



DRAFT
FOUNDATION INVESTIGATION AND DESIGN REPORT
for
HIGHWAY 403 W-N RAMP UNDER QUEEN ELIZABETH WAY WB
WP 2163-10-00, SITE NO. 10-284/1
CENTRAL REGION, ONTARIO

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GEOCRES No.: Not assigned
May 22, 2015



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**DRAFT
FOUNDATION INVESTIGATION REPORT**

for
Highway 403 W-N Ramp under Queen Elizabeth Way WB
WP 2163-10-00, Site No. 10-284/1
Central Region, Ontario

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the replacement of the existing Queen Elizabeth Way (QEW) WB bridge over the Highway 403 W-N ramp (Site 10-284/1). The study was carried out by Peto MacCallum Ltd. (PML) for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The existing bridge is a three-span bridge over the Highway 403 W-N ramp in the Town of Oakville, Regional Municipality of Halton.

This report provides subsurface information pertaining to the foundation of the proposed new bridge and approaches within about 20 m of the abutments.

A preliminary foundation investigation and design report for the replacement of the Highway 403 W-N Ramp under QEW WB bridge was completed in April 2013 by Thurber Engineering Ltd. Information from that study is documented in GEOCREs No. 30M5-285. The foundation investigation and design report completed for the existing bridge in 1978 are documented in GEOCREs No. 30M5-117, with copies of the relevant boreholes provided in Appendix FIR-A for reference.

All elevations in this report are expressed in meters.

2. SITE DESCRIPTION AND GEOLOGY

The QEW WB bridge site is located approximately 275 m north of the existing Ford Drive underpass bridge at QEW. The new Highway 403 W-N Ramp under QEW WB bridge will be located just north of the existing bridge. Drawing WB-1 shows the general alignment of the new bridge and the site.



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 till deposits overlying shale bedrock of the Queenston and Dundas Formations. (L.J. Chapman and D.F. Putnam, *The Physiography of Southern Ontario*, 3rd Edition, 1984).

Lands within the QEW/Highway 403 corridor near the project site are generally vacant, grass covered and have a topography that is gently sloping down to the south. Highway 403 W-N ramp at the site is within a cut, approximately 5 to 7 m below the grades of QEW.

Outside of the highway right of ways, the land use is comprised primarily of 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.

3. INVESTIGATION PROCEDURES

A review of available geotechnical investigations was carried out and relevant subsurface information from these investigations was included in this report. In particular, four boreholes (boreholes 1 to 4) advanced as part of the previous investigation for the existing bridge (GEOCRES No. 30M5-117) were included in this investigation and supplemented by five additional boreholes.

The following reports, including drawings, were available for the proposed bridge.

1. Preliminary Foundation Investigation and Design Report
Highway 403 and QEW Widening
QEW from Trafalgar Road Easterly to East of Winston Churchill Blvd.
And
Highway 403 from QEW Northerly to Highway 407 and Winston Churchill Blvd.
Oakville and Mississauga, Ontario
WP 09-20007
(GEOCRES No. 30M5-285)
(by Thurber Engineering Ltd. dated April 2013)



2. Foundation Investigation and Design Report
W-N Ramp Hwy 403 under QEW
WP 159-75-06, Site 10-284
QEW, District 4, Hamilton
(GEOCREST No. 30M5-117)
(by MTO dated February 1978)

Five supplemental boreholes (11 to 15) were advanced during the period of January 20 to 23, 2015. The borehole locations are shown on Drawing WB-1, attached.

The ground surface elevations for boreholes 11 to 15 were established by Callon Dietz Incorporated.

Boreholes 11 to 15 were advanced using continuous flight hollow stem augers powered by a track mounted D-120 drill rig. Bedrock core samples (3.1 m) were taken in boreholes 13 and 14, using HQ diamond rock coring equipment supplemented by wash boring techniques. All equipment was supplied and operated by a specialist drilling contractor, working under the full-time supervision of a PML field supervisor.

Photographs of the rock cores obtained from boreholes 13 and 14 are shown in Appendix FIR-B.

Representative samples of the soils encountered in the boreholes were recovered at 0.75 and 1.5 m depth intervals. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. Soils were identified in accordance with the MTO soil classification manual procedures. Observations of auger grinding were recorded and included larger particle sizes such as cobbles, boulders and/or bedrock fragments.

The boreholes were backfilled in accordance with the MTO guidelines and MOE regulation 903 for borehole abandonment procedures using a bentonite/cement mixture grout.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open boreholes.



Soils were identified in the field in accordance with the MTO Soil Classification procedures. Recovered soil samples were returned to our laboratory for detailed visual examination and soil classification. The laboratory test program comprised the following tests:

- Natural moisture content determinations (14)
- Atterberg Limits (6)
- Grain size analyses (6)

4. SUMMARIZED SUBSURFACE CONDITIONS

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

The results of the laboratory natural moisture content determinations, grain size analyses, and Atterberg limits are shown on the Record of Borehole sheets. The Atterberg limits and grain size distribution charts are presented in Figures WB-PC-1 and WB-GS-1, respectively.

Two rock core specimens were taken from boreholes 13 and 14 and the results are summarized in section 4.3 of this report and shown on the Record of Borehole sheets.

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

The subsurface stratigraphy encountered in the boreholes comprised surficial fills and native clayey silt to silty clay deposits overlying shallow weathered shale bedrock. The depth of soil cover revealed in the boreholes varied from 1.5 to 3.0 m, elevations ranging from 142.7 to 146.6. Groundwater was encountered during drilling at one borehole location.



With exception to the surficial pavement structures and fills encountered in the five supplemental boreholes the subsurface conditions in the supplemental boreholes were consistent with the conditions encountered in the boreholes completed previously at the site.

The strata encountered are summarized below.

4.1 Fill

A 0.3 to 2.0 m thick fill unit was encountered surficially in boreholes 11 to 15 extending to elevations 143.7 to 147.6. The fill typically comprised clayey silt with exception to borehole 12 where the fill comprised sandy silt with gravel. Trace inclusions of organic matter were noted within the fill. Locally in borehole 11, the fill contained construction debris (such as asphalt, gravel). N values within the fill ranged from 9 to 44. Higher N values are indicative of the presence of cobbles. Moisture contents within the fill ranged from 9 to 22%.

4.2 Clayey Silt to Silty Clay

This stratum was encountered in the previous boreholes as well as the supplemental boreholes.

A 2.1 to 2.4 m thick native silty clay deposit was contacted surficially in boreholes 1 to 4 and extended to elevations 143.2 to 145.4. Boreholes 11 to 15 encountered a 1.0 to 1.5 m clayey silt to silty clay deposit beneath the fill at 0.3 to 2.0 m (elevations 143.7 to 147.6) and extended to 1.5 to 3.0 m (elevations 142.7 to 146.6). N values within the clayey silt to silty clay deposit ranged from 11 to 38 (typically 15 to 30) indicating very stiff consistency. The results of Atterberg limits testing and grain size distribution analyses conducted on six samples of the clayey silt to silty clay deposit are presented in respective Figures WB-PC-1 and WB-GS-1. The clayey silt to silty clay had a liquid limit that ranged between 27 and 38, a plastic limit that ranged between 17 and 20 and a corresponding plasticity index ranging between 10 and 18. The moisture content of the clayey silt to silty clay ranged between 11 to 16%, typically below the plastic limit indicating an over consolidated cohesive deposit of low to medium plasticity.

4.3 Bedrock

The presence and quality of the bedrock underlying the site was verified by extracting HQ-size cores from the rock mass in boreholes 13 and 14 at 7.6 and 7.3 m, some 5.8 m below the Clay/bedrock interface. The bedrock surface was inferred by auger and/or split-spoon sampler refusal in boreholes 11 to 15 at 1.5 to 3.0 m, elevation 142.7 to 146.6. Boreholes 11, 12 and 15 were terminated in the bedrock upon meeting split spoon sampler refusal at 1.7 to 10.2 m, elevation 144.4 to 133.8 below the clay/bedrock interface.

The bedrock is slightly to highly weathered and is generally red to greyish red in colour with interbedded greenish grey limestone. Seams / layers of clayey silt to silty clay were also noted within the highly weathered zones of the bedrock.

Overall, the measured core recovery varied between 83 and 100%. The Rock Quality Designation (RQD) determined from the rock cores was typically in the range of 25 to 78% (locally 100%) indicating poor to fair (locally excellent) quality rock.

A detailed description of the rock cores retrieved from boreholes 13 and 14 at the west and east abutments is provided in Table 1 and summarized on the Record of Borehole Sheets. Photographs of the rock cores are shown in Appendix FIR-B.

Bedrock cores obtained from boreholes 2 and 3 had measured core recovery typically between 80 and 100% with two isolated values of 41 and 70% in boreholes 2 and 3, respectively. The RQD values typically ranged between 29 and 75% (locally 13%) indicating poor (very poor locally) to fair quality rock.

At the west abutment and approach, the bedrock/probable bedrock surface was encountered at 1.8 to 3.0 m, elevations ranging from 142.7 to 144.3 in boreholes 11, 12, 13, 3 and 4. Bedrock core samples were obtained from borehole 13 from 7.6 to 10.7 m, elevations 138.5 to 135.4 and from borehole 3 from 3.0 to 12.9 m, elevations 142.9 to 133.0.



At the west abutment, the measured core recovery from boreholes 13 and 3 ranged from 70 to 100%. The RQD determined from the rock cores was typically in the range of 25 to 78%, indicating improving rock quality with depth, ranging from poor to fair quality rock. The bedrock surface elevations indicate a maximum bedrock surface relief of 1.3 m between borehole locations. The slope of the bedrock surface ranges from less than 1 to 3°.

At the east abutment and approach, the bedrock/probable bedrock surface was encountered at 1.5 to 2.1 m, elevations ranging from 144.8 to 146.6 in boreholes 14, 15, 1 and 2. Bedrock core samples were obtained from borehole 14 from 7.3 to 10.4 m, elevations 140.6 to 137.5 and from borehole 2 from 2.7 to 12.8 m, elevations 144.3 to 134.2.

At the east abutment, the measured core recovery from boreholes 14 and 2 ranged from 80 to 100% with one isolated value of 41% within the upper 2.0 to 3.0 m of borehole 2. The RQD determined from the rock cores was typically in the range of 29 to 75% indicating improving rock quality with depth, ranging from poor to fair quality rock. One isolated RQD value of 13% in borehole 2 indicates locally very poor quality rock. The bedrock surface elevations indicate a maximum bedrock surface relief of 1.2 m between borehole locations. The slope of the bedrock surface ranges from less than 1 to 2°.

4.4 Groundwater

Groundwater was observed during augering in borehole 12 at 10.7 m, elevation 135.4.

Boreholes 11, 13 to 15 and 1 to 4 (Geocres No.: 30M5-117) were dry during and after augering.

The groundwater is subject to fluctuations at the site due to seasonal conditions and rainfall patterns.




5. CLOSURE


The field work was carried out under the supervision of Mr. S. Aziz and direction of Mr. K.R. Daly, B.Eng., EIT. The equipment was supplied by Altech Ltd Drilling and Investigative Services. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.


This Foundation Investigation Report was prepared by Mr. R. Agahzadeh, P.Eng. and reviewed by Mr. David Dundas, 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.


for Romin Agahzadeh, P.Eng.
Senior Engineer, Geotechnical Services


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


C. M. P. Nascimento, P.Eng.,
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



TABLE 1
 ROCK CORE DESCRIPTIONS

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
13	1	7.6 – 8.2	100	100	7.6 – 10.7	SHALE WITH INTERBEDDED LIMESTONE: Light brown to greenish grey, fine grained, occasional interbedded grey limestone (effervesces freely in dilute (5%) hydrochloric acid), soft to medium strength, bedding in shale horizontal, laminated and fissile, slightly weathered to highly weathered, close spaced flat partings, smooth planar, tight, poor to excellent quality.
	2	8.2 – 9.1	100	25		
	3	9.1 – 10.7	97	78		
14	1	7.3 – 7.6	83	41	7.3 – 10.4	SHALE WITH INTERBEDDED LIMESTONE: Light brown to greenish grey, fine grained, occasional interbedded grey limestone (effervesces freely in dilute (5%) hydrochloric acid), soft to medium strength, bedding in shale horizontal, laminated and fissile, slightly weathered to highly weathered, close spaced flat partings, smooth planar, tight, poor to fair quality.
	2	7.6 – 8.8	100	54		
	3	8.8 – 10.4	100	60		

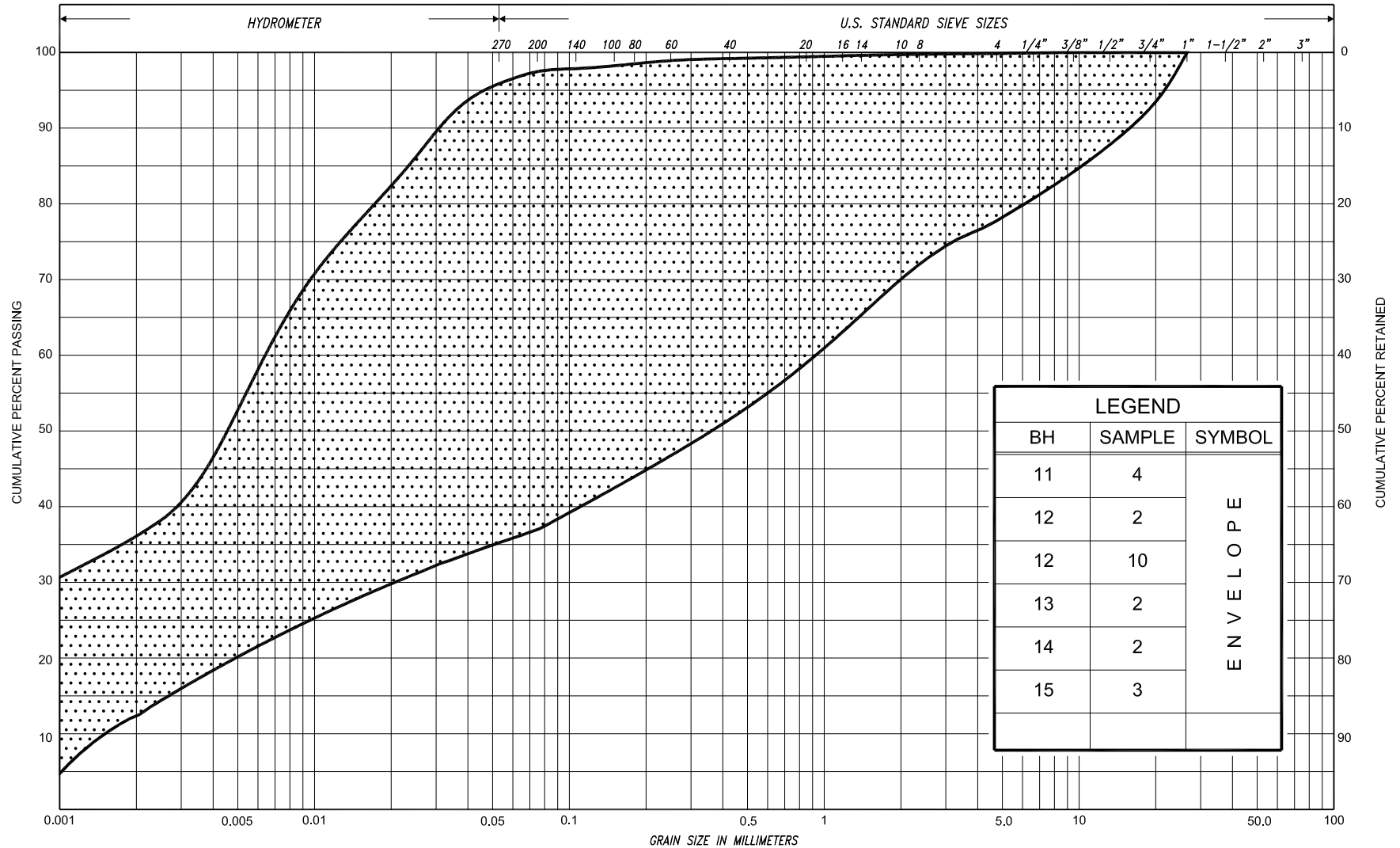
Notes:

Drilled: January 12 to January 23, 2015

Logged: January , 2015

RQD = Rock Quality Designation

Originated:	FP
Compiled:	PML
Checked:	CN



LEGEND		
BH	SAMPLE	SYMBOL
11	4	E N V E L O P E
12	2	
12	10	
13	2	
14	2	
15	3	

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								

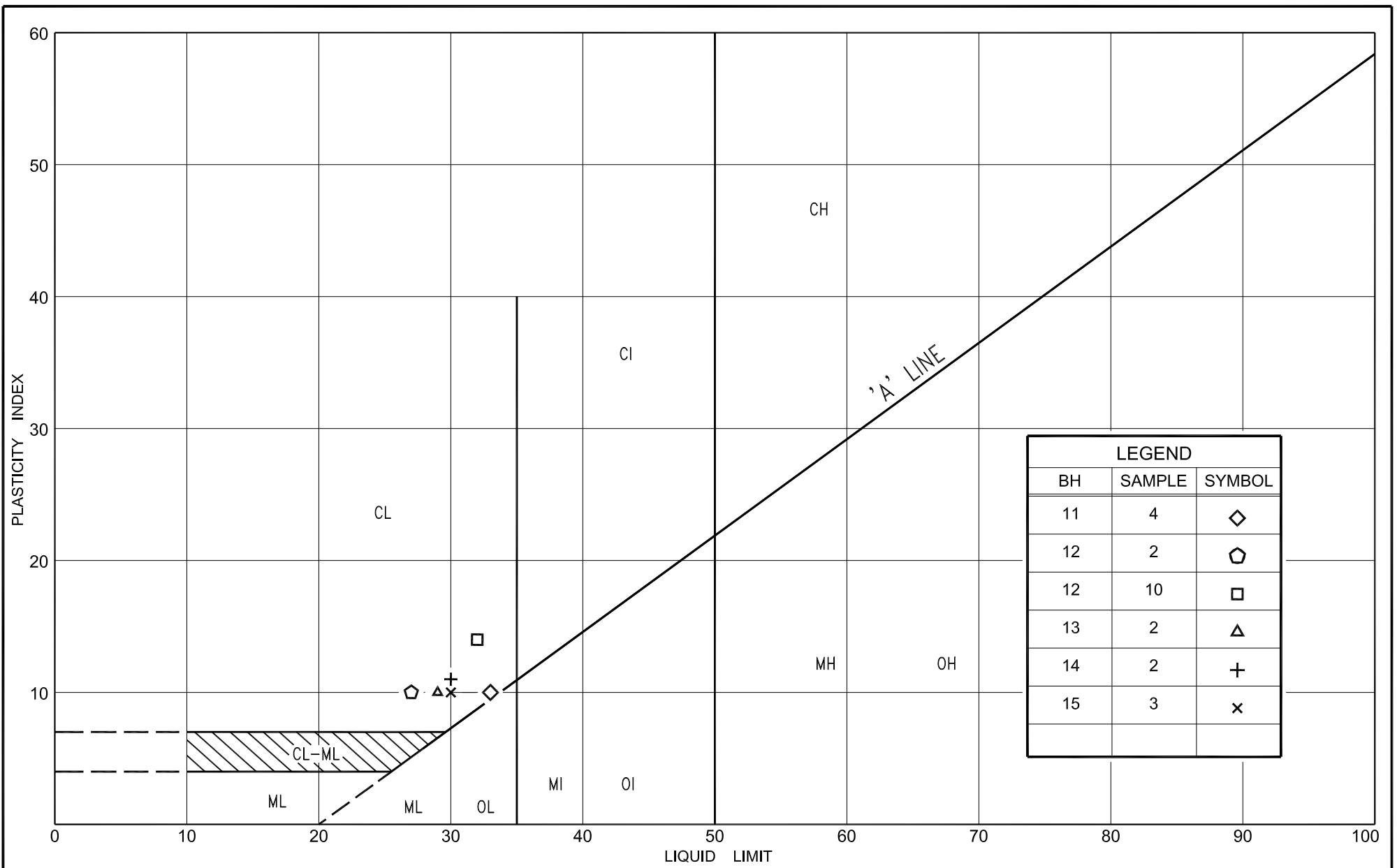


GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY trace sand to sandy, trace to with gravel (CL-CI)

FIG No. WB-GS-1

HWY: 403 / QEW

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



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

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

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No 11

1 of 1

METRIC

G.W.P. 2163-10-00 **LOCATION** Coords: 4 817 713.8 N; 290 736.6 E **ORIGINATED BY** S.A.
DIST Central **HWY** QEW / 403 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** R.A.
DATUM Geodetic **DATE** January 23, 2015 **CHECKED BY** D.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									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
145.7	Ground Surface					20	40	60	80	100							
0.0	Clayey silt rootlets, organics topsoil inclusions		1	SS	23	145											
	Very stiff Dark Moist brown/ reddish brown		2	SS	9									○			
	asphalt debris, organics gravel fill inclusions		3	SS	9		144										
143.7	(FILL)																
2.0	Clayey silt to silty clay trace sand, silt seams					143										0 3 62 35	
	Very stiff Reddish Moist to hard brown/ grey		4	SS	17									○ H			
142.7						142											
3.0	Shale bedrock limestone embedded clayey silt to silty clay seams/layers		5	SS	40												
	Highly weathered																
141.0			6	SS	50/10cm	141											
4.7	End of borehole																

RECORD OF BOREHOLE No 12

1 of 1

METRIC

G.W.P.	2163-10-00	LOCATION	Coords: 4 817 739.5 N; 290 731.8 E	ORIGINATED BY	S.A.
DIST	Central	HWY	QEW / 403	BOREHOLE TYPE	Continuous Flight Hollow Stem Augers
DATUM	Geodetic	DATE	January 22, 2015	COMPILED BY	R.A.
				CHECKED BY	D.D.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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		20 40 60 80 100														
							SHEAR STRENGTH kPa										20 40 60				
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										WATER CONTENT (%)				
146.1 0.0	Ground Surface					146															
145.2 0.9	Sandy silt, with gravel rootlets, organic inclusions		1	SS	44																
	Dense Dark Moist brown/ brown/ brown (FILL)																				
	Clayey silt to silty clay sandy, some gravel, sand seams, weathered shale fragments		2	SS	11	145										20 43 24 13					
			3	SS	37																
144.0 2.1	Stiff Reddish Moist brown/ grey					144															
	Shale bedrock limestone embedded clayey silt to silty clay seams/layers		4	SS	80/28cm																
	Highly weathered		5	SS	50/10cm	143															
			6	SS	50/8cm	142															
						141															
			7	SS	50/10cm	140															
						139															
			8	SS	50/10cm	138															
						137															
			9	SS	50/8cm																
						136															
	clayey silt layer		10	SS	50/5cm	135										22 20 36 22					
133.8 12.3	gravelly sand layer		11	SS	50/10cm	134															
	End of borehole																				
	Sample #10: Sampler bouncing																				
	* 2015 01 22																				
	▽ Water level observed during drilling																				

RECORD OF BOREHOLE No 13

1 of 1

METRIC

G.W.P. 2163-10-00 **LOCATION** Coords: 4 817 768.2 N; 290 724.6 E **ORIGINATED BY** S.A.
DIST Central **HWY** QEW / 403 **BOREHOLE TYPE** C.F.H.S.A. and Rock Core **COMPILED BY** R.A.
DATUM Geodetic **DATE** January 23, 2015 **CHECKED BY** D.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												
146.1	Ground Surface						20	40	60	80	100									
0.0 145.8 0.3	Clayey silt rootlets, organics, topsoil inclusions		1	SS	15															
	Very stiff Reddish Moist brown/ dark brown (FILL)		2	SS	30												1 8 65 26			
144.3	Clayey silt to silty clay trace sand, trace gravel, sand seams, rootlets		3	SS	93/25cm															
1.8	Very stiff Reddish Moist brown/ grey		4	SS	50/13cm															
	Shale bedrock limestone embedded clayey silt to silty clay seams/layers																			
	Highly weathered		5	SS	50/8cm															
			6	SS	50/5cm															
			7	SS	50/10cm															

RECORD OF BOREHOLE No 14

1 of 1

METRIC

G.W.P.	2163-10-00	LOCATION	Coords: 4 817 832.6 N; 290 757.2 E	ORIGINATED BY	S.A.
DIST	Central	HWY	QEW / 403	BOREHOLE TYPE	C.F.H.S.A. and Rock Core
DATE	Geodetic	DATE	January 20, 2015	COMPILED BY	R.A.
				CHECKED BY	D.D.

[illegible]

RECORD OF BOREHOLE No 15

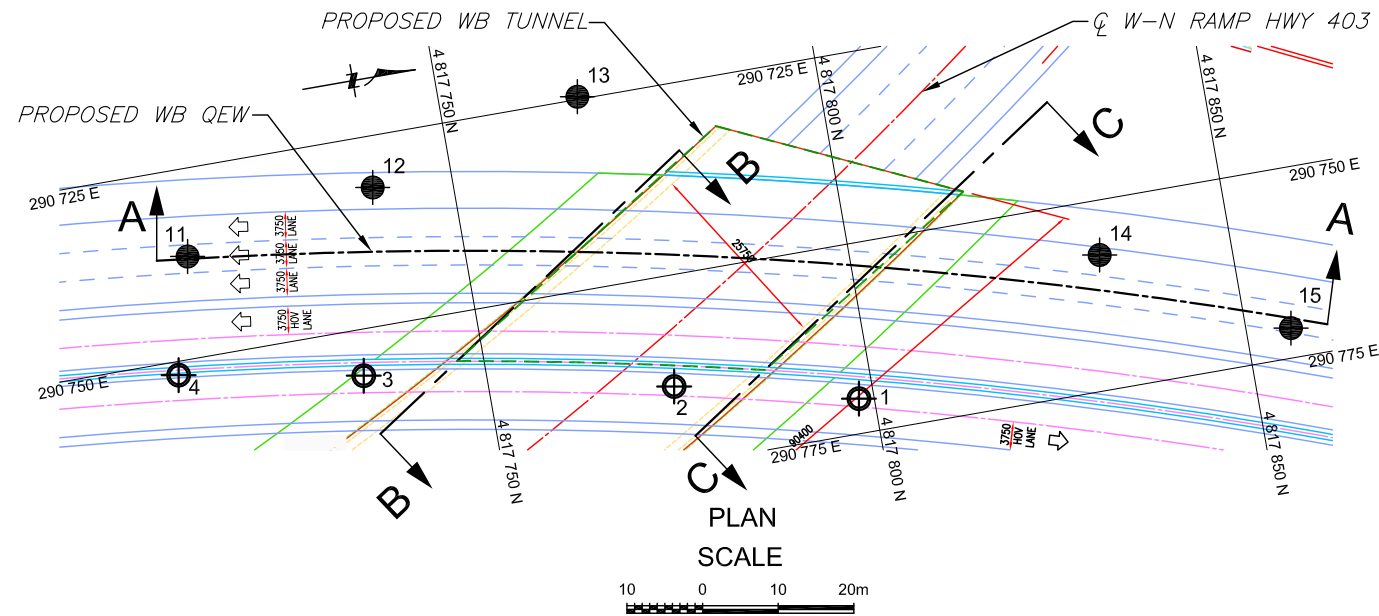
1 of 1

METRIC

G.W.P. 2163-10-00 **LOCATION** Coords: 4 817 855.9 N; 290 771.1 E **ORIGINATED BY** S.A.
DIST Central **HWY** QEW / **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** R.A.
DATUM Geodetic 403 **DATE** January 20, 2015 **CHECKED BY** D.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			20 40 60 80 100									
								SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
					20 40 60 80 100					WATER CONTENT (%)							
148.4	Ground Surface																
0.0	Clayey silt, with gravel organic inclusions, cobbles		1	SS	31												
147.6	Dense Reddish Moist brown grey (FILL)																
0.8	Clayey silt to silty clay trace sand, trace gravel		2	SS	25								○				
146.6	Very stiff Reddish Moist brown		3	SS	50/10cm								○	—		8 4 61 27	
1.8	Shale bedrock limestone embedded sandy silt seams/layers Highly weathered		4	SS	50/5cm												
			5	SS	50/5cm												
			6	SS	50/3cm												
144.4	End of borehole																
4.0	Sample #4 - #6: Sampler bouncing																
	* Borehole dry																

DRAFT NOTE:
DRAFTING TO BE
COMPLETED UPON
RECEIPT OF GA DRAWING
FROM STANTEC
CONSULTING LTD.

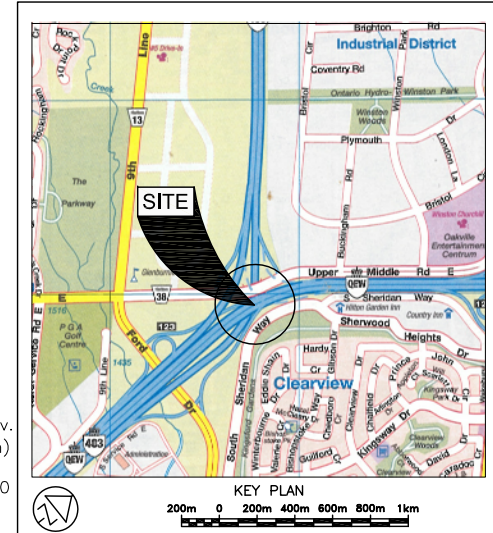


CONT No XXXX-XXX
GWP No 2163-10-00



HIGHWAY 403 W-N RAMP
UNDER QUEEN ELIZABETH WAY WB
Borehole Locations and Soil Strata

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS



LEGEND			
	Borehole		
	Cone		
	Borehole and Cone		
	Geocres Report Borehole (30M5-117)		
N	Blows/0.3m (Std. Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60 Cone, 475 J/blow)		
	WL at time of investigation Jan. 2015		
	Water level not established		
	Head		
	ARTESIAN WATER Encountered		
	PIEZOMETER		

BH No	ELEVATION	NORTHINGS	EASTINGS
11	145.7	4 817 713.8	290 736.6
12	146.2	4 817 739.5	290 731.8
13	146.1	4 817 768.2	290 724.6
14	147.9	4 817 832.6	290 757.2
15	148.4	4 817 855.9	290 771.1
GEOCRES REPORT BOREHOLES			
1	147.6	4 817 798.0	290 770.4
2	147.0	4 817 774.2	290 764.6
3	145.9	4 817 734.0	290 756.1
4	145.6	4 817 709.9	290 751.8

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

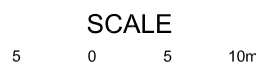
HWY No	QEW/403	DIST	CENTRAL
SUBM'D	NA	CHECKED	RA
DRAWN	NL	CHECKED	DD
DATE	MAY 22, 2015	APPROVED	CN
SITE	10-284	DWG	WB-1

REF Stantec Drawing: 893-10-284-opt_c-ultimate.dwg undated

14T005 HWY 403 / QEW EBL Bridge
MAY 2015
MODIFIED:

DRAWING NAME:
CREATED:

- NOTES:
- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
 - 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.



APPENDIX FIR-A

Relevant GEOCREs Data

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES					
484.1	Ground Level											
0.0	SILTY CLAY					480						
	Very Stiff To Hard		1	SS	15							
477.1			2	SS	60							
7.0	QUEENSTON SHALE BEDROCK		3	SS	100	12"						
			4	SS	100	9"						
	Red To Gray Red		5	SS	100	5"						
468.9			6	SS	50	3"	470					
15.2	End Of Borehole											
	Note: W.L. Not Established											

W P 159-75-06 LOCATION N 15 805 618 E 953 900 Co-ords. ORIGINATED BY P.J.S.
DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Augers, NX Casing and NXL Core COMPILED BY P.J.S.
DATUM Geodetic DATE December 15, 1977 CHECKED BY R.S.

[illegible]

RECORD OF BOREHOLE No 3

W P 159-75-06 LOCATION N 15 805 486 E 953 872 Co-ords. ORIGINATED BY P.J.S.
DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Auger, B Casing and BXL Core COMPILED BY P.J.S.
DATUM Geodetic DATE December 14, 1977 CHECKED BY R.S.

[illegible]

RECORD OF BOREHOLE No 4

W P 159-75-06 LOCATION N 15 805 407 E 953 858 Co-ords. ORIGINATED BY P.J.S.
DIST 4 HWY Q.E.W. BOREHOLE TYPE Solid Auger COMPILED BY P.J.S.
DATUM Geodetic DATE December 22, 1977 CHECKED BY R.S.

[illegible]



APPENDIX FIR-B

Rock Core Photographs



Photograph 1: Cores retrieved from borehole 13. Rock cores 1 to 3 from 7.6 to 10.7 m. RQD values ranged from 25% to 100%, indicating poor to excellent rock quality.



Photograph 2: Cores retrieved from borehole 14. Rock cores 1 to 3 from 7.3 to 10.4 m. RQD values ranged from 41% to 60%, indicating poor to fair rock quality.



**DRAFT
FOUNDATION DESIGN REPORT**

for

**HIGHWAY 403 W-N RAMP UNDER QUEEN ELIZABETH WAY WB
WP 2163-10-00, SITE NO. 10-284/1
CENTRAL REGION, ONTARIO**

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May 22, 2015



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**DRAFT
FOUNDATION DESIGN REPORT**

for
Highway 403 W-N Ramp Under Queen Elizabeth Way WB
WP 2163-10-00, Site No. 10-284/1
Central Region, Ontario

6. GENERAL

This report provides foundation engineering comments and recommendations for detail design and construction of the foundations and approach embankments within 20 m of the abutments of the proposed new bridge for the replacement of the existing Queen Elizabeth Way (QEW) WB bridge over the Highway 403 W-N Ramp (Site 10-284/1). The study was carried out by Peto MacCallum Ltd. (PML) for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The new QEW WB bridge over the Highway 403 W-N Ramp will be constructed on an offset alignment, north of the existing bridge, as part of the staging strategy to maintain the existing traffic in both directions on QEW. This configuration is part of the scheme to accommodate the proposed HOV lanes.

There is no existing bridge along the alignment for the proposed QEW WB bridge. The existing QEW bridges to be replaced are along different alignments and were constructed in 1980. They are three-span concrete bridges with main spans measuring about 42.7 m in length and approach spans measuring about 21.9 m in length. The widths of the decks of these existing bridges are about 17.7 m.

Details of the new bridge have not been finalized at this time. However, based on recent preliminary information it is understood that the new bridge will be about a 26 m long single span bridge constructed as a tunnel. The proposed single-span tunnel design alternative is shown on a drawing provided by Stantec entitled 'QEW Cross-Section Alternatives' attached in Appendix FDR-A. Highway 403 at the bridge site is at about elevation 140 and the grades of QEW at the bridge vary from about 145 to 147 m.

All the elevations presented in this report are in meters.



7. FOUNDATIONS

7.1 Foundation Alternatives

A comparison of the relative advantages and disadvantages related to each of the feasible foundation alternatives are discussed below.

Foundation	Advantages	Disadvantages	Relative Cost	Risk / Concerns
Spread Footings	<ul style="list-style-type: none"> • Contractor availability • Ease of mobilizing equipment • Ease of installation • Lower cost than deep foundations • May be used with semi-integral abutments 	<ul style="list-style-type: none"> • Potential disturbance of founding surface after exposure • Larger excavations requiring shoring support systems 	<ul style="list-style-type: none"> • Moderate 	<ul style="list-style-type: none"> • Weathering of bedrock prior to foundation construction could reduce bearing resistance
Caissons	<ul style="list-style-type: none"> • Minimized excavation • Higher load capacities • May be used with semi-integral abutments 	<ul style="list-style-type: none"> • Contractor availability • More cost prohibitive 	<ul style="list-style-type: none"> • High 	<ul style="list-style-type: none"> • Complexity of installation may require temporary liners
Piles	<ul style="list-style-type: none"> • Integral abutments 	<ul style="list-style-type: none"> • Deep rock excavation required to obtain 3 m zone for free pile bending and 5 m for pile length 	<ul style="list-style-type: none"> • High due to requirement for excavation of bedrock 	<ul style="list-style-type: none"> • Potential complications with deeper excavations into bedrock for integral abutment piles



Conventional or semi-integral abutments are considered practical at this site. Integral abutments are feasible, but not 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 bending lengths and minimum pile lengths for pile seating.

Based on the encountered subsurface conditions, it is considered practical to construct the proposed new bridge foundations on spread footings on the bedrock. Shale bedrock is present at relatively shallow depth below ground surface at the site such that shallow foundations (spread footings) supported on bedrock are a feasible alternative for foundation support.

With space restrictions and associated requirements for temporary roadway protection for construction of spread footings adjacent to the travelled lanes of Highway 403 W-N ramp and potentially in close proximity to existing QEW bridges, 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 free pile lengths.

7.2 Spread Footings on Bedrock

The bedrock surface level within the footprints of the foundation elements ranges from elevation 143.8 to 144.3 at the west abutment and from elevation 144.8 to 146.4 at the east abutment.

The shale bedrock at the site is highly weathered and of very poor to poor quality within the upper 4.0 to 6.5 m becoming slightly weathered and of fair to good quality with depth. Consideration could be given to placing the footings at shallow depths within the upper highly weathered shale bedrock or deeper within the slightly weathered shale bedrock.



The recommended founding levels for spread footings placed on the bedrock at the west and east abutments are provided in the following table.

Foundation Unit	Borehole	Upper Highly Weathered Shale Bedrock		Lower Slightly Weathered Shale Bedrock	
		Approximate Founding Depth (m)	Maximum Founding Elevation (m)	Approximate Founding Depth (m)	Maximum Founding Elevation (m)
West Abutment	12	3.1	143.0	7.1	139.0
	13	3.1		7.1	
	3	5.0		6.9	
East Abutment	14	3.9	144.0	7.9	140.0
	2	3.0		7.0	
	1	3.6		7.6	

The following values for factored geotechnical resistance at ULS and geotechnical reaction at SLS should be used for design of the spread footings for the bridge elements:

Founding Stratum	Geotechnical Resistance (kPa)	
	Factored ULS	SLS
Highly Weathered Bedrock	750	500
Slightly Weathered Bedrock	1000	750

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 skin 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 6.7.4 of the CHBDC.

The nearest edge of footings perched above the grade of Highway 403 should be set back at least 1.0 m from a plane inclined upwards and away from the toe of the road cut at 2H:1V. Forward slopes of rock should be protected from erosion and weathering.



There are 3 planes to consider when computing horizontal resistance – concrete to concrete at the underside of the footing, concrete to shale and within the shale to shale interbeds. The following table provides unfactored friction coefficients for each plane.

Resistance Planes	Friction Coefficient
Concrete to Concrete	0.75
Concrete to Shale	0.45
Shale to Shale	0.30

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

7.3 Caissons

As discussed above, the upper 4.0 to 6.5 m of bedrock is considered to be highly weathered becoming slightly weathered below these depths.

Resistance values for caissons of selected diameters and socket lengths are tabulated below. Refer to Section 4.3 of the foundation investigation report for elevations of the bedrock surface at the foundation elements.

Caisson Diameter (m)	Axial Geotechnical Resistance for Minimum Socket Length			
	3.0 m (Elevation 139.5 West Abutment) (Elevation 141.5 East Abutment)		4.5 m (Elevation 138.0 West Abutment) (Elevation 140.0 East Abutment)	
	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 may be required to support the overburden during construction. Refer to the related NSSP for Caissons in Appendix FDR-B.



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.

7.4 Frost Protection

All footings and/or caisson caps subject to frost action should be provided with 1.2 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 0.6 m of soil cover. This recommendation applies to the soil and bedrock at this site.

7.5 Seismic Considerations

The seismic site coefficient for the conditions at this site is 1.0 (soil profile Type 1, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6).

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

7.6 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 along with draft NSSP's.

8. 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 pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation:

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

where K = coefficient of lateral earth pressure (dimensionless)

γ = unit weight of free-draining granular material, kN/m^3

h = depth below final grade, m

q = surcharge load, kPa, if present

C_p = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)

C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)

where ϕ = angle of internal friction of retained soil (35° for Granular B Type II)

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

The seismic site coefficient for the conditions at this site was provided in Section 2.1.

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 parameters are recommended for design:

PARAMETER	GRANULAR A, GRANULAR B TYPE II or TYPE III	ROCKFILL
Angle of Internal Friction, degrees	35	42
Unit weight, kN/m^3	22.8	18.0
Coefficient of Active Earth Pressure, K_a	0.27	0.20
Coefficient of Earth Pressure At Rest, K_o	0.43	0.33
Coefficient Passive Earth Pressure, K_p	3.69	5.04

Refer to MTO Report SO-96-01 for procedures to determine the earth pressure coefficient to be employed in design of integral abutments. 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 (Clause C6.9.1) 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 6.9.3 of the CHBDC. Refer to OPSS.PROV 501 for additional information in this regard.

RSS walls can be used for retaining walls as required. Refer to Appendix FDR-B for an NSSP for design and construction of RSS systems.

9. APPROACH EMBANKMENTS

As noted previously, surficial fill soils containing construction debris (i.e. asphalt, gravel) and or traces of organic matter were encountered within the upper 2.0 m of the existing fill that forms part of the side embankments along Highway 403. That portion of the fill appears to have been placed as uncontrolled fill (i.e. contains construction debris, organic matter) and therefore is not considered suitable as support for the approach embankments without undergoing settlements. Since this bridge is on a new alignment, there are no existing approach fills for the QEW. 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 alignment of the QEW approach fill within 20 m of the abutments should be excavated down to native clayey silt to silty clay (up to 2.0 m) prior to placement of the new embankment fill. This zone of fill would have to be removed to meet the final grade of the QEW WB 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 1 to 2% 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 gap 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.



10. EXCAVATION AND GROUNDWATER CONTROL

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 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. 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.

Depending on the required depth of excavation, a roadway protection system may be necessary along the existing QEW WB bridge, lanes and or embankments. The roadway protection system is required where excavation geometry is steeper than 1H:1V and 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. It should be noted that groundwater



was encountered at only one borehole (borehole 12). The groundwater at that borehole was at a depth of 10.7 m (elevation 135.4).

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. Subject to the groundwater level at the time of construction, consideration could be given to employing a system of oversize perimeter ditching and sump pumps to control groundwater seepage into the open excavations during construction. 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.

Surface water run-off should be diverted away from excavation to ensure that the foundations are constructed in the dry.

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

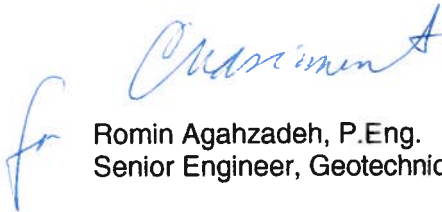


11. **CLOSURE**


The report was prepared by Mr. R. Agahzadeh, P.Eng. and reviewed by Mr. David Dundas, 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.


Romin Agahzadeh, P.Eng.
Senior Engineer, Geotechnical Services


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


C. M. P. Nascimento, P.Eng.,
MTO Designated Principal Contact

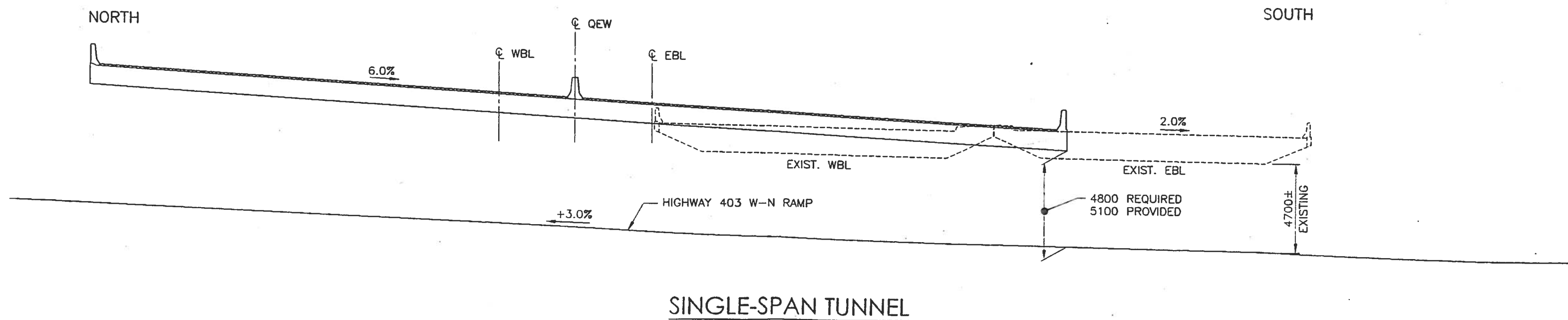
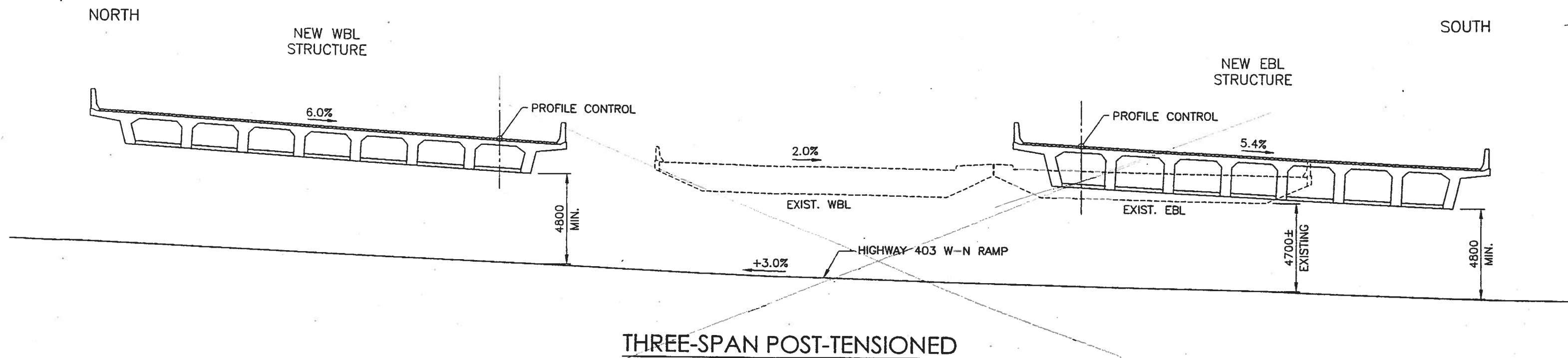
NOTE: The Final Report will be signed and stamped by two Professional Engineers licensed by PEO, one of whom shall be the Designated Principal Contact for MTO foundation projects

RA/DD/CMP:ra-jk



APPENDIX FDR-A

QEW Cross-Section Alternatives





APPENDIX FDR-B

OPSS's and NSSP's



LIST OF OPSS's and STANDARD SPECIFICATIONS RELEVANT TO THE REPORT

DOCUMENT	TITLE
OPSS 405	Construction Specification for Pipe Subdrains
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 206	Construction Specifications for Grading
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	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3190.100	Walls Retaining and Abutment Wall Drain



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 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.

NSSP for Caissons – Addition to OPSS 903

The Contractor shall be responsible for maintaining the stability and integrity of the sides and bases of caissons without disturbance until concrete is placed. The Contractor shall be advised that the shale bedrock is susceptible to weathering and exposed bearing surfaces must be protected with concrete cover within 12 hours of exposure.



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
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.