, erosion protection along the creek will consist of rock protection. A vegetation cover should be established on all other exposed surfaces to protect against surficial erosion, in general accordance with OPSS.PROV 804.

## **8.6 Frost Cover**

The depth of frost penetration at this site is 2.2 m. Typically, the base of all footings, if employed, must be provided with a minimum of 2.2 m of earth cover as protection against frost action.

Considering the negligible susceptibility to frost action of the on-site sand and gravel, frost protection is not required for spread footings supporting a flexible modular structure that can accommodate potential movement.

## **8.7 Impact on Existing and Detour Bridges**

During construction of foundations for the detour bridge, care should be taken to avoid undermining and disturbing the crib abutments supporting the existing bridge. Further, care must be taken to avoid disturbance of the detour bridge foundation (if supported on footings) during construction of the new foundations for the permanent bridge.

Implementation of a monitoring program (including establishment of adequate benchmarks outside the zone of potential influence and acquirement of baseline readings in advance of construction) should be considered for the duration of foundation construction to identify any movement of the existing and detour structures. Appropriate monitoring points and tolerable levels of movement should be specified by the structural designer. If movements exceed tolerable levels, the Contractor must be prepared to jack and/or shim the bridge structure.

# **9 LATERAL EARTH PRESSURES**

Backfill to the structure (if applicable) should be in accordance with OPSS 902 and should consist of Granular A, Granular B Type II or Granular B Type III material. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures, if required, should be restricted in accordance with OPSS 501.

Earth pressures acting on the walls may be assumed to impose a triangular distribution. For a fully drained backfill, the pressures should be computed in accordance with the CHBDC 2014, Clause 6.12, but are generally 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 below)

$\gamma$  = unit weight of retained soil (see table below)

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

$q$  = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment walls are dependent on the material used as backfill. Recommended values of the earth pressure coefficients are shown in Table 9.1.

**Table 9.1 – Coefficients of Lateral Earth Pressure (K)**

Wall Condition	Earth Pressure Coefficient, K			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$ ; $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Existing Embankment Fill $\phi = 32^\circ$ ; $\gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-

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 factors in Table 9.1 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.

## 10 SEISMIC CONSIDERATIONS

Based on the Standard Penetration Test N-values obtained in the underlying silty sand to sandy silt, Site Class E (soft soil) should be assumed to evaluate the seismic site response, as per Table 4.1, Clause 4.4.3.2 of the CHBDC 2014.

The peak ground acceleration, PGA, for a 2% in 50 year probability of exceedance at this site is 0.033 as per the National Building Code of Canada (NBCC).

In accordance with Clause 4.6.5 of the CHBDC 2014, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 10.1 may be used:

**Table 10.1 – Earth Pressure Coefficients for Earthquake Loading**

Condition	Earth Pressure Coefficient (K)	
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I or Existing Sand Fill $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$
Active ( $K_{AE}$ )*	0.28	0.31
Passive ( $K_{PE}$ )	3.7	3.2
At Rest ( $K_{OE}$ )**	0.44	0.49

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

\*\* After Woods

The loose, fine-grained cohesionless soils underlying this site are considered to be prone to liquefaction. However, in view of the estimated peak ground acceleration, liquefaction is not a concern at this site.

## 11 APPROACH EMBANKMENTS

The existing approach embankments are in the order of 1.5 to 2.0 m above the surrounding ground and appear to be performing satisfactorily. No grade raise is envisioned.

Provided that the embankment is reconstructed at the same slope inclination as the existing embankment, but not steeper than 2H:1V, the restored embankment slope should remain stable. As the roadway grade will not be raised, settlement of the embankment is not a concern.

Embankment restoration after bridge replacement should be carried out in accordance with OPSS.PROV 206. In general, surface vegetation, topsoil, organic deposits, disturbed material or otherwise loose/soft soils should be stripped from the areas within the embankment footprints.

## 12 EXCAVATION AND DEWATERING

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

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the approach fill and native sand/gravel above the water table may be classified as Type 3 soils. The cohesionless soils below the water table are classified as Type 4 soil.

It is recommended that excavation for footing or pile cap construction be maintained above the water level in the creek, and that construction be carried out when creek water levels are normally lowest. Excavation within the cohesionless sand and gravel with cobbles and possible boulders below the water level may be problematic due to washing-in of materials, however installation of a sheet pile cofferdam with dewatering to enable construction in the dry is unlikely to be practical for the relatively shallow excavation depths anticipated.

Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor. The Contract Documents should contain a NSSP advising the Contractor of the high groundwater levels, cohesionless soils, cobbles and possible boulders at this site that may impact foundation construction. Suggested wording for an NSSP in this regards is provided in Appendix E.

If roadway protection is required for any stage of the bridge replacement, the roadway protection should be provided in accordance with OPSS.PROV 539 and designed for Performance Level 2. The design of any road protection system that may be required is the responsibility of the Contractor, and any shoring systems should be designed by a Professional Engineer experienced in such designs.

### **13 CONSTRUCTION CONCERNS**

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

- Foundation excavation and/or pile installation for the detour bridge must be carried out in a manner that does not undermine or disturb the existing crib abutments. Similarly, foundation construction for the new permanent bridge should not disturb the temporary bridge foundations.
- The base level of the existing timber cribs is unknown, and may extend below the water level. If spread footings are employed for the new bridge, crib removal should be carried out in a manner that minimizes disturbance of the subgrade.
- Piles should be located to avoid encountering the existing timber cribs during installation.
- Water levels in the creek may fluctuate during construction, and may be at a higher level than indicated on the GA drawing or measured during the site investigation.
- Frequent cobbles and large boulders may be encountered within the excavation depth and during H-pile installation. Removal of these boulders will require suitable excavating equipment, and may result in areas of over-excavation requiring backfill.

## 14 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Ms. Anna Piascik, P.Eng. and Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

### THURBER ENGINEERING LTD.

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P.K. Chatterji, P.Eng., Ph.D.  
Review Principal

**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


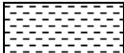



### ROCK WEATHERING CLASSIFICATION

<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



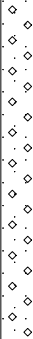

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, 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 <sub>L</sub> < 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 <sub>L</sub> < 30%).
		CI	Inorganic clays of medium plasticity, silty clays. (30% < W <sub>L</sub> < 50%).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS W <sub>L</sub> > 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 BH-01

1 OF 2

METRIC

W.P. 6905-10-00 LOCATION N 5 410 901.2 E 274 605.5 ORIGINATED BY CAM  
 HWY 7047 BOREHOLE TYPE Hollow Stem Augers/Casing/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2015.11.04 - 2015.11.04 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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
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	W <sub>p</sub> W    W <sub>L</sub>														
187.0	GROUND SURFACE															GR   SA   SI   CL							
0.0	<b>SAND</b> , trace gravel, trace silt Very Loose to Loose Brown Moist (FILL)		1	SS	7		186							○			5   90   5 (SI+CL)						
			2	SS	3									○									
185.5															○				Water level at 2.1m				
1.5	<b>SAND</b> and <b>GRAVEL</b> , occasional cobbles Compact to Very Dense Brown Moist  Frequent cobbles from 1.5m to 2.1m and from 3.0m to 3.6m		3	SS	100/ 0.250			185								○							
																		○					
																				○			
					4				SS	19	184										○		
					5				SS	17											○		
183.2									183									○			0   43   53   4		
3.8	Silty <b>SAND</b> to Sandy <b>SILT</b> , trace gravel, trace clay, occasional sand seams Loose to Compact Grey Wet		6	WH				182															
					7	SS	4		181								○						
					8	SS	22		180									○					
							179																
			9	SS	10		178								○								

Continued Next Page

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

20  
15 10 5  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-01

2 OF 2

METRIC

W.P. 6905-10-00 LOCATION N 5 410 901.2 E 274 605.5 ORIGINATED BY CAM  
 HWY 7047 BOREHOLE TYPE Hollow Stem Augers/Casing/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2015.11.04 - 2015.11.04 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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 (%)		
								○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE							PLASTIC LIMIT    NATURAL MOISTURE CONTENT    LIQUID LIMIT w <sub>p</sub> w    w <sub>L</sub>		
							20    40    60    80    100						20    40    60	GR   SA   SI   CL			
	Continued From Previous Page		10	SS	5		176							0   41   53   6			
			11	SS	10		175										
174.0																	
13.0	Silty <b>CLAY</b> , trace sand, occasional silty sand seams Soft Grey Wet		12	SS	3		174							0   0   53   47			
							173										
172.5																	
14.5	Silt <b>SAND</b> , with gravel Very Dense Grey Wet		13	SS	91/ 0.050		172						FI				
14.7	<b>GRANITE TO GRANODIORITE</b> with light grey quartz inclusions, slightly weathered to fresh, grey to reddish brown, occasional mechanical breaks throughout		1	RUN			171						2	RUN #1 TCR=100% SCR=100% RQD=100% UCS=183MPa			
			2	RUN			170						2	RUN #2 TCR=100% SCR=81% RQD=81% UCS=205MPa			
													0				
169.1													1				
17.9	END OF BOREHOLE AT 17.9m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.												2				

ONTMT4S 5220.GPJ 2015TEMPLATE(MTO).GDT 2/9/16

# RECORD OF BOREHOLE No BH-02

1 OF 2

METRIC

W.P. 6905-10-00 LOCATION N 5 410 895.1 E 274 603.9 ORIGINATED BY CAM  
HWY 7047 BOREHOLE TYPE Hollow Stem Augers/Casing/NQ Coring COMPILED BY AN  
DATUM Geodetic DATE 2015.11.05 - 2015.11.05 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  <b>γ</b>  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
186.8	GROUND SURFACE													
0.0	<b>SAND</b> , some gravel, frequent cobbles at top of layer Loose to Very Loose Brown Moist (FILL)		1	SS	7									
			2	SS	3									
185.3														
1.5	<b>SAND</b> and <b>GRAVEL</b> , trace silt, occasional cobbles Compact to Dense Brown Wet		3	SS	15									
			4	SS	38									
			5	SS	19									
183.1														
3.7	Silty <b>SAND</b> to Sandy <b>SILT</b> , trace clay, occasional seams of sand Compact to Loose Grey Wet		6	SS	11									
			7	SS	3									
	0.5m layer of sand, trace silt at 7.0m		8	SS	6									
			9	SS	7									

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 BH-02

2 OF 2

METRIC

W.P. 6905-10-00 LOCATION N 5 410 895.1 E 274 603.9 ORIGINATED BY CAM  
 HWY 7047 BOREHOLE TYPE Hollow Stem Augers/Casing/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2015.11.05 - 2015.11.05 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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
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		10	SS	1								GR   SA   SI   CL	
							176							
			11	SS	5		175							
174.0			12	SS	9		174							
12.8	Silty <b>CLAY</b> , trace sand, occasional silty sand seams Stiff Grey Wet		13	AS				8.5 +					0   0   54   46	
							173							
172.0							172					FI		
14.8	<b>GRANITE TO GRANODIORITE</b> with light grey quartz inclusions, slightly weathered to fresh, grey to reddish brown, occasional mechanical breaks throughout		1	RUN			171					4	RUN #1 TCR=68% SCR=68% RQD=52% UCS=175MPa	
												4		
												3		
												3		
			2	RUN			170					4	RUN #2 TCR=100% SCR=89% RQD=89% UCS=233MPa	
												2		
							169					2		
												1		
			3	RUN								3	RUN #3 TCR=97% SCR=90% RQD=90% UCS=205MPa	
							168					1		
167.4														
19.4	END OF BOREHOLE AT 19.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.													

ONTMT4S 5220.GPJ 2015TEMPLATE(MTO).GDT 2/9/16



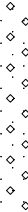

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No BH-03

1 OF 1

METRIC

W.P. 6905-10-00 LOCATION N 5 410 899.1 E 274 594.6 ORIGINATED BY CAM  
 HWY 7047 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2015.11.06 - 2015.11.06 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 (%)							
186.9	GROUND SURFACE							20	40	60	80	100					GR	SA	SI	CL					
0.0	<b>SAND</b> , trace to some gravel Compact Brown Moist (FILL)		1	SS	11		186										51	46	3 (SI+CL)						
186.1																									
0.8	<b>SAND</b> and <b>GRAVEL</b> , trace silt, occasional cobbles Compact Brown Wet		2	SS	20																				
			3	SS	30				185																
184.6																									
2.3	Silty <b>SAND</b> to Sandy <b>SILT</b> , trace gravel, trace clay, occasional sand seams Loose Grey Wet		4	SS	7	184											2	67	28	3					
			5	SS	8																				
								183																	
			6	SS	5																				
							182																		
			7	SS	4																				
181.7																									
5.2	END OF BOREHOLE AT 5.2m. WATER LEVEL AT 3.6m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.																								

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

# RECORD OF BOREHOLE No BH-04

1 OF 2

METRIC

W.P. 6905-10-00 LOCATION N 5 410 889.7 E 274 639.9 ORIGINATED BY CAM  
HWY 7047 BOREHOLE TYPE Hollow Stem Augers/Casing/NQ Coring COMPILED BY AN  
DATUM Geodetic DATE 2015.11.03 - 2015.11.03 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 (%)	
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE							
187.0	GROUND SURFACE															
0.0	<b>SURFACE TREATMENT</b> {30mm}															
	<b>SAND</b> , some gravel Compact Brown Moist (FILL)		1	SS	21											
			2	SS	17											
185.0			3	SS	19									39 47 14 (SI+CL)		
2.0	<b>SAND</b> and <b>GRAVEL</b> , some silt, frequent cobbles Compact to Dense Brown Wet		4	SS	34											
183.3																
3.7	Silty <b>SAND</b> to Sandy <b>SILT</b> , trace clay, occasional sand seams Loose to Compact Brown Wet		5	SS	4									0 53 44 3		
			6	SS	16											
			7	SS	29											
			8	SS	3									Probable hydraulic disturbance		
177.3																
9.7	Silty <b>CLAY</b> , trace sand, occasional silty sand seams															

Continued Next Page

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



# RECORD OF BOREHOLE No BH-04

2 OF 2

METRIC

W.P. 6905-10-00 LOCATION N 5 410 889.7 E 274 639.9 ORIGINATED BY CAM  
 HWY 7047 BOREHOLE TYPE Hollow Stem Augers/Casing/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2015.11.03 - 2015.11.03 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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)					
								○ UNCONFINED   + FIELD VANE	W P                      W                      W L						
							● QUICK TRIAXIAL   × LAB VANE								
	Continued From Previous Page		9	WH				20   40   60   80   100	20   40   60					GR   SA   SI   CL	
175.7	Silty <b>CLAY</b> , trace sand, occasional silty sand seams Soft Grey Wet						176							0   0   55   45	
11.3	<b>GRANITE TO GRANODIORITE</b> with light grey quartz inclusions, slightly weathered to fresh, grey to reddish brown, occasional mechanical breaks throughout		1	RUN			175						FI 1	RUN #1 TCR=100% SCR=100% RQD=86% UCS=200MPa	
			2	RUN			174						1	RUN #2 TCR=100% SCR=100% RQD=100% UCS=194MPa	
			3	RUN			173						0 0 0 2	RUN #3 TCR=100% SCR=80% RQD=80% UCS=178MPa	
172.1															
14.9	END OF BOREHOLE AT 14.9m. WATER LEVEL AT 2.1m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.														

# RECORD OF BOREHOLE No BH-05

1 OF 1

METRIC

W.P. 6905-10-00 LOCATION N 5 410 887.0 E 274 641.2 ORIGINATED BY CAM  
 HWY 7047 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2015.11.04 - 2015.11.04 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
187.0	GROUND SURFACE							<div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × LAB VANE</div></div>							
0.0	<b>SAND</b> , trace gravel, trace silt, occasional cobbles Compact Dark Brown Moist (FILL)		1	SS	18	▽	186								2944 (SI+CL)
			2	SS	10										
185.5															
1.5	<b>SAND</b> and <b>GRAVEL</b> , trace silt, frequent cobbles Dense Brown Moist		3	SS	45		185								
			4	SS	100/ 0.125										
184.0							184								
3.0	Silty <b>SAND</b> to sandy <b>SILT</b> , trace clay Very Loose to Loose Grey Wet		5	SS	2		183								047503
			6	SS	6										
			7	SS	5	182									
181.8															
5.2	END OF BOREHOLE AT 5.2m. WATER LEVEL AT 1.4m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO SURFACE.														

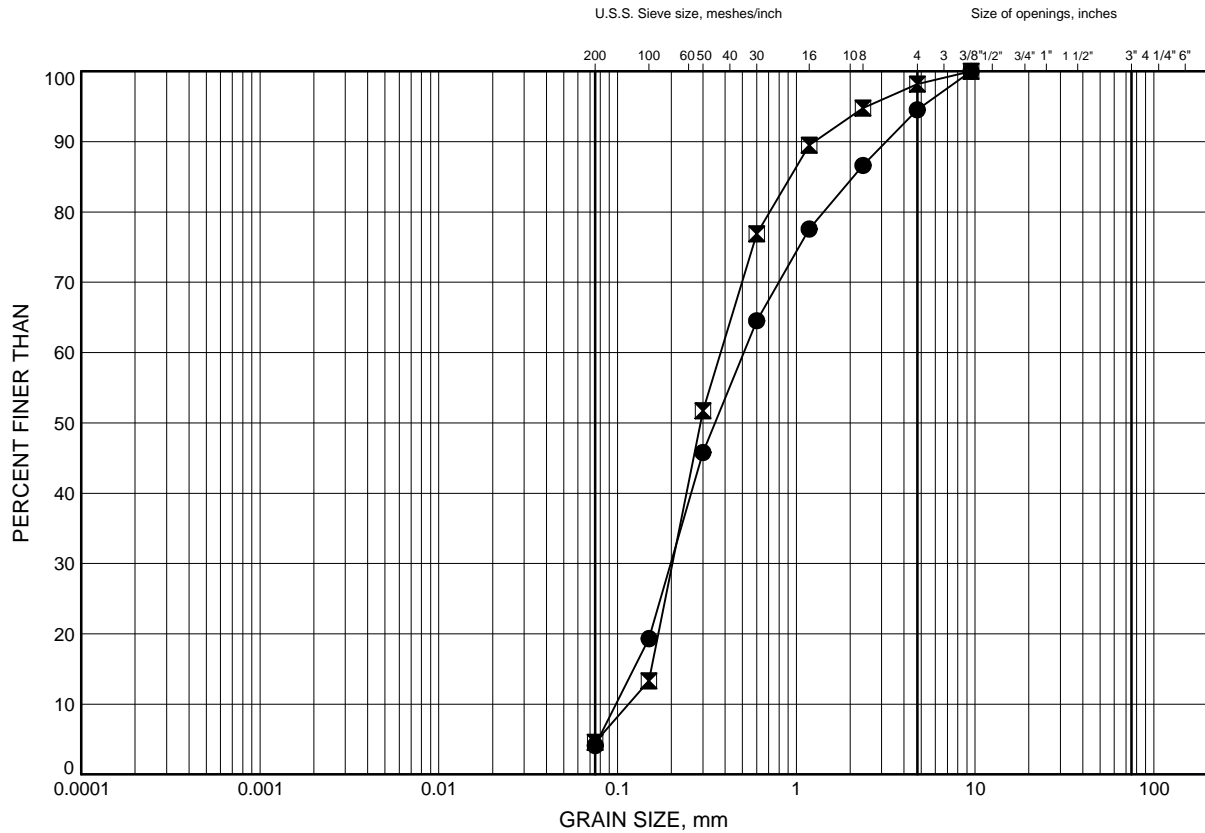
+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity  
 20  
15  
10  
(%) STRAIN AT FAILURE

**Appendix B**  
**Laboratory Test Results**

# Hewitson River Bridge Replacement GRAIN SIZE DISTRIBUTION

FIGURE B1

## SAND FILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH-01	0.50	186.50
⊠	BH-05	0.38	186.62

Date February 2016  
W.P. 6905-10-00



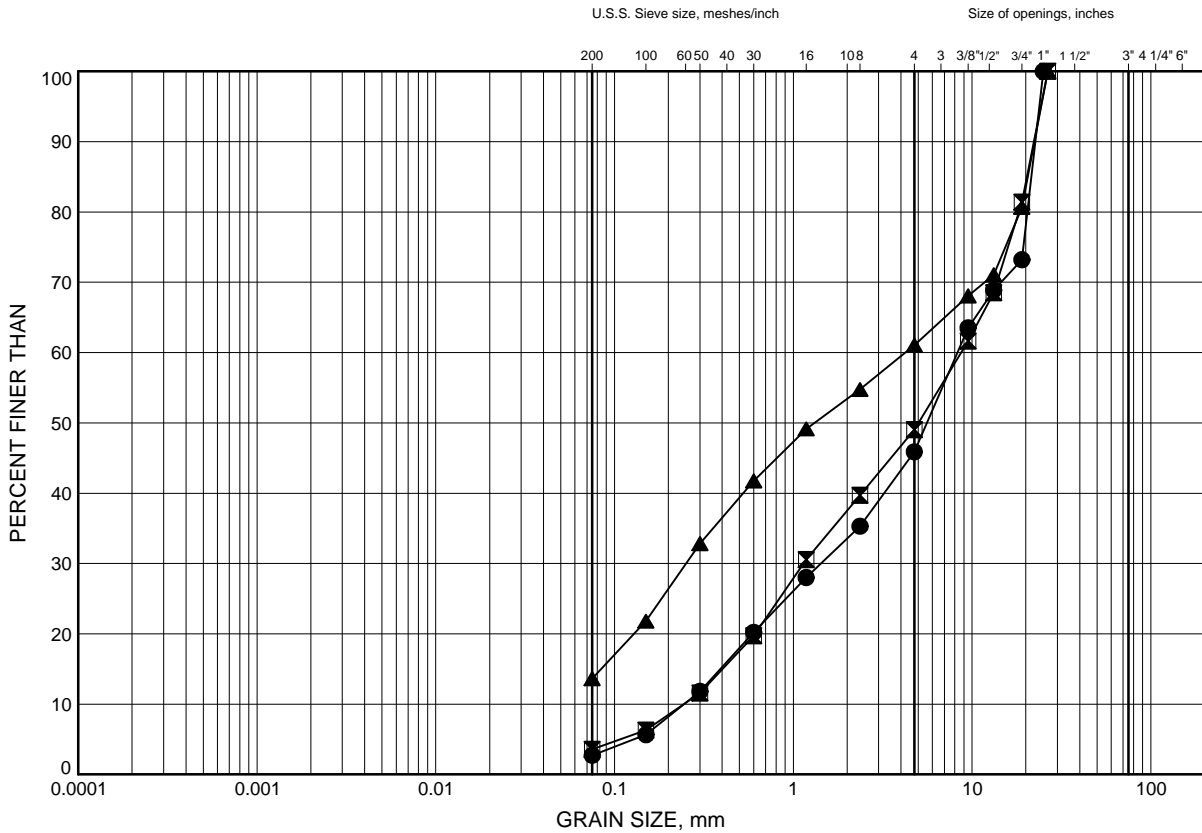
Prep'd AN  
Chkd. AMP

# Hewitson River Bridge Replacement

## GRAIN SIZE DISTRIBUTION

FIGURE B2

### SAND & GRAVEL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH-02	2.59	184.21
■	BH-03	1.07	185.83
▲	BH-04	1.75	185.25

Date February 2016  
W.P. 6905-10-00



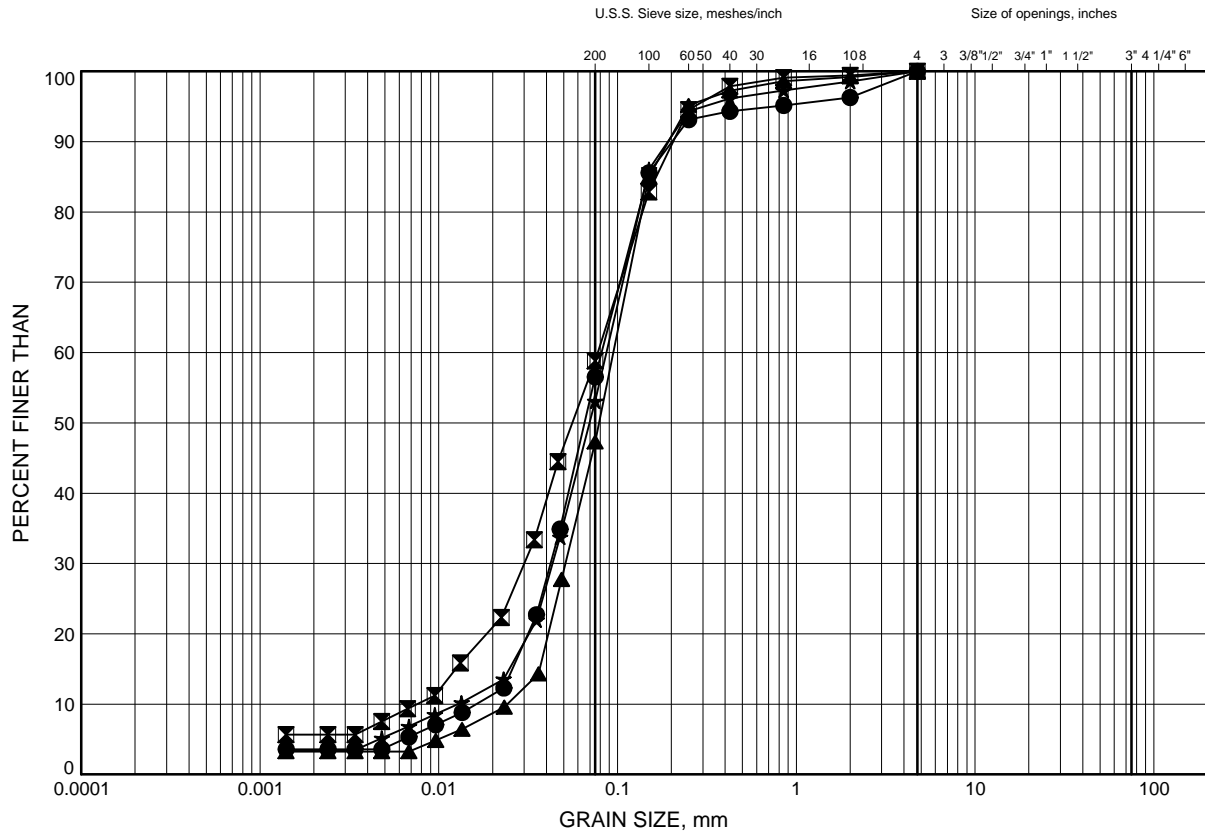
Prep'd AN  
Chkd. AMP

# Hewitson River Bridge Replacement

## GRAIN SIZE DISTRIBUTION

FIGURE B3

### SAND & SILT



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH-01	4.11	182.89
⊠	BH-01	10.21	176.79
▲	BH-04	4.11	182.89
★	BH-05	3.35	183.65

Date February 2016  
W.P. 6905-10-00



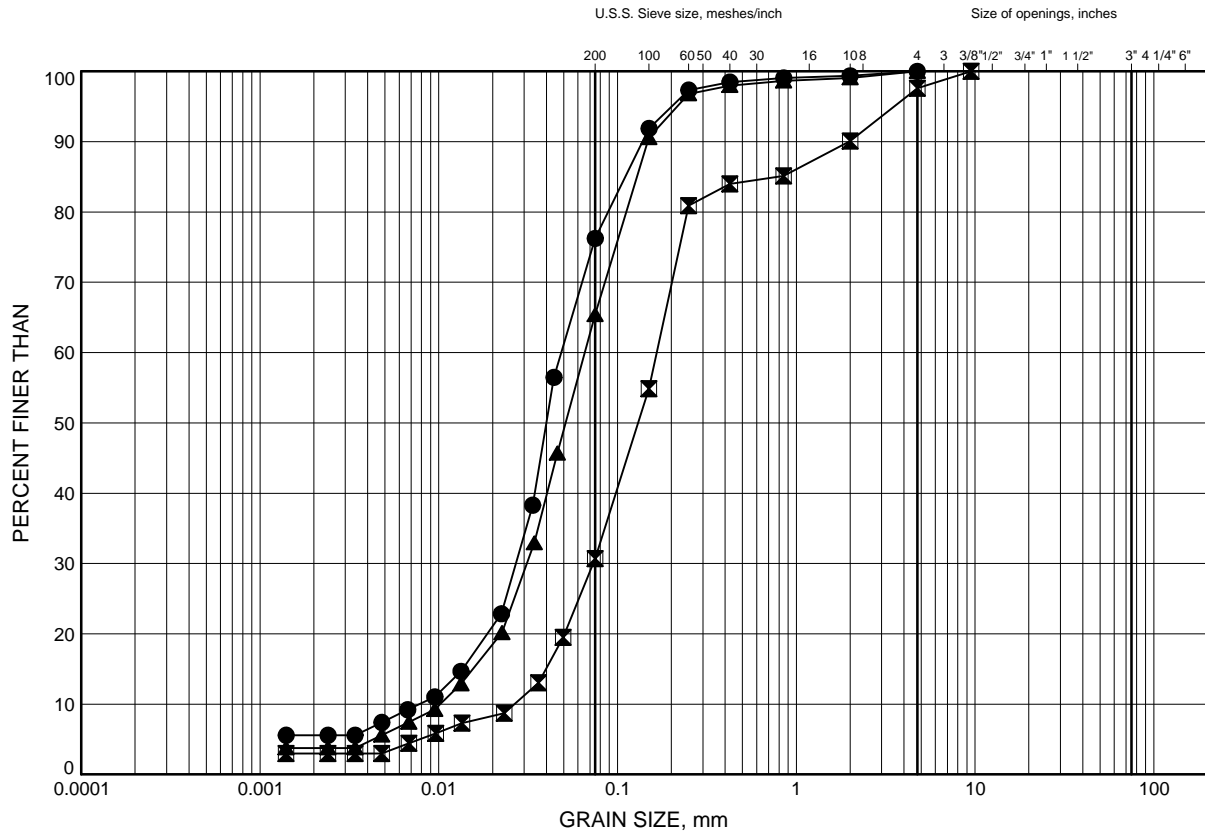
Prep'd AN  
Chkd. AMP

# Hewitson River Bridge Replacement

## GRAIN SIZE DISTRIBUTION

FIGURE B4

### Silty SAND to Sandy SILT



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH-02	10.21	176.59
⊠	BH-03	3.35	183.55
▲	BH-03	4.88	182.02

Date February 2016  
W.P. 6905-10-00

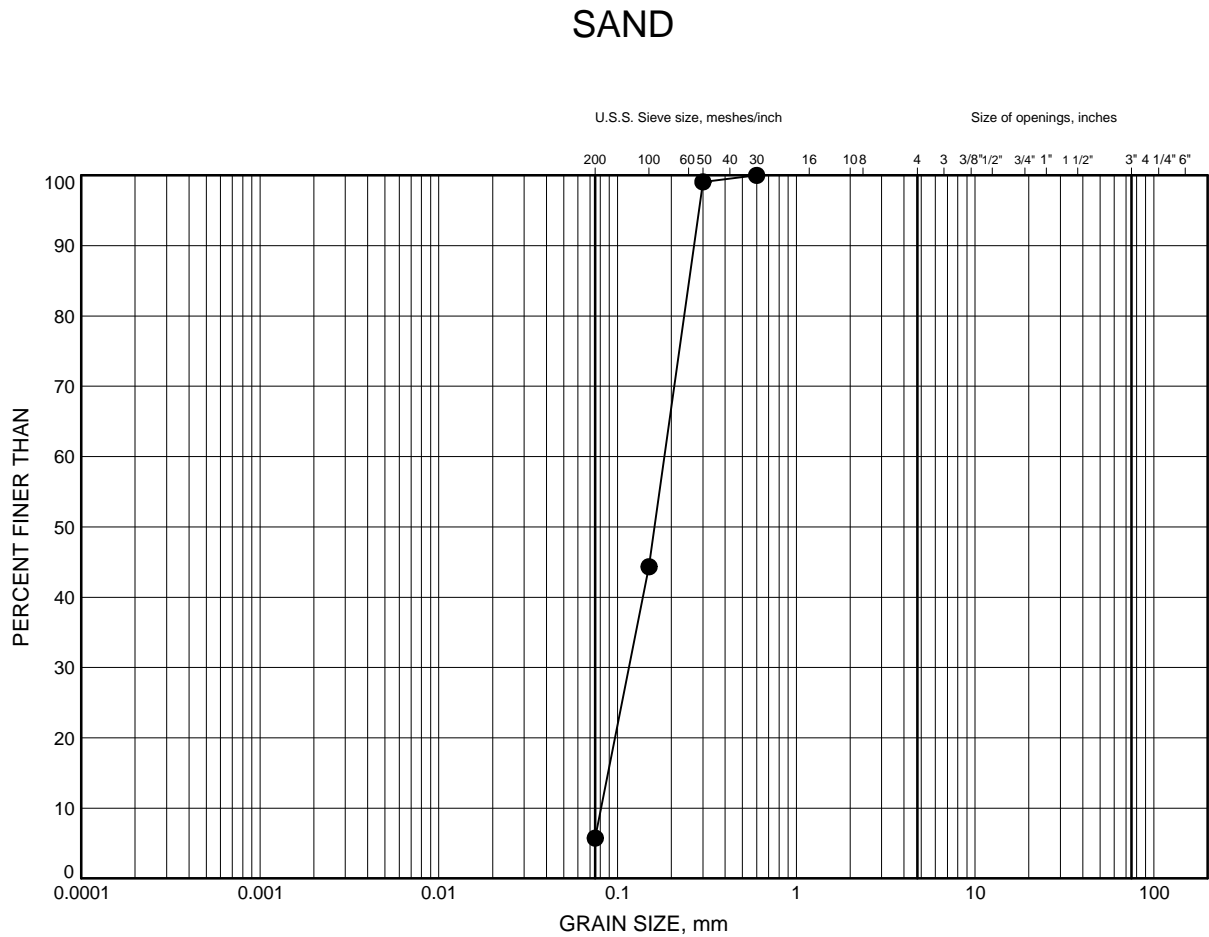


Prep'd AN  
Chkd. AMP

# Hewitson River Bridge Replacement

## GRAIN SIZE DISTRIBUTION

FIGURE B5



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH-02	7.16	179.64

Date February 2016  
W.P. 6905-10-00



Prep'd AN  
Chkd. AMP

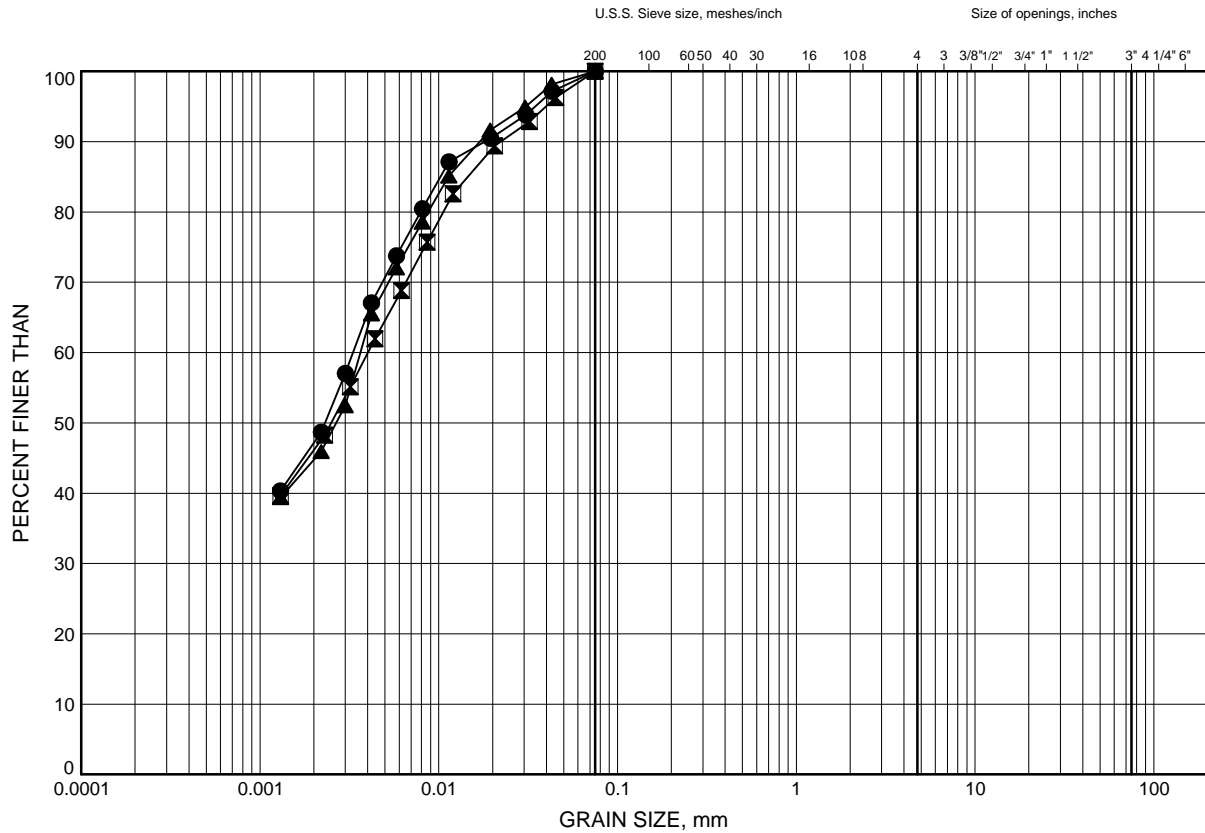


# Hewitson River Bridge Replacement

## GRAIN SIZE DISTRIBUTION

FIGURE B6

### Silty CLAY



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH-01	13.26	173.74
⊠	BH-02	13.26	173.54
▲	BH-04	10.21	176.79

Date February 2016  
W.P. 6905-10-00

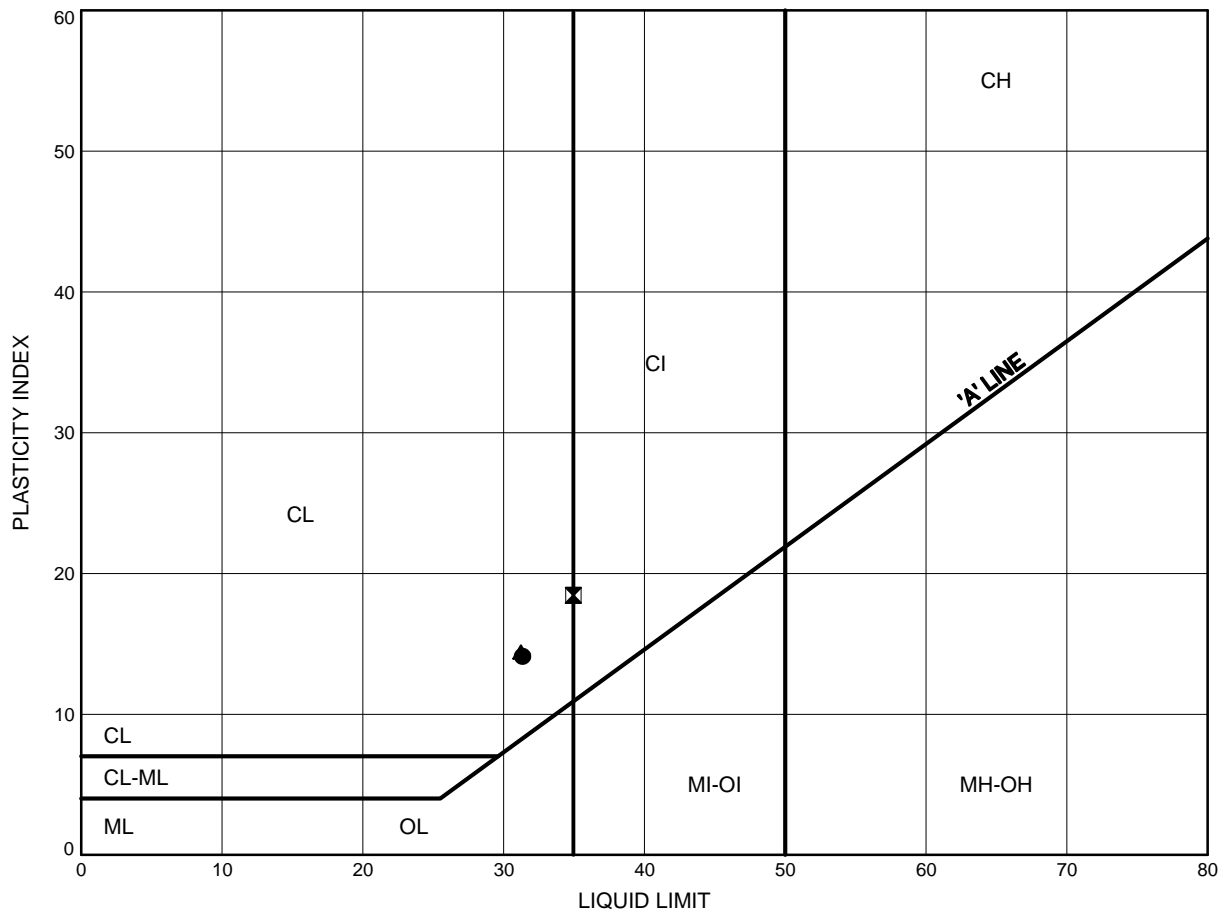


Prep'd AN  
Chkd. AMP

# Hewitson River Bridge Replacement ATTERBERG LIMITS TEST RESULTS

FIGURE B7

Silty CLAY



## LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BH-01	13.26	173.74
⊠	BH-02	13.26	173.54
▲	BH-04	10.21	176.79

Date February 2016  
W.P. 6905-10-00



Prep'd AN  
Chkd. AMP

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



**Photograph 1 - Hewitson Creek Bridge – looking north**



**Photograph 2 – Looking at the east abutment from west bank of creek**





**Photograph 3 – East abutment and view of timber crib**



**Photograph 4 – Looking at the west abutment**





**Photograph 5 – Looking at the west approach**



**Photograph 6 – Looking at the east approach**

**Appendix D**  
**Comparison of Foundation Alternatives**

### COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Native Soil or Engineered Fill	Driven H-Piles	Caissons
<p><b>Advantages</b></p> <ul style="list-style-type: none"> <li>i. Ease of construction.</li> <li>ii. Lower cost than deep foundations.</li> <li>iii. Reasonable geotechnical resistance is available in native soil/rock fill.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Excavation may extend below creek water level in cohesionless soils.</li> <li>ii. Potential environmental impact due to excavation adjacent to creek.</li> <li>iii. Excavation dewatering/cofferdams may not be practical considering shallow excavation and possible cobbles and boulders.</li> <li>iv. Potential disturbance to existing timber cribs or detour bridge foundations under service.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Piles will develop high geotechnical resistance on bedrock.</li> <li>ii. Installation of piles could continue in freezing weather.</li> <li>iii. Requires less excavation than footings.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit costs than spread footings.</li> <li>ii. Cobbles and boulders could be encountered in the fill and native deposits. Provision to remove the obstructions will be required.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High resistance is available for caissons founded on bedrock.</li> <li>ii. Construction of caissons could continue in freezing weather.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher cost than footings or H-piles.</li> <li>ii. Specialized installation methods such as temporary liners and drilling mud will be required to install caissons in cohesionless sands/silts below water level.</li> <li>iii. Difficulty in sealing liners at bedrock surface.</li> <li>iv. Cobbles and boulders may obstruct augering and liner installation.</li> <li>v. Difficulty in cleaning and inspecting bases.</li> </ul>
<b>FEASIBLE</b>	<b>RECOMMENDED</b>	<b>NOT RECOMMENDED</b>



## **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 501
- OPSS.PROV 539
- OPSS.PROV 804
- OPSS 902
- OPSS 903
- OPSS.PROV 1010

**2. Suggested Text for NSSP on “Construction of Driven H-piles”**

Installation of H-piles shall be in accordance with OPSS 903 and the following.

Cobbles and possibly boulders are present within the existing embankment fill and underlying sand and gravel on site. The cobbles and boulders may interfere with pile installation and some piles may meet refusal on boulders above the bedrock surface. The Contractor must be prepared to remove, dislodge or otherwise penetrate these obstructions to advance the piles to bedrock while meeting the specified deflection tolerances.

The H-piles must be provided with pile tip protectors to minimize tip damage.

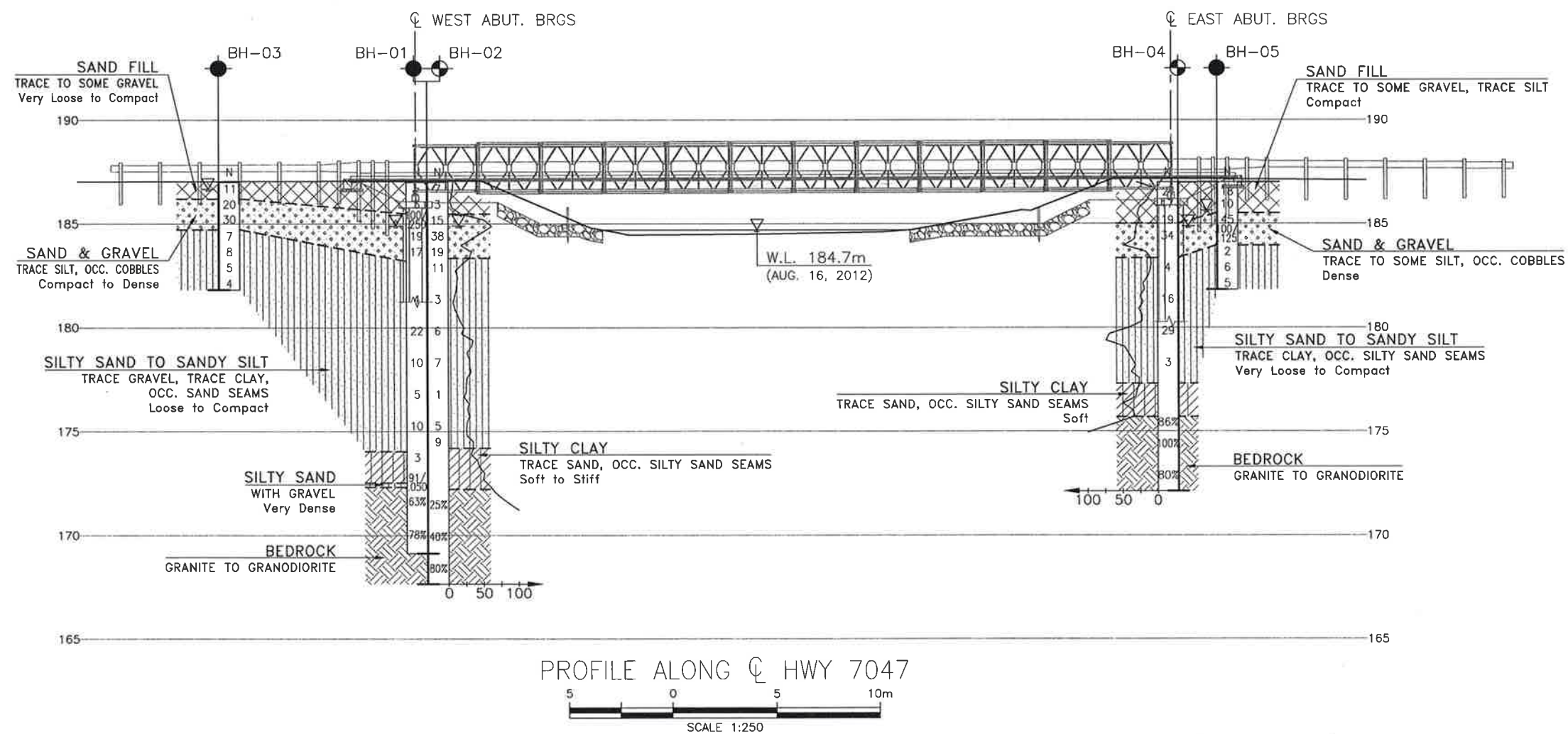
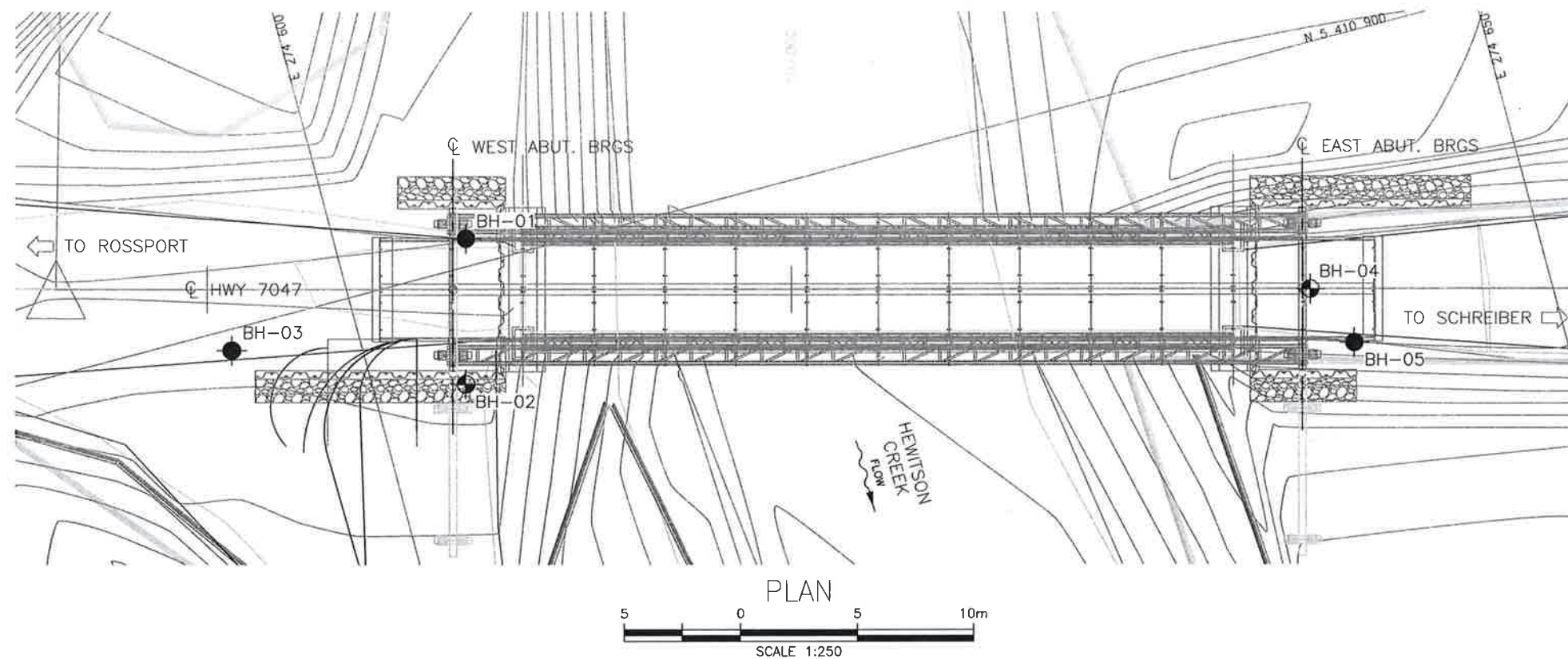
**3. Suggested Text for NSSP - Foundation Excavation**

The Contractor is advised that the existing sand fill and underlying native sand and gravel on the site contains cobbles and possibly boulders that may require handling during foundation excavation. Removal of large boulders will require appropriate excavating equipment, and may result in areas of over-excavation requiring backfill. The work should be carried out in a manner which minimizes disturbance to the excavation base as well as to foundations supporting the existing and temporary detour bridges.

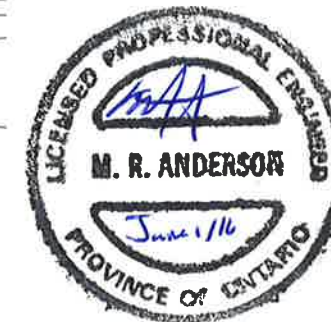
It is further noted that the water level in the creek may fluctuate. Instability of the excavation base and sidewalls should be anticipated if excavation extends below the water level within cohesionless soils. The presence of cobbles and boulders may impact installation of driven sheet piles for cofferdam or roadway protection systems if required.

Selection of the equipment and methodology to excavate and prepare the founding surface remains the responsibility of the Contractor, and should be based on his interpretation of the subsurface conditions presented in the Foundation Investigation Report as well as the surface conditions exposed at the site.

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



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



CONT No
GWP No 6905-10-00





HIGHWAY 7047  
HEWITSON CREEK BRIDGE  
REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA

## HATCH



## KEYPLAN

### LEGEND

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

NO	ELEVATION	NORTHING	EASTING
BH-01	187.0	5 410 901.2	274 605.5
BH-02	186.8	5 410 895.1	274 603.9
BH-03	186.9	5 410 899.1	274 594.6
BH-04	187.0	5 410 889.7	274 639.9
BH-05	187.0	5 410 887.0	274 641.2

-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.

GEOCRES No. 42D-41

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