



**FOUNDATION INVESTIGATION AND DESIGN REPORT
REHABILITATION OF FOUR (4) STRUCTURES IN HUNTSVILLE AREA
KAHSHE RIVER CULVERT EMBANKMENT STABILITY
HIGHWAY 11
GWP 5451-09-01; SITE NO. 42-105C
GRAVENHURST, ONTARIO**

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PML Ref.: 14TF009
Index No.: 023FIR and 024FDR
GEOCRES No.: 31D-586
February 2, 2015



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FOUNDATION INVESTIGATION REPORT
for
Kahshe River Culvert Embankment Stability
Highway 11
GWP 5451-09-01; Site No. 42-105C
Gravenhurst, Ontario

1. INTRODUCTION

This report summarizes the results of a foundation investigation carried out on the west side of Highway 11 at the Kahshe River Culvert location (Site # 42-105/C) in Gravenhurst, Ontario. The study is part of a project to rehabilitate 4 structures in the Huntsville Area. The study was carried out by Peto MacCallum Ltd. (PML) for AECOM, on behalf of the Ministry of Transportation of Ontario (MTO).

This study is intended to obtain the soils information necessary to evaluate the stability of the embankment slope and to provide foundation recommendations for a possible retaining wall across the top of the culvert, if necessary.

The purpose of this report is to summarize the subsurface stratigraphy encountered at the site, during the investigation.

2. SITE DESCRIPTION AND GEOLOGY

An existing twin cell concrete box culvert is present at the site and numerous rock fragments were observed on the surface of the embankment. It is understood from the request for proposal provided by the MTO, that there has been some movement in the embankment slope in the study area.

Refer to Appendix A for site photos of the upstream and downstream embankments.

Land use in the vicinity of the site includes the Highway 11 transportation corridor and scattered residential / commercial properties to the north and south along Highway 11. Sparse to dense stands of trees are located to the east and west of the site, with numerous bedrock outcrops



visible along the river banks. The local topography is governed by the existing road embankment and river valley, with topography generally sloping down from the roadway and towards the river. The embankment slopes from Highway 11 towards Kahshe River at an approximate inclination of between 1 horizontal to 1 vertical (1H:1V) and 1.5H:1V. At the time of the investigation the river was about 15 to 20 m wide.

The project site is located within the physiographic region known as the Number 11 Strip. The Number 11 Strip is a narrow strip of land which extends from Gravenhurst to North Bay where extensive sand deposits interspaced with rock knobs and outcrops are encountered. The bedrock at this site was visible at ground surface along Kahshe River and was encountered in the boreholes at a depth of up to 9 m beneath the Highway 11 embankment.

3. INVESTIGATION PROCEDURES

The field work for this study was carried out on June 19, 2014. Three boreholes were drilled through the existing Highway 11 embankment from the road surface, two auger probes were advanced using manual equipment through the side of the Highway 11 embankment, one dynamic cone penetration test was completed 1.0 m south of borehole 3 and one dynamic cone penetration test was completed between boreholes 1 and 2. The investigations at the site were advanced to depths of 0.3 to 8.6 m and at the locations shown on Drawing 1, appended.

The borehole locations were strategically located to provide data for a slope stability analysis and retaining wall foundations. The borehole locations and elevations were surveyed in the field by exp Geomatics for AECOM. All elevations in this report are expressed in metres.

The boreholes were advanced using continuous flight hollow stem augers with a truck-mounted D-90 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a PML field supervisor.



Soil samples were recovered from the boreholes at regular 0.75 and 1.5 m intervals following the standard penetration testing. Standard penetration tests and dynamic cone penetration tests were conducted to assess the strength characteristics of the substrata. Soils were identified in accordance with the MTO soil classification manual procedures.

The groundwater conditions in the boreholes were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved.

The boreholes were backfilled with a bentonite/grout mixture where required in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment procedures.

The recovered soil samples were returned to our laboratory in Toronto for detailed visual examination, laboratory testing and classification. The laboratory testing program included the following tests:

- Natural moisture content determinations (20)
- Grain size distribution analyses (6)

The charts prepared using the results of the laboratory grain size distribution analyses are presented in Figures K-GS-1 and K-GS-2. All of the test results are summarized on the Record of Borehole sheets.

4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, standard penetration test data, dynamic cone penetration data and groundwater observations. The results of laboratory grain size distribution analyses and moisture content determinations are also shown on the Record of Borehole sheets.



The borehole locations and stratigraphic profile prepared from the borehole data are shown on Drawing K-1. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised the existing Highway 11 pavement structure over compact to very loose fill which locally extended to probable bedrock in borehole 1 at 1.7 m and to a gravelly sand to sand layer in boreholes 2 and 3 at 6.4 and 7.9 m. The gravelly sand to sand layers were 0.6 m thick and overlaid probable bedrock at 7.0 and 8.5 m in boreholes 2 and 3, respectively. The auger probes revealed 0.3 m of sand over refusal to auger on likely rockfill placed over the typical sandy fill embankment. The dynamic cone penetration tests encountered refusal to advance at 2.6 and 8.5 m.

A summary of the findings is given below.

4.1 Fill

A 1.7 to 7.9 m thick embankment fill was encountered in all the boreholes and extended to estimated elevation 224.2 to 229.9. The fill included 80 to 200 mm of asphaltic concrete, underlain by granular fill grading from sand and gravel to sand trace silt. Cobbles were noted within the fill in borehole 2. The fill was typically compact (locally dense) in the upper 1.5 to 2.3 m, SPT-'N' values of 17 to 41 and became loose to very loose with depth, SPT-'N' values of 1 to 5 in the lower portion. The material was moist with moisture contents varying from 3 to 12%. The results of grain size distribution analyses performed on 5 samples of the sand fill are presented in Figure K-GS-1. Although the fill is interpreted to be granular backfill to the culvert, its composition may differ depending across the site and near the surface of the slope. The fill may contain different materials such as rock fill, cobbles and/or boulders.



4.2 Sand to Gravelly Sand

A 0.6 m thick gravelly sand and sand layer was contacted beneath the fill at 6.4 and 7.9 m in boreholes 2 and 3 (elevations 225.3 and 224.2) that extended to the 7.0 and 8.5 m borehole termination by refusal depths (elevations 224.7 and 223.6), respectively. The gravelly sand from borehole 2 was loose (SPT-'N' value of 9) and the sand from borehole 3 was very dense (SPT-'N' value of 50 blows for 10 cm). The material was moist to wet, with a moisture content of 18% recorded in the sand layer from borehole 3. Organic inclusions were noted within the layer in both boreholes. The results of a grain size distribution analysis performed on a sample of the gravelly sand from borehole 2 are presented on Figure K-GS-2. The deposit may contain cobbles and boulders.

4.3 Probable Bedrock

Refusal to auger on probable bedrock was encountered in boreholes 1 to 3 and dynamic cone penetration test DCPT-1 at 1.7 to 8.5 m (elevations 229.9 to 223.6). The probable bedrock sloped downwards from the elevation 229.9 at the south (borehole 1) to elevation 223.6 at the north (borehole 3). The probable bedrock surface was determined at boreholes locations and is expected to be variable across the site due the undulating nature of the bedrock in this area.

4.4 Groundwater

In the process of augering, a water strike was observed at of 7.9 m (elevation 224.2) in borehole 3. All other boreholes remained dry during and upon completion of drilling.

The water level in the Kahshe River was at elevation 224.5 on June 19, 2014. The groundwater levels in the embankment at the site are influenced by the water level in the river and also subject to seasonal fluctuation and precipitation patterns.



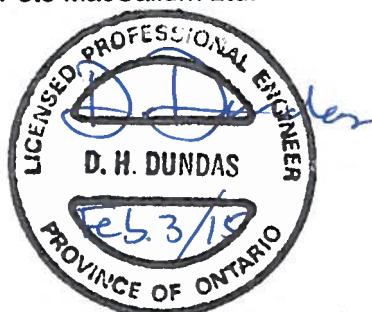
5. CLOSURE

Mr. F. Portela carried out the field investigation for this study under the supervision of Mr. A. DeSira, MEng, P. Eng. Walker Soil Drilling supplied the drill rig for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.

This Foundation Investigation report was prepared by Mr. A. DeSira, MEng, P.Eng., and reviewed by Mr. D. Dundas, P.Eng., Senior Engineer, Geotechnical Services. Mr. C.M.P. Nascimento, P.Eng., Project Manager and MTO Designated Principal contact, conducted an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.

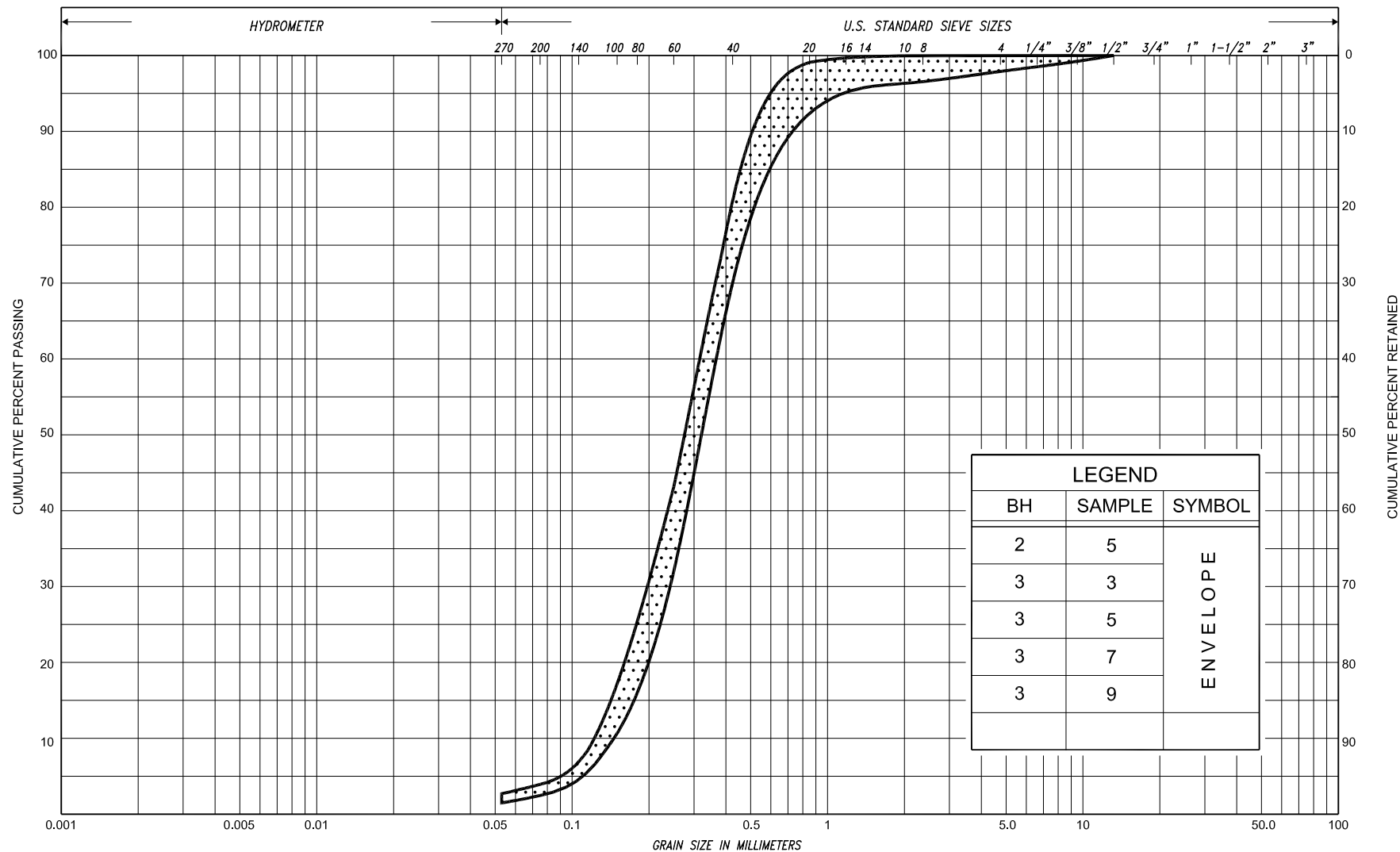


David Dundas, P.Eng.
Senior Engineer, Geotechnical Services



Carlos M.P. Nascimento, P.Eng.
Project Manager and
MTO Designated Principal Contact

DD/CN:dd-mi



SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED		
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT															
CLAY			SILT			V. FINE	FINE	MED.	COARSE		GRAVEL					U.S. BUREAU
						SAND										



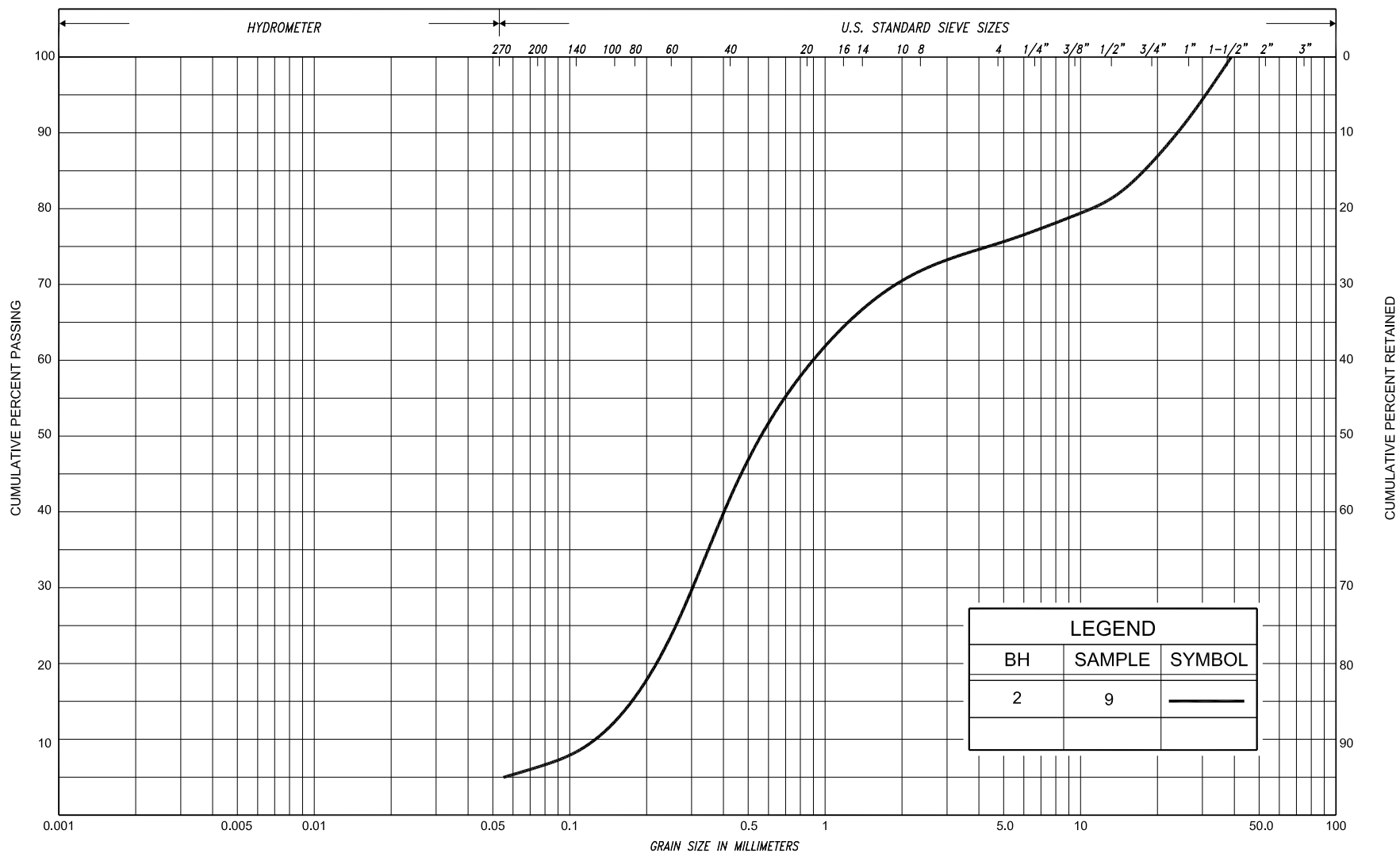
GRAIN SIZE DISTRIBUTION

SAND, trace silt, trace gravel
(FILL)

FIG No. K-GS-1

HWY: 11

G.W.P. No. 5451-09-01



SILT & CLAY				GRAIN SIZE IN MILLIMETERS			GRAVEL		COBBLES	UNIFIED
				FINE	MEDIUM	COARSE				
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL		COBBLES	M.I.T.
CLAY		SILT		V. FINE	FINE	MED.	COARSE	GRAVEL		U.S. BUREAU



GRAIN SIZE DISTRIBUTION

GRAVELLY SAND, trace silt

FIG No. K-GS-2

HWY: 11

G.W.P. No. 5451-09-01

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 - 10	10 - 20	20 - 30	30 - 40	> 40
	TRACE	SOME	WITH	ADJECTIVE (SILTY)	AND (AND SILT)

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S SPLIT SPOON	T P THINWALL PISTON
W S WASH SAMPLE	O S OSTERBERG SAMPLE
S T SLOTTED TUBE SAMPLE	R C ROCK CORE
B S BLOCK SAMPLE	P H T W ADVANCED HYDRAULICALLY
C S CHUNK SAMPLE	P M T W ADVANCED MANUALLY
T W THINWALL OPEN	F S FOIL SAMPLE
F V FIELD VANE	

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{v0}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m ³	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m ³	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m ³ /s	RATE OF DISCHARGE
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	WTP		WETTER THAN PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No. 1

1 of 1

METRIC

G.W.P. 5451-09-01 **LOCATION** Coords: 4 965 759.3 N; 318 963.2 E **ORIGINATED BY** F.P.
DIST Huntsville **HWY** 11 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** A.D.
DATUM Geodetic **DATE** June 19, 2014 **CHECKED BY** B.R.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
231.6	Ground Surface						20	40	60	80	100						
0.0	180mm asphalt over sand and gravel	⊗					231										
	Dense Brown Moist	⊗	1	SS	41										○		
	(FILL)	⊗	2	SS	31										○		
		⊗															
229.9		⊗	3	SS	25/5cm		230										
1.7	End of borehole Refusal on probable bedrock Sample 3: Sampler bouncing 																

RECORD OF BOREHOLE No. 2

1 of 1

METRIC

G.W.P.	5451-09-01	LOCATION	Coords: 4 965 770.5 N; 318 964.9 E	ORIGINATED BY	F.P.
DIST	Huntsville	HWY	11	BOREHOLE TYPE	Continuous Flight Hollow Stem Augers
DATUM	Geodetic	DATE	June 19, 2014	COMPILED BY	A.D.
				CHECKED BY	B.R.G.

[illegible]

RECORD OF BOREHOLE No. 3

1 of 1

METRIC

G.W.P.	5451-09-01	LOCATION	Coords: 4 965 789.8 N; 318 968.1 E	ORIGINATED BY	F.P.
DIST	Huntsville	HWY	11	BOREHOLE TYPE	Continuous Flight Hollow Stem Augers
DATUM	Geodetic	DATE	June 19, 2014	COMPILED BY	A.D.
				CHECKED BY	B.R.G.

[illegible]

RECORD OF PENETRATION TEST No DPCT-1

1 of 1

METRIC

G.W.P. <u>5451-09-01</u>	LOCATION <u>Coords: 4 965 764.9 N; 318 964.1 E</u>	ORIGINATED BY <u>F.P.</u>
DIST <u>Huntsville</u> HWY <u>11</u>	BOREHOLE TYPE <u>Dynamic Cone Penetration Test</u>	COMPILED BY <u>A.D.</u>
DATUM <u>Geodetic</u>	DATE <u>June 19, 2014</u>	CHECKED BY <u>B.R.G.</u>

[illegible]

RECORD OF BOREHOLE No. AP-1

1 of 1

METRIC

W.P.	5451-09-01	LOCATION	Coords: 4 965 764.2 N; 318 956.1 E	ORIGINATED BY	F.P.
DIST	Huntsville	HWY	11	BOREHOLE TYPE	Manual Probing
DATUM	Geodetic	DATE	June 19, 2014	CHECKED BY	B.R.G.
COMPILED BY A.D.					

[illegible]

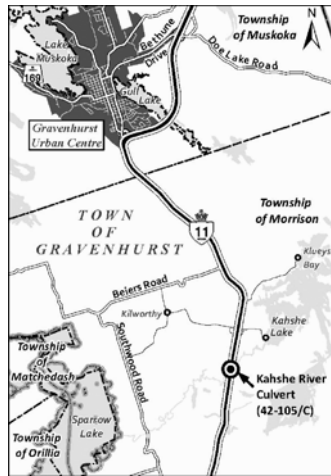
RECORD OF BOREHOLE No. AP-2

1 of 1

METRIC

W.P.	5451-09-01	LOCATION	Coords: 4 965 786.0 N; 318 960.0 E	ORIGINATED BY	F.P.
DIST	Huntsville	HWY	11	BOREHOLE TYPE	Manual Probing
DATUM	Geodetic	DATE	June 19, 2014	CHECKED BY	B.R.G.
COMPILED BY A.D.					

[illegible]



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Borehole and cone
- Cone penetration test
- Auger Probe
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WR denotes penetration due to weight of rods only
- W L at time of investigation June 2014
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER
- 2 Photograph location and direction

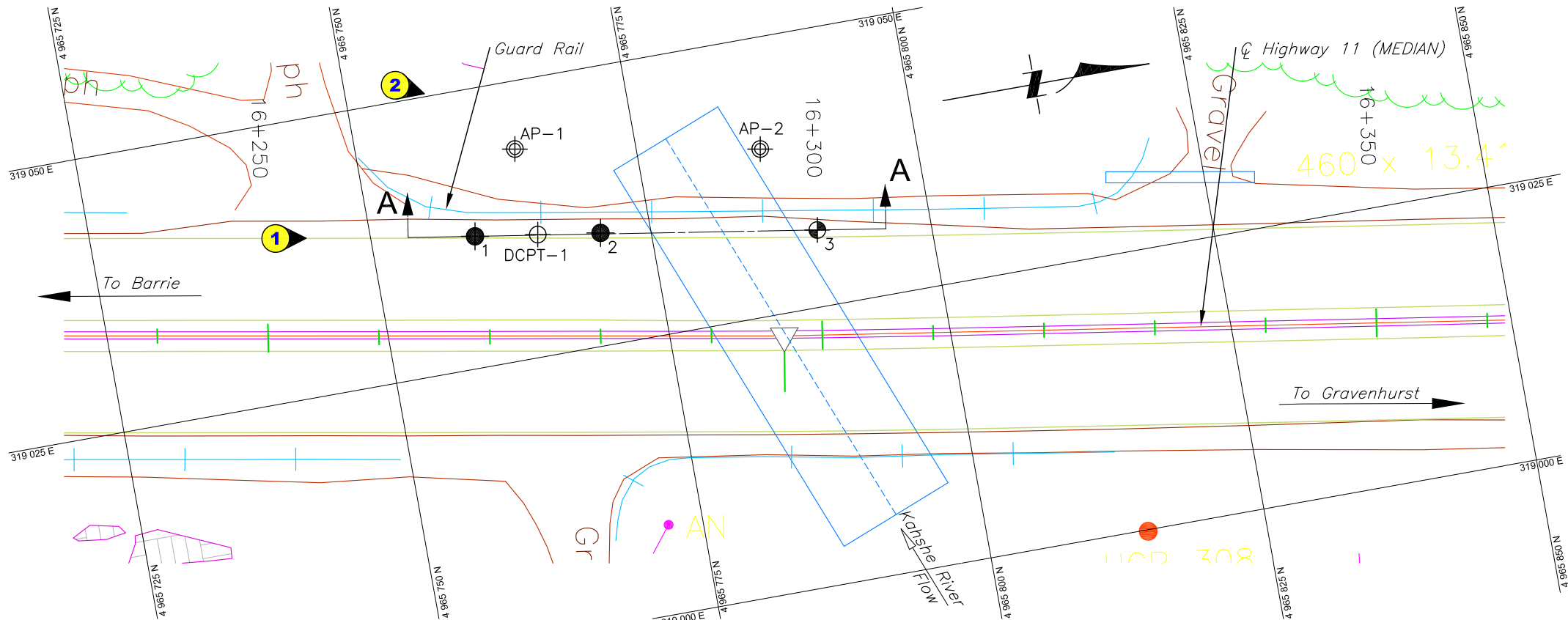
BH No	ELEVATION	COORDINATES	
		NORTHINGS	EASTINGS
1	231.6	4 965 759.3	318 963.2
2	231.7	4 965 770.5	318 964.9
3	232.1	4 965 789.8	318 968.1
DCPT-1	231.6	4 965 764.9	318 964.1
AP-1	228.0	4 965 764.2	318 956.1
AP-2	228.3	4 965 786.0	318 960.0

NOTE

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

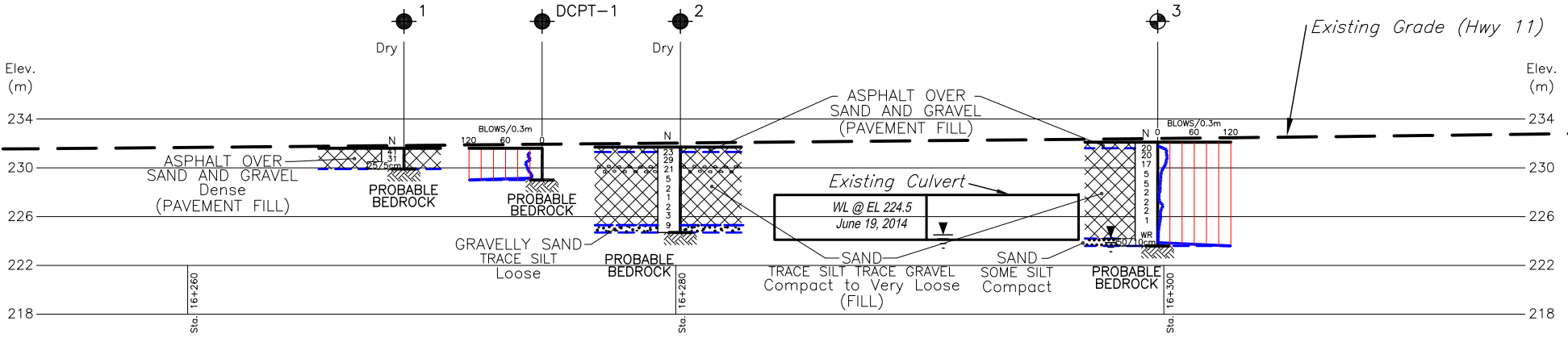
REVISIONS	DATE	BY	DESCRIPTION

HWY No	11	DIST	Huntsville
SUBMD	NA	CHECKED	AD
DRAWN	NA	CHECKED	DD
DATE	FEB, 02, 2015	APPROVED	CN
SITE	42-105C	DWG	1



PLAN

SCALE
0 5 10m

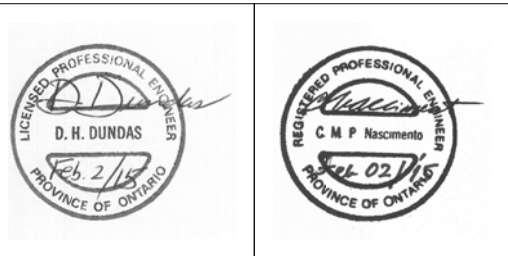


CROSS SECTION A-A

SCALE
HORIZONTAL
0 2.5 5m
VERTICAL
0 5 10m

NOTES:

- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH RECORD OF BOREHOLES AND REPORT
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
- DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.



Reference MMM Drawing: kashe creek north culvert.dwg, undated



APPENDIX FIR-A

Site Photographs



Photograph 1: Taken from the Highway 11 southbound lane outside shoulder, south of the Kahshe River looking north. Note the longitudinal crack approximately 0.3 m east of the Highway 11 southbound lane shoulder. (Photograph taken on April 14, 2014)



Photograph 2: Taken from the south river bank of Kahshe River looking north east at the Highway 11 embankment and Kahshe River Culvert. (Photograph taken on April 14, 2014)



Photograph 3: Upstream Wall, looking south. Note the obvert of the culvert to be higher than the outlet in Photograph 2. (Photograph taken on April 14, 2014)



**FOUNDATION DESIGN REPORT
REHABILITATION OF FOUR (4) STRUCTURES IN HUNTSVILLE AREA
KAHSHE RIVER NORTH CULVERT EMBANKMENT STABILITY
HIGHWAY 11
GWP 5451-09-01; SITE NO. 42-105C
GRAVENHURST, ONTARIO**

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Appendix FDR-A – Previous Technical Memorandum

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for Conceptual Flattened Slope Option

FOUNDATION DESIGN REPORT

for

Kahshe River Embankment Stability

Highway 11

GWP 5451-09-01; Site No. 42-105C

Gravenhurst, Ontario

1. INTRODUCTION

Peto MacCallum Ltd. (PML) was retained by AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO) to provide Foundations Engineering Services to analyse the slope stability and to provide recommendations for remedial slope stabilization measures for the embankment at the west side of Highway 11 at the Kahshe River North Culvert.

This report provides an assessment of the slope stability of the downstream embankment, and foundation design recommendations for a possible retaining wall at the base of the slope and extending across the top of the culvert as requested by MTO further to a preliminary evaluation of the slope dated August 26, 2014. A copy of the technical memorandum from this earlier phase of the project is attached in Appendix FDR-A. In addition, this report presents conceptual options, for consideration, to achieve the stabilization of the embankment.

All elevations in this report are expressed in metres.

2. BACKGROUND

Refer to the Foundation Investigation Report for subsurface information including a drawing illustrating the location of the site, boreholes and a stratigraphical profile based on results from boreholes as well as for elevation view photographs of the existing upstream and downstream embankments.

In general, the subsurface stratigraphy included the existing Highway 11 pavement structure over fill made up of layers of compact (locally dense) sand and gravel becoming sand trace gravel trace silt in about the upper 2.3 m. This upper fill covers loose to very loose sand trace silt in the remainder of the fill layer. The fill locally extended to probable bedrock in borehole 1 at 1.7 m and to a native sand layer in boreholes 2 and 3 at 6.4 and 7.9 m. The native sand layer was 0.6 m thick and overlaid probable bedrock at depths of 7.0 and 8.5 m in boreholes 2 and 3, respectively. The bedrock surface was contacted only at the borehole locations and is expected to be variable across the site. The overburden at the site may vary and depending on proximity to the slope may contain rock fill, cobbles and/or boulders.



The Kahshe River culvert consists of a twin cell concrete culvert with each cell having the approximate dimensions of 6.25 m wide and 3.65 m high. The height of the downstream (west side) embankment is approximately 7 m.

The surface of the Highway 11 roadway adjacent to the location of the Kahshe River twin culvert exhibits a longitudinal crack that is indicative of marginal surficial slope stability. Also, the surface of the west slope appears to have experienced localized erosion that has contributed to the steep downstream slope geometry.

3. EMBANKMENT STABILITY

Based on a visual assessment of the downstream slope, the distress to the pavement surface at the downstream side of the road is a result of marginal surficial slope stability and embankment spreading caused by lack of confinement of the embankment fill and an over-steepened downstream embankment slope.

The existing downstream (west side) embankment of Highway 11 at Kahshe River North was analysed using the limit equilibrium methods and the SLOPE/W software developed by Geo-Slope International Ltd. A factor of safety of 1.3 was targeted to provide a stable embankment slope at this site.

The embankment stability was analysed at sections immediately south and north of the existing culvert near Stations 16+280 and 16+300, respectively. The existing slope geometries, based on a topographical survey of the site provided by AECOM, are shown in the Figures 1 to 3 in Appendix FDR-C. The embankment south of the existing culvert near station 16+280 is approximately 7 m high and the west side slope has an inclination of about 1 Horizontal to 1 Vertical (1H:1V) near the bottom of the slope and 1.5H:1V on the middle and upper slope sections. The embankment north of the existing culvert near Station 16+300 is approximately 8.0 to 9.0 m high and the west side slope has a relatively uniform inclination of approximately 1.2H:1V.

The analysis model utilized the soil stratigraphy encountered in boreholes 2 and 3 for the embankment composition south and north of the existing culvert, respectively. A summary of the engineering properties assumed for the calculations is as follows.



		ASSUMED GEOTECHNICAL PARAMETERS	
		EFFECTIVE STRESS	
SOIL TYPE	UNIT WEIGHT (kN/m ³)	SHEAR STRENGTH (kPa)	INTERNAL FRICTION (Degrees)
Sand Fill (Compact)	22	0	33
Sand Fill (Loose to Very Loose)	20	0	28.5
Sand	21	0	32

The results of the analyses, indicate that the existing embankment immediately south and north of the existing culvert have factors of safety of 1.12 and 1.16 respectively, as shown on the Figures 1 and 2 in Appendix FDR-C. These factors of safety do not meet the target factor of safety value of 1.3. Measures to increase the factor of safety for slope stability to the target value will be required at this site.

4. CONCEPTUAL OVERVIEW OF OPTIONS FOR EMBANKMENT STABILIZATION

Measures to increase the factor of safety at this site could include the alternatives listed in the following table. Per the Terms of Reference for this assignment, the retaining wall option (Option 5) has been identified as the option for which detail design foundation recommendations are required. The remaining options in the table are provided for discussion purposes only. Further analysis would be required for detail design of these remaining options and is beyond the scope of the assignment.

Advantages, disadvantages, risk/consequences and relative costs of each alternative are provided in the table for discussion purposes only.

Option		Advantages	Disadvantages	Risks/Consequences	Relative Costs
#	Description				
1	Patch pavement and maintain	<ul style="list-style-type: none"> - fast - inexpensive - no hwy temporary roadway protection or retaining walls required 	<ul style="list-style-type: none"> - shorter term solution - requirement for continued maintenance 	<ul style="list-style-type: none"> - uncertain duration of effectiveness - minor consequence level requiring future remedial measures 	- very low



Option		Advantages	Disadvantages	Risks/Consequences	Relative Costs
#	Description				
2	Slope flattening of downstream embankment by extending downstream culvert	- no hwy temporary roadway protection or retaining walls required	- significant and costly construction undertaking requiring diversion of river, cofferdams, extension of the existing twin cell culvert and construction of wing walls - flattened slope may extend into river and beyond existing right of way (ROW)	- potential costly construction problems related to river diversion and dewatering	- very high
3	Slope steepening by replacement of surface layer (shell) of existing downstream side of embankment with minimum 2.0 m thickness of rock fill	- no additional property requirements as embankment will not be widened outside MTO ROW - no culvert extension would be required	- would require staged construction with centreline shoring and lane closures	- moderate risk of ineffective performance requiring future maintenance	- moderate
4	Slope steepening with geogrid reinforcement of upper half of downstream side hwy embankment	- no additional property requirements as embankment will not be widened outside MTO ROW - no culvert extension would be required	- would require staged construction with centreline shoring and lane closures	- moderate risk of ineffective performance requiring future maintenance	- moderate
5	Retaining wall at base of embankment to permit flatter 2H:1V earth slopes	- positive solution - provides additional protection from erosion from high water levels in river - no additional property requirements as embankment would not be widened outside MTO ROW - no culvert extension would be required	- would require staged construction with shoring and lane closures	- depending on typed of retaining wall, moderate risk of dewatering problems and with founding on uncertain bedrock surface	- moderate to high depending on type of retaining wall
6	Innovative RSS with vegetated surface	- appearance - implementation of innovative technology	- would require extensive engineering	- potentially less resistant to erosion than conventional solutions	- relatively high



5. RETAINING WALL AT BASE OF EMBANKMENT TO FLATTEN SLOPE (OPTION 5)

5.1 Retaining Wall Type Assessment

The following recommendations pertain to the retaining wall options.

The slope can be stabilized by constructing toe retaining walls. The retaining walls can be either cast-in-place walls on bedrock or gabion walls at the invert elevation of the existing culvert and higher where bedrock is encountered.

Installing a toe retaining wall would permit flattening the embankment slope above the wall to 2H:1V. As shown in the Figure 3, Appendix FDR-C, the safety factor of slope stability would increase to 1.3. The height of the retaining wall should be 0.3 m higher than the height that would permit an embankment slope above the retaining wall of 2H:1V or flatter. The additional retaining wall height would retain potential additional surficial slope treatments to mitigate surficial erosion. The retaining wall should extend across the downstream river valley and across the top of the culvert to provide a slope geometry in which retained earth slopes are 2H:1V or flatter.

Based on the results of the slope stability analysis, the geometry of the existing slope, proximity to the river and the difficulties associated with extending the existing culvert, it is considered that widening the existing embankment at the base to decrease the inclination of the existing side slope would likely be impractical and it is therefore considered that a retaining wall along the length of the embankment over Kahshe River should be considered.

Based on the site conditions a gabion wall retained soil system (RSS) or a cast-in-place concrete retaining wall (cantilever wall or gravity) are recommended. The gabion wall would be considerably less expensive and easier to construct, but would carry slightly more risk of slope erosion if the river should flood. Mechanically stabilized earth type RSS were considered but are not recommended because they would require more extensive excavations and roadway protection systems.

The following table summarise the advantages, disadvantages of different types of retaining system from the foundation perspective.



Retaining System Option		Advantages	Disadvantages	Risks/Consequences	Relative Costs
#	Description				
1	RSS Gabion Wall founded on bedrock or on rock fill	<ul style="list-style-type: none"> - less excavation required than for CIP wall - can be founded at specified elevation with minimal if any bedrock trimming and dewatering 	<ul style="list-style-type: none"> - less durable and less erosion resistance than CIP wall 	<ul style="list-style-type: none"> - potential variable bedrock surface between borehole locations - excavations for installation require shoring and lane closure 	- low
2	Cast-in-place concrete retaining wall founded on bedrock or on rock fill	<ul style="list-style-type: none"> - better erosion resistance to river flooding than RSS Gabion Wall 	<ul style="list-style-type: none"> - preferably requires excavation to bedrock, which would potentially involve some bedrock trimming and would involve dewatering - may require footing dowels to supplement sliding resistance - less tolerant to settlement thus requiring higher performing foundation 	<ul style="list-style-type: none"> - potential variable bedrock surface between borehole locations - excavations for installation require shoring and lane closure - undermining of culvert if bedrock surface is below culvert invert - difficulties in establishing bearing surface on bedrock and with dewatering 	- high

5.2 Retaining Wall Design Recommendations

5.2.1 Bearing Resistance

The cast-in-place retaining wall option can be founded on bedrock. The geotechnical resistances at factored Ultimate Limit State and Serviceability Limit States at the recommended founding level are provided below:

$$\begin{aligned} \text{Factored Geotechnical axial Resistance at ULS} &> 2000 \text{ kPa} \\ \text{Geotechnical axial Resistance at SLS} &> 2000 \text{ kPa} \end{aligned}$$

The gabion wall option can be founded at, but not below, the elevation of the culvert invert or on the bedrock surface, whichever is at a higher elevation. Mass concrete or uniformly graded rock fill with maximum dimension less than 150 mm can be used to level the bearing surface. The gabion wall should be canted into the slope a minimum of 15%. The design could be by the RSS



supplier or by the structural designer. The geotechnical resistances at factored Ultimate Limit State and Serviceability Limit States at the recommended founding level are provided below:

Factored Geotechnical axial Resistance at ULS	= 300 kPa
Geotechnical axial Resistance at SLS	= 200 kPa

The geotechnical bearing resistance for inclined loads should be considered in the design in accordance with the requirements of clause 6.7.4 of the CHBDC.

If a section of the retaining wall is required to span the culvert to provide the required slope geometry, the capacity of the culvert to resist the imposed loads should be verified and if necessary the load on the culvert could be reduced by using a grade beam to span the culvert.

5.2.2 Sliding Resistance

An unfactored friction factor of 0.6 is considered to be suitable at this site on the bedrock surface. The lateral resistance of footing founded on bedrock could be increased, if required, by installing shear keys, or also in the case of cast-in-place walls, by installing footing dowels. If dowels are used, the NSSP – DOWELS INTO ROCK, in Appendix FDR-B should be included in the tender documents to provide specific direction for contractor during installation and testing of the dowels.

Preparation of the subgrade for construction of the foundation should be performed and monitored in accordance with OPSS 902.

5.2.3 Erosion Protection

The flattened slope should be protected against flooding and runoff erosion. This can be achieved with a stone cover or with equivalent protection and should be established on all exposed slope surfaces.

Granular backfill to the existing culvert should be separated by a geosynthetic filter fabric to prevent loss of the granular materials into the voids of the rockfill. The rockfill surface should be chinked in accordance with the requirements of SP 206S03, prior to placing the geotextile. The filter fabric should conform to OPSS 1860 and comprise a Class II non-woven geotextile with a filtration opening size (FOS) of 105 to 210 μm .



5.2.4 Frost Depth

The foundation frost depth for structure foundations at this site is 1.8 m, according to OPSD-3090.101. Frost protection is not required for footings on bedrock and is not a requirement for gabion walls.

5.2.5 Earth Pressure

Backfill to cast-in-place retaining walls should be in accordance with OPSS 902 and should consist of Granular A or Granular B Type I or II material. All granular material should meet the specifications of OPS.PROV 1010. Compaction equipment to be used adjacent to retaining structure should be restricted in accordance with OPSS 501.

The retaining walls or retaining soil system should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure, p (kPa) may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation.

$$p = K (\gamma(h-h_w) + \gamma' h_w + q) + \gamma' h_w + C_p + C_s$$

where K = coefficient of lateral earth pressure (dimensionless)
 γ = unit weight of free-draining granular material, kN/m³
 γ' = the submerged unit weight of the saturated soil, (γ -9.8 kN/m³)
 h = depth below final grade, m
 h_w = the depth below the ground water level, m
 q = surcharge load, kPa, if present
 C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
 C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where ϕ = angle of internal friction of retained soil
 δ = angle of friction between the soil and wall



Free-draining granular material should be used as backfill behind the wall but the effect of water level should be considered. The following parameters are recommended for design:

PARAMETERS	GRANULAR A OR GRANULAR B TYPE II		GRANULAR B, TYPE I	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal I Backfill	Sloping Backfill (2H:1V)
Coefficient of Active Earth Pressure, K_a	0.27	0.39	0.31	0.47
Coefficient of Earth Pressure At Rest, K_o	0.43	-	0.47	-
Coefficient of Passive Earth Pressure, K_p	3.69	-	3.25	-
Internal Friction Angle, ϕ (degrees)	35°		32°	
Unit weight, γ (kN/m ³)	22.8		21.2	

The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures. The earth pressure coefficients should be reviewed if the slope of the backfill exceeds 10° to the horizontal. Alternatively, the material above the top of the wall could be treated as a surcharge load (q in the preceding equation).

It is recommended that the passive resistance from cover in front of walls should not be considered for this site in view of the hydraulic conditions.

For cast-in-place retaining walls, consideration should be given to control rapid drawdown in retaining wall design by providing closely spaced through-wall drain holes.



5.2.6 Seismic Design

The following seismic parameters should be used for design accordance with the CHBDC for a design earthquake with 475-year return period:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- PEAK Ground Acceleration 0.04 g

The soil profile type at this site has been classified as Type I. Therefore, according to Clause 4.4.6.1 of CHBDC, a site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

5.2.7 Dewatering

Dewatering would be required for construction of the cast-in-place retaining wall option as the bedrock surface is probably below the river level near the culvert. The dewatering would require the installation of sealed cofferdams adjacent to the existing culvert prior to retaining wall construction. An NSSP alerting the contractor of this requirement should be included in the contract documents. Refer to Appendix FDR-B for a draft NSSP for dewatering.

Dewatering may not be required for construction of the gabion wall option. If excavations extend below the river level, they could be backfilled with rock fill to a founding level above the river.

5.2.8 Erosion Protection

The surface of the slope should be protected from erosion as soon as possible after construction. These erosion stabilization measures could consist of installing a minimum 1m thickness of uniformly graded rock with dimensions in the order of 150 mm or a vegetation cover on all exposed earth surfaces in general accordance with OPSS 804.



6. CONSTRUCTION CONSIDERATIONS

Refer to Appendix FDR-B for a list of OPS specifications and an SP for Dowels in Rock that should be included in the contract documents if the cast-in-place retaining wall option is adopted. Construction at this site should preferably be performed when the river is at the lowest water level to minimized dewatering issues. It would be advantageous to have construction occur above the river level. If excavations extend below the river level cofferdams would be required for construction in the dry

6.1 New Fill Construction

The new fill should be placed against the existing fill with appropriate benching in accordance with OPSD 208.010 (Benching of Earth Slope).

6.2 Temporary Roadway Protection

Temporary roadway protection should be used where the existing slope is excavated for the proposed works. Temporary protection should be constructed in accordance with OPSS 539. A minimum performance level of 2, according to OPSS 539 is recommended. The contractor is responsible for selection, preparation of a detailed design and performance for the roadway protection system. The presence of cobbles and boulders within the fill material and the possibility of the granular fill migrating through the temporary protection should be considered during detailed design.

Provided adequate groundwater control is achieved, fill and site soils are classified as Type 3 soils according to Occupational Health and Safety Act, 1990 and Regulation 213/1991 for Construction Projects. Excavations in Type 3 soil should be cut back at an inclination of 1H:1V from the base of the excavation.



7. CLOSURE

This report was prepared by Mr. A. DeSira, MEng, P.Eng., and Mr. M. Khorsand, EIT, and reviewed by Mr. D. Dundas, P.Eng. MTO Designated Principal Contact Mr. C.M.P. Nascimento, P.Eng., Project Manager, conducted an independent review of the report.

Yours very truly,

Peto MacCallum Ltd.



Mansoor Khorsand, EIT
Project Supervisor



David Dundas, P.Eng.
Senior Engineer, Geotechnical Services



Carlos M.P. Nascimento, P.Eng.
Project Manager and MTO Designated Principal Contact



APPENDIX FDR-A

Previous Technical Memorandum

August 22, 2014

PML Ref.: 14TF009
Index No.: 012PTM

Mr. Jon Newman, P.Eng.
AECOM
300 Water Street
Whitby, Ontario
L1N 9J2

Dear Mr. Newman

Technical Memorandum
Rehabilitation of four Structures in Huntsville Area
Kahshe River Culvert North Embankment Stability
Site No.42-105
Highway 11
G.W.P. 5451-09-01
Gravenhurst, Ontario

The purpose of this technical memorandum is to briefly outline the findings of the foundation investigation recently completed at the Kahshe River culvert site on Highway 11 and to present the preliminary results of the stability assessment carried out for the west side of the Highway 11 embankment north of Kahshe River.

Summarized Subsurface Conditions

We refer to the draft Record of Boreholes and draft borehole location plan for details of the boreholes carried out at this site.

In general the subsurface stratigraphy included the existing Highway 11 pavement structure over fill made up of layers of compact (locally dense) sand and gravel becoming sand trace gravel trace silt in about the upper 2.3 m. This upper fill covers loose to very loose sand trace silt in the remainder of the fill layer. The fill locally extended to probable bedrock in borehole 1 at 1.7 m and to a native sand layer in boreholes 2 and 3 at 6.4 and 7.9 m. The native sand layer was 0.6 m thick and overlaid probable bedrock at 7.0 and 8.5 m in boreholes 2 and 3, respectively.

Preliminary Embankment Stability Analysis

In accordance with the RFP requirements the stability of the existing embankment on the west side of Highway 11 at Kahshe River North was analysed using the limit equilibrium methods and the SLOPE/W software developed by Geo-Slope International Ltd. The software analyzes numerous potential failure surfaces and establishes a minimum safety factor aided by the user's input.

A minimum factor of safety of 1.3 is recommended for the embankment slope to be considered stable under the long-term effective stress analysis condition. Since only cohesionless, free draining soils were encountered at the site it is considered that the undrained, total stress (short-term) analysis was not required, as effective stress parameters of the soil will govern the stability at this site.



The analysis considered the embankment stability immediately north of the existing culvert near Station 16+300 which was considered to be the most critical section based on its geometry, according to the cross sections provided by AECOM on August 8, 2014. The analysis model utilized the soil stratigraphy encountered in borehole 3, which was drilled immediately north of the existing culvert. The existing embankment is approximately 8.0 m high at this location and the west side slope has an inclination of approximately 1.2 Horizontal to 1 Vertical (1.2H:1V).

A summary of the engineering properties assumed for the calculations is as follows:

		ASSUME STRENGTH PARAMETERS	
		EFFECTIVE STRESS	
SOIL TYPE	UNIT WEIGHT (kN/m ³)	SHEAR STRENGTH (kPa)	INTERNAL FRICTION (Degrees)
Sand Fill (Compact)	22	0	33
Sand Fill (Loose to Very Loose)	20	0	28.5
Sand	21	0	32

The result of the effective stress analysis, which governs in this scenario, indicates that the existing embankment has a factor of safety of 1.156, as shown on the appended Figure 1. This factor of safety does not meet the minimum recommended factor of safety value of 1.3. Consequently it is recommended that measures to increase the slope stability be implemented at this site to meet the minimum recommended factor of safety.

Measures to increase the factor of safety at this site may include the following alternatives. Advantages and disadvantages of each alternative are listed on the table below for discussion purposes:

ALTERNATIVE EMBANKMENT CONSTRUCTION METHOD	ADVANTAGES	DISADVANTAGES
Flatten the slope by widening the embankment westerly	No retaining structure required	Requires extension of the existing non-standard culvert Requires construction of wing walls Flattened slope may extend beyond existing right of way resulting in property acquisitions
Flatten the slope and support the slope with an earth retaining wall	No property restrictions as embankment will not be widened No wing walls or culvert extension will be required	Requires construction of a retaining structure Requires installation of temporary roadway protection



PRELIMINARY Foundation Technical Memorandum
Kahshe River Culvert Embankment Stability
GWP 5451-09-01, Site No.: 42-105C, Index No.: 012PTM
PML Ref.: 14TF009, August 22, 2014, Page 3

Based on the geometry of the existing slope, proximity to the river and the difficulties associated with extending the existing non-standard culvert, it is considered that widening the existing embankment to decrease the inclination of the existing side slope would likely be impractical. It is therefore considered that a retaining wall along the length of the embankment over Kahshe River should be considered.

We trust that the foregoing is sufficient for your present purposes.

Should you have any questions, please do not hesitate to contact our office.

Sincerely

Peto MacCallum Ltd.

Andrew DeSira, MEng, P.Eng.
Project Engineer, MTO Foundation Services

Carlos M.P. Nascimento, P.Eng.
Manager, MTO Foundation Services
MTO Designated Principal Contact

AD/CN:ad-mi

Enclosure(s):
Figure 1 – Preliminary Slope Stability Results

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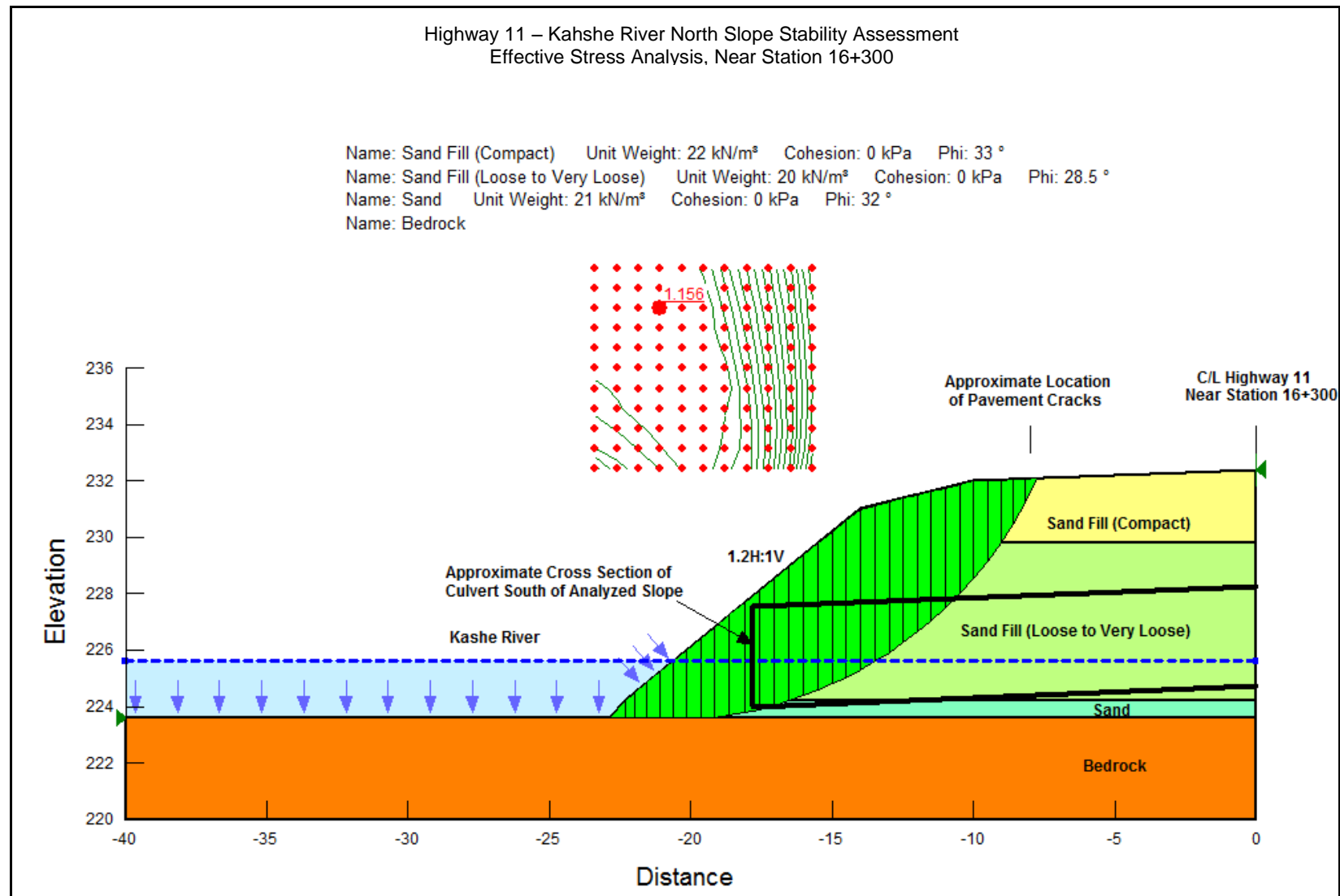


FIGURE 1 – PRELIMINARY SLOPE STABILITY ASSESSMENT



APPENDIX FDR-B

OPSS's, SP's and NSSP's



List of Ontario Provincial Standard Documents

List of Ontario Provincial Standard Documents
and Non-Standard Specific Provision (NSSP)
Referenced in Report

DOCUMENT	TITLE
OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification For Temporary Protection Systems
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
OPSS 1860	Material Specification for Geotextiles
SP 206S03	Construction Specification for Grading
OPSD 208.010	Benching of Earth Slopes
OPSD 3090.100	Foundation Frost Penetration Depths for Northern Ontario

DOWELS INTO ROCK - Item No.

Special Provision

**CONSTRUCTION SPECIFICATION FOR THE SUPPLY, INSTALLATION AND TESTING OF
DOWELS INTO ROCK FOR PIER FOOTINGS**

1.0 SCOPE

The work for the above noted tender item shall be in accordance with OPSS 904, including all Special Provisions, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the pier footing.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications, or publications:

ASTM International

D1143M Standard Test Methods for Deep Foundations Under Static Axial Compressive Load



3.0 DEFINITIONS

For the purpose of this Special Provision, the following definitions apply:

Dowels into Rock means reinforcing steel bar and non-shrink grout.

Design Engineer means an Engineer who has a minimum of five (5) years experience in all aspects associated with the underwater installation of Dowels into Rock, including drilling, underwater grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.

Quality Verification Engineer means an Engineer who has a minimum of five (5) years experience in all aspects associated with the underwater installation of Dowels into Rock, including drilling, underwater grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Working Drawings

Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.

The Contractor shall submit Working Drawings to the Contract Administrator as follows:

- a) All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.
- b) All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.

Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.

Working Drawings consisting of testing and installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.

Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:

- a) Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.



- b) Test results verifying the 28 day strength of non-shrink grout.
- c) The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
- d) The procedures to verify hole length. Records of measurements that verify the hole length.
- e) Records of all drilling procedures, rock conditions encountered, and installation times.
- f) Test procedures for Dowels into Rock.
- g) Drawings and design calculations for a suitable reaction system for the applied test loads.
- h) Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.
- i) Drawings and details for reference system arrangement.
- j) Current calibration curves shall be provided for all gauges.
- k) Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
- l) Remedial measures for unacceptable stressing results.

5.0 MATERIALS

5.01 Non-Shrink Grout

The non-shrink grout shall be an approved product from the MTO's Pre-Qualified Products List.

5.02 Anti-Washout Agent

The anti-washout agent shall be used with the non-shrink grout for the Dowels into Rock. The anti-washout agent shall be one of the following proprietary products:

- 1) Sikament 100 SC Anti-Washout Admixture
Sika Canada Inc.
6915 Davand Drive
Mississauga, ON, L5T 1L5
Toll Free Phone: 800-933-7452



- 2) Rheomac UW 450 Anti-Washout Admixture
BASF Construction Chemicals Canada Ltd (Master Builders)
1800 Clark Blvd
Brampton, ON, L6T 4M7
Toll Free Phone: 416-520-1392

5.03 Manufacturer Information

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

- a) Data sheets for the non-shrink grout and anti-washout agent,
- b) Technical information that proves that the non-shrink grout and anti-washout agent are compatible, and
- c) Installation procedures.

6.0 EQUIPMENT

All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

The equipment shall not cause damage to the reinforcing steel bars.

7.0 CONSTRUCTION

7.01 Instructions to Contractor

These instructions are to be read in conjunction with the Contract Drawings.

A total of 2 test Dowels into Rock are required for the Dowels into Rock at the pier.

Dowels into rock at the pier shall be installed prior to unwatering the structure excavation. Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

7.02 Responsibilities of the Contractor

The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.

The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.



The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.

The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 4.0.

7.03 Subsurface Conditions

Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

7.04 Construction of Holes

The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.

The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.

At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

7.05 Installation of Reinforcing Steel Bar

Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.

Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.

Dowels into Rock at the pier shall be installed prior to unwatering the structure excavation. Dowels shall extend through the tremie concrete for the pier footing and into sound bedrock.

Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

7.06 Grout and Anti-Washout Agent

The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.



The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

Anti-washout agent shall be used in accordance with the specifications of the manufacturer.

Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

8.0 QUALITY ASSURANCE

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.01 Qualifications

8.01.01 Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock

All work shall be performed under the direction of personnel experienced with all aspects associated with the underwater installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.

8.01.02 Qualifications of the Quality Verification Engineer

A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.

8.01.03 Qualifications of the Design Engineer

A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

8.02 Testing Requirements

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

8.02.01 General Testing Requirements

Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.

The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.



The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the pier. The Dowels into Rock for testing shall be 55M dowels grouted into 140 mm diameter holes filled with an approved non-shrink grout with a minimum 4,000 mm embedment into sound bedrock.

The Contractor shall submit Working Drawings that include proposed procedures for testing of the Dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.

The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.

The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.

The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

8.02.02 Testing Location

The Contractor shall remove all loose rock down to sound bedrock at the test location.

The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator. The water depth at the location of the test shall be at least 0.5 m deep.

If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days' notice in writing of this operation.

8.02.03 Testing Equipment

The dowels into rock will be carried out generally in accordance with the prevailing requirements of ASTM International D1143M superseded where applicable by the procedures specified in this document.

The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.

The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:



The beams shall be independently supported with the support firmly embedded in the ground.

The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.

Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.

The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

8.02.04 Testing for Dowels Into Rock, and Report

At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Jacks used for reinforcing steel bars shall have a minimum ram dimension of 152.6 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

8.02.05 Testing Loading

The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the test load of 1,150 kN. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.



8.03 Acceptance Criteria

The following acceptance criteria apply:

- a) The testing of dowels shall be carried out in advance of the instalment of Dowels into Rock at the pier footing.
- b) Tests for Dowels into Rock shall have a capacity of at least 1035 kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

9.0 MEASUREMENT FOR PAYMENT

For measurement purposes, a count shall be made of the number of dowels installed.

10.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.



NSSP for Dewatering – addition to OPSS 902

Although the selection and performance of dewatering schemes remains the responsibility of the Contractor, the Contractor is advised that dewatering below the river level will require the installation of sealed cofferdams on an uneven bedrock surface.

NSSP for Temporary Roadway Protection – addition to OPSS 539

The Contractor is advised that cobbles, boulder and/or rock fill may be encountered during installation of temporary roadway protection.

NSSP for RSS Wall

RETAINED SOIL SYSTEM, TRUE ABUTMENT - Item No.

RETAINED SOIL SYSTEM, FALSE ABUTMENT - Item No.

RETAINED SOIL SYSTEM, WALL/SLOPE, HIGH PERFORMANCE - Item No.

BACKFILL FOR RETAINED SOIL SYSTEM, HIGH PERFORMANCE - Item No.

RETAINED SOIL SYSTEM, WALL/SLOPE, MEDIUM PERFORMANCE - Item No.

BACKFILL FOR RETAINED SOIL SYSTEM, MEDIUM PERFORMANCE - Item No.

RETAINED SOIL SYSTEM, WALL/SLOPE, LOW PERFORMANCE - Item No.

BACKFILL FOR RETAINED SOIL SYSTEM, LOW PERFORMANCE - Item No.

Non Standard Special Provision

January, 2008

1.0 SCOPE

This special provision covers the requirements for the design and construction of Retained Soil Systems (RSS) walls and steep slopes.

Additional requirements for RSS precast concrete facing elements shall be as specified in the Contract documents.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, General:

OPSS 102	Weighing of Materials
OPSS 180	Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Construction

OPSS 501	Compacting
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Canadian Standards Association Standards:

CAN/CSA-S6-00 Canadian Highway Bridge Design Code (CHBDC)

Ministry of Transportation Publications:

MTO Designated Sources of Materials (DSM)
Qualification Criteria for RSS

3.0 DEFINITIONS

For the purposes of this special provision the following definitions apply:

Alignment Elements: means components specified by the manufacturer that are constructed on the foundation for RSS to facilitate placing of the facing elements to the correct lines and grades, such as concrete levelling pads and soldier piles.

Approved Product Drawings: means the documentation for an RSS that has been submitted by the manufacturer and accepted by the Ministry for listing in the DSM, according to the Qualification Criteria for RSS.

Backfill for RSS: means the material specified by the manufacturer as part of the engineered materials comprising the backfill for the RSS.

Constructed Height: means the vertical distance between the foundation for RSS and the top of the currently placed and compacted backfill for RSS, measured at the point of the design height.

Corrective Work: means work carried out by the Contractor to repair deficiencies identified by the Owner during the RSS warranty period.

Design Checking Engineer: means the Engineer retained by the Contractor who checks the original design and working drawings.

Design Engineer: means the Engineer retained by the Contractor who produces the original design and working drawings.

Design Height: means the maximum difference in elevation between the foundation for RSS and the corresponding top of backfill for RSS, over the full length or perimeter of the RSS.

External Stability: means stability against deep-seated failure of the foundation for RSS, including adequate bearing capacity at specified settlements of the foundation.

Facing Elements: means components specified by the manufacturer that delineate the front face of the RSS and to which reinforcing elements may be attached, such as precast concrete panels, split-face concrete blocks, and geo-synthetic panels.



Foundation for RSS: means the base on which the RSS is constructed, such as excavation to a specified elevation and construction of a granular 'A' pad.

Internal Stability: means stability against failure of the engineered materials comprising the RSS, including adequate resistance against excessive elongation, breakage and pullout of the reinforcing elements.

Manufacturer: means the firm who supplies the design and proprietary components, and who specifies the backfill and other materials, for the RSS selected by the Contractor.

Manufacturer's Representative: means an individual with continuous full-time employment with the manufacturer for a period of at least three (3) years, and who is knowledgeable in the design and construction of the RSS selected by the Contractor.

Obstruction: means any part of the work and any existing condition within the Contract limits that affects the design, construction and performance of the RSS, such as structures, catch basins and manholes, drainage pipes and sewers, and utilities.

Performance Tolerance – Local: means the joint gap between any two constructed facing elements, measured at any point along the joint between the facing elements and perpendicular to the line of the joint.

Performance Tolerance – Global: means the vector distance between any point on the constructed RSS and the corresponding point on the theoretical RSS surface as defined in the Contract documents.

Placing Tolerances: means tolerances specified by the manufacturer on the placing of the RSS components and backfill for RSS to ensure compliance of the constructed RSS with the performance tolerances.

Reinforcing Elements: means components specified by the manufacturer that are placed within the backfill for RSS and connected to the facing elements to mechanically stabilize the backfill for RSS, such as metal tie strips, metal grids and geo-synthetic grids,

Retained Soil System (RSS): means a proprietary system listed in the DSM used to retain horizontal loads for applications such as true and false abutment structures, retaining walls and steep slopes; or, to retain vertical loads for applications such as embankments over soft ground.

RSS Superintendent: means the Contractor's authorized representative in responsible charge of the construction of the RSS.

Structure: means any bridge, culvert, tunnel, retaining wall, overhead sign, high mast light pole, wharf, dock, or any part thereof.



4.0 SUBMISSION AND DESIGN REQUIREMENTS

4.1 Submissions

4.1.1 Working Drawings

The Contractor shall submit working drawings for all RSS. A separate submission shall be made for each RSS in the Contract. All submissions shall bear the seal and signature of the Design Engineer and the Design Checking Engineer.

The RSS Superintendent shall have a copy of the working drawings on site at all times during the construction of the RSS.

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

4.1.2 Working Drawing Requirements

Working drawings shall include at least the following:

- Statement from the manufacturer confirming the experience and expertise of the Design Engineer and Design Checking Engineer to provide design services for the manufacturer's RSS;
- All design, fabrication and construction drawings and specifications for the RSS;
- Location and value of the design height of the RSS;
- Defined lines and grades, type, and quantity in m³ of the backfill for RSS;
- Details at obstructions, and connections to other structures, where shown in the Contract drawings;
- Statement of bearing resistance required by the RSS foundation according to the CHBDC;
- Statement of satisfactory internal and external stability;
- Placing tolerances for the RSS.

4.1.3 RSS Superintendent

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit in writing to the Contract Administrator the name(s) of the RSS Superintendent for each RSS in the Contract.

During construction of an RSS, the Contractor shall not change the RSS Superintendent for that RSS without written permission from the Contract Administrator. The Contractor shall submit in writing to the Contract Administrator the proposed change for RSS Superintendent at least one week prior to the actual change in RSS Superintendent.

4.1.4 Manufacturer's Representative

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit in writing to the Contract Administrator the name(s) of the manufacturer's representative for each RSS in the Contract.



For each occasion the Contractor arranges for the manufacturer's representative to be on site, the Contractor shall submit 48 hours advance notice in writing to the Contract Administrator giving the dates and locations the manufacturer's representative will be on site.

4.1.5 Certificates of Conformance

For each RSS in the Contract, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the QVE upon completion of the RSS.

4.1.6 Milestone Inspection

For each RSS in the Contract, the Contractor shall submit to the Contract Administrators Milestone Inspection Report following an Interim Inspection by the QVE at each of the following milestones, and prior to commencement of subsequent operations on that RSS:

- a) Layout and marking of all lines and grades needed to construct the RSS; and construction of the alignment elements, where applicable;
- b) Delivery and storage on site of facing elements and reinforcing elements, where applicable;
- c) Installation of the facing elements; placement and compaction of the backfill for RSS; and installation of the reinforcing elements, where applicable;

For RSS where the design height is greater than 5.0 m, the Contractor shall submit a series of Written Permissions to Proceed for milestone c) corresponding to the constructed height of the RSS at 5.0 m, 10.0 m, and 15.0 m, as applicable, up to and including the design height.

The Milestone Inspection submissions in no way supersede the inspection and testing intervals required for the construction of the RSS, as specified in the working drawings.

4.1.7 RSS Warranty

The Contractor shall submit a warranty to the Owner to address all deficiencies identified by the Owner related to the performance of the RSS for a period of 36 months from the date of certification of completion of the Contract.

4.1.8 Repair Procedures for Corrective Work

At least two weeks prior to commencement of any corrective work at an RSS during the warranty period, the Contractor shall submit to the Manager of Contracts, for information purposes only, three copies of his repair procedures for that RSS.

The repair procedures shall include a description of the cause and fully detail the corrective work required to correct the deficiencies identified by the Owner.

The repair procedures shall bear the seal and signature of an Engineer (who may be different than the Design Engineer and Design Checking Engineer), and be signed by the manufacturer's representative.

4.2 Design

4.2.1 General

The Contractor shall be responsible for the design of the RSS and for ensuring the RSS as designed is compatible with the work.

The geometric requirements of the RSS, such as lines and grades of the facing elements and typical cross-sections, shall be as specified in the Contract drawings.

The foundation for RSS shall be as specified in the Contract documents.

4.2.2 RSS Selection

The Contractor shall select an RSS from the DSM that meets the Application, Performance and Appearance requirements for that RSS, as specified in the Contract drawings.

The Contractor shall select an RSS from the DSM designated as either 'A' (Accepted) or 'DE' (Demonstration). RSS designated as 'DE' status require inspection, instrumentation and monitoring of the constructed RSS, and reporting of the findings to the Ministry by the manufacturer, according to the Qualification Criteria for RSS.

Where there is more than one RSS in the Contract, the Contractor shall select the RSS from the same DSM listing, including type and colour of facing elements, according to the following groupings:

- a) All RSS covered under the same tender item number(s) for payment;
- b) All RSS with the same Performance and Appearance requirements that about the same structure, existing and/or part of the work.

4.2.3 Performance Tolerances

Performance tolerances for the RSS shall be according to Table 1.

TABLE 1 – PERFORMANCE TOLERANCES FOR RSS		
Performance Requirement	Performance Tolerance (mm)	
	Local	Global
Abutments	Joint Gap ¹ ± 5	≤ 20
High	Joint Gap ¹ ± 10	≤ 30
Medium	N/A	≤ 50
Low	N/A	≤ 100

Note 1: Joint Gap shall be as specified in the working drawings.



4.2.4 Obstructions

The Contractor shall be responsible for developing design details of the RSS at obstructions, for all obstructions shown in the Contract drawings.

Where an obstruction is shown in the Contract drawings but not located to sufficient accuracy for the design of the RSS, the Contractor shall locate the obstruction in the field to sufficient accuracy as required to design the RSS.

4.2.5 Foundation Report

A Foundation Investigation Report that describes the subsurface conditions at the RSS is available, as specified in the Contract documents.

The Owner warrants the data in the Foundation Investigation Report, except that interpretations of the data and opinions expressed in the Foundation Investigation Report are not warranted.

5.0 MATERIALS

5.1 General

All materials for the selected RSS shall be according to the Approved Product Drawings for that RSS.

6.0 EQUIPMENT

6.1 Restriction on Skid-Steer Vehicles

Skid-steer vehicles will not be permitted on any area where the depth of backfill for RSS over installed reinforcing elements is less than 0.5 m.

7.0 CONSTRUCTION

7.1 General

The RSS shall be constructed according to the working drawings and this Special Provision.

Construction of the RSS shall not commence until the Contractor has submitted all applicable Certificates of Conformance for the foundation for RSS.

7.2 RSS Superintendent

The Contractor shall schedule his operations such that the construction of an RSS is at all times under the responsible charge of an RSS Superintendent who has been advised on site by the manufacturer's representative as to the required procedures for the construction of that RSS, for the specified operations and time periods.



7.3 Manufacturer's Representative

The manufacturer's representative shall be on site to advise the RSS Superintendent as to the procedures and placing tolerances required for the construction of the RSS.

For each RSS in the Contract, the Contractor shall arrange for the manufacturer's representative to be on site at commencement of each of the following operations, for a time period of three (3) working days per operation or until the operation is complete, whichever is less:

- a) Layout of the RSS; and construction of the alignment elements, where applicable;
- b) Installation of the facing elements;
- c) Placement and compaction of the backfill for RSS; and installation of the reinforcing elements, where applicable.

Whenever there is a change in the RSS Superintendent during construction of an RSS, the Contractor shall arrange for the manufacturer's representative to return to the site for the same operations and time periods as at commencement.

7.4 Backfill for RSS

Backfill for RSS shall be placed within the lines and grades shown on the working drawings. All backfill for RSS shall be compacted according to OPSS 501.

Unless otherwise shown in the Contract drawings, the Contractor shall not place backfill for RSS against an adjacent concrete structure that is part of the work until the concrete in that structure has obtained a compressive strength at least 70% of the concrete strength specified in the Contract.

7.5 Management of Excess Materials

Management of excess materials shall be according to OPSS 180.

7.6 Corrective Work

At least one week prior to commencement of any corrective work at an RSS during the warranty period, the Contractor shall submit written notice of commencement to the Manager of Contracts.

The Contractor shall repair all deficiencies according to the repair procedures for corrective work. All corrective work shall be done within the RSS warranty period, unless prevented by seasonal shutdown, in which case the corrective work shall be done during the first eight weeks of the following construction season.

The Contractor shall provide access to the corrective work for inspection by the Owner when requested.



8.0 QUALITY ASSURANCE

8.1 Acceptance Criteria at End of the RSS Warranty Period

The Owner will accept the RSS at the end of the RSS warranty period if none of the deficiencies listed in Table 2 are found during the warranty inspections. Where deficiencies are found, the RSS will not be accepted until the Contractor has carried out corrective work to repair the deficiencies.

TABLE 2 – RSS DEFICIENCIES	
Number	Description of Deficiency
1.	Performance tolerance exceeds tolerances given in Table 1.
2.	Damaged facing elements and damaged alignment elements, where applicable.
3.	Dead and dying vegetative elements that are an integral part of the RSS.

8.2 Warranty Inspections

Throughout the warranty period the Owner will carry out warranty inspections of the RSS for deficiencies as per Table 2. The Owner will notify the Contractor as to the date and time of the inspection(s) and the Contractor may, at his discretion, be present during the inspection(s).

Within two weeks following a warranty inspection the Owner will notify the Contractor in writing of all deficiencies that require corrective work.

9.0 MEASUREMENT FOR PAYMENT

9.1 Actual Measurement

9.1.1 Backfill for Retained Soil System, High Performance Backfill for Retained Soil System, Medium Performance Backfill for Retained Soil System, Low Performance

Measurement will be of the mass in tonnes of the material placed within the theoretical lines and grades shown in the stamped working drawings. The method of determining the mass shall be according to OPSS 102.



10.0 BASIS OF PAYMENT

10.1 Retained Soil System, True Abutment - Item Retained Soil System, False Abutment - Item Retained Soil System, Wall/Slope, High Performance – Item Retained Soil System, Wall/Slope, Medium Performance – Item Retained Soil System, Wall/Slope, Low Performance – Item

Payment at the contract price for the above tender items shall be full compensation for all labour, equipment and material to do the work, including all costs associated with the manufacturer's representative on site.

Payment for construction of the foundation for RSS will be made under the appropriate tender items in the Contract.

No payment will be made for corrective work, including investigation of deficiencies, design of repairs, site access, traffic staging and removal of existing work, except where the corrective work is required as a result other than an act or fault of the Contractor.

10.2 Backfill for Retained Soil System, High Performance – Item Backfill for Retained Soil System, Medium Performance – Item Backfill for Retained Soil System, Low Performance – Item

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

When the Contract does not contain a separate tender item for backfill for RSS, the contract price for the RSS contract items in which the backfill for RSS is incorporated shall include full compensation for all labour, equipment and material required to place and compact the backfill for RSS.

WARRANT: Always with these tender items.

APPENDIX FDR-C

Slope Stability Analyses

Figures 1 and 2 – Existing Conditions

Figure 3 – Conceptual Flattened Slope Condition



FIGURE 1 – Existing Conditions
 Highway 11 – Kahshe River North Slope Stability Assessment
 Effective Stress Analysis, Near Station 16+300

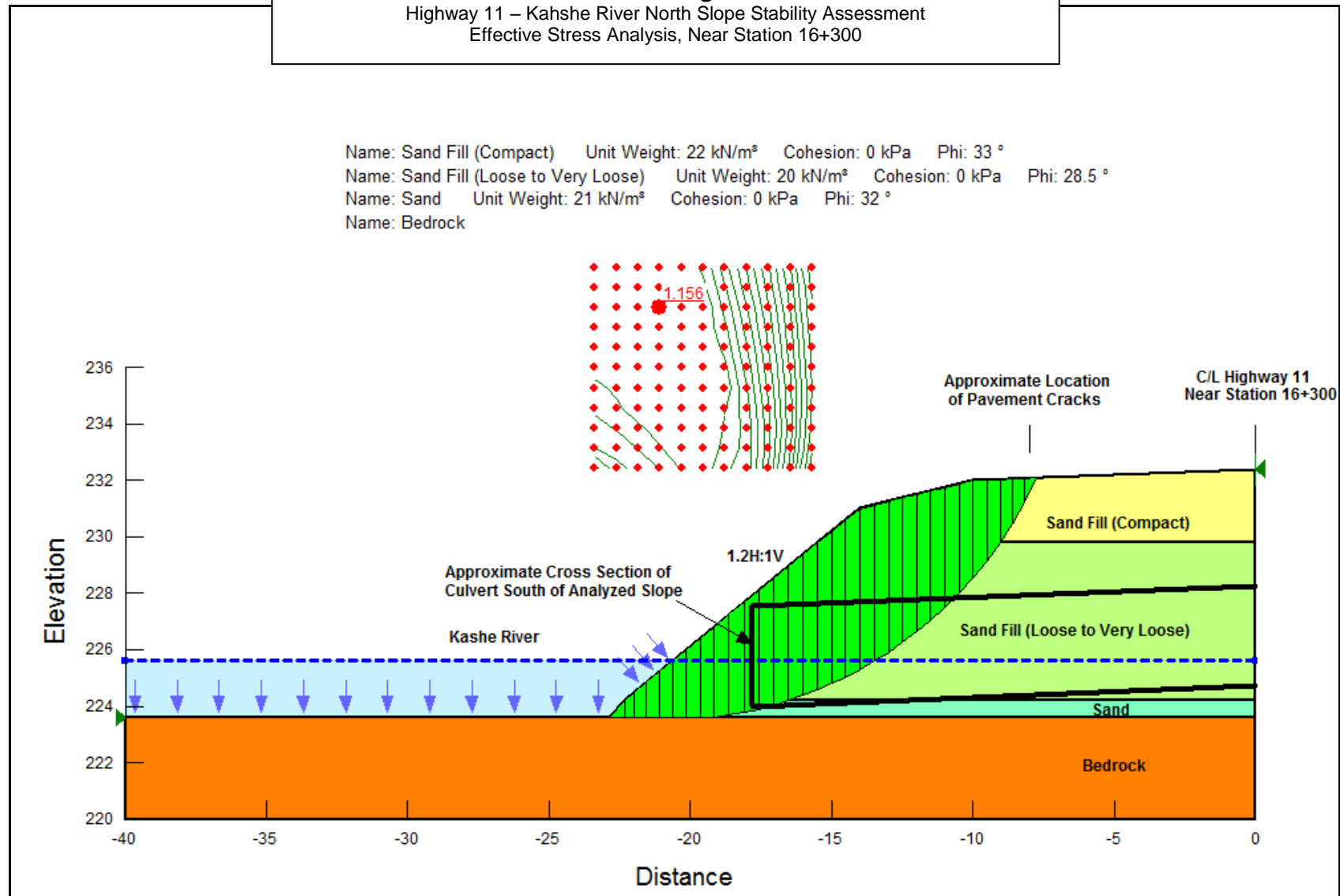


FIGURE 2 – Existing Conditions
Highway 11 – Kahshe River North Slope Stability Assessment
Effective Stress Analysis, Near Station 16+280

Name: Sand Fill (Compact) Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 33 °
Name: Sand Fill (Loose to Very Loose) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 28.5 °
Name: Gravelly Sand Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 °
Name: Bedrock

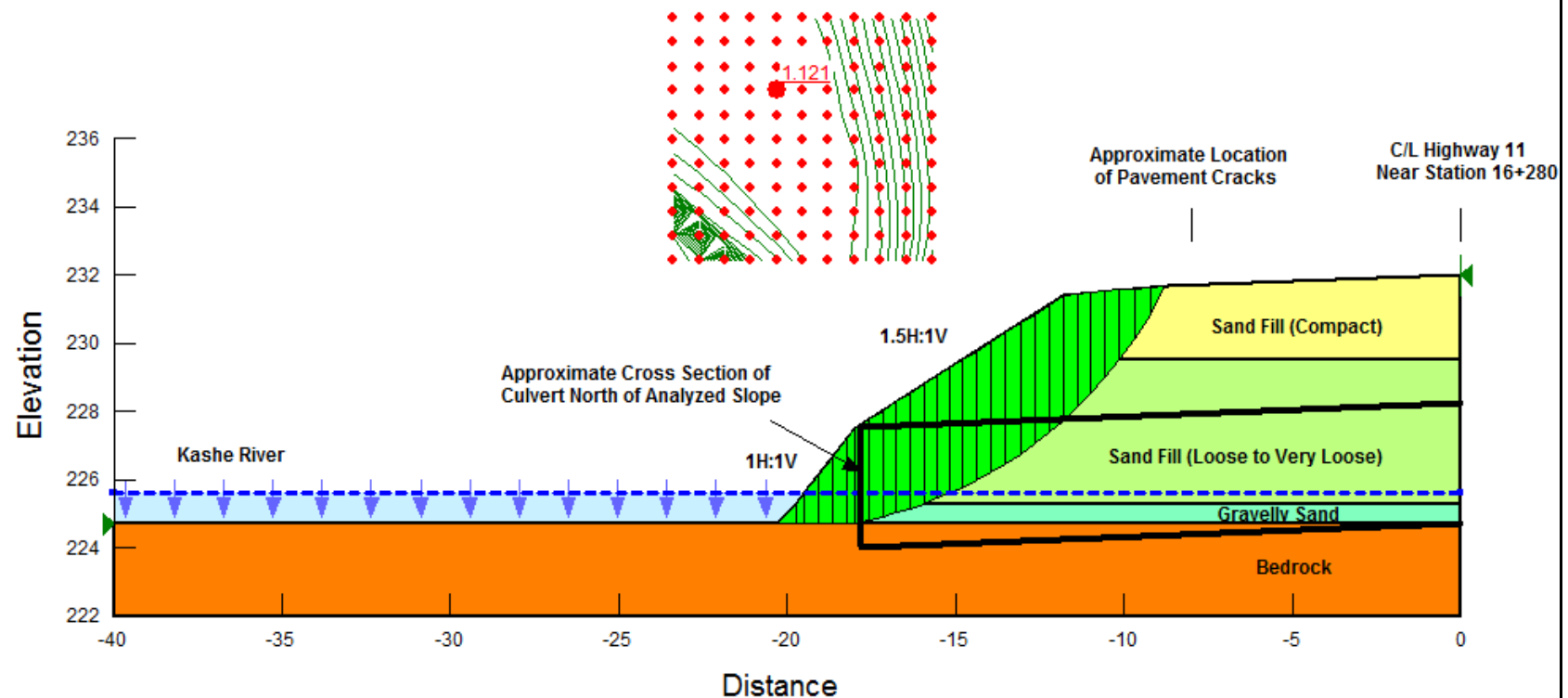


FIGURE 3 – Conceptual Flattened Slope Condition

Highway 11 – Kahshe River Slope Stability Assessment
 Effective Stress Analysis, Near Station 16+280

Name: Sand Fill Compact Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 33 °
 Name: Sand Fill (Loose to Very Loose) Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 28.5 °
 Name: Sand and Gravel Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 °
 Name: Bedrock

