



THURBER ENGINEERING LTD.



**FOUNDATION INVESTIGATION AND DESIGN REPORT
BLANCHE RIVER BRIDGE REPLACEMENT
HIGHWAY 569
NEW LISKEARD DISTRICT, ONTARIO
G.W.P. 5163-13-00, SITE NO. 47-038**

GEOCRES No. 31M-120

Report

to

WSP

Date: June 9, 2017
File: 19-5161-251

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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 Blanche River Bridge on Highway 569, in the District of New Liskeard, Ontario.

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

Thurber carried out the investigation as a sub-consultant to MMM Group Limited, under the Ministry of Transportation Ontario (MTO) Agreement Number 5014-E-0019.

2. SITE DESCRIPTION

The Blanche River Bridge is located on Highway 569, approximately 10.9 km east of Highway 11. Originally built in 1923 and rehabilitated in 1992, the existing bridge is a three-span structure with a total length of approximately 92.5 m and a deck width of 5.4 m. The middle span is supported on a 64 m long arched steel truss.

Blanche River flows easterly to southeasterly at the bridge site. The land surrounding the site has a generally flat to gently undulating terrain and is typically used for agricultural purposes. Residential houses exist sporadically to the north and south of the site.

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

The site lies within the physiographical area of Cobalt Embayment. Surficial geology at the site is featured by glacio-lacustrine silts and clays and swamp deposits consisting of peat, muck and marl. The bedrock consists typically of Ordovician sedimentary rocks of Liskeard Group.

3. INVESTIGATION PROCEDURES

The field investigation and testing was carried out between November 18 and 27, 2015. A total of four boreholes, identified as BR-01 to BR-04, were drilled in conjunction with Standard Penetration Testing (SPT) to depths of 34.1 to 52.4 m (Elev. 147.0 to 127.2) from the ground surface or river bed. A Dynamic Cone Penetration Test (DCPT) was carried out below the drilled portion of BR-02 to a depth of 45.4 m below the water surface (Elev. 133.3). BR-01 and BR-04 were drilled on the land near the proposed south and north abutments, respectively. BR-02 and BR-03 were drilled from a barge in the river near the proposed south and north piers, respectively.

The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing included in Appendix D. Completion details of the piezometer and boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Element	Borehole	Piezometer Installation		Completion Details
		Screen Depth / Elevation	Stratum	
South Abutment	BR-01	10.7 – 12.2 / 175.1 – 173.6	Silty Clay	Grout from 52.4 m to 15.2 m, bentonite holeplug to 12.2 m, sand to 10.4 m, and bentonite holeplug to surface.
South Pier	BR-02	None Installed		Grout from 37.5 m to riverbed.
North Pier	BR-03	None Installed		Grout from 51.8 m to riverbed.
North Abutment	BR-04	None Installed		Grout from 38.7 m to 2.1 m and bentonite holeplug to surface.

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

A track-mounted D-25 drill rig was used to advance all four boreholes in the overburden using NW casing/wash boring techniques. Boreholes BR-02 and BR-03 were advanced from the river water surface on a barge. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Field vane shear testing (VST)

was carried out at selected depths to measure the undrained shear strength of the silty clay. A total of six (6) undisturbed silty clay samples were collected using Shelby Tube samplers.

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 Boreholes BR-01 and BR-04 were observed throughout the drilling operations. Groundwater conditions observed after completion of drilling were not representative of site conditions as water was used to assist drilling during wash-boring operations. A standpipe piezometer was installed in Borehole BR-01 to monitor the groundwater level after drilling. All boreholes were backfilled in general accordance with MOE Regulation 903.

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 Atterberg Limits tests. One undisturbed Shelby Tube sample was subjected to standard 24-hour incremental loading (IL) consolidation test as per ASTM D2435-04 Test Method A. 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.

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. It must be recognized that soil conditions may vary between and beyond the borehole locations.

The soil stratigraphy encountered below the existing embankment fill generally comprises a thick deposit of grey varved silty clay underlain by clayey silt to silt. More detailed descriptions of the individual strata are presented below.

5.1 Topsoil

Topsoil was encountered in Boreholes BR-01 and BR-04 drilled near the existing abutments. The topsoil thickness was approximately 50 mm in both boreholes. Topsoil thickness may vary beyond

borehole locations and in other areas of the site. This limited data should not be used for estimating topsoil quantity.

5.2 Embankment Fill

Sand to gravelly sand fill was encountered below the topsoil in Boreholes BR-01 and BR-04. Silty clay fill was encountered below the gravelly sand fill in BR-04. The fill thickness ranged between 1.4 and 2.4 m with the lower boundaries at Elev. 184.4 and 183.3, respectively.

SPT-N values recorded in the cohesionless fill ranged from 4 to 10 blows per 0.3 m of penetration, indicating a loose relative density. One SPT-N value obtained in the silty clay fill was 4 blows per 0.3 m of penetration indicating a firm consistency. Moisture contents ranged from 6 to 15%.

The results of two gradation analyses performed on the sand to gravelly sand fill samples are provided on the Record of Borehole sheets in Appendix A and plotted in Figure B1 of Appendix B. The results are summarized below.

Particle Size	Percentage (%)
Gravel	1 to 33
Sand	59 to 95
Silt & Clay	4 to 8

5.3 Silty Clay

A thick deposit of grey varved silty clay was encountered below the fill in Boreholes BR-01 and BR-04 and from the riverbed in Boreholes BR-02 and BR-03. The water depth during drilling was about 3.1 m and 4.6 m at BR-02 and BR-03, respectively. BR-02 (drilled portion) and BR-04 were terminated within the silty clay layer at a depth of 34.1 m below the riverbed and 38.7 m below the ground surface or Elev. 141.5 and 147.0, respectively. A Dynamic Cone Penetration Test (DCPT) was carried out in BR-02 and terminated at a depth of 8.2 m below the base of the drilled portion or Elev. 133.3. The thickness of the silty clay layer fully penetrated in BR-01 and BR-03 ranged between 47.1 and 40.8 m with the lower boundary at Elev. 137.3 and 133.3, respectively.

SPT-N values recorded in the silty clay ranged from 0 to 7 blows per 0.3 m of penetration. Field vane shear tests measured undrained shear strengths ranging typically from 35 to 60 kPa. The field test data indicates that the silty clay has a very soft to stiff consistency.

The results of the Atterberg Limits tests conducted on samples of the silty clay are provided on the Record of Borehole sheets in Appendix A and illustrated in Figures B7 to B10 of Appendix B. The results of the Atterberg Limits testing indicated that the silty clay has plastic limits ranging

from 17 to 26% and liquid limits ranging from 26 to 59%, yielding plasticity indices ranging from 9 to 37%. Moisture contents of the silty clay ranged from 24 to 62%.

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 illustrated in Figures B2 to B5 of Appendix B. The results for the typical deposit and one sample immediately beneath the embankment fill are summarized below.

Particle Size	Percentage (%)	
	Typical	Immediately Below Fill
Gravel	0	0
Sand	0	25
Silt	20 to 64	50
Clay	36 to 80	25

One incremental loading (IL) consolidation test was carried out on an undisturbed silty clay sample collected using thin-wall Shelby Tube sampler. The test results are included in the Appendix B and summarized below.

BH No.	Sample Depth (m)	Moisture Content w_n (%)	Initial Void Ratio e_o	In-situ Vertical Stress σ_{vo}' (kPa)	Pre-consolidation Pressure P'_c (kPa)	Compression Index C_c	Recompression Index C_r
BR-1	4.6 – 5.2	52.4	1.391	50	135	0.57	0.039

5.4 Clayey Silt to Silt

A layer of grey clayey silt to silt was encountered below the silty clay in BR-01 and BR-03. The boreholes were terminated within the clayey silt to silt layer at a depth of 52.4 m below the ground surface or Elev. 133.4 and 46.9 m below the riverbed or Elev. 127.2, respectively.

SPT-N values recorded in the layer ranged from 4 to 8 blows per 0.3 m of penetration, indicating a firm to stiff consistency or loose relative density. Natural moisture contents of the deposit ranged from 25% to 33%.

The results of two gradation analyses performed on the clayey silt to silt samples are provided on the Record of Borehole sheets in Appendix A and plotted in Figure B6 of Appendix B. The results are summarized below.

Particle Size	Percentage (%)
Gravel	0
Sand	0
Silt	63 to 82
Clay	18 to 37

The results of one Atterberg Limits test conducted on a clayey silt sample is provided on the Record of Borehole sheets in Appendix A and illustrated in Figure B11 of Appendix B. The test results indicated that the soil has a plastic limit of 18% and a liquid limit of 26%, yielding a plasticity index of 8%.

5.5 Groundwater Conditions

Where possible, water levels were monitored in the open boreholes during drilling operations. Wash boring was used to advance all boreholes and therefore water levels recorded during or upon completion of drilling may not reflect natural groundwater conditions. A standpipe piezometer was installed in Borehole BR-01 after completion of drilling. The water level measured in the piezometer and in open boreholes are presented in Table 5.1.

Table 5.1 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
BR-01	Nov. 26, 2015	2.0	183.8	In piezometer
	Nov. 27, 2015	1.4	184.4	
	Nov. 28, 2015	1.4	184.4	
BR-02	Nov. 18, 2015	-	178.7	Observed River Level
BR-03	Nov. 20, 2015	-	178.7	Observed River Level
BR-04	Nov. 27, 2015	2.1	183.6	In Open Borehole

The river level in the GA drawing was reported at Elev. 179.43 on May 22, 2015. The water levels recorded in the boreholes are short-term readings and seasonal fluctuations of the groundwater and river level are to be expected. The GA drawing indicates a 100-year flood level at Elev. 183.3.

6. MISCELLANEOUS

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

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

Walker Drilling Limited of Utopia, Ontario supplied a track-mounted D-25 drill rig and a barge, and conducted the drilling, sampling and in-situ testing operations for the boreholes. The drilling operations were supervised by Mr. George Azzopardi of Thurber.

Overall supervision of the field program, interpretation of the data, and preparation of the report were carried out by Ms. Deanna Pizycki, E.I.T., and Mr. Keli Shi, P. Eng.

The report was reviewed by Mr. Alastair Gorman, P. Eng., Dr. P.K. Chatterji, P. Eng., a Designated Principal Contact for MTO Foundations Projects.

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File No.: 19-5161-251

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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 provides geotechnical recommendations to assist the design team in selecting and designing a suitable foundation system for the proposed Blanche River Bridge Replacement on Highway 569 in the District of New Liskeard, Ontario.

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

The existing Highway 569 roadway grade on the bridge is at approximately Elev. 185.8. The proposed replacement alignment will be located approximately 11 m to the east of the existing bridge. The replacement structure will be a three-span (36 m-60 m-36 m) bridge with a total length of 132 m and a 10 m wide deck to accommodate two lanes of traffic. The new structure will require up to 2.5 m and 3.2 m of new fill to be placed at the new south and north abutments, respectively.

8. STRUCTURE FOUNDATIONS

In general, the soil stratigraphy below the existing fill consists of a deep deposit of firm to stiff silty clay grading into clayey silt to silt at depth. The thickness of the silty clay was 47.1 m near the

south abutment and 40.8 m near the north pier (base Elev. 137.3 to 133.3). The inferred thickness of the silty clay at the south pier based on the DCPT results was on the order of 39 m. Borehole BR-04 near the north abutment was terminated within the silty clay at 38.7 m depth or 36.3 m below the top of silty clay layer. Bedrock was not encountered within the depth of drilling.

The river level shown in the GA was at Elev. 179.43 on May 22, 2015. A 100-year flood level at Elev. 183.3 was also indicated. Groundwater level were measured at Elev. 184.4 at the south abutment in a standpipe piezometer two days following the installation in the current investigation. The river water level was observed at Elev. 178.7 in November 2015 during the current investigation.

In view of the presence of deep compressible silty clay deposit at the site, spread footings founded on native soil or engineered fill are not considered a feasible foundation option due to the low bearing resistances available and the potential for large footing settlement over an extended period under the foundation loads. Use of driven steel H-piles to support the abutments and piers are deemed the most suitable option at this site.

Recommendations for design of the recommended foundation alternative are presented in the following sections together with the corresponding geotechnical design parameters.

8.1 Driven H-Pile Foundations

8.1.1 Axial Resistance

Use of driven steel H-piles to support the abutments and piers is deemed suitable at the site.

The estimated geotechnical resistances at the factored ULS and the SLS reaction are provided in Table 8.1 for varying pile lengths for steel HP 310x110 and HP 360x132 piles.

Table 8.1 – Axial Geotechnical Resistances for Piles

Location (Borehole)	Top of Pile Elevation (m)	Pile Length (m)	HP 310x110		HP 360x132	
			ULS _r (kN)	SLS (kN)	ULS _r (kN)	SLS (kN)
South Abutment (BR-01)	183.3*	20	300	250	360	300
		30	450	375	560	460
		40	625	520	750	625
Pier 1 (BR-02)	175.4*	20	300	250	360	300
		30	450	375	560	460
		40	600	500	720	600
Pier 2 (BR-03)	177.4*	20	300	250	360	300
		30	450	375	560	460
		40	600	500	720	600

Location (Borehole)	Top of Pile Elevation (m)	Pile Length (m)	HP 310x110		HP 360x132	
			ULS _r (kN)	SLS (kN)	ULS _r (kN)	SLS (kN)
North Abutment (BR-04)	183.0*	20	300	250	360	300
		30	450	375	560	460
		40	625	520	750	625

Note: (*) Top of pile elevation based on the information provided in the GA drawing.

The axial pile resistances shown above will primarily rely on the resistances along the pile shaft embedded within the silty clay. Therefore, pile tip protection or pile shoe should not be used.

8.1.2 Pile Installation

Pile installation must be in accordance with OPSS 903.

Piles must not be installed until after 90% degree of consolidation has been achieved in the foundation clay. Details of the foundation consolidation settlement are discussed in Section 13.

A waiting time of 7 days between pile installation and retapping is recommended to allow for pile setup or gain in pile capacity with time. Suggested wording for NSSP is included in Appendix E.

Pile driving may be controlled in accordance with Standard Drawing SS103-11 (Hiley Formula) and an ultimate pile resistance should be specified by the designer. The Hiley formula need not be used until the piles are within 2.0 m of the design tip elevation. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS103-11 using an ultimate resistance of 'R' kN per pile". 'R' must have a minimum value of twice the design load at ULS, but must not exceed twice the factored ULS resistance of the selected pile length at each foundation element, e.g., 'R' must not exceed 1,250 kN for 40 m long HP 310x110 at the South Abutment.

Consideration may also be given to high-strain dynamic testing using pile driving analyzer for pile retapping and assessing ultimate pile capacity. An example NSSP is included in Appendix E.

8.1.3 Lateral Pile Resistance

The geotechnical lateral resistance acting on a pile in cohesive soils may be calculated using 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
(0.310 m for HP 310x110 and 0.373 m for HP 360x132)

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

k_s = $n_h z / D$ (kN/m³)

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

Where z = depth of embedment of pile (m)

D = pile width or diameter
(0.310 m for HP 310x110 and 0.373 m for HP 360x132)

n_h = coefficient related to soil relative density (kN/m³)

γ' = effective unit weight (kN/m³)

K_p = passive earth pressure coefficient

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 must not exceed the ultimate lateral resistance.

Table 8.2 – Soil Parameters for Lateral Pile Resistance

Soil Unit	Elevation (m)		γ' (kN/m³)	n_h (kN/m³)	K_p	S_u (kPa)
	Top	Bottom				
South Abutment (BR-01)						
Silty Clay	183.3	151.0	8.0 (*)	-	-	40
Silty Clay	151.0	137.3	8.0 (*)	-	-	50
Silt to Clayey Silt	137.3	133.4	9.0 (*)	500	2.5	-
Pier 1 (BR-02)						
Silty Clay	175.4	173.0	7.5 (*)	-	-	25
Silty Clay	173.0	151.0	8.0 (*)	-	-	40
Silty Clay	151.0	141.5	8.0 (*)	-	-	50
Pier 2 (BR-03)						
Silty Clay	177.4	145.0	8.0 (*)	-	-	40
Silty Clay	145.0	133.3	8.0 (*)	-	-	45
Silt to Clayey Silt	133.3	127.2	9.0 (*)	500	2.5	-
North Abutment (BR-04)						
Silty Clay	183.0	177.0	9.0 (*)	-	-	50
Silty Clay	177.0	147.0	8.0 (*)	-	-	35

Note: (*) Submerged unit weight

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

The modulus of subgrade reaction and ultimate lateral resistance may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 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 perpendicular to direction of loading	4D	1.0
	1D	0.5
Pile group oriented parallel to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

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

8.2 Downdrag

Up to 2.5 m and 3.2 m of new fill will be placed to construct the approach embankments to the proposed south and north abutments, respectively, to accommodate the replacement bridge. The new fill placement at the abutments will result in development of downdrag forces along the length of abutment piles associated with consolidation of the silty clay foundation under the weight of the new fill.

For design purposes, an unfactored downdrag load of 750 kN per pile is recommended to evaluate the impact of downdrag on the abutment piles.

This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C6.11.4.10 to obtain a factored downdrag load. In accordance with Section 6.11.4.10 of the CHBDC and Clause C6.11.4.10 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects

of downdrag. The factored dead and downdrag load should not exceed the factored structural resistance of a pile at the neutral plane.

8.3 Integral Abutment Considerations

The soil conditions at this site are suitable for the design of an integral abutment structure. The pile flexibility requirements of this design must be checked by the structural designer, taking account of the lateral resistance of the soil surrounding the piles, assuming no CSP is used.

If this system is too stiff, the piles should be placed in CSPs as described in the integral abutment guidelines. The order of construction should be:

- 1) Auger a hole with suitable diameter to the depth required;
- 2) Install the corrugated steel pipe (CSP);
- 3) Drive the pile;
- 4) Fill the CSP with sand.

8.4 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, driven steel H-piles are considered a suitable foundation option at this site.

8.5 Frost Cover

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

8.6 Impact on Existing Foundations

Placement of the new approach fill will potentially induce settlement of the existing abutment foundations. Piles for the new piers will be driven adjacent to the existing bridge piers for construction of the replacement bridge. It is recommended the structural designer select appropriate settlement monitoring points on the existing structure and specify a monitoring program for the duration of pile driving. Suggested wording for an NSSP for monitoring of the existing structure during construction has been included in Appendix E.

9. EXCAVATION AND DEWATERING

The river level was reported at Elev. 179.43 in May 2015. Groundwater levels measured in the standpipe piezometer in BR-01 ranged from Elev. 183.8 and 184.5 between November 26 and 28, 2015. Excavation for abutment construction will extend below the groundwater table.

Dewatering scheme consisting of sump and pump with surface run-off diversion is anticipated to be appropriate at the abutments.

The preliminary GA indicates that excavation for construction of the pile caps at pier locations will extend below the riverbed. Where excavations extend below the groundwater table or river level, the Contractor must implement effective groundwater control measures to prevent disturbance of the base and sides of the excavation and to permit construction in the dry. The Contractor must also take effective measures to prevent inundation by the river or surface run-off. At the proposed pier locations, use of watertight sheet pile cofferdam installed into silty clay and pumping from within the excavation is expected to be required. The depths of penetration of the sheet piles into the silty clay must be adequately designed to effectively cut off the seepage flow and retain the soils above the base of excavation. The construction sequence should be as follows:

- 1) Install sheet pile cofferdam at the pier locations;
- 2) Excavate inside the cofferdam to the underside of tremie plug;
- 3) Drive the piles within the cofferdam;
- 4) Pour tremie plug to the base of excavation;
- 5) Unwater inside the cofferdam;
- 6) Construct the pile cap.

The required thickness of the tremie plug is approximately 1 m, but this must be confirmed by the designer of the cofferdam and unwatering system.

Design and implementation of the dewatering procedures remain the responsibility of the Contractor.

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

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

The selection of the method of excavation and its temporary support is the responsibility of the Contractor and must be based on his equipment, experience and interpretation of the site conditions. It is anticipated that a hydraulic excavator will be suitable.

10. ROADWAY PROTECTION

Client: WSP

File No.: 19-5161-251

E file: H:\19\5161\251 Foundations - Temiscaming Cochrane Rehab Replacement 5014-E-0019\Reports & Memos\Blanche River Bridge\Final FIDR\Blanche River Bridge_FIDR.docx

Date: June 9, 2017

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

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

γ	=	19 kN/m ³	(bulk unit weight of fill)
	=	18 kN/m ³	(bulk unit weight of native silty clay)
γ'	=	9 kN/m ³	(submerged unit weight of fill)
	=	8 kN/m ³	(submerged unit weight of native silty clay)
K_a	=	0.33	(active earth pressure coefficient of sand fill)
	=	0.38	(active earth pressure coefficient of silty clay fill)
	=	0.38	(active earth pressure coefficient of native silty clay)
K_p	=	3.0	(passive earth pressure coefficient of sand fill)
	=	2.7	(passive earth pressure coefficient of silty clay fill)
	=	2.7	(passive earth pressure coefficient of native silty clay)
H_w	=	179.4 m	(design river level at pier locations)
	=	184.0 m	(design groundwater table at abutment locations)

The actual lateral earth pressure distribution acting on the shoring system is a function of construction sequence and the relative rigidity of the shoring wall and these factors must be accounted for when designing the shoring system.

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

11. LATERAL EARTH PRESSURES ON ABUTMENTS

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

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

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

K = coefficient of lateral earth pressure (see Table 11.1)

γ = unit weight of retained soil (see Table 11.1)

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

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 11.1.

Table 11.1 – Coefficients of Lateral Earth Pressure (K)

Loading Condition	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Type III $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active K_A (Unrestrained Wall)	0.27	0.39*	0.31	0.47*
At-rest K_0 (Restrained Wall)	0.43	-	0.47	-
Passive K_P	3.7	-	3.3	-

* For wing walls

The active and passive earth pressure coefficients in Table 11.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code (CHBDC).

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

12. SEISMIC CONSIDERATIONS

According to Clause 4.4.4 of the CHBDC, an earthquake with a 2475-year return period or 2% probability of exceedance in 50 years should be used for seismic design. The peak ground acceleration (PGA) associated with the design earthquake is 0.116 g for Site Class C.

Based on the encountered soil conditions, this site is assessed to be Site Class E for seismic site response according to Table 4.1 of the CHBDC. The above PGA value should be modified by a site coefficient of 1.72 based on Table 4.8 of the CHBDC.

For the design of retaining walls under seismic loading, the coefficients of horizontal earth pressure in Table 12.1 may be used:

Table 12.1 – Earth Pressure Coefficient for Earthquake Loading (K_E)

Loading Condition	Granular A or Granular B Type II $f' = 35^\circ$; $g = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I or Type III $f' = 32^\circ$; $g = 21.2 \text{ kN/m}^3$
Active (K_{AE}) *	0.36	0.40
At-rest (K_{OE}) **	0.69	0.74
Passive (K_{PE})	3.4	3.0

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

** After Woods (1973).

Given the presence of deep silty clay deposit and the clay plasticity, the potential for liquefaction at this site is assessed to be low.

13. APPROACH EMBANKMENTS

The preliminary GA drawing indicates that fill placement will be required at new bridge approaches. The maximum heights of fill placement above the original ground surface at the abutments are approximately 2.5 m at the south abutment and 3.2 m at the north abutment. The new fill placement will cause ground settlement associated with consolidation of the foundation clay.

13.1 Settlement Analysis

Settlement analysis was carried out based on Terzaghi's 1D consolidation theory in conjunction with the soil compressibility parameters obtained from the laboratory consolidation test. The normalized recompression index ($C_{r\varepsilon}$) and normal compression index ($C_{c\varepsilon}$) used in the analysis ranged from 0.015 to 0.025 and 0.15 to 0.25, respectively. The effects of the use of wick drains to expedite the foundation consolidation were examined for 1.5 m and 2.0 m wick spacing installed in a triangular pattern. One way drainage is assumed for wick drain design. The diameter ratio of smear zone to wick drain and the permeability ratio of undisturbed clay to the disturbed clay within the smear zone are both assumed to be 5:1.

The estimated consolidation settlements under the approach fill and the corresponding time to 90% degree of consolidation are summarized in the table below. The foundation settlement behaviours for the three analysis cases at each abutment are illustrated in the settlement vs. time plots included in Appendix F. Post-construction settlements due to secondary compression of the foundation clay were estimated to be in the order of 10 to 15 mm in 20 years.

Approach Embankment	Max. New Fill Height	Estimated Primary Consolidation Settlement	Wick Drain Spacing	Estimated Time to 90% Degree of Consolidation
South Abutment	2.5 m	110 mm	No Wick	152 months
			2.0 m	10 months
			1.5 m	6 months
North Abutment	3.2 m	130 mm	No Wick	152 months
			2.0 m	10 months
			1.5 m	6 months

The settlement analysis assumes that the wick drains will be installed to 3 m above the non-plastic silt deposit or approximately Elev. 140 m, and laterally to the limits of new fill placement on both west and east sides. No wick drain is required within the cut area. A minimum 300 mm thick granular drainage blanket shall be placed on the existing ground to 1 m beyond the wick drain extent prior to wick drain installation in accordance with OPSS.PROV 220. Based on the latest design sections available, the wick drains shall be installed to the following extents to meet the MTO Post-Construction Settlement Criteria behind bridge abutments:

- South Approach: From Sta. 16+956 or ~2 m north of the south abutment to Sta. 16+910 or ~45 m south of the south abutment
- North Approach: From Sta. 17+085 or ~2 m south of the north abutment to Sta. 17+110 or ~25 m north of the north abutment

Based on the above estimates, ground improvement including wick drains and embankment preloading will be required to accommodate the construction schedule and to reduce the post-construction settlement. Settlement of the approach embankments will also induce downdrag load and lateral load on the abutment piles if installed prior to substantial completion of the ground settlement. It is therefore recommended that the abutment piles not be installed until substantial completion of consolidation under the new fill, e.g. 90% degree of consolidation, is achieved in the foundation clay. The preload embankments should be constructed to the subgrade level using earth fill or select subgrade material (SSM), and then pavement granular material to the top of pavement level. The crest of the preload embankment slopes facing the river valley should be

constructed to the abutment stations with an inclination no steeper than 2H:1V towards the river valley. The new embankments must be overbuilt to accommodate the settlement.

North approach embankment from Sta. 17+110 to 17+135 or the area extending from 25 to 50 m north of the north abutment may experience up to 70 mm of post-construction settlement, which exceeds MTO's settlement criterion of 50 mm within this area, as wick drains and preloading can not be installed in this area due to various construction constraints and staging requirements. More regular roadway maintenance may be anticipated in these areas following construction.

It is also understood that preloading of the existing roadway as part of the new approach embankment between approximately 17+090 and 17+110 will not be possible due to the need to keep the existing roadway open during construction. It is however recommended that wick drains be installed in the area through the existing roadway after the traffic is diverted to the new roadway to allow the foundation settlement to achieve substantial completion within a relatively short period following construction, e.g., 6 months for 1.5 m wick spacing. If it is not possible to install wick drains within the existing roadway, the post-construction foundation settlement of up to 65 mm under the higher embankment is expected to occur over an extended period of 10 to 15 years during which the roadway may require regular maintenance.

Surcharging above the new fill is not considered necessary given the potential for increased settlement at the existing bridge abutments and adjacent roadway embankments.

Settlements of the existing bridge abutments due to new approach fill placement for the replacement structure were estimated to be in the order of 10 mm.

The new approach fill will be placed in the vicinity of the existing roadway embankments and induce settlement of the existing roadway. It is estimated that 20 to 30 mm settlement will occur on the existing roadway near the proposed south abutment, and approximately 50 to 100 mm settlement will occur on the existing roadway near the proposed north abutment during construction. Maintenance of the existing roadway within the settlement zone of influence is to be anticipated during construction.

An embankment monitoring program is recommended to allow assessment of the foundation performance during fill placement and determination of the timing for pavement construction. As a minimum, the embankment sections within 50 m of the abutments should be instrumented and monitored. The monitoring instrumentation may include vibrating wire piezometers (VWP) and settlement rods (SR).

Alternatively, the approach embankments may be constructed using light-weight fill such as expanded polystyrene blocks (EPS) in order to produce a zero net loading situation at the abutments. Excavation below the existing ground surface to compensate for the pavement loading and minimum cover requirement may be needed to minimize foundation loading. A comparison of the advantages and disadvantages between light-weight fill and wick drains is provided in the table below.

Wick Drain	Light-Weight Fill (EPS)
<p><u>Advantages:</u></p> <ol style="list-style-type: none"> 1) Less costly than light-weight fill option. 2) Ease of construction. 3) Significantly expedite clay consolidation. 	<p><u>Advantages:</u></p> <ol style="list-style-type: none"> 1) Permits zero net foundation loading condition. 2) Requires no wait period prior to pile installation.
<p><u>Disadvantages:</u></p> <ol style="list-style-type: none"> 1) Requires wait time before pile installation. 2) May require benching for wick installation given slope height at south approach. 3) Deep installations at this site. 	<p><u>Disadvantages:</u></p> <ol style="list-style-type: none"> 1) More costly than wick drain option. 2) High river level at this site. 3) May require excavation below existing ground to compensate for pavement and soil cover and associated dewatering. 4) Requires benching of existing slope for EPS placement. 5) Extraction of temporary roadway protection through EPS fill may be problematic.

13.2 Embankment Stability

Analysis of the global stability of the approach embankments has been conducted for both river-facing forward slopes and embankment side slopes. The analysis indicates that the factors of safety for global stability were typically greater than 1.3 for the short-term condition (undrained analysis) and greater than 1.5 for the long-term condition (drained analysis). These factors of safety are deemed satisfactory from a geotechnical perspective. Results of selected stability analysis runs are included in Appendix G.

13.3 Cut Slope Construction

Excavation for cut slope construction should be carried out in accordance with OPSS.PROV 206.

Slope inclination of earth cuts should not be steeper than 2H:1V for cut slopes less than 4.5 m depth and not steeper than 3H:1V for cut slopes at or deeper than 4.5 m. A 2 m wide mid-height

bench should be incorporated along the length of earth cut with depths at or exceeding 6 m. The bench should maintain a 2% slope to shed surface runoff.

During construction of earth cuts some areas may reveal a final subgrade to be soft and moisture sensitive. Any soft and weak soils in the exposed subgrade should be removed and replaced with granular material compacted as per OPSS.PROV 501.

Temporary drainage of the cuts should be provided to maintain a relatively dry, stable excavation. Permanent drainage of the cuts must also be provided. Roadside ditches are expected to provide an adequate level of surface drainage in most areas. An interceptor ditch should be provided at the top of the earth cuts as per OPSD 200.020 and 201.020.

Where fine-grained silt and clay soils are exposed on a cut slope, the native soils are soft and moisture sensitive and may become negatively impacted after spring thaw and/or ingress of surface water and/or changes in the water table. The properties of the soils are such that the fluctuation in moisture content is likely to soften the soils and to result in erosion and/or sloughing of the soils and resulting in instability of the cut slopes. Such areas must be protected from erosion both on a temporary and permanent basis.

14. SCOUR AND EROSION PROTECTION

Erosion protection should be provided along any soil surfaces that may be in contact with the river flow. Scour protection should be provided for the pile caps at the piers. Design of such protection measures should be carried out by a professional engineer experienced in such design.

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

15. CONSTRUCTION CONCERNS

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

- The Contractor should be aware of the potentially fluctuating river level and anticipate the possible impacts on construction and dewatering requirements.
- Variations in ground conditions may require longer piles to achieve design capacity.
- Construction staging will require careful planning to keep the existing roadway open to public traffic, especially at the north approach.

- The Contractor's selection of construction equipment and methodology should include assessment of the capability of the subgrade soils to support the proposed construction equipment and any temporary structures or fill (i.e. as a pad for crane support). Site conditions may limit the type of equipment suitable for use. The design and safety of any temporary works is the responsibility of the Contractor. Recommended wording for an NSSP addressing this issue is provided in Appendix E.

16. CLOSURE

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

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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


4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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

 Water Level
 Shear Strength Determination by Pocket Penetrometer

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

EXPLANATION OF ROCK LOGGING TERMS


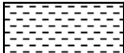



ROCK WEATHERING CLASSIFICATION

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

DISCONTINUITY SPACING

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

SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

STRENGTH CLASSIFICATION

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

TERMS

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

UNIFIED SOILS CLASSIFICATION

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

RECORD OF BOREHOLE No BR-01

1 OF 6

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2015.11.24 - 2015.11.26 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
185.8	GROUND SURFACE							20	40	60	80	100						
0.0	TOPSOIL: (50mm)							20	40	60	80	100						
	SAND , trace silt, trace gravel Loose Brown Moist (FILL)		1	SS	7		185											1 95 4 (SI+CL)
			2	SS	4													
184.4																		
1.4	Silty CLAY , occasional sand seams, varved Firm to Stiff Grey Wet		3	SS	7		184											
			4	SS	4		183											0 0 44 56
			5	SS	2		182											
			1	TW			181											0 0 28 72
							180											
			6	SS	2		179											
			7	SS	1		178											
							177											
			8	SS	1		176											0 0 20 80

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

RECORD OF BOREHOLE No BR-01

2 OF 6

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2015.11.24 - 2015.11.26 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page													
	Silty CLAY , occasional sand seams, varved Firm to Stiff Grey Wet		2	TW			175	4.0						
							174							
			9	SS	1		173							
							172	5.0						
			10	SS	2		171							
							170							
			11	SS	2		169	5.0						
							168							
			12	SS	2		167							
							166	4.0						

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METRIC

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RECORD OF BOREHOLE No BR-01

5 OF 6

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 448.0 E 402 608.3 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2015.11.24 - 2015.11.26 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page													
	Silty CLAY , varved Firm to Stiff Grey Wet		21	SS	4		145							0 0 32 68
							144	3.0						
							143							
							142							
			22	SS	4		141							
							140							
							139							
			23	SS	3		138							
							137							
137.3 48.5	SILT , some clay to clayey silt, occasional clay seams Loose Grey Wet						136							

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Sensitivity 20
15 10 5 (%) STRAIN AT FAILURE

METRIC

[illegible]

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BR-02

2 OF 5

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 475.4 E 402 612.8 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing/Dynamic Cone Penetration Test COMPILED BY AN
 DATUM Geodetic DATE 2015.11.18 - 2015.11.19 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	W _p W W _L	WATER CONTENT (%)				
	Continued From Previous Page													
	Silty CLAY , varved Very Soft to Stiff Grey Wet		7	SS	2		168							
			8	SS	1		167							
			9	SS	1		166	6.0						
			10	SS	2		165							
			11	SS	1		164							
			12	SS	1		163	6.0						
			13	SS	6		162							
							161	5.0						
							160							

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

METRIC

[illegible]


+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No BR-02

5 OF 5

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 475.4 E 402 612.8 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing/Dynamic Cone Penetration Test COMPILED BY AN
 DATUM Geodetic DATE 2015.11.18 - 2015.11.19 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	W _p	W	W _L			
	Continued From Previous Page													
133.3	45.7													
	END OF BOREHOLE AT 45.7m. BOREHOLE OPEN TO 45.7m AND WATER LEVEL IN CASING AT 0.3m. BOREHOLE BACKFILLED WITH GROUT TO SURFACE.													

ONTMT4S 19-5161-265B.GPJ 2015TEMPLATE(MTO).GDT 5/11/16

RECORD OF BOREHOLE No BR-03

1 OF 6

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 520.3 E 402 610.0 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2015.11.20 - 2015.11.21 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
179.0	TOP OF BARGE													
0.0	DECK													
178.7														
0.3	WATER													
174.1	Bottom of river													
4.9	Silty CLAY , varved Very Soft to Stiff Grey Wet		1	SS	2		174							0 0 32 68
			2	SS	2		173							
			3	SS	2		172							
			1	TW			171							
			4	SS	2		170							
			5	SS	2									

Continued Next Page


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

RECORD OF BOREHOLE No BR-03

2 OF 6

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 520.3 E 402 610.0 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2015.11.20 - 2015.11.21 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 ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				GR	SA	SI	CL		
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × LAB VANE											
	Continued From Previous Page							20 40 60 80 100				W _p W W _L								
	Silty CLAY , varved Very Soft to Stiff Grey Wet		6	SS	2															
			7	SS	1															
			8	SS	1															
			9	SS	2															
			10	SS	1															
			11	SS	2															
			12	SS	2															

Continued Next Page

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

RECORD OF BOREHOLE No BR-03

3 OF 6

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 520.3 E 402 610.0 ORIGINATED BY GA
HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
DATUM Geodetic DATE 2015.11.20 - 2015.11.21 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
	Continued From Previous Page															
	Silty CLAY , varved Very Soft Grey Wet															
			13	SS	2											
			14	SS	2											
			15	SS	4											

Continued Next Page

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

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

ONTMT4S 19-5161-265B.GPJ 2015TEMPLATE(MTO).GDT 5/11/16

RECORD OF BOREHOLE No BR-03

5 OF 6

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 520.3 E 402 610.0 ORIGINATED BY GA
HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
DATUM Geodetic DATE 2015.11.20 - 2015.11.21 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	Continued From Previous Page							20 40 60 80 100						
	Silty CLAY , varved Very Soft Grey Wet		19	SS	4		138							
							137	4.0 +						
							136							
			20	SS	4		135							0 0 64 36
							134							
133.3							133							
45.7	SILT , some clay to clayey silt, occasional silty clay seams Loose Grey Wet		21	SS	4		132							
							131							
							130							

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No BR-03

6 OF 6

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 520.3 E 402 610.0 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2015.11.20 - 2015.11.21 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
	Continued From Previous Page		22	SS	6												
	SILT, some clay, occasional silty clay seams Loose Grey Wet																
127.2			23	SS	8												
51.8	END OF BOREHOLE AT 51.8m. BOREHOLE OPEN TO 51.8m AND WATER LEVEL IN CASING AT 0.3m. BOREHOLE BACKFILLED WITH GROUT TO SURFACE.																

RECORD OF BOREHOLE No BR-04

1 OF 4

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 551.0 E 402 605.2 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2015.11.26 - 2015.11.27 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa													
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							PLASTIC LIMIT W _P		NATURAL MOISTURE CONTENT W		LIQUID LIMIT W _L		
								20 40 60 80 100							WATER CONTENT (%) 20 40 60						
185.7	GROUND SURFACE					▽															
0.0	TOPSOIL: (50mm)																				
	Gravelly SAND , trace silt, occasional rootlets Compact to Loose Grey Moist (FILL)		1	SS	10																33 59 8 (SI+CL)
			2	SS	9																
184.2																					
1.5	Silty CLAY , sandy, trace gravel Firm Brown Wet (FILL)		3	SS	4																
183.3																					
2.4	Silty CLAY , occasional sand seams, varved Very Soft to Stiff Grey Wet		4	SS	4																
			5	SS	2																0 25 50 25
			6	SS	2																
			7	SS	6																
			1	TW																	
			8	SS	2																

Continued Next Page

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

RECORD OF BOREHOLE No BR-04

2 OF 4

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 551.0 E 402 605.2 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2015.11.26 - 2015.11.27 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE		WATER CONTENT (%) W _p W W _L				
	Continued From Previous Page													
	Silty CLAY , occasional sand seams, varved Very Soft to Stiff Grey Wet		9	SS	2		175							
							174	5.0						
			10	SS	1		173							0 0 36 64
							172							
			2	TW			171	5.0						
							170							
			11	SS	1		169							
							168	5.0						
							167							
			12	SS	0		166							
			13	SS	1									

Continued Next Page

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

RECORD OF BOREHOLE No BR-04

3 OF 4

METRIC

W.P. 5163-13-00 LOCATION Blanche River Bridge N 5 288 551.0 E 402 605.2 ORIGINATED BY GA
 HWY 569 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2015.11.26 - 2015.11.27 CHECKED BY AMP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page		14	SS	1									
	Silty CLAY , occasional sand seams, varved Very Soft to Stiff Grey Wet						165	5.0 +						
							164							
			15	SS	0		163							
							162	5.0 +						
							161							
							160							
			16	SS	0		159							
							158	4.0 +						
							157							
			17	SS	1		156							

Continued Next Page

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

METRIC

+³, ×³: Numbers refer to Sensitivity



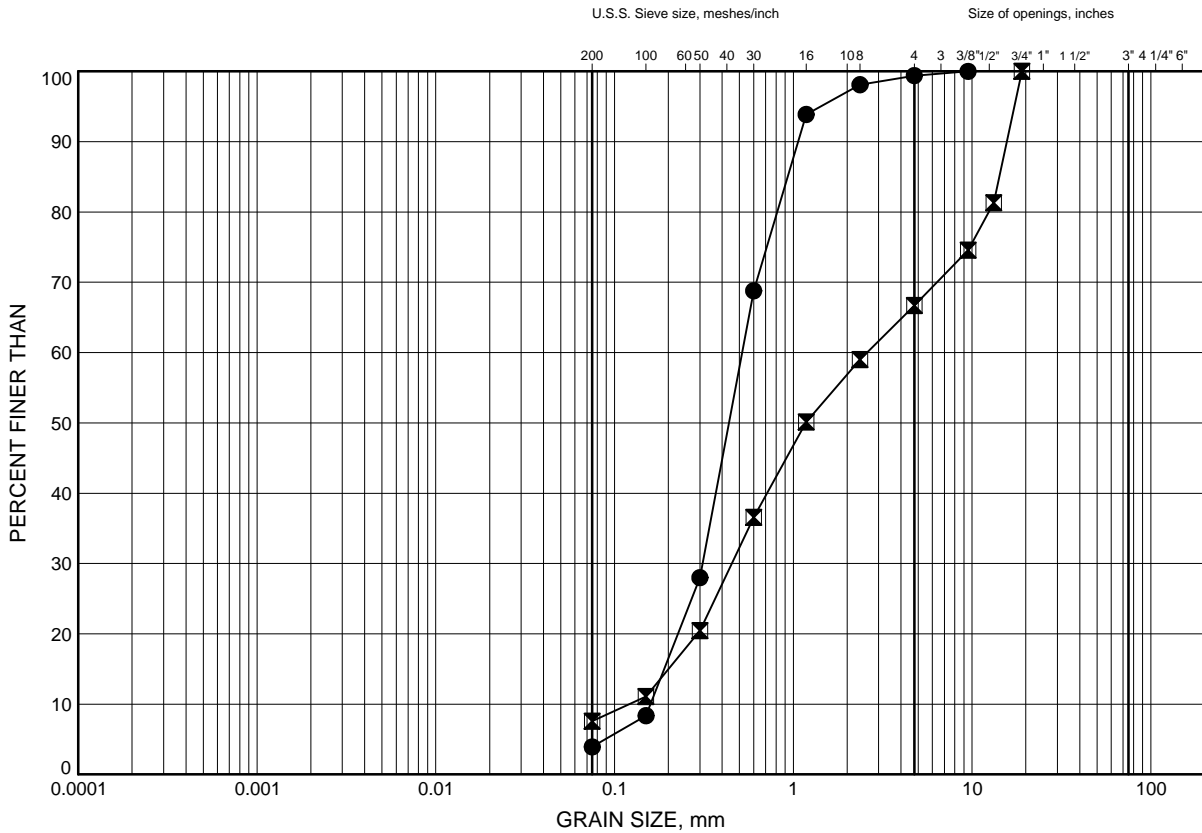
Appendix B

Laboratory Test Results

Blanche River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND to Gravelly SAND FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BR-01	1.07	184.73
⊠	BR-04	0.30	185.40

Date December 2015
W.P. 5163-13-00

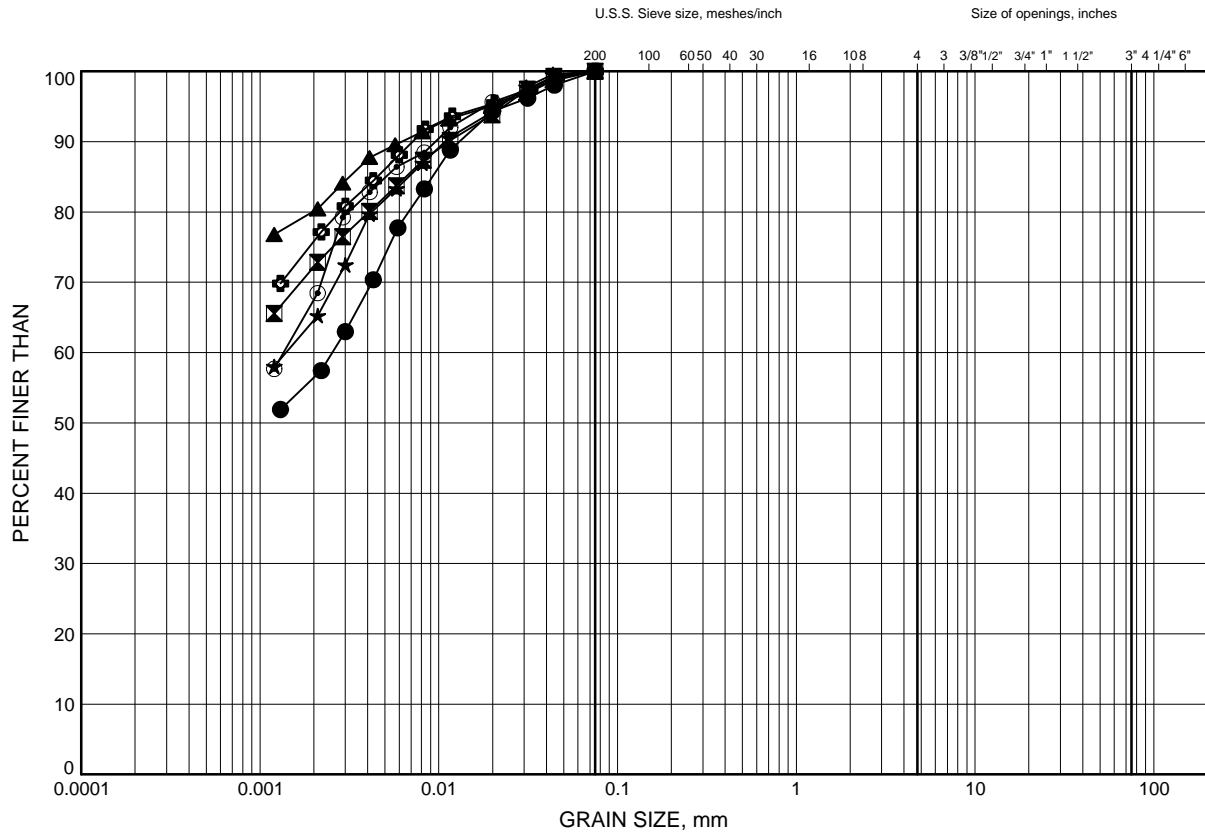


Prep'd AN
Chkd. AMP

Blanche River 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)
●	BR-01	2.59	183.21
⊠	BR-01	4.88	180.92
▲	BR-01	9.45	176.35
★	BR-01	23.16	162.64
⊙	BR-01	41.45	144.35
⊕	BR-02	5.18	173.82

Date December 2015
W.P. 5163-13-00

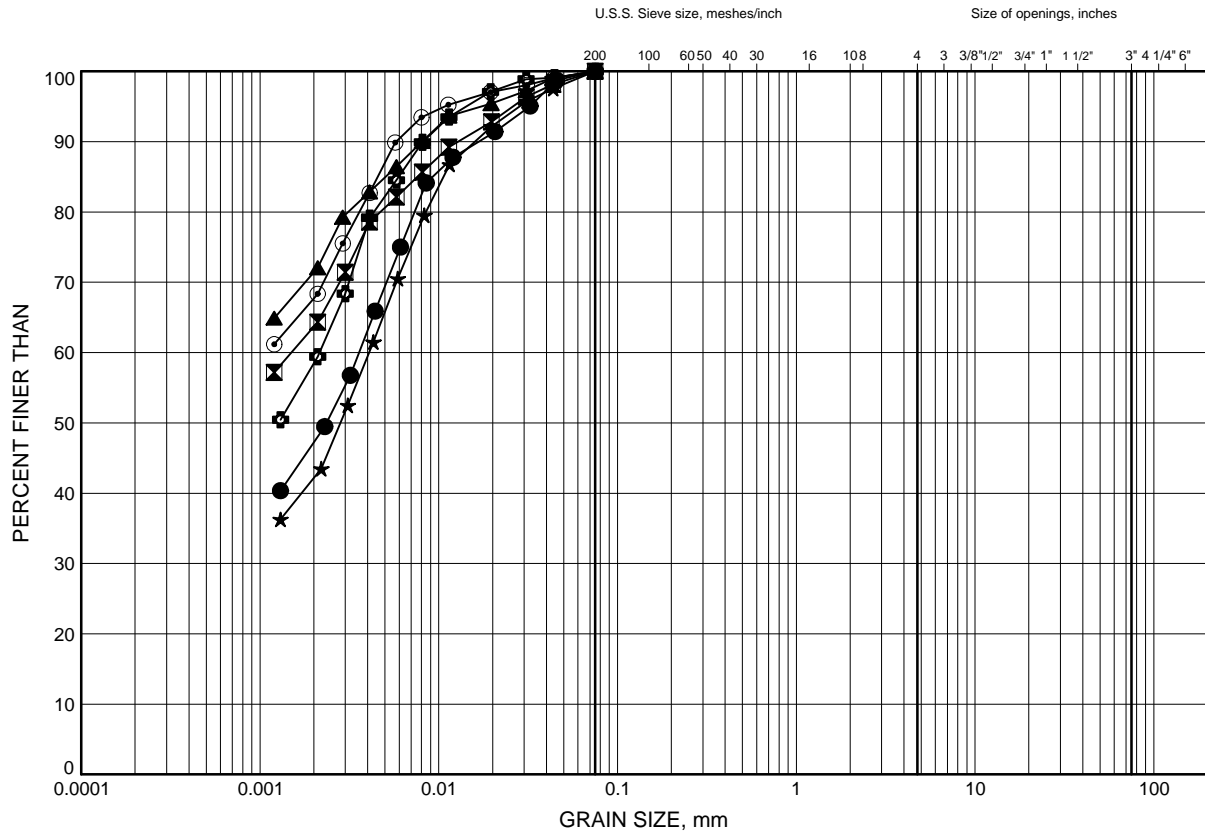


Prep'd AN
Chkd. AMP

Blanche River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B3

Silty CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BR-02	8.84	170.16
⊠	BR-02	13.41	165.59
▲	BR-02	19.51	159.49
★	BR-02	36.27	142.73
⊙	BR-03	5.18	173.82
⊕	BR-03	14.33	164.67

Date December 2015
W.P. 5163-13-00

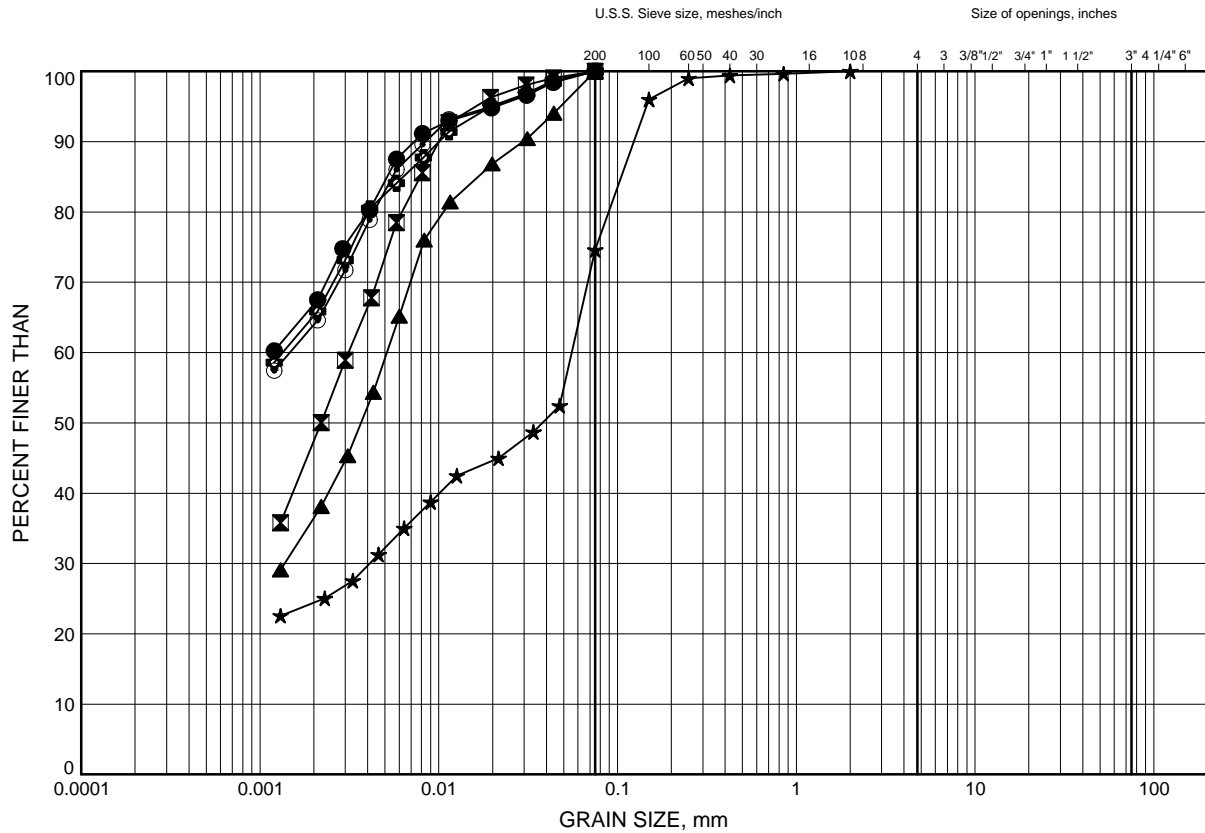


Prep'd AN
Chkd. AMP

Blanche River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B4

Silty CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BR-03	19.51	159.49
⊠	BR-03	37.80	141.20
▲	BR-03	43.89	135.11
★	BR-04	3.35	182.35
⊙	BR-04	12.50	173.20
⊕	BR-04	20.12	165.58

Date December 2015
W.P. 5163-13-00

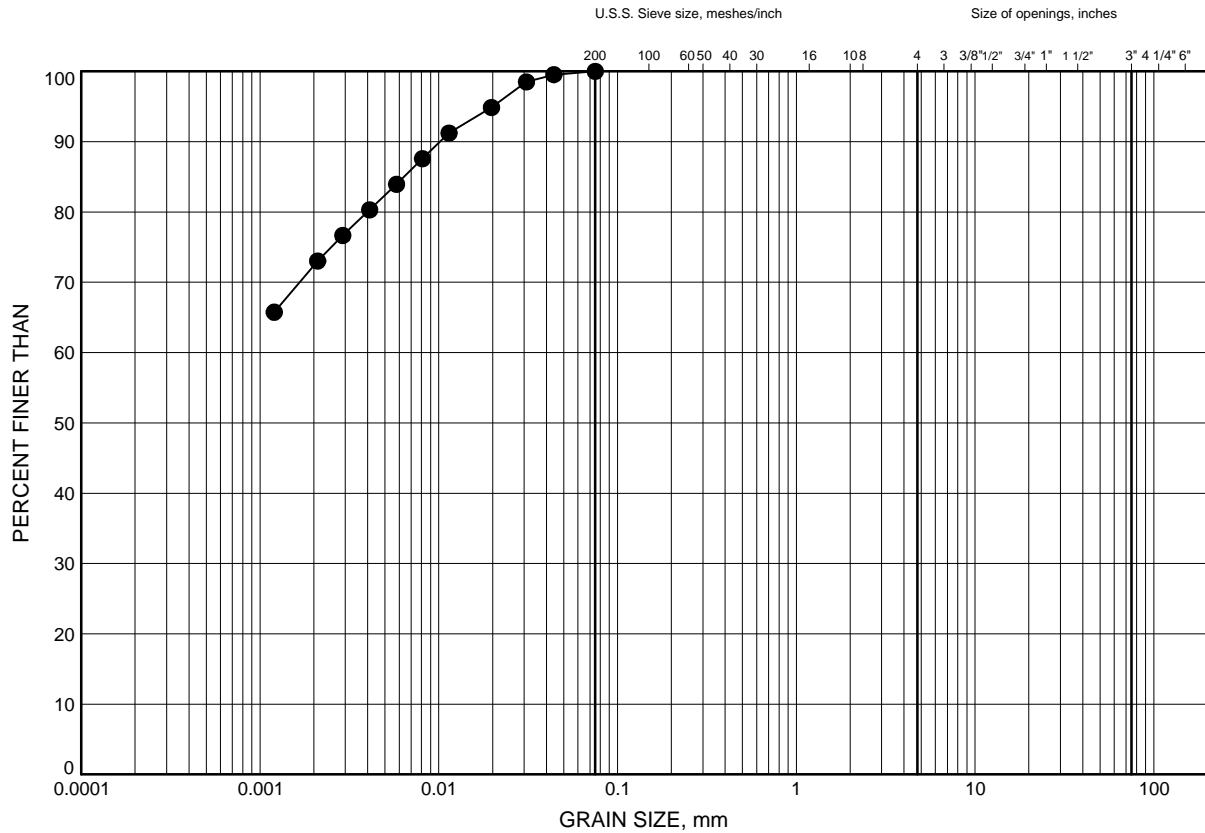


Prep'd AN
Chkd. AMP

Blanche River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B5

Silty CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BR-04	35.36	150.34

Date December 2015
W.P. 5163-13-00

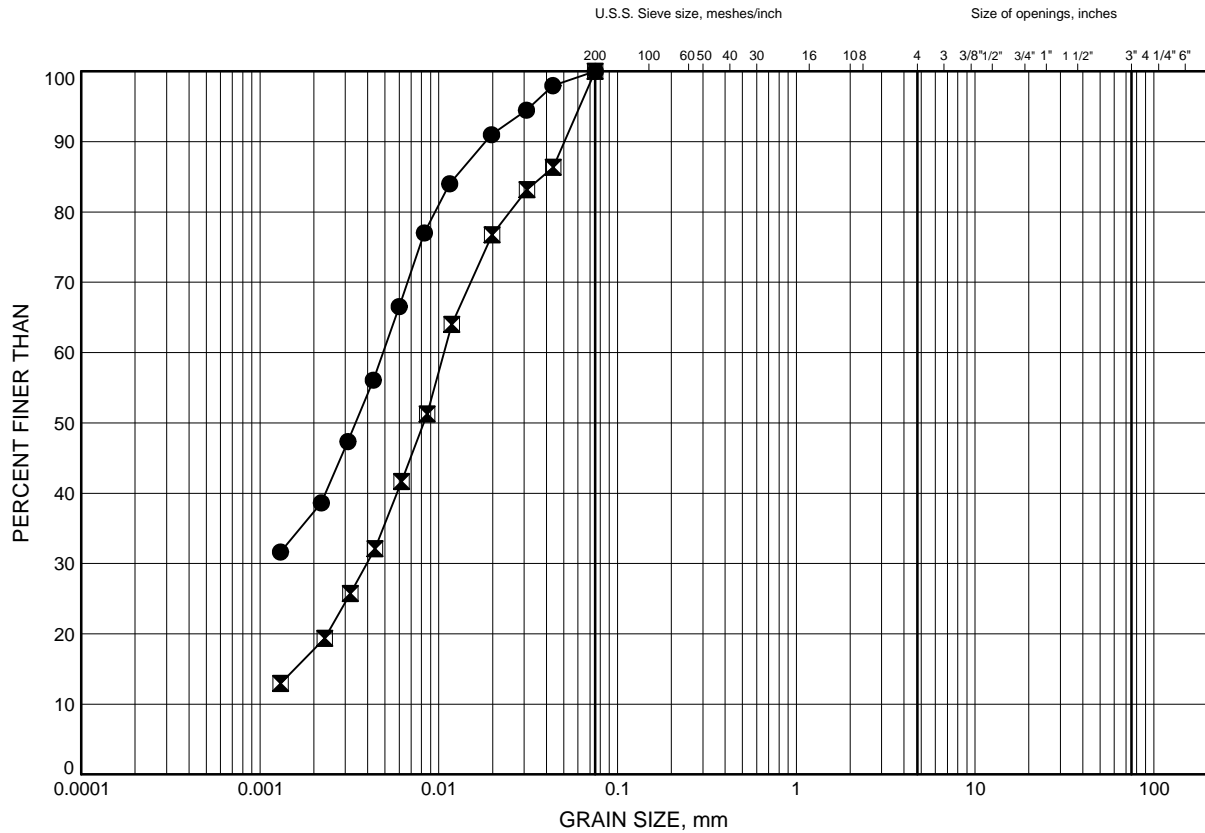


Prep'd AN
Chkd. AMP

Blanche River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B6

SILT to Clayey SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BR-01	52.12	133.68
⊠	BR-03	49.99	129.01

Date December 2015
W.P. 5163-13-00

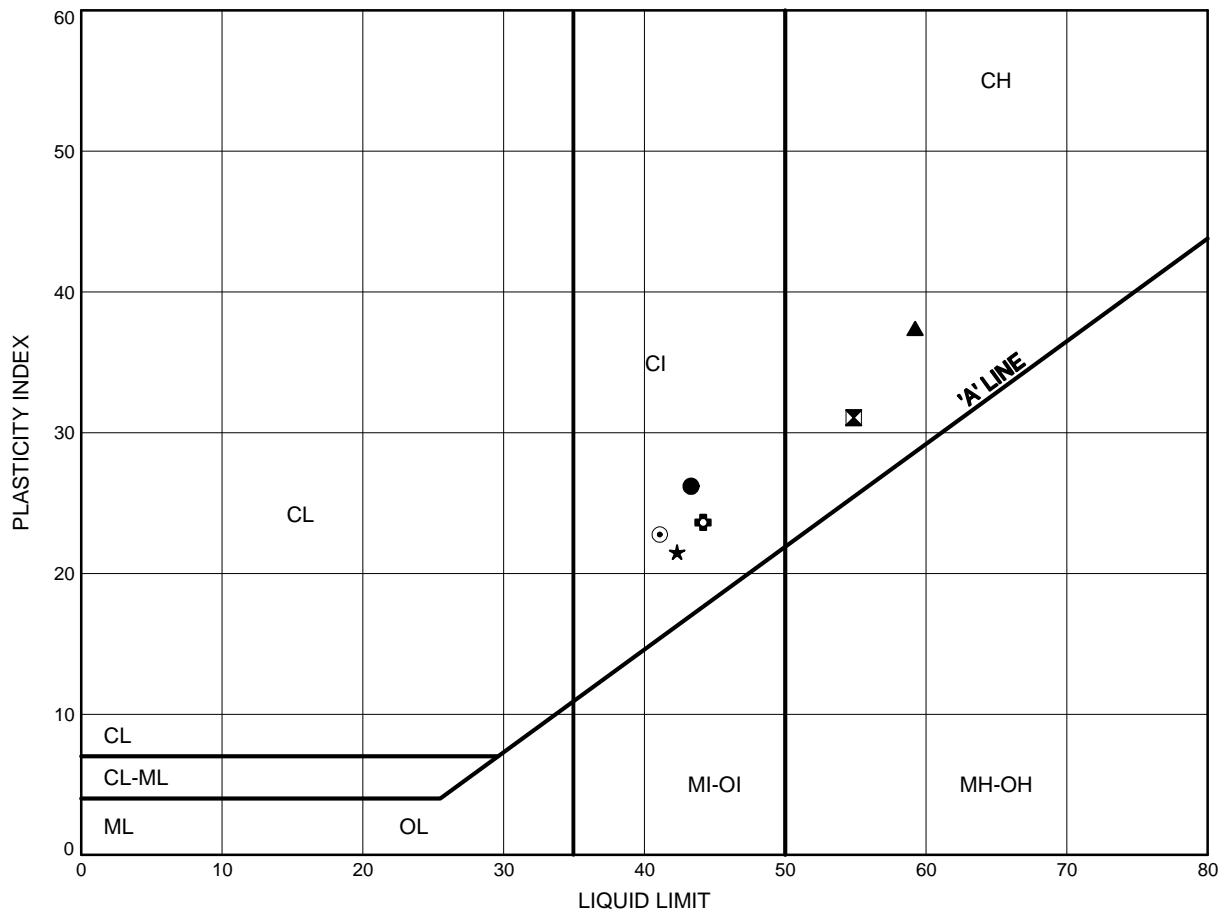


Prep'd AN
Chkd. AMP

Blanche River Bridge ATTERBERG LIMITS TEST RESULTS

FIGURE B7

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BR-01	2.59	183.21
⊠	BR-01	4.88	180.92
▲	BR-01	9.45	176.35
★	BR-01	23.16	162.64
⊙	BR-01	41.45	144.35
⊕	BR-02	5.18	173.82

Date December 2015
W.P. 5163-13-00

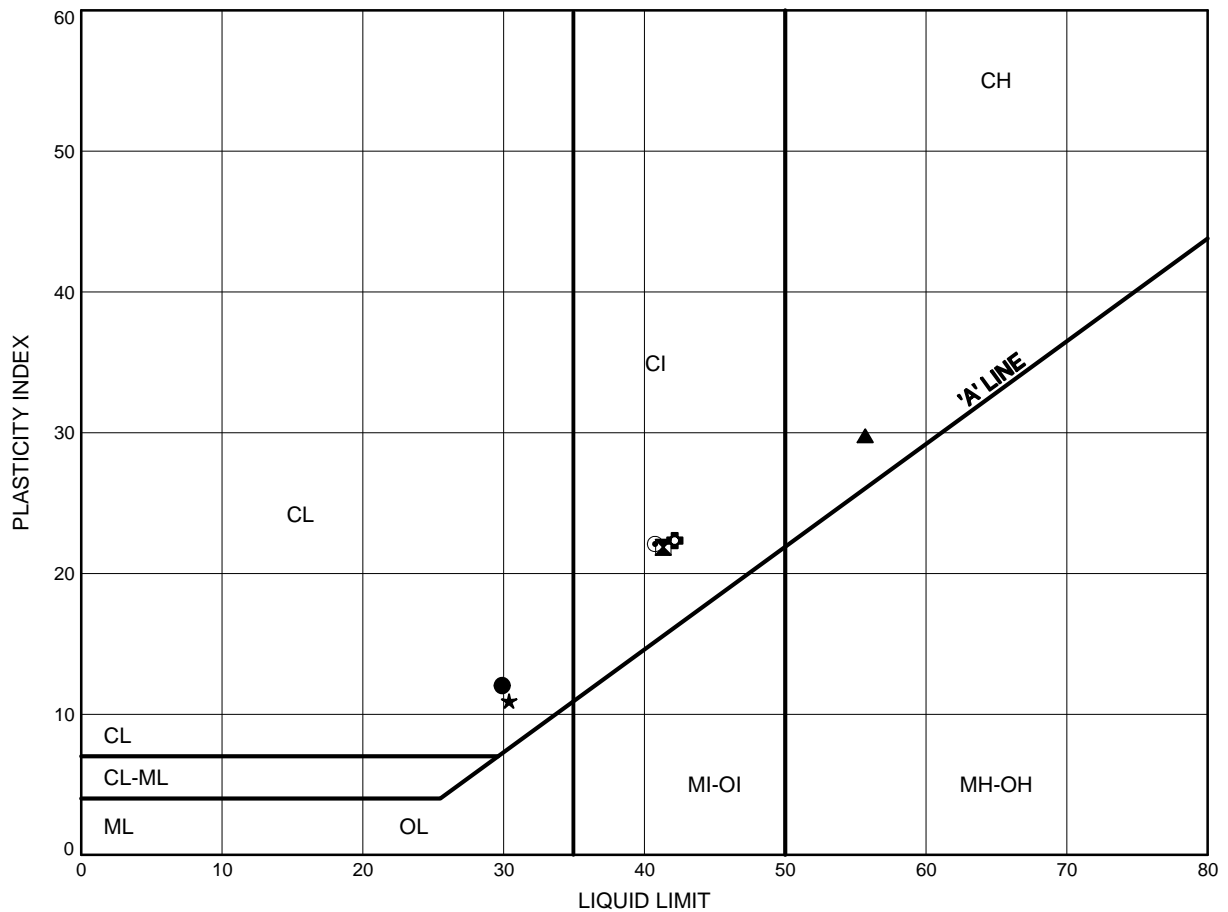


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Chkd. AMP

Blanche River Bridge ATTERBERG LIMITS TEST RESULTS

FIGURE B8

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BR-02	8.84	170.16
⊠	BR-02	13.41	165.59
▲	BR-02	19.51	159.49
★	BR-02	36.27	142.73
⊙	BR-03	5.18	173.82
⊕	BR-03	14.33	164.67

Date December 2015
W.P. 5163-13-00

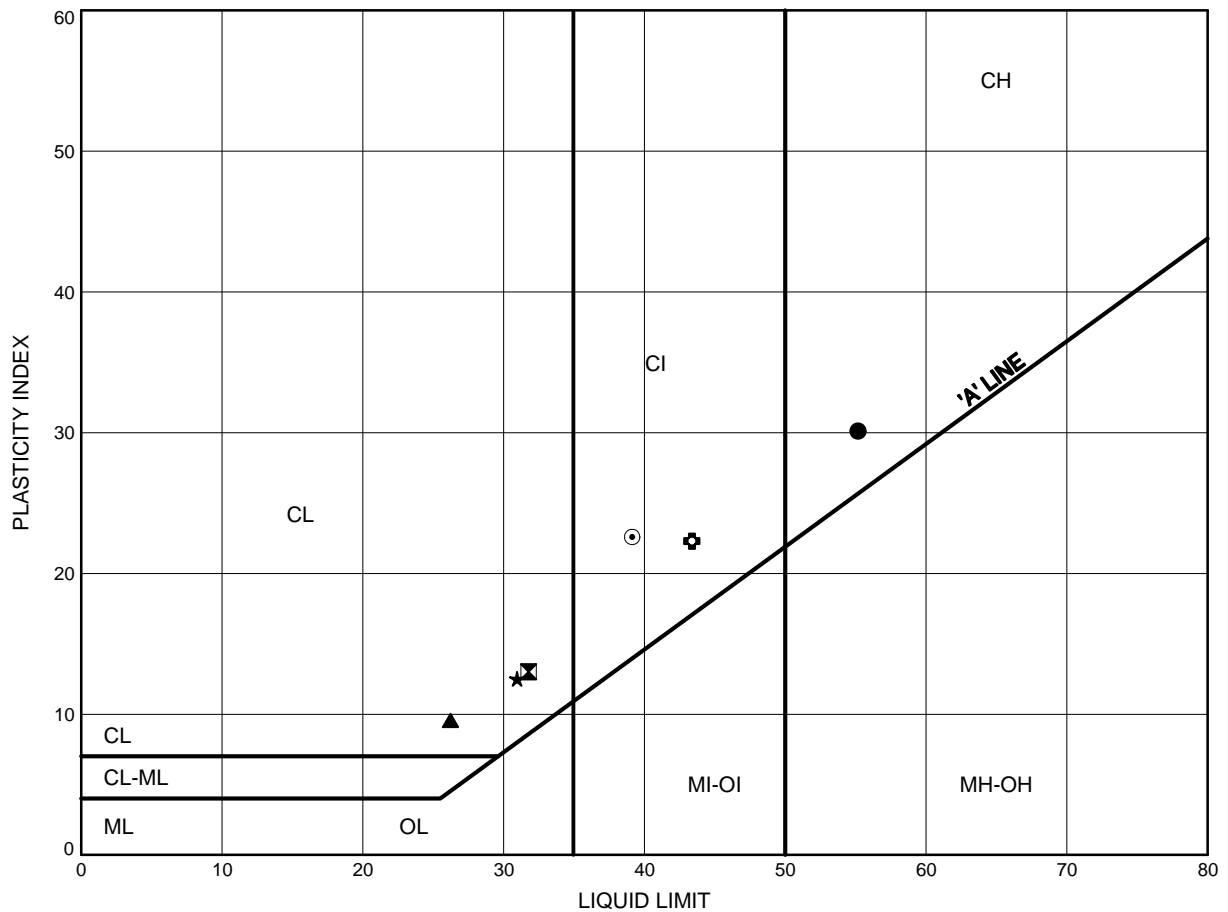


Prep'd AN
Chkd. AMP

Blanche River Bridge ATTERBERG LIMITS TEST RESULTS

FIGURE B9

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BR-03	19.51	159.49
⊠	BR-03	37.80	141.20
▲	BR-03	43.89	135.11
★	BR-04	3.35	182.35
⊙	BR-04	12.50	173.20
⊕	BR-04	20.12	165.58

Date December 2015
W.P. 5163-13-00

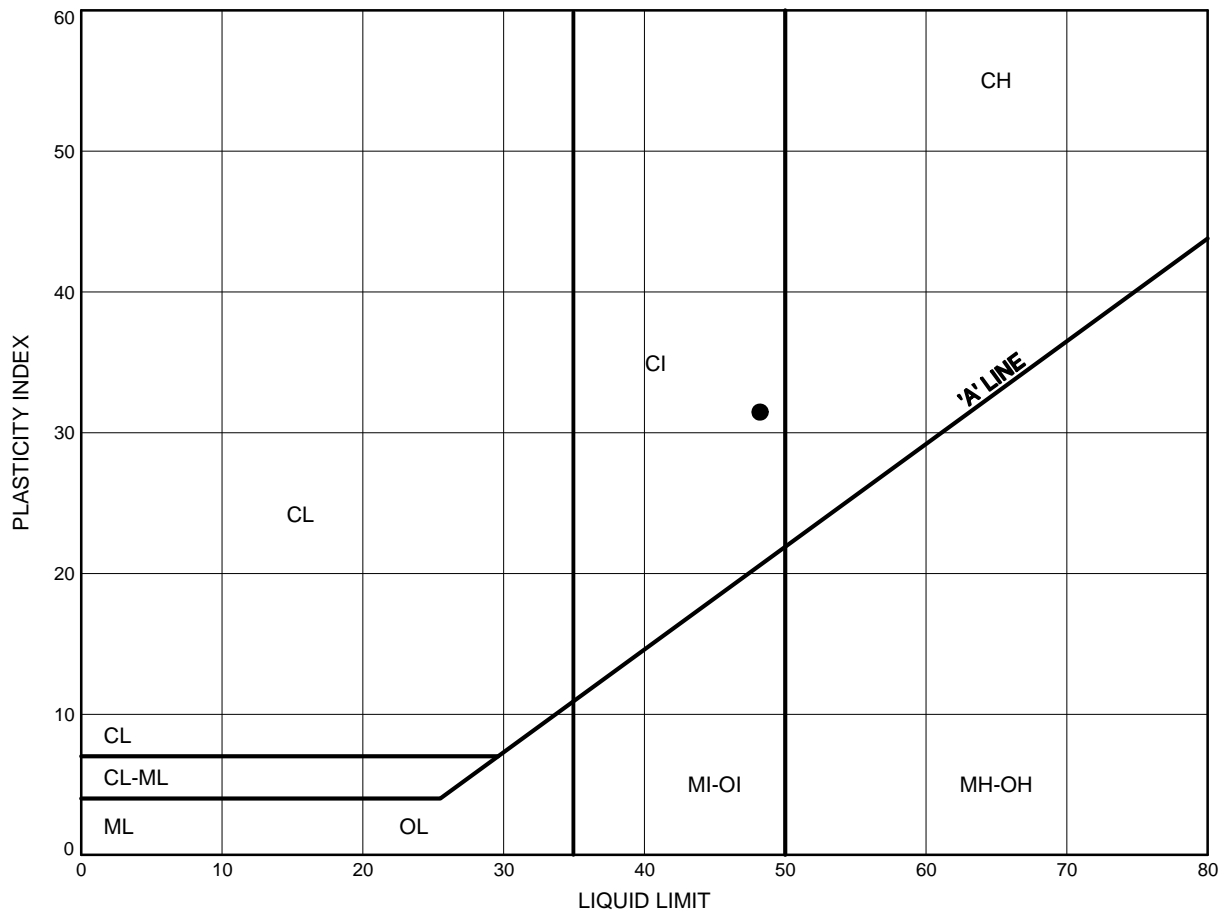


Prep'd AN
Chkd. AMP

Blanche River Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B10

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BR-04	35.36	150.34

Date December 2015
W.P. 5163-13-00

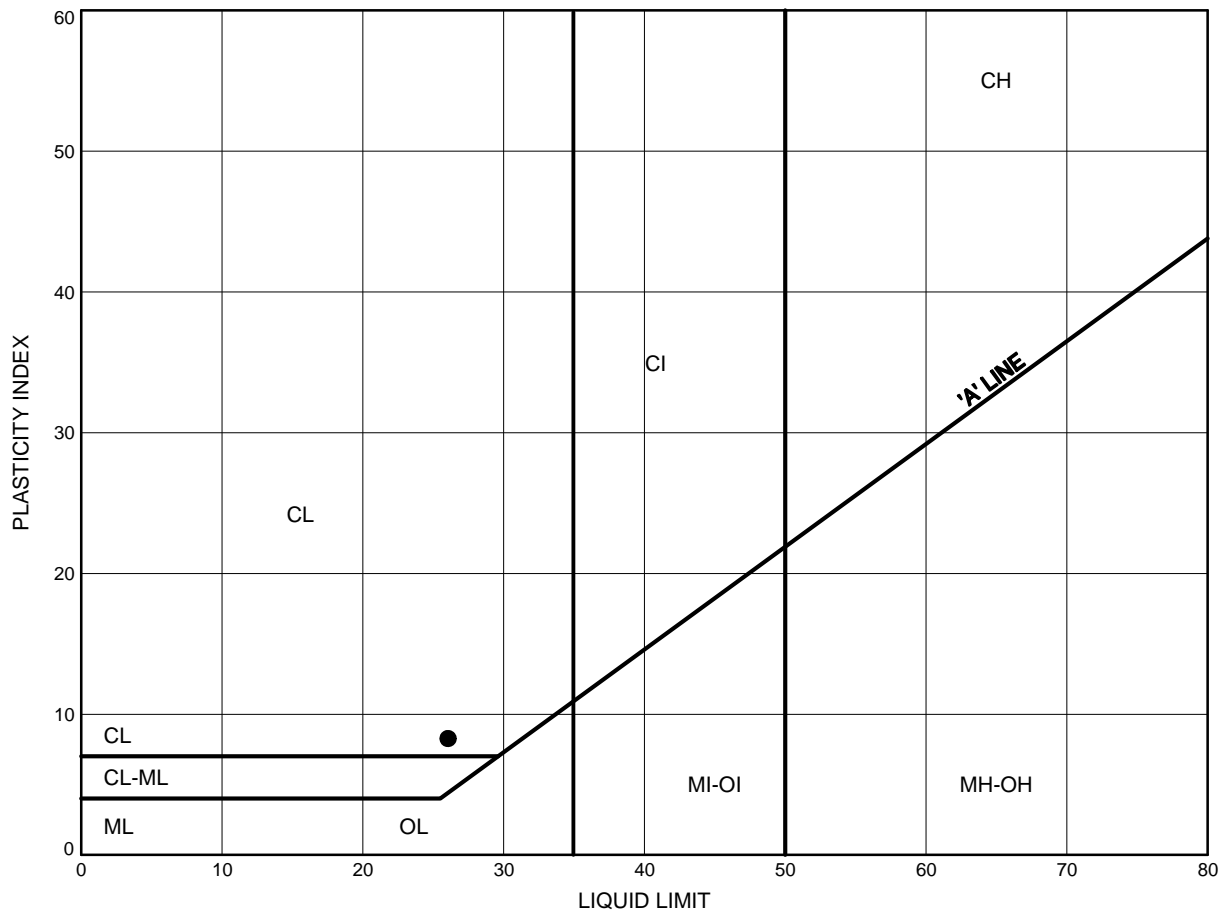


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Chkd. AMP

Blanche River Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B11

Clayey SILT



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	BR-01	52.12	133.68

Date December 2015
 W.P. 5163-13-00



Prep'd AN
 Chkd. AMP

Consolidation Test Report

CLIENT: **MMM Group Limited**

FILE NUMBER: **19-5161-251**

PROJECT: **Highway 569 Blanche River Bridge**

REPORT DATE: **4-Jan-2016**

TEST DATES: **December 09, 2015 - December 18, 2015**

SAMPLE: **BR-1-TW1 (15'-17')**
Silty Clay, varved, grey, 27% Silt and 73% Clay, LL=54.9%, PL=23.8%.

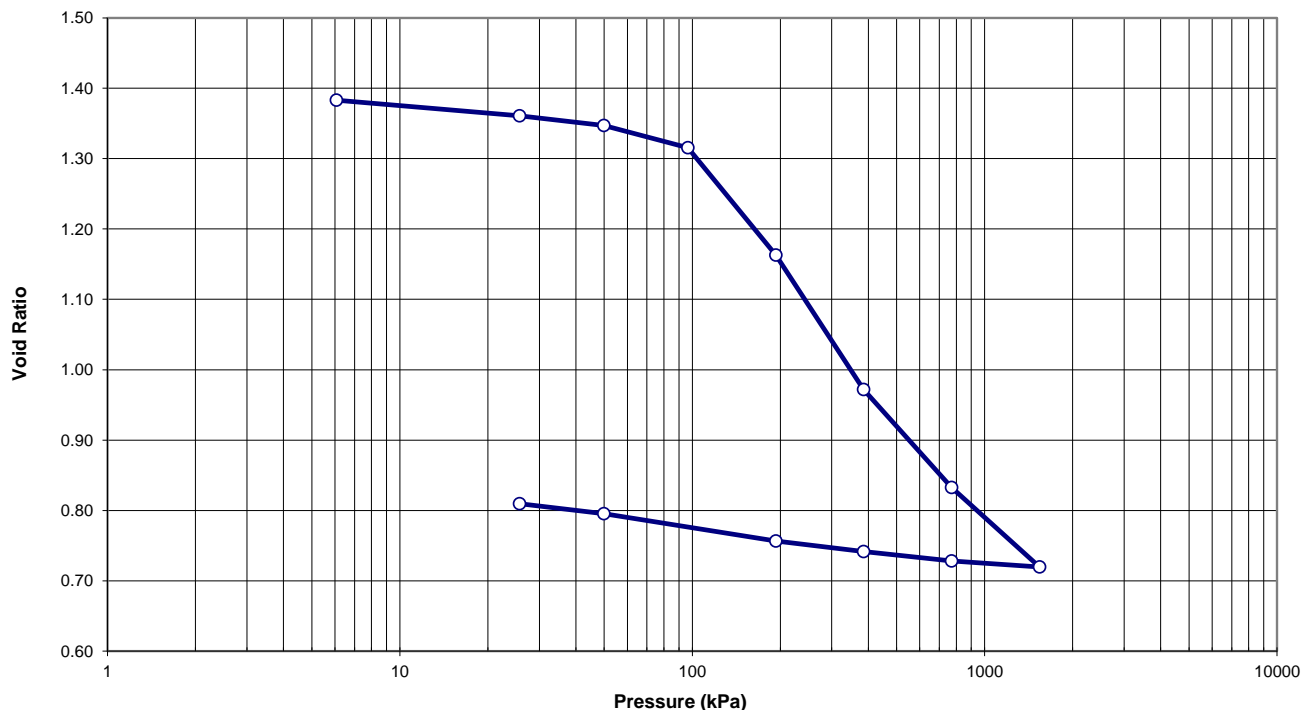
PROCEDURE: Test carried out in accordance with Standard Test Method for One-Dimensional Consolidation Properties of Soils, ASTM D 2435-04, method A

	<u>Start of Test</u>	<u>End of Test</u>
Wet Dens. (kg/m ³)	1775.6	2028.1
Dry Dens. (kg/m ³)	1164.7	1539.1
Moisture Cont. (%)	52.4	31.8
Void Ratio	1.391	0.809

Note: A Specific Gravity (Gs) of 2.79 was measured for the void ratio and saturation calculations.

Project #: 19-5161-251
 Client: MMM Group Limited
 Project Name: Highway 569 Blanche River Bridge
 Sample: BR-1-TW1 (15'-17')

Void Ratio vs. Pressure



Consolidation Test Report

Highway 569 Blanche River Bridge
19-5161-251

BR-1-TW1 (15'-17')

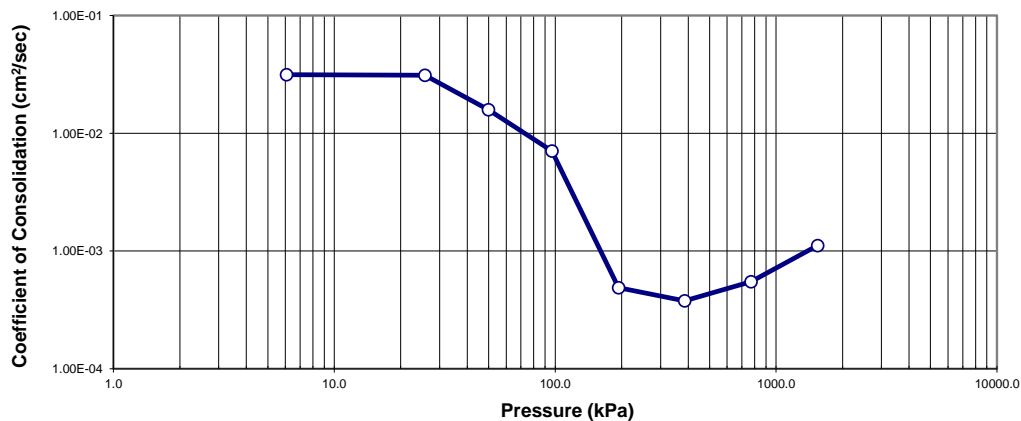
TRIMMING: The Specimen was manually trimmed to the size of consolidation ring, then mounted in a fixed ring consolidometer.

LOADING: A seating load of 6.1 kPa was applied and the consolidometer was flooded with distilled water. Sample was monitored to ensure no swelling effect occurred before the start of the test. Subsequent loads were applied after a constant load increment duration of 24 hours.

CALCULATIONS: Coefficients of Consolidation were calculated by the square root time method.

Pressure (kPa)	Corr. H. (mm)	Avg. H. (mm)	D_{90} (mm)	t_{90} (min)	c_v (cm ² /s)	Void Ratio	m_v (m ² /kN)	k (cm/s)
0.0	25.400					1.391		
6.1	25.316	25.358	-0.055	0.72	3.14E-02	1.383	5.45E-04	1.68E-06
25.7	25.082	25.199	-0.199	0.72	3.11E-02	1.361	4.72E-04	1.44E-06
49.9	24.935	25.009	-0.080	1.39	1.59E-02	1.347	2.42E-04	3.77E-07
96.6	24.599	24.767	-0.147	3.06	7.08E-03	1.316	2.88E-04	2.00E-07
193.2	22.978	23.789	-0.930	40.96	4.88E-04	1.163	6.82E-04	3.27E-08
385.7	20.947	21.963	-1.535	45.16	3.77E-04	0.972	4.59E-04	1.70E-08
770.7	19.470	20.209	-1.075	26.32	5.48E-04	0.833	1.83E-04	9.85E-09
1540.7	18.268	18.869	-0.764	11.29	1.11E-03	0.720	8.02E-05	8.76E-09
770.7	18.357	18.313				0.728		
385.7	18.499	18.428				0.741		
193.2	18.658	18.579				0.756		
49.9	19.074	18.866				0.795		
25.7	19.222	19.148				0.809		

Coefficient of Consolidation vs. Pressure



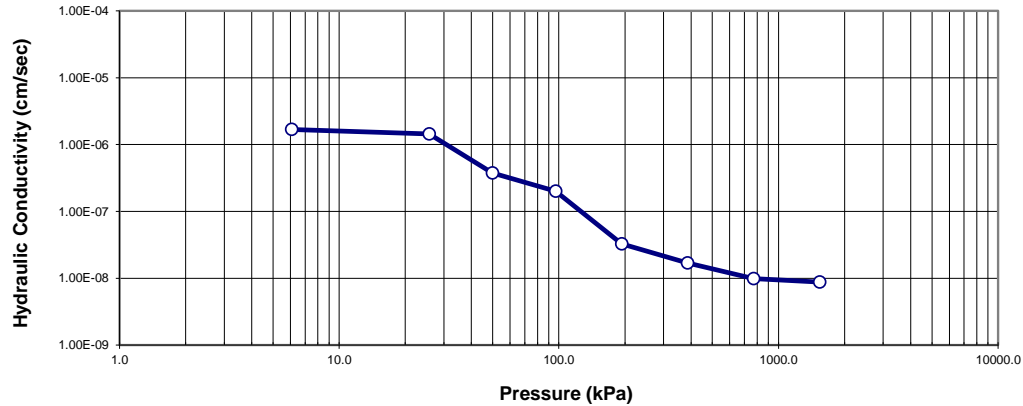
Notes: C_v and k calculated using t_{90} values

Consolidation Test Report

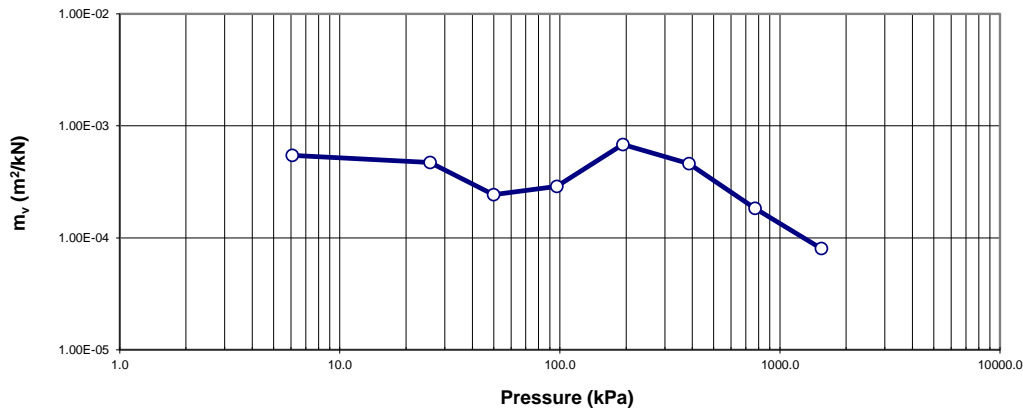
Highway 569 Blanche River Bridge
19-5161-251

BR-1-TW1 (15'-17')

Hydraulic Conductivity vs. Pressure



m_v vs. Pressure





Appendix C

Site Photographs



Photograph 1 – East Elevation Looking North



Photograph 2 – South Approach Looking North



Photograph 3 – North Abutment

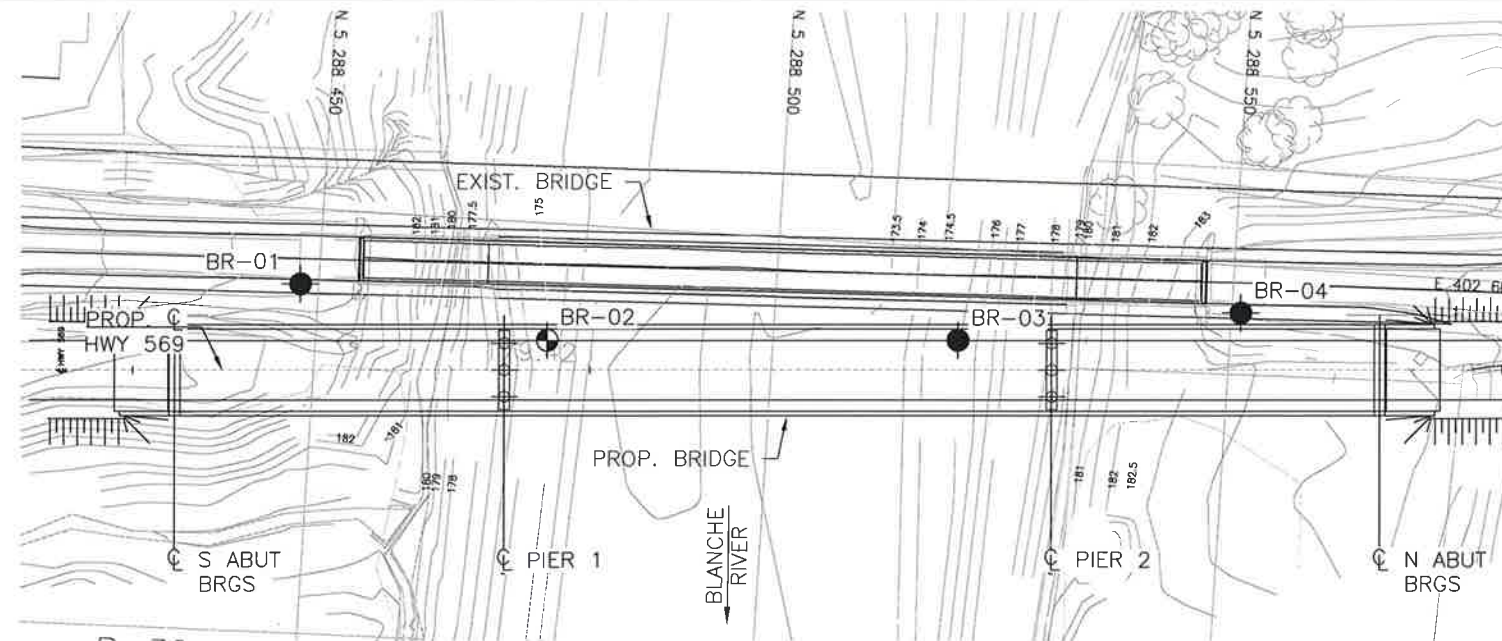


Photograph 4 – Typical Pier

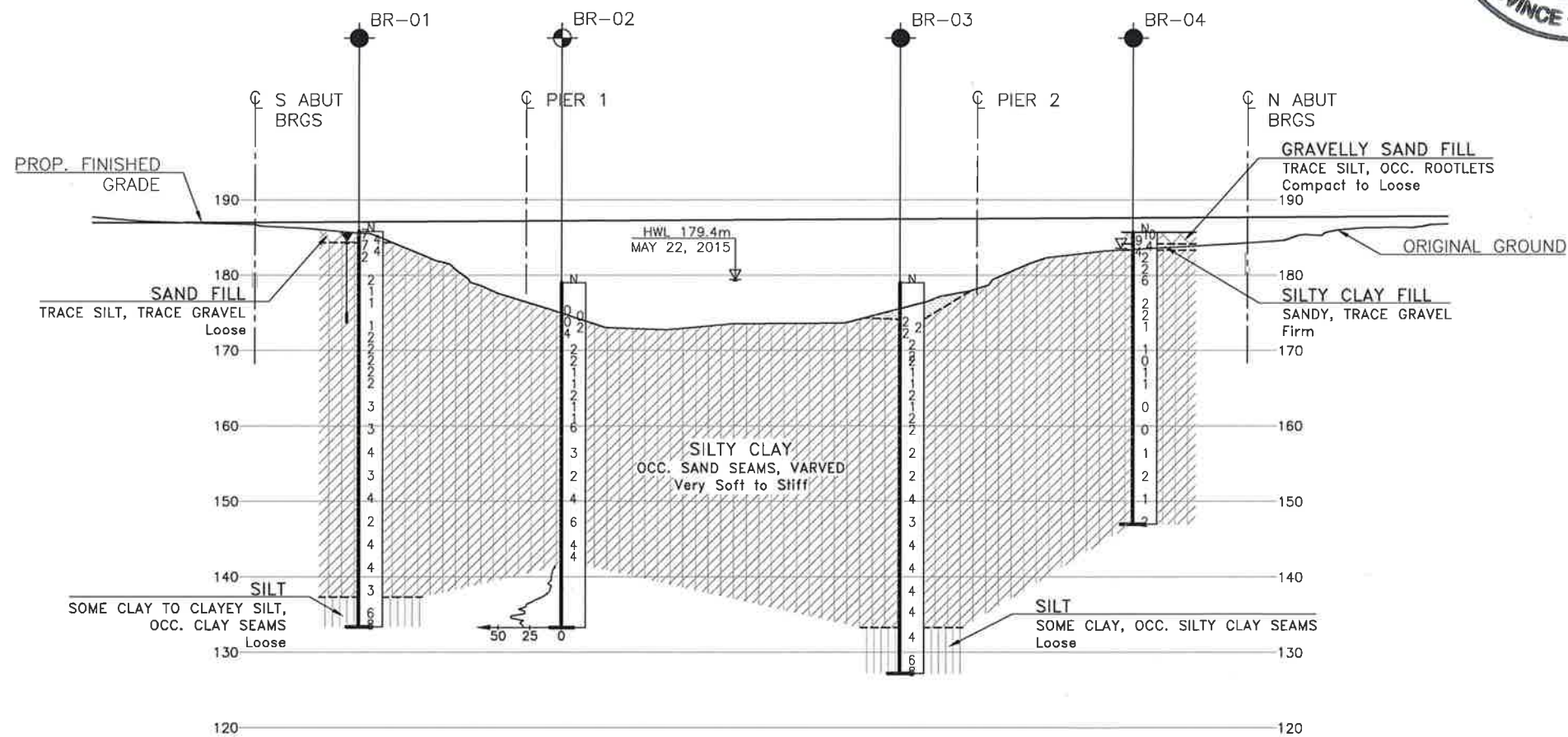


Appendix D

Borehole Locations and Soil Strata Drawing



PLAN
SCALE 1:800



PROFILE ALONG C HWY 569

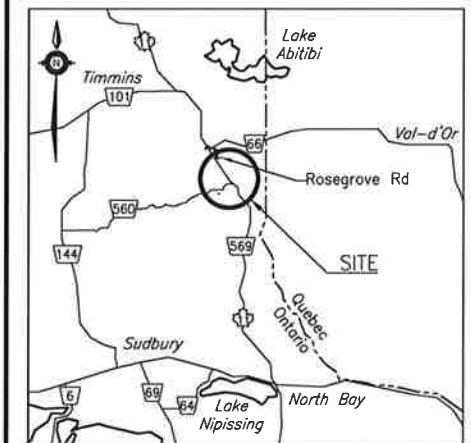
SCALE 1:800
SCALE 1:200

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



CONT No
WP No 5163-13-00

HIGHWAY 569
BLANCHE RIVER BRIDGE
REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING (MTM)	EASTING (MTM)
BR-01	185.8	5 288 448.0	402 608.3
BR-02	179.0	5 288 475.4	402 612.8
BR-03	179.0	5 288 520.3	402 610.0
BR-04	185.7	5 288 551.0	402 605.2

-NOTES-

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

GEOCRES No. 31M-120

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	AMP	CHK	KS
DRAWN	AN	CHK	SITE
LOAD	DATE	JUN 2017	
STRUCT	DWG	1	



Appendix E

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

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

- OPSS.PROV 206
- OPSS.PROV 220
- OPSS.PROV 501
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS 903
- OPSS.PROV 1010

2. Suggested text for NSSP on “Monitoring of Existing Structure”

The Contractor shall ensure that the existing structure remains stable while it remains in service and is not adversely affected by construction.

A monitoring program is required to confirm that any movements of the existing structure remain within tolerable levels. As a minimum, the monitoring program should require the Contractor to establish reference points over each foundation element of the existing structure and to monitor movement of these points relative to known, fixed reference points on a regular basis. The suggested frequency is:

- Three readings on separate days prior to construction to establish a baseline;
- Twice daily while any foundation construction or other subsurface construction is in progress;
- Daily for one week after completion of foundation construction;
- On such continuing basis as may be directed by the Contract Administrator.

The vertical and horizontal accuracy of readings should be ± 2 mm. All readings should be reported to the Contract Administrator within 24 hours and immediately if any movement exceeds limits set by the structural designer.

The Contract Administrator should be advised of the importance of monitoring and be required to advise the Ministry immediately if the vertical and horizontal movements exceed the specified limits.

3. Suggested text for NSSP on “Use of Heavy Construction Equipment”

The use of heavy construction equipment and in particular heavy lift cranes may be required during removal of the existing bridge and erection of the new bridge. The impact of the heavy

equipment loads on the existing embankment, the soft soils (silty clay) underlying the embankment, and the existing bridge foundations must be considered during selection of the methodology and equipment employed for construction.

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to assess the impact of the proposed equipment loads and methodology, and determine requirements and/or restrictions necessary to safely support the loads. All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO's RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – Medium Complexity.

The assessment shall include, but not be limited to, the following:

- Determining appropriate setbacks for heavy equipment from the bridge abutments and existing foundations;
- Determining the permissible ground pressure that may be applied to the foundation soils by the equipment; and
- Providing recommendations for crane pad design to distribute the crane loads without causing foundation failure.

The Contractor shall submit the findings of the geotechnical assessment and details of the proposed equipment and construction methodology to the Contract Administrator for information purposes a minimum of two weeks prior to the start of construction.

4. Suggested text for NSSP on “Pile Retapping”

A pile shall be retapped no sooner than 7 days after installation of the individual pile to confirm that the ultimate axial resistance has been achieved.

5. An example NSSP on “High-Strain Dynamic Testing”

HIGH-STRAIN DYNAMIC TESTING, DEEP FOUNDATIONS – Item No.

Special Provision

May 2017

Amendment to OPSS 903, April 2016

903.02 REFERENCES

Section 903.02 of OPSS 903 is amended by the addition of the following under **ASTM International**:

D 4945-12 Standard Test Method for High-Strain Dynamic Testing of Deep Foundations

903.03 DEFINITIONS

Section 903.03 of OPSS 903 is amended by the addition of the following:

High Strain Dynamic Testing means a method of evaluating the quality of deep foundations and/or performance of the drive system. It is a form of load testing and involves the instrumenting and application of dynamic loads to a tested pile.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.02 Submission Requirements

Subsection 903.04.02 of OPSS 903 is amended by the addition of the following clause:

903.04.02.06 High-Strain Dynamic Testing

Prior to commencing high-strain dynamic testing, calibration certificates of all equipment used shall be submitted to the Contract Administrator. Strain gauges and accelerometers shall be in good working condition, and shall have been calibrated in accordance with ASTM D 4945 within the last 2 years.

A preliminary report on the test results and its analysis shall be submitted to the Contract Administrator on the same day of the testing. The analysis shall be based on a closed-form solution (Case Method or approved equivalent) or signal-matching analyses (CAPWAP or approved equivalent). As a minimum, the preliminary report shall include:

- a) The estimated pile capacity and integrity.
- b) Calculated driving stresses.
- c) Transferred energy and hammer efficiency at the time of the test.

A digital copy of the measured signal/wave data file shall be submitted to the Contract Administrator for information purposes. This data shall include the raw data prior to signal matching analysis.

A final report and the digital signal/wave file (both, raw and post-analysis) shall be provided to the Contract Administrator within 10 Calendar Days of the field testing. The final report shall include the following:

- a) Results of pile capacity and pile integrity based on signal-matching analyses (CAPWAP or approved equivalent), hammer performance and comparisons with any applicable static load test.
- b) Discussion and recommendations for soil setup/relaxation, and/or revised pile installation criteria.
- c) An appendix containing the following documents:
 - i. Pile installation record
 - ii. Reference subsurface information (borehole record)
 - iii. Pile location drawing
 - iv. Initial calibration check by the test computer unit

The report shall be signed and sealed by two Engineers of the testing company, one of whom shall be identified as MTO's designated contact.

903.07 CONSTRUCTION

903.07.02.07 Monitoring Driven Piles

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

Clause 903.07.02.07.03.01 of OPSS 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified resistance, the pile resistance shall be validated using high-strain dynamic testing.

The results of the high-strain dynamic tests shall be to the Contract Administrator who shall, in collaboration with the independent testing company, verify that the design ultimate resistance has been achieved.

903.07.02.07.04 Wave Equation Analysis

Clause 903.07.02.07.04 is deleted in its entirety and replaced with the following:

903.07.02.07.04 High-Strain Dynamic Testing

An independent testing company with no corporate affiliation with the Contractor shall be employed to perform the high-strain dynamic testing. The independent testing company shall be RAQs qualified (Specialty: Geotechnical (Structures and Embankments – Medium or High Complexity)).

High-strain dynamic tests shall be performed by an Engineer employed by the independent testing company. The Engineer shall have documented evidence of training and experience in foundation engineering and wave equation analyses, and have at least 5 years of experience in high-strain dynamic testing or a certificate of proficiency (intermediate level or better) in the pile driving analyzing (PDA) proficiency test.

High-strain dynamic testing shall be performed using the PDA, or approved equivalent, for the determination of pile capacity, establishment of pile installation criteria, assessment of pile integrity and monitoring of hammer/drive system performance, as specified in the Contract Documents. The method and equipment for testing and its reporting shall be according to ASTM D 4945.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing, and shall be submitted to the Contract Administrator for approval.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles; or as indicated on the Contract Documents.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out during the retapping of piles, as specified in the Retapping Tests on Piles clause. Restrike testing shall be performed on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles; or as specified in the Contract Documents. Restrike testing shall be carried out no sooner than 7 days after installation of the individual pile or at a time specified in the Contract Documents. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

903.10 BASIS OF PAYMENT

Section 903.10 of OPSS 903 is amended by the addition of the following subsection:

903.10.04 High-Strain Dynamic Testing, Deep Foundations - Item

Payment for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

Where additional tests are required, payment shall be made based on the ratio of the additional tests to number of tests specified in the High-Strain Dynamic Testing clause (i.e. tests at end of initial drive and at restrike), times the tender price for high-strain dynamic testing.



Appendix F

Primary Consolidation Settlement / Degree of Consolidation vs. Time Plots

FIGURE F-1

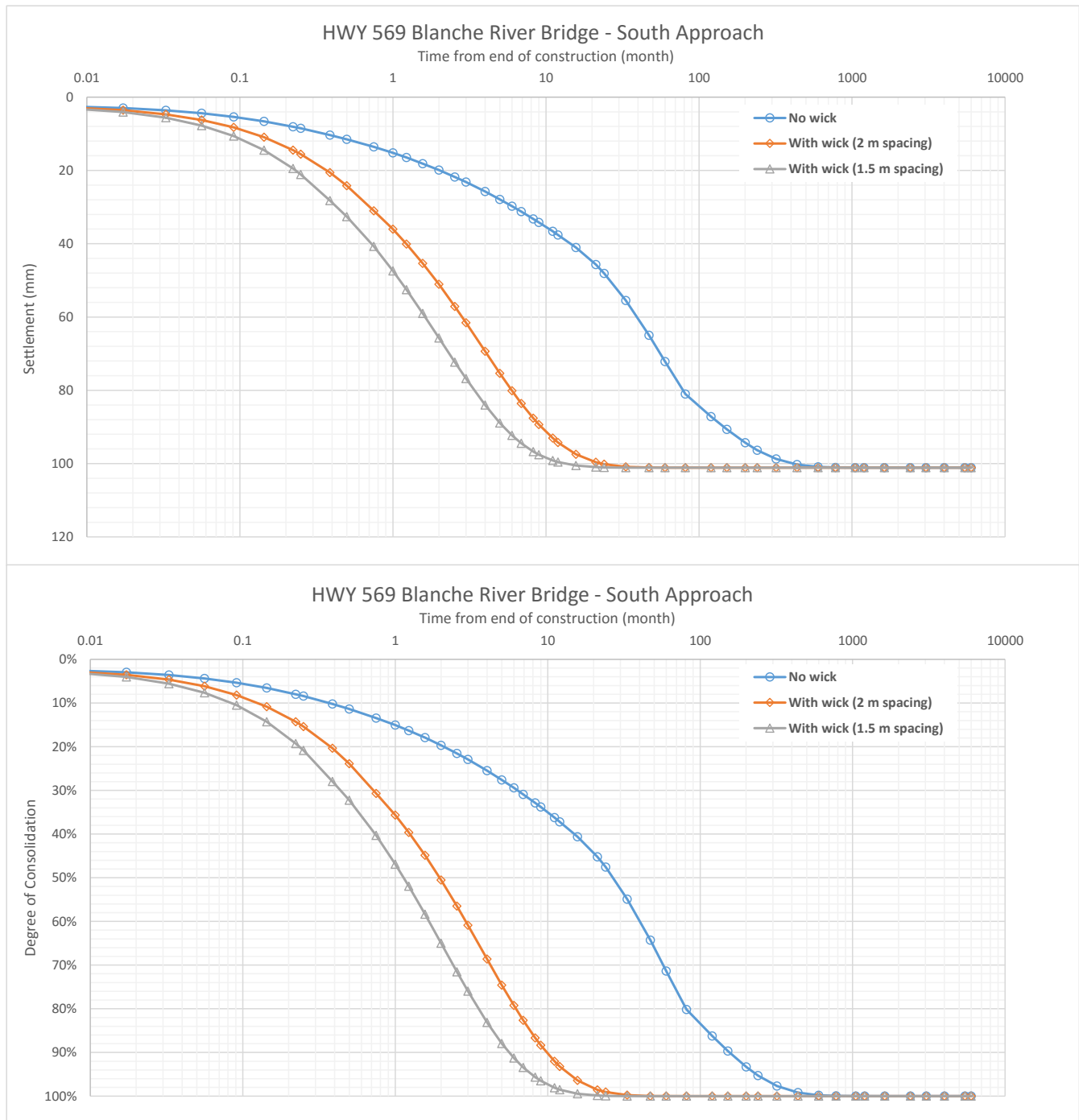
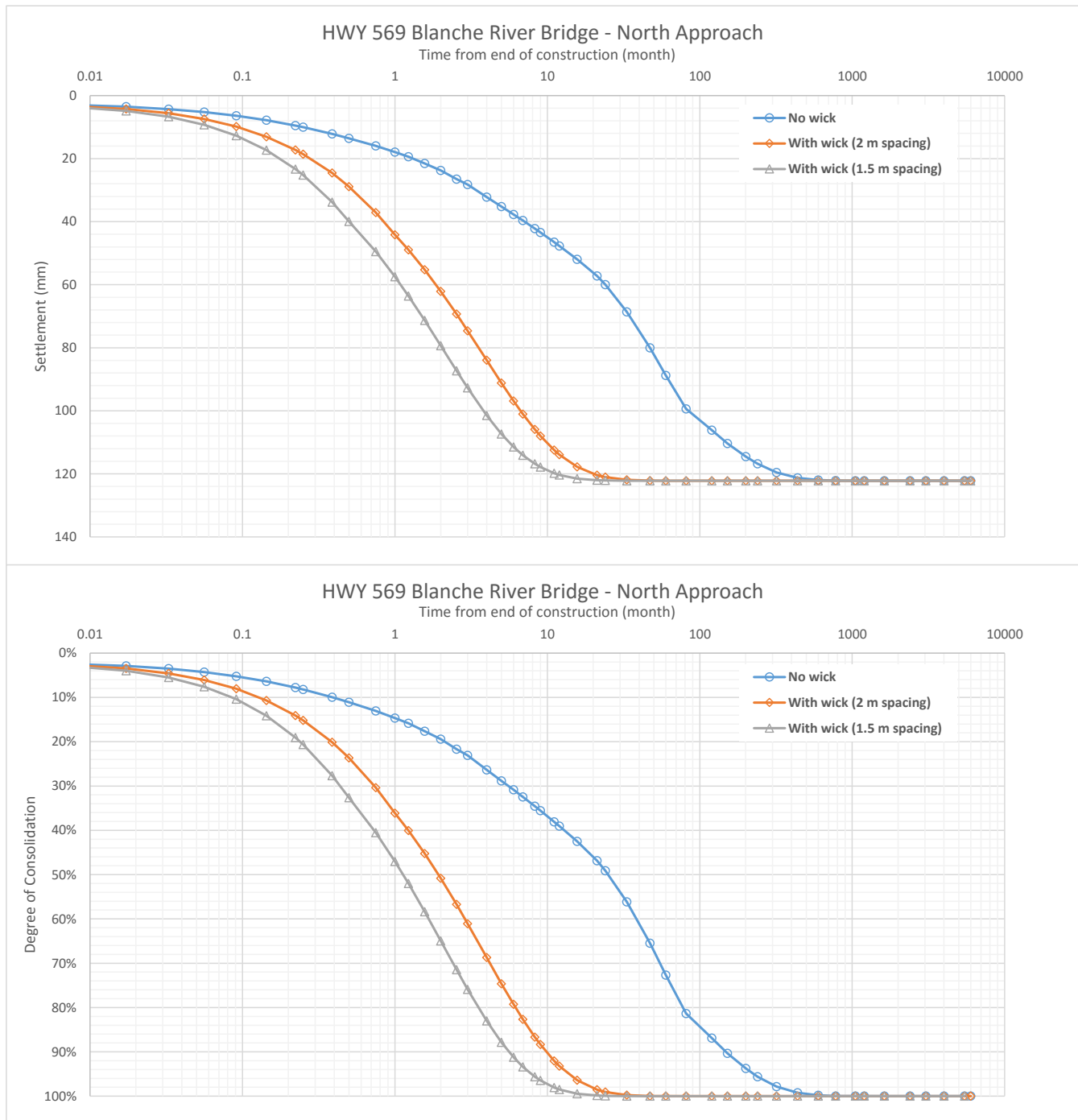


FIGURE F-2





Appendix G

Select Stability Analysis Runs

FIGURE G-1

**BLANCHE RIVER BRIDGE - SOUTH ABUTMENT
STATION 16+950
UNDRAINED CONDITION**

Name: New Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Clay Model: Undrained (Phi=0) Unit Weight: 18 kN/m³ Cohesion': 40 kPa

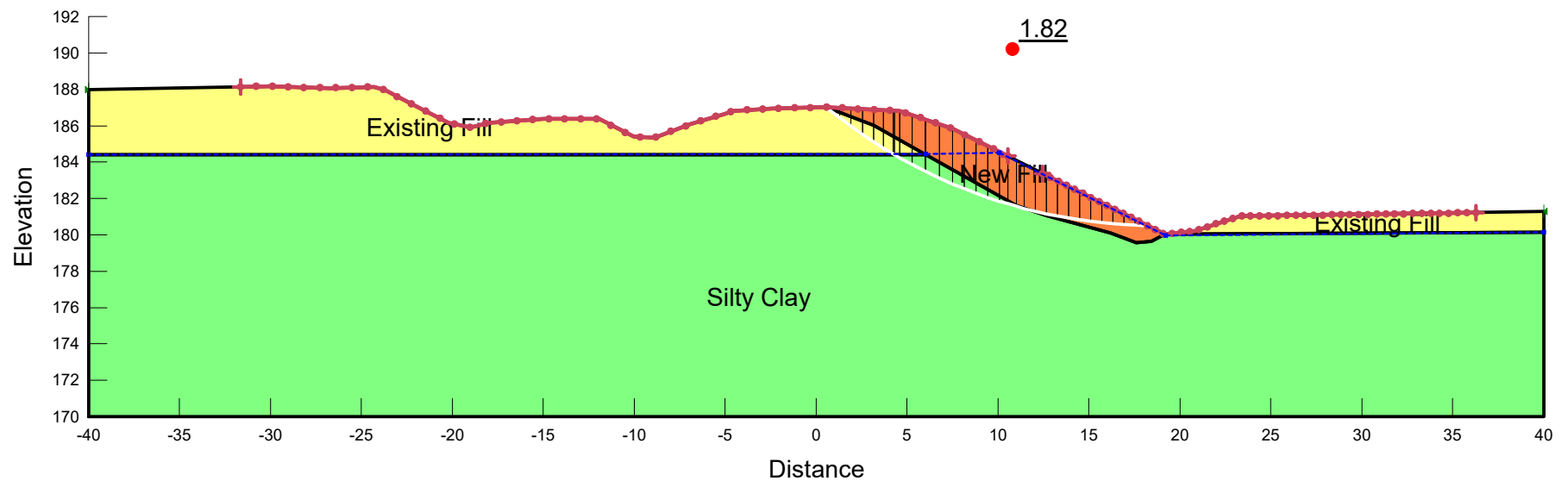


FIGURE G-2

**BLANCHE RIVER BRIDGE - SOUTH ABUTMENT
STATION 16+950
DRAINED CONDITION**

Name: New Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Clay Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 5 kPa Phi': 28 °

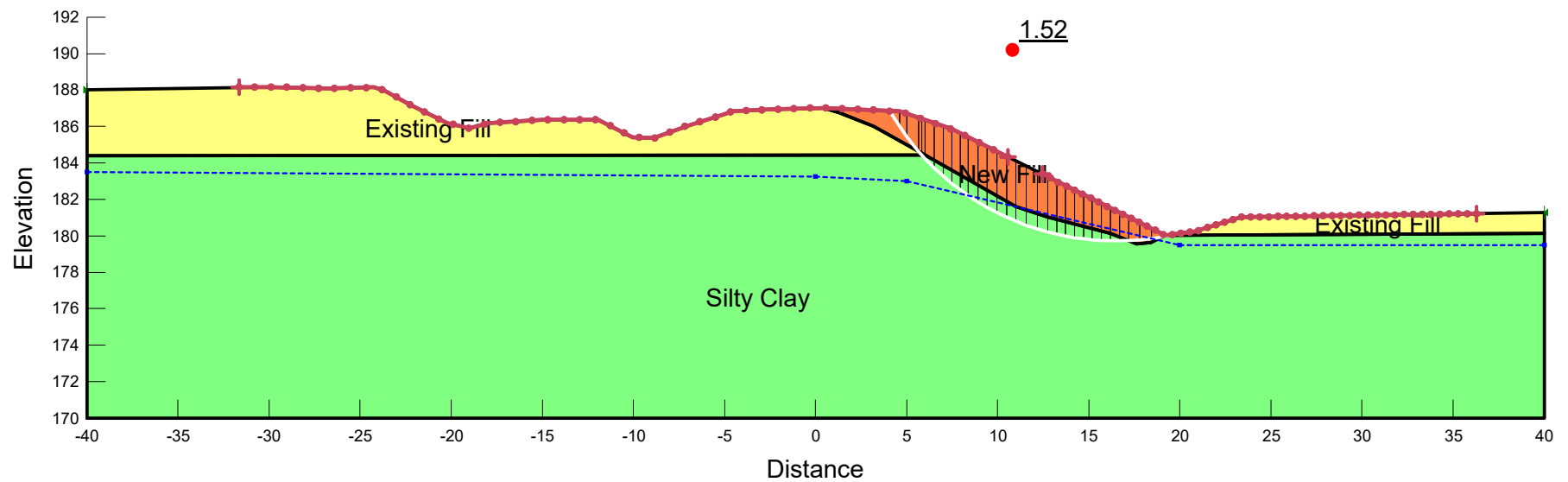


FIGURE G-3

**BLANCHE RIVER BRIDGE - NORTH ABUTMENT
STATION 17+090
UNDRAINED CONDITION**

Name: New Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Clay Model: Undrained (Phi=0) Unit Weight: 18 kN/m³ Cohesion': 40 kPa

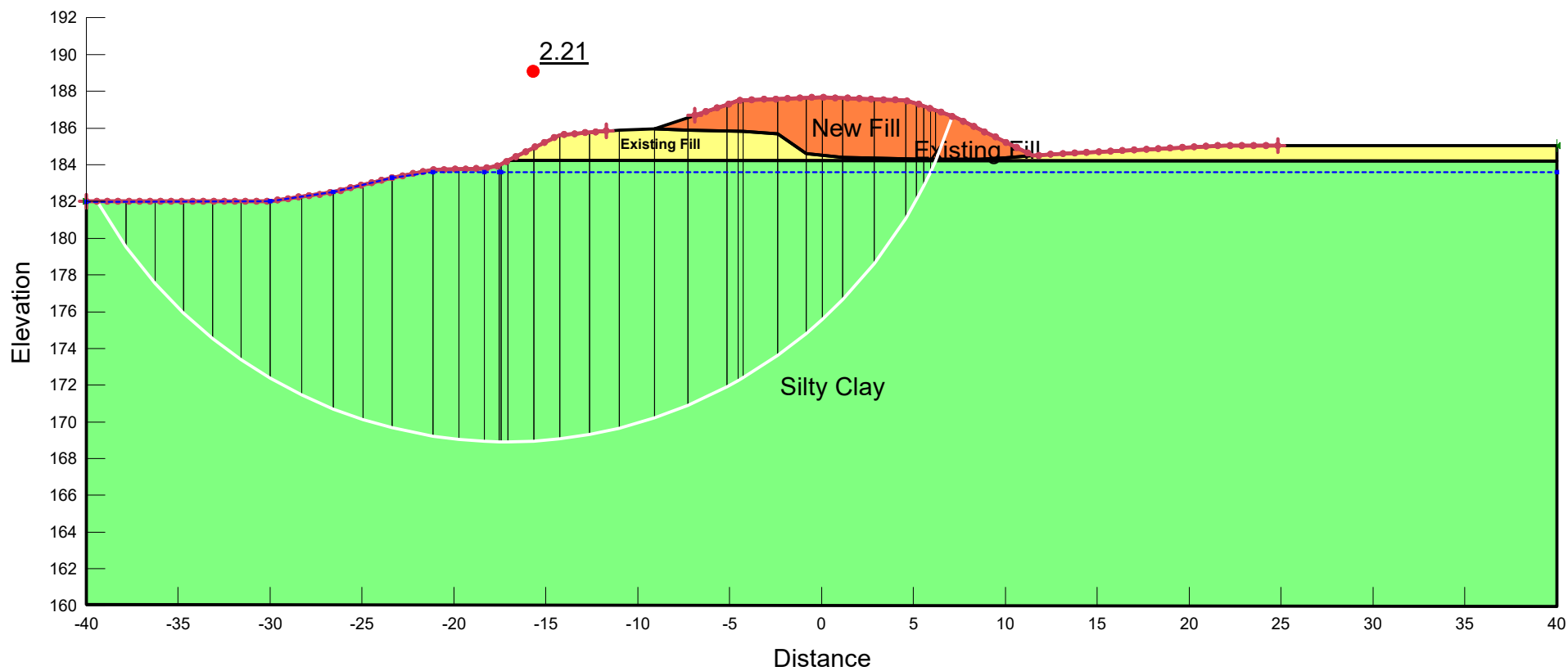


FIGURE G-4

**BLANCHE RIVER BRIDGE - NORTH ABUTMENT
STATION 17+090
DRAINED CONDITION**

Name: New Fill Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion': 0 kPa Phi': 35 °
Name: Existing Fill Model: Mohr-Coulomb Unit Weight: 19 kN/m³ Cohesion': 0 kPa Phi': 32 °
Name: Silty Clay Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion': 5 kPa Phi': 28 °

