



**FOUNDATION INVESTIGATION AND DESIGN REPORT
for
PROPOSED HIGHWAY 403 W-E RAMP OVERPASS AT
QUEEN ELIZABETH WAY WBL
WP 2163-10-03, SITE NO. 10-284/1
CENTRAL REGION, ONTARIO**

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

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PML Ref.: 14TF005
Index No.: 057FIR and 058FDR
GEOCRES No.: 30M5-318
December 9, 2015



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for

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FOUNDATION INVESTIGATION REPORT

for

Proposed Highway 403 W-E Ramp Overpass at Queen Elizabeth Way WBL
WP 2163-10-03, Site No. 10-284/1
Central Region, Ontario

1. INTRODUCTION

This report summarises the results of a foundation investigation carried out for a replacement of existing Queen Elizabeth Way (QEW) westbound overpass over the Highway 403 W-E ramp in Oakville, Ontario. The study was conducted for Stantec Consulting Ltd. (Stantec) on behalf of the Ministry of Transportation of Ontario (MTO).

The QEW passes over the Highway 403 W-E Ramp at approximate Station 23+950, QEW WBL chainage (ref. General Arrangement Drawing 'Hwy 403 W-E Ramp WBL Overpass Replacement' prepared by Stantec in October 2015). The existing overpass is a three span structure. The replacement bridge is proposed to be constructed on a new alignment just north of the existing overpass.

The ramp from QEW EB to Hwy 403 EB is referred to as the Highway 403 W-E Ramp in this Contract. The contractor is advised that some background drawings and reports may refer to it as the Highway 403 W-N Ramp.

The existing road grade on Highway 403 at the bridge location varies between approximate elevation 139.5 and 141.5. The approach embankments are about 7 to 8 m high at the abutments.

The report provides subsurface information pertaining to the proposed structure and approaches within about 20 m of the abutments.

A preliminary foundation investigation and design report for the replacement of the Highway 403 W-E ramp under the QEW WBL bridge was completed in April 2013 by Thurber Engineering Ltd. and documented in GEOCREC No. 30M5-285. A foundation investigation and design report



completed for the existing bridge in 1978 is documented in GEOCRES No. 30M5-117. Copies of the relevant borehole logs are provided in Appendix FIR-A for reference.

All elevations in this report are expressed in meters.

2. SITE DESCRIPTION AND GEOLOGY

The site is situated about 0.5 km north of the QEW / Ford Drive interchange. The structure to be replaced carries QEW westbound traffic over the W-E ramp connecting the QEW and Highway 403 in the Town of Oakville, Regional Municipality of Halton. The QEW runs approximately in the south-north direction at the overpass location.

The study area lies in the physiographic region known as the South Slope that is bounded by the Peel Plain to the north and the Iroquois Plain to the south and extends from the Niagara escarpment to the Trent River, covering approximately 2,435 square kilometers. The South Slope is characterised 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 site are generally vacant and grass covered. The topography is gently sloping down towards the south. The Highway 403 W-E ramp is within a cut, some 5 to 7 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.

3. INVESTIGATION PROCEDURES

A review of the following reports available for the existing bridge was carried out. The subsurface information from boreholes 1 to 4, 13-25 and 13-26 advanced as part of the previous investigations (GEOCRES Nos. 30M5-117 and 30M5-285) is considered to be relevant and used in this report.



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
(GEOCREC 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
(GEOCREC No. 30M5-117)
(by MTO dated February 1978)

The field work for this study was carried out during the periods of January 20 to 23 and October 22 to November 28, 2015 and comprised 12 boreholes and 5 auger probes advanced to depths of 0.8 to 12.3 m. The locations of the boreholes and auger probes are indicated on Drawing WR-1, attached.

The borehole and auger probe locations were established in the field by Peto MacCallum Ltd. The coordinates and ground surface elevations at the boreholes were provided by Callon-Dietz Inc.

The boreholes and auger probes were advanced using continuous flight hollow and solid stem augers, powered by a track-mounted drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. A total of five boreholes (13, 14, 17, 25 and 25A) were extended 3.0 to 3.1 m into bedrock using NQ diamond rock coring equipment supplemented by wash boring techniques.

Representative soil samples were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration tests (SPT) were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. The results of the field tests and observations are reported on the Record of Borehole sheets.



Groundwater conditions at the borehole and auger probe 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 borehole. Upon completion of drilling, the boreholes and auger probes were backfilled with bentonite-cement grout in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures.

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, classification and routine moisture content determination. Atterberg limits testing (6) and grain size distribution analyses (7) were conducted on selected soil samples. The laboratory test results are presented in Figures WR-GS-1, WR-GS-2 and WR-PC-1 and on the corresponding logs.

4. SUMMARISED 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 laboratory Atterberg limits testing, grain size distribution analyses and natural moisture content determinations are also shown on the Record of Borehole sheets.

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

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised surficial topsoil or fill and a cohesive deposit of clayey silt / silty clay overlying shale bedrock. The bedrock was contacted or inferred below the fill or clayey silt / silty clay at depths of 0.3 to 3.7 m (elevation 136.4 to 146.6). The groundwater was at elevation 137.7 to 143.4.

The strata encountered are summarised below.



4.1 Topsoil

Surficial topsoil was identified in boreholes 13-25, 13-26 and auger probes AP-A, AP-B. The silty topsoil was 100 to 200 mm thick and penetrated at elevation 139.5 to 147.7.

4.2 Fill

Asphalt 100 to 180 mm in thickness was present surficially in boreholes 16 to 18, 25, 25A, 26 and 27 put down on the shoulders of the Highway 403 W-E ramp and overlay fill of various granulometric composition (sand and gravel, gravelly sand, sand and silt, clayey silt, silty clay). The fill had a moisture content of 7 to 12% and was typically penetrated at depths of 0.5 to 0.7 m (elevation 139.5 to 141.8). It is noted that the deeper fill encountered in borehole 17 extended to 2.2 m depth (elevation 138.0) and was due to a previous excavation. The results of grain size distribution analysis performed on the sand and silt fill from borehole 16 are presented in Figure WR-GS-1.

Surficial fill was present in boreholes 11 to 15 and auger probes AP-C and AP-D. The fill was composed of clayey silt, sandy silt or sand and gravel, locally containing organics and construction debris. The fill was 9 to 22% in moisture content and penetrated at depths of 0.1 to 2.0 m (elevation 138.5 to 147.6).

Directly beneath the topsoil at 0.1 m depth (elevation 146.2 and 147.7) in boreholes 13-25 and 13-26 was sand / gravelly sand fill. This unit was compact in relative density (SPT-'N' values of 18 and 21) and about 5% in moisture content. The sand / gravelly sand was 700 mm thick and penetrated at 0.8 m depth (elevation 145.5 and 147.0) in boreholes 13-25 and 13-26 respectively.

4.3 Clayey Silt / Silty Clay

A cohesive deposit of clayey silt / silty clay was present surficially in boreholes 1 to 4 and auger probe AP-L and overlain by the topsoil or fill at depths of 0.1 to 2.0 m (elevation 138.5 to 147.6) in boreholes 11 to 15, 17, 18, 25, 26, 13-25, 13-26 and auger probes AP-A to AP-D. Stiff to hard in



consistency, this deposit had a thickness of 0.2 to 3.5 m and was penetrated at depths of 0.3 to 3.7 m (elevation 136.4 to 146.6).

The results of Atterberg limits testing and grain size distribution analyses conducted on 6 cohesive samples of the deposit are presented in respective Figures WR-PC-1 and WR-GS-2. The liquid and plastic limits of the clayey silt / silty clay ranged from 27 to 38 and from 17 to 20 respectively, with the plasticity index of 10 to 18. The moisture content of the deposit varied between 11 and 16%.

4.4 Bedrock

Bedrock was contacted or inferred by refusal below the fill or clayey silt / silty clay at depths of 0.3 to 3.7 m (elevation 136.4 to 146.6) in all the boreholes and auger probes. The bedrock comprises a reddish brown highly to slightly weathered soft to medium strength shale with interbedded greenish grey limestone. Seams or layers of clayey silt / silty clay were also noted within the highly weathered zones of the bedrock.

The rock cores retrieved from boreholes 2, 3, 13, 14, 17, 25, 25A, 13-25 and 13-26 are described on the corresponding borehole logs. A detailed description of the rock cores in boreholes 13, 14, 17 and 25 is given in Table A, appended. Photographs of the rock cores are shown in Appendix FIR-B.

The measured core recovery varied between 41 and 100%. The Rock Quality Designation (RQD) determined from the rock cores was in a range of 13 to 100%, typically 25 to 75%, thus indicating a poor to fair quality rock. The rock quality was very poor (RQD of 13%) in a core sample in borehole 2 at elevation 143.3, however this zone was previously excavated below the Highway 403 W-E Ramp where the existing pavement grades currently range from elevation 139.1 to 141.1. The RQD was good to excellent in a few core samples in boreholes 13, 17 and 13-25.

At the west abutment and approach, the bedrock surface was contacted or inferred at depths of 0.3 to 3.7 m (elevation 136.4 to 144.7) in boreholes 3, 4, 11 to 13, 16 to 18, 13-25 and auger



probes AP-A to AP-D. The measured core recovery in boreholes 3, 13, 17 and 13-25 varied between 70 and 100%. The RQD determined from the rock cores was in a range of 25 to 100%, indicating poor to excellent quality rock. The bedrock surface has a maximum relief of 8.3 m.

At the east abutment and approach, the bedrock surface was contacted or inferred at depths of 0.7 to 2.4 m (elevation 138.3 to 146.6) in boreholes 1, 2, 14, 15, 25, 25A, 26, 27, 13-26 and auger probe AP-L. The measured core recovery in boreholes 2, 14, 25, 25A and 13-26 varied between 41 and 100%. The RQD determined from the rock cores was in a range of 25 to 75%, indicating poor to fair quality rock, with the exception of one core sample in borehole 2 where the rock quality was very poor (RQD of 13%). The bedrock surface has a maximum relief of 8.3 m.

4.5 Groundwater

In the process of augering, water was detected at 10.7 m depth (elevation 135.4) in borehole 12 and a depth of 1.2 m (elevation 139.0) in borehole 17. Groundwater was measured in auger probes AP-B, AP-D and AP-L to be at depths of 0.9 to 1.7 m (elevation 137.7 to 138.7) upon completion of drilling.

The groundwater measured in the piezometers installed in boreholes 13-25 and 13-26 was at depths of 4.4 to 5.5 m (elevation 140.8 to 143.4).

The groundwater levels at the site are subject to seasonal fluctuations and precipitation patterns.



5. CLOSURE

The field work was carried out under the supervision of Mr. S. Aziz and Mr. M. Khorsand, under the coordination of Mr. K. Daly, B.Eng, and direction of Mr. C.M.P. Nascimento, P.Eng., Project Manager. The equipment was supplied by Altech Drilling & Investigative Services Ltd. The laboratory testing of selected samples was carried out in the PML laboratory in Toronto.

This report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng., Project Manager and MTO Designated Principal Contact.

Yours very truly,

Peto MacCallum Ltd.



Grigory O. Degil, PhD, P.Eng.
Senior Foundation Engineer



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

GD:gd-mi



TABLE A
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		

Originated: FP / JO / SAT

Compiled: PML

Checked: CN



TABLE A
ROCK CORE DESCRIPTIONS

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
17	1	3.2 – 4.7	87	37	3.2 – 6.2	LIMESTONE WITH INTERBEDDED SHALE: Dark grey to green, occasional purple shale, fine crystalline to aphanitic, with few stylitic partings, small chert nodules, soft to medium strength, slightly weathered to unweathered, close spaced flat partings, smooth to rough planar, tight, with dipping to vertical joints, poor to fair quality.
	2	4.7 – 6.2	100	58		
25	1	0.9 – 2.4	90	52	0.9 – 4.0	LIMESTONE WITH INTERBEDDED SHALE : Dark grey to dark green, occasional purple shale, fine crystalline to aphanitic, with few stylitic partings, small chert nodules, soft to medium strength, slightly weathered to unweathered, close spaced flat partings, smooth to rough planar, tight, with dipping to vertical joints, fair quality.
	2	2.4 – 4.0	93	58		
25A	1	1.2 – 2.7	63	25	1.2 – 4.3	SHALE WITH INTERBEDDED LIMESTONE : Dark green to purple shale, 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 moderately weathered, close spaced flat partings, smooth planar, tight, with dipping to vertical joints, poor quality.
	2	2.7 – 4.3	57	25		

Notes:

Drilled: January 20 and 23, October 23 and November 5, 2015

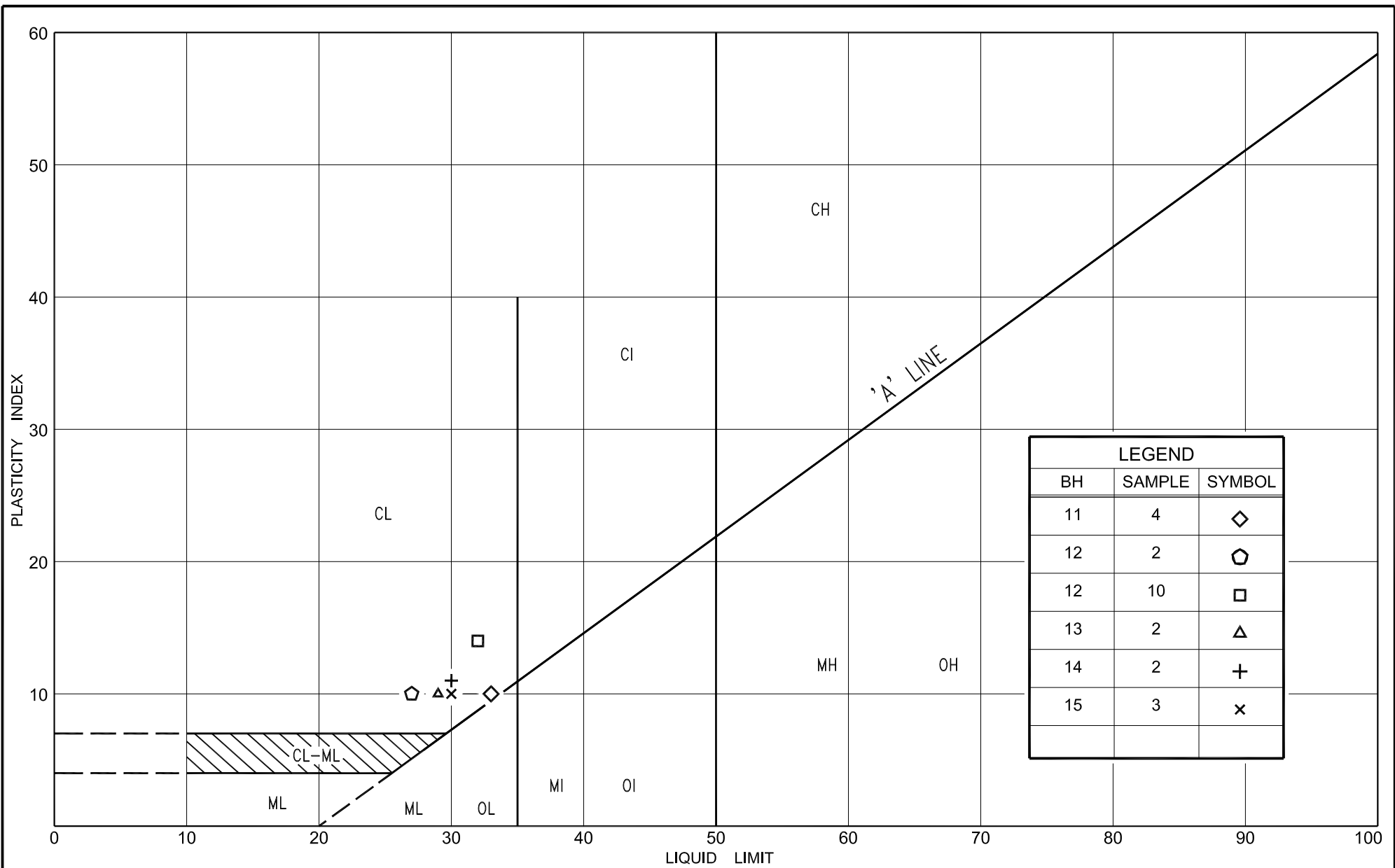
Logged: January 27, October 28 and November 12, 2015

RQD = Rock Quality Designation

Originated: FP / JO / SAT

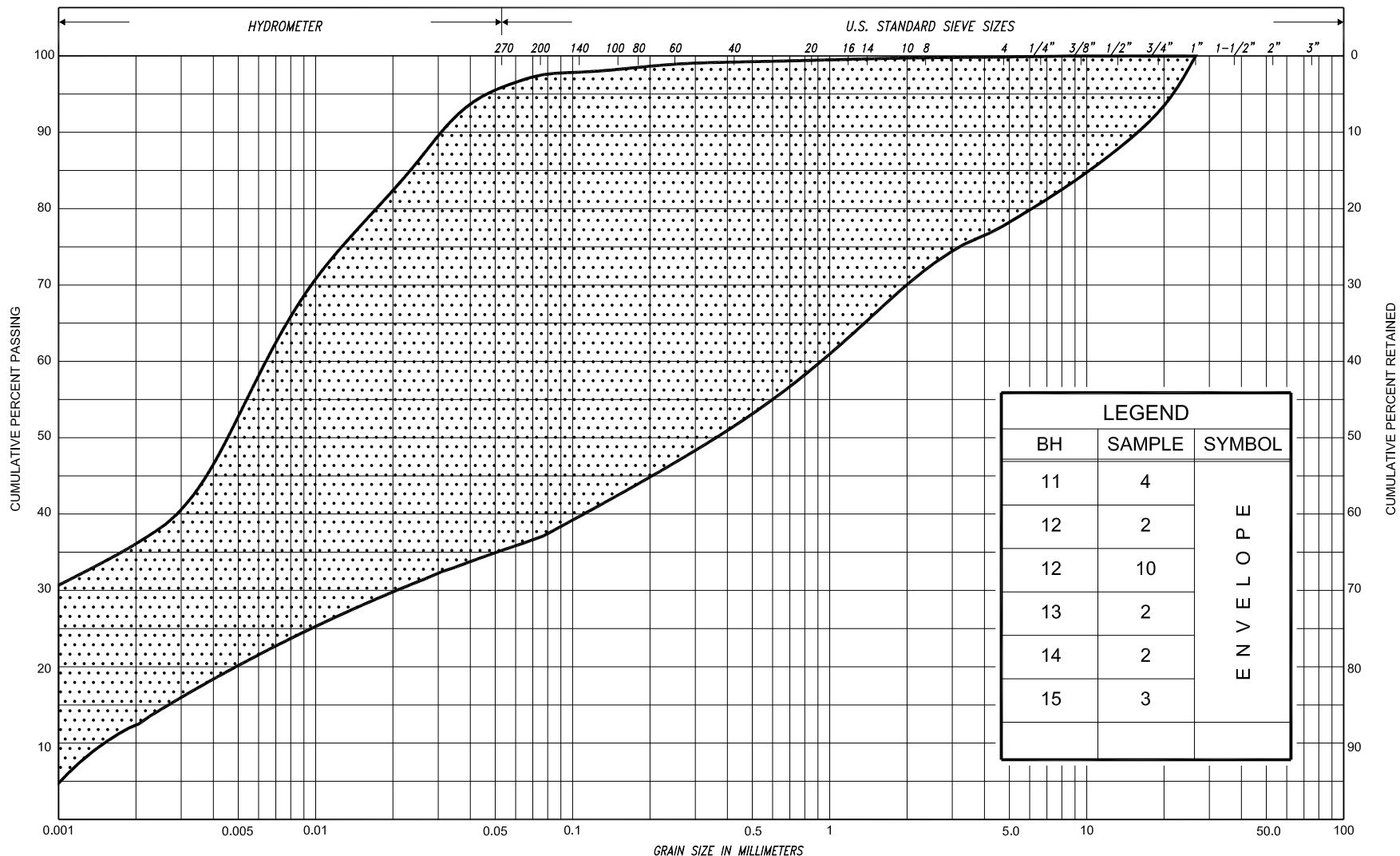
Compiled: PML

Checked: CN



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

FIG No. WR-PC-1
 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 sand to sandy, trace to with gravel (CL-CI)

FIG No. WR-GS-2

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'_{vo}	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	WTPL		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										WATER CONTENT (%)		
								○ 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															
	Very stiff Dark Moist brown/ reddish brown		2	SS	9							○								
	asphalt debris, organics gravel fill inclusions																			
143.7	(FILL)		3	SS	9															
2.0	Clayey silt to silty clay trace sand, silt seams																			
	Very stiff Reddish Moist to hard brown/ grey		4	SS	17							○	H			0 3 62 35				
142.7	Shale bedrock limestone embedded clayey silt to silty clay seams/layers		5	SS	40															
	Highly weathered																			
141.0			6	SS	50/10cm															
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 **COMPILED BY** R.A.
DATUM Geodetic **DATE** January 22, 2015 **CHECKED BY** D.D.


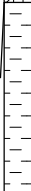
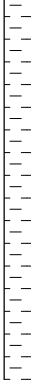
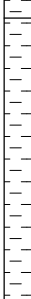


SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa									
							○ UNCONFINED + FIELD VANE									
							● QUICK TRIAXIAL × LAB VANE									
					WATER CONTENT (%)											
146.1	Ground Surface					20	40	60	80	100	20	40	60			
0.0	Sandy silt, with gravel rootlets, organic inclusions		1	SS	44						○					
145.2	Dense Dark Moist brown/ brown (FILL)		2	SS	11						○					20 43 24 13
0.9	Clayey silt to silty clay sandy, some gravel, sand seams, weathered shale fragments		3	SS	37											
144.0	Stiff Reddish Moist brown/ grey		4	SS	80/28cm						○					
2.1	Shale bedrock limestone embedded clayey silt to silty clay seams/layers		5	SS	50/10cm											
	Highly weathered															
			6	SS	50/8cm											
			7	SS	50/10cm											
			8	SS	50/10cm											
			9	SS	50/8cm											

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															
																				
	Slightly weathered to highly weathered		7	SS	50/10cm															
	Soft to medium strength																			
	Poor to fair quality																			
																				
			9	RC NQ	REC 100%												RQD 100%			
			10	RC NQ	REC 100%												RQD 25%			
																				
			11	RC NQ	REC 97%												RQD 78%			
135.4 10.7	End of borehole																			
	Sample #6: Sampler bouncing																			

RECORD OF BOREHOLE No 14

1 of 1

METRIC

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










[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/m³	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									
148.4	Ground Surface						20	40	60	80	100						
0.0	Clayey silt, with gravel organic inclusions, cobbles		1	SS	31		148										
147.6	Hard Reddish Moist brown/grey (FILL)																
0.8	Clayey silt to silty clay trace sand, trace gravel		2	SS	25		147							○			
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		4	SS	50/5cm		146										
	Highly weathered																
			5	SS	50/5cm		145										
																	
144.4			6	SS	50/3cm												
4.0	End of borehole																
	Sample #4 - #6: Sampler bouncing																
	 <																

RECORD OF BOREHOLE No 16
1 of 1
METRIC

G.W.P. 2163-10-00 **LOCATION** Coords: 4 817 800.7 N; 290 729.5 E **ORIGINATED BY** M.Kh
DIST Central **HWY** 403 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** M.Kh
DATUM Geodetic **DATE** October 23, 2015 **CHECKED BY** G.D.




SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT CONTENT			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		GR	SA	SI	CL
								20 40 60 80 100					20 40 60							
141.0	Ground Surface																			
0.0	130mm asphalt over sand and silt with gravel, some clay		1	SS	50/10cm														22 35 31 12	
140.4																				
0.6	Very dense Brown Moist (FILL)		2	SS	50/5cm															
139.9	Highly weathered shale						140													
1.1	End of borehole																			
	Refusal on probable bedrock																			
	Samples 1 & 2: Sampler bouncing																			
	* Borehole dry																			

RECORD OF BOREHOLE No 18

1 of 1

METRIC

G.W.P. 2163-10-00 LOCATION Coords: 4 817 775.2 N; 290 746.5 E ORIGINATED BY M.Kh
DIST Central HWY 403 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY M.Kh
DATUM Geodetic DATE October 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									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
140.0	Ground Surface							20	40	60	80	100					
0.0	150mm asphalt over silty clay some sand, some gravel shale fragments		1	SS	50/13cm												
139.5	(FILL)																
139.2	Clayey silt, trace sand weathered shale fragments		2	SS	50/0cm												
0.5	Hard Reddish Moist brown																
0.8	End of borehole																
	Refusal on probable bedrock																
	Sample 2: Sampler bouncing																
	* Borehole dry																

RECORD OF BOREHOLE No 17

1 of 1

METRIC

G.W.P. 2163-10-00 **LOCATION** Coords: 4 817 781.1 N; 290 743.1 E **ORIGINATED BY** S.A.
DIST Central **HWY** 403 **BOREHOLE TYPE** C.F.S.S.A. and Rotary Diamond Coring **COMPILED BY** M.Kh
DATUM Geodetic **DATE** November 04 and 05, 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											
								○ UNCONFINED + FIELD VANE											
								● QUICK TRIAXIAL × LAB VANE											
					WATER CONTENT (%)														
					20 40 60 80 100					20 40 60									
140.2	Ground Surface																		
0.0	130mm asphalt over sand and gravel																		
139.8	(PAVEMENT FILL)		1	SS	17														
0.4	Clayey silt with gravel, some sand																		
	Stiff Reddish Moist brown/grey		2	SS	12														
	Gravelly sand, trace silt																		
	Compact Reddish Moist brown/ to wet grey		3	SS	14														
138.0	(FILL)																		
2.2	Highly weathered shale		4	SS	55/10cm														
	Reddish brown																		
137.0			5	SS	50/10cm														
3.2	Limestone bedrock with shale fragments gravelly layered at 3.6m depth		6	RC NQ	REC 87%											RQD 62%			
	weathered shale in cells																		
			7	RC NQ	REC 100%											RQD 97%			
134.0	End of borehole																		
6.2	Samples 4 and 5: Sampler bouncing.																		
	 <																		

RECORD OF BOREHOLE No 25

1 of 1

METRIC

G.W.P.	2163-10-00	LOCATION	Coords: 4 817 790.7 N; 290 754.2 E	ORIGINATED BY	M.Kh
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DIST	Central	HWY	403	BOREHOLE TYPE	C.F.H.S.A. and Rotary Diamond Coring	COMPILED BY	M.Kh
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DATUM Geodetic **DATE** October 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			20 40 60 80 100														
								SHEAR STRENGTH kPa										WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE														
141.1	Ground Surface						141															
0.0	130mm asphalt over gravelly sand (PAVEMENT FILL)		1	SS	68																	
140.4																						
0.7	Clayey silt		2	SS	50/8cm																	
140.2	Highly weathered shale																					
0.9	Hard Reddish Moist brown		3	RC NQ	REC 90%																	
	Shale Bedrock with limestone fragments																					
	Limestone bedrock with clay and shale inclusions		4	RC NQ	REC 93%																	
137.1																						
4.0	End of borehole																					
	Sample 2: Sampler bouncing.																					

RECORD OF BOREHOLE No 25A

1 of 1

METRIC

G.W.P. 2163-10-00 **LOCATION** Coords: 4 817 809.9 N; 290 742.9 E **ORIGINATED BY** M.Kh
DIST Central **HWY** 403 **BOREHOLE TYPE** C.F.H.S.A. and Rotary Diamond Coring **COMPILED BY** G.D.
DATUM Geodetic **DATE** November 27 and 28, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
142.0	Ground Surface							20	40	60	80	100					GR SA SI CL			
0.0	180mm asphalt over gravelly sand		1	SS	25		141										RQD 25%			
141.3	Compact Reddish Moist brown (PAVEMENT FILL)		2	SS	50/8cm															
0.7	Highly weathered shale																			
	Reddish brown Slightly weathered shale		3	RC NQ	REC 63%															
	Limestone bands						140													
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				
																				

RECORD OF BOREHOLE No 26

1 of 1

METRIC

G.W.P. 2163-10-00	LOCATION	Coords: 4 817 821.2 N; 290 733.7 E	ORIGINATED BY M.Kh
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DIST Central HWY 403 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY M.Kh

DATUM Geodetic DATE October 22, 2015 CHECKED BY G.D.

[illegible]

RECORD OF BOREHOLE No 27

1 of 1

METRIC

G.W.P. 2163-10-00 **LOCATION** Coords: 4 817 826.9 N; 290 729.0 E **ORIGINATED BY** M.Kh
DIST Central **HWY** 403 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** G.D.
DATUM Geodetic **DATE** November 27 and 28, 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
142.5	Ground Surface																
0.0	150mm asphalt over gravelly sand																
141.8	Compact Reddish brown (PAVEMENT FILL)		1	SS	13		142										
0.7	Highly weathered shale bedrock		2	SS	50/8cm												
	Reddish brown limestone bands		3	SS	50/8cm		141										
140.4	End of borehole																
2.1	Refusal on slightly weathered bedrock																
	Sample 3: Sampler bouncing																
	* Borehole dry																

RECORD OF BOREHOLE No AP-A

1 of 1

METRIC

G.W.P.	<u>2163-10-00</u>	LOCATION	<u>Coords: 4 817 795.2 N; 290 727.8 E</u>	ORIGINATED BY	<u>M.Kh</u>
DIST	<u>Central</u>	HWY	<u>403</u>	BOREHOLE TYPE	<u>Continuous Flight Solid Stem Augers</u>
COMPILED BY					<u>M.Kh</u>
DATUM	<u>Geodetic</u>	DATE	<u>October 23, 2015</u>	CHECKED BY	<u>G.D.</u>



[illegible]

RECORD OF BOREHOLE No AP-B

1 of 1

METRIC

G.W.P.	2163-10-00	LOCATION	Coords: 4 817 772.9 N; 290 744.0 E	ORIGINATED BY	M.Kh		
DIST	Central	HWY	403	BOREHOLE TYPE	Continuous Flight Solid Stem Augers	COMPILED BY	M.Kh
DATUM	Geodetic	DATE	October 23, 2015	CHECKED BY	G.D.		

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT						PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa											WATER CONTENT (%)								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											w _p w w _L								
139.7	Ground Surface					139	20	40	60	80	100	20	40	60													
139.5	Topsoil																										
139.5 0.2	Silty clay																										
138.5	Highly weathered shale bedrock					138																					
138.0	End of auger probe																										
1.7	Refusal on probable bedrock																										
* 2015 10 23																											
 Water level measured after drilling																											
NOTE: Auger advanced fairly fast to 1.2m, then grinding to 1.7m depth.																											

RECORD OF BOREHOLE No AP-C

1 of 1

METRIC

G.W.P.	<u>2163-10-00</u>	LOCATION	<u>Coords: 4 817 749.7 N; 290 754.2 E</u>	ORIGINATED BY	<u>M.Kh</u>
DIST	<u>Central</u>	HWY	<u>403</u>	BOREHOLE TYPE	<u>Continuous Flight Solid Stem Augers</u>
COMPILED BY					<u>M.Kh</u>
DATUM	<u>Geodetic</u>	DATE	<u>October 23, 2015</u>	CHECKED BY	<u>G.D.</u>

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 (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
139.1	Ground Surface						20	40	60	80	100									
139.0	Sand and gravel (FILL)																			
138.9	Silty clay																			
137.9	Reddish brown																			
137.6	Highly weathered shale bedrock																			
137.5	End of auger probe																			
	Refusal on probable bedrock																			

RECORD OF BOREHOLE No AP-D

1 of 1

METRIC

G.W.P.	2163-10-00	LOCATION	Coords: 4 817 735.7 N; 290 760.7 E	ORIGINATED BY	M.Kh
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DIST	Central	HWY	403	BOREHOLE TYPE	Continuous Flight Solid Stem Augers	COMPILED BY	M.Kh
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DATUM Geodetic **DATE** October 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 (%)		
								○ UNCONFINED		+ FIELD VANE		● QUICK TRIAXIAL						× LAB VANE		
138.6	Ground Surface																			
138.5	Sand and gravel																			
138.3	(FILL)																			
138.3	Silty clay																			
0.3																				
137.7	Reddish brown																			
0.9	Highly weathered shale bedrock																			
	End of auger probe																			
	Refusal on probable bedrock																			
														</						

RECORD OF BOREHOLE No AP-L

1 of 1

METRIC

G.W.P. 2163-10-00 **LOCATION** Coords: 4 817 788.1 N; 290 762.9 E **ORIGINATED BY** M.Kh
DIST Central **HWY** 403 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** M.Kh
DATUM Geodetic **DATE** October 22, 2015 **CHECKED BY** G.D.

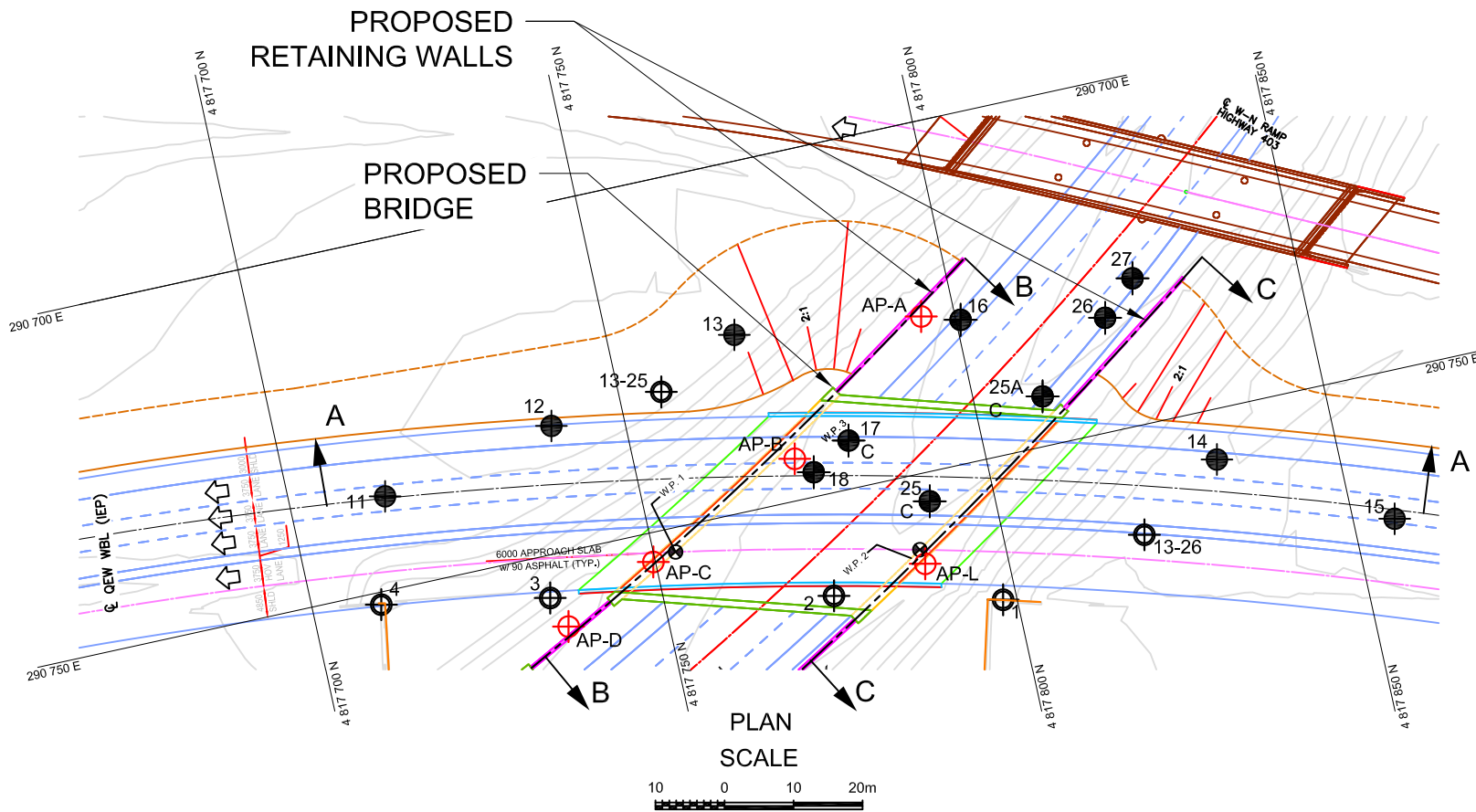
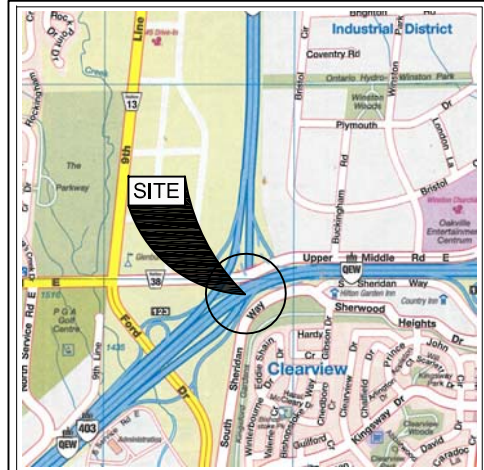
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
140.4	Ground Surface							20	40	60	80	100								
0.0	Silty clay, some gravel weathered shale fragments Reddish brown						140													
							139													
138.3	End of auger probe																			
2.1	Refusal on probable bedrock																			
	<div>* 2015 10 22</div> <div> Water level measured after drilling</div>																			

CONT No XXXX-XXX
GWP No 2163-10-00
WP No 2163-10-03

HIGHWAY 403 W-E RAMP OVERPASS
AT QUEEN ELIZABETH WAY WBL
Borehole Locations and Soil Strata

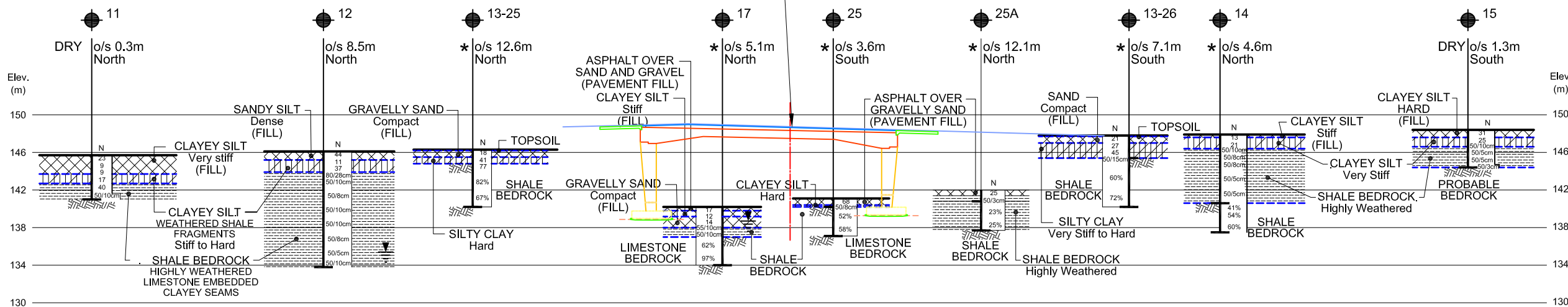


SHEET



NORTH
FOR CONSTRUCTION

PROPOSED BRIDGE



PROFILE A - A

SCALE



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.
- CHAINAGES FOR THE PROPOSED QEW WBL WERE NOT AVAILABLE.

BH No	ELEVATION	NORTHINGS	EASTINGS
AP-A	140.1	4 817 795.2	290 727.8
AP-B	139.7	4 817 772.9	290 744.0
AP-C	139.1	4 817 749.7	290 754.2
AP-D	138.6	4 817 735.7	290 760.7
AP-L	140.4	4 817 788.1	290 762.9

GEOCRIS REPORT BOREHOLES

1	2	3	4	13-25	13-26
147.6	147.0	145.9	145.6	146.3	147.8
4 817 798.0	4 817 774.2	4 817 734.0	4 817 709.9	4 817 756.1	4 817 820.0
290 770.4	290 764.6	290 756.1	290 751.8	290 730.4	290 765.6



(Legend Continued)

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. 30M5-318			
HWY No	QEW / 403	DIST	CENTRAL
SUBMD	NA	CHECKED	GD
DATE	DEC. 08, 2015	SITE	10-284
DRAWN	NA	CHECKED	GD
APPROVED	CN	DWG	WR-1



1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. REFER TO DRAWING WR-1 FOR BOREHOLE AND SECTION LOCATIONS
3. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
4. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.



REVISIONS			
	DATE	BY	DESCRIPTION

HWY No QEW / 403					DIST CENTRAL	
SUBM'D	NA	CHECKED	GD	DATE DEC. 08, 2015	SITE	10-284
DRAWN	NA	CHECKED	GD	APPROVED CN	DWG	WR-2

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



APPENDIX FIR-A

Relevant GEOCRETS Data

[illegible]

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 13-26

1 OF 1

METRIC

W.P. _____ LOCATION N 4 817 820.0 E 290 765.6 ORIGINATED BY GA
 HWY 403/QEW BOREHOLE TYPE Solid Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.05.30 - 2013.05.30 CHECKED BY LRB

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
147.8								20 40 60 80 100						
0.0	TOPSOIL: (100mm)							○ UNCONFINED + FIELD VANE						
0.1	SAND, some gravel, some silt Compact Brown		1	SS	21			● QUICK TRIAXIAL x LAB VANE						
147.0	Damp (FILL)							20 40 60 80 100						
0.8	Silty CLAY, trace sand, trace shale fragments Very Stiff to Hard		2	SS	27									0 4 69 27
			3	SS	45									
145.4			4	SS	50/ 0.150									
2.4	SHALE, highly weathered, thinly bedded, reddish brown													
	Start coring at 4.6m Slightly weathered to fresh, thinly bedded, reddish brown, occasional limestone interbeds Limestone interbeds (25mm thick) at 4.5m, 5.4m Highly broken zone (125mm) at 5.8m Soft zone (300mm) at 4.5m		1	RUN										RUN #1 TCR=100% SCR=83% RQD=60% UCS=8MPa (Average)
	Horizontal fractures (25mm thick) at 4.9m, 5.2m, 5.6m, 5.7m													
	Limestone interbeds (25mm thick) at 6.0m, 6.1m, 6.4m, 6.5m, 6.7m, 6.8m, 6.9m, 7.0m and (100mm) at 7.3m		2	RUN										RUN #2 TCR=100% SCR=93% RQD=72% UCS=16MPa (Average)
	Horizontal fractures from 6.4m to 7.0m													
140.2	END OF BOREHOLE AT 7.6m. BOREHOLE OPEN TO 7.6m AND WATER LEVEL AT 2.7m UPON COMPLETION OF CORING. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) May 30/13 4.8 143.0 Jun 26/13 4.4 143.4													
7.6														

ONTMT4S 1184 G.P.J. 2012 TEMPLATE (MTO) GDT 8/8/13

+ 3 x 3 Numbers refer to
Sensitivity 15 5 10 (%) STRAIN AT FAILURE



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.



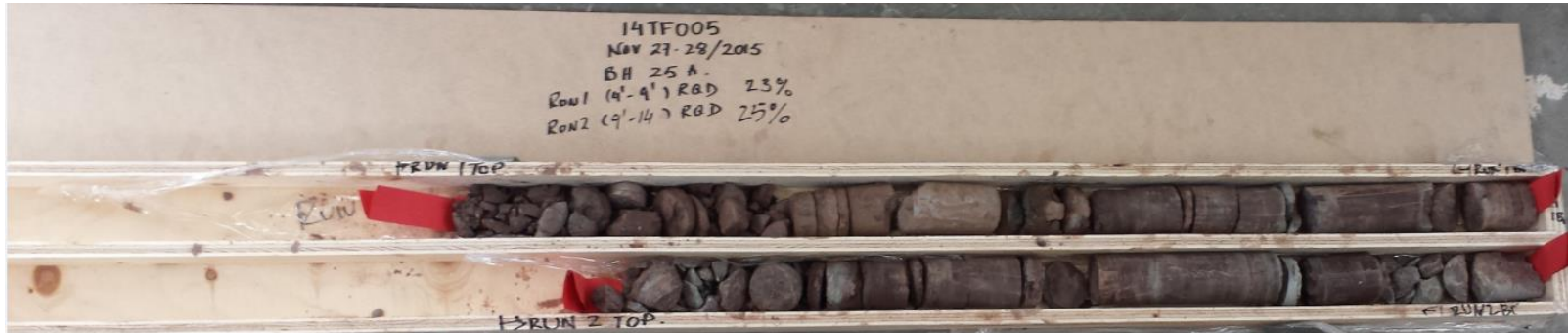
Photograph 2: Cores retrieved from borehole 14. Rock cores 1 to 3 from 7.3 to 10.4 m. RQD values ranged from 41% to 60%, indicating poor to fair rock quality.



Photograph 3: Cores retrieved from borehole 17. Rock cores 1 to 2 from 3.2 to 6.2 m. RQD values ranged from 37 to 58%, indicating poor to fair rock quality.



Photograph 4: Cores retrieved from borehole 25. Rock cores 1 to 2 from 0.9 to 4.0 m. RQD values ranged from 52% to 58%, indicating fair rock quality.



Photograph 5: Cores retrieved from borehole 25A. Rock cores 1 to 2 from 1.2 to 4.3 m. Both RQD values were 25%, indicating poor rock quality.



FOUNDATION DESIGN REPORT

for

**PROPOSED HIGHWAY 403 W-E RAMP OVERPASS AT
QUEEN ELIZABETH WAY WBL
WP 2163-10-03, SITE NO. 10-284/1
CENTRAL REGION, ONTARIO**

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:

- 1 cc: Stantec Consulting Ltd. for distribution to MTO,
Project Manager + 1 digital copy
- 3 cc: Foundation Investigation Report only to
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- 1 cc: PML Toronto

PML Ref.: 14TF005
Index No.: 058FDR
GEOCRES No.: 30M5-318
December 9, 2015



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Appendix FDR-A – Standard and Non-Standard Specifications

FOUNDATION DESIGN REPORT

for
Proposed Highway 403 W-E Ramp Overpass at Queen Elizabeth Way WBL
WP 2163-10-03, 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 overpass for the Highway 403 W-E Ramp at the relocated Queen Elizabeth Way (QEW) westbound lanes. 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 overpass will carry the QEW WBL traffic over the Highway 403 W-E Ramp and will be constructed on an offset alignment, north of the existing structure. The realignment is part of the staging strategy to maintain the existing traffic in both directions on QEW throughout the construction and also part of the scheme to accommodate the proposed HOV lanes. Retaining walls 18 and 20 m in length are planned to support the west and east embankments on the north side of the overpass.

The ramp from QEW EB to Hwy 403 EB is referred to as the Highway 403 W-E Ramp in this Contract. The contractor is advised that some background drawings and reports may refer to it as the Highway 403 W-N Ramp.

The alignment for the proposed QEW WB overpass structure is adjacent to the north side of the existing QEW WBL structure that was constructed in 1980.

The proposed Highway 403 W-E Ramp overpass for the QEW eastbound will be addressed under a separate cover.

Based on preliminary GA drawing dated October 2015, the new overpass will be an approximately 24 m and 36 m long single span structure. The Highway 403 at the bridge site is at about



elevation 139 to 141 and the grades of the QEW WBL at the overpass will vary from about elevation 147.6 to 148.3.

All the elevations presented in this report are in meters.

7. FOUNDATIONS

7.1 Existing Site Conditions

The foundation options discussed in this report consider the presence of bedrock at shallow depths. This condition resulted from the lowering of the grades to accommodate the Highway 403 W-E Ramp grades during the construction of the existing structures in the 1980's. The previous cuts penetrated into the weathered zones of the bedrock where the pavement of the Highway 403 W-E Ramp was constructed.

7.2 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.

7.3 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.



7.4 Foundation Alternatives

A comparison of the relative advantages and disadvantages related to each of the feasible foundation alternatives for the proposed rigid frame overpass and retaining walls is tabulated below.

Table 7.4: Comparison of Foundation Alternately

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 • Higher cost than spread footings • Temporary liners required 	<ul style="list-style-type: none"> • High 	<ul style="list-style-type: none"> • Complex installation may be required to excavate limestone bands
Piles	<ul style="list-style-type: none"> • Integral abutments • Higher load capacities 	<ul style="list-style-type: none"> • Deep rock excavation required to obtain 5 m free one for pile length 	<ul style="list-style-type: none"> • High due to requirement for excavation of bedrock 	<ul style="list-style-type: none"> • Complex installation may be required to excavate limestone bands
Piles in Drilled Socketed Holes	<ul style="list-style-type: none"> • Higher load capacities 	<ul style="list-style-type: none"> • Higher cost than spread footings • Temporary liners required 	<ul style="list-style-type: none"> • High 	<ul style="list-style-type: none"> • Complex installation may be required to excavate limestone bands



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.

Since the design calls for a rigid frame structure, and based on the encountered subsurface conditions including bedrock near the ground surface, it is considered that the most practical alternative is to construct the proposed new overpass foundations on spread footings placed on the bedrock.

With space restrictions and associated requirements for temporary roadway protection for construction of spread footings adjacent to the travelled lanes of Highway 403 W-E Ramp and potentially in close proximity to existing QEW 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.

7.5 Spread Footings on Bedrock

The existing ground surface along the Highway 403 W-E Ramp ranges from elevation 139.1 to 141.1. The proposed bedrock surface level within the footprints of the foundation elements ranges from elevation 138.0 to 139.9 at the west abutment and retaining wall and from elevation 139.3 to 139.8 at the east abutment and retaining wall.

The upper zone of the shale bedrock below the Highway 403 W-E Ramp is highly weathered and of poor quality to approximate levels ranging from elevations 137.7 to 139.2, with elevation 137.0 at borehole 17 where a previous excavation was likely carried out. Below these levels, the bedrock becomes slightly weathered and of fair to good quality. These levels are summarized in Table 7.5 for each of the boreholes and auger probes, together with the levels to provide the 1.2 m foundation frost protection to the spread footings.



Table 7.5: Proposed Overpass Foundations - Hwy 403 W - N Ramp Under QEW WBL

Overpass Component	Element	BH No.	Current Surface Elevation	Proposed Footing Subgrade Elevation from Stantec GA Drawings	Hard Silty Clay Shale Fragment (Highly Weathered Shale Bedrock)	Slightly Weathered Shale Bedrock	Minimum Level for Frost Protection (1.2 m Below Ground)
Ramp W-E Hwy 403 QEW WBL - West Abutment and Retaining Walls	Abutment	3(*)	139.4	138.0	143.8(**)	140.8 (**)	138.2
	Retaining Wall	16	141.0	139.8	140.1	139.2	139.8
	Abutment	17	140.2	138.5	N/A (***)	137.0 (***)	139.0
	Abutment	18	140.0	138.0	139.5	139.2	138.8
	Retaining Wall	AP-A	140.1	139.0	139.9	138.0	138.9
	Abutment	AP-B	139.7	138.5	139.5	138.5	138.5
	Abutment	AP-C	139.1	138.0	138.7	137.9	137.9
	Retaining Wall	AP-D	138.6	139.9	138.3	137.7	137.4
Ramp W-E Hwy 403 QEW WBL - East Abutment and Retaining Walls	Abutment	25	141.1	139.8	140.3	140.2	139.9
	Abutment	25A	142.0	139.8	141.3	140.8	140.8
	Retaining Wall	26	142.2	139.8	141.7	141.4	141.0
	Retaining Wall	27	142.5	139.8	141.8	140.4	141.3
	Abutment	AP-L	140.4	139.3	140.1	139.2	139.2
	Retaining Wall	AP-K	139.9	139.3	139.4	138.7	138.7
	Abutment	1(*)	140.4	139.3	145.5(**)	144.6(**)	139.2
	Abutment	2(*)	140.4	139.3	144.9(**)	144.0 (**)	139.2

Notes: (*) - Borehole surface elevations before previous excavations for present highway configuration:

BH No.	Elev.
3	145.9
1	147.6
2	147.0

(**) Relates to site conditions before previous excavations for present highway configuration.
(***) Probable previous local excavation.



7.5.1 Recommended Spread Footings Founding Levels

Considering that the requirements for foundation frost protection resulted in founding levels into or near the slightly weathered shale bedrock, the founding levels for spread footings in Table 7.5.1 are recommended for the overpass and associated retaining walls.

Table 7.5.1: Recommended Founding Levels for Spread Footings

Overpass Component	Foundation Element	Recommended Founding Level (*)		Foundation Bedrock Type
		Borehole	Elevation	
Ramp W-E Hwy 403 QEW WBL - West Abutment and Retaining Walls	West Abutment	3	138.0	Slightly weathered shale
		17 (**)	137.0	
		18	138.0	
		AP-B	138.5	
		AP-C	137.9	
	West Retaining Wall	16	139.8	Highly weathered shale
		AP-A	138.9	Slightly weathered shale
		AP-D	137.4	
Ramp W-E Hwy 403 QEW WBL - East Abutment and Retaining Walls	East Abutment	25	139.8	Slightly weathered shale
		25A	139.8	
		AP-L	139.2	
		1	139.2	
		2	139.2	
	East Retaining Wall	26	139.8	Slightly weathered shale
		27	139.8	
		AP-K	138.7	

Notes: (*) Foundation levels are bottom of footing, do not include the thickness of working slabs.

(**) Represents local condition due to previous local excavation.

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.



7.5.2 Recommended Geotechnical Resistances

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 structure elements:

Table 7.5.2: Recommended Geotechnical Resistances

Founding Stratum	Geotechnical Resistance (kPa)	
	Factored ULS	SLS
Highly Weathered Bedrock	800	600
Slightly Weathered Bedrock	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 6.7.4 of the CHBDC.

7.5.3 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 provides unfactored friction coefficients for each plane.



Table 7.5.3: 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, shear keys, 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.

7.5.4 Uplift and Lateral Resistance

Grouted dowels or rock anchors installed into the rock 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 weathered shale conditions in the boreholes. Higher bond stress, up to 700 kPa, is possible in sound shale, however they will require further rock core investigation at the specific anchor location selected. A minimum depth of embedment of 40 bar diameters 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 NSSP 999F29 for Dowels Into Concrete.



Should uplift resistance be required, pre-stressed rock anchors should be used to transfer the horizontal stresses into the rock mass. The rock anchors should be connected to the abutment spread footings.

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. This test should be taken to 150% of the design load or until there is a significant increase in the pull-out rate. In the latter case, the design load must be limited to 50% of the load at which the rate of pull-out increases.

Shear keys may be constructed into the upper zones of the slightly weathered bedrock to increase the sliding resistance of the foundations. The passive resistance of the bedrock should be computed using the parameters provided in Table 8 of this report.

7.6 Caisson Foundations

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 embedded into the slightly weathered shale bedrock account for resistance being developed by both end bearing and shaft adhesion and are tabulated below. Refer to Table 7.5 of this report for elevations of the bedrock surface at the foundation elements.

Table 7.6: Caisson Axial Resistances

Caisson Diameter (m)	Axial Geotechnical Resistance for Minimum Socket Length			
	3.0 m (Elevation 135.0 West Abutment) (Elevation 136.2 East Abutment)		4.5 m (Elevation 133.5 West Abutment) (Elevation 134.7 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 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.

7.7 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-A.

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 7.3.

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 parameters are recommended for design.

TABLE 8: Earth Pressure Parameters

Parameter	Granular A, Granular B Type II or Type III	Weathered Rock
Angle of Internal Friction, degrees	35	42
Unit weight, kN/m^3	22.8	23..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

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-A for an NSSP for design and construction of RSS systems.

9. APPROACH EMBANKMENTS

The approach embankments to the proposed overpass will be approximately 7.5 m high at the abutments where the previously existing overburden was cut approximately 6 m for the W-E Ramp construction. Away from the abutments, the fill will be approximately 1.5 m high over original ground. Since the embankment for the WBL structure is proposed on a new alignment, the typical thickness of the existing fill is not significant.

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 slope 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. 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 new alignment of the QEW WBL approach fill within 20 m of the abutments should be excavated down to native clayey silt to silty clay (locally up to 2.0 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 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 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 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.

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 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.



10. 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 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.

10.1 Roadway Protection Systems

We understand 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 and will remain in place after completion of Contract 1.

In addition, depending on the required depth of excavation, a roadway protection system may be necessary along the existing QEW WB 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.

11. GROUNDWATER CONTROL

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 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-A.

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.



12. CLOSURE

This report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng., Project Manager and MTO Designated Principal Contact.

Yours very truly,

Peto MacCallum Ltd.



Grigory O. Degil, PhD, P.Eng.
Senior Foundation Engineer



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

GD/CN:gd-mi



APPENDIX FDR-A

Standard and Non-Standard Specifications



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.