



**DRAFT
FOUNDATION INVESTIGATION AND DESIGN REPORT
for**

**EASTBOUND QUEEN ELIZABETH WAY OVERPASS AT FORD DRIVE
QUEEN ELIZABETH WAY AND HIGHWAY 403
TOWN OF OAKVILLE
REGIONAL MUNICIPALITY OF HALTON, ONTARIO
G.W.P. 2163-10-00, SITE NO. 10-286/1
CENTRAL REGION, ONTARIO**

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PML Ref.: 14TF005-EBFD
Index No.: 080DIR and 081DDR
GEOCRES No.: Not Assigned
March 13, 2017



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PART A – DRAFT FOUNDATION INVESTIGATION REPORT

for
Eastbound Queen Elizabeth Way Overpass at Ford Drive
GWP 2163-10-00, Site 10-286/1
Town of Oakville
Regional Municipality of Halton, Ontario

1. INTRODUCTION

This report presents the factual findings obtained from the geotechnical foundation investigation carried out for the detail design of the replacement of the existing structure carrying the Queen Elizabeth Way (QEW) eastbound lane traffic over Ford Drive, in the Town of Oakville, Regional Municipality of Halton.

The foundation investigation was conducted by Peto MacCallum Ltd. (PML), retained as a sub-consultant to Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

This report provides subsurface information pertaining to the proposed structure and approaches within approximately 20 m of the abutments. A review of the following reports for the existing structure was carried out.

1. Preliminary Foundation Investigation and Design Report
QEW Overpass at Ford Drive - Reconstruction
Queen Elizabeth Way / Highway 403 Improvements
Oakville, Ontario
W.O. 09-20007
GEOCRES No. 30M5-297
Thurber Engineering Ltd. dated October 15, 2013
2. Foundation Investigation Report
QEW Over Ford Drive
W.P. 125-66-17, Site 10-286
QEW, District 4, Hamilton
GEOCRES No. 30M5-116
Engineering Materials Office – Soil Mechanics Section dated January 25, 1978

The subsurface information from GEOCRES boreholes 3, 4, 13-23 and 13-24 advanced as part of the previous investigations (GEOCRES Nos. 30M5-116 and 30M5-297) is considered to be relevant and used in this report.



All elevations in this report are expressed in meters.

2. SITE DESCRIPTION

The site is situated approximately 0.5 km south of the QEW / Highway 403 interchange. The existing structure carries the QEW eastbound traffic over Ford Drive. The performance of the existing structure foundations and related approach embankments appears to be satisfactory.

Lands within the QEW / Highway 403 right of way near the project site are generally vacant and grass covered. The topography of the area is gently sloping down towards the south. The Ford Drive roadway is located within a cut, some 6 to 8 m below the QEW road grade.

Outside of the highway right of ways, land use primarily includes commercial and light industrial buildings and businesses. The Ford Motor Company occupies the majority of the land to the south of the QEW / Highway 403. Site photographs are included in Appendix FIR-A.

3. FIELD INVESTIGATION PROCEDURES

The field work for this study was carried out during the period of November 22 to November 25, 2015 and comprised 3 new boreholes (15-1, 15-3 and 1-NEW) drilled to depths ranging from 5.8 m to 8.8 m. The records of the current boreholes and of the GEOCRETS boreholes 3, 4, 13-23 and 13-24 are attached in Appendix FIR-B. The locations of the boreholes are shown on Drawing EBFD-1, provided in Appendix FIR-C.

The locations of the new boreholes were established in the field by PML. All three boreholes were advanced using continuous flight hollow and solid stem augers, powered by a truck mounted D50 drill rig, supplied and operated by Tri-Phase Group, a specialist drilling subcontractor, working under the full-time supervision of a member of the PML engineering staff. Two boreholes (15-1 and 15-3) were extended 3.0 m into bedrock using NQ diamond rock coring equipment.



Representative soil samples were recovered at 0.75 m and 1.5 m depth intervals using the standard penetration test (SPT) method. Standard penetration tests were conducted to assess the strength characteristics of the substrata. The results of the field tests and observations are reported on the Record of Borehole sheets, provided in Appendix FIR-B.

Soils were identified and described 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 and, where encountered, by measuring the groundwater level in the open boreholes.

Surveying of the boreholes was conducted after drilling by Callon Dietz Inc., under contract to Stantec and the coordinates and ground surface elevations of all three boreholes were collected and provided on the Record of Borehole sheets and on Drawing EBFD-1.

During drilling the target termination criterion of 100 blows per 0.3 m penetration or refusal on bedrock was met for all three boreholes. The boreholes were backfilled in accordance with the MTO guideline and MOE Reg. 903 for borehole abandonment procedures.

4. LABORATORY TEST PROCEDURES

The recovered soil samples were returned to the PML laboratory in Toronto for detailed visual examination, laboratory testing and classification. Table 4.0 provides the types and quantities of the laboratory tests completed for the foundation investigation.

Table 4.0: Laboratory Testing Program

LABORATORY TEST	QUANTITY
Natural Moisture Content	13
Grain Size Distribution Analyses	5
Atterberg Limits	4

The grain size distribution curves of selected soil samples are presented on Figures EBFD-GS-1 to EBFD-GS-3. The results of the Atterberg Limits tests are given on Figures EBFD-PC-1 and



EBFD-PC-2. All of the laboratory figures from the current and previous investigations are provided in Appendix FIR-D and the test results are summarized on the Record of Borehole sheets, provided in Appendix FIR-B.

5. SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Site Geology

The project area lies within the physiographic region known as the South Slope. The South Slope is bounded by the Peel Plain to the north and the Iroquois Plain to the south. The physiographic region extends from the Niagara escarpment to the Trent River and covers approximately 2,435 square kilometers. The South Slope is characterized by glacial deposits overlying shale bedrock of the Queenston and Meaford-Dundas Formations. (L.J. Chapman and D.F. Putnam, *The Physiography of Southern Ontario*, 3rd Edition, 1984).

Locally, the Meaford-Dundas Formation is a medium gray shale with good fissility and resistant interbeds of gray fossiliferous limestones and siltstones.

5.2 Subsurface Conditions

Reference is made to the Record of Borehole sheets for details of the subsurface conditions including soil classifications, bedrock descriptions, inferred stratigraphy, boundary elevations, SPT data and groundwater observations.

A stratigraphic profile and sections along the proposed abutments were prepared from the borehole data and are shown on Drawing EBFD-1 and Drawing EBFD-2, respectively. The boundaries between soil strata were established at borehole locations only. The soil boundaries between and beyond the boreholes are assumed and may vary from what is shown on Drawing EBFD-1.

The subsurface stratigraphy revealed in the boreholes generally comprised fill layers and a cohesive deposit of clayey silt / silty clay overlying low to medium strength, highly weathered



shale bedrock. Limestone interbedded with slightly weathered shale bedrock was encountered within the Meaford-Dundas Formation, underlying the highly weathered shale.

Groundwater was not observed in any of the boreholes during or upon completion of augering. Boreholes 15-1 and 15-3 were charged with drilling water to facilitate the rock coring operations.

5.2.1 Fill

Asphalt, 150 mm to 230 mm in thickness was present surficially in boreholes 15-1, 15-3 and 1-NEW advanced on the shoulder of the QEW eastbound lane. Asphalt is underlain by non-cohesive sand and gravel pavement fill which extends to elevation 128.6 in borehole 15-1, 130.5 in borehole 15-3 and 128.6 in borehole 1-NEW.

Underlying the 150 mm of asphalt in boreholes 13-23 and 13-24 was sand and gravel fill. The non-cohesive fill is compact to dense in relative density with SPT-N values ranging from 19 to 31. The sand and gravel fill was 1.3 m thick in both boreholes and penetrated at elevation 128.2 and 130.0 in boreholes 13-23 and 13-24, respectively.

Sandy gravel fill was encountered underlying the pavement fill in borehole 15-3. The sandy gravel fill was 0.7 m thick and extended to elevation 129.8. The results of the grain size distribution analysis performed on the sandy gravel fill sample is presented in Figure EBFD-GS-1. Cobbles were observed within the sandy gravel fill layer in borehole 15-3.

Clayey silt fill material was encountered underlying the pavement fill in boreholes 15-1 and 1-NEW. The thickness of the clayey silt fill was 1.3 m in borehole 15-1 and 1.6 m in borehole 1-NEW. The clayey silt fill extended to elevation 127.3 and 127.0 in boreholes 15-1 and 1-NEW, respectively.

SPT-N values of the clayey silt fill ranged from 4 to 13, indicating firm to stiff consistency. Organic inclusions were observed within the clayey silt fill in borehole 1-NEW.

The results of grain size distribution analysis for a sample collected from the cohesive fill is shown on Figure EBFD-GS-2. The Atterberg plasticity chart is presented on Figure EBFD-PC-1.



Table 5.2.1 summarizes the results of the grain size distribution analysis conducted on the sample of clayey silt fill material.

Table 5.2.1: Grain Size Distribution – Fill

MATERIAL	PERCENTAGE
Gravel	20
Sand	31
Silt	29
Clay	20

The liquid and plastic limits of the fill were 29 and 16 respectively, with the corresponding plasticity index of 13. The moisture content determinations ranged from 16 to 27%, corresponding to a moist soil condition.

5.2.2 Clayey Silt / Silty Clay

A cohesive deposit of clayey silt / silty clay was present surficially in boreholes 3 and 4. The deposit ranged in thickness from 2.2 m to 2.3 m and penetrated into weathered shale bedrock at elevation 126.3 in borehole 3 and 128.0 in borehole 4.

Clayey silt / silty clay was encountered in boreholes 15-1, 15-3, 1-NEW, 13-23 and 13-24, overlain by the fill material. The thickness of the cohesive deposit ranged from 0.3 m to 2.2 m and extends to highly weathered shale bedrock at elevation 125.9 in borehole 15-1, 128.6 in borehole 15-3, 126.2 in borehole 1-NEW, 126.1 in borehole 13-23 and 129.7 in borehole 13-24. Shale bedrock fragments were encountered in the cohesive deposit in boreholes 15-1, 15-3 and 13-24.

SPT-N values of the clayey silt / silty clay ranged from 6 to 49 indicating firm to hard consistency, typically stiff to very stiff consistency. The results of grain size distribution analyses and Atterberg limits testing conducted on three samples of the native clayey silt to silty clay are presented in respective Figures EBFD-GS-3 and EBFD-PC-2.



Table 5.2.2 summarizes the results of the grain size distribution analyses conducted on the native soil from boreholes 15-1, 15-3 and 1-NEW.

Table 5.2.2: Grain Size Distribution – Clayey Silt / Silty Clay

MATERIAL	PERCENTAGE
Gravel	0-24
Sand	6-28
Silt	32-52
Clay	16-52

The clayey silt to silty clay had a liquid limit ranging from 27 to 44, a plastic limit ranging from 17 to 22 and a corresponding plasticity index ranging from 10 to 22. The moisture content of the deposit ranged from 13 to 24%, below the plastic limit, indicating a moist soil condition.

5.2.3 Bedrock

Bedrock was contacted or inferred by split spoon refusal and auger grinding below the native clayey silt to silty clay material in the three boreholes drilled during the current investigation and in the four boreholes drilled from the previous investigations. The depths and elevations at which the top of the bedrock was encountered are summarised in Table 5.2.3.

Table 5.2.3: Depths and Elevations of Bedrock Surface

STRUCTURE ELEMENT	BOREHOLE	BEDROCK SURFACE	
		DEPTH (m)	ELEVATION (m)
South (Construction West) Approach	1-NEW	3.0	126.2
South (Construction West) Abutment	15-1	3.6	125.9
	13-23	3.7	126.1
	3	2.3	126.3
North (Construction East) Abutment	15-3	2.6	128.6
	13-24	1.8	129.7
	4	2.2	128.0



The bedrock in Meaford-Dundas Formation comprised a grey to dark grey highly to slightly weathered low to medium strength shale bedrock with limestone interbeds. The shale bedrock has thin horizontal bedding and dipping to vertical joints. Seams or layers of clayey silt / silty clay were also noted within the highly weathered zones of the bedrock. The shale bedrock is susceptible to wetting/drying cycles and not durable upon exposure to the elements.

During the drilling operation of boreholes 15-1, 15-3 and 1-NEW, within the shale bedrock formation, auger refusal was encountered on the limestone interbeds. The limestone interbeds are significantly harder to penetrate than the highly weathered shale bedrock.

GEOCRETS 30M5-297 estimated the unconfined compression strength (UCS) of the shale with hard limestone interbeds ranging from 42 MPa to 97 MPa, indicating a medium to strong bedrock strength classification. The values were interpreted from point load tests conducted on intact cores.

The rock cores retrieved from boreholes 3, 4, 15-1, 15-3, 13-23 and 13-24 are described on the corresponding borehole logs attached in Appendix FIR-B. A detailed description of the bedrock cores retrieved from boreholes 15-1 and 15-3 is given in Table A, attached in Appendix FIR-E. Photographs of the bedrock cores retrieved from borehole 15-1 and 15-3 are also attached in Appendix FIR-E.

The measured core recovery varied between 43% and 100%. The Rock Quality Designation (RQD) determined from the rock cores ranged from 11% to 83%, typically 25% to 70% thus indicating a poor to fair quality bedrock.

The low RQD values of 11% and 18%, presented in boreholes 15-1 and 15-3 respectively, likely reflect local conditions of weathered bedrock that differ from the RQD values of 40% to 83% seen in the slightly weathered bedrock in the MTO GEOCRETS reports.



5.2.4 Groundwater

During the process of augering, groundwater was not detected in any of the boreholes drilled during the current investigation (15-1, 15-3 and 1-NEW). Boreholes 15-1 and 15-3 were charged with drilling water during the process of coring the bedrock.

Boreholes 13-23 and 13-24 drilled during October 2013 were also dry upon completion of augering and charged with drilling water to facilitate the coring operation.

Groundwater levels observed during the 1978 investigation ranged from 1.2 to 1.8 m below the ground surface. However, an indication was not provided in the GEOCRE 30M5-116 report whether the recorded water level was a result of the coring operation.

It should be noted that groundwater levels are susceptible to seasonal fluctuations. In particular, the groundwater level may increase after the spring snowmelt or periods of significant and/or prolonged precipitation events.



6. CLOSURE

The field work was carried out under the supervision of Mr. S. Aziz, under the direction of Mr. K. R. Daly, P.Eng. The drilling equipment was supplied and operated by Tri-Phase Group. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.

This Foundation Investigation Report was prepared by Mr. K. R. Daly, P.Eng, and reviewed by Mr. G.O. Degil, PhD, P.Eng., Senior Engineer. 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.

A handwritten signature in black ink, appearing to read "Kyle R. Daly", written in a cursive style.

Kyle R. Daly, P.Eng.
Project Engineer, Geotechnical Services

A handwritten signature in black ink, appearing to read "Grigory O. Degil", written in a cursive style.

Grigory O. Degil, PhD, P.Eng.
Senior Engineer, Geotechnical Services

A handwritten signature in black ink, appearing to read "Carlos M.P. Nascimento", written in a cursive style.

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

KD/CN/GD:nk

NOTE: The Final Report will be signed and stamped by two Professional Engineers licensed by PEO, one of whom shall be the Designated Principal Contact for MTO foundation projects



APPENDIX FIR - A

Site Photographs



Photograph 1: Existing structures carrying QEW EB and WB Lanes over Ford Drive.



Photograph 2: Taken near the QEW Eastbound construction east abutment.



APPENDIX FIR – B

Explanation of Terms Used in Report

Record of Borehole Sheets

GEOCRES Boreholes Logs from 30M5-297 and 30M5-116

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				i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO	WTPL		WETTER THAN PLASTIC LIMIT			

RECORD OF BOREHOLE No 15-1

1 of 1

METRIC

G.W.P. 2163-10-00 LOCATION Coords: 4 817 185.5 N ; 290 788.9 E ORIGINATED BY S.A.
 DIST Central HWY QEW BOREHOLE TYPE Continuous Flight Hollow Stem Augers and NQ Coring COMPILED BY K.D.
 DATUM Geodetic DATE November 23 and 24, 2015 CHECKED BY G.D.

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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE							
129.5	Ground Surface						20	40	60	80	100							
0.0	180 mm asphalt over sand and gravel, trace silt																	
128.6	Compact Grey/red (PAVEMENT FILL)		1	SS	13													
0.9	Clayey silt mixed with sand and gravel		2	SS	14													
127.3	Stiff Red/brown Moist (FILL)		3	SS	8											20 31 29 20		
2.2	Silty clay, trace sand		4	SS	14													
	Stiff Red/brown Moist															0 6 42 52		
	some sand shale fragments		5	SS	23													
125.9	Very stiff																	
3.6	Highly weathered shale bedrock silty clay seams Grey/red		6	SS	50/10cm													
123.7	Limestone with interbedded shale bedrock		7	RC NQ	REC 78%											RQD 31%		
	Slightly weathered to moderately weathered																	
	Low to medium strength		8	RC NQ	REC 43%											RQD 11%		
	Poor to very poor quality																	
120.7	End of borehole																	
8.8																		
	* Borehole charged with coring water																	

RECORD OF BOREHOLE No 15-3

1 of 1

METRIC

G.W.P.	2163-10-00	LOCATION	Coords: 4 817 242.4 N ; 290 787.1 E	ORIGINATED BY	S.A.
DIST	Central	HWY	QEW	BOREHOLE TYPE	Continuous Flight Hollow Stem Augers and NQ Coring
DATUM	Geodetic	DATE	November 24 and 25, 2015	CHECKED BY	G.D.

[illegible]

RECORD OF BOREHOLE No 1-NEW

1 of 1

METRIC

G.W.P. 2163-10-00 LOCATION Coords: 4 817 174.1 N ; 290 789.7 E ORIGINATED BY S.A.
DIST Central HWY QEW BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY K.D.
DATUM Geodetic DATE November 22 and 23, 2015 CHECKED BY G.D.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										20 40 60		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
129.2	Ground Surface																			
0.0	230 mm asphalt over sand and gravel (PAVEMENT FILL)		1	SS	14		129													
128.6	Clayey silt some sand, trace gravel organic inclusions		2	SS	13		128						○							
0.6	Stiff Reddish Moist to firm brown (FILL)		3	SS	4								○							
127.0	Silty clay, trace sand						127													
2.2	Stiff Grey Moist		4	SS	13								○			0 9 52 39				
126.2																				
3.0	Highly weathered shale bedrock silty clay seams Grey/red		5	SS	50/10cm		126													
			6	SS	50/8cm		125						○							
123.4			7	SS	50/3cm		124													
5.8	End of borehole Refusal to augering																			
	* Borehole dry upon completion of augering																			

RECORD OF BOREHOLE No 13-23

1 OF 2

METRIC

W.P. _____ LOCATION N 4 817 184.8 E 290 769.4 ORIGINATED BY GA
 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.05.25 - 2013.05.25 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				W P W W L				
								20 40 60 80 100	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE			WATER CONTENT (%) 20 40 60				
129.7																
0.0	ASPHALT: (150mm)															
0.2	SAND and GRAVEL, some silt Dense to Compact Brown to Reddish Brown Damp (FILL)		1	SS	31		129								39 46 15 (SI+CL)	
			2	SS	19											
128.2																
1.5	Silty CLAY, trace sand Firm to Very Stiff Reddish Brown		3	SS	6		128									
			4	SS	8		127								0 4 40 56	
			5	SS	19											
126.1																
3.7	SHALE, with limestone interbeds, highly weathered, grey						126									
			6	SS	50/ 0.125		125									
	Start coring at 6.1m						124									
	Slightly weathered to fresh, thinly bedded, grey, occasional limestone interbeds Clay seam (200mm) at 6.1m		1	RUN			123								RUN #1 TCR=100% SCR=80% RQD=53% UCS=97MPa (Average)	
	Limestone interbeds (25mm to 75mm) at 6.3m, 6.4m, 6.5m, 6.7m, 6.8m, 7.0m, 7.2m and (125mm) at 7.4m Vertical fracture (125mm) at 7.4m						122								RUN #2 TCR=100% SCR=97% RQD=83% UCS=72MPa (Average)	
	Horizontal fracture at 6.4m, 6.5m, 6.6m, 6.7m, 6.8m, 6.9m, 7.7m, 7.9m, 8.1m, 8.5m, 8.7m		2	RUN			121									
	Limestone interbeds (25mm) at 7.6m, 7.9m, 8.0m, 8.2m, 8.5m, 8.9m, 9.1m and (75mm) at 8.7m															
120.6																
9.1	END OF BOREHOLE AT 9.1m. BOREHOLE OPEN TO 9.1m AND WATER LEVEL AT 4.8m UPON COMPLETION OF CORING. BOREHOLE BACKFILLED WITH															

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

[illegible]

RECORD OF BOREHOLE No 13-24

1 OF 1

METRIC

W.P. _____ LOCATION N 4 817 241.5 E 290 767.3 ORIGINATED BY GA
 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.05.24 - 2013.05.25 CHECKED BY LRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
131.5								20 40 60 80 100					
0.0	ASPHALT: (150mm)												
0.2	SAND and GRAVEL, some silt Compact Brown Damp (FILL)		1	SS	30		131						
			2	SS	28								
130.0							130						
1.5	Silty CLAY, trace sand, occasional shale fragments		3	SS	16								
129.7	Very Stiff Reddish Brown												
1.8	SHALE, with limestone interbeds, highly weathered, grey		4	SS	50/ 0.150		129						
	Start coring at 3.3m		5	SS	50/ 0.100								
			1	RUN			128						
							127						
			2	RUN			126						
							125						
			3	RUN			124						
123.6													
7.9	END OF BOREHOLE AT 7.9m. BOREHOLE OPEN TO 7.9m AND WATER LEVEL AT 3.9m UPON COMPLETION OF CORING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m, CONCRETE TO 0.15m, THEN ASPHALT COLD PATCH TO SURFACE.												

ONTMT4S 1184.GPJ 2012TEMPLATE(MTO).GDT 11/10/13

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

W P L25-06-17 LOCATION Co-ords N 15 803 724; E 954 012 ORIGINATED BY CTJ
 DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger, BXL Core COMPILED BY CTJ
 DATUM Geodetic DATE March 23, 1977 CHECKED BY RS

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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
422.0	Ground Level							20 40 60 80 100								
0.0	Clayey Silt To Silty Clay, Some Sand, Trace Of Gravel		1	SS	46		420									
414.5	Hard		2	SS	32											
412.3	(Weathered)		3	SS	112		10"									
9.5	(Sound)		4	BXL	91% REC		410							RQD 30%		
	Shale Bedrock (See Below)*		5	BXL	100% REC		400							RQD 63%		
392.3																
29.7	End Of Borehole															
	*Intermittent shale, shaly limestone & limestone, fine tex- ture, soft to med.hard light grey, shale is fissile, thin bedding with Limestone (med. hard, fine texture, light grey, fossil- iferous) seams from 12'8" to 13'6" 19'6" to 20'2" 25'3" to 26'2"															

RECORD OF BOREHOLE No 4

W P 125-66-17 LOCATION Co-ords N 15 803 823; E 954 023 ORIGINATED BY CTJ
 DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger, BXL Core COMPILED BY CTJ
 DATUM Geodetic DATE March 22, 1977 CHECKED BY RS

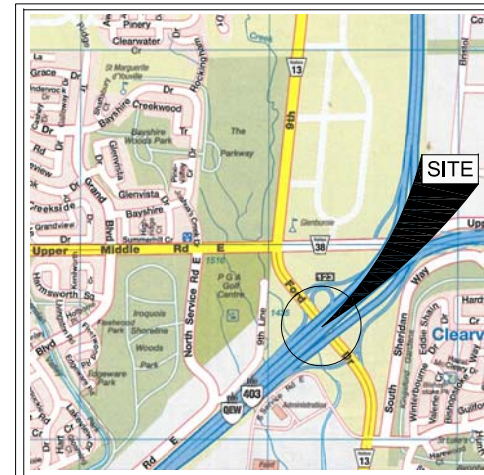
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT Wp	NATURAL MOISTURE CONTENT W	LIQUID LIMIT Wl	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE										10 20 30		
427.1	Ground Level																			
0.0	Clayey Silt To Silty Clay, Some Sand Traces Of Gravel (Reworked) Very Stiff		1	SS	16		420									5 31 39 25				
420.0			2	SS	111/8"															
7.1			3	SS	131/9"															
417.1	(Weathered)		4	BXL	84% REC		410									RQD 25%				
10.0	(Sound)		5	BXL	100% REC											RQD 15%				
	Shale Bedrock (See Below)*		6	BXL	97% REC											RQD 60%				
397.9							400													
29.2	End Of Borehole																			
	*Intermittent Shale, Shaly Limestone & Limestone Beds, Soft To Hard, Fine Texture, Shale ls Fissile, Light Grey Colour, Thin hori- zontal Bedding With Limestone (Hard, Fine Texture fossiliferous) seams from 11'10" to 12'4" 13' 6" to 14'2" 22' 2" to 22'6" 23' 0" to 23'10" 28'10" to 29'2"																			



APPENDIX FIR – C

Drawing EBFD-1 – Borehole Locations and Soil Strata

Drawing EBFD-2 – Soil Strata



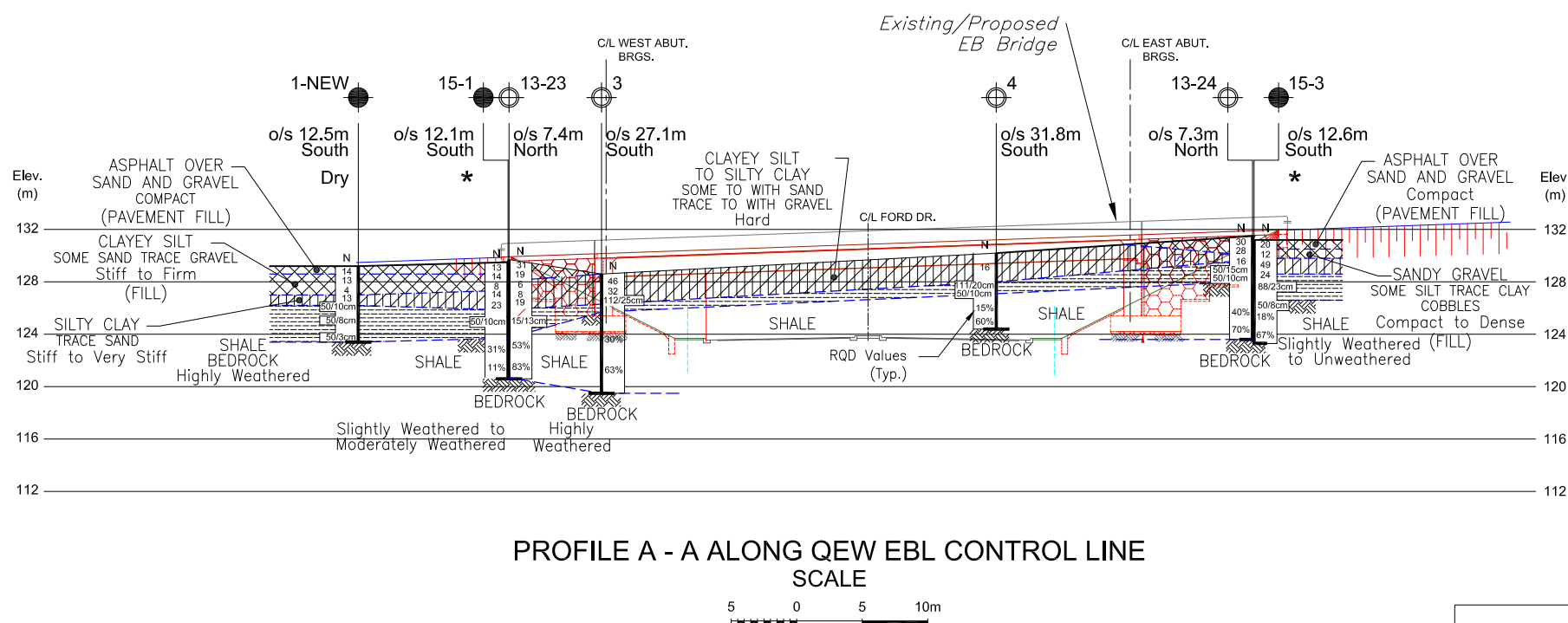
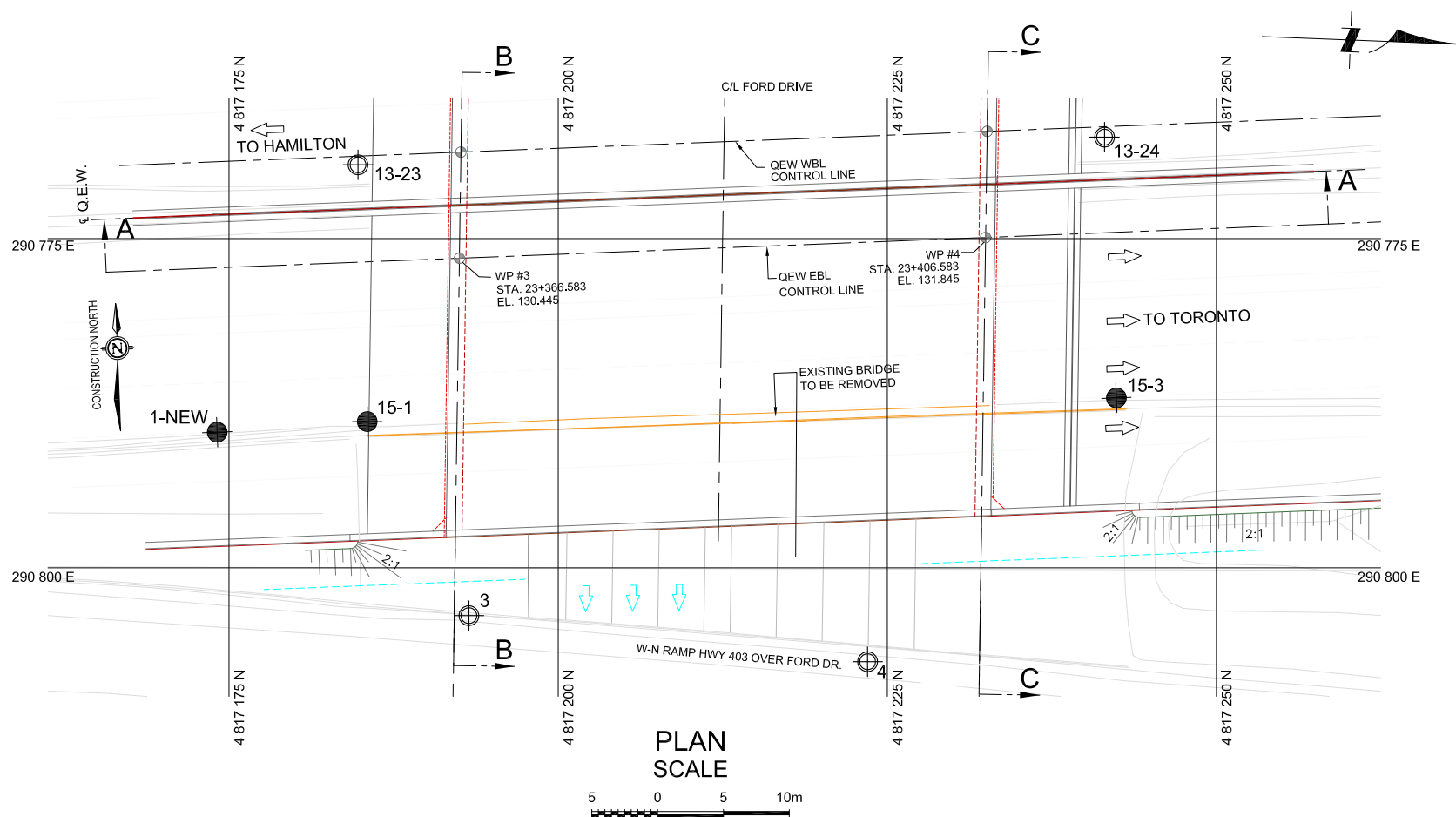
LEGEND			
	Borehole		
	Geocres Report Borehole (30M5-116 & 30M5-297)		
N	Blows/0.3m (Std. Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60 Cone, 475 J/blow)		
	WL at time of investigation March 1977, May 2013 and Nov. 2015		
*	Water level not established		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		

BH No	ELEVATION	NORTHINGS	EASTINGS
15-1	129.5	4 817 185.5	290 788.9
15-3	131.2	4 817 242.4	290 787.1
1-NEW	129.2	4 817 174.1	290 789.7
GEOCRES REPORT BOREHOLES			
13-23	129.7	4 817 184.8	290 769.4
13-24	131.5	4 817 241.5	290 767.3
3	128.6	4 817 193.2	290 803.9
4	130.2	4 817 223.5	290 807.1

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

DATE	BY	DESCRIPTION

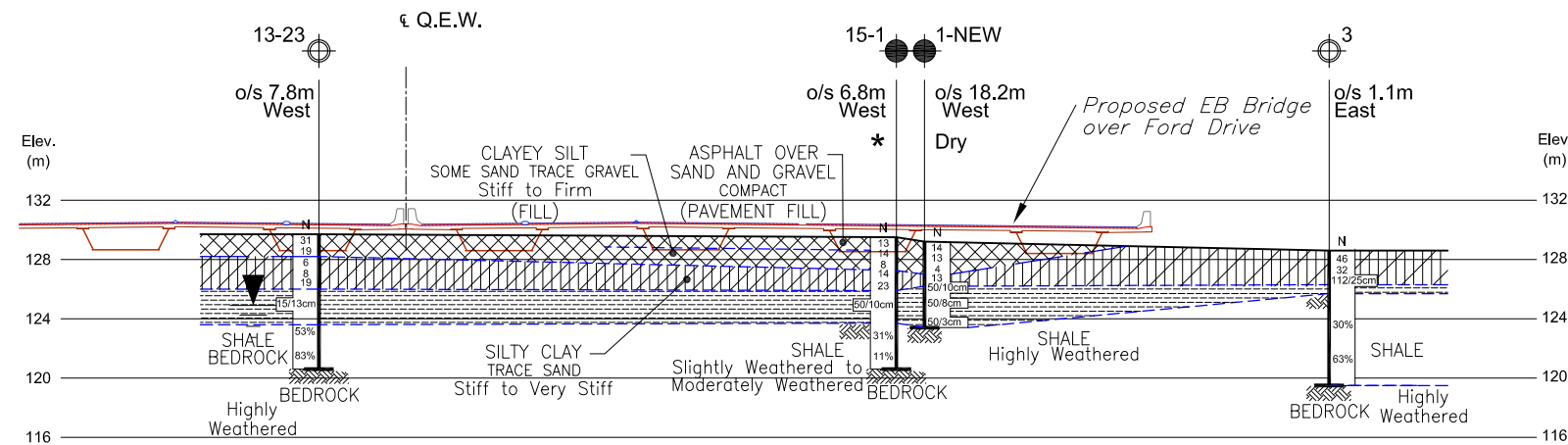
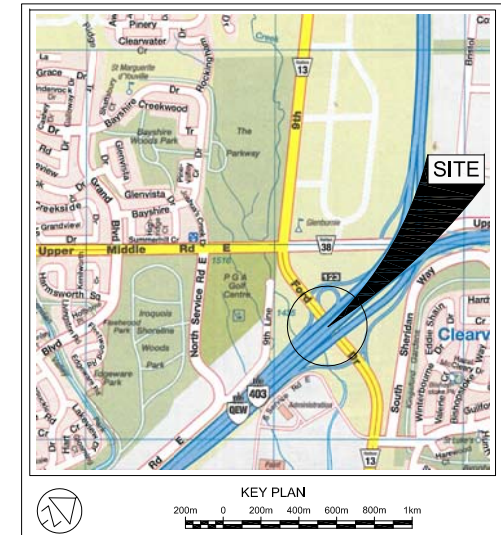
Geocres No. XXX-XXX			
HWY No	QEW / 403	DATE	MARCH 13, 2017
SUBMT	NA	CHECKED	KD
DRAWN	NA	CHECKED	GD



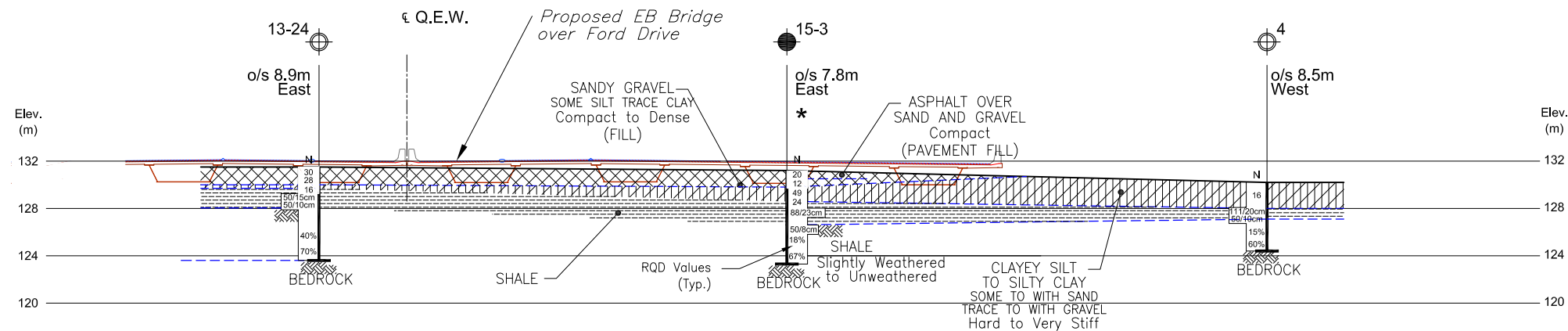
- NOTES:
- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
 - REFER TO DRAWING EBFD-2 FOR SECTIONS B-B AND C-C.
 - THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
 - DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

DRAFT NOTE:
STAMP TO BE ADDED
FOR FINAL REPORT.

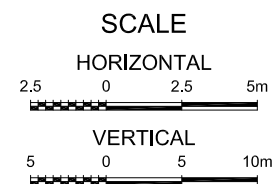
DRAFT NOTE:
STAMP TO BE ADDED
FOR FINAL REPORT.



SECTION B - B (ALONG C/L WEST ABUTMENT)



SECTION C - C (ALONG C/L EAST ABUTMENT)



NOTES:

- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
- REFER TO DRAWING EBFD-1 FOR BOREHOLE AND SECTION LOCATION PLAN AND PROFILE A-A.
- 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.

DRAFT

DRAFT NOTE:
STAMP TO BE ADDED
FOR FINAL REPORT.

DRAFT NOTE:
STAMP TO BE ADDED
FOR FINAL REPORT.

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

DATE	BY	DESCRIPTION

Geocres No. XXX-XXX			
HWY No	QEW / 403	DIST	CENTRAL
SUBMD	NA	CHECKED	KD
DATE	MARCH 13, 2017	SITE	10-286/1
DRAWN	NA	CHECKED	GD
APPROVED	CN	DWG	EBFD-2



APPENDIX FIR – D

Figure EBFD-GS-1 – Grain Size Distribution for Sandy Gravel Fill

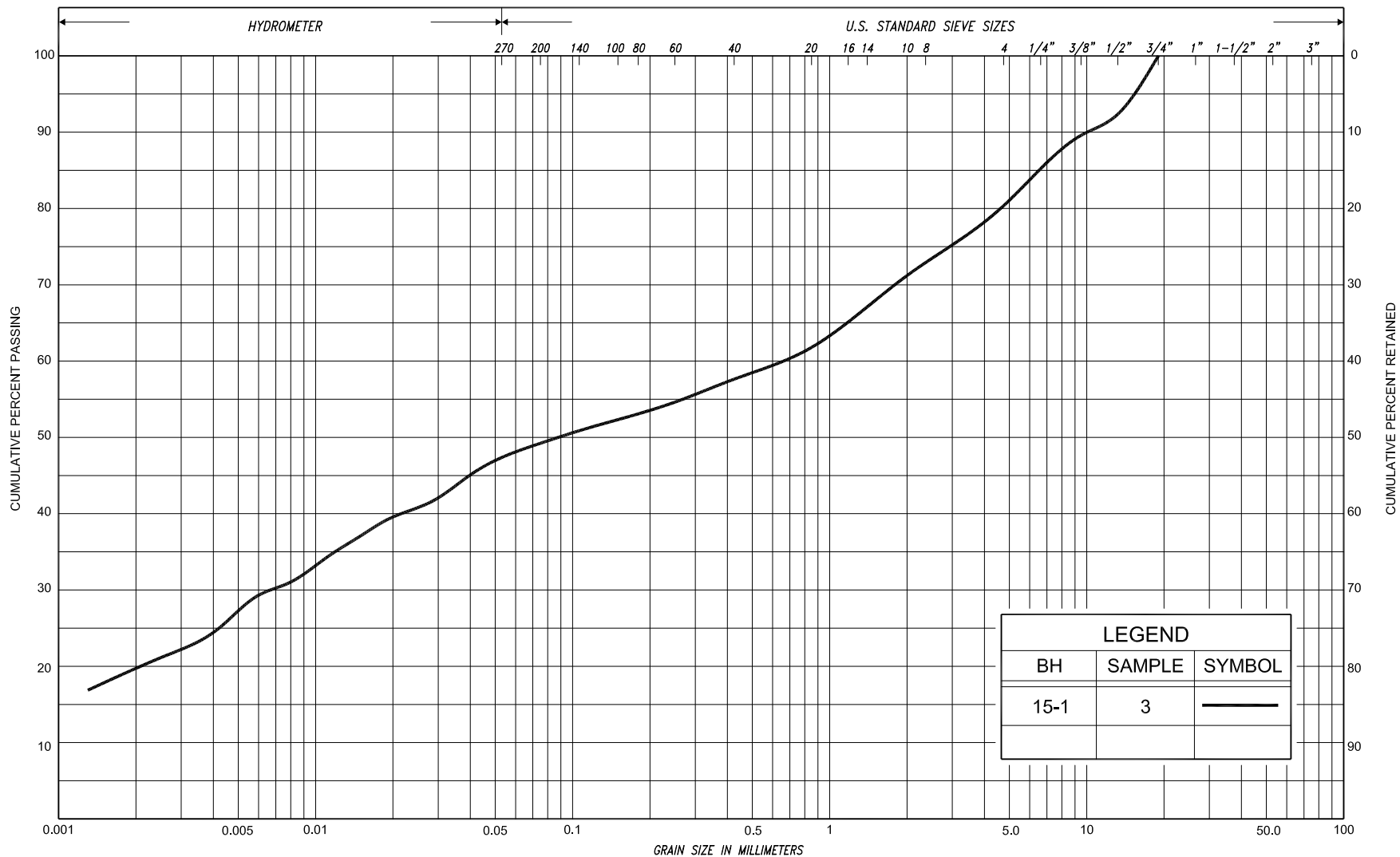
Figure EBFD-GS-2 – Grain Size Distribution for Clayey Silt Fill

Figure EBFD-GS-3 – Grain Size Distribution for Clayey Silt to Silty Clay

Figure EBFD-PC-1 – Plasticity Chart for Clayey Silt Fill

Figure EBFD-PC-2 – Plasticity Chart for Clayey Silt to Silty Clay

Figure B1 and B2 – GEOCRETS 30M5-297 Laboratory Test Results



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

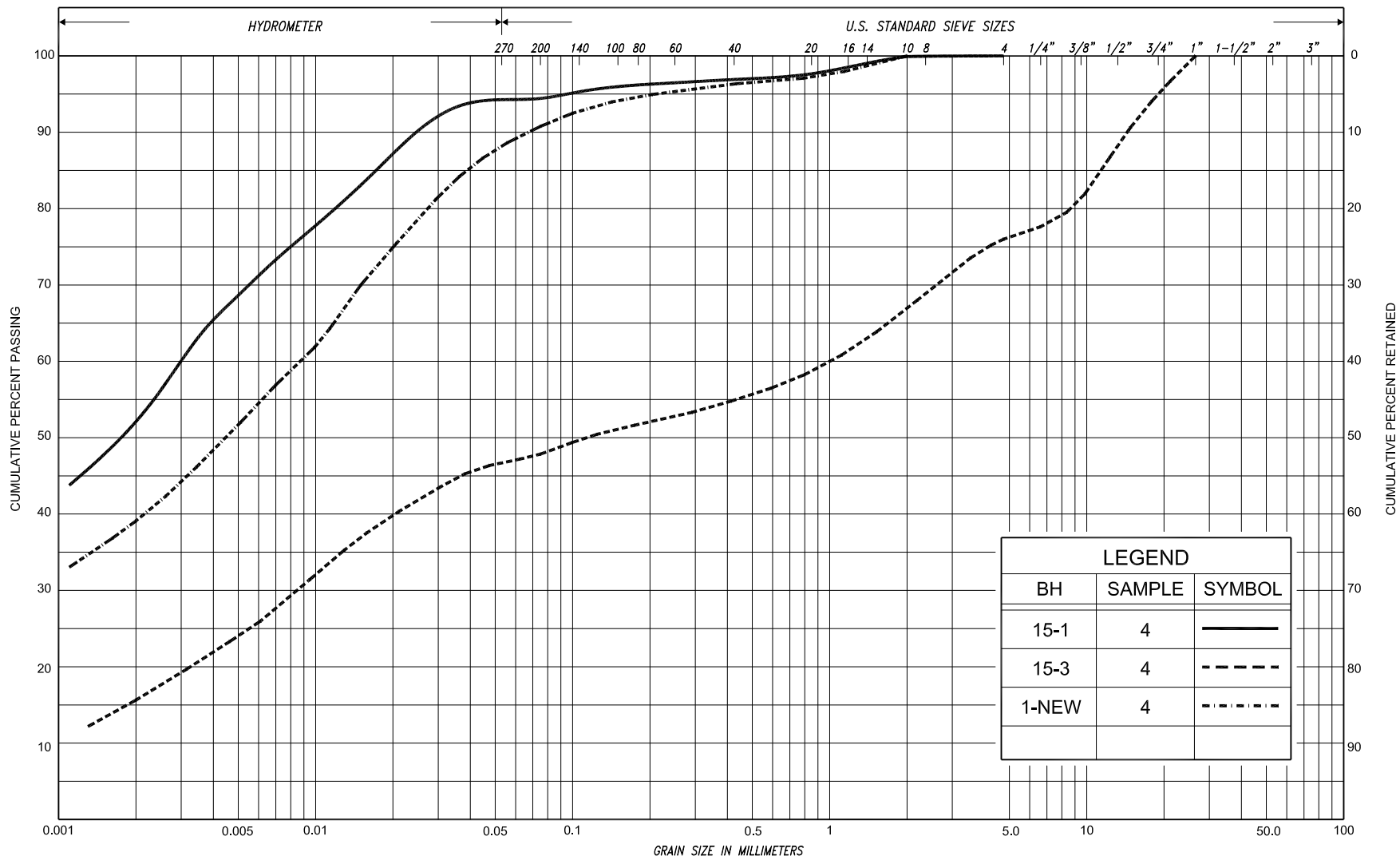


GRAIN SIZE DISTRIBUTION CLAYEY SILT, sandy, with gravel (CL) (FILL)

FIG No. EBFD-GS-2

HWY: 403 / QEW

G.W.P. No. 2163-10-00

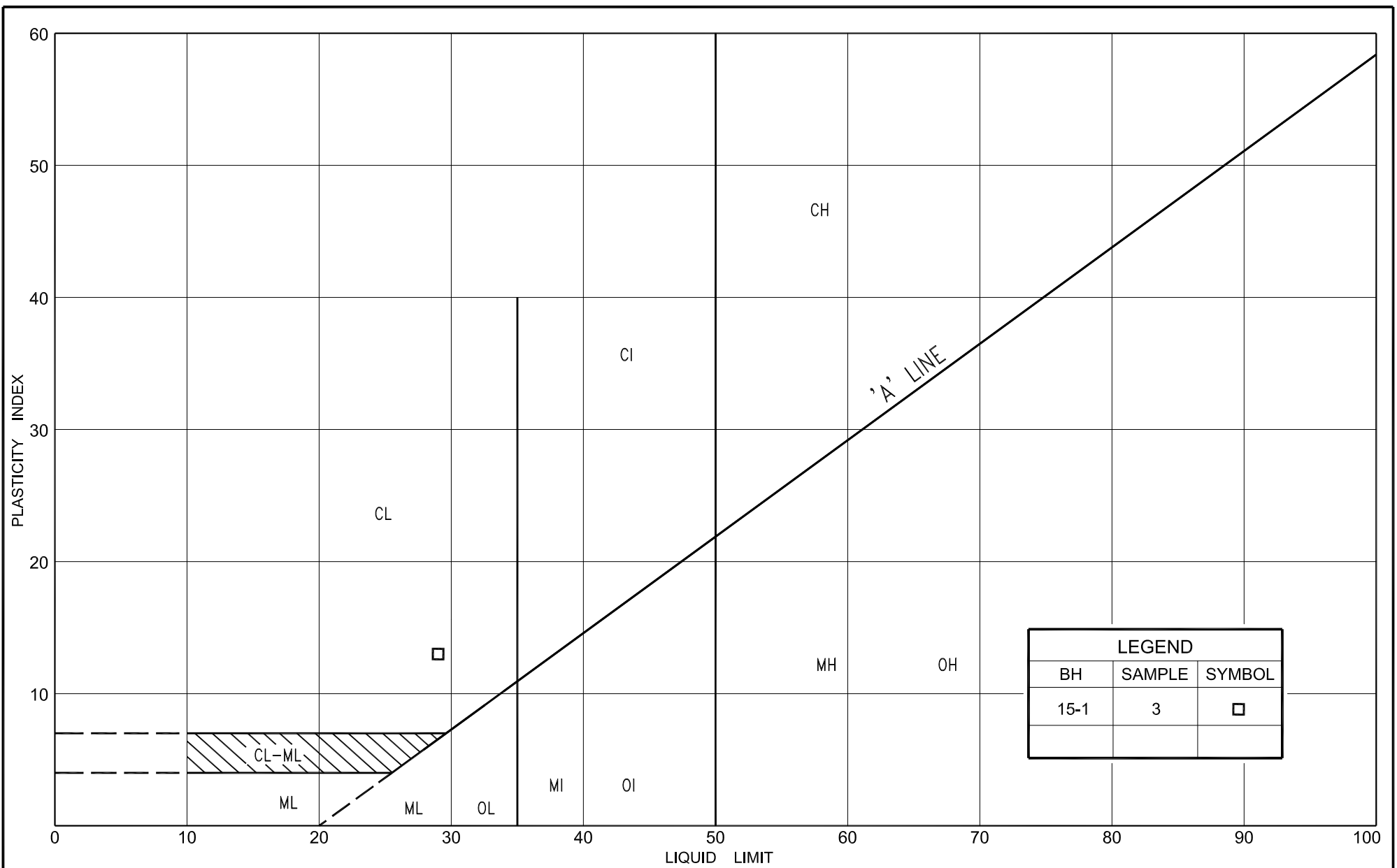


SILT & CLAY				FINE		MEDIUM		COARSE	GRAVEL		COBBLES	UNIFIED					
				SAND													
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL	COBBLES	M.I.T.		
	SILT																
CLAY			SILT			V. FINE		FINE		MED.		COARSE		GRAVEL		U.S. BUREAU	
						SAND											



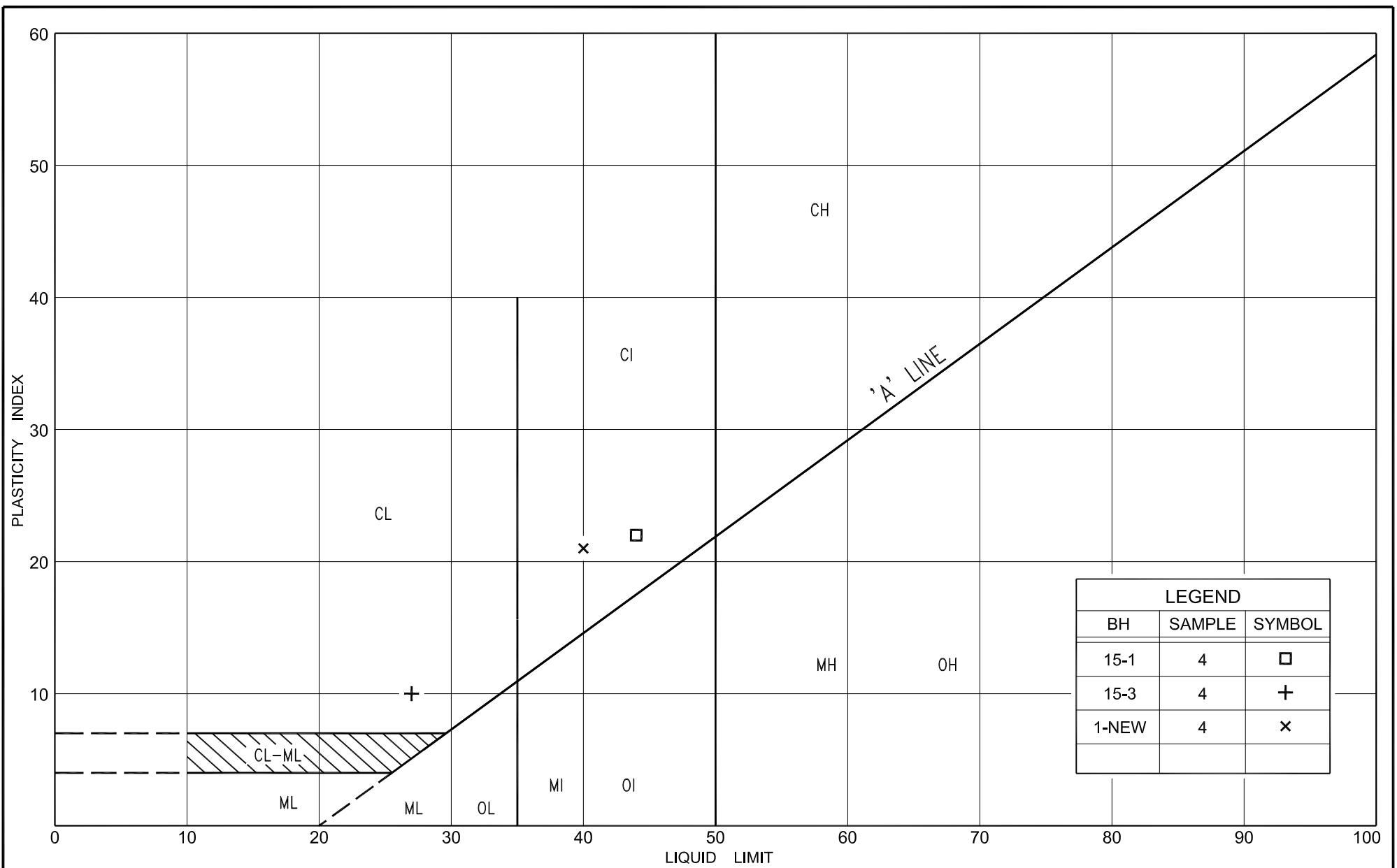
GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY
 trace to with sand, trace to with gravel (CL-CI)

FIG No. EBFD-GS-3
 HWY: 403 / QEW
 G.W.P. No. 2163-10-00



PLASTICITY CHART
 CLAYEY SILT, sandy, with gravel (CL)
 (FILL)

FIG No.	EBFD-PC-1
HWY:	403 / QEW
G.W.P. No.	2163-10-00



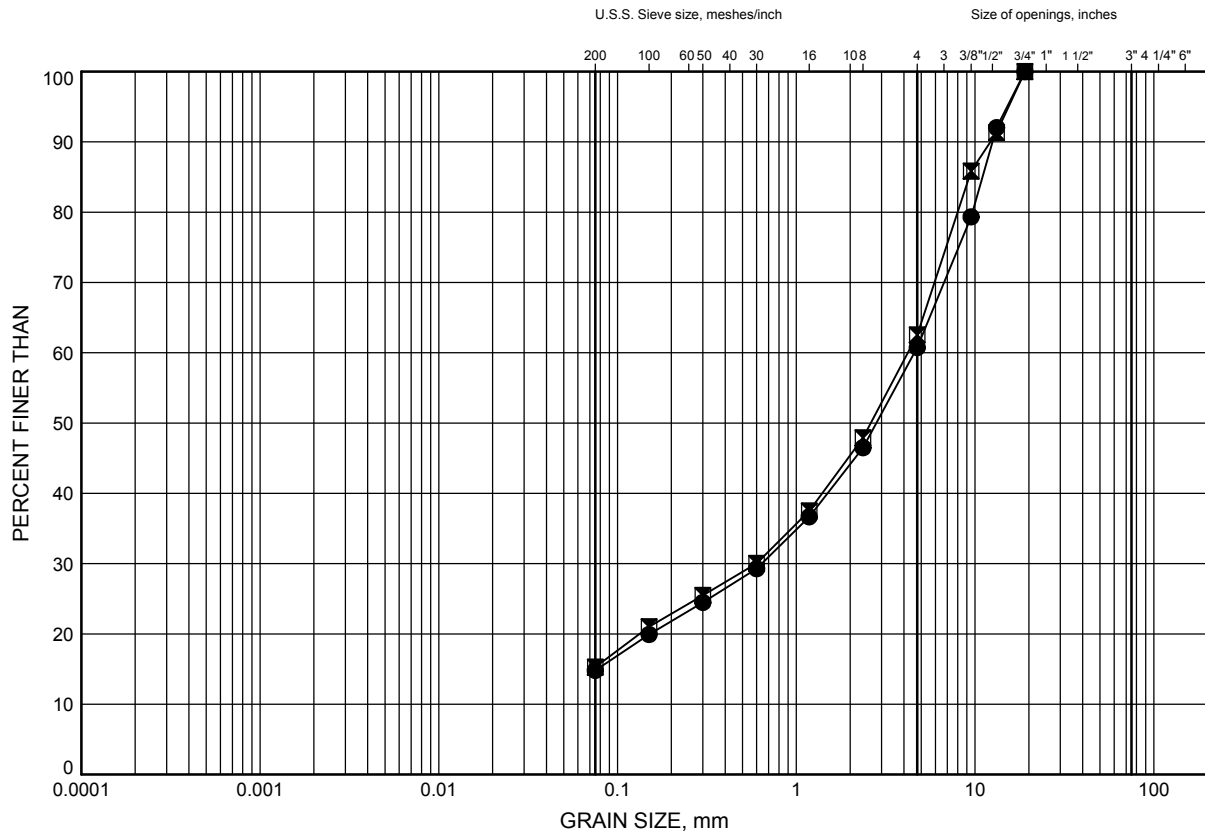
PLASTICITY CHART
 CLAYEY SILT TO SILTY CLAY
 trace to with sand, trace to with gravel (CL-CI)

FIG No. EBFD-PC-2
 HWY: 403 / QEW
 G.W.P. No. 2163-10-00

QEW and Hwy 403
GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND and GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-23	0.38	129.35
⊠	13-24	1.07	130.42

Date August 2013
W.P.

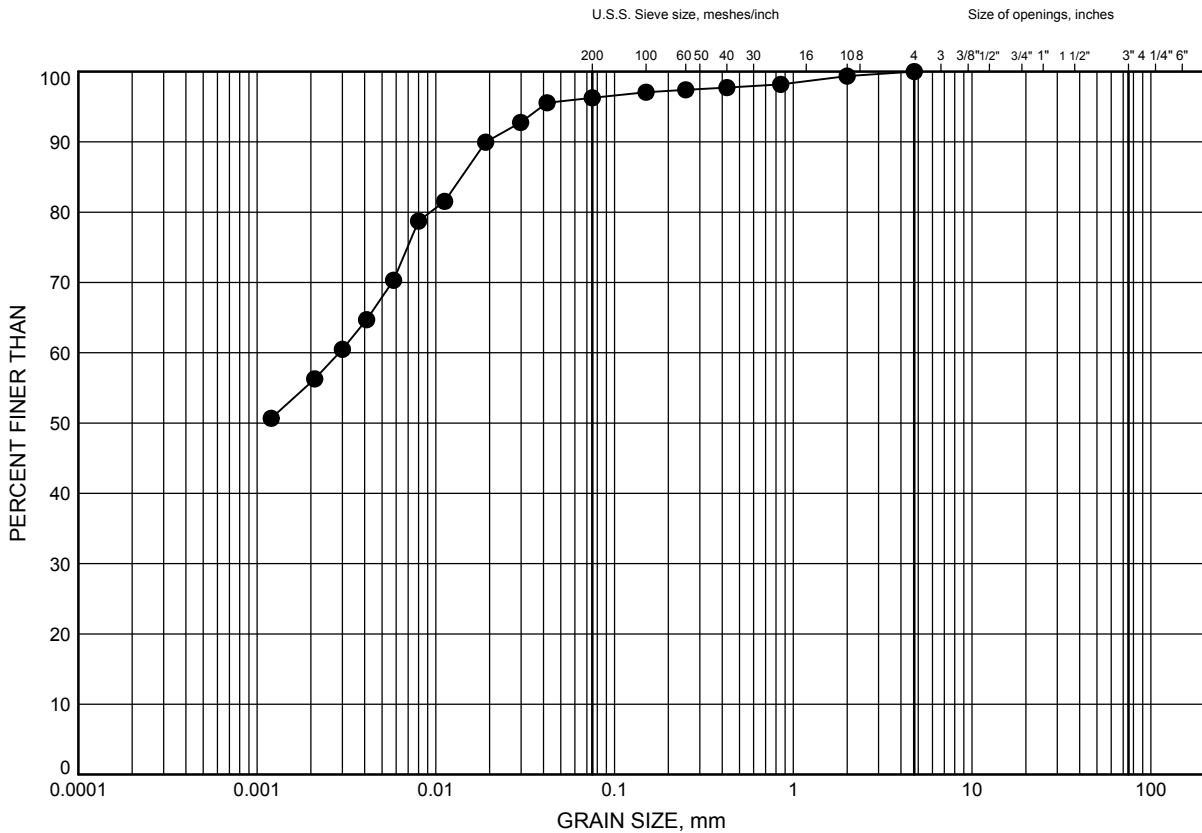


Prep'd SBP
Chkd.

QEW and Hwy 403
GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	13-23	2.59	127.14

Date August 2013
W.P.



Prep'd SBP
Chkd.



APPENDIX FIR – E

Table A – Rock Core Descriptions
Rock Core Photographs



TABLE A

ROCK CORE DESCRIPTIONS

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
15-1	7	5.8 ⁽¹⁾ – 7.3	78	31	5.8 – 8.8	SHALE WITH INTERBEDDED LIMESTONE: Grey to dark grey, occasional dark grey to purple shale, fine crystalline to aphanitic, with few stylitic partings, small chert nodules, low to medium strength, occasional fossils, bedding in shale horizontal, laminated and fissile in shale, slightly weathered to moderately weathered, close spaced flat partings, smooth to rough planar, tight, with dipping to vertical joints, poor to very poor quality.
	8	7.3 – 8.8	43	11		
15-3	7	4.9 ⁽²⁾ – 6.4	70	18	4.9 – 7.9	SHALE WITH INTERBEDDED LIMESTONE: Grey to dark grey, occasional dark grey shale, fine crystalline to aphanitic, with few stylitic partings, small chert nodules, medium strength, occasional fossils, bedding in shale horizontal, laminated and fissile, slightly weathered to unweathered, close spaced flat partings, smooth to rough planar, tight, with dipping to vertical joints, very poor to fair quality.
	8	6.4 – 7.9	100	67		

Notes:

Drilled: November 23 to 25, 2015

Logged: December , 2015

RQD = Rock Quality Designation

5.8⁽¹⁾, 4.9⁽²⁾ : Bedrock core starts at 5.8 m at BH15-1, 4.9 m at BH15-3

Originated: SA/JO/SAT
 Compiled: JO/SAT
 Checked: SS/KD



Photograph 1: Cores retrieved from borehole 15-1. Rock cores 7 and 8 from 5.8 to 8.8 m. RQD values ranged were 31% and 11% respectively, indicating poor to very poor rock quality.



Photograph 2: Cores retrieved from borehole 15-3. Rock cores 7 and 8 from 4.9 to 7.9 m. RQD values were 18% and 67% respectively, indicating very poor to fair rock quality.



**DRAFT
FOUNDATION DESIGN REPORT
for**

**EASTBOUND QUEEN ELIZABETH WAY OVERPASS AT FORD DRIVE
QUEEN ELIZABETH WAY AND HIGHWAY 403
TOWN OF OAKVILLE
REGIONAL MUNICIPALITY OF HALTON, ONTARIO
G.W.P. 2163-10-00, SITE NO. 10-286/1
CENTRAL REGION, ONTARIO**

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GEOCRES No.: Not Assigned
March 13, 2017



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Appendix FDR-A Comparison of Foundation Alternatives

Appendix FDR-B Standard Specifications and Non Standard Special Provision

PART B – DRAFT FOUNDATION DESIGN REPORT

for
Eastbound Queen Elizabeth Way Overpass at Ford Drive
GWP 2163-10-00, Site 10-286/1
Town of Oakville
Regional Municipality of Halton, Ontario

7. GENERAL

This report provides detail foundation design recommendations based on the interpretation of the geotechnical data presented in the factual report (Part A) to assist the design team in selection of a suitable type of foundation for the proposed replacement structure carrying the eastbound traffic lanes of the Queen Elizabeth Way over Ford Drive.

7.1 Statement of Limitations

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

8. PROJECT DESCRIPTION

8.1 General

The discussions and recommendations presented in this report are based on the preliminary General Arrangement drawing (GA) (ref. General Arrangement Drawing 'Pre-GA1.dwg') prepared by the Ministry of Transportation of Ontario in August 2016.



8.2 Existing Structure

The existing QEW passes over Ford Drive at approximate Station 23+387, QEW centreline chainage. The existing QEW eastbound overpass of Ford Drive is a single span 40 m long structure.

The existing road grade on the QEW at the bridge location varies between approximate elevation 130.4 and 131.9 at the south (construction west) and north (construction east) abutment respectively. The approach embankments are about 1 to 2.5 m high at the abutments.

8.3 Proposed Structure

Our understanding of the project, based on the GA drawing received from the design team dated August 2016, consists of:

- The proposed new and widened structure will carry the QEW eastbound traffic lanes over Ford Drive on the existing alignment as the existing structure.
- The proposed structure will consist of a single 40.0 m span, including retained soil system (RSS) walls and will carry four lanes of traffic including the proposed HOV lane and two fully paved shoulders in the eastbound direction over Ford Drive.
- The proposed construction west and east abutments will be located at approximate Station 23+367 and 23+407, respectively.
- The proposed road grade of the QEW eastbound lanes at the construction west and east abutments will be approximately 130.4 and 131.8 m, respectively.
- The road grade of Ford Drive will be at approximate elevation 123.5 m.

The proposed QEW westbound lane overpass at Ford Drive will be addressed under a separate cover.



9. FOUNDATION RECOMMENDATIONS

9.1 Existing Site Conditions

In summary, the soil stratigraphy generally comprised a surficial layer of asphalt overlying a layer of mixed non cohesive and cohesive fill. The fill is underlain by a deposit of native clayey silt to silty clay, which is underlain by highly weathered shale bedrock. Limestone interbedded with slightly weathered shale bedrock was encountered within the Meaford-Dundas Formation, underlying the highly weathered shale. No groundwater was observed in any of the boreholes during and upon completion of drilling. However, this could vary in the long-term due to seasonal fluctuations and rainfall patterns.

9.2 Frost Protection

All footings and/or caisson caps should be provided with a minimum of 1.2 m of earth cover or equivalent thermal insulation as protection against frost action. Equivalent thermal insulation can be provided by appropriate thickness of rigid polystyrene. Polystyrene insulation should meet the requirements of National Standards of Canada CAN/ULC-S701 (Standard for Thermal Insulation, Polystyrene, Boards and Pipe Covering). Appropriate thickness can be determined from manufacturer's literature.

9.3 Seismic Considerations

The reference Peak Ground Acceleration (PGA_{ref}) is 0.180 for the Town of Oakville, Ontario (National Building Code of Canada, 2015). Based on the subsurface conditions, the site classification for seismic design purposes is Type C in accordance with clause 4.4.3.2 of the Canadian Highway Bridge Design Code (CHBDC) 2014 Edition.

The liquefaction potential is not applicable to the bedrock foundation subgrade.



9.4 Foundation Alternatives

Considering the subsoil conditions and due to the shallow depth of overburden overlying bedrock, spread footing founded on the weathered shale bedrock are considered feasible to support the structural loads and are the recommended alternative for the foundations of the structure.

Consideration was given to the following feasible foundation alternatives discussed below and a comparison of the technical advantages and disadvantages for the replacement structure are presented in Appendix FDR-A.

Conventional or semi-integral abutments were considered to be feasible. Integral abutments are feasible, but not considered to be practical due to the deep rock excavation (in the order of 5 m below the stem level) that would be required to obtain adequate free pile lengths and minimum pile lengths for pile seating.

With space restrictions and associated requirements for temporary roadway protection for construction of spread footings adjacent to the travelled lanes of the QEW eastbound and westbound structures, the use of deep foundations (caissons) for the new bridge supports offers a feasible foundation alternative that minimizes the depth of excavations by maintaining the caisson cap level as high as possible.

The use of a driven pile foundation system is considered impractical due to the deep rock excavation required to obtain adequate 5 m free pile lengths through hard limestone bands.

9.5 Spread Footings on Bedrock

The existing ground surface along the QEW eastbound lane ranges from elevation 129.2 m to 131.5. The proposed bedrock surface level within the footprints of the foundation elements ranges from elevation 125.9 m to 126.3 m at the construction west abutment and from elevation 128.0 to 129.7 at the construction east abutment.



The upper zone of the shale bedrock below the QEW eastbound lane is highly weathered and of poor quality to approximate levels ranging from elevations 123.6 to 125.7 at the construction west abutment and from elevations 126.3 to 127.1 at the construction east abutment. Below these levels, the bedrock becomes slightly weathered with interbedded limestone and of fair to good quality. These levels are summarized in Table 9.5 for each of the boreholes, together with the levels to provide the 1.2 m foundation frost protection for the spread footings.

Table 9.5: Proposed Overpass Foundations – QEW EBL over Ford Drive

Overpass Component	Element	BH No.	Current Surface Elevation	Proposed Footing Subgrade Elevation from GA Drawings	Highly Weathered Shale Bedrock	Slightly Weathered Shale Bedrock / Limestone Interbeds	Minimum Level for Frost Protection (1.2 m Below Ground)
QEW EBL Construction West Abutment	Abutment	15-1	129.5	124.2	125.9	123.7	128.3
	Abutment	13-23	129.7	124.2	126.1	123.6	128.5
	Abutment	3	128.6	124.2	126.3	125.7	127.4
QEW EBL Construction East Abutment	Abutment	15-3	131.2	124.2	128.6	126.3	130.0
	Abutment	13-24	131.5	124.2	129.7	126.7	130.3
	Abutment	4	130.2	124.2	128.0	127.1	129.0

9.5.1 Recommended Spread Footings Founding Levels

The proposed footing subgrade elevations from the GA Drawing are considered feasible at the foundation element locations. The footing elevation along the construction west abutment will border the slightly weathered shale bedrock / limestone interbeds. While the east abutment footing elevation will penetrate approximately 2.5 m into the slightly weathered shale bedrock / limestone interbeds. Refer to Section 12 of this report for details pertaining to excavation considerations for the project.

In view of natural variations in the level of the slightly weathered shale bedrock, the levels indicated should be considered approximate, with a probable variation of plus or minus 0.5 m.



9.5.2 Recommended Geotechnical Bearing Resistances

The values in the following Table 9.5.2 for factored geotechnical resistance at ULS and geotechnical reaction at SLS should be used for design of the spread footings for the structure foundation elements:

Table 9.5.2: Recommended Geotechnical Resistances

Founding Stratum	Geotechnical Resistance (kPa)	
	Factored ULS	SLS at 25 mm Settlement
Highly Weathered Shale Bedrock	800	600
Slightly Weathered Shale Bedrock / Limestone Interbeds	1000	>1000

It is noteworthy that the upper highly weathered shale will be excavated to reach the founding levels. As a consequence, seams or layers of clayey silt/silty clay within the highly weathered zones of the bedrock will be removed and only negligible differential settlements are expected to occur after construction.

The shale bedrock is prone to weathering when exposed to the elements. To prevent further weathering of the shale at the founding levels, a lean concrete (minimum 20 MPa) working slab with a minimum thickness of 100 mm should be placed on all bearing surfaces as soon as possible but within 4 hours after the bearing surfaces have been exposed to prevent further weathering, softening and deterioration.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.4 of the 2014 CHBDC.

9.5.3 Geotechnical Sliding Resistance

There are three planes to consider when computing horizontal sliding resistance. These include concrete to concrete at the underside of the footing, concrete to shale and within the shale to shale interbeds. The following Table 9.5.3 provides unfactored friction coefficients for each plane.



Table 9.5.3: Geotechnical Sliding Resistance

Resistance Planes	Friction Coefficient
Concrete to Concrete	0.75
Concrete to Shale	0.45
Shale to Shale (Interbeds)	0.30

Construction of the footings should be performed and monitored in accordance with OPSS 902 to verify the competency of the founding surface.

Where required, dowels and/or rock anchors into the bedrock may be used to augment the resistance of the structure to sliding as discussed in the following section of this report.

9.5.4 Uplift and Lateral Resistance

Grouted dowels or rock anchors installed into the bedrock may be used to resist the uplift force induced by wind loads and lateral forces. A design rock to grout bond stress of 200 kPa may be assumed. This bond stress is based on the highly weathered shale conditions in the boreholes. Higher bond stress, up to 700 kPa, is possible in sound shale / limestone interbeds, however they will require further rock core investigation at the specific anchor location selected. A minimum depth of embedment of 40 times the bar diameter should be used for design.

The failure plane may be assumed at an inclination of 45° to the horizontal from the bottom of the anchor for a single anchor. In the case of a group of rock anchors, a truncated cone should be assumed. The shear strength of bedrock of 250 kPa may be used to determine the adequacy of the anchor depth.

To compute the horizontal resistance provided by the dowels or rock anchors, a shear strength of 250 kPa should also be considered. The anchors should be installed no closer than 5 times the bar diameter.

Dowels should be installed in accordance with an NSSP for Dowels into Rock in Appendix FDR-B.



Rock anchors should be installed and tested in accordance with OPSS 942. It is noted that at least one full scale pull-out test should be carried out on an anchor, in accordance with the current post-tensioning practice manual.

9.6 Caisson Foundations

Resistance values for caissons of selected diameters and socket lengths embedded into the slightly weathered shale bedrock/limestone interbeds account for resistance being developed by both end bearing and shaft adhesion are tabulated in Table 9.6 below. Refer to Table 9.5 of this report for elevations of the bedrock surface at the foundation elements.

Table 9.6: Caisson Axial Geotechnical Resistances

CAISSON DIAMETER (m)	3.0 m LENGTH (ELEVATION 121.2 FOR BOTH CONSTRUCTION EAST AND WEST ABUTMENTS)		4.5 m LENGTH (ELEVATION 119.7 FOR BOTH CONSTRUCTION EAST AND WEST ABUTMENTS)	
	FACTORED ULS (kN)	SLS (kN)	FACTORED ULS (kN)	SLS (kN)
0.9	3,500	>3,500	4,200	>4,200
1.2	4,000	>4,000	4,800	>4,800
1.5	4,500	>4,500	5,500	>5,500

It is noted that the SLS values for 10 mm of settlement will be greater than the factored ULS values, therefore the ULS conditions will govern the design.

A temporary liner will be required to support the sides of the drilled holes during construction.

The shale bedrock contains limestone interbeds within its matrix that are significantly harder/stronger than the shale. These hard rock obstructions may pose difficulties during the advancing of caissons/liners. Where encountered, these interbeds may require significant effort to penetrate, depending on their thicknesses and the techniques employed by the contractor.



9.7 Construction Considerations

The "red flag" issues outlined in the preceding paragraphs and the recommended methods of overcoming these issues noted in the following sections of the report are intended to alert and aid the designer and the contractor. These comments and recommendations are based on the conditions revealed during the investigations and no responsibility is assumed by the consultants or the MTO for alerting the contractor to all critical issues for each foundation alternative. The requirements to deliver acceptable construction quality remain the responsibility of the contractor.

A list of the Ontario Provincial Standard documents relevant to this report is provided in Appendix FDR-B.

10. ABUTMENT, WING WALLS AND RETAINING WALLS

The abutment and wing walls 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 pressures presented in Section 6.12 of the CHBDC 2014 or employing the following equation assuming a triangular pressure distribution.

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

where K = coefficient of lateral earth pressure (dimensionless)

γ = unit weight of free-draining granular material, kN/m³

h = depth below final grade, m

q = surcharge load, kPa, if present

C_p = compaction pressure, kPa (refer to clause 6.12.3 of CHBDC 2014)

C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.5 of CHBDC 2014)

where \emptyset = angle of internal friction of retained soil (35° for Granular A or 30° for B Type II)

δ = angle of friction between the soil and wall (23.5° for Granular A or B Type II)

The seismic site coefficient for the conditions at this site was provided in Section 9.3.



Hydrostatic pressures were not included in the equation since free-draining granular material or rockfill will be used as backfill behind the wall. The following Table 10 provides parameters that are recommended for design.

Table 10: Earth Pressure Parameters

PARAMETER	GRANULAR A	GRANULAR B TYPE II	WEATHERED BEDROCK
Angle of Internal Friction, degrees	35°	30°	30°
Unit weight, kN/m ³	22.5	21.5	20.0
Coefficient of Active Earth Pressure, K_a	0.27	0.33	0.33
Coefficient of Earth Pressure At Rest, K_o	0.43	0.50	0.50
Coefficient Passive Earth Pressure, K_p	3.69	3.00	3.00

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).

The magnitude of the passive resistance and active pressure is dependent on the actual lateral movement of the structure toward and away from the adjacent soil, respectively. We refer to Figure C6.16 of the CHBDC for these computations. The backfill should be considered as medium dense sand for this project.

A weeping tile system (OPSS 405 and OPSD 3190.100) and/or weep holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet.



Backfilling adjacent to retaining structures should be carried out in conformance with OPSD 3101.150 for granular backfill at abutments.

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.12.3 of the CHBDC. Refer to OPSS.PROV 501 for additional information in this regard.

RSS walls can be used for retaining walls as required. High appearance, high performance RSS walls are recommended. Refer to Appendix FDR-B for an NSSP for design and construction of RSS systems.

11. APPROACH EMBANKMENTS

The approach embankments to the proposed structure will be approximately 2.5 m high at the abutments. Away from the abutments, the fill will be approximately 1.5-2.0 m high over original ground.

Construction of the embankment fill remote from the abutments on the native clayey silt to silty clay deposits is considered to be feasible. It is recommended that the existing fill materials at the abutment locations and along the existing alignment of the QEW EBL approach fill within 20 m of the abutments be excavated down to native clayey silt to silty clay (locally up to 2.2 m) prior to placement of the new embankment fill. This zone of fill would have to be removed and may be reused if approved by the geotechnical engineer upon inspection to meet the final grade of the QEW eastbound lanes.

Embankment fill should be placed and compacted in accordance with OPSS.PROV 206 and OPSS.PROV 501. New embankment fill placed against existing embankment slopes or on a sloping ground surface should be benched into the existing slope in accordance with OPSD 208.010. The magnitude of the fill surface settlement by self-compression that would occur during and after construction of the embankment depends on the type of fill and the height of the material placed but is expected to be in the order of 0.5 to 1% of fill height. It is recommended that paving should be delayed for as long as possible after placement of the fill but for a minimum



period of 1 month to permit some of the settlement within the fill and in the underlying ground to occur. The area immediately behind abutments should be preloaded for a minimum of 1 month to minimize post construction settlements.

Maximum settlements of the approach fill as a result of consolidation of the underlying cohesive deposit induced by the low embankment loads are estimated to be less than 25 mm. To minimize differential settlement due to self-settlement of the fill, the use of granular fill could be considered since the majority of their settlements will occur during construction.

Provided that the new fill embankments are constructed as recommended in this report, these fill slopes will be stable at an inclination of 2H:1V. Rock cuts into the Queenston Shale Formation and Meaford-Dundas Formation should be sloped at 3H:1V in view of susceptibility of the rock to weathering. The earth fill and rock cut slopes should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 803 and OPSS.PROV 804 for time constraints and the type of seed and mulch required.

12. EXCAVATION CONSIDERATIONS

According to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria, the existing overburden fill and native clayey silt to silty clay soils are classified as Type 3 soils. The bedrock is classified as Type 1 soil. Since open cut procedures are governed by soils with the highest soil type number, the requirements for Type 3 soils govern and temporary cut slopes over the full depth of excavation inclined at 1H:1V should be provided assuming adequate drainage measures are in place. Cobbles and boulders should be expected in the excavations.

The equipment required and method of excavation within the bedrock will be dependent upon the geometry of the cut and relative depth of excavation into the bedrock. Although the method of excavation should remain the responsibility of the Contractor, as noted previously, the shale bedrock is highly to slightly weathered and subject to adequate groundwater control, excavations of the upper highly weathered shale should be possible with conventional excavation techniques for shale bedrock. More advanced excavation techniques and equipment such as a hoe ram, jack hammer or equivalents should be considered by the Contractor and may be required to penetrate



relatively harder zones (limestone interbeds) within the bedrock. Progressively more difficult conditions should be anticipated with increasing depth of excavation. All excavations should be conducted in accordance with OPSS 902. Weathered shale fragments found in the native clayey silt to silty clay materials similar in size to cobbles and boulders may be encountered during excavation.

12.1 Roadway Protection Systems

It is understood that roadway protection will be required to keep the new embankments adjacent to the new bridge off the existing highway and away from the existing overpass. This roadway protection will be installed as part of the bridge backfilling operation.

In addition, depending on the required depth of excavation, a roadway protection system may be necessary along the existing QEW EB structure, lanes and/or embankments. The roadway protection system is required where excavation geometry is steeper than 1H:1V. The roadway protection system should be designed according to OPSS.PROV 539. It is recommended that a minimum performance level 2 be implemented to prevent excessive lateral movement of the adjacent existing embankment during construction.

Although a roadway protection scheme consisting of soldier piles and lagging, anchored as required, could be considered, the Contractor should be responsible for the selection, detailed design and performance of the roadway protection scheme. OPSS.PROV 539 also calls for monitoring of the roadway protection system by the Contractor to check the horizontal and vertical displacements of the roadway surface during construction.

13. GROUNDWATER CONTROL

Groundwater was not observed in any of the boreholes during or upon completion of drilling. However, the Contractor should be responsible for installing a dewatering system to lower the groundwater a minimum of 0.5 m below the base of excavations for construction in the dry, should seepage from perched water deposits, soil/bedrock fissures or surface water run-off enter the excavations. Subject to the groundwater level at the time of construction, consideration could be given to employing sump pumps to control groundwater seepage into the open excavations.



If caissons are selected, sump pumping may be adequate or tremie techniques may be required for installation in the wet. Refer to the related NSSP for dewatering in Appendix FDR-B.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.



14. CLOSURE

This Foundation Design Report was prepared by Mr. K. R. Daly, P.Eng, and reviewed by Mr. G.O. Degil, PhD, P.Eng., Senior Geotechnical Engineer. 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.

A handwritten signature in black ink, appearing to read 'K. R. Daly'.

Kyle R. Daly, P.Eng.
Project Engineer, Geotechnical Services

A handwritten signature in black ink, appearing to read 'G. O. Degil'.

Grigory O. Degil, PhD, P.Eng.
Senior Engineer, Geotechnical Services

A handwritten signature in black ink, appearing to read 'C. M. P. Nascimento'.

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

NOTE: The Final Report will be signed and stamped by two Professional Engineers licensed by PEO, one of whom shall be the Designated Principal Contact for MTO foundation projects

KD/CN/GD:nk

APPENDIX FDR – A

Comparison of Foundation Alternatives



Comparison of Foundation Alternatives

Option 1: Footings on Bedrock	Option 2: Caissons/Drilled Shaft	Option 3: H-Piles in Pre-Augured Holes	Option 4: Driven H-Piles
Advantages: <ol style="list-style-type: none"> 1. Ease of construction using conventional excavation and construction techniques 2. Lower cost compared to deep foundation 3. High geotechnical resistance available at shallow depth 	Advantages: <ol style="list-style-type: none"> 1. High geotechnical resistance available for caissons placed on sound bedrock 2. Less number of deep foundation elements compared to H-piles 3. Permits installation through winter weather 4. Requires less excavation to construct abutments 	Advantages: <ol style="list-style-type: none"> 1. High geotechnical resistance is available for piles founded in slightly weathered bedrock 2. Requires less excavation compared to footing 3. Allows for design of integral abutments 4. Piles can be lowered to predetermined depth 5. Permits installation of piles through winter weather 	Advantages: <ol style="list-style-type: none"> 1. High geotechnical resistance available for piles driven to slightly weathered bedrock 2. Allows for design of integral abutments 3. Requires less excavation compared to footings 4. Permits installation through winter weather 5. Requires less excavation to construct abutments 6. Limited groundwater control
Disadvantages: <ol style="list-style-type: none"> 1. Significant depth of excavation below the existing ground level 2. Excavation may require temporary shoring 3. Dewatering may be required depending on the surface drainage conditions and depth of excavation 	Disadvantages: <ol style="list-style-type: none"> 4. Preclude use of integral abutments 5. Temporary liners may be required to the slightly weathered bedrock level 6. Difficulty in cleaning and inspection of base under water 7. Complex excavations may be encountered to excavate limestone interbeds 	Disadvantages: <ol style="list-style-type: none"> 8. Temporary liners may be required to keep the fill from collapsing 9. Higher unit cost compared to footings 10. May require auguring equipment and piling rig for tapping the piles at the founding level 11. Complex excavations may be encountered to excavate limestone interbeds 	Disadvantages: <ol style="list-style-type: none"> 12. Considering the nature of bedrock, individual piles within the group may encounter refusal at varying elevations 13. Higher unit cost compared to footings 14. Minimum length of pile required for integral abutments may not be feasible
Recommended	Feasible but not Recommended	Feasible but not Recommended	Not Recommended

APPENDIX FDR – B

Standard Specifications and Non Standard Special Provision

LIST OF STANDARD SPECIFICATIONS RELEVANT TO THE REPORT

DOCUMENT	TITLE
OPSS 405	Construction Specification for Pipe Subdrains
OPSS 803	Construction specification for Sodding
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 942	Construction Specification for Prestressed Soil and Rock Anchors
OPSS.PROV 206	Construction Specifications for Clearing, Close Cut Clearing, Grubbing, and Removal of Surface and Piled Boulders
OPSS.PROV 501	Construction Specifications for Compaction
OPSS.PROV 539	Construction Specifications for Temporary Protection Systems
OPSD 208.010	Benching of Earth Slopes
OPSD 3101.150	Minimum Granular Backfill Requirements - Abutments
OPSD 3190.100	Retaining Wall and Abutment Wall Drain Detail

NSSP for Rock Excavation - Addition to OPSS 902

The Contractor shall be advised that the equipment required and method of excavation within the bedrock will be dependent upon the geometry of the excavation and the depth of excavation into the bedrock. Although the method of excavation should remain the responsibility of the Contractor, the shale bedrock is slight to highly weathered and subject to adequate groundwater control, excavations of the upper highly weathered shale should be possible with conventional excavation techniques for shale bedrock. A hoe ram or jack hammer may be required to penetrate relatively harder zones (limestone bands) within the shale. Progressively more difficult conditions should be anticipated with increasing depth of excavation.

NSSP for Working Slab and Exposure Hours – Addition to OPSS.PROV 206

The Contractor shall be advised that a lean concrete (minimum 20 MPa) working slab a minimum 100 mm in thickness should be placed on all bearing surfaces as soon as possible but not later than 4 hours after the bearing surfaces were exposed to prevent further weathering, softening and deterioration of shale bedrock at the founding levels.

NSSP for Dewatering - Addition to OPSS 902

The Contractor shall take measures to lower the prevailing groundwater level a minimum of 0.5 m below the base of excavations or foundation bases for construction in-the-dry.



DOWELS INTO ROCK - Item No.

Non Standard Special Provision

CONSTRUCTION SPECIFICATION FOR THE SUPPLY, INSTALLATION AND TESTING OF DOWELS INTO ROCK FOR 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.



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

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)
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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 Administrator a 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 abut 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

TABLE 1 – PERFORMANCE TOLERANCES FOR RSS		
Performance Requirement	Performance Tolerance (mm)	
	Local	Global
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.