



# FOUNDATION INVESTIGATION AND DESIGN REPORT

## PROPOSED IVANHOE RIVER BRIDGE REPLACEMENT, HIGHWAY 101 AND 7172 (FOLEYET)

SITE LOCATION (Long. -82.448293°, Lat. 48.247514°)

PLANMAC ENGINEERING INC.

GWP 5266-13-00

GEOCRES NO. 42B-013

WSP PROJECT NO.: 171-02344-00

JUNE 10, 2019



# FOUNDATION INVESTIGATION REPORT

## PROPOSED IVANHOE RIVER BRIDGE REPLACEMENT, HIGHWAY 101 AND 7172 (FOLEYET)

SITE LOCATION (Long. -82.448293°, Lat. 48.247514°)

PLANMAC ENGINEERING INC.

GWP 5266-13-00

GEOCRES NO. 42B-013

WSP PROJECT NO.: 171-02344-00

JUNE 10, 2019



# TABLE OF CONTENTS

1	INTRODUCTION .....	1
2	BACKGROUND INFORMATION .....	2
2.1	GEOLOGICAL SETTING .....	2
2.2	PREVIOUS GROUND INVESTIGATIONS .....	2
2.3	PROJECT BACKGROUND .....	3
2.4	SITE AND STRUCTURE DESCRIPTION .....	3
3	FIELD AND LABORATORY INVESTIGATIONS .....	5
3.1	FIELD INVESTIGATION .....	5
3.2	LABORATORY INVESTIGATIONS .....	7
3.3	GROUNDWATER INVESTIGATION .....	7
4	SUBSURFACE CONDITIONS .....	9
4.1	GENERAL .....	9
4.2	OVERVIEW .....	9
4.3	SUBSURFACE CONDITIONS .....	10
4.4	GROUNDWATER LEVEL OBSERVATIONS .....	16

## TABLES

TABLE 2 - 1 PILE DRIVING DETAILS AT EXISTING BRIDGE ABUTMENTS - TIMBER PILES.....	2
TABLE 3 - 1 SUMMARY OF EXPLORATORY HOLE DETAILS .....	6
TABLE 3 - 2 PIEZOMETER INSTALLATION DETAILS .....	7
TABLE 4- 1 GRAIN SIZE DISTRIBUTION SUMMARY - EMBANKMENT FILL.....	11
TABLE 4- 2 GRAIN SIZE DISTRIBUTION SUMMARY - SANDY SILT/SILTY SAND.....	11
TABLE 4- 3 GRAIN SIZE DISTRIBUTION SUMMARY - SILT .....	12
TABLE 4- 4 GRAIN SIZE DISTRIBUTION SUMMARY - SILTY SAND/SANDY SILT/SAND AND SILT (TILL).....	14
TABLE 4- 5 PROPERTIES OF BEDROCK.....	15
TABLE 4- 6 SUMMARY OF GROUNDWATER LEVEL OBSERVATIONS .....	16

## DRAWINGS

DRAWING 1	BOREHOLE LOCATION PLAN AND SUBSOIL PROFILE
DRAWING 2	EMBANKMENT CROSS-SECTIONS AT PROPOSED ABUTMENTS

## APPENDICES

APPENDIX A	RECORD OF BOREHOLE SHEETS
APPENDIX B	LABORATORY TEST RESULTS
APPENDIX C	SITE PHOTOGRAPHS
APPENDIX D	ROCK CORE PHOTOGRAPHS



# 1 INTRODUCTION

This work was carried out under Assignment No: 5016-E-0027 for the MTO Northeast Region. WSP Environment Toronto (“WSP”) was retained by Planmac Engineering Inc. (“TPM”), as foundation sub-consultants to provide foundation information and recommendations to enable detailed design for the proposed Bridge Replacement over the Ivanhoe River on Highway 101. This existing bridge is located in the Township of Foleyet in the County of the New Liskeard Area. The abutments at the Foleyet end and Timmins end will be referred to as the South Abutment and the North Abutment respectively following geographic north. The scope of work under the original assignment was significantly revised during the course of the project to achieve beneficial project gains and related foundation impacts are summarized under Project Background in Section 2.3. The alignment chosen for the detailed investigation lies to the west of the existing 3-span bridge with a centre line offset of 10 m and the intention is subject proving reliable foundation conditions to build a single span bridge of approximate span of 45 m over the river.

The purpose of the Geotechnical Investigation was to determine the sub-surface conditions and groundwater observations at the site by means of boreholes, field and laboratory tests. Based on the information obtained, the engineering characteristics of the subsurface soils were assessed and site conditions described to develop geotechnical recommendations to address the foundation scope.

Part A of this report presents factual information concerning the subsurface conditions based on all of the subsurface information at hand and is followed by Part B wherein engineering discussion and foundation recommendations are made for the design and construction of the proposed Ivanhoe River Bridge Replacement.

This report is based on the revised General Arrangement (GA) Drawings provided to WSP by the TPM on Dec 8, 2018.

## 2 BACKGROUND INFORMATION

### 2.1 GEOLOGICAL SETTING

Site geology as per OGS Geology Map No: 2671 on the Quaternary Geology of the Foleyet Area (1:50,000 scale) indicates the surficial geology to be recent alluvial deposits comprising sand, silt and organic rich deposits around the floodplain of the Ivanhoe River. According to the OGS Northern Ontario Engineering Geology Terrain Study 75 on the Foleyet Area (1980), the alluvial plains are usually composed of micaceous silt and local patches of peat. Further, the morainal terrain in the Foleyet area was found to be non-plastic with nearly equal proportions of sand and silt and commonly bouldery. Ontario, Division of Mines MAP 2221 on Bedrock Geology of Chapleau-Foleyete (Geological Compilation Series) indicates the bedrock in the locality of Foleyet to be Metasedimentary, granitic gneiss rock.

### 2.2 PREVIOUS GEOTECHNICAL INFORMATION

No foundation report for the existing bridge was available. However, pile driving records for the existing bridge (3-span) in 1960 (#59-F-242-C; W.P. 263-59 Foleyet – Chapleau Road) are available although the legibility of the records is poor. Timber piles were used for the abutments and steel H-piles were used for the two river piers. Some details are given for the abutment piles. Table 2-1 has been compiled based on the decipherable details.

**Table 2 - 1 Pile Driving Details at Existing Bridge Abutments – Timber Piles**

Driving Location	Pile Penetration (m)	Drop of Piling Hammer (m); Set (mm/Blow)	Interpretation by Piling Foreman
Abutment at Foleyet End (South)	0 to 4.9	2.4; 600	
	4.9 to 7.0	3.05; 150	
	Final Strike	4.6; 7.6	Large boulders or bedrock
Abutment at Timmins End (North)	0 to 4.3	2.4; 4.3 m	Muskeg
	4.3 to 6.7	2.4; 300	
	6.7 to 8.5	2.4; 75	

	8.5 to 8.84	NA; 12	
	Final Strike	4.6; 3	Large boulders or bedrock

Based on the intercepted ground conditions during pile driving, the piling notes mention that better ground conditions were intercepted at the north abutment. Subsequent to the original construction in 1960, it is also noted that superstructure rehabilitation was done in 1993. Emergency rehabilitation due to severely corroded top portion of pier piles was completed in 2014.

---

## 2.3 PROJECT BACKGROUND

The RFQ under assignment no. 5016-E-0027 stipulated requirements for foundation services for the replacement of the existing Ivanhoe bridge along the same alignment with a temporary detour bridge adjacent to the existing and the replacement of two centreline culverts at stations 17+534 and 17+755. Improvements to the existing alignment were put forward by the TPM to the MTO Northeast Region. This implied amongst other things in so far as foundation issues were concerned the elimination of the replacement of the two centreline culverts mentioned above and consideration of a new bridge alignment to the west of the existing bridge. In view of the pile driving observations noted during the original construction in 1960 where surficial peat (muskeg) was suspected (as discussed in Section 2.2) and the general geology of the wider area which is known to have deposits of peat, concern was expressed by the Region that any new alignment could be confronted with compressible ground issues.

To allay these concerns WSP proposed a preliminary foundation investigation to explore the presence of compressible deposits along the proposed alignment which also included a borehole each at the existing approaches. This investigation did not intercept any peat or soft clays in any of the explored locations. Following these findings and structural submissions by the TPM, the Region confirmed the adoption of the new alignment for the replacement bridge. The decision to adopt a single span bridge which will have significant cost and time savings was deferred till the completion of a detailed foundation investigation to prove competent abutment foundation conditions. If favourable findings can be established to the satisfaction of the Region, then the need for expensive river piers can be eliminated and the need for a detour can be dispensed with the existing bridge to be used during construction of the replacement.

These issues are discussed in the present report.

---

## 2.4 SITE AND STRUCTURE DESCRIPTION

The key plan of the site is shown on **Drawing No.1**. Highway 101 is a two-lane undivided highway with paved shoulders. Highway 101 runs approximately in a north-south direction while the Ivanhoe river has an approximate east-west orientation with respect to the geographic north. The existing bridge is 45.4 m long with 3 spans (each span approximately 15 m long). The surrounding area of the bridge site is generally well-vegetated with tall trees and grassy landscape. All site and structure related photographs are in Appendix C.

The Ivanhoe River flows in an approximate east-west direction (see Photo C1-1) at the subject bridge crossing. Differential settlements were observed at the edge of the approach slabs with the approach embankments (see Photo C1-2 to C1-3). There are no visible signs of erosion scars at the riverbanks at the proposed bridge alignment (see Photos C1-4 and C1-5) and the banks were grassed.

The existing embankment heights on the centreline at the abutments are 2.6 m on the south (Sta 12+839) and 2.3 m (Sta 12+900), with side slopes 2H:1V.

# 3 FIELD AND LABORATORY INVESTIGATIONS

---

## 3.1 FIELD INVESTIGATION

The reconnaissance observations about the nature of terrain and access constraints for conventional drilling gear were carefully considered in planning the field investigation program. Field investigations commenced initially with an exploration phase consisting of six (6) land boreholes. This phase commenced in October 2017 and after the completion of four boreholes (BH 17-1, BH 17-2, BH 17-5 and BH 17-6), fieldwork was disrupted due to winter weather. The two remaining boreholes (BH 17-3 and BH 17-4) were completed in July 2018. Bedrock was intercepted in BH 17-4 with 3 m coring. Based on the findings, MTO approval was given to extend the drilling programme with three additional boreholes, BH 18-1, BH 18-2 and BH 17-5A to prove bedrock with BH 18-1 and BH 17-5A to be advanced to 6 m into bedrock. This was undertaken in September 2018. One of the principal aims of the latter investigation was to prove bedrock. As BH 17-5A was in close proximity to BH 17-5, it was advanced with spoon sampling only undertaken beyond the explored depth of BH 17-5, since the main objective of this borehole was to prove bedrock. The exploratory hole locations are shown on **Drawing 1** following the text of the FIR.

The traffic set-up indicated a single lane closure with two flag men on opposite ends of the work closure in accordance with MTO Book 7 (Photos C2-1 and C2-2).

Rig access to boreholes at the south abutment and approach of the proposed bridge alignment can be seen in Photo C2-3. The locations of the three boreholes (BH 17-3, BH 17-4 and BH 18-1) are shown in Photos C2-4 and C2-5. Further, access to the proposed bridge alignment boreholes at the Timmins end required the construction of a temporary access ramp made of wooden logs (Photos C2-6 and C2-7). The locations of these boreholes (BH 17-5, BH 17-5A, BH 17-6 and BH 18-2) are clearly indicated in Photos C2-8 and C2-9. The remaining boreholes (BH17-1 and BH17-2) were located on the approaches of the existing bridge (Photos C2-10 and C2-11).

Based on the site reconnaissance, supplemented by provision of temporary access where required, a conventional drilling approach was deployed, i.e. the use of a track mounted CME 55/850 type of machine.

The Fieldwork was carried out under full-time supervision of WSP technical staff who directed the exploration and sampling operation, logged borehole data in accordance with MTO Soils Classification System and took custody of soil/rock samples retrieved for subsequent laboratory testing and identification. Soil/rock samples were visually classified in the field and later re-evaluated by an engineer. The recovered soil/rock samples were placed in labelled moisture-proof bags/rock core boxes and returned to WSP's laboratory for further assessment.

**Table 3-1** presents the exploratory hole details of the WSP foundation investigation program.

**Table 3 - 1 Summary of Exploratory Hole Details**

<b>Borehole No.</b>	<b>Co-Ordinates: Eastings/ Northings (m)</b>	<b>Ground El. (m)</b>	<b>Explored Depth (m)</b>	<b>Drilling Methodology/Remarks</b>
BH 17 - 1	E420051.5 N5346609.5	322.5	21.3	Wash Boring/coring
BH 17 - 2	E420052.0 N5346547.0	322.6	12.2	Hollow Stem/Wash Boring/Coring
BH 17 - 3	E420039.5 N5346539.3	321.3	11.6	Hollow Stem Auger/DCPT
BH 17 - 4	E4200037.7 N5346550.4	321.3	18.8	Wash Boring/NQ Rock Coring to 3 m
BH 17 - 5	E420033.0 N5346598.5	321.6	17.9	Wash Boring/Coring
BH 17 - 5A	E420032.5 N5346595.9	321.6	23.8	Wash Boring/NQ Rock Coring to 6 m
BH 17 - 6	E420031.3 N5346613.1	321.7	7.6	Hollow Stem Auger/DCPT
BH 18 - 1	E420036.3 N5346553.1	321.3	21.9	Wash Boring/NQ Rock Coring to 6 m
BH 18 - 2	E420028.3 N5346593.6	321.6	20.4	Wash Boring/NQ Rock Coring to 3 m

**Notes\*:**

- 1) Locates done by G-TEL with participating companies such as Hydro One & Bell Canada
- 2) The spacing and quantity of boreholes generally conform to RFQ requirements;
- 3) Type of drilling rig Used: Track mounted - CME 55/850 rig (used by both Marathon and Landcore);
- 4) Co-ordinates: based on MTM NAD 83 Zone 13 coordinates; Terminology of directions, e.g., Reference to North is geographic;
- 5) Names of Drilling Company: Marathon Drilling, Ottawa, Ontario (for drilling of BH17-1, BH17-2, BH17-5 & BH17-6) and Landcore, Chelmsford, Ontario (for drilling of BH17-3, BH17-4, BH17-5A, BH18-1 & BH18-2);
- 6) Drilling Supervision: by WSP staff from Toronto office;
- 7) Traffic control to Book 7 by Demora Construction Services Inc. New Liskead, Ontario and Caron Equipment Inc., Timmins, Ontario;

8) Borehole Survey by Talbot Surveys Inc.

Samples were retrieved at regular intervals with a 50 mm Outer Diameter (O.D.), split-barrel sampler driven with a hammer weighing 624 N and dropping 760 mm in accordance with the Standard Penetration Test (ASTM D 1586) method. This sampling method recovers samples from the soil strata, and the number of blows required to drive the sampler 300 mm depth into the undisturbed soil (SPT 'N'-values) gives an indication of the compactness condition or consistency of the sampled soil material. The SPT 'N' values are indicated on the Record of Borehole Sheets (Refer to Appendix A). At the proposed north approach, in BH 17-6, hollow-stem drilling was followed by DCPT probing due to possible cobble/boulder obstructions and at the proposed south approach, in BH 17-3, beyond a depth of 9.8 m, DCPT probing was carried out till probe refusal.

Rock coring was carried out at selected boreholes as indicated in Table 3-1 using NQ core barrels to approximate depths of 3 m and 6 m below auger/casing refusal depths (See Table 3-1) and occasional rock coring was carried out through boulder/cobble material to overcome drilling obstructions.

## 3.2 LABORATORY INVESTIGATIONS

Visual examination and classification were undertaken on the soil/rock samples returned to WSP laboratory. A routine laboratory testing program consisting of natural water content tests, grain size analysis, hydrometer testing and Atterberg Limit Test was carried out on selected representative soil samples.

Point load tests were conducted generally on every core run. UCS testing on an intact rock specimen from each abutment was carried out at Golder Associates, Mississauga Laboratory. The results of the laboratory tests are summarized on the appropriate Record of Borehole Sheets in Appendix A and compiled test results are given in Appendix B.

## 3.3 GROUNDWATER INVESTIGATION

Groundwater conditions in the boreholes were observed during and on completion of drilling in the open boreholes. A 19 mm standpipe piezometer was installed in Borehole BH 17-5 and a 50 mm monitoring well installed in BH18-1 upon completion of drilling to enable long-term groundwater level monitoring and water levels read subsequently. The rest of the boreholes were grouted (decommissioned) using a cement/bentonite mixture as per MTO procedures. As part of the construction, the two installed piezometers need to be decommissioned in accordance with Ontario Regulation 903 (amended to Ontario Regulation 372/07).

Table 3-2 provides information about the piezometers installed for this investigation.

**Table 3 - 2 Piezometer Installation Details**

BH No.	Ground Surface Elevation (m)	Borehole Bottom		Well Screen Interval Depth, m		Well Screen Interval Elevation, m		Remarks
		Depth (m)	Elevation (m)	From	To	From	To	

BH 17-5	321.6	17.9	303.7	7.6	10.7	314.0	310.9	Proposed north abutment
BH 18-1	321.3	21.9	299.4	19.2	21.9	302.1	299.4	Proposed south abutment



# 4 SUBSURFACE CONDITIONS

---

## 4.1 GENERAL

The subsurface conditions encountered at the bridge location are described in the following sections.

**Drawing 1** shows a borehole/DCPT location plan with a longitudinal profile (projected along the centreline of the proposed Hwy 101 at the proposed bridge crossing). The inferred stratigraphic profile at the bridge location is based on the borehole data. The soil descriptions are based on visual and tactile observations and complemented by the results of field and laboratory soil test results. **Drawing 2** shows embankment/stratigraphic cross-sections close to the proposed bridge abutments.

For purposes of soil description, the MTO Soil classification manual was generally followed. The strata boundaries shown on the subsurface profile must not be interpreted as exact planes of geological change but rather as inferred transitions from one soil type to another since they are based on non-continuous sampling information at discrete borehole locations. It should be noted that the subsurface conditions and the topsoil thicknesses encountered might vary in between and beyond the borehole locations and the topsoil thicknesses could vary especially in depressed areas and near watercourses. All topsoil and pavement fill thicknesses reported should not be relied upon for quantity estimation as they may vary beyond the borehole locations. Unless otherwise stated, all SPT 'N' values quoted are for 300 mm of penetration.

Groundwater observations on completion of drilling will not be reliable wherever wash boring technique was followed and hence not reported. However, observed borehole stability conditions upon completion of boreholes are described. All groundwater levels observed in the exploratory holes are subject to seasonal fluctuations and variations due to precipitation events.

An overview of subsurface conditions is described below. All depths quoted are below the existing ground surface.

---

## 4.2 OVERVIEW

The native stratigraphy (below pavement structure/topsoil/embankment fill as applicable) predominantly consisted of a silt deposit underlain by cohesionless till intercepted in all the boreholes overlying bedrock. The intercepted till was very bouldery and required intermittent coring in most of the boreholes.

The intercepted silt was dilatant and of thickness ranging from 3.2 m to 7.1 m with measured moisture content varying from 10% to 52% with an average of 24% and was found to be non-plastic. Grain size information revealed a silt content between 71% and 95 %. Accordingly, based on the MTO Frost Susceptibility Chart and the MTO Erosion and Sediment Control (ESC) guideline, the silt can be ranked as of "High" frost susceptibility and "Most Erodible" on the erodibility classification, respectively. The deposit had an average SPT 'N' Blow count of 9 based on 42 SPT probes and hence can be typically described as of loose compactness. Discontinuous sandy interbeds were intercepted in a few boreholes (BH 17-5, BH 18-1 and BH 18-2). In borehole BH 17-5, a few SPT spoons witnessed trace to some organics with other isolated occurrences of traces of organics; a flowing sand condition was also observed within a sandy silt interbed in this borehole. The compactness of the interbeds can be described as very loose.

The dominant native soil deposit underlying the silt was a cohesionless till comprising sandy silt/silty sand / sand and silt. The intercepted thickness of the deposit ranged from 1.5 m to 14.8 m with an average of 5.7 m. It was predominantly sand

and silt with numerous boulder beds. More than 70% of the SPT probes indicated the deposit to be dense or better; however, the bouldery nature will likely have influenced the recorded blow counts.

The intercepted quaternary geology closely resembles the geology reported in the literature (See Section 2.1).

Gneissic bedrock was proven in four boreholes at the proposed abutments, two boreholes at each abutment. Two of the boreholes proved bedrock each with 6 m coring, one at each abutment while in the other two boreholes, bedrock was proven with 3 m coring. The elevations of top of bedrock at the proposed south abutment were 305.8 m and 305.5 m in Boreholes BH 17-4 and BH 18-1 respectively. The corresponding elevations at the proposed north abutment were 303.9 m and 304.6 m in Boreholes BH 17-5A and BH 18-2 respectively. The average RQD was 70% (range 27% to 98%) and based on Canadian Foundation Engineering Manual (CFEM) Table 3.10, it is indicative of a rock mass of fair to good quality. The fracture index was predominantly 2 per 0.3 m or less. The UCS values based on average point load strength index test values were 109 MPa and 71 MPa for axial and diametral loading modes respectively. The weathering index was W2 to W1 throughout, i.e. Fresh to Slightly Weathered. Two UCS tests on intact rock specimens yielded uniaxial compressive strengths of 146 MPa and 39 MPa. Based on the intact rock strengths, following CFEM Table 3.5, the rock strength can be described as typically strong, although the possibility of very strong bands (e.g. UCS of 146 MPa) of rock cannot be ruled out.

Shallow groundwater levels were observed in piezometers installed one at each proposed abutment.

---

## 4.3 SUBSURFACE CONDITIONS

### 4.3.1 TOPSOIL

A veneer of topsoil was intercepted at the ground surface in BH 17-3, BH 17-4, BH 17-5 and BH 17-6 with thickness of 50 mm and in BH 18-1 and BH 18-2 with thickness of 100 mm. The measured moisture contents ranged from 7% to 23%.

### 4.3.2 ASPHALT

The asphalt thickness intercepted in BH 17-1 and BH 17-2 was 140 mm on the existing bridge approaches.

### 4.3.3 EMBANKMENT FILL

Embankment fill comprising sand and gravel /gravelly sand was intercepted in BH 17-1 and BH 17-2 with thickness of 2.2 m and 2.5 m respectively. The grain size distribution of a selected sample from the fill was determined in the laboratory and gave the grain size distribution shown in **Table 4-1**.

**Table 4- 1 GRAIN SIZE DISTRIBUTION SUMMARY - EMBANKEMENT FILL**

Samples Tested	Size Fraction (%)			Remarks
	Gravel	Sand	Fines (Silt & Clay)	
BH17-2/SS4	29	55	(16)	Shown in Figure B-1, <b>Appendix B</b> Summarized on the relevant Record of Borehole Sheet

Based on the grain size distribution, the tested sample can be described as gravelly sand and can be classified as SM.

Measured moisture contents of the spoon samples ranged from 2% to 11% indicative of a moist condition. SPT 'N' values recorded in the range 13 to 32 indicate the fill to be of compact to dense, relative density.

#### 4.3.4 SILTY SAND/SAND

Silty sand/sand deposits were intercepted underlying topsoil in BH 17-3, BH 17-4, BH 18-1 and BH 18-2, but in BH 17-6, it was underlying a silt deposit (Section 4.3.5), with thickness varying from 0.5 m to 2.1 m.

The grain size distribution of selected samples from this fill were determined in the laboratory and gave the grain size distribution shown in **Table 4-2**.

**Table 4- 2 GRAIN SIZE DISTRIBUTION SUMMARY - SANDY SILT/SILTY SAND**

Samples Tested	Size Fraction (%)				Remarks
	Gravel	Sand	Silt	Clay	
BH17-3/SS2 BH17-6/SS5	0-1	77 - 87	20	3	Shown in Figure B-2, <b>Appendix B</b>  Summarized on the relevant Record of Borehole Sheets
			12		

Based on the visual, tactile and grain size information, the deposit can be described as sand/silty sand and essentially of a cohesionless nature.

The moisture contents of the spoon samples ranged from 13% to 28% and the material was found to be wet. The SPT 'N' blow counts ranged from 3 to 13 and this is indicative of the deposit being of very loose to compact but typically very loose. A flowing sand condition was observed in BH 17-6.

#### 4.3.5 SILT

A non-plastic silt deposit was intercepted in all the boreholes with thickness ranging from 3.1 m (BH 17-6) to 7.1 m (BH 17-5) and was observed to be dilatant. The intercepted top elevation of the deposit ranged from El. 321.6 m (BH 17-6) to 319.2 m (BH 17-3). Sandy silt/silty sand interbeds were intercepted in a few boreholes (See Table 4-3). In borehole BH 17-5, a few SPT spoons witnessed trace to some organics and there were other isolated occurrences of traces of organics. The grain size distribution of thirteen (13) samples from the native silt deposit (including the interbeds) were determined in the laboratory and gave the grain size distribution shown in **Table 4-3**.

**Table 4- 3 GRAIN SIZE DISTRIBUTION SUMMARY - SILT**

Samples Tested	Size Fraction (%)				Remarks
	Gravel	Sand	Silt	Clay	
BH17-1/SS5 BH17-2/SS8 BH17-3/SS7 BH17-4/SS3 BH17-4/SS4 BH17-5/SS3 BH17-5/SS5 BH17-6/SS3 BH18-2/SS4 BH18-2/SS9	0 - 9	1 - 21	71 - 95	4 - 9	Shown in Figure B-3, <b>Appendix B</b>  Summarized on the relevant Record of Borehole Sheets
<b>Interbeds</b>	<b>Gravel</b>	<b>Sand</b>	<b>Silt</b>	<b>Clay</b>	

BH17-5/SS5			63 – 69	5 - 7	
BH18-1/SS4					
BH18-2/SS6	0 - 2	24 - 54	44		

Based on the visual, tactile and grain size information, the silt deposit can be described as typically silt with trace to some sand traces of gravel and clay of a cohesionless nature. The interbeds can be described as sandy silt/silty sand and also of a cohesionless nature.

Accordingly, based on the MTO frost susceptibility chart and the MTO ESC guideline the silt can be ranked as of High frost susceptibility and Most Erodible on the erodibility classification, respectively. The measured moisture contents varied from 10% to 52% with an average of 24%. Moisture was measured on 48 spoon samples (within the silt and the interbeds) and eight samples recorded moisture over 30% indicative of a wet condition.

The silt deposit had an average SPT ‘N’ Blow Count of 9 based on 42 SPT probes and hence can be typically described as of loose compactness.

The interbeds had an average SPT ‘N’ Blow Count of 2 and thus can be described as very loose. Flowing sand conditions were observed in the sandy silt interbed in BH 17-5.

#### 4.3.6 COHESIONLESS TILL (SANDY SILT/SILTY SAND/SAND AND SILT)

The dominant native soil deposit underlying the silt intercepted in all the boreholes was a cohesionless till comprising sandy silt/silty sand / sand and silt and was bouldery. Intermittent coring was required in many boreholes to advance through boulders. The intercepted thickness of the deposit ranged from 2.1 m (BH 17-3) to 14.2 m (BH 17-1) with an average of 8.2 m. The intercepted top elevation of the deposit varied from 315.4 m (BH 17-2) to 313.7 m (BH 17-3, BH 17-4 and BH 18-1).

The grain size distribution of twelve samples from the till deposit was determined in the laboratory and gave the grain size distribution shown in **Table 4-4**.

**Table 4- 4 GRAIN SIZE DISTRIBUTION SUMMARY - SILTY SAND/SANDY SILT/SAND AND SILT (TILL)**

Sample Tested	Size Fraction (%)				Remarks
	Gravel	Sand	Silt	Clay	
BH17-1/SS12	3 - 16	27 - 50	35 - 64	2 - 12	Shown in Figure B-4, <b>Appendix B</b> Summarized on the relevant Record of Borehole Sheets
BH17-1/SS16					
BH17-2/SS12					
BH17-3/SS10					
BH17-4/SS10					
BH17-4/SS11					
BH17-4/SS14					
BH17-5/SS9					
BH17-5/SS14					
BH18-1/SS10					
BH18-1/SS17					
BH18-2/SS20					

Based on the visual, tactile and grain size information, the deposit can be described as predominantly sand and silt (with some to traces of gravel and clay) with numerous boulder beds.

The measured moisture contents ranged from 8% to 38% with an average of 11% based on 34 measurements. Flowing sand conditions were observed in BH 17-2 and BH 17-4. More than 70% of the SPT probes indicated the deposit to be dense or better; however, the bouldery nature will likely would have influenced the recorded blow counts.

#### 4.3.7 BEDROCK

Bedrock was proven in four boreholes, two at each proposed abutment. The elevations of top of bedrock at the proposed south abutment were 305.8 m and 305.5 m in Boreholes BH 17-4 and BH 18-1 respectively. The corresponding elevations at the proposed north abutment were 303.9 m and 304.6 m in Boreholes BH 17-5A and BH 18-2 respectively. The bedrock was contacted at depths ranging from 15.5 m (BH 17-4) to 17.7 m (BH 17-5A). Based on visual examination, the cored bedrock appears to be metasedimentary, granitic gneiss as was also indicated on MAP 2221 (bedrock geology of Chapleau-Foleyet).

Bedrock was cored to 3.3 m length in BH 17-4, 3.4 m length in BH 18-2, while BH 17-5A and BH 18-1 were cored to 6.1 m and 6.0 m length respectively. Photos of the rock cores are attached in **Appendix D**. The bedrock properties are summarized in **Table 4-5**.

**Table 4- 5 PROPERTIES OF BEDROCK**

BH No.	Termination Depth of Bedrock Below Ground Surface (m)	Top Elevation/(Depth) of Bedrock (m)	Cored Length (m)	RQD% (RQD In Core Runs)	Uniaxial Compressive Strength Based on Point Load Test, MPa (Run)  A – Axial D - Diametral	UCS* Test, MPa (Run)
BH 17-4	18.8	305.8/(15.5)	3.3	52/72/76	Run 1, 2 & 3 42.3 – 94.0 (D) 102.6 – 115.0 (A)	
BH 17-5A	23.8	303.9/(17.7)	6.1	60/27/80/98/98/88	Run 1, 2, 3, 4, 5 and 6 67.6 – 104.0 (D) 95.9 – 147.8 (A)	146.1 (Core Run 1)
BH 18-1	21.9	305.5/(15.9)	6.0	50/68/69/86/48	Run 1, 2, 3, 4 and 5 37.9 – 66.7 (D) 87.1 – 112.1 (A)	39.1 (Core Run 4)
BH 18-2	20.4	304.6/(17.0)	3.4	50/80/96	Run 1 and 2 57.2 – 82.1 (D) 85.5 – 150.5 (A)	

\*UCS – Uniaxial Compressive Strength Test

The average RQD was 70% (range 27% to 98% based on 17 measurements) and based on Canadian Foundation Engineering Manual (CFEM) Table 3.10, it is indicative of a rock mass of fair to good quality. The fracture index was predominantly 2

per 0.3 m or less. The average point load strength index based UCS values were 109 MPa and 71 MPa for axial and diametral loading modes respectively based on 16 tests. The weathering index was W2 to W1 throughout, i.e. Fresh to Slightly Weathered. Two UCS tests on intact rock specimens yielded uniaxial compressive strengths of 146 MPa and 39 MPa. Based on the intact rock strengths, following CFEM Table 3.5, the rock strength can be described as typically strong, although the possibility of very strong bands (e.g. UCS of 146 MPa) of rock cannot be ruled out. The higher UCS of 146.1 MPa was closer to the point load based **axial** UCS of 147.8 MPa for the same core run in BH 17-5A and the lower UCS of 39.1 MPa was closer to the point load based **diametral** UCS of 37.9 MPa for the same core run in BH 18-1.

## 4.4 GROUNDWATER LEVEL OBSERVATIONS

Piezometers were installed in Boreholes BH 17-5 (standpipe) & BH18-1 (monitoring well) for long-term groundwater monitoring. The screen for the piezometer was installed spanning the silty sand/sandy silt till in BH 17-5 and in bedrock in BH 18-1. On-completion groundwater level observed in other boreholes are not considered reliable due to wash boring method being used and are not reported. However, observed borehole stability conditions upon completion of boreholes are described. It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to major weather events. Findings are summarized in **Table 4-6**.

**TABLE 4- 6 SUMMARY OF GROUNDWATER LEVEL OBSERVATIONS**

BH No.	Existing Ground Elevation	Date of Measurement	Groundwater Level- Depth (m)	Groundwater Level - Elevation (m)	Notes
BH 17-1	322.5				Wash Boring Borehole cave-in at 1.7 m below GL
BH 17-2	322.6				Hollow Stem/Wash Boring Borehole cave-in at 1.7 m below GL
BH 17-3	321.3	July 20, 2018 (upon completion)	4.6	316.7	Hollow Stem Auger used; Borehole cave-in at 4.6 m below GL
BH 17-4	321.3				Wash Boring Borehole cave-in at 9.9 m below GL



BH 17-5 (19 mm dia. Piezometer)	321.6	October 26, 2017 (upon completion)	1.5	320.1	Hollow Stem Auger used; Unstabilized water level reported; Borehole cave-in at 17.4 m below GL
		December 18, 2017	2.3	319.3	
		September 27, 2018	1.8	319.2	
		November 18, 2018	1.6	320	
		January 19, 2019	2.1	319.5	
BH 17-5A	321.6				Wash Boring Borehole cave-in at 9.2 m below GL
BH 17-6	321.7	October 26, 2017 (upon completion)	1.8	319.9	Hollow Stem Auger Borehole cave-in at 3.0 m below GL
BH 18-2	321.6				Wash Boring Borehole cave-in at 7.9 m below GL
BH 18-1 (50 mm dia. piezometer)	321.3	September 27, 2018 (upon completion)	1.6	319.7	Indicative of a sub- artesian condition in the fractured bedrock
		November 18, 2018	1.5	319.8	
		January 10, 2019	2.3	319.0	

---

## CLOSURE

The "Limitations of Report" as presented in **Appendix H** are an integral part of this report.

---

## SIGNATURES

We trust that the information contained in this foundation investigation report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

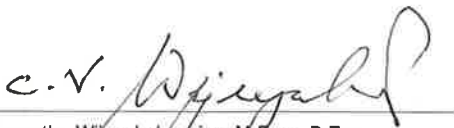
### WSP Canada Inc.



Franklin Oliha, MSc., EIT



Masud Karim, Ph.D., P.Eng.  
Senior Geotechnical Engineer/Independent Technical Reviewer



Vasantha Wijeyakulasuriya, M.Eng., P.Eng.  
Senior Technical Director, Geotechnical  
MTO Designate (Foundations).



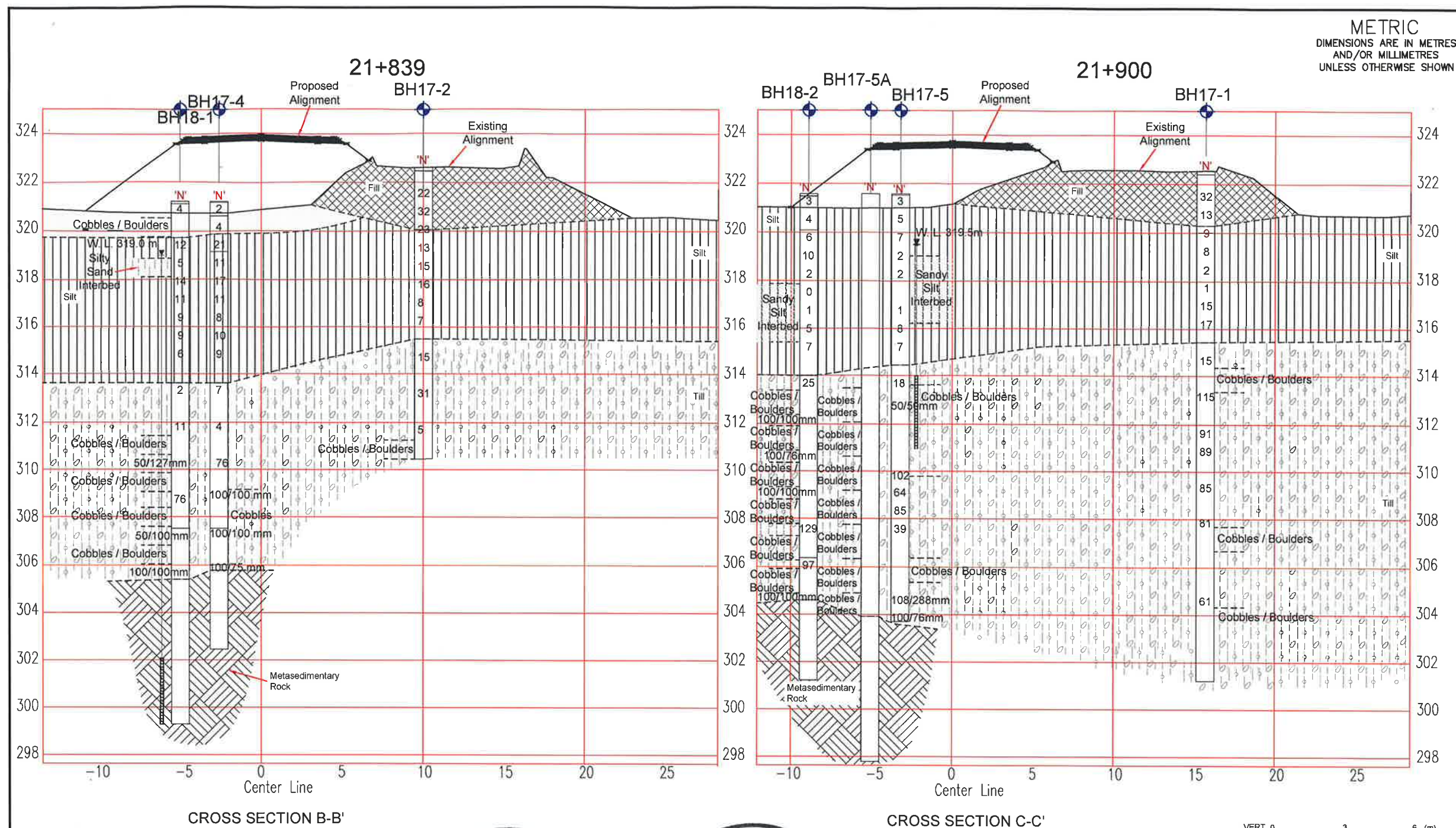
# DRAWINGS

---









CONT No:  
5266-13-00

Hwy 101 and Hwy 7172  
EMBANKMENT CROSS SECTIONS  
AT PROPOSED ALIGNMENT

**wsp** 51 Constellation Court  
Toronto, Ontario  
M9W 1K4



KEY PLAN  
NOT TO SCALE

LEGEND			
	Borehole		
N	Blows/0.3m (Std Pen Test, 475 J/blow)		
	WL upon completion		
	WL in Piezometer		
	Piezometer		
BH No.	ELEV. (m)	MTM NAD83 ZONE 13 CO-ORDINATES	
		NORTH (m)	EAST (m)
BH17-1	322.5	5346609.5	420051.5
BH17-2	322.6	5346547.0	420052.0
BH17-3	321.3	5346539.3	420039.5
BH17-4	321.3	5346550.4	420037.7
BH17-5	321.6	5346598.5	420033.0
BH17-5A	321.6	5346595.9	420032.5
BH17-6	321.7	5346613.1	420031.3
BH18-1	321.3	5346553.1	420036.3
BH18-2	321.6	5346593.6	420028.3

— NOTES —

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

Drawings are based on "WP 5266-13-00-SB bridge&hwy 7172 NEW DESIGN 11.5m OFFSET- PLAN&PROFILE WITH BOREHOLE LOCATIONS.dwg" received Nov. 18 2018 and "GA-90%" received Dec. 8 2018

LICENSED PROFESSIONAL ENGINEER  
A.S.M.M. KARIM  
100223054  
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER  
V. WIJEYAKULASURIYA  
100186910  
PROVINCE OF ONTARIO

SOIL STRATA SYMBOLS

Fill	Silty Sand / Sandy Silt / Sand and Silt Till	Silt	Sand	Sandy Silt	Silty Sand	Metasedimentary Rock
------	--	------	------	------------	------------	----------------------

# APPENDIX

**A**

RECORD OF BOREHOLE SHEETS

**METRIC** 1 OF 3

[illegible]

Measurement      1st      2nd      3rd      4th



RECORD OF BOREHOLE No BH17-1

METRIC 2 OF 3

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420051.5, N 5346609.5 ORIGINATED BY DW  
 DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FL  
 DATUM \_\_\_\_\_ DATE Oct/24/2017 to Oct/25/2017 CHECKED BY MP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	10 20 30	GR SA SI CL			
	Continued														
	<b>SANDY SILT (TILL):</b> trace gravel, trace to some clay, contains cobbles / boulders, grey, wet, compact to very dense. (continued)														
11			13	SS	91		312								
			14	SS	89		311								
12															
			15	SS	85		310								
13															
14															
			16	SS	81		308								
15															
	cobbles/boulders														
			17	RC			307								
16															
17															
			18	SS	61		305								
18															
	cobbles/boulders														
19															
20			19	RC			303								

Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

+ 3, X 3: Numbers refer to  
Sensitivity

○ 3% Strain at Failure

171-02344-00



RECORD OF BOREHOLE No BH17-1

METRIC 3 OF 3

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420051.5, N 5346609.5 ORIGINATED BY DW  
 DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FL  
 DATUM \_\_\_\_\_ DATE Oct/24/2017 to Oct/25/2017 CHECKED BY MP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>			
	Continued																
	<b>SANDY SILT (TILL):</b> trace gravel, trace to some clay, contains cobbles / boulders, grey, wet, compact to very dense. (continued)					302											
21																	
301.2																	
21.3	<b>End of Borehole</b>  Note: 1) Borehole caved-in at 1.7 m upon completion. 2) Water level was at a depth of 0.8 m upon completion.																

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

+ 3, X 3: Numbers refer to  
Sensitivity

○ 3% Strain at Failure

171-02344-00

RECORD OF BOREHOLE No BH17-2

METRIC 1 OF 2

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420052, N 5346547 ORIGINATED BY DW  
DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Hollow Stem/Wash Boring/Coring COMPILED BY FL  
DATUM \_\_\_\_\_ DATE Oct/20/2017 to Oct/20/2017 CHECKED BY MP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100			
322.6	Ground Surface														
0.1	ASPHALT (140 mm)		1	AS			322								
	FILL: gravelly sand, trace silt, trace clay, brown to grey, moist to wet, compact to dense		2	AS											
1			3	SS	22										
2			4	SS	32										
320.0			5	SS5A	23		320								
2.6	SILT: trace clay, greyish brown to grey, wet, loose to compact.			SS5B											
3			6	SS	13										
4	trace organics dilatant		7	SS	15										
5			8	SS	16										
6			9	SS	8										
7			10	SS	7										
315.4															
7.2	SILTY SAND (TILL): some gravel, trace clay, grey, wet, loose to dense		11	SS	15										
8															
9															
10			12	SS	31										

Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

+ 3, X 3: Numbers refer to  
Sensitivity

○ 3% Strain at Failure

171-02344-00

**METRIC** 2 OF 2

SOIL PROFILE						SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			POCKET PEN. (C <sub>u</sub> ) (kPa)		NATURAL UNIT WT (kN/m³)		REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa							WATER CONTENT (%)												
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							W <sub>P</sub> W      W <sub>L</sub> —————○—————												
								20 40 60 80 100							10 20 30					GR SA SI CL							
Continued	SILTY SAND (TILL): some gravel, trace clay, grey, wet, loose to dense ( <i>continued</i> )		13	SS	5		312																Flowing sand				
	cobbles /boulders		14	RC			311																				
End of Borehole	Note: 1) Borehole caved-in at 1.7 m upon completion. 2) Water level was at a depth of 0.8 m upon completion.																										

171-02344-00

# RECORD OF BOREHOLE No BH17-3

METRIC 1 OF 2

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420039.5, N 5346539.3 ORIGINATED BY FO  
 DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Hollow Stem Auger/DCPT COMPILED BY FO  
 DATUM \_\_\_\_\_ DATE Jul/18/2018 to Jul/18/2018 CHECKED BY V.W

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			POCKET PEN (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								○ UNCONFINED	+ FIELD VANE				
								● QUICK TRIAXIAL	× LAB VANE				
321.3	Ground Surface						20 40 60 80 100	W <sub>P</sub>	W	W <sub>L</sub>	γ		GR SA SI CL
0.1	TOPSOIL 50 mm												
	SAND: some silt , trace clay, trace organics, trace rootlets, blackish brown, moist, loose		1	SS	10		321						
1			2	SS	9		320						0 77 20 3
319.6			3	SS3A									
1.7	SILT: trace clay, grey, wet, loose to compact		3	SS3B	13		319						
2			4	SS	9		318						
3			5	SS	14		317						
4			6	SS	11		316						
5			7	SS	14		315						Non-plastic 0 1 95 4
6			8	SS	10		314						
7			9	SS	7		313						
313.7			10	SS	9		312						6 40 54 2
7.6	SAND AND SILT TILL: trace clay, trace gravel, grey, wet, very loose to loose		11	SS	2								
311.6	End of Sampling, DCPT below to 11.6 m												DCPT starts at 9.8 m
9.8													

Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

+ 3, X 3: Numbers refer to  
Sensitivity


○ 3% Strain at Failure

171-02344-00

RECORD OF BOREHOLE No BH17-3

METRIC 2 OF 2

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420039.5, N 5346539.3 ORIGINATED BY FO  
 DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Hollow Stem Auger/DCPT COMPILED BY FO  
 DATUM \_\_\_\_\_ DATE Jul/18/2018 to Jul/18/2018 CHECKED BY V.W

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT  SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE									
	Continued												
	End of Sampling, DCPT below to 11.6 m (continued)												
11													
309.7													
11.6	End of Borehole/DCPT												
	Notes: 1. Water level was at 4.6 m upon completion 2. Borehole cave at 4.6 m upon completion												

171-02344-00

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

+ 3, × 3: Numbers refer to Sensitivity

○ 3% Strain at Failure

RECORD OF BOREHOLE No BH17-4

METRIC 1 OF 2

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420037.7, N 5346550.4 ORIGINATED BY FO  
 DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FO  
 DATUM \_\_\_\_\_ DATE Jul/16/2018 to Jul/17/2018 CHECKED BY V.W

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100			
321.3	Ground Surface														
0.1	TOPSOIL 50 mm														
320.7	SAND: trace organics, trace rootlets, blackish brown, moist, very loose		1	SS	2		321								
0.6	SILTY SAND: trace organics, trace rootlets, brown, wet, very loose		2	SS	4										
319.9	SILT: trace to some sand, trace clay, grey, wet, loose to compact		3	SS	21		320								0 21 71 8
1.4			4	SS	11		319								Non-plastic 0 17 78 5
			5	SS	17		318								
			6	SS	11		317								
			7	SS	8		316								
			8	SS	10		315								
			9	SS	9		314								
313.7	SILTY SAND (TILL): trace to some gravel, grey, wet, very loose to very dense		10	SS	7		313								17 53 29 1
7.6			11	SS	4		312								Non-plastic 8 50 37 5 Flowing sand observed

Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

+ 3, X 3: Numbers refer to  
Sensitivity

○ 3% Strain at Failure

171-02344-00

RECORD OF BOREHOLE No BH17-4

METRIC 2 OF 2

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420037.7, N 5346550.4 ORIGINATED BY FO  
DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FO  
DATUM \_\_\_\_\_ DATE Jul/16/2018 to Jul/17/2018 CHECKED BY V.W

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>	POCKET PEN. (C <sub>u</sub> ) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60						80	100	20	40	60
	Continued							SHEAR STRENGTH kPa			WATER CONTENT (%)									
								○ UNCONFINED + FIELD VANE			● QUICK TRIAXIAL × LAB VANE									
								20	40	60	80	100	10	20	30					

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

+ 3, × 3: Numbers refer to Sensitivity ○ 3% Strain at Failure

171-02344-00

LOG of ROCK CORE BH17-4

METRIC 1 OF 1

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420037.7, N 5346550.4 ORIGINATED BY FO  
DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Track mount CME55 / Wash Boring/Coring COMPILED BY FO  
DATUM \_\_\_\_\_ DATE Jul/16/2018 to Jul/17/2018 CHECKED BY V.W

(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE		TOTAL CORE RECOVERY (%)	SOLID CORE RECOVERY (%)	HARD LAYER (%)	RQD (%)	FRACTURE INDEX (per 0.3 m)	DISCONTINUITIES	Weathering Index HYDRAULIC CONDUCTIVITY (cm/sec)	POINT LOAD TEST UCS AXIAL (MPa)	POINT LOAD TEST UCS DIAMETRAL (MPa)*	UNIAXIAL COMPRESSION (MPa)	DENSITY (g/cm <sup>3</sup> ) E (GPa)
			NUMBER	SIZE											
305.8	Rock Surface														
15.5	<b>METASEDIMENTARY ROCK, grey, fresh Granitic Gneiss</b>		1	NQ	100	76		52	1	Fracture: 15.51m-15.54m, $\theta=55^\circ$ 15.81m-15.93m, $\theta=30^\circ$ to $20^\circ$	W2 to W1	104.6	42.3		
305.3									1						
16.0			2	NQ	100	80		72	1	Fracture: 16.00m-16.03m, $\theta=65^\circ$ 16.87m-16.90m, $\theta=45^\circ$ 17.17m-17.22m, $\theta=55^\circ$ 17.22m-17.37m, $\theta=20^\circ$ 17.41m-17.44m, $\theta=70^\circ$ Joint: 16.76m-16.81m, $\theta=50^\circ$	W2 to W1				
303.8									3			102.6	61.5		
17.5									2						
18			3	NQ	100	80		76	2	Fracture: 17.53m-17.58m, $\theta=45^\circ$ 17.70m-17.73m, $\theta=65^\circ$ 17.74m-17.78m, $\theta=50^\circ$ 17.98m-18.03m, $\theta=40^\circ$ 18.50m-18.53m, $\theta=60^\circ$ 18.63m-18.68m, $\theta=35^\circ$ 18.67m-18.77m, $\theta=25^\circ$	W2 to W1	115.1	94.0		
302.5									5						
18.8	<b>END OF BOREHOLE</b> Notes: 1) Water level was at 1.3 m upon completion. 2) Borehole cave-in at 9.9 m upon completion.														



**METRIC** 1 OF 2

[illegible]

171-02344-00

	1st	2nd	3rd	4th
Measurement				

**METRIC** 2 OF 2

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			POCKET PEN. (C <sub>u</sub> ) (kPa)	NATURAL UNIT WT (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	W <sub>p</sub>				W	W <sub>L</sub>	
	Continued																	
	<b>SILTY SAND (TILL):</b> trace to some gravel, trace clay, contains cobbles / boulders, grey, wet, compact to very dense. (continued)		11	RC														
	silt and sand till		12	SS	102													
			13	SS	64													
			14	SS	85													
			15	SS	39													
	cobbles/boulders																	
			16	RC														
			17	SS	108/ 288mm													
			18	SS	100/ 76mm													
303.7 17.9	<b>End of Borehole</b>																	
Note: 1)Borehole caved-in at 17.4 m upon completion. 2)Water level was at a depth of 1.5 m upon completion  <b>Water level readings:</b> Date    Depth (m)    Elevation (m) Oct26-17    1.5    319.5  Dec18-17    2.3    318.7 Sept 28-18    1.8    319.2 Nov18-18    1.6    320.0 Jan10-19    2.1    319.5																		
No Recovery																		

WSP-SOIL-ROCK-MAY-29-2017.GLB  
ALMTO. 2016 WITH WSP 171-0234-00-GINT REVISED. EQ. JAN. 8 10:01 11/11/19

171-02344-00

### GROUNDWATER ELEVATIONS

Measurement      1st      2nd      3rd      4th

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ **8**=3% Strain at Failure

**METRIC** 1 OF 3

10 9 8  
ON-MIC-2016-MITH WSP 171-0234-00-GINT REVISED TO JAN 8 19 45 P 19/19

## GROUNDWATER ELEVATIONS

171-02344-00

RECORD OF BOREHOLE No BH17-5A

METRIC 2 OF 3

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420032.5, N 5346595.9 ORIGINATED BY FO  
 DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FO  
 DATUM \_\_\_\_\_ DATE Sep/25/2018 to Sep/25/2018 CHECKED BY VW

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W <sub>p</sub>	W			
	Continued Wash boring done from surface to bedrock surface without sampling (continued)																
11	Cobbles/Boulders		3	RC		311											Recovered core thickness 20 mm
12	Cobbles/Boulders		4	RC		310											Recovered core thickness 200 mm
13	Cobbles/Boulders		5	RC		309											Recovered core thickness 350 mm
14	Cobbles/Boulders		6	RC		308											Recovered core thickness 450 mm
15	Cobbles/Boulders		7	RC		307											Recovered core thickness 470 mm
16	Cobbles/Boulders					306											
17	Cobbles/Boulders					305											
18	303.9 17.7 META SEDIMENTARY ROCK: Granitic Gneiss		1	RC		304											
19			2	RC		303											
20			3	RC		302											

Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

+ 3, × 3: Numbers refer to  
Sensitivity


○ 3% Strain at Failure

171-02344-00

**METRIC** 3 OF 3

SOIL PROFILE				SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)				
	Continued													
	META SEDIMENTARY ROCK: (continued) Granitic Gneiss		4	RC			301							
			5	RC			300							
			6	RC			299							
297.9 23.8	END OF BOREHOLE Notes: 1)Borehole caved-in at 9.2 m upon completion. 2)Water level at 0.6 m upon completion						298							

171-02344-00

Measurement 

1st 2nd 3rd 4th

LOG of ROCK CORE BH17-5A

METRIC 1 OF 1

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420032.5, N 5346595.9 ORIGINATED BY FO  
DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FO  
DATUM \_\_\_\_\_ DATE Sep/25/2018 to Sep/25/2018 CHECKED BY VW

(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE		TOTAL CORE RECOVERY (%)	SOLID CORE RECOVERY (%)	HARD LAYER (%)	RQD (%)	FRACTURE INDEX (per 0.3 m)	DISCONTINUITIES	Weathering Index	HYDRAULIC CONDUCTIVITY (cm/sec)	POINT LOAD TEST UCS AXIAL (MPa)	POINT LOAD TEST UCS DIAMETRAL (MPa)*	UNIAXIAL COMPRESSION (MPa)	DENSITY (g/cm <sup>3</sup> ) E (GPa)
			NUMBER	SIZE												
303.9	Rock Surface															
17.7	<b>METASEDIMENTARY ROCK, grey, fresh Granitic Gneiss</b>		1	NQ	100	88		60	1	Fracture: 17.72m-17.74m, $\theta=55^\circ$ 18.07m-18.16m, $\theta=35^\circ$ and $30^\circ$ , two sets 18.15m-18.17m, $\theta=75^\circ$ 18.26m-18.31m, $\theta=40^\circ$ Joint: 17.75m-17.77m, $\theta=75^\circ$ 18.00m-18.07m, $\theta=30^\circ$ Fragmented zone: 18.31m-18.55m Fracture: 18.64m-18.68m, $\theta=45^\circ$ 18.84m-18.87m, $\theta=55^\circ$ Joint: 18.61m-18.62m, $\theta=70^\circ$ 18.71m-18.77m, $\theta=30^\circ$ Fracture: 19.15m-19.25m, $\theta=25^\circ$ to $0^\circ$ 19.27m-19.29m, $\theta=75^\circ$ Fracture: 20.96m-21.01m, $\theta=55^\circ$ Joint: 20.09m-20.14m, $\theta=40^\circ$  Fracture: 21.67m-21.70m, $\theta=65^\circ$ Joint: 21.45m-21.49m, $\theta=50^\circ$  Fracture: 23.22m-23.25m, $\theta=50^\circ$ 23.27m-23.30m, $\theta=55^\circ$ Joint: 23.09m-23.14m, $\theta=40^\circ$ 23.66m-23.71m, $\theta=45^\circ$	W1 to W1					
18									4		W2 to W1		147.8	104.0	146.1	
303.3			2	NQ	100	46		27	24		W1 to W1					
18.3									2		W2 to W1		139.2	91.6		
302.6			3	NQ	100	80		80	2		W1 to W1					
19.0									0		W2 to W1		110.0	94.5		
301.9			4	NQ	100	98		98	0		W1 to W1					
19.7									0		W2 to W1		103.2	92.0		
20									0							
21									0							
300.4			5	NQ	100	98		98	0		W1 to W1					
21.2									1		W2 to W1		95.9	67.6		
22									0							
298.9									0							
22.7			6	NQ	100	98		88	2		W1 to W1		128.0	94.6		
23									0		W2 to W1					
297.9									0							
23.8	<b>END OF BOREHOLE</b> Notes: 1) Borehole caved-in at 9.2 m upon completion. 2) Water level at 0.6 m upon completion															

**METRIC** 1 OF 1

[illegible]





Measurement      1st      2nd      3rd      4th

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ **8**=3% Strain at Failure

171-02344-00

**METRIC** 1 OF 3

SOIL PROFILE												
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	PCKET PRN. (GJ) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS & GRAIN SIZE DISTRIBUTION (%)
			NUMBER			TYPE	"N" VALUES	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	w <sub>p</sub>	w	w <sub>L</sub>	
321.3	Ground Surface											
0.1	TOPSOIL (100 mm) <b>SAND:</b> trace silt, trace organics, some rootlets, brown, moist very loose.		1	SS	4							
	Cobbles/Boulders		2	RC								Grinding Noise recovered rock core thickness 350 mm
319.8	<b>SILT:</b> trace sand, trace gravel, trace clay, grey, wet, loose to compact.		3	SS	12							3 90 7
1.5	Silty Sand		4	SS	5							2 54 (44)
			5	SS	14							
			6	SS	11							
			7	SS	9							
			8	SS	9							
			9	SS	6							
313.7	<b>SANDY SILT (TILL):</b> trace clay, trace gravel, trace cobble, grey, moist to wet, very loose to very dense.		10	SS	2							Non-plastic 5 27 64 4
7.6			11	SS	11	Holeplug						Recovered core thickness 230

	1st	2nd	3rd	4th
Measurement				

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ **8**=3% Strain at Failure

171-02344-00



**METRIC** 2 OF 3

[illegible]

Measurement 

171-02344-00

RECORD OF BOREHOLE No BH18-1

METRIC 3 OF 3

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420036.3, N 5346553.1 ORIGINATED BY FO  
 DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FO  
 DATUM \_\_\_\_\_ DATE Sep/27/2018 to Sep/28/2018 CHECKED BY VW

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>	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (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										
	Continued																	
	META SEDIMENTARY ROCK: (continued)																	
	Granitic Gneiss		4	RC			301											
			5	RC			300											
21																		
299.4	End of Borehole																	
21.9	Note: 1) No Cave-in upon completion. 2) Water level at 1.6 m upon completion																	
	Water level readings:																	
	Date Depth (m) Elevation (m)																	
	Sept 27-18 1.6 319.7																	
	Nov 18-18 1.5 319.8																	
	Jan 10-19 2.3 319.0																	

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

+ 3, X 3: Numbers refer to Sensitivity

○ 3% Strain at Failure

171-02344-00

LOG of ROCK CORE BH18-1

METRIC 1 OF 1

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420036.3, N 5346553.1 ORIGINATED BY FO  
DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FO  
DATUM \_\_\_\_\_ DATE Sep/27/2018 to Sep/28/2018 CHECKED BY VW

(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE		TOTAL CORE RECOVERY (%)	SOLID CORE RECOVERY (%)	HARD LAYER (%)	RQD (%)	FRACTURE INDEX (per 0.3 m)	DISCONTINUITIES	Weathering Index W1 to W5	HYDRAULIC CONDUCTIVITY (cm/sec)	POINT LOAD TEST UCS AXIAL (MPa)	POINT LOAD TEST UCS DIAMETRAL (MPa)*	UNIAXIAL COMPRESSION (MPa)	DENSITY (g/cm <sup>3</sup> ) E (GPa)
			NUMBER	SIZE												
305.5	Rock Surface															
16 15.9	METASEDIMENTARY ROCK, grey, fresh Granitic Gneiss		1	NQ	100	61		50	23	Fragmented zone: 15.85m-15.90m; 15.98m-16.15m Fracture: 15.90m-15.94m, $\theta=45^\circ$ 15.98m-16.01m, $\theta=0^\circ$ 16.15m-16.22m, $\theta=30^\circ$ 16.56m-16.59m, $\theta=60^\circ$ 16.59m-16.61m, $\theta=70^\circ$ Fracture: 16.76m-16.78m, $\theta=65^\circ$ 16.95m-16.98m, $\theta=55^\circ$ 17.36m-17.40m, $\theta=40^\circ$ 17.36m-17.49m, $\theta=15^\circ, 10^\circ$ and 30°, three sets 17.48m-17.55m, $\theta=30^\circ$ 17.64m-17.78m, $\theta=20^\circ$ and 25°, two sets 17.78m-17.86m, $\theta=30^\circ$ 17.88m-18.02m, $\theta=10^\circ$ and 15°, two sets 17.94m-17.97m, $\theta=35^\circ$ Joint: 17.20m-17.31m, $\theta=35^\circ$ 17.34m-17.39m, $\theta=30^\circ$ Fracture: 18.46m-18.54m, $\theta=25^\circ$ 18.68m-19.98m, $\theta=25^\circ$ 19.98m-20.02m, $\theta=50^\circ$ Joint: 18.46m-19.86m, $\theta=5^\circ$ 20.03m-20.09m, $\theta=30^\circ$ Voids at: 18.90m-18.92m; 19.02m-19.04m 19.06m-19.07m Fracture: 20.52m-20.65m, $\theta=25^\circ$ 20.55m-20.59m, $\theta=45^\circ$ 20.92m-20.96m, $\theta=50^\circ$ Joint: 19.86m-19.92m, $\theta=40^\circ$ 20.69m-20.85m, $\theta=15^\circ$ Fracture: 21.62m-21.73m, $\theta=30^\circ$ 21.72m-21.82m, $\theta=30^\circ$	W2 to W1		112.1	54.6		
304.5 16.8			2	NQ	100	68		68	1		W2 to W1					
17									1							
18									5							
303.0 18.3									8							
19									0							
301.5 19.8									1							
20									2							
300.0 21.3									6		W2 to W1					
21									0							
300.0 21.3									0							
299.4									0							
21.9	END OF BOREHOLE Notes: 1) Borehole caved-in at 21.9 m upon completion. 2) Water level at 1.6 m upon completion  Water level readings: Date Depth (m) Elevation (m) Sept 27-18 1.6 319.7 Nov 18-18 1.5 319.8 Jan 10-19 2.3 319.0															

RECORD OF BOREHOLE No BH18-2

METRIC 1 OF 3

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420028.3, N 5346593.6 ORIGINATED BY FO  
DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FO  
DATUM \_\_\_\_\_ DATE Sep/26/2018 to Sep/26/2018 CHECKED BY VW

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>	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						
321.6	Ground Surface														
0.1	<b>TOPSOIL</b> (100 mm) <b>SILTY SAND:</b> trace organic, trace rootlets, black to brown, moist, very loose		1	SS	3										
1			2	SS	4										
320.1	<b>SILT:</b> some sand, trace clay, trace gravel, grey, moist to wet, very loose to loose.		3	SS	6										
1.5															
2															
	traces of red colour		4	SS	10										
3															
			5	SS	2										
4															
	Sandy Silt		6	SS	0										
5															
			7	SS	1										
6															
			8	SS	5										
7															
	trace organics, dark grey		9	SS	7										
314.0	<b>SANDY SILT (TILL):</b> trace clay, trace gravel, grey, moist to wet, compact to very dense		10	SS	25										
7.6															
	Cobbles/Boulders		11	RC											
			12	SS	100/100mm										
	Cobbles/Boulders														

Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th


+ 3, X 3: Numbers refer to Sensitivity

○ 3% Strain at Failure

171-02344-00

**METRIC** 2 OF 3

SOIL PROFILE							SAMPLES		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS			
<b>SANDY SILT (TILL):</b> trace clay, trace gravel, grey, moist to wet, compact to very dense ( <i>continued</i> )									
			13	RC					
			14	SS	100/ 76mm				
	Cobbles/Boulders		15	RC					
			16	SS	100/ 100mm				
	Cobbles/Boulders		17	RC					
			18	SS	129				
	Cobbles/Boulders		19	RC					
			20	SS	97				
	Cobbles/Boulders		21	RC					
			22	SS	100/ 100mm				
<b>META SEDIMENTARY ROCK:</b> <b>Granitic Gneiss</b>									
			1	RC					
			2	RC					

Measurement 

171-02344-00

RECORD OF BOREHOLE No BH18-2

METRIC 3 OF 3

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420028.3, N 5346593.6 ORIGINATED BY FO  
 DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FO  
 DATUM \_\_\_\_\_ DATE Sep/26/2018 to Sep/26/2018 CHECKED BY VW

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>	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (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										
	Continued																	
301.3	META SEDIMENTARY ROCK: (continued)		3	RC														
20.4	End of Borehole Note: 1) Borehole caved-in at 7.9 m upon completion. 2) Water level was at ground surface upon completion																	

WSP-SOIL-ROCK-MAY-20-2017-CLB  
 ONATO-2018-11-15-171-02344-00-REVISED-FO JAN 8 10:01 10/19

171-02344-00

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

+ 3, X 3: Numbers refer to Sensitivity

○ 3% Strain at Failure

LOG of ROCK CORE BH18-2

METRIC 1 OF 1

W.P. \_\_\_\_\_ LOCATION MTM NAD 83 (Zone 13), E 420028.3, N 5346593.6 ORIGINATED BY FO  
DIST \_\_\_\_\_ HWY 101 BOREHOLE TYPE Wash Boring/Coring COMPILED BY FO  
DATUM \_\_\_\_\_ DATE Sep/26/2018 to Sep/26/2018 CHECKED BY VW

(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE		TOTAL CORE RECOVERY (%)	SOLID CORE RECOVERY (%)	HARD LAYER (%)	RQD (%)	FRACTURE INDEX (per 0.3 m)	DISCONTINUITIES	Weathering Index	HYDRAULIC CONDUCTIVITY (cm/sec)	POINT LOAD TEST UCS AXIAL (MPa)	POINT LOAD TEST UCS DIAMETRAL (MPa)*	UNIAXIAL COMPRESSION (MPa)	DENSITY (g/cm <sup>3</sup> ) E (GPa)	
			NUMBER	SIZE													
304.6	Rock Surface																
17.0	METASEDIMENTARY ROCK, grey, fresh  Granitic Gneiss		1	NQ	100	54		50	4	Fracture: 17.02m-17.06m, $\theta=25^\circ$ 17.08m-17.12m, $\theta=45^\circ$ 17.12m-17.23m, $\theta=15^\circ$ 17.23m-17.55m, $\theta=5^\circ$ to $25^\circ$ Joint: 17.58m-17.63m, $\theta=45^\circ$	W2 to W1						
									1								
									0								
									0								
									0								
303.4			2	NQ	100	80		80	2	Fracture: 18.38m-18.54m, $\theta=20^\circ$ 18.75m-18.81m, $\theta=30^\circ$ 19.02m-19.05m, $\theta=75^\circ$ Joint: 18.29m-18.38m, $\theta=15^\circ$ to $25^\circ$ 19.18m-19.25m, $\theta=45^\circ$ 19.34m-19.74m, $\theta=5^\circ$ to $10^\circ$	W2 to W1		85.5	57.2			
18.2									2								
									1								
									0								
									0								
301.9			3	NQ	100	96		96	0	Fracture: 20.39m-20.35m, $\theta=45^\circ$ Joint: 19.74m-20.31m, $\theta=5^\circ$ 20.19m-20.20m, $\theta=35^\circ$	W2 to W1						
19.7									0								
									2								
301.3	END OF BOREHOLE Notes: 1)Borehole caved-in at 7.9 m upon completion. 2)Water level was at ground surface upon completion.																
20.4																	

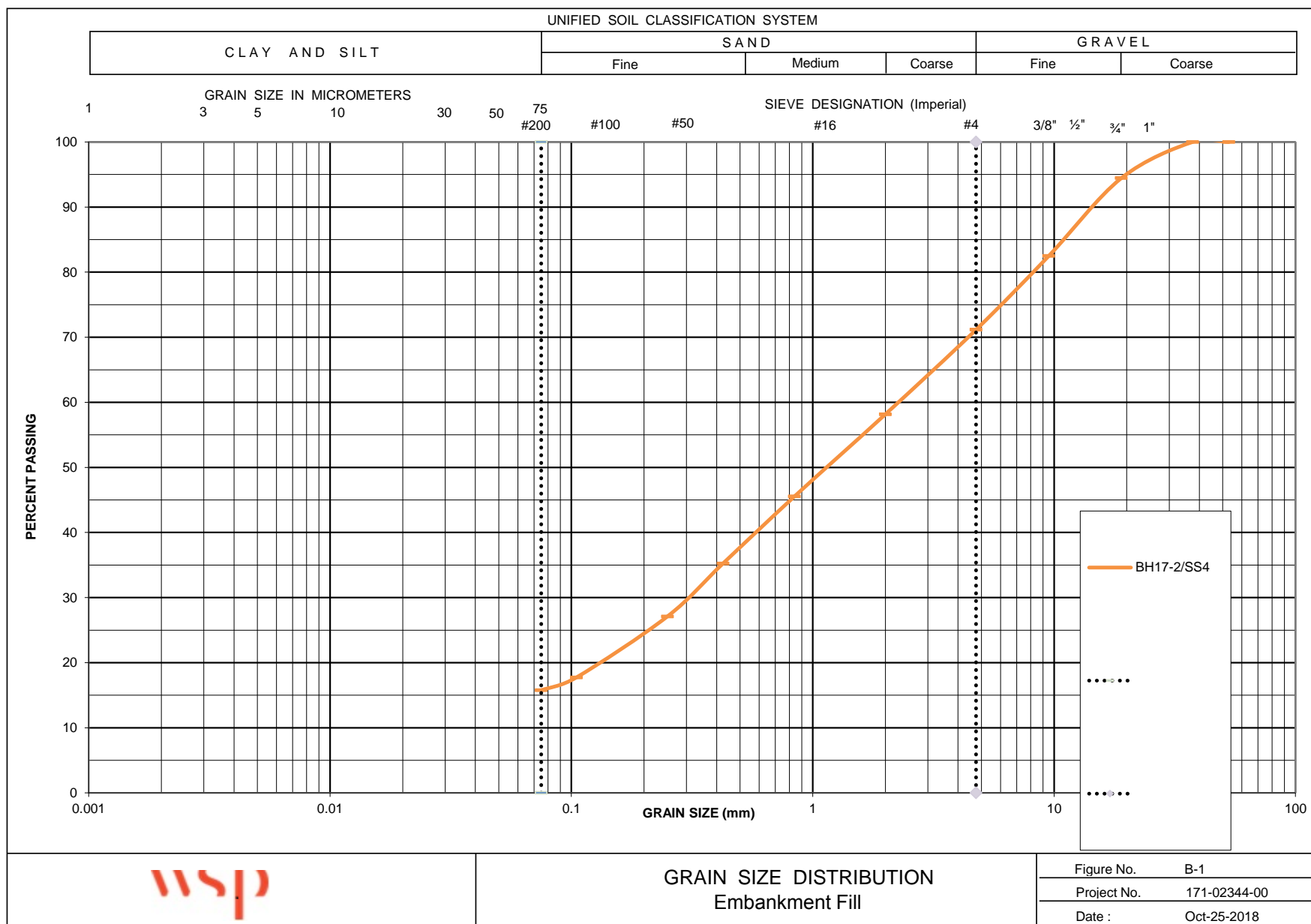
171-02344-00

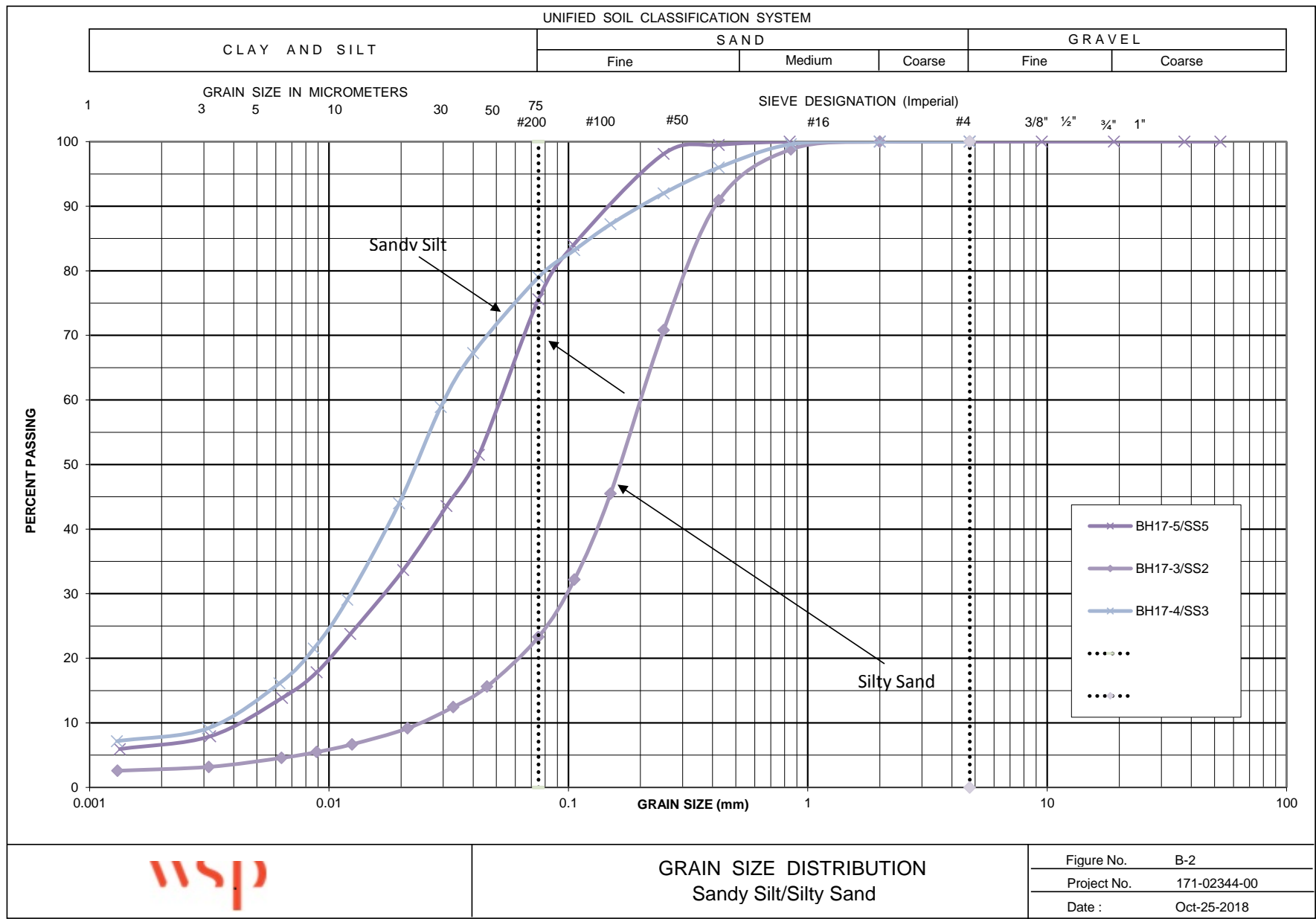
# APPENDIX

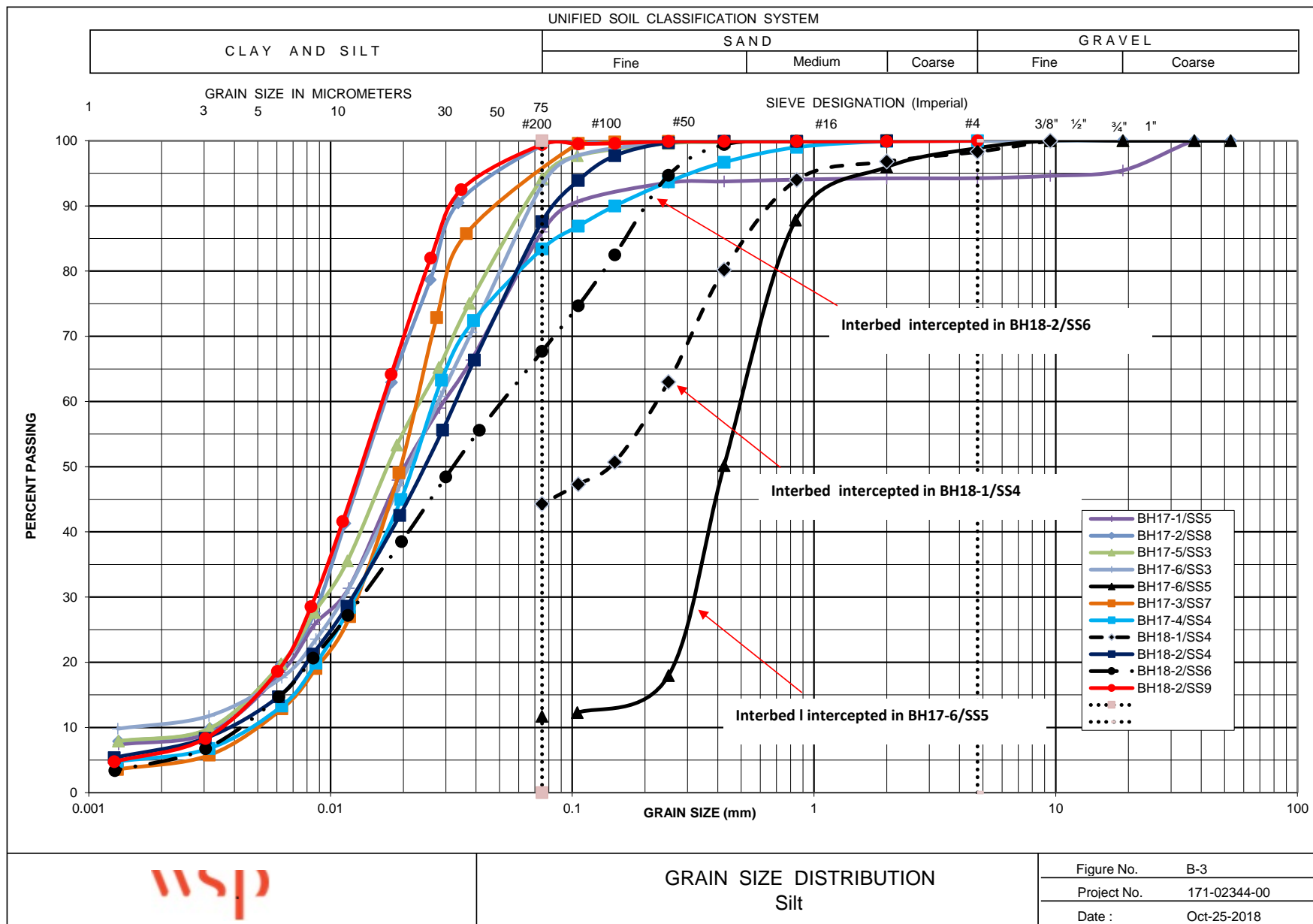
## B

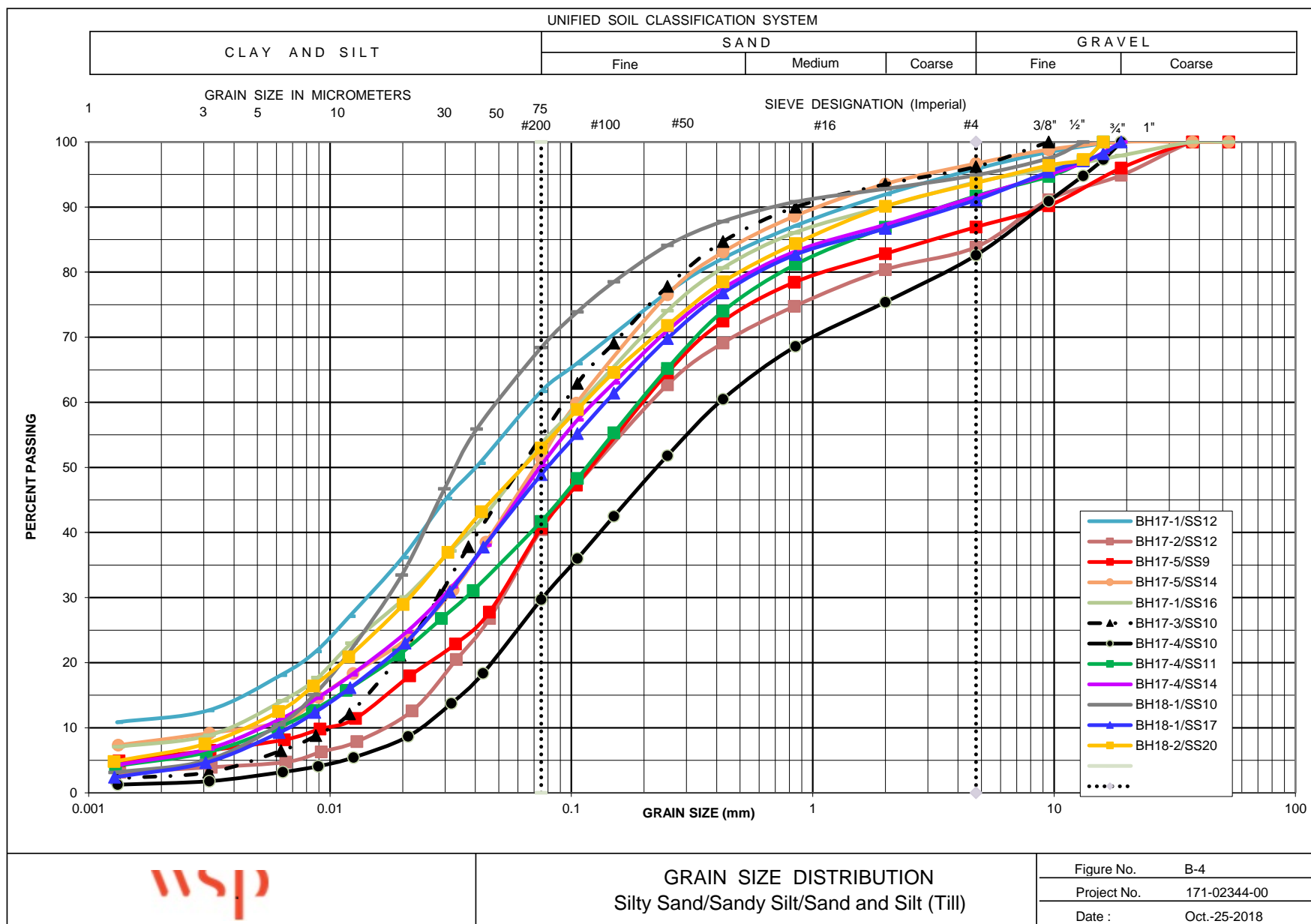
### LABORATORY TEST RESULTS











## UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

### ASTM D7012

#### SAMPLE IDENTIFICATION

PROJECT NUMBER	1894932 (11000)	SAMPLE NUMBER	-
PROJECT NAME	WSP/Lab Testing/Miss	SAMPLE DEPTH, m	17.8-18.0
BOREHOLE NUMBER	17-5A	DATE:	2018-11-02

#### TEST CONDITIONS

MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.25

#### SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.68	WATER CONTENT, (specimen) %	0.00
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m <sup>3</sup>	26.37
SAMPLE AREA, cm <sup>2</sup>	17.65	DRY UNIT WT., kN/m <sup>3</sup>	26.37
SAMPLE VOLUME, cm <sup>3</sup>	188.61	SPECIFIC GRAVITY	-
WET WEIGHT, g	507.42	VOID RATIO	-
DRY WEIGHT, g	507.42		

#### VISUAL INSPECTION

#### FAILURE SKETCH



#### TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	146.1
----------------------	-----	---------------------------	-------

REMARKS:

Checked By: 

**Golder Associates**

**GOLDER****UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS****ASTM D7012****SAMPLE IDENTIFICATION**

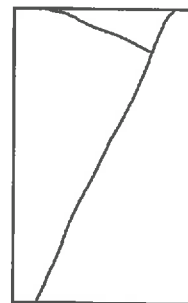
PROJECT NUMBER	1894932 (11000)	SAMPLE NUMBER	-
PROJECT NAME	WSP/Lab Testing/Miss	SAMPLE DEPTH, m	20.6-21.0
BOREHOLE NUMBER	18-1	DATE:	2018-11-02

**TEST CONDITIONS**

MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.28

**SPECIMEN INFORMATION**

SAMPLE HEIGHT, cm	10.82	WATER CONTENT, (specimen) %	0.20
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m <sup>3</sup>	27.39
SAMPLE AREA, cm <sup>2</sup>	17.62	DRY UNIT WT., kN/m <sup>3</sup>	27.34
SAMPLE VOLUME, cm <sup>3</sup>	190.61	SPECIFIC GRAVITY	-
WET WEIGHT, g	532.57	VOID RATIO	-
DRY WEIGHT, g	531.51		

**VISUAL INSPECTION****FAILURE SKETCH****TEST RESULTS**

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	39.1
----------------------	-----	---------------------------	------

REMARKS:

Checked By:

**Golder Associates**

# APPENDIX

C

SITE PHOTOGRAPHS

## **Project: Ivanhoe Bridge Replacement over Hwy 101**

**Assignment No. 5016 – E – 0027/GWP 5266 – 13 - 00**

### **SITE VISIT PHOTOGRAPHS**

#### **C1: Site Description Photographs**

#### **C2: Field Investigation Photographs**





Photo C1-1: Looking Northwest of Highway 101 – Flowing river (September 2018)



Photo C1-2: Looking towards North of Highway 101 – Differential Settlement at the Edge of South Approach Slab (September 2018)





Photo C1-3: Looking towards South of Highway 101 – Differential Settlement at the Edge of the North Approach Slab (September 2018)

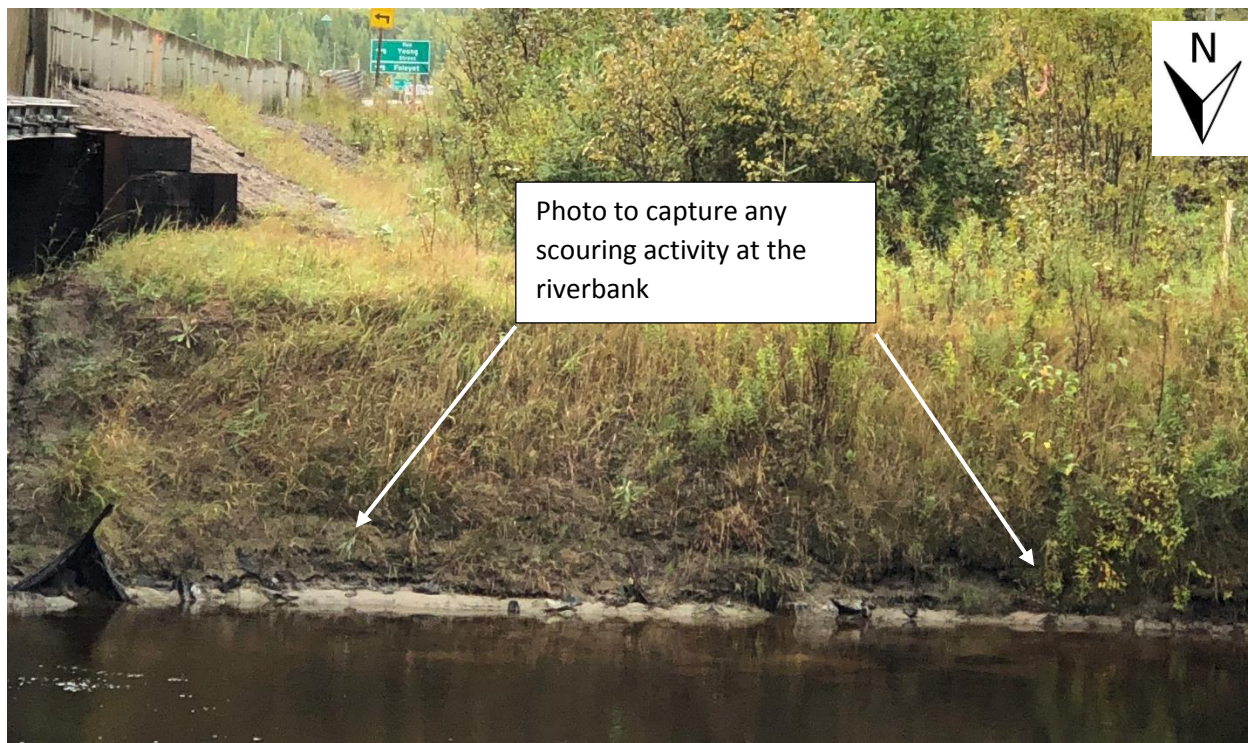


Photo C1-4: Looking towards South at the river bank for erosion activity at the south abutment along the proposed bridge alignment (September 2018)





Photo C1-5: Looking towards North at the river bank for erosion activity at the north abutment along the proposed bridge alignment (September 2018)





Photo C2-1: Looking towards South of Highway 101 – Traffic Control Set-up (September 2018)



Photo C2-2: Looking South of Highway 101 at a closer range (September 2018)



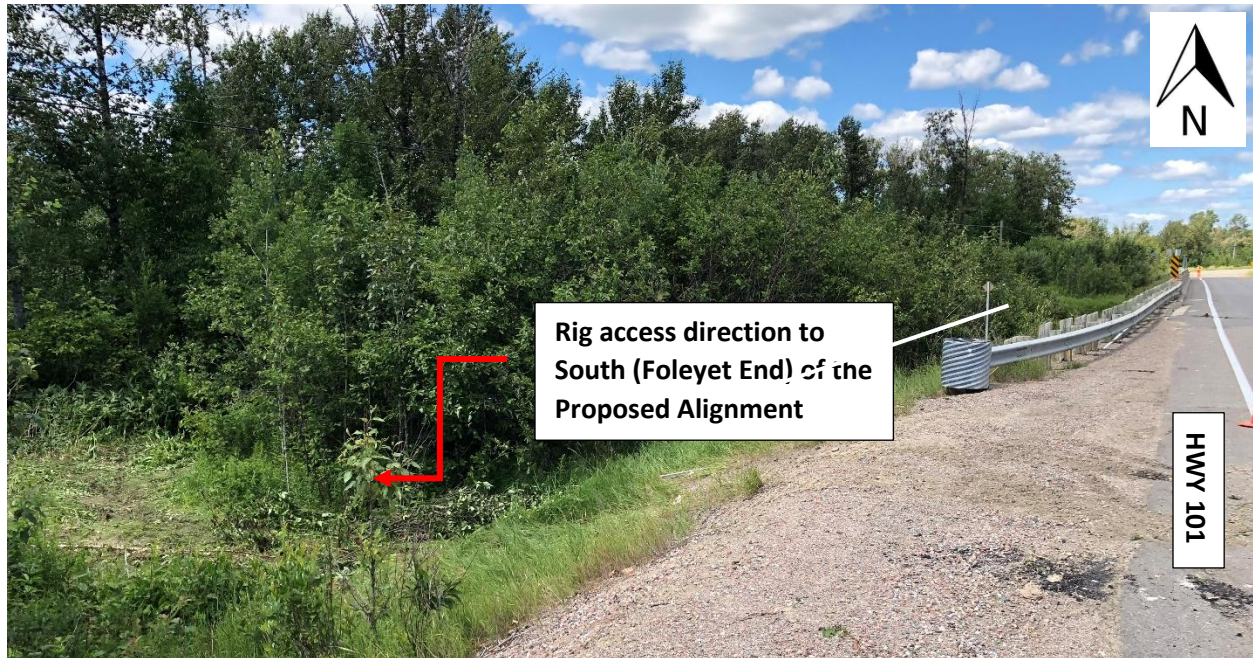


Photo C2-3: Looking Northwest – Rig Access to BH17-3, 17-4 & BH18-1 (June 2018)



Photo C2-4: Looking towards East – Approach to the south abutment at the proposed bridge alignment (June 2018)





Photo C2-5: Looking towards East – South abutment along the proposed bridge alignment (September 2018)



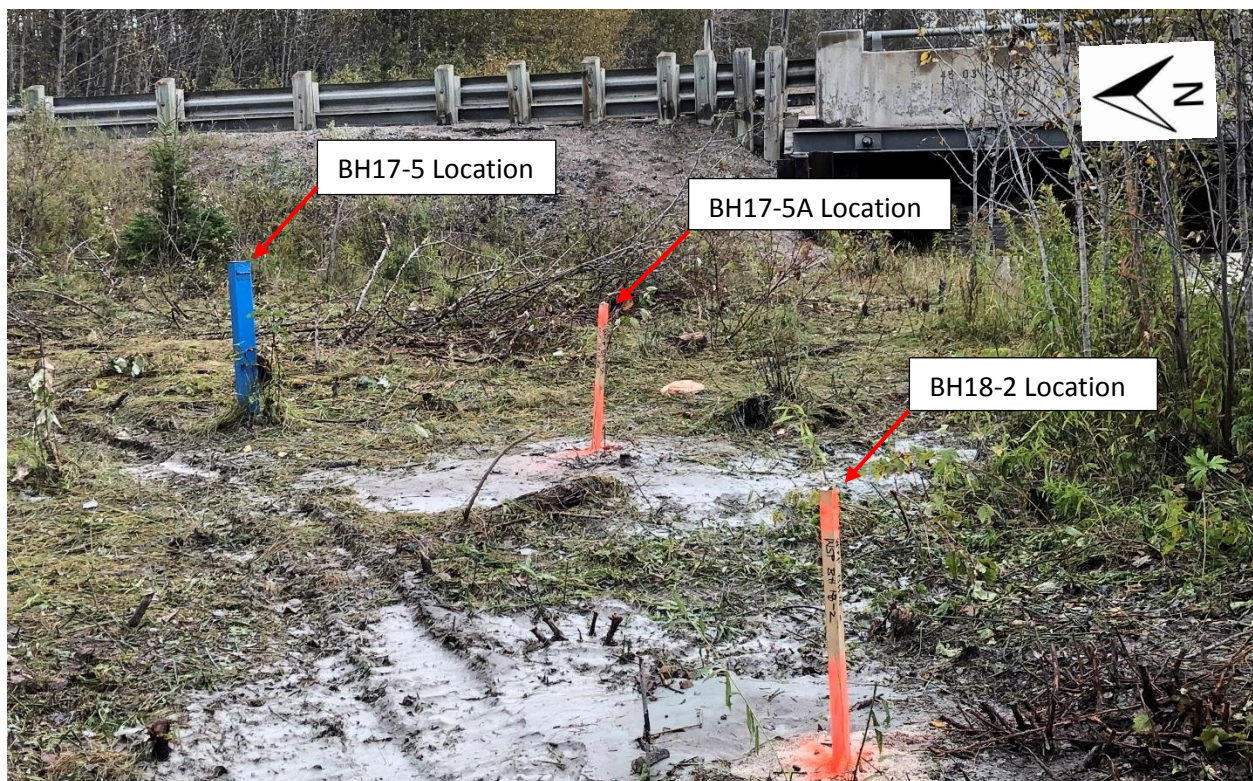
Photo C2-6: Looking towards South – Rig Access to BH17-5, BH17-6, BH17-5A & BH18-2 (September 2018)





Temporary access ramp  
made of wood logs

Photo C2-7: Looking Southwest – Showing temporary access ramp constructed of wood logs (September 2018)



BH17-5 Location

BH17-5A Location

BH18-2 Location

Photo C2-8: Looking towards East – North Abutment along the proposed bridge alignment (September 2018)





Photo C2-9: Looking Southwest – Approach to the North Abutment of the proposed bridge alignment (September 2018)



Photo C2-10: Looking towards South of Highway 101 (October 2017)



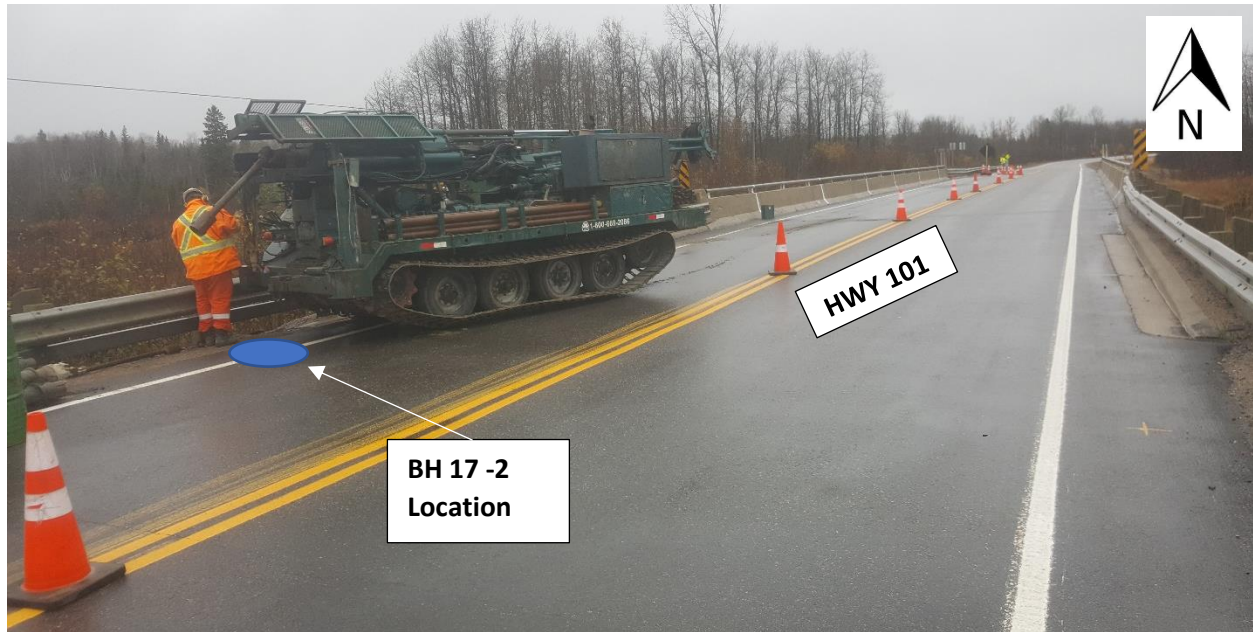


Photo C2-11: Looking towards North of Highway 101 (October 2017)

# APPENDIX

D

ROCK CORE PHOTOGRAPHS



Run 1: 50' 9" ~ 52' 6" (15.47m ~ 16.00m) Run 2: 52' 6" ~ 57' 6" (16.00m ~ 17.53m)

Run 3: 57' 6" ~ 59' 1.5" (17.53m ~ 18.02m)



Run 3: 59' 1.5" ~ 61' 7" (18.02m ~ 18.77m)





Run 1: 58' 0" ~ 60' 1" (17.68m ~ 18.29m) Run 2: 60' 1" ~ 62' 3" (18.29m ~ 18.97m)

Run 3: 62' 3" ~ 64' 9" (18.97m ~ 19.74m)



Run 4: 64' 9" ~ 69' 8" (19.74m ~ 21.23m) Run 5: 69' 8" ~ 72' 9" (21.23m ~ 22.17m)



Run 5: 72' 9" ~ 74' 7" (22.17m ~ 22.73m) Run 6: 74' 7" ~ 77' 11" (22.73m ~ 23.75m)





Run 1: 52' 0" ~ 55' 0" (15.85m ~ 16.76m) Run 2: 55' 0" ~ 60' 0" (16.76m ~ 18.29m)



Run 3: 60' 0" ~ 65' 1" (18.29m ~ 19.84m) Run 4: 65' 1" ~ 67' 0" (19.84m ~ 20.42m)



Run 4: 67' 0" ~ 69' 11" (20.42m ~ 21.31m) Run 5: 69' 11" ~ 71' 10" (21.31m ~ 21.89m)





Run 1: 55' 10" ~ 59' 8" (17.02m ~ 18.19m) Run 2: 59' 8" ~ 63' 8" (18.19m ~ 19.41m)



Run 2: 63' 8" ~ 64' 9" (19.41m ~ 19.74m) Run 3: 64' 9" ~ 66' 9" (19.74m ~ 20.35m)





# **FOUNDATION DESIGN REPORT**

## **PROPOSED IVANHOE RIVER BRIDGE REPLACEMENT, HIGHWAY 101 AND 7172 (FOLEYET)**

**SITE LOCATION (Long. -82.448293°, Lat. 48.247514°)**

**PLANMAC ENGINEERING INC**

GWP 5266-13-00  
GEOCRES NO. 42B-013

**WSP PROJECT NO.: 171-02344-00**

**JUNE 10, 2019**

**WSP CANADA INC.**

**[WWW.WSP.COM](http://WWW.WSP.COM)**

# TABLE OF CONTENTS

<b>5</b>	<b>DISCUSSION AND RECOMMENDATIONS.....</b>	<b>19</b>
<b>5.1</b>	<b>GENERAL .....</b>	<b>19</b>
<b>5.2</b>	<b>GEOTECHNICAL CHARACTERISATION.....</b>	<b>19</b>
<b>5.3</b>	<b>ALTERNATIVE BRIDGE FOUNDATION OPTIONS .....</b>	<b>21</b>
<b>5.4</b>	<b>RECOMMENDED FOUNDATION OPTION .....</b>	<b>25</b>
<b>5.5</b>	<b>DESIGN DETAILS FOR ROCK-SOCKETED CAISSONS .....</b>	<b>25</b>
<b>5.6</b>	<b>APPROACH EMBANKMENTS .....</b>	<b>29</b>
<b>5.7</b>	<b>CONSTRUCTION CONSIDERATIONS.....</b>	<b>37</b>
	<b>REFERENCES .....</b>	<b>42</b>

## TABLES

5- 1 COMPARISON OF BRIDGE FOUNDATION ALTERNATIVES .....	23
5- 2 RECOMMENDED SOCKET SHAFT PARAMETERS .....	26
5- 3 RECOMMENDED SOCKET BASE PARAMETERS .....	26
5- 4 RECOMMENDED LATERAL RESISTANCE PARAMETERS .....	27
5- 5 GEOTECHNICAL PARAMETERS FOR LATERAL PILE RESISTANCE OF OVERBURDEN SOILS .....	28
5- 6 GEOTECHNICAL PARAMETERS FOR SLOPE STABILITY .....	30
5- 7 LONG-TERM STABILITY-CONVENTIONAL EARTHWORKS .....	31

5- 8 COMPARISON OF GROUND IMPROVEMENT STRATEGIES .....	32
5- 9 LONG-TERM STABILITY WITH CELLULAR CONCRETE.....	35
5- 10 INTERPRETED OHSR REQUIREMENTS FOR OPEN-CUT EXCAVATIONS.....	37
5- 11 GEOTECHNICAL DESIGN PARAMETERS (UNFACTORED) – FOR TEMPORARY SHORING .....	39

---

## FIGURES

FIGURE 1	SPT "N" VALUE VS ELEVATION (m)
FIGURE 2	RQD (%) FREQUENCY DISTRIBUTION
FIGURE 3	FRACTURE FREQUENCY VS FRACTURE INDEX
FIGURE 4	FRACTURE ANGLE FREQUENCY DISTRIBUTION
FIGURE 5a	POINT LOAD BASED UCS - DIAMETRAL (MPa)
FIGURE 5b	POINT LOAD BASED UCS - AXIAL (MPa)
FIGURE 6	DEPTH (m) BELOW BEDROCK VS POINT LOAD DIAMETRAL BASED UCS/UCS (MPa)

---

## APPENDICES

APPENDIX E	Global Stability Analysis
APPENDIX F	Settlement Interaction Analysis
APPENDIX G	List of OPSS/SPs/NSSPs
APPENDIX H	Extruded Polystyrene for Treatment
APPENDIX I	Ground Movements/Ground Vibrations Monitoring
APPENDIX J	Limitations of Report

# 5 DISCUSSION AND RECOMMENDATIONS

---

## 5.1 GENERAL

This section of the report addresses pertinent geotechnical design issues for the construction of the Replacement Bridge over Ivanhoe River on Hwy 101 at Foleyet in Northern Ontario. The replacement alignment is 10 m offset from the existing bridge centreline to the west. The existing bridge is a 3-span bridge of approximate length of 45 m. The existing highway is a two-lane undivided rural highway with partially paved shoulders. The abutment locations were based on the 90% GA drawings. Subject to proving of competent abutment foundation conditions, the proposed bridge will be a single span bridge over the river. The proposed approach embankments are 3.20 m and 2.83 m high (higher than the existing) at the south and north abutments respectively and will partially overlap with the existing embankment cross-section on the eastern side slope of the proposed embankment.

The discussions and recommendations presented in this report are intended to assist the designers with sufficient information that would enable them to proceed with the design of the structure foundations and approach embankments.

Construction comments made herein are based on geotechnical considerations only and should not be relied upon without further independent assessment and qualification in the selection of means and methods for construction.

The main thrust of the discussion and recommendations of this report will be on analysis of alternative bridge foundation options to suit project geology, subsoil conditions, groundwater conditions and to assess associated construction impacts on the highway. Valuable information available from the construction records of abutment pile driving of the original bridge construction in 1960 has been taken into consideration. Based on this assessment, recommendations will be made on a foundation option from a geotechnical perspective considering the viability in terms of costs as well, to facilitate the bridge replacement.

---

## 5.2 GEOTECHNICAL CHARACTERISATION

### 5.2.1 OVERVIEW OF SUBSURFACE CONDITIONS

As an overview, the native stratigraphy predominantly consisted of a silt deposit underlain by cohesionless till intercepted in all the boreholes overlying bedrock. The intercepted till was very bouldery and required intermittent coring in most of the boreholes. The intercepted quaternary geology closely resembles the geology reported in the literature (See Section 2.1). In isolated boreholes, only insignificant amounts of organics were contacted. This overview is based on the factual information in Section 4 of the FIR and the interpretation of that data relevant to the particular design issues discussed in this report.

The intercepted silt was dilatant, non-plastic and wet with an average moisture content of 24%. Grain size information revealed very high silt contents. Accordingly, based on the relevant MTO guidelines discussed in Section 4.3.5, the silt material can be ranked as of “High” frost susceptibility and “Most Erodible” on the erodibility classification, respectively. The deposit had typically a loose compactness.

Discontinuous very loose, sandy interbeds were intercepted in three boreholes. Issues such as the flowing sand condition observed within a sandy silt interbed in borehole BH 17-5 has important ramifications for construction.

The average thickness of the composite silt deposit (including the discontinuous interbeds) was 5.5 m. Figure 1 shows the SPT 'N' profile of this composite deposit at the project site and is indicative of very loose to compact state of compactness, but typically loose.

The dominant native soil deposit underlying the silt was a cohesionless till comprising sandy silt/silty sand / sand and silt. The intercepted thickness of the deposit ranged from 1.5 m to 14.8 m with an average of 5.7 m. It was predominantly sand and silt with numerous boulder beds, requiring intermittent rock coring through boulders to advance through this deposit. Limited occurrence of flowing sand conditions observed within the glacial till will have ramifications for deep foundation construction. More than 70% of the SPT probes indicated the deposit to be dense or better; however, the bouldery nature will likely would have influenced the recorded blow counts.

Bedrock proven in each of the boreholes at the proposed abutment locations is indicated to be of gneissic lithology as per geology maps. The elevations of top of bedrock intercepted at the proposed abutment locations imply a relatively flat bedrock surface within the explored boreholes. The weathering index was W2 to W1 throughout, i.e. Fresh to Slightly Weathered. The average RQD was 70% and Fig. 2 shows the RQD distribution and this indicates 70% of the data lies in the RQD range 50% to 90% and indicative of a rock mass of fair to good quality (as per Table 3.10 of CFEM). The 10<sup>th</sup> percentile, RQD is 44. The fracture index was predominantly 2 per 0.3 m or less (see Fig.3) which corresponds to close spacing based on Table 3.9 of CFEM. The UCS values based on average point load strength index test values were 109 MPa and 71 MPa for axial and diametral loading modes respectively, Fig. 5. Two UCS tests on intact rock specimens yielded uniaxial compressive strengths of 146 MPa and 39 MPa. The intact rock strength can be described as typically strong (as per Table 3.5 of CFEM), although the possibility of very strong bands (e.g. UCS of 146 MPa) of rock cannot be ruled out. The 10<sup>th</sup> percentile, diametral point load based UCS is 41 MPa. Based on the discussed bedrock properties, a Geological Strength Index (GSI) of 60 is estimated for the rock mass.

Shallow groundwater levels (long-term) were observed in piezometers installed one at each proposed abutment location. The groundwater level in the piezometer on the south side is indicative of sub-artesian groundwater conditions within the fractured bedrock.

#### Ground Motion Parameters

The subject site is located in a part of the stable interior of the North American Plate. Based on the borehole information and our review of the general subsurface conditions in the area, the subject site for the proposed structures can be classified as 'Class D' for seismic site response according to Table 4.1 of CSA S6-14.

The Peak Ground Acceleration for the Town of Foley (based on Town of Chapleau), Ontario for 2% / 50-year return probability based on National Building Code of Canada (NBCC2015) and Canadian Bridge Design Code (CSA S6-14; hereinafter referred to as S6-14) is 0.041g. The site adjusted amplification factor F(PGA) based on S6-14, Table 4.8 is 1.29. Accordingly, PGA (Site Adjusted) is equal to 0.06.

Accordingly, for pseudo-static analysis for non-yielding walls, the following ground acceleration parameter is appropriate:

- a  $k_h$  value of 0.06

The seismic performance category (for 2475 yr return period) according to S6-14 for the proposed single span bridge in Ivanhoe ( $S(0.2) = 0.071 < 0.2$  and  $S(1.0) = 0.031 < 0.10$ ) is category 1. Based on Section 4.6.1 of S6-14, no further seismic foundation issues need to be addressed for seismic category 1 single span bridges.

### Frost Depth/Susceptibility

The frost depth for the project site is 2.3 m based on the MTO Foundation Frost Depths for Northern Ontario, **OPSD 3090.100**. The predominant native deposit closest to the existing ground level (intercepted at depths between 0.1 m to 2.6 m below existing ground level), Silt, has 'HIGH' frost susceptibility based on grain size information as per the MTO Frost Classification criteria.

---

## 5.3 ALTERNATIVE BRIDGE FOUNDATION OPTIONS

### 5.3.1 GENERAL

Based on the ground conditions encountered in the abutment boreholes, the loose, dilatant silt deposit is considered not competent for any shallow foundations even with a granular pad compounded by the proximity of the abutment axes to the river bank. The potential competent founding strata are the underlying till and the bedrock. Hence, any form of deep foundation must of necessity take into account the influence of cobbles and boulders that were intermittently cored with diamond drilling within the till layer. Pile driving details available from the original construction (1960) of the existing bridge reveal the resistances met by timber piles at the existing bridge abutments (See Section 2.2). It should also be taken into account the impact of any foundation construction methodology, given the proximity of the proposed abutments to the river bank. Based on the 90% GA drawings the offsets are 3.7 m at the south abutment and 2.6 m at the north abutment. The impact of any hard pile driving, for example, on the loose, wet, dilatant silt comprising the surficial native deposit of 5.5 m average thickness would also need careful assessment. Given the very bouldery nature of the till, reliance on pile refusal for driven piles can also be circumspect. Section 5.3.2 evaluates different deep foundation options based on site specific issues relating to subsurface conditions discussed and taking into account the potential impact of these conditions on the construction of alternative deep foundation types.

### 5.3.2 DEEP FOUNDATIONS

A number of deep foundation options which are common in Ontario have been considered as follows:

#### 5.3.2.1 DRIVEN STEEL TUBE PILES/ DRIVEN STEEL H-PILES

Driven steel piles in bouldery till geology can be exposed to uncertainty about refusal interpretation for end bearing piles (See Fig. 1, BH17-1 for decreasing SPT 'N' profile in till; further Borelog, BH17-1 also shows interbedded boulder beds which required rock coring). Although BH17- was located on the existing north abutment, the findings are still relevant for the general site sub-soil conditions.

Difficulties of separating premature pile refusal on boulders (possibly exacerbated by plug formation with open-ended tube piles) and reliability of PDA type of pile testing in such intermittent boulder bed terrain as the subject till deposit do not give a practitioner reasonable confidence in the use of driven steel piles for a 45 m span bridge foundation. Further, the possibility cannot be ruled out that the driven piles can meet refusal at different elevations within the same abutment. For example, at the south abutment, BH17-4 and BH18-1 would indicate that the driven steel piles can refuse at elevations with a difference of more than 2.5 m (estimated refusal at BH 17-4: El. 308.5 and at BH18-1: between El. 310.5 m and El. 311 m).

Attempts to minimize elevation differences with hard driving (e.g. with a heavier steel H-section) even reinforced with rock points can lead to pile damage. Also, if on premature refusal (giving not enough confidence for reliance on such piles), if additional piles need to be driven, then pile bent geometry may pose a restriction leaving aside possible contractual implications during construction. In addition, any attempt to significant hard driving within the till could adversely impact the river bank stability, given the close proximity of the proposed abutments to the respective river banks. This will be exacerbated through

wave amplification induced by heavy vibrations in the underlying dense till deposit, resulting in potential susceptibility of the silt deposit to liquefaction.

The possibility of driven steel piles pre-bored into was also explored. The extent of pile embedment that can be achieved with subsequent driving is difficult to estimate given the findings of BH 17-5A where almost continuous rock coring through the boulders had to be undertaken.

Hence use of driven steel piles is not considered further as a viable foundation option for a 45 m span bridge under consideration.

#### 5.3.2.2 AUGER PRESS CONCRETE PILES (CFA PILES)

This pile type requires a high degree of construction control (given flowing sand conditions, dilatant loose silts) on the relative rates of auger extraction and concrete intrusion and possible auger grinding on boulder beds make the adoption of CFA not particularly suited under the subject geological conditions. Coupled with the inability to resist significant lateral/uplift loads, makes CFA piling unsuitable for the given ground conditions.

#### 5.3.2.3 DRILLED CAISSON FOUNDATIONS

Given the relatively shallow bedrock conditions, drilled caissons end bearing or socketed into bedrock, in principle, can be considered for the design of foundations under the given geological conditions. However, there are important construction considerations that need to be addressed to make the adoption of drilled caissons viable. With the adoption of caissons, one of the drawbacks is the use of integral abutments will be not feasible due to lack of flexibility of the piling type to move with thermal cycling loads of the bridge superstructure. However, caissons can be used with semi-integral abutments.

Table 5.1 compares the proposed bridge foundation options.

**TABLE 5-1 COMPARISON OF BRIDGE FOUNDATION ALTERNATIVES**

Pile Type	Advantages/Disadvantages	Risks/Consequences	Recommendations
DRIVEN STEEL TUBE PILES/ DRIVEN STEEL H-PILES	<p>Advantages:</p> <ol style="list-style-type: none"> <li>1. Widely used in Ontario</li> <li>2. Cost-effective</li> <li>3. Amenable for integral bridge abutments</li> </ol> <p>Disadvantages:</p> <ol style="list-style-type: none"> <li>1. Possibility of pile refusal at significant elevational differences and reservations on computational ability to assess the impact of such scenarios</li> <li>2. Lack of confidence in reliability of PDA type of indirect pile capacity assessments in boulder beds</li> <li>3. Possible pile deflections due to sizeable boulders</li> <li>4. Potential impacts of pile driving on river banks' stability and potential vulnerability of the silt deposit to heavy construction vibrations</li> </ol>	<ol style="list-style-type: none"> <li>1. Driven steel piles in bouldery till geology can be exposed to uncertainty about refusal interpretation for end bearing piles (See Fig. 1 BH17-1 SPT profile); although BH17- was located on the existing north abutment, the findings should be still relevant for the general sub-soil conditions</li> <li>2. Undetected pile damage and pile deflections</li> <li>3. Uncertainty of PDA type of pile capacity assessments in boulder geology</li> <li>4. Need for additional driven piles due to uncertainty on reliance of pile capacity can lead to design constraints with pile cap geometry and possible contractual issues</li> <li>5. Risks of river bank instability due to hard driving</li> </ol>	<b>Not recommended</b>



**TABLE 5-1 COMPARISON OF BRIDGE FOUNDATION ALTERNATIVES**

Pile Type	Advantages/Disadvantages	Risks/Consequences	Recommendations
<b>AUGER PRESS CONCRETE PILES (CFA PILES)</b>	<p>Advantages:</p> <ol style="list-style-type: none"> <li>1. As a kind of bored pile, it has better control of soil loosening around the bored hole. And hence better control of hole stability, for example, in flowing sand/dilatant loose silt situations</li> </ol> <p>Disadvantages:</p> <ol style="list-style-type: none"> <li>1. Possible limitations of augering through boulder beds</li> <li>2. Greater demand on construction control in the retrieval of auger through flowing sands/loose dilatant silt</li> <li>3. Limitations on insertion depths of reinforcing cage into green concrete in the bored hole</li> </ol>	<ol style="list-style-type: none"> <li>1. Limitations on achieving required pile lengths</li> <li>2. Undetected potential pile necking issues in flowing sand zones/loose dilatant silt due to possible variation in construction control in auger retrieval</li> </ol>	<b>Not recommended</b>
<b>DRILLED CAISSONS</b>	<p>Advantages:</p> <ol style="list-style-type: none"> <li>1. With specialized drilling gear cased holes through boulder terrain can be advanced onto rock and <b>pile alignment issues can be managed</b></li> <li>2. Rock socketed solutions can yield much <b>higher and reliable foundation loads</b></li> <li>3. <b>Relatively shallow</b> competent <b>bedrock</b> intercepted (within 20 m)</li> </ol> <p>Disadvantages:</p> <ol style="list-style-type: none"> <li>1. Tremie concreting methods need to be used.</li> <li>2. Casing removal through flowing sands can cause soil intrusion into pile shaft; With sacrificial casing this risk can be managed.</li> <li>4. Contractor pool with the right type of gear and experience will be limited in Northern Ontario</li> </ol>	<ol style="list-style-type: none"> <li>1. Higher construction costs</li> <li>2. Experience in piling in such heavy bouldery terrain is not widespread</li> </ol>	<b>Recommended</b>

---

## 5.4 RECOMMENDED FOUNDATION OPTION

Based on the foregoing discussion on bridge foundation alternatives and the tabular comparison, the adoption of rock socketed drilled caissons with sacrificial liners is the recommended foundation option given the site geology comprising boulder till /flowing sands/dilatant silt and relatively shallow competent bedrock. Suggested wording for an **NSSP** is provided in Appendix G to “red flag” the need for use of sacrificial liners.

---

## 5.5 DESIGN DETAILS FOR ROCK-SOCKETED CAISSONS

### 5.5.1 ROCK SOCKET DESIGN PHILOSOPHY

In a rock socketed caisson foundation, load is carried both in shaft and on the socket base. However, the socket resistance mobilizes with relatively small shear displacement between the socket wall and the caisson shaft (of the order of 0.25 – 1% of the pile diameter with respect to the surrounding rock) unlike base resistance which typically requires base settlement of the order of 10% - 20% of the pile diameter to mobilize ultimate base resistance. Given the difficulties of ensuring the socket bases are clean under a head of water and workplace safety requirements restraining human inspection of socket bases, rock socketed foundations are generally designed to carry the SLS load on the socket shaft and the contribution of the socket base in end bearing is exploited to address the axial capacity difference between the ULS and SLS loading. Any load carried by the shaft side resistance will result in a small settlement, but any load carried in end-bearing generally results in higher settlement. Apart from socket cleaning and restraints on human inspection, this is another reason to carry the SLS load on the socket shaft.

In view of the WSP recommendation to use sacrificial liners to safeguard against potential necking intrusions on the green concrete shafts likely to arise on liner withdrawal as a result of intermittent flowing sands as encountered during WSP field investigations, axial compression resistance needs to be derived from the rock socket shaft. Similar reasoning will apply for uplift resistance. This load sharing approach is recommended. In view of the likely socket base inspection limitations, the use of belled rocket sockets is not recommended.

Design parameter recommendations are compliant with the geotechnical resistance factors of S6-14 Table 6.2. A socket diameter of 1.2 m is recommended. Centre- to -centre spacing of drilled caissons at the abutments must be a minimum of 2.5 diameters (Clause 6.11.4.7, S6-14). In view of the potential for very strong bedrock layers to be intercepted (based on the rock strength testing results), this could have a significant impact on socket drilling production and hence larger socket diameters are not favoured.

Construction considerations are discussed in Section 5.7.

### 5.5.2 ROCKET SOCKET PARAMETERS

#### 5.5.2.1 SHAFT RESISTANCE

As very strong bands of rock cannot be ruled out based on intact rock strength results within bedrock, sockets are likely to be machine drilled that would make the sockets very smooth. In view of this, socket shaft parameters are based on smooth socket wall conditions. In particular, the 10% percentile UCS of 41 MPa was used as the characteristic strength of the intact rock (see Section 5.2.1) and UCS of concrete was taken as 30 MPa (taking into consideration, shaft concreting under tremie pours). Since, no interface can have more strength than the weakest of the two materials it separates, the UCS of concrete will govern

interface behaviour in the present situation. Based on Horvath (1983) and Zhang and Einstein (1998), the following socket shaft parameters are recommended. Fig. 6 shows the point load diametral based UCS values plotted with depth below the top of the bedrock surface. The diametral based values obtained on vertical rock cores were chosen since these values were lower than the axial based UCS values. Further, diametral based UCS values are more relevant for socket shaft considerations as the rock socket shaft is loaded by the pile compression in a radial manner. Also shown are the UCS values based on two intact samples. The adopted design line (the 10<sup>th</sup> percentile) is also shown which gives a reasonable lower bound to the strength values. Based on this observation, the total shaft length, i.e. can be taken as the effective shaft length.

**Table 5.2: Recommended Socket Shaft Parameters**

Limit State	Shaft Resistance in Axial Compression (MPa)	Shaft Resistance in Axial Uplift (MPa)
ULS (factored)	0.51	0.38
SLS	0.41	0.31

Based on the above socket shaft parameters, a socket shaft of 1.2 m diameter and 2.4 m in length, should resist 4617 kN as factored ULS and 3693 kN as SLS load in axial compression. As discussed in Section 5.5.1, it is recommended to carry the axial SLS loads on the rocket socket shaft.

#### Cone Pull-Out:

Fig. 4 shows the distribution fracture angles measured on rock cores (from the core axis, i.e. from the vertical, since boreholes were vertical) and the most dominant fracture angle is seen to be 30 degrees from the vertical. Based on this observation, a pull-out cone of semi-apex angle of 30 degrees is adopted as the most probable pull-out cone geometry. From the rock head a vertical cylindrical failure surface emanating from the cone is adopted as the full pull-out body. Based on Table 4-5, the minimum overburden depth to the rock head is taken as 15 m and was assumed to in equal thicknesses of silt and till. Using effective unit weights of 16.3 kN/m<sup>3</sup> for bedrock (based on UCS testing of rock cores), 7 kN/m<sup>3</sup> for the silt and 12 kN/m<sup>3</sup> for the till, the total buoyant weight of a 2.4 m long socket (i.e. **2x socket diameter**) is 850 kN (with FoS of 1.1).

As this is a single span bridge with no skew, foundation uplift forces are not considered to be significant.

#### 5.5.2.2 BASE RESISTANCE

Subject to good construction control on the cleanliness of socket bases, based on Ladanyi and Roy (1971) and Zhang (2010) and using the rock properties discussed in Section 5.2.1, the following socket base end-bearing parameters are recommended.

**Table 5.3: Recommended Socket Base Parameters**

Limit State	Axial Compression in End Bearing (MPa)
ULS (factored)	6.8
SLS	5.0

The structural capacity of the caisson must be checked by the structural engineer. Downdrag is not considered an issue with rock socketed foundations with sacrificial liners.

As discussed in Section 5.5.1, it is recommended to only carry the applied load difference between the factored axial ULS load and the SLS load on the socket base.

### 5.5.2.3 ROCK SOCKET SETTLEMENTS

A deformation modulus of 0.69 GPa is recommended for the rock mass for rock socket settlement calculations. Using Rowe and Armitage (1987) (CFEM Fig. 18-15), a rock socket (1.2 m dia. and 2.4 m length) carrying a SLS load of 3693 kN as shaft resistance is estimated to settle less than 3 mm including concrete compression of the pile. A Young's modulus of 25 GPa for concrete and length of concrete shaft of 20 m (estimated from the base of the abutment to the tip of the rock socket) were assumed for the above estimation.

## 5.5.3 LATERAL RESISTANCE OF SUBSOILS

### 5.5.3.1 ROCK SOCKETS

Ultimate lateral resistance of the rock socket was estimated using the method of Zhang, et. al. (2000) and was based on a socket diameter of 1.2 m.

**Table 5.4: Recommended Lateral Resistance Parameters – Rock Sockets**

Limit State	Lateral Resistance (MN/m)
ULS (factored)	2.5
SLS	1.9

### 5.5.3.2 OVERBURDEN SOILS

Geotechnical considerations pertaining to lateral capacity and lateral deformations of overburden soils are addressed in the following with respect to lateral loading of piles:

- Lateral capacity of piles: As the intercepted subsoils are cohesionless the passive earth pressure coefficient,  $K_p$  and the effective unit weight of soils address pile lateral capacity issues
- Lateral deformation of piles: the coefficient of horizontal subgrade reaction,  $k_s$  (using  $n_h$  for cohesionless soils) provide the input for lateral deformation analyses

Vertical piles can provide resistance to lateral loading. In cohesionless soils, the coefficient of horizontal subgrade reaction can be estimated from:

$$k_s = n_h z / d$$

where  $k_s$  = coefficient of horizontal subgrade reaction

$n_h$  = coefficient related to soil density as given in **Table 5.5**

$z$  = depth

$d$  = pile diameter

**Table 5.5: Geotechnical Parameters for Lateral Pile Resistance -Overburden Soils**

Location	Elevation (m)	Soil	$\eta_h$ MN/m <sup>3</sup>	$K_p$	Buoyant Unit Weight kN/m <sup>3</sup>	Remarks
South Abutment	320.8 to 313.2	Surficial soils & Silt	NA	NA	7	Loose, wet dilatant silt
	313.2 to 310.2	Upper Cohesionless Till	4	3.2	10	
	310.2 to 307.8	Lower Cohesionless Till	11	4.5	12	
North Abutment	320.9 to 313.8	Silt	NA	NA	7	Loose, wet dilatant silt
	313.8 to 303.0	Cohesionless Till	11	4.5	12	

Note: The design water level is assumed at the ground surface for lateral pile resistance considerations

Reference should be made to Clause 6.11.2.2 of S-14, for assessing lateral pile resistance.

Factored lateral resistance at SLS shall be determined based on the serviceability requirements of the structure.

Spring Stiffness Values -Lateral Loading:

The spring stiffness values for structural analysis purposes can be obtained as follows:

$K_{spring} = k_{subgrade\ modulus} * d * \Delta l$  where d is the diameter of the pile and  $\Delta l$  is the elemental length of the foundation.

### 5.5.3.3 GROUP EFFECT ON PILE LATERAL RESISTANCE

Given the high load carrying capacity of rock sockets, the piles would be in a single row and any lateral load, given a straight single span bridge, will likely be perpendicular to the line of rock sockets, i.e. will be in the longitudinal direction. Therefore no 'shadow effect' will manifest to compromise the lateral capacity of a single row of rock sockets.

## 5.5.4 FROST CONSIDERATIONS

For frost protection, all pile caps should have a permanent earth cover of at least 2.3 m (frost depth for the general project area) or an equivalent thickness of insulation. According to MacMaster and Wrong (1988), the project site is located in the region of the 1750 Freezing Index contour. Based on the contour information, a minimum insulation thickness of 120 mm is estimated (as per Fig. 13.11 of CFEM 2006). Therefore, a minimum 120 mm thick extruded polystyrene rigid board insulation should be placed below the underside of the pile cap if the pile cap is designed to be located within the frost depth, see sketch in Appendix H. Specialist supplier instructions should be followed for specific installation requirements.

The Contract Drawings should address this requirement for the soil insulation. The suggested wordings are included for reference in Appendix H.

---

## 5.6 APPROACH EMBANKMENTS

### 5.6.1 OVERVIEW

The proposed south approach embankment at the abutment location, based on the GA drawing, has a height of 3.20 m while the corresponding height at the north abutment is 2.83 m. They also have different offsets from the abutments to the respective crests of the river bank along the proposed centreline of the bridge with 3.7 m on the south and 2.6 m on the north. River bank slopes based on the GA drawing are variable on the south with the steepest slope of 1H:1V at the bank crest and 2H:1V on the north. Hence it is not immediately clear which approach is more vulnerable from a stability point of view. As a consequence, both approaches were investigated for slope stability. Slope stability in the longitudinal direction was investigated under different limit states that may arise in the life of the highway embankment. The slope stability of the proposed approaches transverse to the proposed centreline is not considered more critical in the absence of a river impact and hence was not investigated in detail but recommendations on stable slopes are made based on engineering judgement.

The abutments of the existing bridge are founded on driven timber piles (constructed in 1960) and their present condition is unknown in so far in its ability to resist additional loading. According to the available pile driving records that were discussed in Section 2.2, it appears they were driven to refusal in what we now know as glacial till with significant but random spatial distribution of cobbles and boulders. What damage did take place in the timber piles especially the pile toes on hard driving is not known. The original piling records indicate the piles had dropped significant heights on initial hammer blows. At that time, it was thought the rapid advancement of the piles were due to the presence of muskeg as muskeg is generally known to occur in the subject area as local patches according to the geological studies (See Section 2.1). Based on the ground information obtained under this assignment, we have found no evidence of muskeg in nine boreholes that were advanced in the present investigation. The borehole evidence we have now indicate that it is very likely the reported rapid pile advancements would have been as a result of the loose, wet dilatant silt undergoing loss of shear strength induced by pile driving. This silt deposit was intercepted on both banks. However, this rapid advancement of timber piles was reported more during the north abutment piling.

The impact of the close proximity of the proposed abutments to the river banks has resulted in the proposed embankment approaches with conventional earthworks to be unstable. Therefore, ground improvement schemes appropriate to the local soil conditions were investigated with particular attention being paid to the reported pile driving experience of the original construction on assessing the effectiveness of the schemes. As the proposed embankment cross-section overlaps on the existing approach embankments, it was also recognized that any proposed embankment loading and/or ground remediation methodology should not impose significant additional stresses on the existing abutment piles as the existing bridge needs to be operational during the proposed construction serving as a “detour”. It was realized that the most reliable way to minimize additional stresses on the existing abutments was to adopt a light-weight fill approach of very limited extents behind the abutments. A detailed quantitative assessment of this interaction would require a full 3D numerical assessment. This is considered beyond the scope of the present assignment. Therefore, a rather cautious approach was pursued.

Taking the above considerations into account, settlement analyses were undertaken in an approximate 3D framework to estimate the settlements on the existing approaches and near the existing abutments, using Settle 3-D software (Rocscience Inc.). Slope stability was investigated using the limit equilibrium method of stability analysis based on the Morgenstern- Price method using SLIDE software (Rocscience Inc.).

No proposed retaining walls were identified in the GA drawings other than the abutment and hence alternative options for retaining walls are not addressed.

A detailed discussion of the issues raised in the overview is given below.

## 5.6.2 GLOBAL SLOPE STABILITY

### 5.6.2.1 OVERVIEW

Geotechnical Parameters for slope stability analysis are given in Table 5.6. They are based on SPT 'N' results, engineering judgement and contemporary engineering literature. As a further check on the appropriateness of the strength parameters given in Table 5.6, the global stability of the existing north abutment was investigated. After proving the geotechnical strength parameters, the stability of the proposed approach embankments was investigated. All global stability analyses for the proposed embankments are analysed for 2H:1V side slopes where applicable.

The following geotechnical limit states were investigated for stability analysis:

- Long-term stability – with long-term river stage (the stage was set at the 10-yr flood level based on TPM's hydraulic expert's advice)
- Seismic stability
- Rapid drawdown

Possible groundwater flow due to differences in the observed groundwater levels in the piezometers and the adopted river stage was modelled within the stability model as provided for in the SLIDE software. This ought to give more accurate stability results than basing analysis on guesstimated phreatic surface trajectories.

End-of-construction stability was not investigated given the cohesionless nature of the subsoils, as undrained conditions cannot develop in free-draining soils. All global stability results are in **Appendix E**.

**Table 5.6: Geotechnical Parameters for Slope Stability**

Location	Degree of Compactness	Unit Weight kN/m <sup>3</sup>	Effective Stress Parameters: c', $\phi'$ (degrees)	Drained Stiffness E' (MPa); $\nu'$
Pavement Fill	Compact	22	1,34°	30,0.25
Embankment Fill	Compact	20	1,32°	25;0.3
Rockfill	Compact	18	0,40	25,0.3
Sand/silty sand	Loose	18	0,30°	15;0.3
Non-Plastic Silt	Loose	17	0,28°	10;0.3
Cohesionless Till	Dense	22	0,40°	80;0.25

### 5.6.2.2 GEOTECHNICAL STRENGTH MODEL CALIBRATION FOR STABILITY ANALYSIS

The existing north abutment was chosen for the ground model calibration for two reasons:

- The existing bank has the shorter offset to the river bank crest out of the two existing approaches
- It is indicated that the original pile driving recorded most of the rapid pile advancements on this approach piling

The ground model of the existing north abutment is shown in Fig. E1.1a. Global stability results are shown in Fig. E1.1b and yielded a FoS of 1.26.

By contemporary geotechnical design standards (typically a FoS of 1.5 is used), the obtained FoS is considered low for an abutment forward stability. However, the obtained FoS value does not indicate impending global instability. It is unknown, conforming to what code of practice standards the existing abutments have been built during the original construction. The global stability of the existing abutment does not conform to the current S6-14, Table 6.2 requirements (a minimum FoS of 1.54 is required for permanent global stability when the degree of understanding of the ground is “Typical”). The new standard also has increased surcharge pressure for live load in addition to the slightly higher global stability standards imposed. Since the proposed abutments are higher than the existing and also closer to the river bank, it became clear that the proposed abutment approaches comprising conventional earthworks will not be able to satisfy S6-14 requirements. It became evident that with the proposed GA abutment fixing, some form of ground improvement would be necessary to satisfy the new code requirements.

### 5.6.2.3 LONG-TERM STABILITY

For long-term stability analyses, the 10-year river stage (i.e. flood return period of 10 years; water level elevation of 320.07 m.) was used for the river flow stage. The proposed abutments were analyzed for global stability based on the above geotechnical model and the following stability results were obtained (see Appendix E2 for stability results):

**Table 5.7: Long-Term Stability (Fore Slope) - Conventional Earthworks**

Embankment Construction Material	Additional Ground Improvement	FoS		Remarks
		South Abutment	North Abutment	
Conventional Earthworks	Nil	0.87 See Fig. E2.1a	0.85 See Fig. E2.1b	Does not comply with S6-14, Minimum FoS=1.54
Conventional Earthworks	Basal Reinforcement (BR)	0.90 See Fig. E2.2a	0.91 See Fig. E2.2b	
Conventional Earthworks	BR+Rockfill Toe Buffer	1.21 See Fig. E2.3a	1.34 See Fig. E2.3b	
Rockfill	Nil	0.94 See Fig. E2.4a	0.93 See Fig. E2.4b	

Basal inclusions are passive reinforcement inclusions and thus require a bond length to develop the interface shear stresses. With a blade cantilever abutment wall, this bond length as would develop under a earthfill forward slope is not available. This is amply reflected in the most critical slip surfaces almost touching the abutment walls. In the light of the approach embankment methodologies with conventional earthworks as well as rockfill not being able to meet the current S6-14 standards, more effective ground improvement strategies were explored, namely the use of rammed aggregate pier columns, the use of light-weight fill. Two light weight fill products were investigated, namely cellular concrete and expanded polystyrene (EPS). Cellular concrete contains a foaming agent that creates air voids within the concrete and thus makes it significantly lighter than earthworks. The two lightweight fill materials considered are listed on the MTO’s Designated Sources of Materials List and



satisfies MTO's prequalification requirements. Table 5.8 compares the relative merits of the different ground improvement approaches.

**Table 5.8: Comparison of Ground Improvement Strategies**

Ground Improvement Strategy	Advantages/Disadvantages	Risks/Consequences	Remarks
Rammed Aggregate Piers	<p>Advantages:</p> <ol style="list-style-type: none"> <li>4. Widely used in Ontario</li> <li>5. Cost effective</li> </ol> <p>Disadvantages:</p> <ol style="list-style-type: none"> <li>1. Resistance to load is developed by lateral bulging in the upper few diameters of the rammed column.; however, the loose, wet dilatant silt is susceptible to provide this resistance under rammed vibrations.</li> <li>2. High water table can require cased boreholes</li> <li>3. Ramming operations very close proximity to the river bank may impact on river bank stability</li> <li>4. Will not reduce the lateral load effects on the existing abutments</li> </ol>	<ol style="list-style-type: none"> <li>1. Silt not being able to provide passive restraint under the disturbance brought by ramming</li> <li>2. Possible impacts on river bank stability and consequent impact on existing abutments</li> <li>3. Ramming operations very close proximity to the river bank may impact on river bank stability</li> <li>4. Will not contribute to reducing lateral load impacts on existing abutment piling</li> </ol>	<b>Not Recommended</b>
Expanded Polystyrene (EPS)	<p>Advantages:</p> <ol style="list-style-type: none"> <li>1. Very light weight</li> <li>2. No construction disturbance on underlying silt or induce instability on the river banks</li> <li>3. No perceptible lateral stress impacts due to polystyrene weight on the existing abutment piles</li> </ol> <p>Disadvantages:</p> <ol style="list-style-type: none"> <li>1. Expensive; labour intensive; time consuming especially if irregular geometries are involved</li> </ol>	<ol style="list-style-type: none"> <li>1. Risk due to high flammability to hydrocarbons</li> <li>2. Risk of extreme buoyancy</li> </ol>	<b>Feasible</b>

	<ol style="list-style-type: none"> <li>2. Special protection required due to its high flammability</li> <li>3. Little resistance to buoyancy effects</li> <li>4. Final embankment mass is a disjointed assembly of blocks</li> <li>5. Cannot be used to anchor road furniture</li> </ol>		
Cellular Concrete	<p>Advantages:</p> <ol style="list-style-type: none"> <li>1. Relatively light weight (half the unit weight of water)</li> <li>2. Less disturbance on underlying silt and reduced impact on the river banks</li> <li>3. Self-compacting</li> <li>4. Reduced lateral stress impacts due to lighter weight of cellular concrete on the existing abutment piles</li> <li>5. Relatively faster construction</li> <li>6. Resistance to freeze thaw</li> <li>7. Irregular geometries can be managed easily compared to polystyrene</li> <li>8. Final embankment mass is a monolithic product</li> <li>9. Cellular concrete is poured in lifts and each lift is preceded by an already hardened cellular concrete lift which is used to support formwork for the currently poured lift, i.e. stepped-pyramidal in geometry</li> <li>10. Can be used to anchor/embed road furniture such as lighting poles</li> <li>11. Low permeability and water absorption</li> <li>12. Resistant to hydrocarbons</li> <li>13. In view of the hardened monolithic nature of cellular concrete vertical faces are self- standing and thus does</li> </ol>	<ol style="list-style-type: none"> <li>1. Extra containment required to prevent environmental impacts during construction especially near a river</li> <li>2. Buoyancy issues but less compared to EPS</li> </ol>	<b>Feasible and Recommended</b>

	<p>not require any retaining wall support.</p> <p>14. Fresh cellular concrete is highly flowable and can be pumped into place over large distances through flexible hoses.</p> <p>Disadvantages:</p> <ol style="list-style-type: none"> <li>1. Expensive</li> <li>2. More protection is required to contain cellular concrete during pours from environmental impacts</li> <li>6. Better resistance to buoyancy effects than polystyrene</li> <li>7. Staging area for plant may be more compared to polystyrene</li> <li>8. Will require erosion protection</li> <li>9. Differential icing of the pavement surface but can be addressed by using engineered fill (OPSS Granular A) to form the sub-base and subgrade</li> <li>10. Need extra environmental controls to ensure river water is not impacted</li> </ol>		
--	--	--	--

We have used cellular concrete (comparatively heavier than expanded polystyrene), which is the recommended choice, with typical material properties and investigated the global stability requirements. MTO has previous experience with the use of cellular concrete at bridge abutments (Ahmad et.al. (2016)). The results of the stability investigation are reported as follows:

**Table 5.9: Global Stability with Cellular Concrete ( $C' = 100 \text{ kPa}$ ,  $\Phi' = 0^\circ$ ;  $\gamma = 5 \text{ kN/m}^3$ )**

Embankment Type	Geotechnical Limit State	FoS		Required Cellular Block Length (m)		Remarks
		South Abutment	North Abutment	South Abutment	North Abutment	
Conventional Earthworks + Cellular Concrete	Long-Term Stability	1.54 See Fig. E2.5a	1.56 See Fig. E2.5b	6.1	6.6	Does comply with S6-14 minimum FoS=1.54
	Rapid Drawdown	1.41 See Fig. E3.1a	1.44 See Fig. E3.1b			Drawdown of 50 yr flood level to 10 yr flood level in one day
	Seismic Stability	1.31 See Fig. E4.1a	1.32 See Fig. E4.1b			$k_h$ value of 0.06 used
Rockfill + Cellular Concrete	Long-Term Stability	1.56 See Fig. E2.4c	1.56 See Fig. E2.4d	6.0	6.5	Does comply with S6-14 minimum FoS=1.54
	Rapid Drawdown	1.44 See Fig. E3.1c	1.46 See Fig. E3.1d			Drawdown of 50 yr flood level to 10 yr flood level in one day
	Seismic Stability	1.32 See Fig. E4.1c	1.32 See Fig. E4.1d			$k_h$ value of 0.06 used

Table 5.9 reveals that the use of cellular concrete will enable the achievement of safety factors required by the S6-14. These safety factors at the proposed abutments also would imply that soil yielding beneath the proposed abutments are not expected to be widespread. This has the positive effect of restraining/reducing lateral thrusts on the existing abutments. This table further indicates the use of rockfill behind the cellular block as far as global stability is concerned almost requires the same length of cellular block length, although requiring a marginally shorter block length by 0.1 m at both abutments. It is our understanding that rock excavations will be required along the project alignment for construction of various other works and it is proposed to use rockfill as embankment fill on this project. Therefore foundation settlement analyses discussed in Section 5.6.3 will be based on use of rockfill behind the respective cellular blocks.

In the analysis of global stability with the cellular concrete block, a 0.5 m thick pavement structure was also incorporated on top of the cellular concrete block. The pavement structure overlying the cellular block will indirectly address the differential icing side effect.

In view of the proposed use of cellular concrete behind abutments, no lateral earth pressures will manifest on the abutment walls as cellular concrete is cast in incremental lifts and each lift hardens overnight to be self- standing.

To enable good interlock between the proposed embankment overlapping on the western side slope of the existing embankment, the existing side slope should be benched as per **OPSD 208.010** and care should be exercised not to cause local slope

failures/slumping of the existing embankment due to excessing intrusion/encroachment by excavation. At each proposed abutment, the cellular concrete blocks will transition into conventional earthworks. As cellular concrete is poured in successive lifts to form a stepped-pyramidal geometry, the conventional earthworks will have stepped benching formed by the cellular concrete structure to promote interlocking.

Side slopes of rockfill earthworks beyond the cellular blocks should not be steeper than 2H:1V global stability was found acceptable (See Fig. E5).

A drainage outlet should be provided at the interface between the cellular block and the abutment wall and also at the interface with the existing embankment side slope to guard against any hydrostatic pressure build-up against the abutment walls or laterally against the cellular block along the interface with the existing side slope.

### 5.6.3 APPROACH EMBANKMENT SETTLEMENTS

Approach embankment settlements were based on cellular concrete/rockfill option as this option requires marginally less cellular concrete block lengths and rockfill will be produced as part of the excavations on this project. A quasi-3D settlement analysis was undertaken to investigate the settlement interaction issues at the north abutment. The presence of the river was simulated by a pseudo excavation. A cellular concrete block of 6 m length was modelled behind the abutment, and beyond that to the north, a rockfill embankment was simulated (See Fig. F1a in **Appendix F**). Although the native non-plastic silt deposit is fast to consolidate under embankment loading, any settlements produced can potentially impact the existing bridge abutment foundations. The existing bridge abutment foundations have not been investigated in this project. Hence it is important to ensure minimization of impacts on the existing foundations produced by the proposed abutment/approach constructions.

Based on the analysis, it is seen the maximum settlement along the line of the existing abutment is less than 10 mm due to the proposed embankment construction (See query line 1 on Fig. F1b). Further, the maximum settlement along a cross-section (perpendicular to the centre line) placed within the proposed conventional earthworks behind the cellular concrete block is less than 40 mm (See query line 3) and occurs within the proposed earthworks. Negligible settlement interaction is shown along the part of the cross-section anywhere within the existing embankment (See segments of query lines 1 and 3 over the existing approach in Fig. F1b). Fig. F1c shows the 3-D settlement profiles as settlement planes.

### 5.6.4 SUMMARY

Approach embankment stability investigated has revealed that in order to achieve the margin of safety as required under S6-14, the inclusion of positive ground improvement measures would be needed. Use of cellular concrete was found to be the most appropriate out of a few alternatives discussed and is recommended. With 6 m long cellular concrete block at the Foleyet end and 6.5 m long cellular concrete block at the Timmins end with the same side slopes as the GA drawing cross-sections. i.e. 2H:1V, the code requirements can be met for global stability. Approximate settlement interaction analysis with this outlined approach reveals the resulting settlement impacts induced on the existing abutment timber piles of unknown reliability is less than 10 mm and the structural designer should assess the impact of this order of settlement on the existing abutment pile integrity.

Based on foundation considerations discussed in Section 5.5 and stability and settlement interaction issues discussed in Section 5.6, from a geotechnical perspective, semi-integral abutments are feasible subject to structural confirmation.

Construction considerations relevant to approach embankments are discussed in Section 5.7.

---

## 5.7 CONSTRUCTION CONSIDERATIONS

---

### 5.7.1 GENERAL

During construction, the contract Administrator should employ experienced geotechnical staff to observe construction activities to ensure geotechnical recommendations are carried out. Additional construction issues that need resolution depending on the contractors' means and methods may be warranted. Potential contractors should seek independent geotechnical advice for such issues. Potential construction considerations include, but are not necessarily limited to the following:

### 5.7.2 SITE PREPARATION

Prior to placement of any fill, all topsoil, rock boulders and other deleterious matter such as organic inclusions, if any, should be stripped in accordance with **OPSS 206** from the embankment footprint to be widened. All vegetation, loose rock/soil on the existing western highway embankment side slope should be removed carefully without causing ground depressions on the road above. The maximum thickness of topsoil intercepted in the boreholes were 100 mm.

---

### 5.7.3 OPEN CUT EXCAVATION STABILITY

All excavations should be carried out in accordance with the Province's Occupational Health and Safety Act (OHSA), O. Reg. 213/91, as well as **OPSS.PROV 539** Construction Specification for Temporary Protection Systems.

In accordance with the Province's Safety Regulation, the following soil classification would be applicable for open-cut. However, based on site specific ground conditions and engineering judgement, OHSA classifications have been qualified and adopted to err on the side of safety.

**Table 5.10 Interpreted OHSA Requirements for Open-Cut Excavations**

Material/Deposit	Groundwater	OHSA Classification/Recommendation	Remarks
Embankment Fill	Above groundwater	Type 3	
	Below groundwater	Not steeper than 2H:1V	
Surficial sands and Silt	Above groundwater	Not steeper than 2H:1V	
	Below groundwater	Type 4	For steeper slopes, dewatering using well points likely to be required.

Cohesionless Till	Below groundwater	Not steeper than 2H:1V	
Bedrock	Not Applicable	Not steeper than 1H: 2V	

**Note:** \* These recommendations are more cautious than OHSA in view of our judgement based on site specific subsurface conditions.

Before worker entry, geotechnical assessment of the excavation slopes should be carried out and is recommended, in addition to surveillance requirements discussed below.

These temporary excavation slopes for the above soil types as per OHSA are to be used only as guidelines for temporary excavation slopes for a short duration. We also recommend that these slopes be visually monitored for any movement especially if workers are present at the toe of the slopes.

All excavations should be undertaken with care to minimize disturbance especially for slopes below the water table and the river banks.

Excavations should be possible in the above soil types except bedrock using equipment such as a hydraulic excavator. Surficial silt is very susceptible to disturbance and for design of granular pads for heavy equipment this should be taken into account. Deeper open-cut excavations into glacial till is unlikely, however, it must be borne in mind that the glacial till is heavily laden with cobbles and boulders.

#### 5.7.4 SHORED EXCAVATIONS

The discussion on shored excavations is provided in case some requirement for temporary works necessitate excavation support. The use of Driving of sheet piles into the glacial till will be significantly impacted by cobbles and boulders. Table 5.10 gives recommended unfactored design parameters for design of temporary shoring. The shoring system should be designed so that the lateral movement of the portion of the 'roadway protection system' will not exceed the established criterion for the structure performance level. In this case, the required Performance Level is considered to be 2 (**OPSS.PROV 539**). The presence of likely random cobbles intercepted within the embankment fill, and the significant contact of boulders in the native till, should be taken into consideration in deciding the means and methods for any shoring support. The shoring design should be carried out by a Professional Engineer, experienced in this type of work and the design will be the sole responsibility of the shoring designer.

**Table 5.11: Geotechnical Design Parameters (Unfactored) – For Temporary Shoring**

Material	D <sub>r</sub> ** (Typical)	Unit weight γ (kN/m³)	Unfactored Strength Parameters*					
			Effective Stress					Remarks
			c' (kPa)	Φ' (deg)	K <sub>a</sub>	K <sub>p</sub>	K <sub>o</sub>	
Existing Embankment Fill – sand and gravel/gravelly sand, fill	Compact	20	0	32	0.31	3.25	0.47	Cohesionless
Surficial Sands/Native Silt	Loose	17	0	28	0.36	2.8	0.53	Cohesionless
Glacial Till	Dense/Very Dense	22	0	40	0.22	4.5	0.45	Cohesionless
Gneissic Bedrock	Typically, strong	27	-	-	-	-	-	Assume a UCS of 50-100 MPa range (see note 4)

\*  $c'$ – Effective cohesion;  $\Phi'$  –Effective friction angle;  $K_a$ -Active Earth Pressure Coefficient;  $K_p$  – Passive Earth Pressure Coefficient;  $K_o$ – At-Rest Earth Pressure Coefficient;  $D_r^{**}$  – Relative Density / Consistency

Notes:

- 1- A factor of safety of 2 shall be applied for computing passive resistance to lateral loads
- 2- Adequate allowance should be made for surcharge loads such as traffic with a minimum of 16 kPa surcharge
- 3- Earth pressure coefficients given in the table are for horizontal backfill and level surface in front (toe area). Any departures from this should be taken into account (for example, at the proposed abutment locations, the proximity of the river banks should be taken into account). Passive earth pressures within the frost depth in soil should be disregarded.
- 4- Very high strength, intact rock cores (bands within the bedrock) were intercepted and it is very important that this be taken into consideration. Bedrock properties discussed in the FIR may be used by contractor to aid in the assessment of possible excavation methods in bedrock. However, the contractor should independently assess if this quantum of information should be sufficient for the planning of means and methods required for such work based on the machinery to be used. This may warrant additional field investigation.
- 5- Design ground water table can be considered at the original ground surface.



## 5.7.5 CONSTRUCTION CONSIDERATIONS FOR ROCK SOCKETS

The boulder laden glacial till would be challenging to drill through to advance the casing and specialized gear such as double rotary rigs could be required. Potential contractors are advised to seek specialist opinion from experienced drilling personnel. Sealing of casing into bedrock would be difficult. In view of the sub-artesian water head observed within the fractured bedrock, tremie concreting will be required. Means and methods of the potential contractor should be able to address the very high strength rock bands during construction of rock sockets. In view of the dilatant wet silt in the overburden and potential adverse impacts on the existing bridge abutments, the use blasting to excavate hard rock bands must not be undertaken. An **NSSP** is attached in **Appendix G** to ‘red flag’ this issue. Drilled caisson construction shall conform to **OPSS 903**.

In view of the flowing sand conditions discussed in Section 5.2.1, the need for sacrificial liners is highlighted in an **NSSP** in Appendix G. This sacrificial liner will also act as a protective jacket for the caisson concrete. Thorough cleaning of rock sockets and socket bases should be ensured during construction.

In view of the close proximity of the existing bridge (which needs to be operational during construction of the new bridge) to the proposed bridge and in addition the condition of the abutment foundations of the existing is not known with reasonable reliability, a settlement and ground vibration monitoring scheme is formulated to monitor and manage risks of construction impacts due to rock socket construction. This monitoring scheme is discussed in Section 5.7.12.

## 5.7.6 BACKFILLING OF PITS

Any temporary excavation needs to be backfilled and returned to pre-construction grades. Any organic, excessively wet, compressible or otherwise deleterious materials should be discarded from being used for backfilling. Any material shortfall should be met with approved materials and backfilling must conform to **OPSS 401** and site restoration to **OPSS 492**.

## 5.7.7 CELLULAR CONCRETE PLACEMENT

The most critical parameter in the present application is to ensure the unit weight of cellular concrete produced is below the design unit weight adopted for analysis ( $5 \text{ kN/m}^3$ ), as the intention is to use it as a light-weight fill. In addition, since the cellular concrete is to be used in close proximity to a river, environmental considerations take very high precedence (requirements for liquid-tight formwork, river water quality monitoring, etc.). An **NSSP** should be included in the contract to address the requirements for cellular concrete. MTO **NSSP** titled Cellular Concrete documents the requirements to be satisfied by the Contractor and the contractor’s Cellular concrete supplier. This **NSSP** is appended in Appendix G.

---

## 5.7.8 EMBANKMENT FILL PLACEMENT

Earth borrow shall conform to **OPSS PROV 212**. Embankment fill placement should conform to **OPSS 501**. The top 600 mm of earthfill prior to placement of the granular subbase and base courses, should be compacted to at least 100 per cent of the standard Proctor Maximum Dry Density (SPMDD) as this is found to enhance pavement life by providing good subgrade support. To ensure appropriate materials are used and adequate levels of compaction are achieved, inspection and field density testing should be carried out by qualified geotechnical personnel.

## 5.7.9 SLOPE PROTECTION/EROSION CONTROL

Rip-rap armouring should be provided at the river banks facing the abutments ends and should generally follow **OPSD 810.010** and any specific recommendations in the hydrology report. The extent along the banks, mass of riprap and the riprap class should be assessed based on specialist discipline advice.

These erosion/scour protection systems should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the hydraulic energy, i.e. velocity of water in the watercourse and its regime and the erodible nature of stream bed material).

The side slopes of the earthen embankments should be provided with adequate erosion protection against surface water runoff. Proper erosion control measures should be implemented both during construction and permanently. This can be achieved by prompt seed and cover (**OPSS 804**) or sodding (**OPSS 803**). If the embankments have to be exposed during winter unprotected, then to minimize adverse spring thaw effects, covering the slopes with straw or gravel sheeting can be considered. Also maintain a cross-fall on top of the embankment so that no puddles will form during snow melt.

Surface runoff flowing from the embankments should be prevented from forming erosion gullies. Cover is the single most effective erosion control practice. Hence, topsoiling to promote grass cover can be effective against gulley erosion.

Erosion protection of cellular concrete bridge abutment approaches are required. A clear cover of 300 mm comprising engineered OPSS Granular BII should be provided as an envelope to cover all exposed cellular concrete faces except the top surface where the pavement structure details will address the situation.

### 5.7.10 SOIL RE-USE/DISPOSAL ISSUES

The excavated silt will not be suitable for any engineered construction, if however, used for general landscaping, the high erosion potential should be guarded against.

The excavated materials from excavations should be checked for contamination prior to removal/disposal off-site, to determine which disposal option is best for the excavated materials (**OPSS 180**). The investigation of disposal requirements is outside WSP's scope.

### 5.7.11 CONSTRUCTION DEWATERING

Due to the cohesionless nature of the subsoils and the sub-artesian nature of the bedrock, any dewatering requirement shall conform to **OPSS 517** and **NSSP FOUND003**. The need for a PTTW should be addressed with hydrogeological input.

### 5.7.12 SETTLEMENT AND GROUND VIBRATION MONITORING

Rock socket excavations for the proposed bridge abutment foundations, given the presence of hard rock bands, are likely to cause ground vibrations and potential associated ground subsidence. These vibrations could have the potential to impact the existing bridge abutment foundations. As a precautionary measure, to alert should such adverse impacts occur and to plan mitigation options, a settlement and ground vibration monitoring scheme is recommended.

Both ground vibration and settlement monitoring should be undertaken during the excavation of rock sockets. However, settlement monitoring should be continued during the approach embankment construction.

The Contractor is responsible for the supply and installation of the instrumentation. The Contract Administrator with a Foundation Engineering Specialist is responsible for the monitoring program. An NSSP is attached giving details of the settlement and ground vibration monitoring programme.

# SIGNATURES



Franklin Oliha, MSc., EIT.



Masud Karim, Ph.D., P.Eng. Senior  
Geotechnical Engineer /Independent  
Technical Reviewer



Vasantha Wijeyakulasuriya, M.Eng., P.Eng.  
Senior Technical Director, Geotechnical  
MTO Designate (Foundations).



Foundation Investigation Report  
Proposed Ivanhoe Bridge Replacement, Township of Foleyet, Hwy 101, Ontario  
WSP No. T7M-023443-00



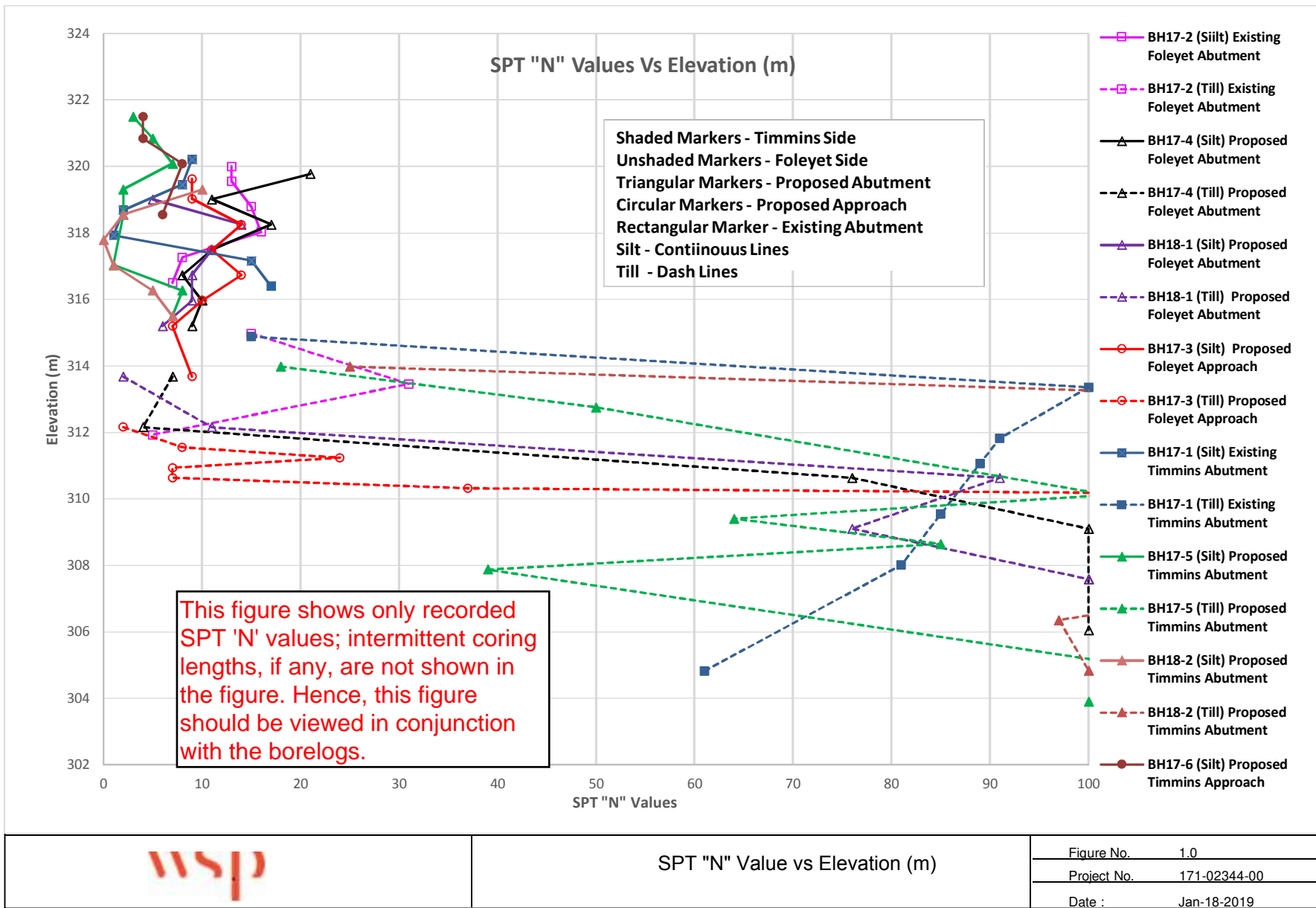
---

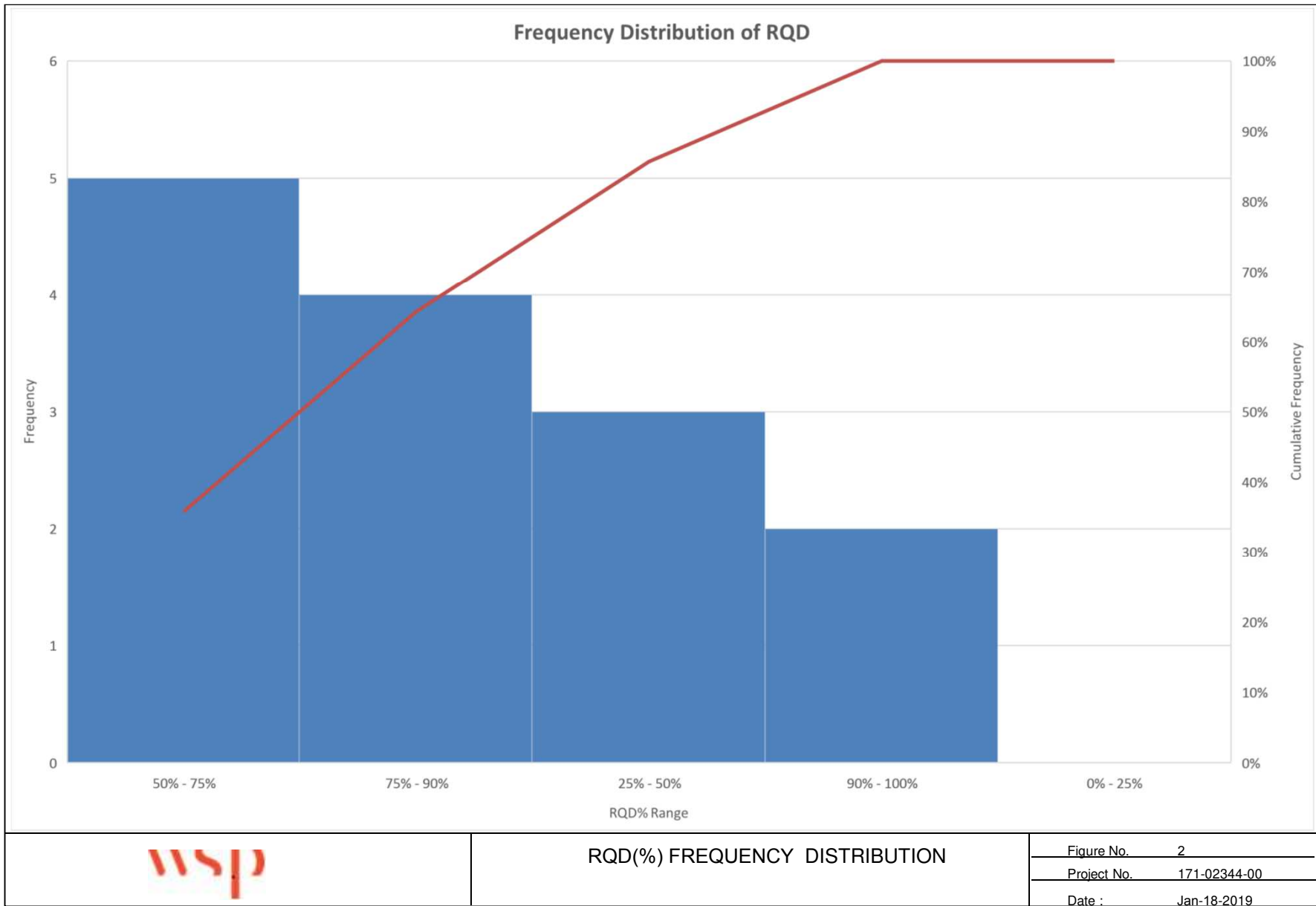
## REFERENCES

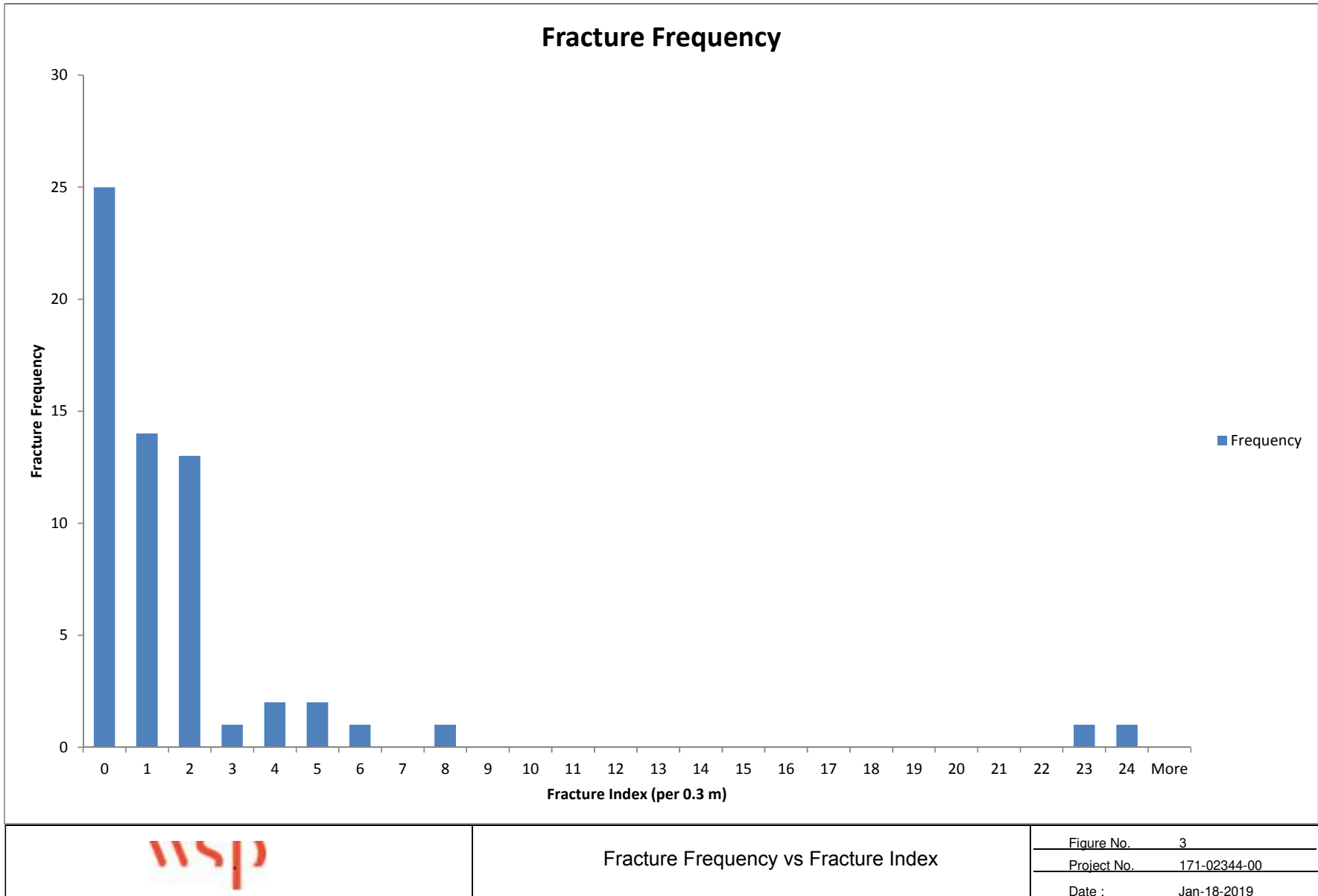
- Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6-14. 2014. CSA Special Publication, S6.1 14. Canadian Standard Association.
- Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4<sup>th</sup> Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd, British Columbia.
- M.T.O Soil Classification Manual, Ministry of Transportation, Ontario.
- MTO Report SO-96-01 Integral Abutment Bridges, Ministry of Transportation, Ontario.
- Zhang, L., Ernst, H. and Einstein, H.H. (2000) Nonlinear Analysis of Laterally Loaded Rock-socketed Shafts, ASCE, Geotechnical Journal
- Ahmad, Ken et al. (2016) Settlement Mitigation by Using Cellular Concrete at Nepewassi River Bridge, Ontario – A Case Study, TAC.

# LIST OF FIGURES

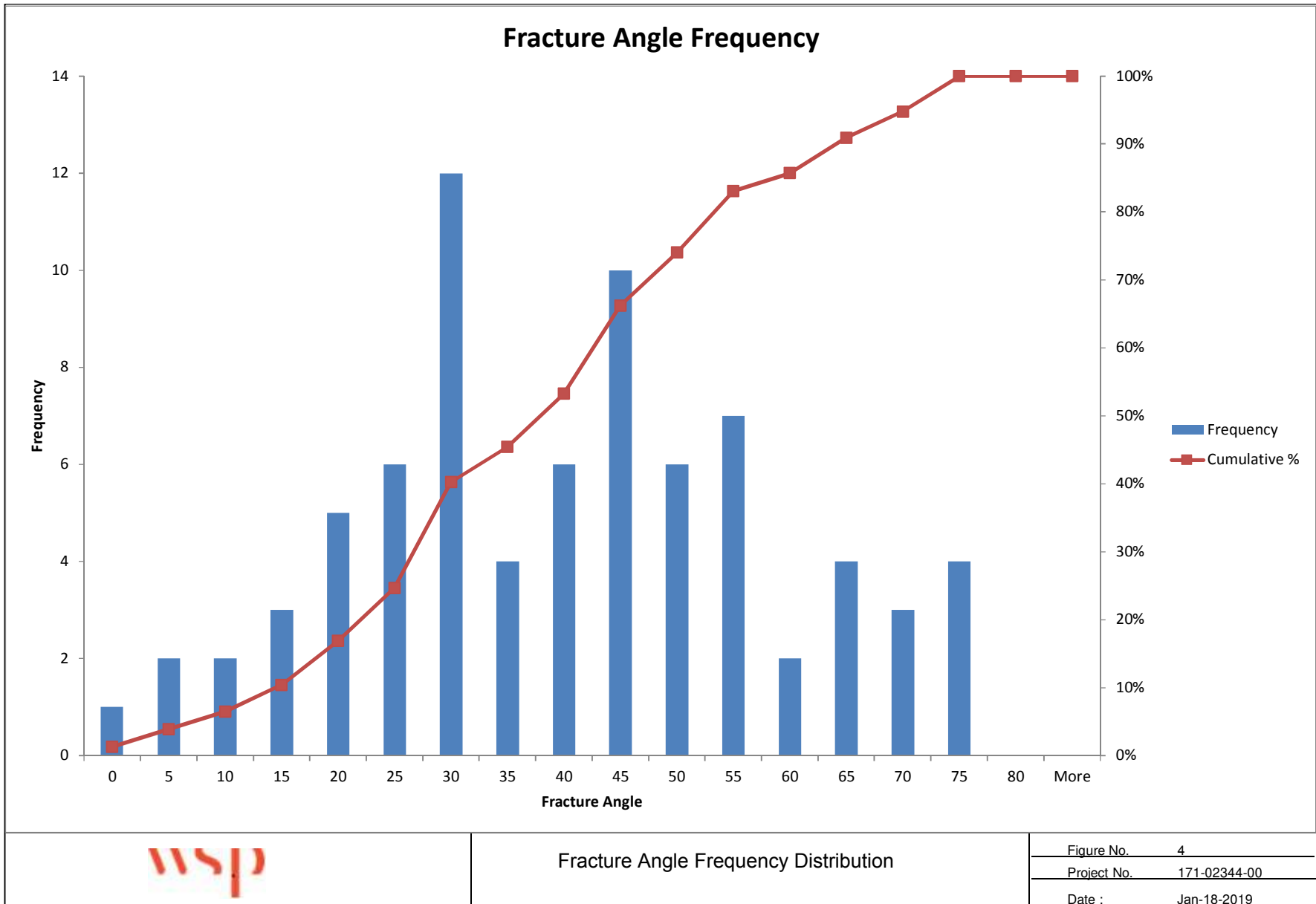
---

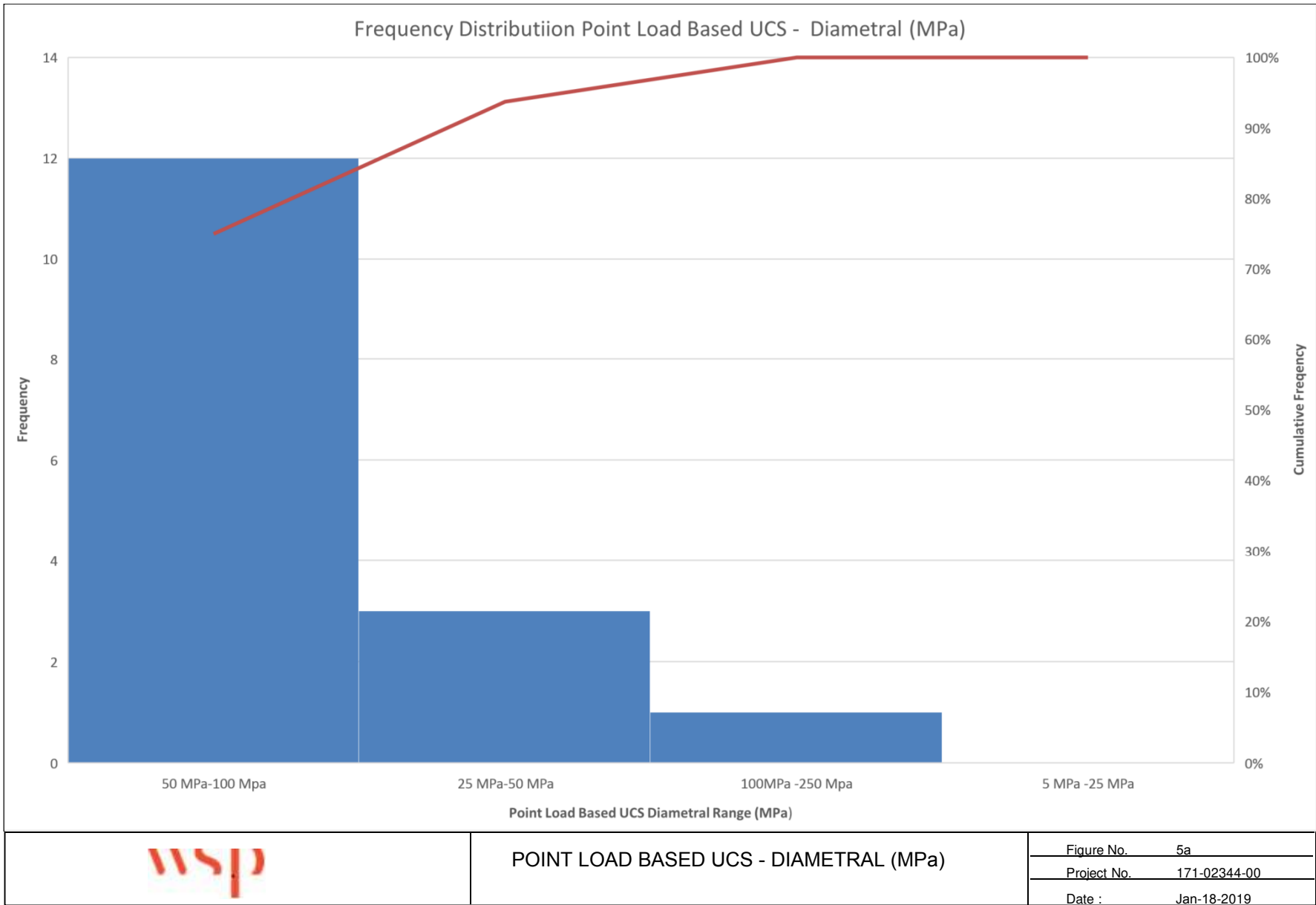


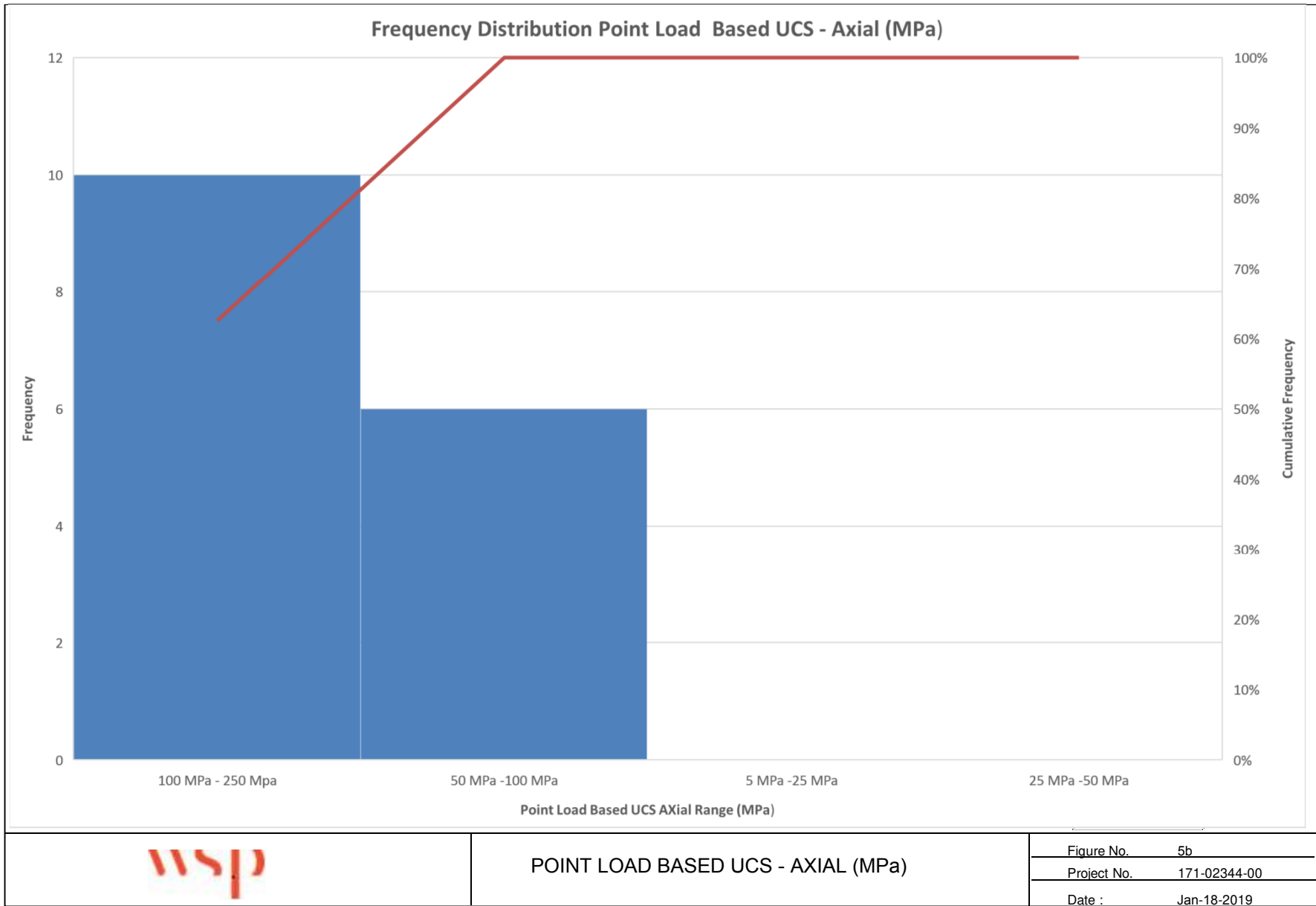




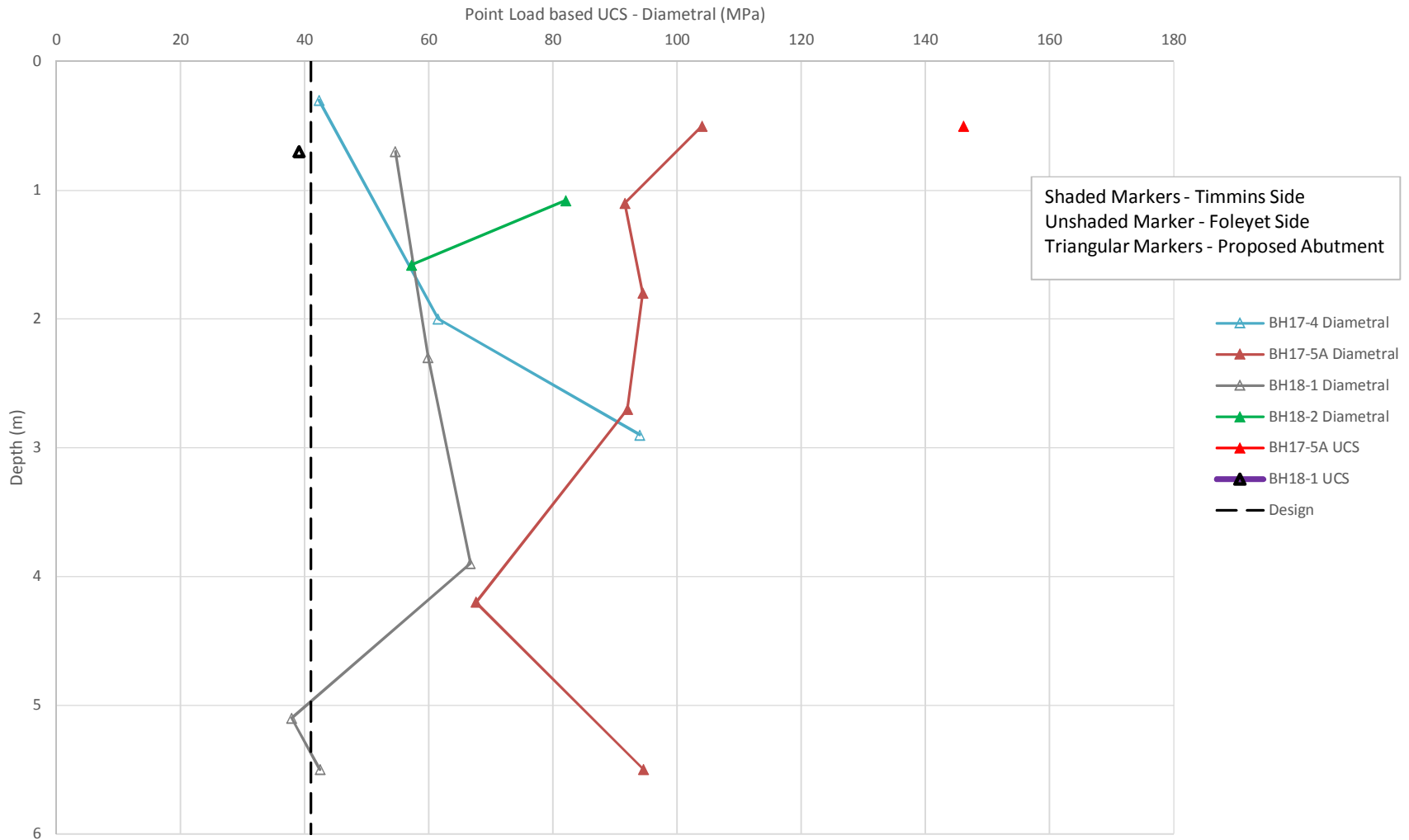








Depth (m) Below Bedrock Vs Point Load Based UCS/UCS (MPa)



Depth (m) Below Bedrock VS Point Load Based-UCS/UCS (MPa)

Figure No. 6

Project No. 171-02344-00

Date : Oct-25-2018

# APPENDIX

## E

### **SLOPE STABILITY ANALYSIS**

- E1) Ground Calibration**
- E2) Long Term Stability**
- E3) Rapid Drawdown**
- E4) Seismic**
- E5) Side Slope Stability**



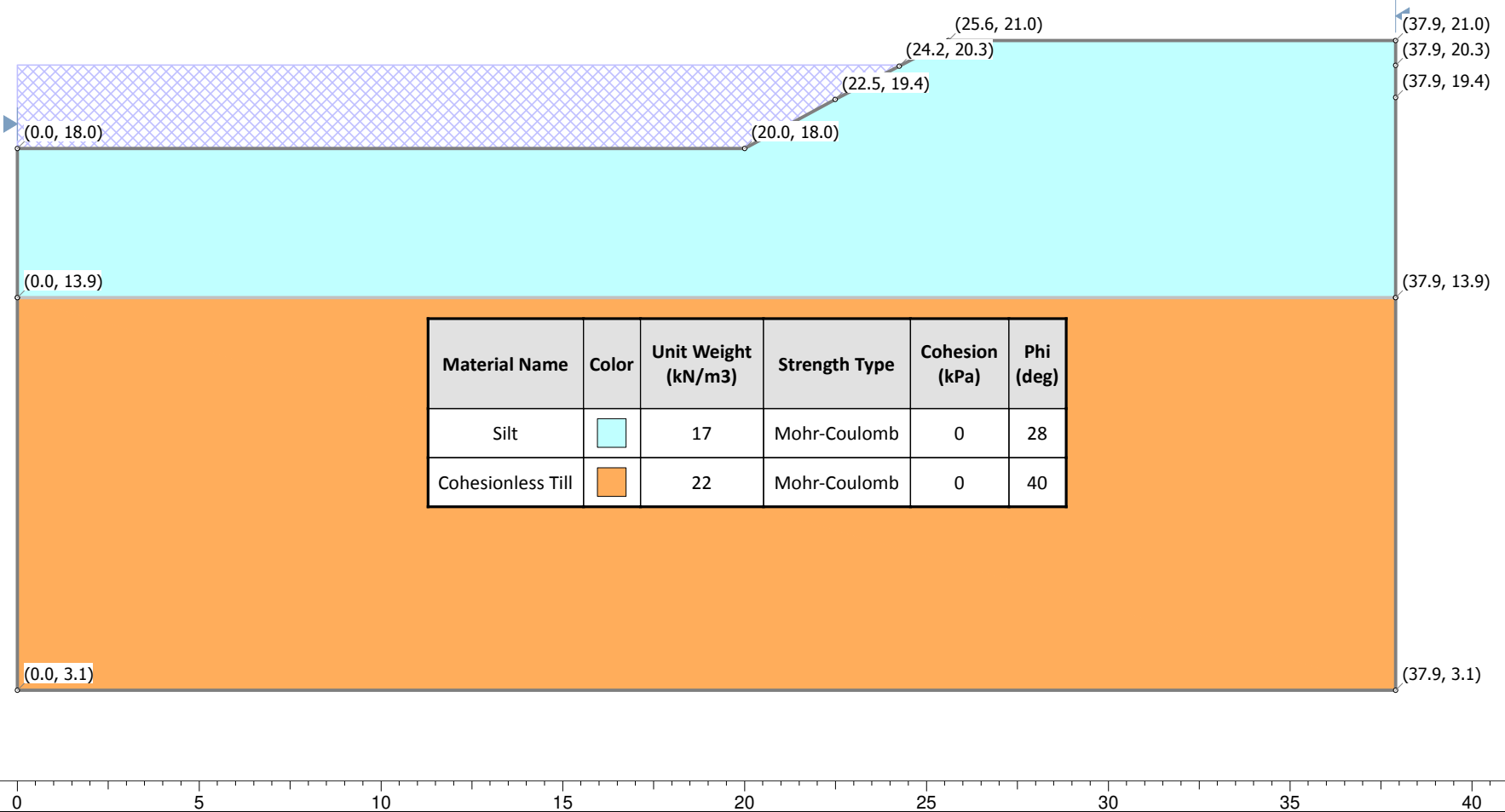
# E1

**Ground Calibration**

# Embankment Geometry at the Existing Abutment (Timmins End)

10 yr Flood Level

Fig. E1.1a



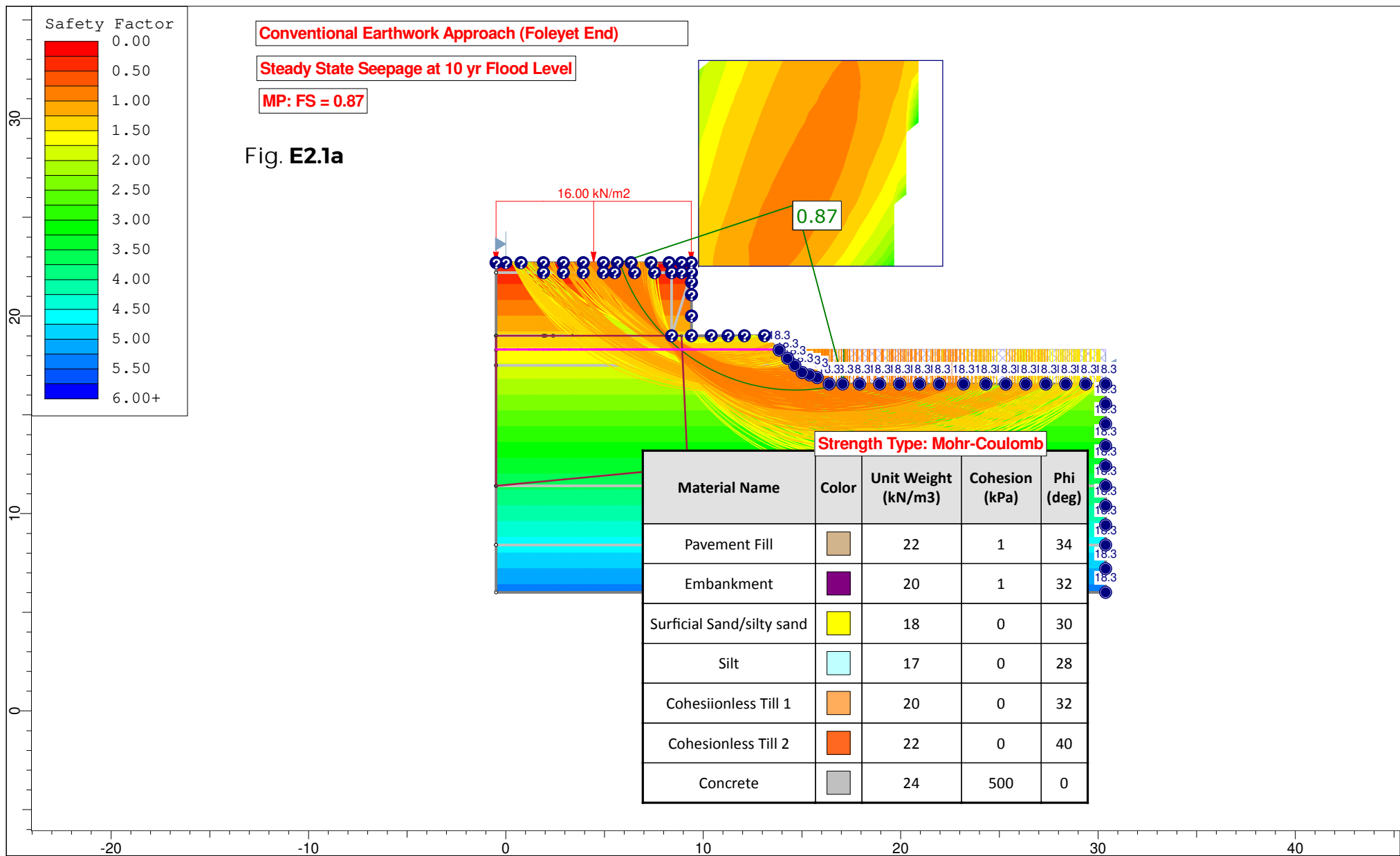




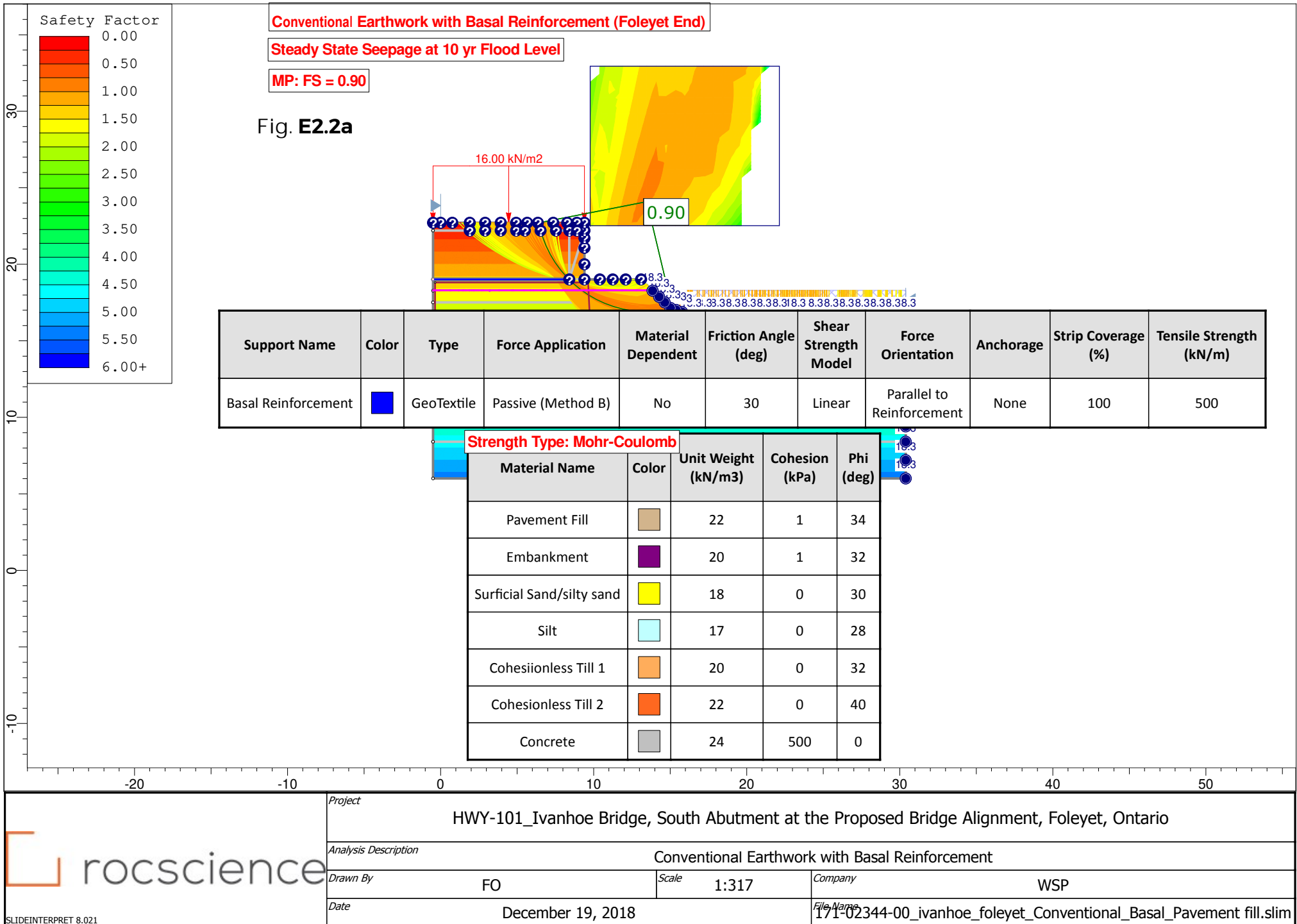
# E2

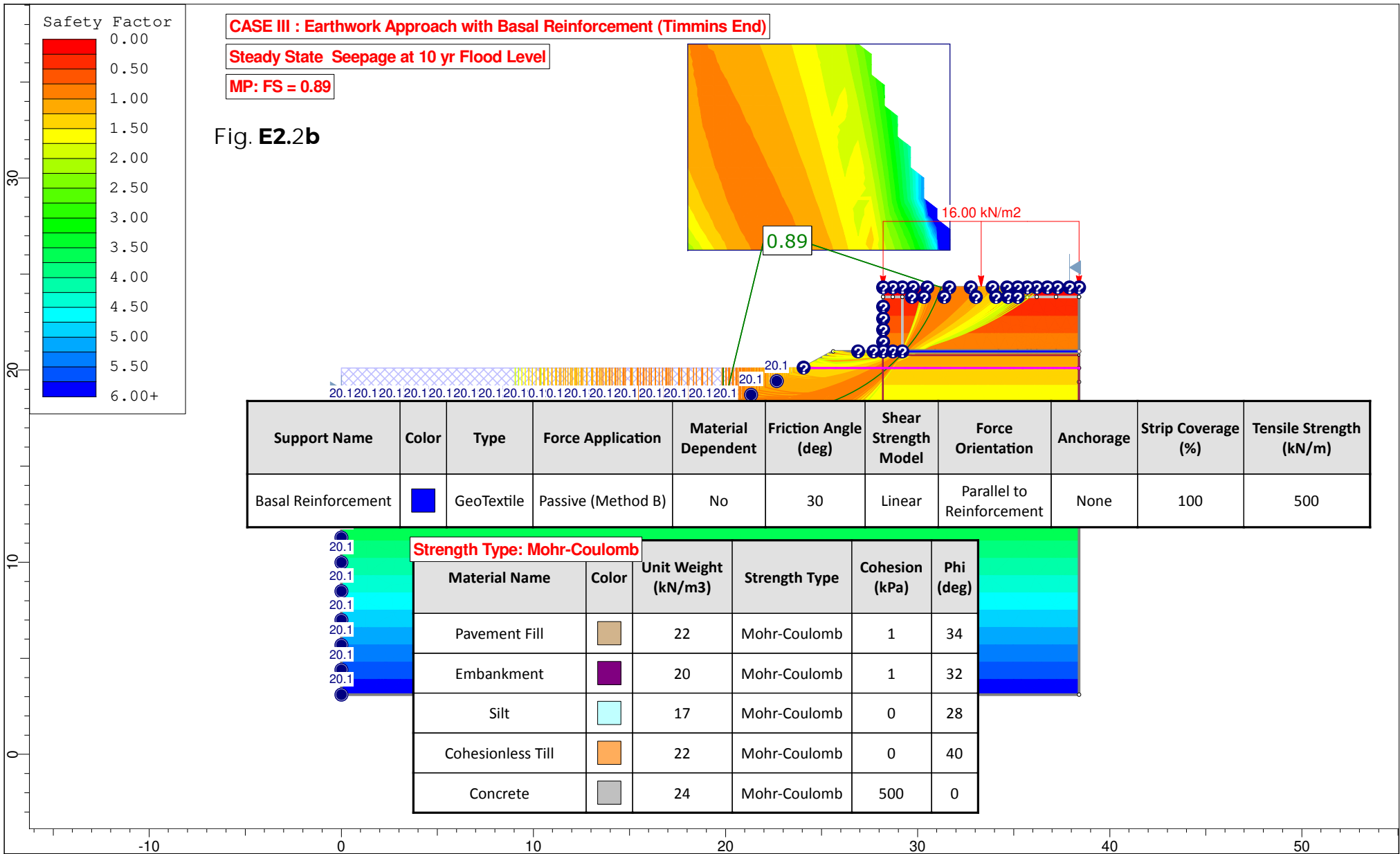
## Long Term Stability


- Conventional Earthwork Approach (Fig E2.1a & Fig E2.1b)
- Conventional Earthwork + Basal Reinforcement (Fig E2.2a & Fig E2.2b)
- Conventional Earthwork + Basal Reinforcement + Rockfill Toe Buffer (Fig 2.3a & Fig 2.3b)
- Embankment RockFill (Fig E2.4a & Fig E2.4b)
- Conventional Earthwork + Cellular Concrete Block (Fig E2.5a & Fig 2.5b)
- Embankment Rockfill + Cellular Concrete Block (Fig E2.5c & Fig 2.5d)

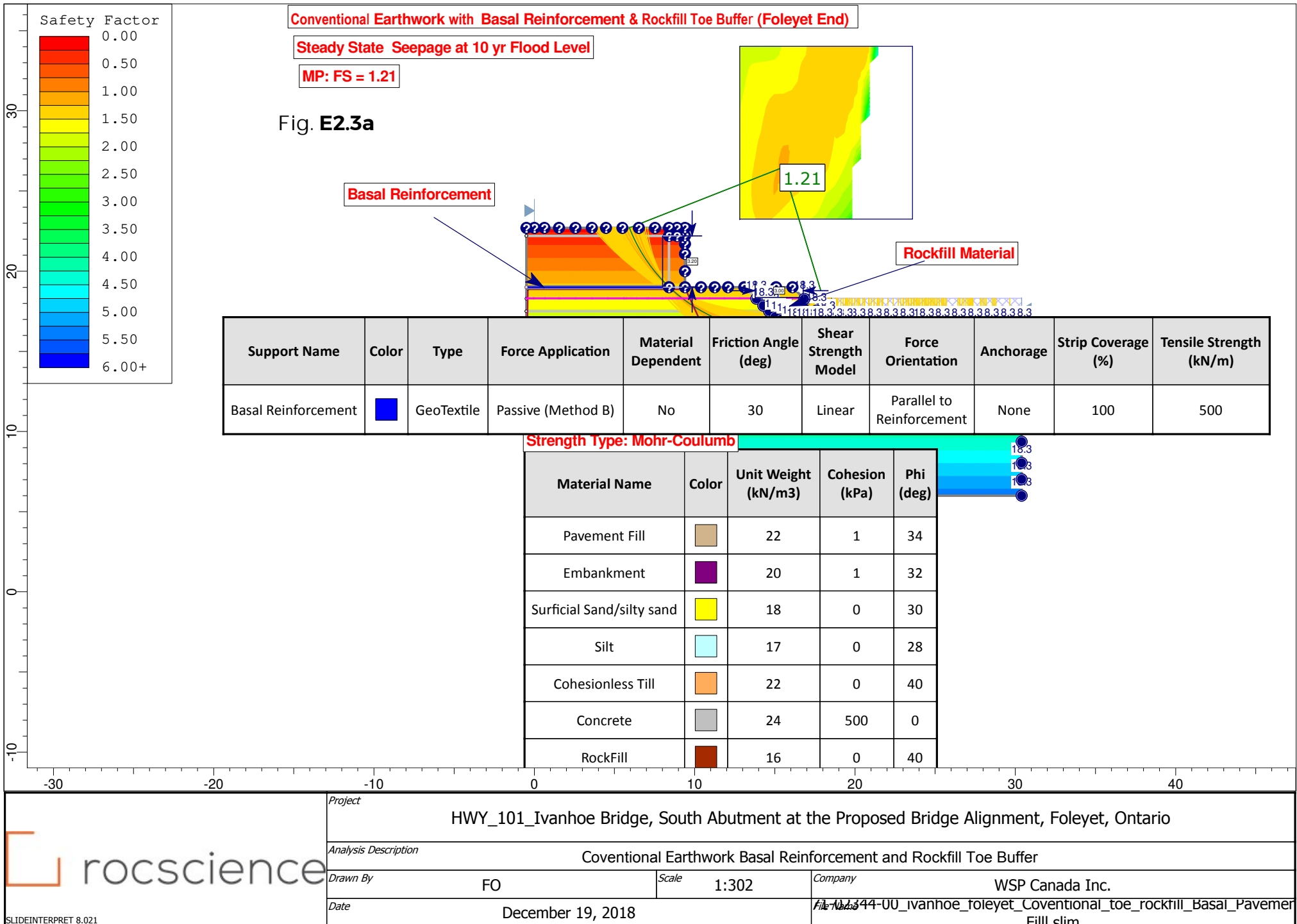




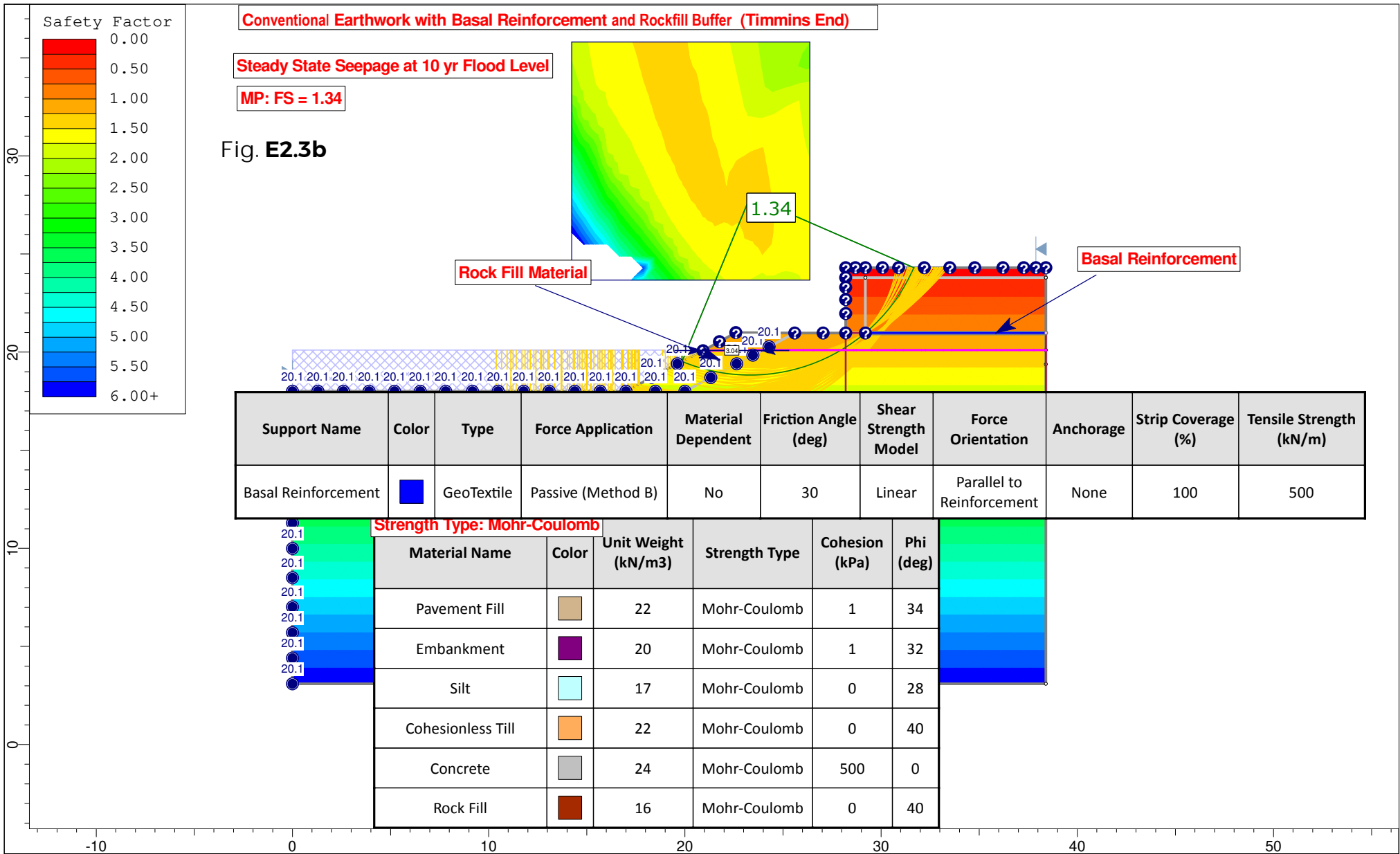




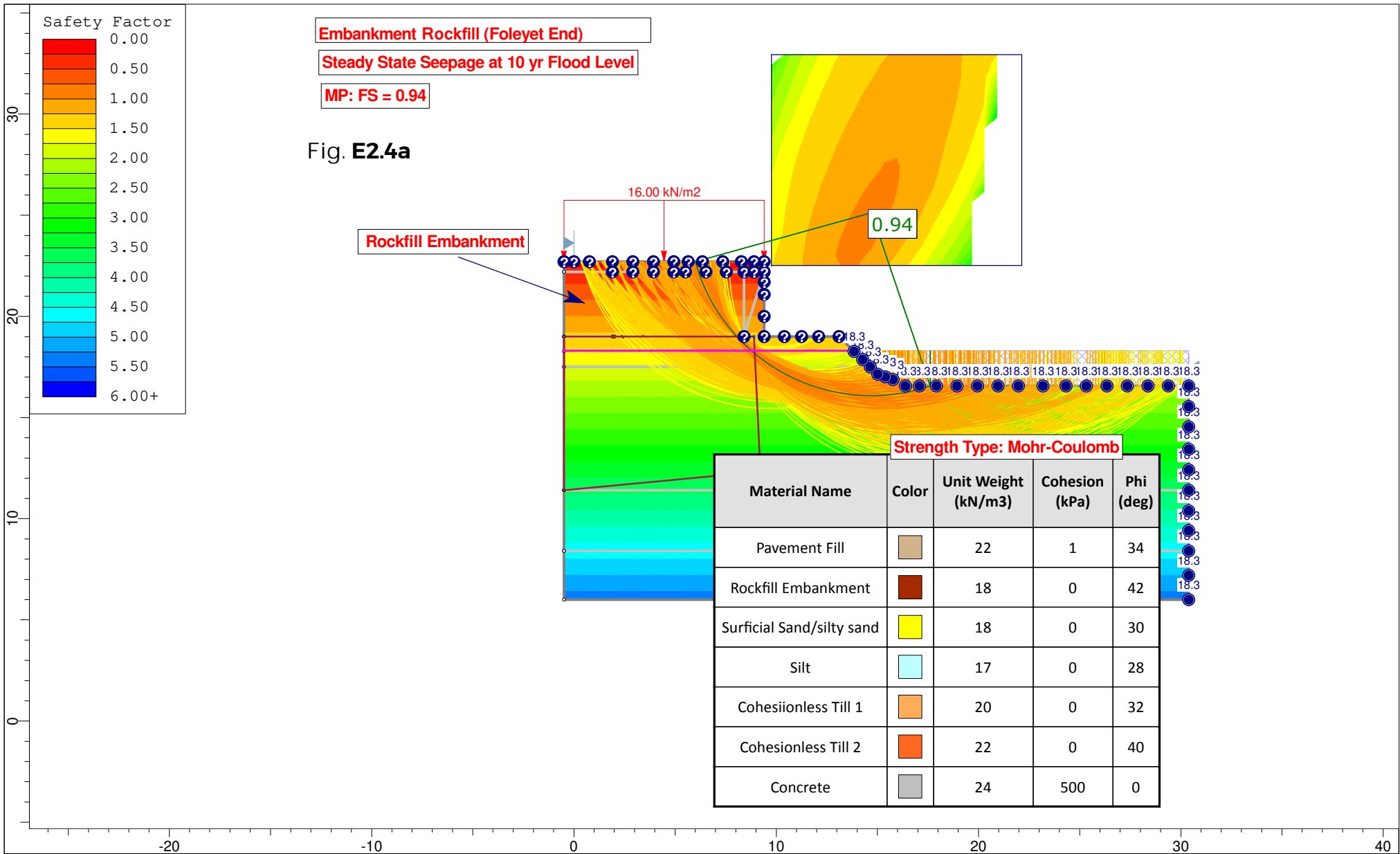
 <small>SLIDEINTERPRET 8.021</small>	<i>Project</i> HWY_101_Ivanhoe Bridge North Abutment at the Proposed Bridge Alignment			
	<i>Analysis Description</i> CASE III : Earthwork Approach with Basal Reinforcement			
	<i>Drawn By</i> FO		<i>Scale</i> 1:272	<i>Company</i> WSP Canada Inc.
	<i>Date</i> December 19, 2018		<i>File Name</i> 11-02344-00_ivanhoe_timmins_conventional_Basal_Pavement fill slim	








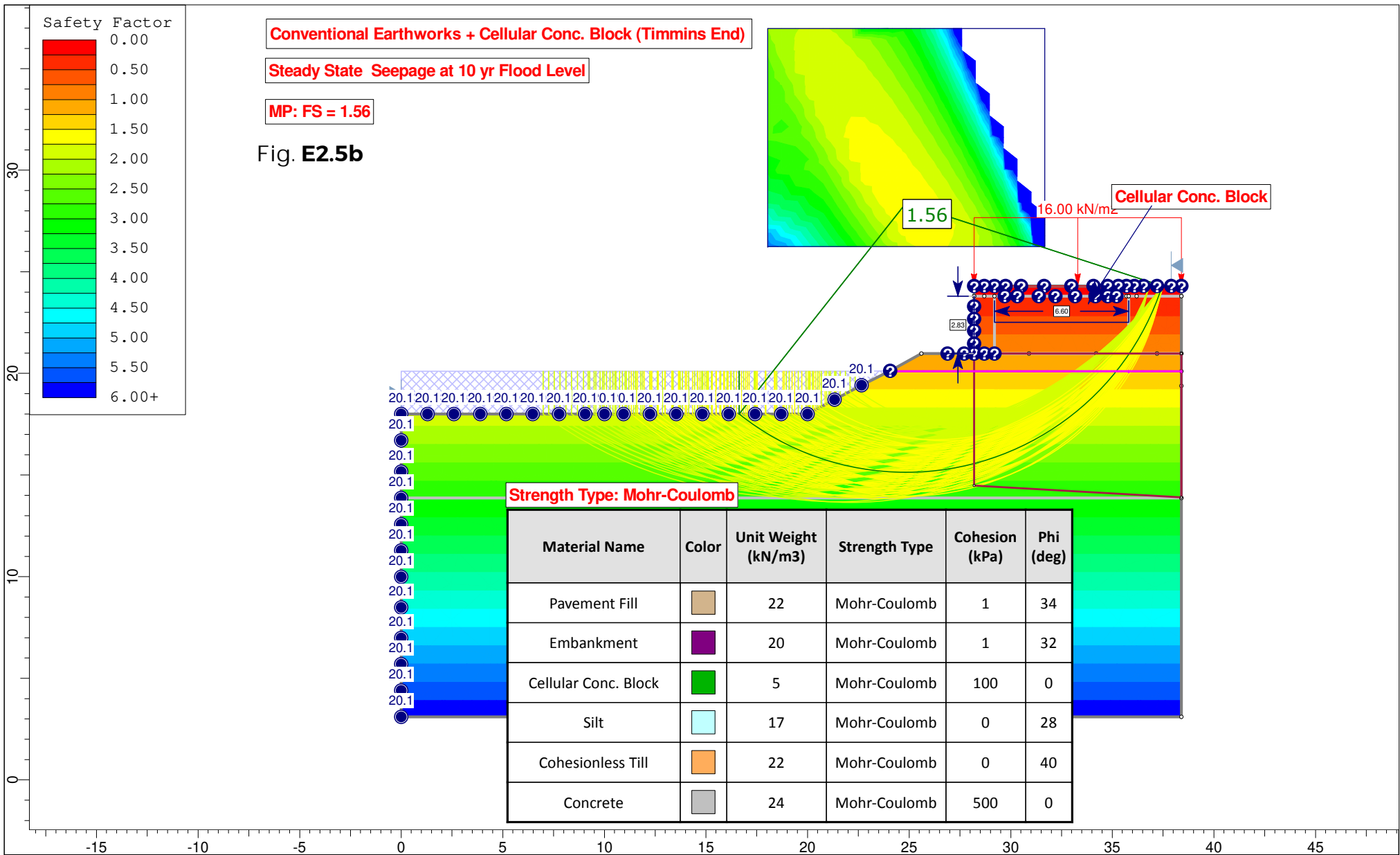
<div> <div></div> <div>rocscience</div> </div>	Project			HWY_101_Ivanhoe Bridge North Abutment at the Proposed Bridge Alignment		
	Analysis Description			Conventional Earthwork with Basal Reinforcement and Rockfill Buffer		
	Drawn By	FO	Scale	1:267	Company	WSP Canada Inc.
	Date	December 19, 2018			File Name	2344-00_ivanhoe_timmins_Earthwork_Basal_Rockfill_toe_rockfill_global



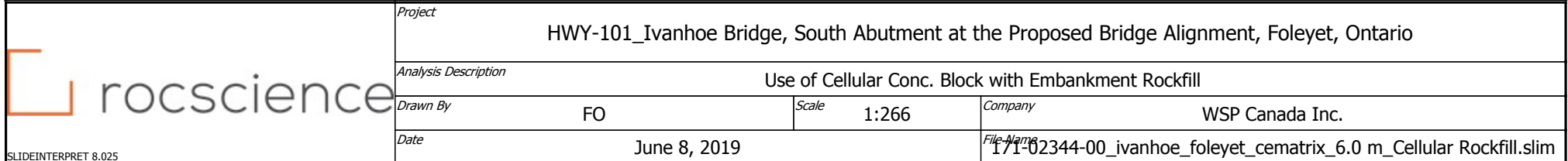
	Project			HWY-101_Ivanhoe Bridge, South Abutment at the Proposed Bridge Alignment, Foleyet, Ontario		
	Analysis Description			Embankment Rockfill		
	Drawn By	FO	Scale	1:259	Company	WSP
	Date	April 22, 2019			File Name	171-02344-00_ivanhoe_foleyet_Emb_Rockfill_pavement fill.slim



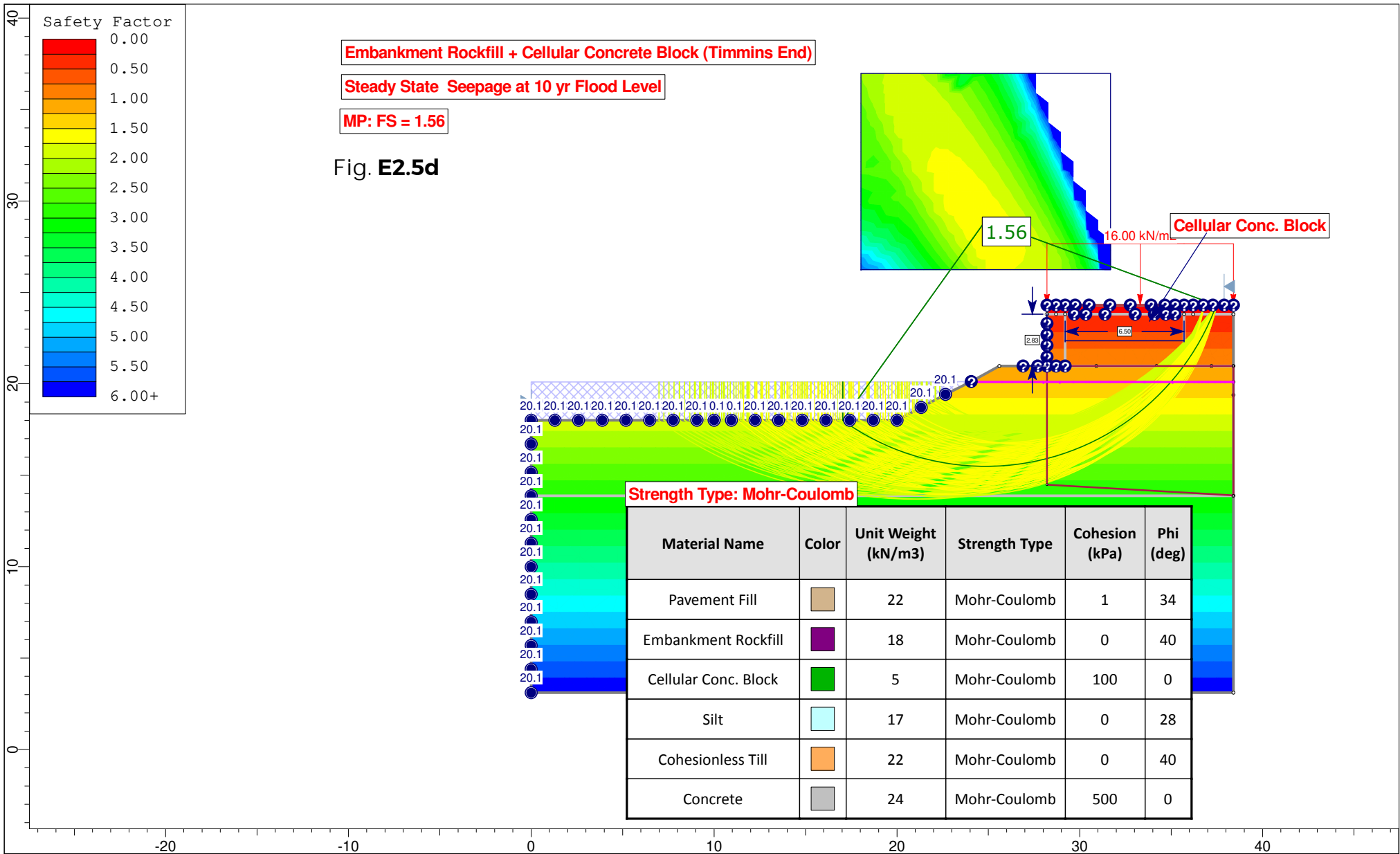




	Project			HWY_101_Ivanhoe Bridge North Abutment at the Proposed Bridge Alignment	
	Analysis Description			Use of Cellular Conc. Block with Conventional Earthworks	
	Drawn By	FO	Scale	1:258	Company WSP Canada Inc.
	Date	June 8, 2019		File Name: 11-07-344-00_ivanhoe_timmins_cematrix_7m_Conv_Earthfill_pavemen fill slim	





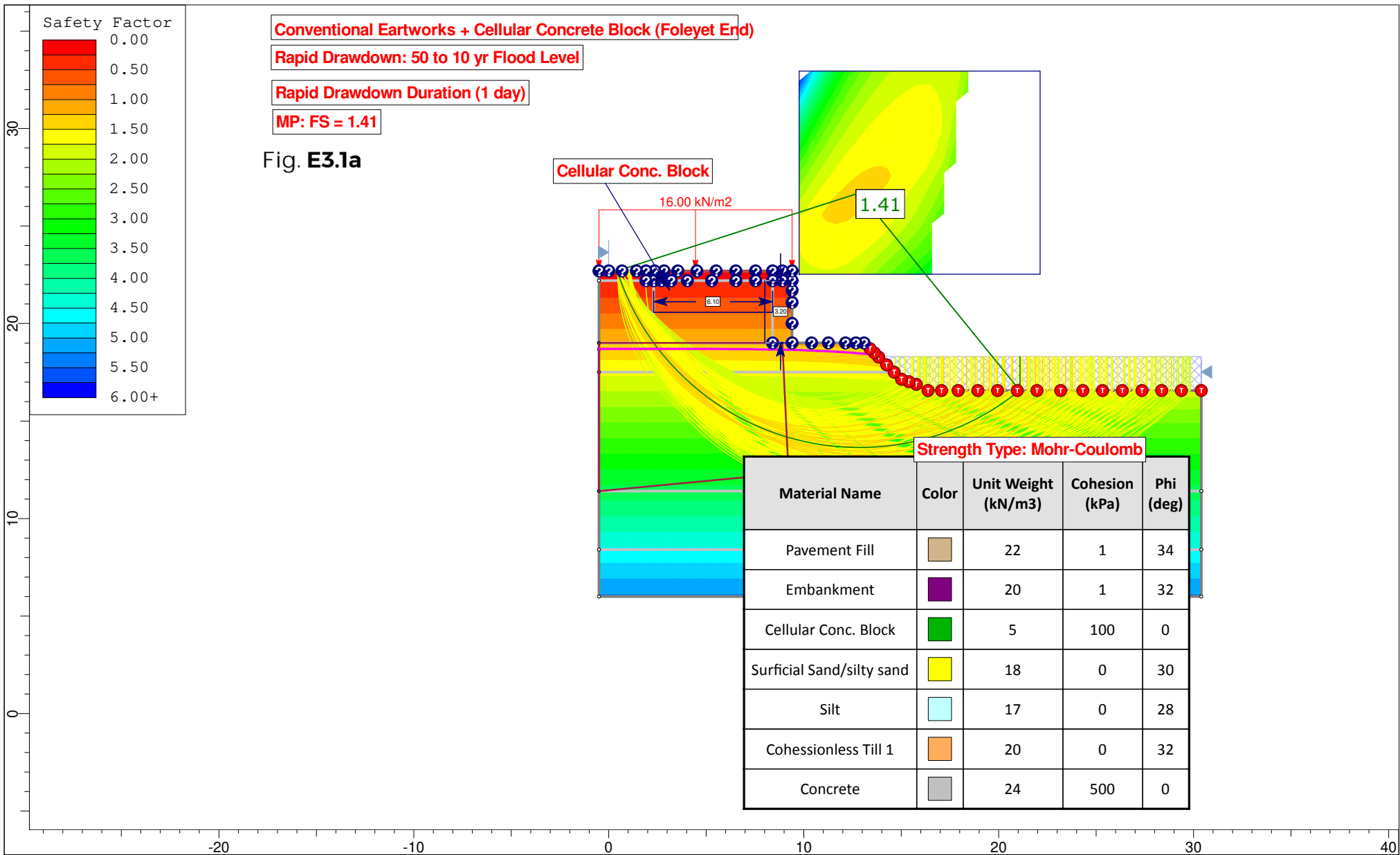




	Project			HWY_101_Ivanhoe Bridge North Abutment at the Proposed Bridge Alignment	
	Analysis Description			Use of Cellular Conc. Block with Embankment Rockfill	
	Drawn By	FO	Scale	1:286	Company WSP Canada Inc.
	Date	June 8, 2019	File Name	171-02344-00_ivanhoe_timmins_cematrix_6.5m_Rockfill Cellular.slim	

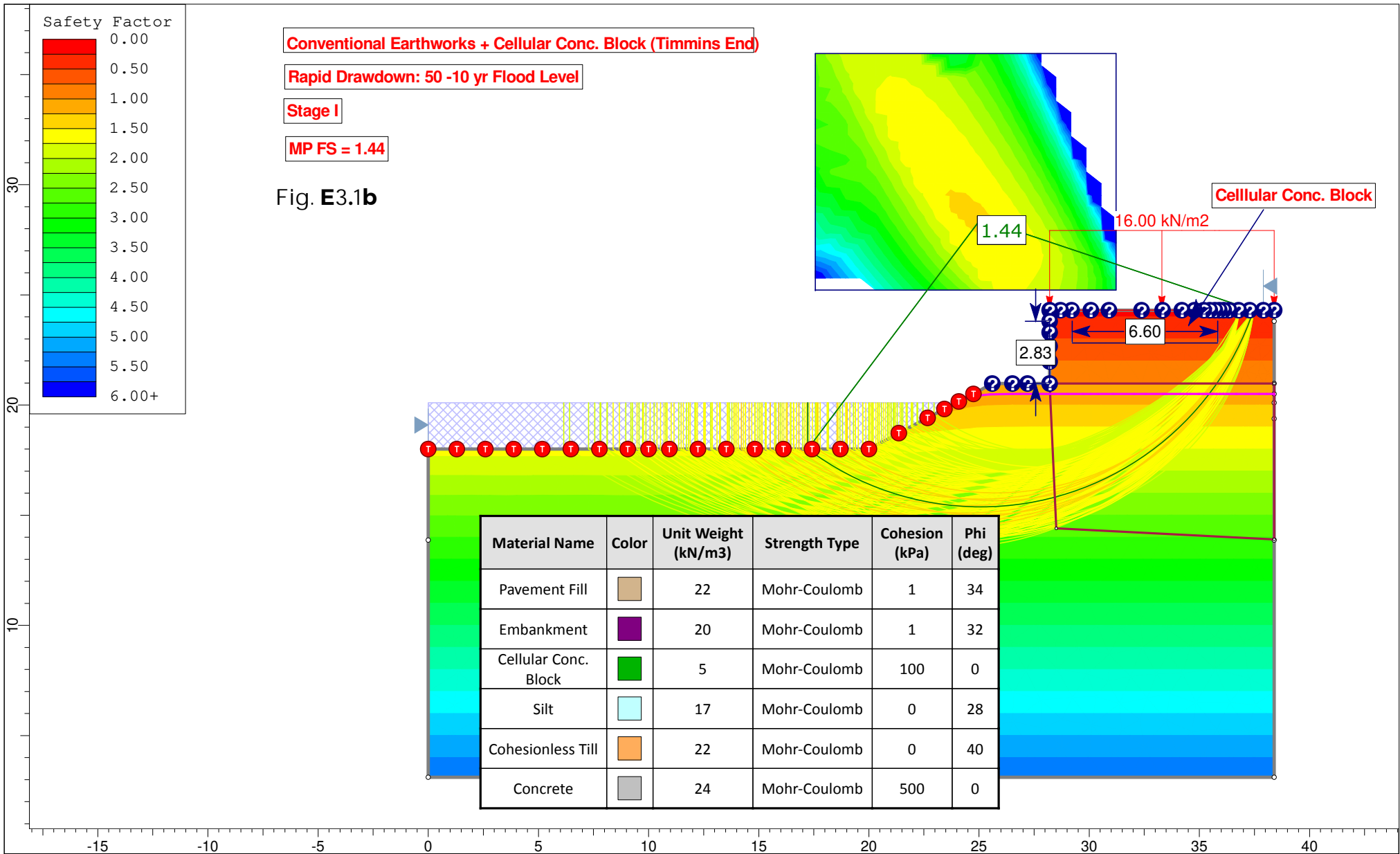
# E3

## **Rapid Drawdown**

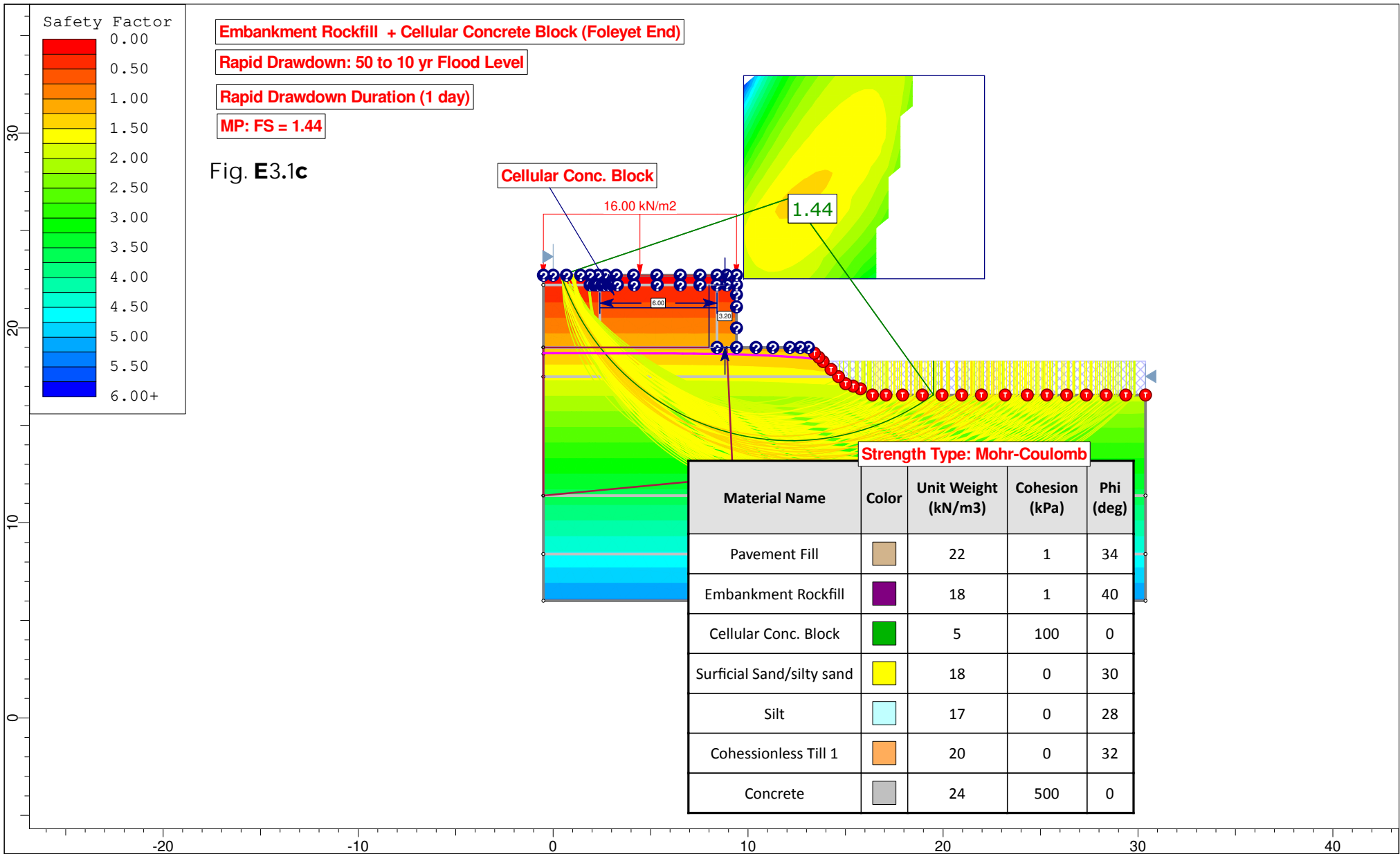
- Conventional Earthworks + Cellular Concrete Block (Fig E3.1a &E3.1b)
- Rocfill Embankment + Cellular Concrete Block (Fig E3.1c &E3.1d)




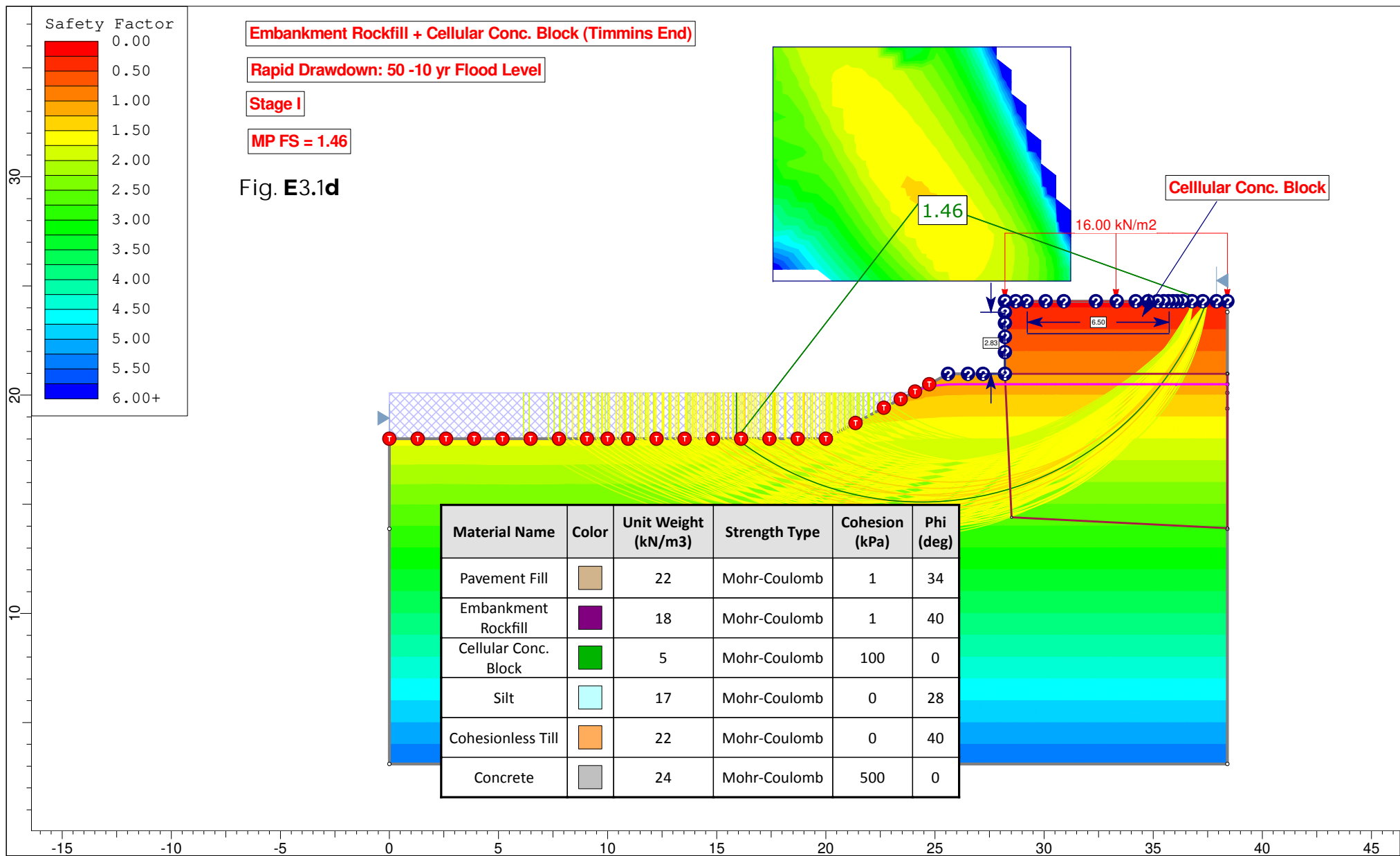
	Project			
	HWY-101_Ivanhoe Bridge, South Abutment at the Proposed Bridge Alignment, Foleyet, Ontario			
	Analysis Description			
	Use of Cellular Concrete Block with Conventional Earthworks			
	Drawn By	FO	Scale	1:268
	Date	June 8, 2019	Company	WSP Canada Inc
SLIDEINTERPRET 8.025		File Name: U2344-00_Ivanhoe_Foleyet_End_Conv Earthfill_Cellular Conc.- 1 Stage_slim		



	Project			HWY_101_Ivanhoe Bridge North Abutment at the Proposed Bridge Alignment	
	Analysis Description			Use of Cellular Concrete Block with Conventional Earthfill	
	Drawn By	FO	Scale	1:237	Company
	Date	June 8, 2019	File Name	WSP Canada Inc.	
	SLIDEINTERPRET 8.025			File Name: 2344-00_Ivanhoe_Timmins_End_Conv. Earthfill_Cellular Conc. 6.6m Case II Stage 1 slim	



	Project			HWY-101_Ivanhoe Bridge, South Abutment at the Proposed Bridge Alignment, Foleyet, Ontario	
	Analysis Description			Use of Cellular Concrete Block with Embankment Rockfill	
	Drawn By	FO	Scale	1:268	Company WSP Canada Inc
	Date	June 8, 2019	File Name	E:\1002344-00_Ivanhoe_Foleyet_End_Emb. Rockfill_Cellular Conc.- 1 Stage slim	



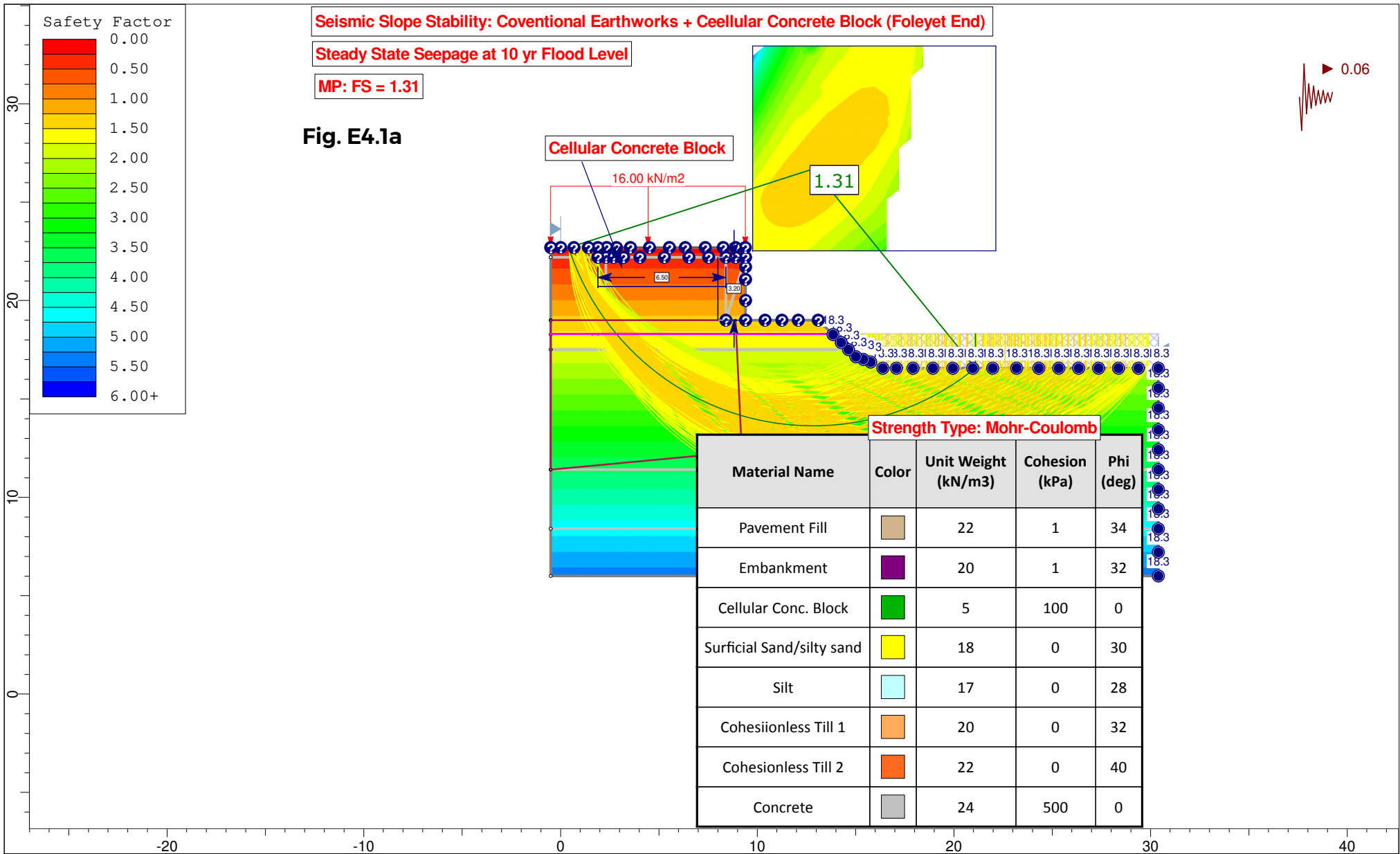


# APPENDIX

## E4

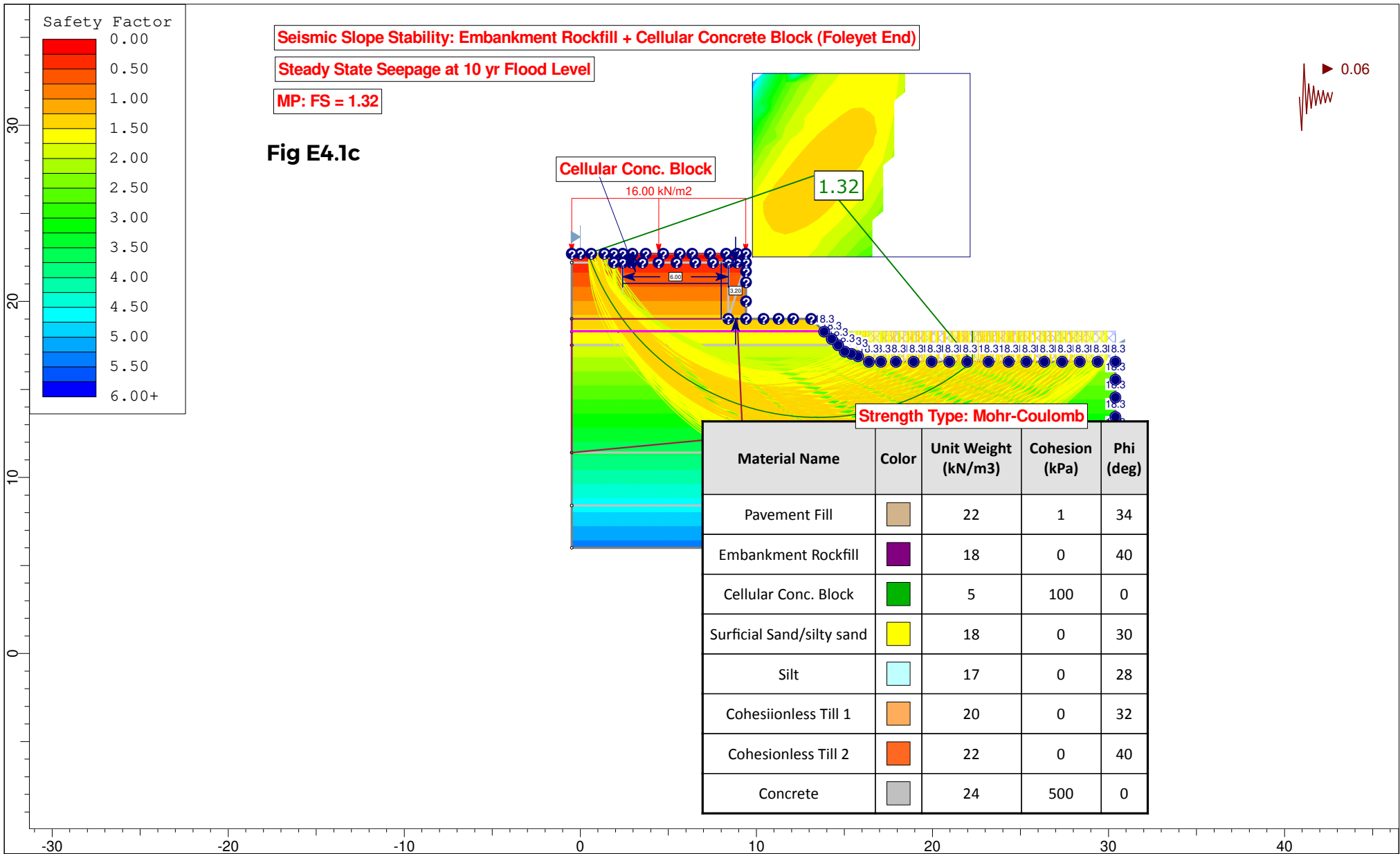
**Seismic**


- Conventional Earthworks + Cellular Concrete Block (Fig E4.1a & Fig E4.1b)
- Embankment Rocfill + Cellular Concrete Block (Fig E4.1c & Fig E4.1d)

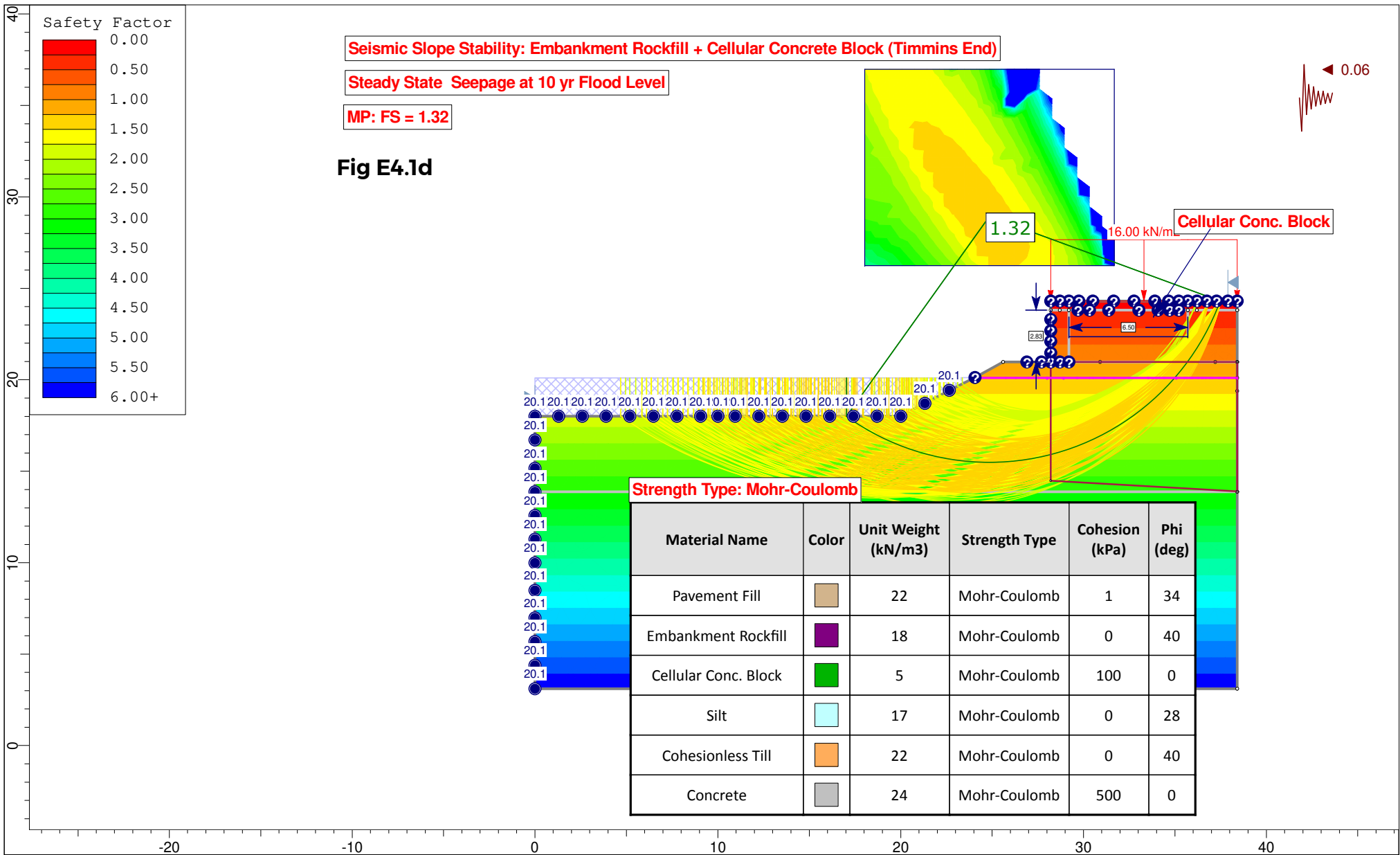



	Project			HWY-101_Ivanhoe Bridge, South Abutment at the Proposed Bridge Alignment, Foleyet, Ontario	
	Analysis Description			Seismic Slope Stability: Use of Cellular Concrete Block with Conventional Earthworks	
	Drawn By	FO	Scale	1:266	Company WSP
	Date	June 8, 2019		File Name	1/1-02344-00_ivanhoe_foleyet_Earthfill_Cellular Conc. Seismic 0.06 slim





	Project			HWY-101_Ivanhoe Bridge, South Abutment at the Proposed Bridge Alignment, Foleyet, Ontario	
	Analysis Description			Seismic Slope Stability: Use of Cellular Conc. Block with Embankment Rockfill	
	Drawn By	FO	Scale	1:297	Company WSP Canada Inc.
	Date	June 8, 2019		File Name 171-02344-00_ivanhoe_foleyet_cematrix_6.0 m_Cellular Rockfill_Seismic_0.06.slim	



 <small>SLIDEINTERPRET 8.025</small>	<i>Project</i>			
	HWY_101_Ivanhoe Bridge North Abutment at the Proposed Bridge Alignment			
	<i>Analysis Description</i>			
	Seismic Slope Stability: Use of Cellular Conc. Block with Embankment Rockfill			
	<i>Drawn By</i>		<i>Scale</i>	<i>Company</i>
	FO	1:286	WSP Canada Inc.	
	<i>Date</i>		<i>File Name</i>	
	June 8, 2019		171-02344-00_ivanhoe_timmins_cematrix_6.5m_Rockfill Cellular_Seismic_0.06.slim	

# APPENDIX

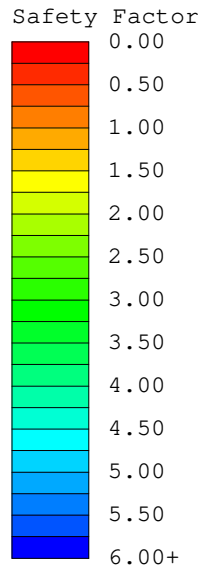
## E5

**LONG TERM STABILITY**

**- Embankment Side Slope**



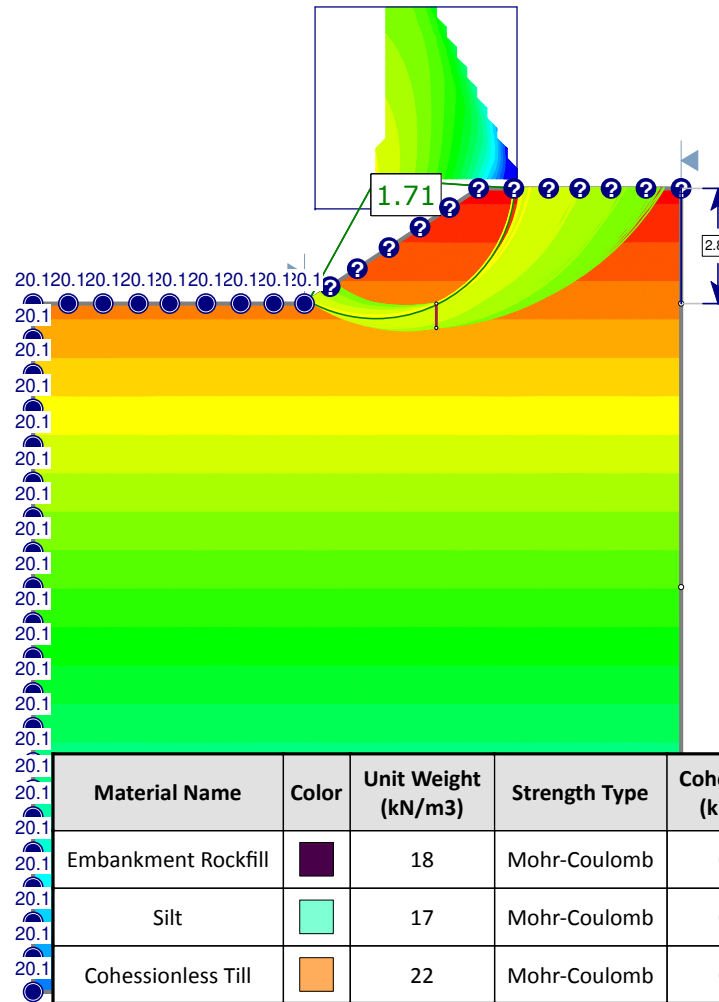
**Fig E5**



**Embankment Side Slope - Timmins End**

**Steady State Seepage at 10 year Flood Level**

**MP: FS = 1.71**



# APPENDIX



**F**

**SETTLEMENT ANALYSIS**

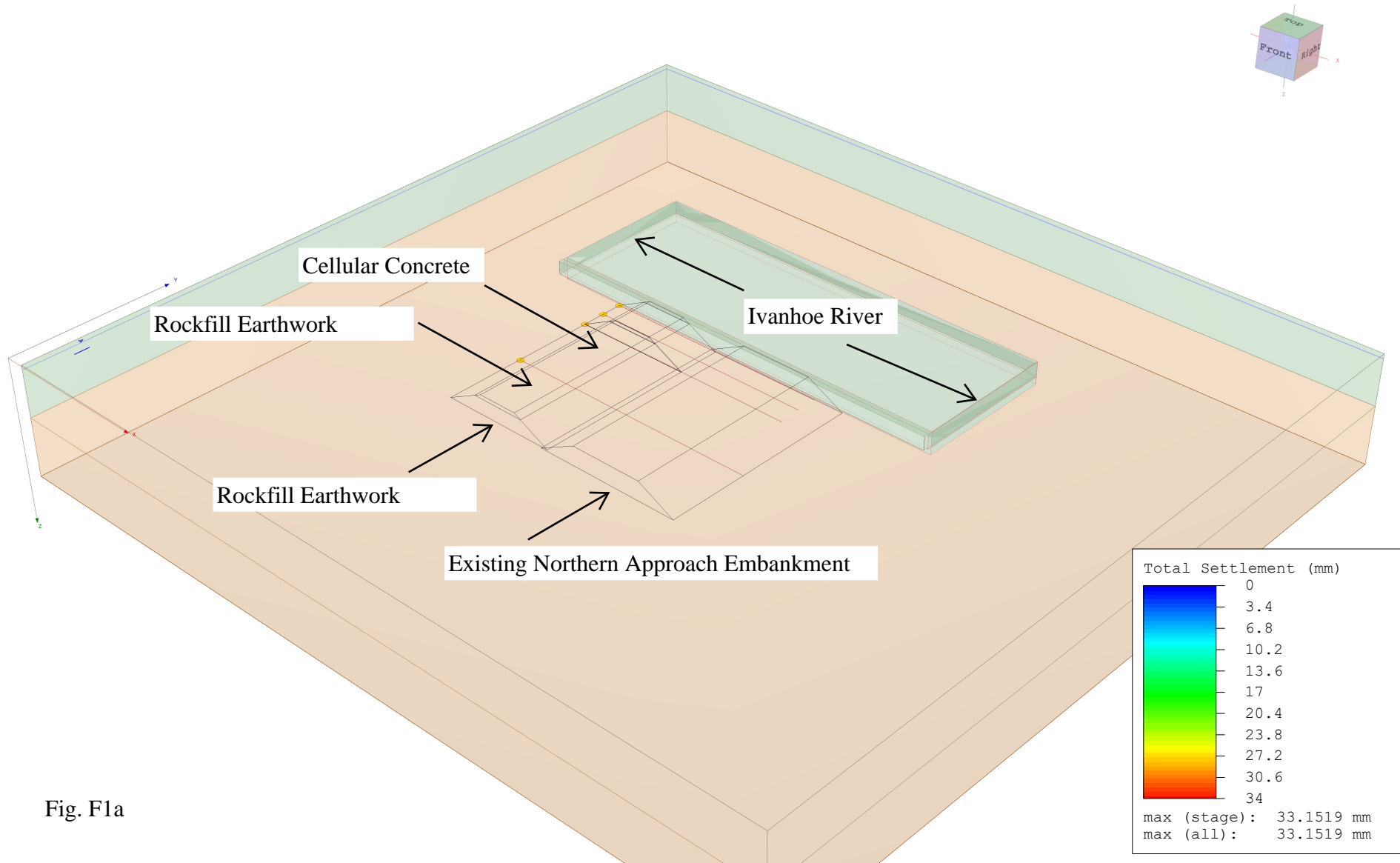


Fig. F1a

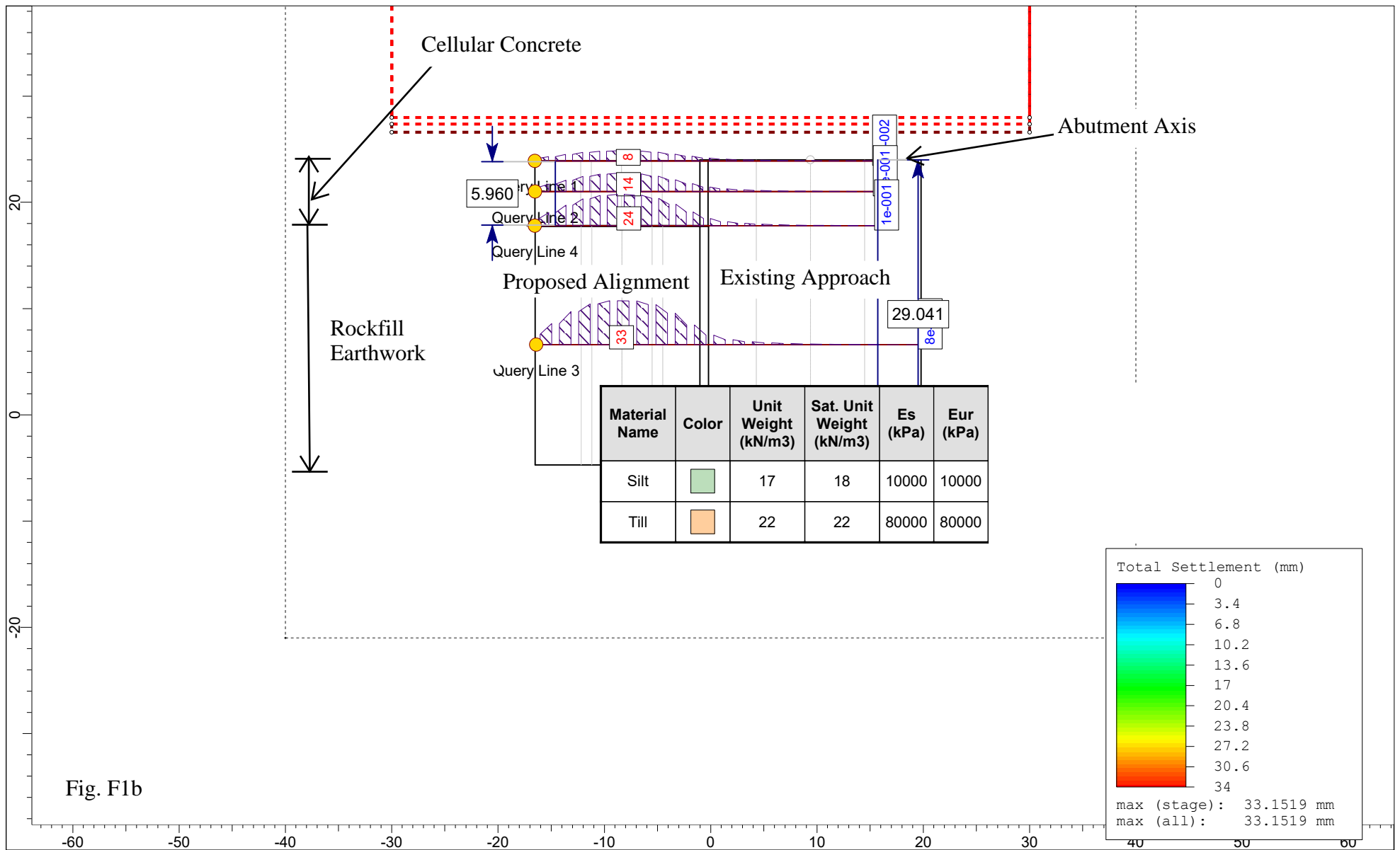
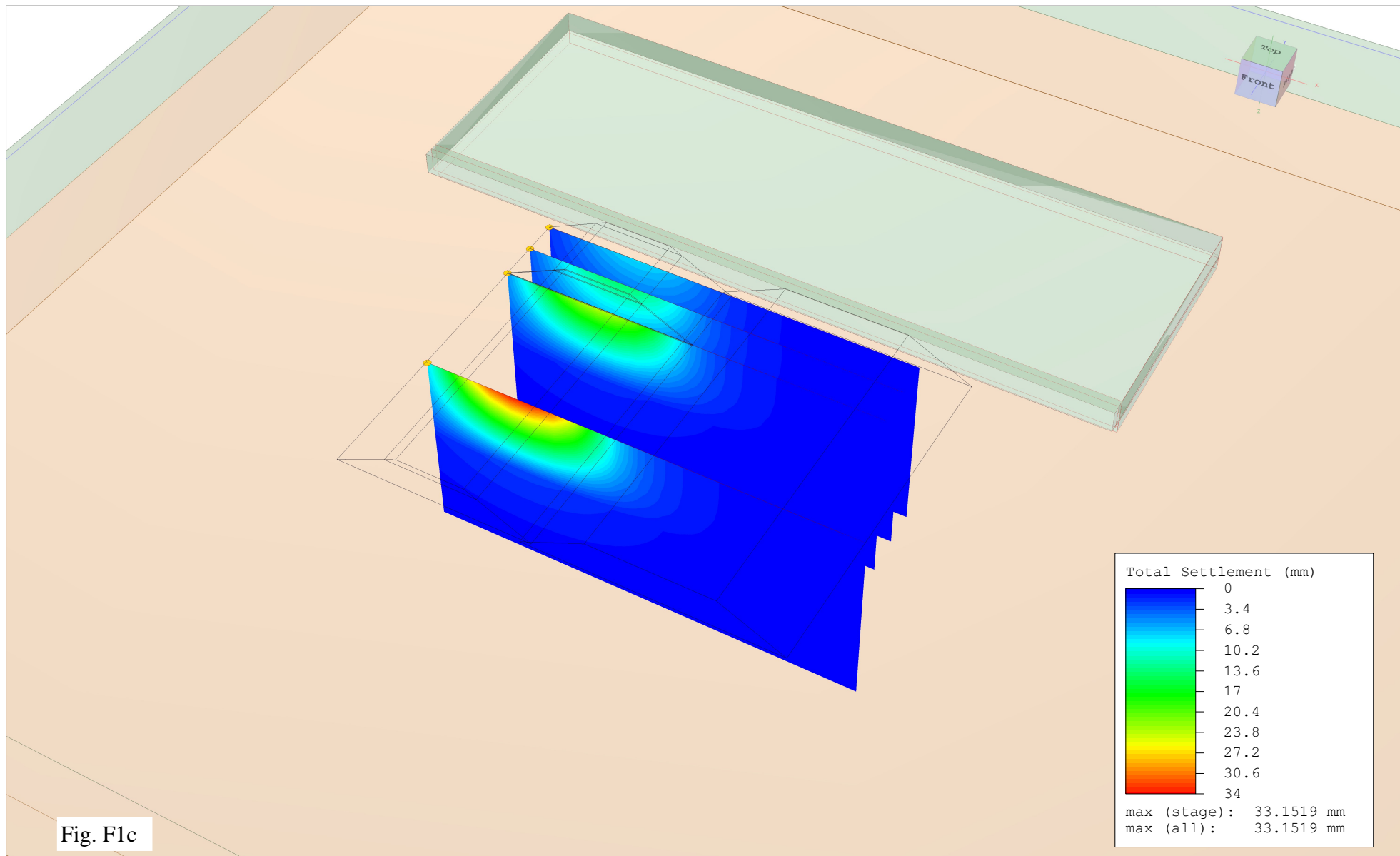



Fig. F1b

	Project	Ivanhoe Bridge	
	Analysis Description	Settlement Interaction	
	Drawn By	Company	WSP Canada Inc.
	Date	June 8, 2019	File Name Section 2-b1 - 20181224_vw_June7 F1.5.s3z



	Project		Ivanhoe Bridge
	Analysis Description		Settlement Interaction
	Drawn By	Company	WSP Canada Inc.
	Date	File Name	Section 2-b1 - 20181224_vw_June7 F1.5.s3z

June 8, 2019

# APPENDIX



# G

LIST OF OPSS, OPSD AND NSSP



## Appendix G: List of OPSSs, OPSDs And NSSPs referenced in the Report

Document	No	TITLE
OPSS	180	GENERAL SPECIFICATION FOR THE MANAGEMENT OF EXCESS MATERIALS
OPSS	206	CONSTRUCTION SPECIFICATION FOR GRADING
OPSS	401	CONSTRUCTION SPECIFICATION FOR TRENCHING, BACKFILLING AND COMPACTING
OPSS	492	CONSTRUCTION SPECIFICATION FOR SITE RESTORATION FOLLOWING INSTALLATION OF PIPELINES, UTILITIES, AND ASSOCIATED
OPSS	501	CONSTRUCTION SPECIFICATION FOR COMPACTING
OPSS.PRO V	539	CONSTRUCTION SPECIFICATION FOR TEMPORARY PROTECTION SYSTEMS
OPSS	803	CONSTRUCTION SPECIFICATION FOR SODDING
OPSS	804	CONSTRUCTION SPECIFICATION FOR SEED AND COVER
OPSS	903	CONSTRUCTION SPECIFICATION FOR DEEP FOUNDATIONS
OPSD	208.010	BENCHING OF EARTH SLOPES
OPSD	3090.100	FOUNDATION FROST DEPTHS FOR NORTHERN ONTARIO
OPSD	810.010	GENERAL RIP-RAP LAYOUT FOR SEWER AND CULVERT OUTLETS
NSSP		EXCAVATION FOR ROCK SOCKETS
NSSP		CELLULAR CONCRETE
NSSP		USE OF SACRIFICIAL LINERS



## **OBSTRUCTIONS**

### **Non-Standard Special Provision**

Foundation exploration boreholes coring bedrock have intercepted very strong rock bands in this project. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures in the excavation for rock sockets. The use of blasting for rock socket excavation is not permitted.

### ***Basis of Payment***

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

WSP Canada Inc.  
51 Constellation Court  
Toronto ON M9W 1K4

Tel: 1+ 416 798-0065  
Fax: 1+ 416 798-0518  
[www.wspgroup.com](http://www.wspgroup.com)

**Cellular Concrete - Item No.**

---

Non Standard Special Provision

Revised: September 26, 2018

---

**1.0 SCOPE**

This specification specifies the requirements for the supply and placement of cellular concrete for use as lightweight fill at the locations and in accordance with the details shown in the plans. The cellular concrete shall be placed in the dry condition and above any groundwater table.

**2.0 REFERENCES**

This specification refers to the following standards, specifications, or publications:

**Ontario Provincial Standard Specifications, Construction:**

OPSS 517      Dewatering  
OPSS 539      Temporary Protection System

**Ontario Provincial Standard Specifications, Material:**

OPSS 1301                      Cementing Materials  
OPSS 1302                      Water  
OPSS.PROV 1303              Admixtures for Concrete  
OPSS.PROV 1350              Concrete – Materials and Production

**ASTM**

ASTM C 150                  Portland Cement  
ASTM C 869                  Standard Specification for Foaming Agents Used in Making Preformed Foam for Cellular Concrete  
ASTM C 796                  Standard Test Method for Foaming Agents for Use in Producing Cellular Concrete Using Preformed Foam  
ASTM C 495-99a              Standard Test Method for Compressive Strength of Lightweight Insulating Concrete

**Ministry of Transportation Publication:**

LS - 407      Method of Test for Compressive Strength of Moulded Cylinders

**3.0 DEFINITIONS**

For the purpose of this specification the following definitions apply:

**Cellular Concrete:** Cellular concrete is a material with flowable consistency during placement, produced by the substitution of a uniform cellular structure of air cells (voids) for some or all of the aggregate particles found in standard concretes.

**Production Lot:** The quantity of cellular concrete produced for a continuously placed lift of cellular concrete.

**Project Superintendent:** means the cellular concrete's authorized representative in responsible charge of the construction of the cellular concrete

**Cellular Concrete Representative:** means an individual with continuous full-time employment with the cellular concrete supplier for a period of at least three (3) years, and who is knowledgeable in the design and construction of the cellular concrete.

#### **4.0 DESIGN AND SUBMISSION REQUIREMENTS**

##### **4.1 Prequalification of Cellular Concrete Product**

Prior to the commencement of work, the Contractor shall submit to the Contract Administrator a statement from the Supplier verifying that the Supplier has successfully put the product through the MTO Prequalification Process for Lightweight Fill and confirming that the product has been prequalified for use as lightweight fill by the MTO .

##### **4.2 Qualifications**

###### **4.2.1 Project Superintendent**

At least two weeks prior to commencement of construction of the cellular concrete, the name(s) of the project superintendent responsible for the placement of the cellular concrete in the Contract shall be submitted in writing to the Contract Administrator.

During construction of the cellular concrete, the project superintendent shall not change without written permission from the Contract Administrator. A proposal for a change in the project superintendent shall be submitted at least one week prior to the actual change in project superintendent.

###### **4.2.2 Cellular Concrete Representative**

At least two weeks prior to commencement of construction of the cellular concrete, the name(s) of the cellular concrete representative shall be submitted in writing to the Contract Administrator.

At least 48 hours written advance notice shall be provided to the Contract Administrator prior to each visit to the site by the cellular concrete representative. The advance notice shall include the dates and locations the cellular concrete representative will be on site.

##### **4.3 Submission of Shop Drawings and Placement Procedures**

The shop drawings and the proposed placement procedures shall be submitted to the Contract Administrator for review at least fifteen (15) business days prior to commencement of the work.

The submission shall include a description of the proposed method of installation including, as a minimum, the following:

- A work plan outlining the schedule, procedures and work site details;
- Proposed dewatering procedure (in accordance with OPSS 517);
- Environmental Protection procedures;
- Method for sealing cracks (if any) to prevent grout leakage;
- Method for bulkhead construction;
- List of equipment to be used;
- List of materials to be used;

The contractor must clearly identify how the grouting procedure will be monitored.

#### **4.4 Submission of Environmental Protection Strategy**

At least fifteen (15) business days before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of an environmental protection strategy as specified under Section 7.5.

### **5.0 MATERIALS**

#### **5.1 Cementing Materials**

Cementing materials shall be according to OPSS 1301. Supplementary cementing materials shall not be used.

#### **5.2 Water**

Water shall be free of contamination and any deleterious substance. Water shall conform to OPSS 1302.

#### **5.3 Admixtures**

Admixtures shall conform to OPSS.PROV 1303.

#### **5.4 Foaming Agents**

Foaming agents shall conform to the requirements of ASTM C 869 when tested in accordance with the provisions of ASTM C 796.

#### **5.5 Cellular Concrete Properties**

Cellular concrete shall have the following properties:

- Minimum unconfined compressive strength at 28 days of 0.5 MPa.
- Wet cast density of 475 kg/m<sup>3</sup> (4.66 kN/m<sup>3</sup>) (+/-5%)
- Must not contain any fly ash or any other waste or process by-product.

## **5.6 Prequalification of Cellular Concrete Product**

Prior to use, the Cellular concrete product must be “Prequalified” for use as lightweight fill by MTO (through the MTO Lightweight Fill Committee Prequalification Process).

## **6.0 EQUIPMENT**

Cellular concrete shall be produced utilizing specialized automated proportioning, mixing, and foam producing equipment, which is capable of meeting the specified properties.

Dry-mix equipment must be able to receive bulk cement and process it continuously from one piece of equipment, and pump through hoses or pipes up to a flat lineal distance of 1000 metres. Wet-mix equipment must be able to receive slurry on-site into the equipment and process it continuously during ready-mix supply, and pump through hoses or pipes up to a flat lineal distance of 200 metres.

Cellular concrete must be pumped by a positive displacement pump. A foam generator shall be used to continuously produce pre-formed foam, which shall be injected and mixed with the cementitious slurry downstream of the positive displacement slurry pump. The equipment shall be calibrated to produce a precise, consistent and predictable volumetric rate of foam with stable uniform microbubbles.

## **7.0 CONSTRUCTION**

### **7.1 Excavation and Subgrade Preparation**

Foundation excavation shall be carried out to the design elevations and the horizontal and vertical limits shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be sub excavated.

The prepared subgrade shall be good competent level ground and snow and ice must be removed from the area prior to placement.

### **7.2 Dewatering**

The prepared subgrade shall be free of standing water during placement of cellular concrete and until backfill is placed on top of the cellular concrete. If necessary, dewatering shall be continuous during placement of materials.

Dewatering shall be according to OPSS 517.

### **7.3 Roadway Protection System**

The construction of all protection schemes shall be according to OPSS 539 and paid for under the appropriate tender item. Where the stability, safety or function of an existing roadway, railway, other works, or proposed works may be impaired due to the method of operation, such protection as may be required shall be provided.

#### **7.4 Placement**

The construction of cellular concrete shall be scheduled such that it is at all times under the responsible charge of the project superintendent who has been advised on site by the manufacturer's representative as to the required procedures and schedule for the placement of the cellular concrete.

The cellular concrete representative shall be on site to oversee the placement of the cellular concrete and to verify that the cellular concrete is being supplied and placement in accordance with the contractual requirements.

Any items to be fully or partially encased in the cellular concrete shall be properly set and stable prior to the installation of the cellular concrete.

Where required, formwork should be designed and installed to withhold cellular concrete. When working near surface water, formworks shall be lined with an impermeable liner to prevent any leakage.

Cellular concrete shall not be allowed on frozen ground. Cellular concrete may be placed during freezing conditions, provided measures are taken to prevent damage to the cellular concrete until sufficient strength has been attained. Care should be taken to avoid freezing before initial set. Cold weather protection shall be provided in accordance with OPSS.PROV 1350.

Cellular concrete must not be placed during heavy or prolonged precipitation.

Once mixed, the cellular concrete shall be conveyed promptly to the location of placement without excessive handling.

The Constructor shall determine the maximum lift thickness based on density and any other considerations that may impact placement. Cellular concrete shall be cast in a formed area within 1 to 2 hours, to permit an undisturbed setting.

Finished surface elevation shall be within  $\pm 25$  mm of the design grades shown on the drawings. Cellular Concrete can be placed with a maximum slope of 1%. Slopes greater than 1% will require profiling by creating steps for the Cellular Concrete with formwork.

Vehicles, equipment, backfills or other loadings on the cellular concrete shall be prevented until the material has attained sufficient strength to withstand the loads with no damage. Backfill can commence on the cellular concrete when the cellular concrete has attained sufficient strength such that foot traffic can be supported without leaving an indentation.

#### **7.5 Environmental Protection**

The Contractor shall handle materials and conduct the work in a manner that will ensure protection of the natural environment and prohibit cellular concrete from entering surface or ground water. The Contractor shall take measures as necessary to prevent the material from entering the natural environment and/or leaking outside of the intended placement location, and shall have established methods for stopping flow of the product as required, and for prompt remediation of any leaks or spills. These measures and any other contingency planning requirements shall be documented in an Environmental Protection Strategy.



## **8.0 QUALITY ASSURANCE**

### **8.1 Field Sampling and Testing**

The fresh cellular concrete shall be collected for density testing once per production run, or once for every 50 cubic metres, or once per 30 minutes, whichever is more frequent.

Cellular concrete samples shall be captured, cured, and tested to verify the specified compressive strength and the dry unit weight. The unit weight shall be maintained within +/- 5 % of the design unit weight and shall be adjusted as required to obtain the specified density at the point of placement. One sample is comprised of one set of four cellular concrete cylinders. One sample should be taken for each placement, or every 100m<sup>3</sup>, whichever is more frequent. Cylinders are cast in 75 mm by 150 mm cylindrical plastic molds. Cellular concrete cylinders shall be cured and tested for compressive strength as per ASTM C495-07 and LS 407.

## **9.0 MEASUREMENT FOR PAYMENT**

Measurement will be Plan Quantity as may be revised by adjusted Plan Quantity of the cellular concrete in cubic metres.

## **10.0 BASIS OF PAYMENT**

Payment at the contract price for the cellular concrete shall be full compensation for all labour, equipment and material to do the work.



### **Use of Sacrificial Liners**

#### **Non-Standard Special Provision**

Foundation exploration boreholes have intercepted flowing sand conditions in the subsurface geology. Sacrificial liners must be used to avoid significant impacts on the drilled sockets which can otherwise manifest on liner retrieval.

#### ***Basis of Payment***

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

WSP Canada Inc.  
51 Constellation Court  
Toronto ON M9W 1K4

Tel: 1+ 416 798-0065  
Fax: 1+ 416 798-0518  
[www.wspgroup.com](http://www.wspgroup.com)

## **DEWATERING STRUCTURE EXCAVATIONS - Item No.**

---

Special Provision No. FOUN 0003

March 8, 2018

---

### **Amendment to OPSS 902, November 2010**

#### **902.02 REFERENCES**

Section 902.02 of OPSS 902 is amended by the addition of the following:

#### **Ontario Provincial Standard Specifications, Construction**

OPSS 517      Dewatering  
OPSS 805      Temporary Erosion and Sediment Control Measures

#### **902.03 DEFINITIONS**

Section 903.03 of OPSS 902 is amended by the addition of the following:

**Automatic Transfer Switch** means as defined in OPSS 517.

**Cofferdam** means as defined in OPSS 539.

**Cut-Off Wall** means as defined in OPSS 517.

**Design Storm Return Period** means as defined in OPSS 517.

**Dewatering System** means as defined in OPSS 517.

**Groundwater Control System** means as defined in OPSS 517.

**Plug** means as defined in OPSS 517.

**Sediment** means as defined in OPSS 517.

**Sediment Control Measure** means as defined in OPSS 517.

**Temporary Flow Passage System** means as defined in OPSS 517.

**Unwatering** means as defined in OPSS 517.

**Vegetated Discharge Area** means as defined in OPSS 517.

**Waterbody** means as defined in OPSS 517.

**Watercourse** means as defined in OPSS 517.

## **902.04 DESIGN AND SUBMISSION REQUIREMENTS**

### **902.04.01 Design Requirements**

#### **902.04.01.01 Dewatering**

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a [No in-water works are required for this project to necessitate temporary flow passages] year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

### **902.04.02 Submission Requirements**

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

#### **902.04.02.01 Working Drawings**

Working Drawings for the dewatering system shall be according to OPSS 517.

#### **902.04.02.02 Preconstruction Survey**

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of [Tremie concrete pouring will be required for this project; No adjacent properties in the immediate local rural neighbourhood] metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

#### **902.04.02.03 Milestone Inspections**

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

## **902.07 CONSTRUCTION**

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

## **902.07.04                      Dewatering Structure Excavation**

### **902.07.04.01                      General**

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

### **902.07.04.02                      Discharge of Water**

The discharge of water shall be according to OPSS 517.

### **902.07.04.03                      Monitoring**

Monitoring shall be according to OPSS 517.

### **902.07.04.04                      System Amendments**

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

### **902.07.04.05                      Removal**

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:


\* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

\*\* Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this standard tender item **only** on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.

# APPENDIX



## H

EXTRUDED POLYSTYRENE FOR FROST TREATMENT



## **Extruded Polystyrene Frost Heave Treatment**

### **Non-Standard Special Provision**

#### ***Scope***

As part of the work under the above tender item, the Contractor shall supply 120 mm thick extruded polystyrene rigid board insulation to manage frost heave effects.

#### ***References***

#### ***Materials***

The thermal insulation shall be of the following:

##### ***Product***

Styrofoam High Load 40

(Or approved equivalent)

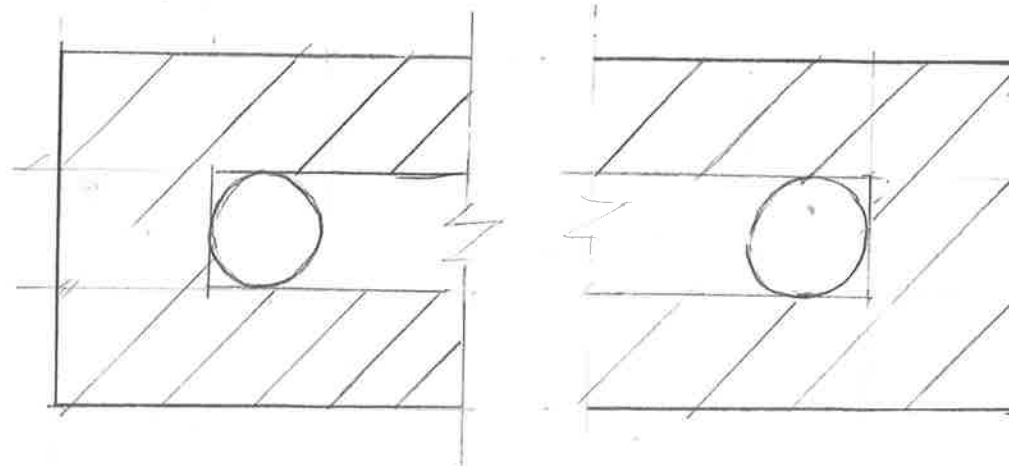
##### ***Manufacturer***

Dow Chemical Canada Inc.

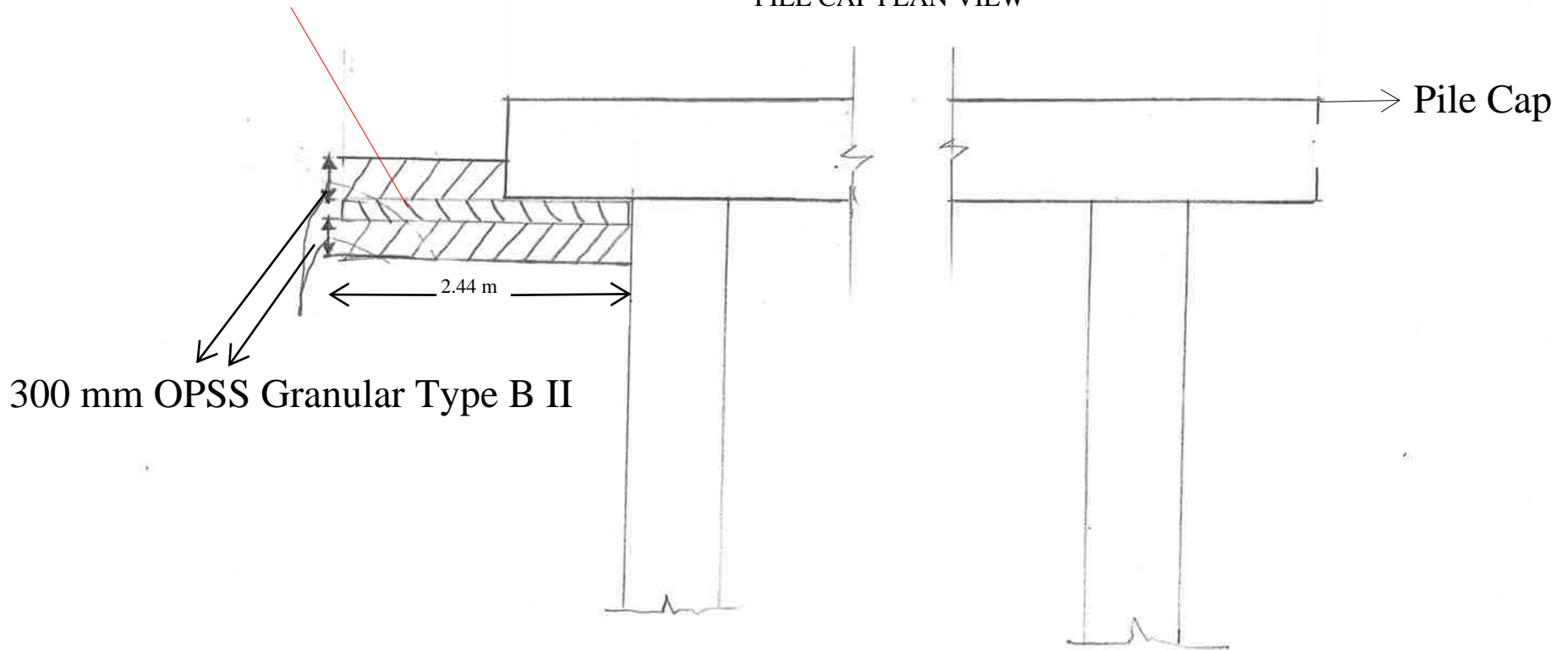
#### ***Basis of Payment***

Payment at the Contract Price for the above tender items shall be full compensation for all labour, equipment and material to do the work.

See Previous page  
Titled "Extruded  
Polystyrene Frost  
Heave Treatment"  
(Thickness 120 mm)



PILE CAP PLAN VIEW

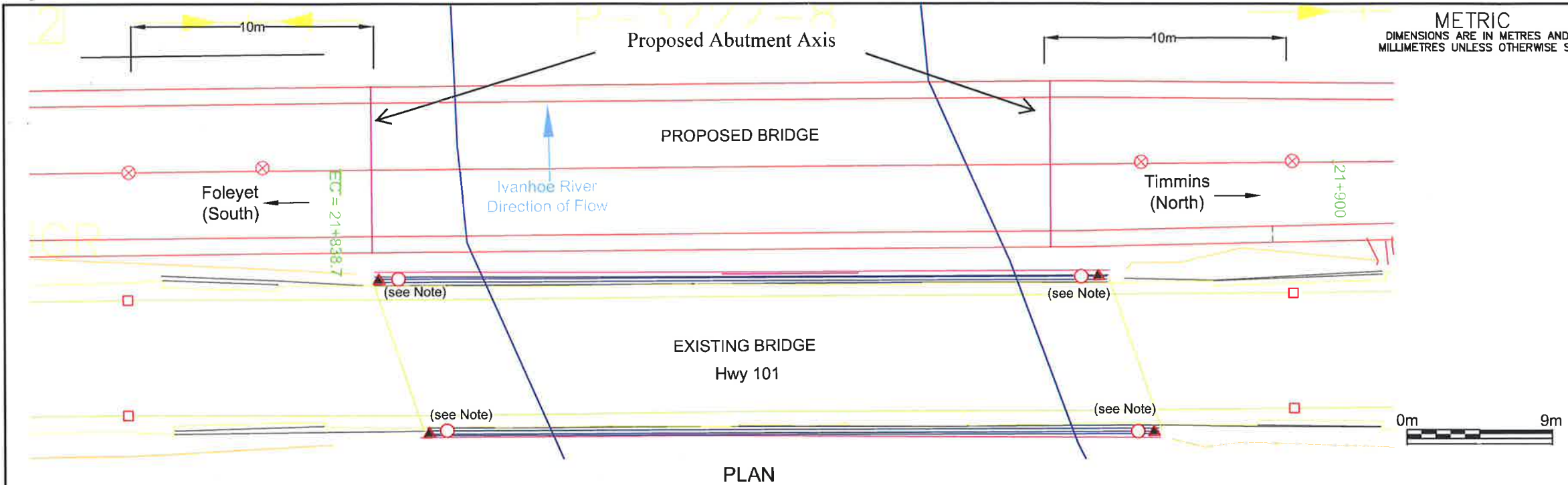


Schematic Diagram For Layout of Soil Insulation

# APPENDIX



GROUND MOVEMENTS/GROUND VIBRATIONS MONITORING

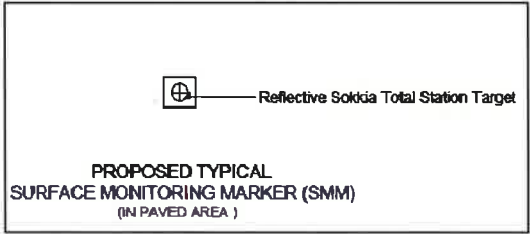
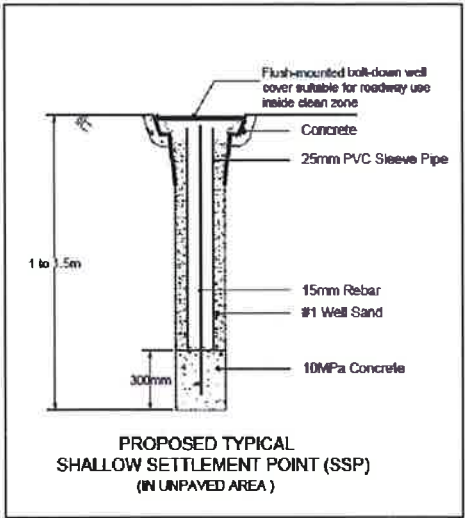


### Notes

The targets shall be placed at the respective ends of the bridge parapet walls

### Legend

- ▲ V Ground Vibration Monitoring Points
- S1 Settlement Targets on Existing Highway 101 (Shallow Settlement Point, SSP)
- S2 Settlement Targets on the existing bridge Abutment (Surface Monitoring Marker, SMM)
- ⊗ S3 Settlement Plates (SP) to Monitor Settlements on the Proposed Bridge Approaches



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

CONT No:  
5266-13-00

Hwy 101 and Hwy 7172  
BOREHOLE LOCATIONS & SOIL STRATA



51 Constellation Court  
Toronto, Ontario  
M9W 1K4



**KEY PLAN**  
NOT TO SCALE

### LEGEND

- ◆ Borehole
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- ⊗ WL upon completion
- ⊗ WL in Piezometer
- ⊗ Piezometer

BH No.	ELEV. (m)	MTM NAD83 ZONE 13 CO-ORDINATES	
		NORTH (m)	EAST (m)
BH17-1	322.5	5346609.5	420051.5
BH17-2	322.6	5346547.0	420052.0
BH17-3	321.3	5346539.3	420039.5
BH17-4	321.3	5346550.4	420037.7
BH17-5	321.6	5346598.5	420033.0
BH17-5A	321.6	5346595.9	420032.5
BH17-6	321.7	5346613.1	420031.3
BH18-1	321.3	5346553.1	420036.3
BH18-2	321.6	5346593.6	420028.3

### NOTES

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

Drawings are based on "WP 5266-13-00-SB bridge&hwy 7172 NEW DESIGN 11.5m OFFSET- PLAN&PROFILE WITH BOREHOLE LOCATIONS.dwg" received Nov. 18 2018 and "GA-90%" received Dec. 8 2018

REVISIONS	DATE	BY	Revised	DESCRIPTION
Jun. 10/19	RA			
DATE	BY			
WSP No. 171-02344-00				
HWY No 101	CHECKED FO	DATE	Jun. 10/19	SITE
SUBM'D	CHECKED FO	APPROVED	VW	DWG
DRAWN	RA	CHECKED FO	APPROVED	VW

SOIL STRATA SYMBOLS

**Non-Standard Special Provision (NSSP)**  
**Ground/structure movement and vibration monitoring**  
Proposed Ivanhoe River Bridge Replacement, GWP: 5266-13-00

## **1.0 SCOPE**

This special provision covers the requirements for ground/structure movement monitoring comprising the supply, installation, monitoring of square steel settlement plates with survey rods (SPs), shallow settlement points (SSP) and surface monitoring markers (SMM) as well as ground vibration monitoring of existing bridge abutments, primarily to monitor the construction impacts of the rock socket constructions.

The locations of the various ground movement (x,y,z) monitoring devices and placement locations for geophones for ground vibration monitoring are shown on Dwg I in Appendix I.

## **2.0 INSTRUMENTATION – Ground Movement Monitoring**

### **2.1 Settlement Plates (SPs)**

#### Steel Base Plate

The Contractor shall supply minimum Grade 300 steel base plates with a thickness of at least 6.35 mm. The plates shall be square and at least 500 mm by 500 mm in plan dimensions. The steel base plates shall be placed on the prepared subgrade for the proposed bridge approach fills.

#### Survey Rod

The Contractor shall supply a Schedule 40 steel pipe with an outside diameter (O.D.) not less than 25.4 mm, supplied in lengths as required to complete the installation as described elsewhere. The lowest rod shall be welded to a settlement monitoring plate. The top end of the topmost length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to for repeated observations. Intermediate rods shall be threaded both ends to facilitate extensions commensurate with embankment height increases.

#### Friction Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 PVC pipe, 50.8 mm O.D., cut perpendicular to the axis of the pipe.

#### Protective Surround

The Contractor shall supply a protective surround for the portion of the rod within the embankment. The surround shall consist of 300 mm diameter corrugated steel pipe (CSP, OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the Friction Reduction Sleeve (PVC pipe) shall be filled with loose dry medium sand.

#### Placement Elevations

Placement elevation of the Settlement Plates should be at a depth of 1.8 m below the elevation of the underside of the pavement structure at the locations marked on Dwg. I.

#### Installation

The actual placement elevation shall be surveyed. As embankment height increases the rods shall be extended above the new top of embankment. Sleeves shall be installed around the rods to reduce friction and allow uninhibited movement of the rod with the plate. The protective surround shall be extended with the rods as embankment construction proceeds.

Settlement plates shall be installed on horizontal ground surface with the SP base placed on locally chinked levelled surface. The elevation of the top of plate shall be surveyed before backfilling and further embankment construction. The rods shall be fixed to the centres of, and perpendicular to, the plates. The coupling of the rods shall be such that all sections have the

same axis, with no separation or contraction at the couplings. The friction reducing sleeve shall be placed over the entire length of the rod that is within the embankment, except that the cap on top of the settlement rod shall extend 25 mm above the top of the friction sleeve at all times. The settlement rods shall be extended upwards as fill is placed such that the top of the rod is always at least 0.3 m but not more than 2 m above the surrounding fill. The CSP, Friction Reducing Sleeve and sand protective surround shall be extended with the rods. The settlement rod shall be in the centre of the CSP and friction-reducing sleeve. The annulus between the CSP and the friction-reducing sleeve shall be filled with loose dry medium sand to a level not higher than the top of the sleeve.

The elevation and location coordinates of the centres of the plates and the tops of rods shall be surveyed during the embankment build-up.

## **2.2 Shallow Settlement Points (SSPs) and Surface Monitoring Markers (SMMs)**

Schematics in Dwg. I give main details of these targets. These targets must be installed on each abutment side as per the Dwg. I before the commencement of construction of rock sockets or any temporary sheet pile driving undertaken by the Contractor on the respective abutment side for the new alignment.

## **2.3 Survey Benchmarks**

Ground movements shall be measured by level surveying with total station equipment at the top of the settlement rods, top of the inner rod of SSPs and on top of SSMs.

The Contractor shall provide local, stable and non-settling survey benchmarks located a minimum distance of 25 m from any instrument location. The number and locations of benchmarks shall be such that direct sighting is possible from all settlement target locations to at least one bench mark. Elevations shall be surveyed to an accuracy of  $\pm 2$  mm or better. Prior to the installation of the instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain ground elevations at each instrument location.

## **2.4 Alternative Methods of Ground Movement Monitoring**

The Contractor may propose alternative methods for measuring the movement of the ground beneath the approach embankments. The proposal shall be signed off by a Professional Engineer and shall be subject to approval prior to implementation.

## **2.5 Ground Movement Monitoring Data Requirements**

Ground movement monitoring data shall be provided to the Contract Administrator along with copies of all field notes, in accordance with the schedule shown below:

- Immediately after installation, indicating the location of the permanent reliable benchmarks used for ongoing ground movement monitoring (Section 2.3).
- Ground Movements shall be recorded in conformance with the schedule shown in Table 1.

**Table 1. Ground Movement Monitoring Schedule**

Type of Target	Frequency	Alert Level
SSPs and SMMs	<ul style="list-style-type: none"> <li>After the base reading (base reading needs to be taken before the construction of rock sockets or any adjacent sheet pile driving on the particular side of the bridge, before first such activity; Twice a day (AM/PM) during the construction of rock sockets or any adjacent sheet pile driving; daily during the construction of the approach embankments (i.e. within 20 m from each proposed bridge abutment); thereafter weekly; or terminated on earlier reaching of practical cessation of ground movements <u>after embankment has reached the underside of the pavement structure elevation</u>; practical cessation is defined as 2 mm or less of movement increase in a month (accuracy of the instrument).</li> </ul>	<ul style="list-style-type: none"> <li>10 mm in either x or y or z for SSMs</li> <li>25 mm in either x or y or z for SSPs</li> </ul>
SPs	<ul style="list-style-type: none"> <li>After the base reading (base reading shall be taken before the construction of the approach rockfill embankment (i.e. within 20 m from each proposed bridge abutment); Once a day during the construction of the approach embankment; thereafter weekly; or terminated on earlier reaching of practical cessation of ground movements <u>after embankment has reached the underside of the pavement structure elevation</u>; practical cessation is defined as 2 mm or less of movement increase in a month (accuracy of the instrument).</li> </ul>	<ul style="list-style-type: none"> <li>50 mm in either x or y or z for SPs</li> </ul>

## 2.6 Measurement Tolerances

The tolerances for settlement monitoring shall be as follows:

- Instrument location:  $\pm 100$  mm.

Error of survey closure from permanent bench mark  $\pm 2$  mm.

## 3.0 INSTRUMENTATION – Ground Vibration Monitoring

### 3.1 Instrumentation Requirements

A seismograph configured to produce a continuous record shall be used for vibration monitoring that will record vibrations in three orthogonal directions including peak particle velocity, half-wave-frequency, peak acceleration, peak displacement, and resultant particle velocity.



### 3.2 Ground Vibration Monitoring Data Requirements

Type of Intervention	Frequency	Alert Level
Pre-Construction Measurements of background vibration	<ul style="list-style-type: none"> <li>The device shall be set to the lowest possible trigger threshold of peak particle velocity (PPV) and monitoring durations of 2 seconds, 15 seconds and 1 minute shall be monitored. The measurement cycle shall be repeated a second time at each geophone location. After the base reading – (Base Reading)</li> </ul>	<ul style="list-style-type: none"> <li>Not Applicable</li> </ul>
During rock socket construction or any vicinity sheet pile driving or extraction	<ul style="list-style-type: none"> <li>Continuous monitoring <u>for the duration of the particular activity</u> of the geophones on the particular side of the existing bridge abutment; ground vibration monitoring on a bridge abutment can be terminated if such rock socket constructions or any temporary sheet pile driving or extraction works are complete and not required during the Contract on the particular side of the bridge.</li> </ul>	<ul style="list-style-type: none"> <li>PPV of 30 mm/s</li> </ul>

### 4.0 RESONSE PLANS FOR ALERT LEVELS

1. Alert levels are not to be exceeded. Activities may be suspended in affected area with the exception of those actions necessary to avoid the exceedance of the Alert Level or to make the work and affected properties safe and secure.
2. If the Alert Level is reached:
  - a. Inform the Contract Administrator immediately
  - b. Work may be stopped by Contract Administrator
  - c. The Contractor shall meet with Contract Administrator to collaborate and discuss response action (s) and participate in the development of an Alert-Level Response Plan of Action and mitigation resolution for the alert level.
  - d. Install and monitor additional instruments as directed by the Contract Administrator.
  - e. The Contractor shall implement, the reviewed and accepted Alert-Level Response Plan of Action.

## **5.0 SAFETY AND PROTECTION**

The above ground location of target monitoring fixtures shall be made clearly visible to avoid accidental damage at all times. Markings shall be of sufficient size to be visible to construction equipment operators and after heavy snowfalls.

Instruments shall be clearly labelled in the field, each instrument having a unique identifier. The labelling shall remain legible for the duration of the monitoring period.

All instruments shall be adequately protected by the Contractor such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced and resurveyed at the Contractor's cost.

# APPENDIX

**J**

**LIMITATIONS OF REPORT**

## **LIMITATIONS OF REPORT**

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to WSP Canada Inc. at the time of preparation. Unless otherwise agreed in writing by WSP Canada Inc., it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. WSP Canada Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.