

**FOUNDATION INVESTIGATION AND  
DESIGN REPORTS  
PROPOSED RETAINING WALLS FOR  
EXISTING CULVERT AT STATION 10+369  
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET  
PETERBOROUGH, ONTARIO  
W.P. 581-93-00 CONTRACT NO. 2007-4005**

**GEOCRES NO. 31D-444**

**Prepared For:**

**UMA/AECOM ENGINEERING LIMITED**

**Prepared by:**

**SHAHEEN & PEAKER**

**Project: SPT1182A-04  
August 12, 2008**



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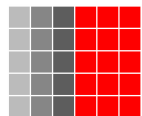
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## **1. INTRODUCTION**

Shaheen & Peaker Limited (S&P) was retained by UMA/AECOM Engineering Limited (UMA) to conduct a foundation investigation for proposed headwalls at the existing culvert location at Station 10+369, Highway 7, immediately east of Burnham Line intersection, Peterborough, Ontario.

The purpose of this investigation was to obtain subsurface information at the site by means of exploratory boreholes.

This report presents the findings of the geotechnical investigation.

## **2. PHYSIOGRAPHY**

The project site is located east of the junction of Highway 7/115 and Highway 7 near Peterborough, Ontario.

Based on the Physiography of Southern Ontario (by Putnam & Chapman), the project site is located within the Physiographic Region known as the Peterborough Drumlin Fields, which is notable for its eskers, as well as drumlins. While the general orientation of the drumlin axes in this field is from northeast to southwest, there are local variations worth noting. The Peterborough Drumlins are composed of limestone till, which may vary from highly calcareous till to angular limestone rubble, with the occurrence of boulders (many having a diameter of 600 to 900 mm and more numerous on or near the surface compared to deeper excavations) of Precambrian origin. The ridges of gravel are valuable as sources of road material, as other local sources of good quality gravel are rare. Characteristic shallow overburden soil types are expected to vary from sandy to clayey soils. In places where the hills are widely spaced, swamps may intervene.

Bedrock underlying this region is mostly limestone with minor dolostone and shale, of the Trenton and Black River Groups. These formations are approximately 480 million years old. They are highly fossiliferous and disintegrate easily.

The topography of Peterborough County is flat to gently rolling. The site is presently under construction for the rehabilitation and minor pavement/shoulder widening of Highway 7 from Highway 7/115.

### 3. INVESTIGATION PROCEDURES

The fieldwork at the site was carried out during the period of June 16-19, 2008 and, as shown on Drawing No. 1, consisted of drilling and sampling two boreholes (Boreholes 7 and 8).

Both boreholes were put down from the highway embankment, using a drilling rig, equipped with hollow-stem augers, supplied and operated by Eastern Soil Investigation Limited of Courtice, Ontario. The two boreholes were extended to depths of 11.3 and 10.8 m below the ground surface. Sampling in the boreholes was conducted at frequent intervals of depth by the Standard Penetration Test (SPT) method, as specified in ASTM D1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm o.d., split-barrel (split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground in the borehole by a distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil and this gives an indication of the consistency or the compactness condition of the soil deposits.

Below the bottom of Borehole 7, a Dynamic Cone Penetration test was performed. In Dynamic Cone Penetration Test (DCPT), a 51 mm diameter, 60 deg. apex cone point, screw-attached to the tip of A-size rods, is driven into the ground using the same driving energy as in the SPT method. By recording the number of blows to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of relative density/consistency is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT method and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic effects which in many cases affect the SPT values, especially in the fine-grained granular soils. The DCPT was terminated when the number of blows to drive the cone/rod assembly 0.3 m exceeded 100.

Water level observations in the open boreholes were made during drilling and at the completion of each borehole. At the completion of drilling, the boreholes were grouted and sealed using a cement/bentonite mixture.

The drilling, sampling and field testing operations were carried out under the supervision and direction of a Geotechnical Engineer from S&P.

The borehole locations were measured by S&P field staff in relation to stations and the centerline of the on-going highway construction. The geodetic elevations for the boreholes were provided to us by the surveyor for The Greer Galloway Group Inc, Contract Administrator at the site.

A laboratory testing programme, consisting of natural moisture content measurements, Atterberg Limits test and grain-size analyses, was performed on selected soil samples.

The results of drilling, in-situ testing and water level measurements, as well as laboratory test results, are summarized on the Record of Borehole Sheets in Appendix A. The results of grain-size analyses and Atterberg limits tests are also presented separately in Appendix B.

#### **4. SUBSURFACE CONDITIONS**

In general below some embankment fill and organic rich soils, the boreholes show the presence of a glacial till deposit consisting of sandy silt till to the full depth of the boreholes. In Borehole 8, however, a 1.9 m thick clayey silt deposit with thin clay seams was contacted sandwiched between the embankment fill and the sandy silt till.

Details of the stratigraphy encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The following paragraphs are only meant to complement and amplify these data.

##### **4.1 EMBANKMENT FILL**

Boreholes 7 and 8 were drilled from the roadway, which at the time of our investigation, was under construction (i.e. being widened). The boreholes encountered embankment fill, to depths of 2.3 m (El. 201.7 m) and 3.3 m (El. 200.3 m), respectively.

The upper portion of the embankment fill in Borehole 7 to a depth of 0.7 m consists of granular pavement fill (i.e. gravelly sand). Standard Penetration tests performed in this granular pavement fill yielded an N-value of 26 blows/0.3 m, indicating a compact condition. In this borehole, the granular pavement fill is underlain by a clayey silt to sandy silt embankment fill, with traces of gravel. Based on N-values of 17 and 19 blows/0.3 m, the fill is described as having a very stiff consistency in the cohesive zones and a compact relative density where it is primarily non-cohesive (i.e. granular).

In Borehole 8, a 25 mm thick layer of crushed stone layer had been placed to expedite the construction activities over the sandy silt embankment fill (which was found in the borehole). From the recorded N-values of 9 and 10 blows/0.3 m, the compactness condition of the fill is described as loose. The sandy silt fill extends to a depth of 1.5 m below the ground surface or to El. 202.1 m and is underlain by a 0.5 m thick sand fill layer. This fill layer extends to 2.0 m or El. 201.6 m and contains some gravel and silt. An N-value of 11 blows/0.3 m was recorded in this granular fill material and from this its relative density is described as compact.

The sand layer is underlain by a basically cohesive embankment fill to 3.3 m depth or to El 200.3 m. This lower embankment fill consists of clayey silt with some gravel and organic soil inclusions. A Standard Penetration Test performed in this lower fill zone yielded an N-

value of 2 blows/0.3 m. This indicates a very soft consistency, as well as leading us to believe that proper compaction was not applied when the material was first placed.

#### 4.2 TOPSOIL

In Borehole 7, the embankment fill is underlain by an approximately 0.4 m thick topsoil layer.

#### 4.3 CLAYEY SILT

Underlying the embankment fill at 3.3 m (El. 200.3 m), Borehole 8 contacted a 1.9 m clayey silt deposit, with thin clay seams, to a depth of 5.2 m or El. 198.4 m. The clay seams appeared to be typically 5 to 12 mm thick and also appeared to be highly plastic with a blocky structure, an indication of a probable sensitive soil.

Atterberg limits tests performed on three samples from this cohesive soil deposit yielded the following index values:

Liquid Limit:	30 - 35%
Plastic Limit	16 - 24%
Plasticity Index:	11 - 15%

As shown in the Plasticity Chart, presented in Figure B-1, Appendix B, these results are characteristic of clayey soils of low plasticity. The measured natural moisture contents range from 26 to 35%. These values are in excess of the measured liquid limit values or closer to the liquid limit in comparison with the Plastic Limit values, which indicates little or no pre-consolidations.

N-values recorded range from 4 to 8 blows/0.3 m. These values indicate a firm consistency.

Based on the field and laboratory test results, together with a visual and tactile examination of the soil samples, the deposit is considered to have a weak and compressible structure.

#### 4.4 SANDY SILT TILL

Beneath the embankment fill and the topsoil in Borehole 7 and the clayey silt layer in Borehole 8, a basically granular (non-cohesive) glacial deposit consisting of a heterogeneous mixture of silt and sand with some gravel and traces of clay particles was contacted, at depths 2.7 m/ El. 201.3 m and 3.3 m/El. 200.3 m, respectively. The deposit was found to extend to the full depth of the boreholes and possibly beyond (i.e. to a depth of about 11 m or El. 193 m).

Sieve analyses were performed on five samples from the deposit which show the following grain-size distribution:

Gravel:	7-19%
Sand:	32-40%
Silt & Clay:	45-53%

These results are presented in an envelope form in Figure B-2, in Appendix B.

It should also be pointed out that owing to their mode of deposition, the presence of cobbles and boulders should always be anticipated in the glacial till deposits.

Standard Penetration tests performed in Borehole 7, yielded an N-value of 5 within the upper 0.5 ±m, indicating a loose condition. Below this depth (e.g. below about El. 201.0 m) the recorded N-values are typically in the 16 to 26 blows/0.3 m range to the full borehole depth, indicating a compact condition. A dynamic cone penetration test performed from the bottom of the borehole encountered refusal (i.e. 100 blows for 5 cm penetration) about 1.9 m below or at El. 190.8 m.

In Borehole 8, the recorded N-values in the upper 1.5 ±m of the deposit (i.e. to El. 196.8 m) were 2 blows/0.3 m, followed by an N-value of 6 to about El. 196.0 m. These results indicate a very loose condition in the upper zone, followed by a loose condition. It should be pointed out that these results were obtained beneath an approximately 3 m surcharge (e.g. embankment fill) and that N-values before the placement of the embankment or (at present) outside the embankment fill area could even lower. Below this weak zone, the recorded N-value was found to increase to 16 blows/0.3 m (i.e. a compact condition) followed by high values of in excess of 100 blows/0.3 m below about El. 194.5 m.

#### 4.5 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were measured at depths of 8.2 and 8.4 m or at about El. 195.5 m. These values were, however, obtained upon completion of the boreholes and are therefore believed not to be representative of stabilized groundwater levels. Based on the observations made at the site and the moisture conditions of the samples, the groundwater table at the time of our investigation expected to be near the o.g. levels or at about El. 200 to 201 m.



It should however be pointed out that the groundwater at the site would be subject to seasonal fluctuations and fluctuations in response to major weather events. As well it would be influenced by the water level in the watercourse at the site.

**SHAHEEN & PEAKER LIMITED**



Ramon Miranda, P.Eng.



Z.S. Ozden, P.Eng.



K. R. Peaker, Ph.D., P.Eng.



ZO:tr/drive

# Drawings

METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
ARE IN KILOMETRES + METRES.

NOTES:  
FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

CONT No 2007-4005

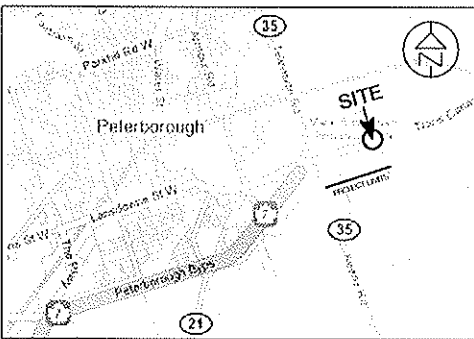
WP No 581-93-00

HIGHWAY 7, PETERBOROUGH  
CULVERT @ 10+369  
BOREHOLE LOCATIONS



SHEET

SHAHEEN & PEAKER LIMITED



KEY PLAN  
N.T.S.

LEGEND

Borehole

No.	ELEV.	STATION NO
BH-7	204.03	10+374
BH-8	203.60	10+365

NOTE

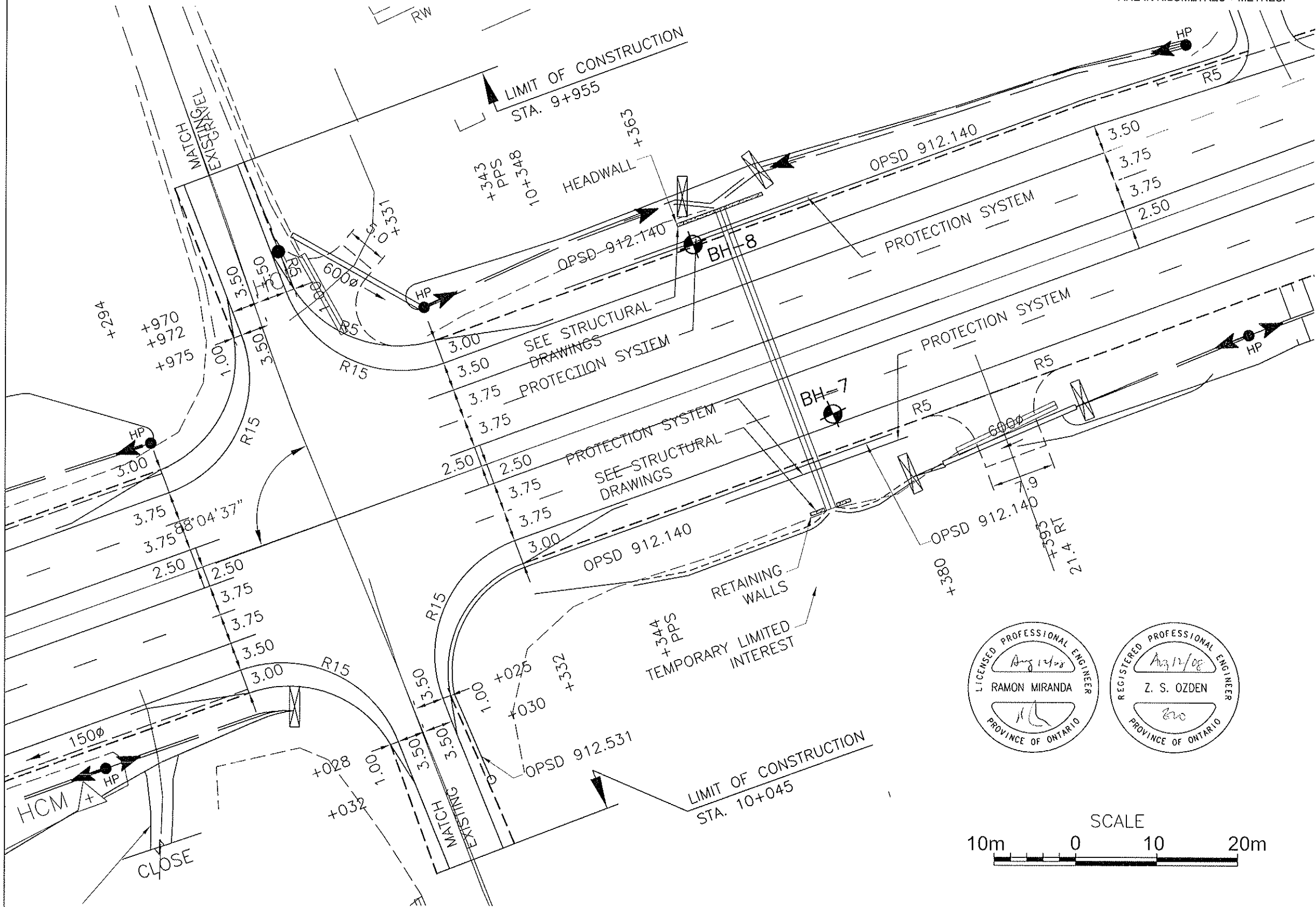
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: This drawing is for subsurface information only. Subsurface details and features are for conceptual illustration.

REV.	DATE	BY	DESCRIPTION

Geocres No. 31 D-444

SUBM'D		CHECKED	DATE JUN 2008	SITE
DRAWN PK		CHECKED RM	APPROVED ZO	DWG 1



BOREHOLE LOCATION PLAN

METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
ARE IN KILOMETRES + METRES.

NOTES:  
FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

CONT No 2007-4005

WP No 581-93-00

HIGHWAY 7, PETERBOROUGH  
CULVERT @ 10+369  
STRATIGRAPHY



SHEET

SHAHEEN & PEAKER LIMITED



KEY PLAN  
N.T.S.

LEGEND

Borehole

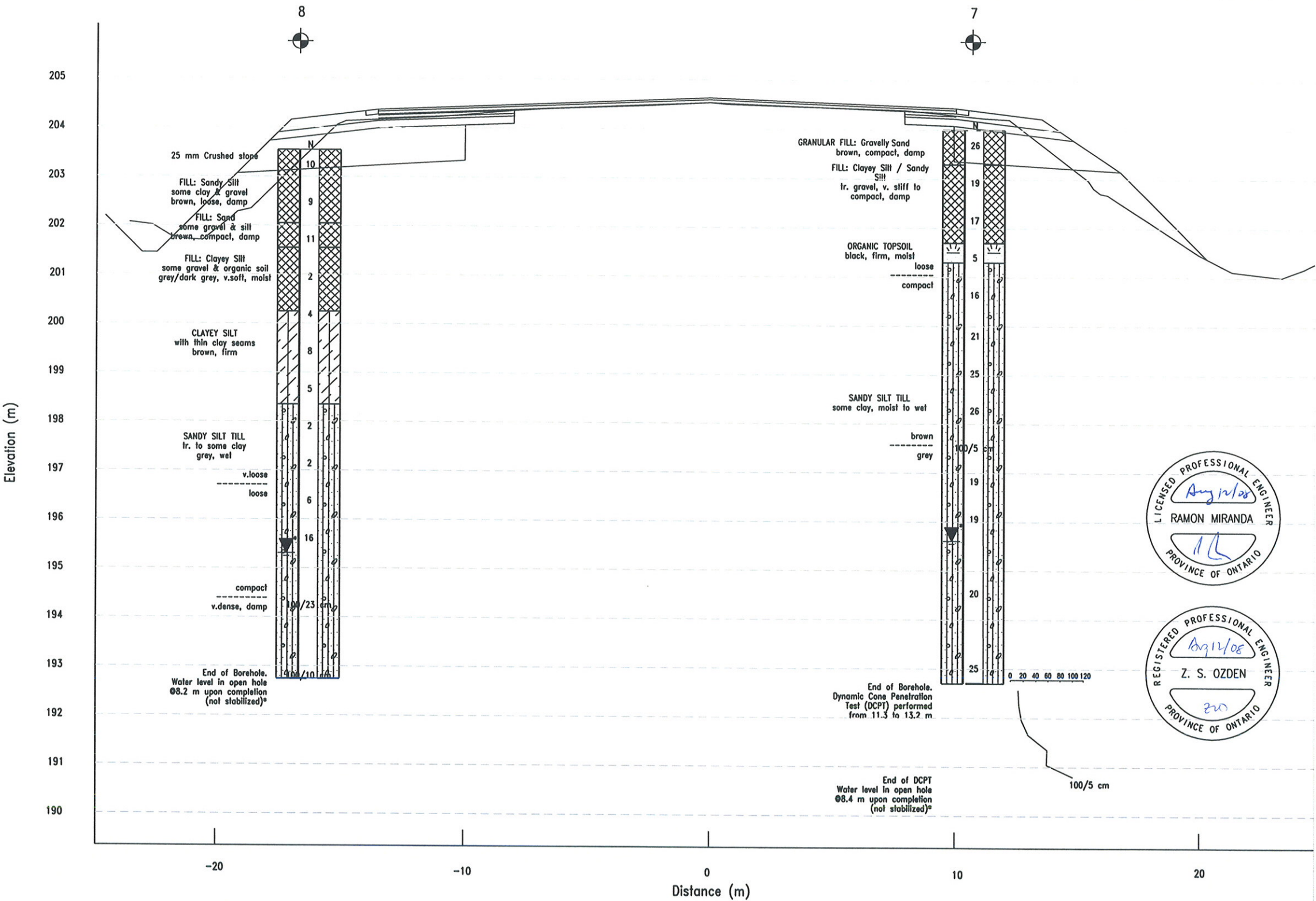
No.	ELEV.	STATION NO
BH-7	204.03	10+374
BH-8	203.60	10+365

NOTE

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

NOTE: This drawing is for subsurface information only. Subsurface details and features are for conceptual illustration.

REV	DATE	BY	DESCRIPTION
Geocres No.			
SPT 1182 A-04			DIST
SUBM'D	CHECKED	DATE JUN 2008	SITE
DRAWN PK	CHECKED RM	APPROVED ZO	DWG 2



STRATIGRAPHY

# Appendix A

## Record of Borehole Sheets



SPT1182A-04: Highway 7 (Peterborough)

# RECORD OF BOREHOLE No 7

1 OF 1

METRIC

GWP G.W.P. 173-98-00 LOCATION (Sta: 10+374) 10.5 m Rt C/L of Hwy 7, Peterborough ORIGINATED BY SK  
 DIST HWY 7 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS  
 DATUM Geodetic DATE 6/16/2008 6/19/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
204.0	GROUND SURFACE													
0.0	GRANULAR FILL: Gravelly Sand brown, compact, damp		1	SS	26									
203.3														
0.7	FILL: Clayey Silt / Sandy Silt tr. gravel, v. stiff to compact, damp		2	SS	19									
			3	SS	17									
201.7														
2.3	ORGANIC TOPSOIL black, firm, moist		4	SS	5									
201.3														
2.7														
			5	SS	16									
			6	SS	21									
			7	SS	25									
			8	SS	26									
			9	SS	100/5 cm									
			10	SS	19									
			11	SS	19									
			12	SS	20									
			13	SS	25									
192.7														
11.3	End of Borehole. Dynamic Cone Penetration Test (DCPT) performed from 11.3 to 13.2 m													
190.8														
13.2	End of DCPT Water level in open hole @8.4 m upon completion (not stabilized)*													

SPT1182A-04: Highway 7 (Peterborough)

# RECORD OF BOREHOLE No 8

1 OF 1

METRIC

GWP G.W.P. 173-98-00 LOCATION (Sta : 10+365) 15.0 m LI C/L of Hwy 7, Peterborough ORIGINATED BY SK  
 DIST HWY 7 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS  
 DATUM Geodetic DATE 8/16/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
203.6	GROUND SURFACE						20	40	60	80	100		
0.0	25 mmCrushed stone FILL: Sandy Silt some clay & gravel brown, loose, damp		1	SS	10								
			2	SS	9								
202.1													
1.5	FILL: Sand some gravel & silt brown, compact, damp		3	SS	11								
201.6													
2.0	FILL: Clayey Silt some gravel & organic soil grey/dark grey, v soft, moist		4	SS	2								
200.3			5	SS	4								
3.3	CLAYEY SILT with thin clay seams brown, firm		6	SS	8								
			7	SS	5								
198.4			8	SS	2								
5.2	SANDY SILT TILL tr. to some clay grey, wet		9	SS	2								
			10	SS	6								
			11	SS	16								
			12	SS	100/23 cm								
192.8			13	SS	100/10 cm								
10.8	End of Borehole Water level in open hole @8.2 m upon completion (not stabilized)*												

+ 3, X 3: Numbers refer to  
Sensitivity

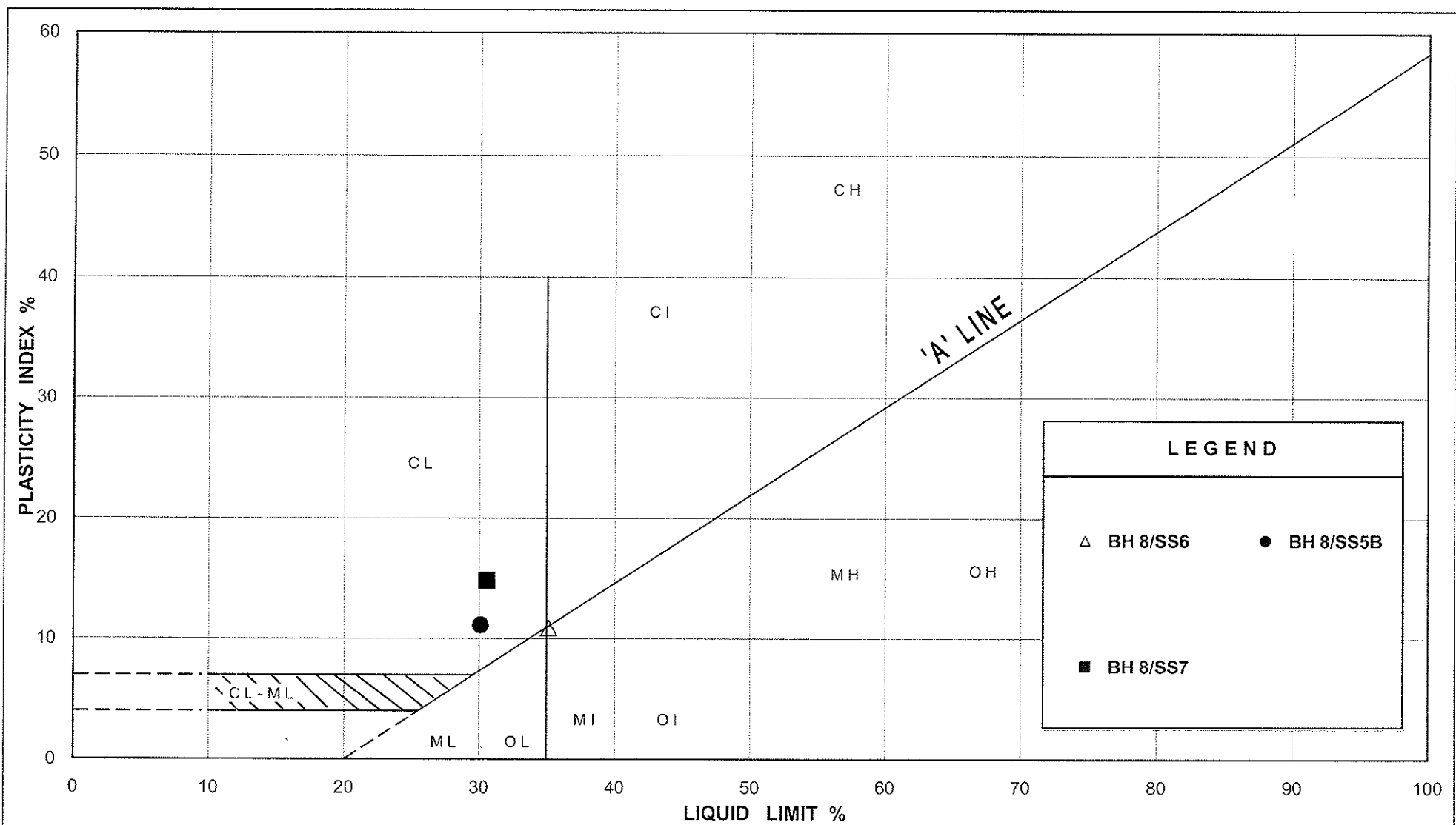
20  
15  
10

(%) STRAIN AT FAILURE

# Appendix B

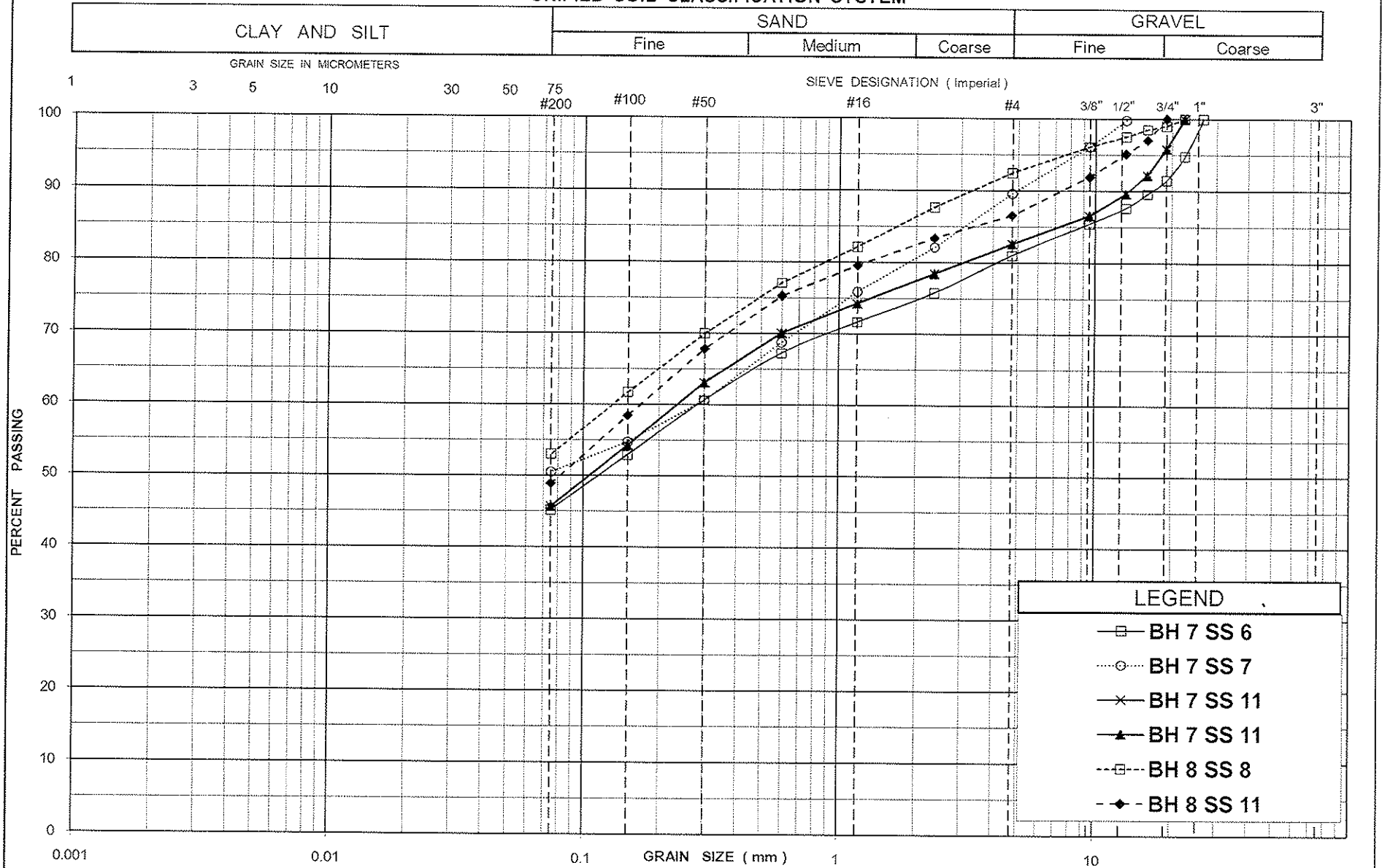
## Laboratory Test Results





SHAHEEN & PEAKER LIMITED  A COFFEY GEOTECHNICS COMPANY	PLASTICITY CHART		FIGURE No. B-1
	CALYHEY SILT with thin clay seams		REF. No. SPT 1182 A-04
			DATE June 27, 2008

# UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER LIMITED  
A Coffey Geotechnics Company

GRAIN SIZE DISTRIBUTION  
SANDY SILT TILL

FIGURE No. B-2  
REF. No. SPT 1182 A-04  
DATE July 2008

# Appendix C

## Site Photographs



Photograph 1. Borehole 7 (looking west)



Photograph 2. Borehole 8 (looking east)

## Appendix D

# Explanation of Terms Used in Report

## 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 475J 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.

**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 STRUCUTRAL 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 (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINT 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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICALL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$c_c$	1	COMPRESSION INDEX
$c_s$	1	SWELLING INDEX
$c_a$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $c_u / \tau_r$

## PHYSICAL PROPERTIES OF SOIL

$P_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$j_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$P_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$j_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$s_r$	%	DEGREE OF SATURATION	$D_n$	mm	N PERCENT – DIAMETER
$P$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$j$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$P_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$j_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $(W_L - W_p) / I_p$	v	m/s	DISCHARGE VELOCITY
$P_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDAULIC GRADIENT
$j_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_c$	1	CONSISTENCY INDEX = $(W_L - W) / 1_p$	k	m/s	HYDRAULIC CONDUCTIVITY
$P'$	kg/m <sup>3</sup>	DENSITY OF SUBMERED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>3</sup>	SEEPAGE FORCE
$j'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT  
PROPOSED RETAINING WALLS FOR  
EXISTING CULVERT AT STATION 10+369  
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET  
PETERBOROUGH, ONTARIO  
W.P. 581-93-00 CONTRACT NO. 2007-4005**

**GEOCRES NO. 31D-444**

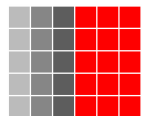
**Prepared For:**

**UMA/AECOM ENGINEERING LIMITED**

**Prepared by:**

**SHAHEEN & PEAKER**

**Project: SPT1182A-04  
August 12, 2008**



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### APPENDIX E: LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT  
PROPOSED RETAINING WALLS FOR EXISTING CULVERT AT STATION 10+369  
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET,  
PETERBOROUGH, ONTARIO  
W. P. NO. 581-93-00 CONTRACT NO. 2007-4005**

**5. DISCUSSION AND RECOMMENDATIONS**

Improvements to Highway 7, east of Peterborough, includes, under Contract No. 2007-4005, the construction of headwalls at the inlet and outlet of the existing culvert at Station 10+369.

In general, Borehole 7 on the south side shows, below some embankment fill and underlying an organic topsoil layer, the presence of a compact sandy silt till deposit to the full depth of the borehole. Borehole 8, however, presents a different picture on the north side. In this borehole, below the embankment fill, a clayey silt deposit was contacted to El. 198.4 m. This approximately 2 m thick layer is weak and compressible (i.e. firm consistency). Below this a sandy silt till was contacted (similar to Borehole 7 location). However, in this borehole the upper 2.5 ±m of the till deposit is very loose to loose, becoming compact below El. 195 ±m and then very dense with increasing depth.

The groundwater table at the site is believed to be at or near the o.g. levels at about El. 200 to 201 m, but would be subject to fluctuations.

From the data supplied to us, the invert of the existing culvert is at El. 201.4 ±m on the north side and 201.1 ±m on the south side. We understand that a reinforced concrete headwall is proposed and that the bottom of footings will be at about El. 200.2 m on the north side and El. 199.9 ±m on the south side.

It is also our understanding that the wall will be about 2.0 m high and the soil behind the wall will slope at 2H:1V up by about another 1.0 ±m to the shoulder of the existing highway embankment.

The following alternatives were considered to support the proposed headwall at this site.

### Summary of Foundation Alternatives Considered

Type of Wall	Comments	Recommendations
Reinforced Concrete Retaining Wall	Spread footing foundations suitable at Borehole 7 but unsuitable at Borehole 8.  Caisson foundations suitable at both borehole locations.	Dewatering requirements as well as possible temporary shoring would make the use of spread footings a poor choice. Caisson foundations would likely be economical if temporary shoring is not necessary during the construction. Recommended for consideration.
Retained Soil System (RSS)	May be suitable if scour is not a problem.	Unlikely to be economical if temporary shoring is required. Will require dewatering.
Permanent Soldier Pile & Lagging System	Less sensitive to dewatering than the other two alternatives. Will not require temporary shoring. Scour should be checked.	Recommended for serious consideration. Will require dewatering but less than most other options. Recommended for consideration.
Gabion or Armour Stone Wall	Can be considered if some distortions and lateral yield can be acceptable.	Economical but may not be reliable due to scour and also due to lateral yield which may cause some cracking in the embankment.

## 5.1 REINFORCED CONCRETE RETAINING WALL

The present design is based on a reinforced concrete retaining wall supported on shallow spread footing foundations. The borehole data show that while the use of normal spread footing foundations is feasible on the south side based on Borehole 7 findings, the soil encountered on the north side in Borehole 8 is unsuitable (i.e. too weak and compressible) for the use of shallow spread footing foundations to support a reinforced concrete retaining wall.

### 5.1.1 SPREAD FOOTING FOUNDATIONS

The use of spread footing foundations is feasible at Borehole 7 which was drilled from the top embankment on the south side. Here the bottom of footing elevation (as presently designed) is 199.9 m. At the borehole location the following geotechnical resistances are available at or below El. 200.5 m for footings placed on undisturbed natural compact sandy silt till.

$$\begin{aligned}\text{Factored Bearing Resistance at U.L.S.} &= 200 \text{ kPa} \\ \text{Bearing Resistance at S.L.S.} &= 130 \text{ kPa}\end{aligned}$$

The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively. To achieve this, the founding subgrade must be properly dewatered. Otherwise, it may be disturbed and dilate, leading to excessive settlements when structural loads are applied.

Under inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with CHBDC.

The structure will need to be checked against overturning and sliding, with an appropriate factor of safety. The unfactored horizontal resistance against sliding between poured concrete and approved sandy silt till subgrade surface can be calculated using a friction angle of 28 degrees.

All footing excavations will need to be certified as per SP 902S01.

Upon completion of the footing excavation, inspection and approval, a 150 mm thick layer of skim coat of concrete (mud slab) should be placed on the approved surface, without delay (i.e. within four hours) to prevent dilation and disturbance of the approved bearing subgrade.

As the groundwater table at the site is high and the sandy silt till is a dilatent material, it is recommended that the footing levels be kept as high as possible and that the construction be carried out during a relatively dry season, if at all possible.

On the north side, the design bottom of footing elevation is 200.2 m. Borehole 8 shows the presence of a clayey silt deposit at this elevation. The clayey silt is weak and compressible and is underlain by sandy silt till below El. 198.4 m. The upper zones of the till is also weak to about El. 196.0 m with recorded N-values of 2, 2 and 6 blows/0.3 m indicating a very loose condition becoming loose to about El. 196.5 m. The actual N-values beyond the toe of the embankment may be even lower, since the soil beneath the embankment would have gained strength under the embankment weight. A rigid reinforced concrete structure will likely undergo excessive settlements due to the poor foundation soils. For these reasons and considering the high water table and the dilatant nature of the soil, the use of spread footing foundations at this borehole location is not recommended.

#### 5.1.2 DEEP FOUNDATIONS

The very dense sandy silt till encountered below about El. 194.5 m in Borehole 8 would be suitable to support deep foundations. The presence of a similar competent deposit was inferred at Borehole 7 below culvert El. 191.5 m from a Dynamic Cone Penetration test (DCPT) performed from the bottom of this borehole.

The use of driven piles is considered to be uneconomical. As well, vibrations induced during the driving of the piles may be objectionable. However, the use of drilled and poured in placed concrete (caisson) foundations may be an attractive option for the project, especially if it is combined with other sites within the present contract (i.e., equipment and mobilization costs would be shared).

Caisson foundations socketed at least 1.0 m into the very dense soil (e.g. about El. 190.2 m at Borehole 7 and El. 193.3 m at Borehole 8) can be designed for the following geotechnical resistances.

Factored Bearing Resistance at U.L.S. = 1800 kPa  
Bearing Resistance at S.L.S. = 1200 kPa

During their installation, the caissons would require the use of temporary steel casing to enable the bases to be properly cleaned of any disturbed soils and to enable the inspection and approval of the base by the engineer. For these reasons, the minimum caisson diameter should be 0.76 m. QVE inspection should be implemented as per SP903 S01. The steel casing would be carefully withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent a 'necking' condition.

Higher bearing resistances are available if the caissons are socketed further into the very dense soil. This, however, is unlikely to be necessary and further more is not recommended since the base of the caisson would be subject to upward migration of groundwater, resulting in disturbance and loss of strength of the subgrade. For the same reason concrete must be poured immediately after augering, inspection and the approval of the caisson base, without undue delay.

The Contractor should be advised that cobbles and boulders may be encountered in the glacial till deposits.

## 5.2 RETAINED SOIL SYSTEM (RSS)

In principle, a retained soil system consists of fastening vertical facing units into a soil mass, with their tensile strips. It consists of four elements:

- A soil backfill
- Tensile reinforcing strips
- Facing elements at boundaries
- Mechanical connections between reinforcing elements

The soil backfill is generally a granular material with not more than 10 to 15% by weight passing #200 mesh size sieve. It should not contain materials corrosive to reinforcing strips. Within the reinforced zone, the soil is able to stand at much steeper slopes than possible without reinforcing.

This is a patented method and the provider of the system normally guarantees its stability.

This system should have a high appearance and medium performance MTO rating.

A MESA type wall (provided by Tensar) or its equivalent (must be on MTO's approved list) would likely be suitable. Depending on the details, this type of wall would be placed on a reinforced granular pad.

The granular pad would be constructed from Granular 'B' Type II soil. The pad would likely extend about 0.6 m beyond the perimeter of the footprint of the wall and would include the placement of a biaxial reinforcing geo-grid such as Terrafix BX 1200 or equivalent. The thickness of the granular pad would be 350 mm and the subgrade would consist of sufficiently competent natural soil. After the excavation to 450 mm below the bottom of the proposed wall elevation, the exposed subgrade would be inspected, evaluated and approved by the Geotechnical Engineer appointed by QVE and a 100 mm thick layer of skim coat of concrete would be placed on the approved surface without undue delay. On top of the concrete mud coat, a 100 mm thick Granular B Type II soil would be placed, overlain by the geo-grid and the balance of the granular material (i.e. 250 mm). The wall would be placed on this mat. The highest elevation of suitable subgrade for this purpose at the borehole locations are as follows:

Borehole No.	Existing Ground Elevation (m)	Highest Suitable Subgrade Elevation (m)
7	204.0	201.2
8	203.6	200.0

Proper dewatering would be required during construction to facilitate the construction and to preserve the load carrying capability of the subgrade soil supporting the reinforced mat.

This type of support may undergo greater than normal settlement at Borehole 8 location, say about 50 mm, depending on the details. As well, scour may need to be considered. In addition, it will likely require a temporary support system to facilitate the construction. In consideration of these and the anticipated dewatering requirements, this system may not present a cost-effective solution. We will however be pleased to further discuss its details, if you wish us to do so. As well, we recommend that its details be also discussed with a specialized contractor, who is on MTO's approved list.

### 5.3 SOLDIER PILE AND LAGGING SYSTEM

A permanent soldier pile and lagging system is another alternative. This consists of installing a caisson type hole at about every 2.5 to 3.0 m and inserting a steel I-beam. The section below the ground surface level to be supported (e.g. below the o.g. level) is filled with concrete, in the same manner as a temporary system. As such a temporary system would likely be required for the construction of alternative wall systems (such as reinforced concrete wall or a retained soil system), this permanent type system would likely present an economical and attractive solution, since it will serve to retain the soil during its construction. Permanent (concrete) lagging is provided between the caissons in order to support the

retained soil. Due to the low height of the retained embankment soil, tiebacks are unlikely to be required. The concrete facing will need to be scour and pollution (e.g. acid water flowing in the ditch, salt, elements, etc) resistant. The exposed portions of the steel I-beam will also need to be protected. Proper measures will need to be taken to prevent loss of soil between precast concrete lagging units (e.g. a suitable geotextile will need to be placed).

The dewatering required during construction would be much less strict than would be required for normal spread footing foundations as well as for an RSS system.

Proper permanent drainage measures will need to be provided similar to a normal retaining wall, to reduce earth pressures.

Machine access to install the caisson holes should be verified.

This type of retaining structure would be reliable, as well as being cost effective, especially if caissons are utilized in other areas, under the same contract administration and if they can be done at the same time. Another advantage of this method, as was mentioned before, temporary shoring will not be required. In other words, during its installation the structure itself will serve as the temporary support system. For these reasons, in our opinion, this method presents a good solution.

#### 5.4 GABION WALL/ARMOUR STONE WALL

Owing to the relatively low height of the embankment to be retained, consideration can be given to the use of a flexible gravity type wall such as the use of gabions or an armour stone wall, provided some lateral yield and distortions would be acceptable. This would present an inexpensive and yet expedient solution, but may not be acceptable due to scour considerations, as well as the lateral yield that may occur.

If acceptable this system would be placed on a granular pad constructed from Granular 'B' Type II soil. The pad would typically extend about 0.6 m beyond the perimeter of the footprint of the wall (this can be reduced to 0.3 m at the back where loads will be less) and would include the placement of a biaxial reinforcing geo-grid such as Terrafix BX 1200 or equivalent. A biaxial geo-grid is recommended because it would help distribute stresses over a larger area in both directions under the wall. The thickness of the granular pad should be 350 mm and the subgrade should consist sufficiently competent natural soil. After excavation to 450 mm below the bottom of the proposed wall elevation, the exposed subgrade should be inspected, evaluated and approved by the Geotechnical Engineer appointed by QVE and a 100 mm thick layer of skim coat of concrete should be placed on the approved surface without undue delay. On top of the concrete mud coat, a 100 mm thick Granular 'B' Type II soil would be placed, overlain by the geo-grid and the balance of the granular material (i.e. 250 mm). The wall would be placed on this mat. The highest elevation of suitable subgrade for this purpose at the borehole locations are as follows:

Borehole No.	Existing Ground Elevations (m)	Highest Suitable Subgrade Elevation (m)
7	204.0	201.3
8	203.6	200.3

Proper dewatering would be required during construction to prevent the disturbance of the subgrade soil supporting the reinforced mat.

These support systems are unlikely to be acceptable due to scour considerations but if required we will be pleased to discuss further details.

## 5.5 LATERAL EARTH PRESSURES

Backfill behind retaining walls should consist of non-frost susceptible, free draining granular materials in accordance with the Ontario Ministry of Transportation and Ontario Provincial Standards.

Granular backfill to be placed behind the retaining walls should conform to the minimum requirements illustrated in OPSD 3101.150 or OPSD 3120.100, whichever is applicable. The granular backfill should conform to OPSS 1010 for either Granular 'A', 'B' Type I or Type II. To maintain free draining characteristics in these granular fill materials, the maximum percentage passing the No. 200 sieve (75  $\mu$ m) should be limited to 5%.

The backfill should be placed in accordance with OPSS 501. A perforated subdrain should be installed behind the base of the walls as shown in OPSD 3101.150 to maintain the granular fill in a drained condition. The subdrain should be directed to a positive outlet. The position of the subdrain should be selected in consideration with the groundwater level and the water level in the existing watercourse.

Computation of earth pressures acting against the retaining wall should be in accordance with the current addition of the Canadian Highway Bridge Design Code, (CHBDC). For design purposes, the following properties can be assumed for backfill.

### **Compacted Granular 'A' and Granular 'B' Type II**

Angle of Internal Friction  $\phi=35^\circ$  (unfactored)

Unit weight = 22 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressure:



Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.27$	$K_a=0.34$	$K_a=0.40$
$K_b=0.35$	$K_b=0.44$	$K_b=0.50$
$K_o=0.43$	$K_o=0.56$	$K_o=0.62$
$K^*=0.45$	$K^*=0.60$	$K^*=0.66$

### Compacted Granular 'B' Type I

Angle of Internal Friction  $\phi=32^\circ$  (unfactored)

Unit Weight = 21 kN/m<sup>3</sup>

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.31$	$K_a=0.42$	$K_a=0.54$
$K_b=0.41$	$K_b=0.52$	$K_b=0.64$
$K_o=0.47$	$K_o=0.66$	$K_o=0.76$
$K^*=0.57$	$K^*=0.74$	$K^*=0.86$

NOTE:

$K_a$  is the coefficient of active earth pressure

$K_b$  is the backfill earth pressure coefficient for an unrestrained structure including compaction efforts

$K_o$  is the coefficient of earth pressure at rest

$K^*$  is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

These values are based on the assumption that the backfill behind the retaining structure is free-draining granular material and adequate drainage is provided.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. In the case of a rigid structure where yielding is unlikely, at rest pressures should be used, as per Clause 6.9.2 of CAN/CSA-S6-06 CHBDC. The effect of compaction during construction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6.9.2 of CAN/CSA-S6-06 CHBDC. The use of vibratory compaction equipment behind the retaining walls should be restricted in size as per current MTO practice. Vibration generated by traffic should also be considered in the selection of appropriate earth pressure coefficients.

## 5.6 CONSTRUCTION

The excavation should be carried out in accordance with the Occupational Health and Safety Act, Reg 213/91, as well as the following specifications:

SP105 S19 – Protection Systems

SP902 S01 – Excavation and Backfilling to Structures

The boreholes show that the excavations can be expected to extend through some embankment fill followed by sandy silt till (Borehole 7) or clayey silt underlain by sandy silt till (Borehole 8). These soils can be classified as follows:

Granular Embankment (Pavement) Fill:	Type 3 soil above water level Type 4 soil below water level
Sandy Silt to Clayey Silt Embankment Fill:	Type 3 soil above water level Type 4 soil below water level
Clayey Silt	Type 3 soil above water table Type 4 soil below water table
Sandy Silt Till	Type 3 soil above water table Type 4 soil below water table (if the soil was not dewatered)

Dewatering will be required during the construction to stabilize the soil and to prevent its dilatation. It is our opinion that the groundwater level can be lowered by up to about 0.6 m by means of gravity drainage and pumping from strategically located filtered sumps, depending on the site conditions at the time of construction. Closely spaced deep filtered sumps may be required if deeper water level lowering is required. For more than about 0.8 m water lowering, well points or deep wells may be required. For this reason, we recommend that, if possible, the construction be carried out during a dry period. As well, care should be taken to avoid disturbing the foundation soils by minimizing construction traffic (including foot traffic) and minimizing vibrations.

By means of careful construction and dewatering techniques the disturbance of this subgrade soil should be prevented, especially if spread footing foundations are employed.

We recommend that the Contractor be alerted to the fact that special care is needed to avoid disturbing the founding soils. As well, the Contractor should be required to submit their dewatering and excavation proposal to the CA for information purposes.

Temporary shoring may be required to support the excavations. The shoring should be designed by a Professional Engineer, experienced in this type of work. Shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this

case, the required performance level is considered 2. The coefficient of lateral earth pressures given in Table 5.5.1 can be used for the design of the shoring system.

Table 5.5.1  
Recommended Unfactored Parameters for Shoring Design

Soil Type	$K_a$	$K_o$	$K_p$	Unit Weight ( $\text{kN/m}^3$ )
Granular Fill	0.30	0.50	3.3	21.5
Organic Rich Soils	0.6	0.8	1.0	14.0
Clayey Silt	0.48	0.66	2.0	16.5
Sandy Silt Till dense to very dense	0.30	0.45	3.5	22.0
Sandy Silt Till Compact	0.33	0.50	3.0	20.5
Sandy Silt Till very loose to loose	0.38	0.55	2.6	19.5

## 5.7 FROST PROTECTION

Design frost protection depth for the area is 1.6 m. Therefore, a permanent earth cover of 1.6 m or its thermal equivalent of artificial insulation is required for frost protection of foundation, including pile caps (if any). In case of riprap or rock fill, only one half of the riprap thickness should be assumed to be effective in providing protection against frost heave.

## 6. CLOSURE

The Limitations of Report, as quoted in Appendix E, are an integral part of this report.

### SHAHEEN & PEAKER LIMITED



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ZO:tr/idrive

# Appendix E

## Limitations of Report

## **LIMITATIONS OF REPORT**

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

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

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

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

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