

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
PROPOSED HEADWALLS FOR
EXISTING CULVERT AT STATION 10+012
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET
PETERBOROUGH, ONTARIO
W.P. 581-93-00 CONTRACT NO. 2007-4005**

GEOCRES NO. 31D-443

Prepared For:

UMA/AECOM ENGINEERING LIMITED

Prepared by:

SHAHEEN & PEAKER

**Project: SPT1182A-01
August 12, 2008**



A Division of Coffey Geotechnics, Inc.

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DRAWINGS

DRAWING NO.

BOREHOLE LOCATIONS & SOIL STRATA

1 & 2

APPENDIX A: RECORD OF BOREHOLE SHEETS

APPENDIX B: LABORATORY TEST RESULTS

APPENDIX C: SITE PHOTOGRAPHS

APPENDIX D: EXPLANATION OF TERMS USED IN REPORT

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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+012, Highway 7, east of Highway 7/115, 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 investigation.

2. PHYSIOGRAPHY

The project site is located immediately east of the junction of Highway 7/115.

Based on the Physiography of Southern Ontario (by Putnam & Chapman), the proposed area is located within the Physiographic Region known as the Peterborough Drumlin Field 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 project site is presently under construction (contract No. 2007-4005) for the rehabilitation and minor pavement/shoulder widening of Highway 7.

3. INVESTIGATION PROCEDURES

The field investigation at the site was carried out on June 11 and 12, 2008 and, as shown on Drawing No. 1, consisted of putting down four boreholes.

Boreholes 1 and 2 were put down from the highway embankment, using a motorized drilling rig, equipped with hollow-stem augers, supplied and operated by Eastern Soil Investigation Limited of Courtice, Ontario. These boreholes were extended to depths of 11.0 and 9.2 m, respectively, 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 by a vertical 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.

Boreholes HD1 and HD2 were put down from the o.g. level, near the toe of the Highway embankment, using hand drilling methods, to depths of 1.2 and 1.4 m below the o.g. levels, respectively. In these boreholes a 31.8 kg hammer was used to drive the sampler, instead of the standard 63.5 kg hammer. The number of blows of the hammer to drive the sampler into the ground was divided by two to come up with an approximate (equivalent) N-value.

The drilling and sampling operations were carried out under the direction and supervision of a geotechnical engineer from S&P.

Water level observations in the open boreholes were made during drilling and at the completion of each borehole. At the completion of drilling, Boreholes 1 and 2 were grouted and sealed using a cement/bentonite mixture.

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 tests 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 an approximately 1.5 m thick clayey silt/silt deposit underlain by silt till and sandy silt till to the full depth of the boreholes.

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 1 and 2, drilled from the roadway, which was under construction (i.e. being widened), encountered embankment fill, to depths of 4.6 m (El. 198.2 m) and 3.9 m (El. 198.3 m), respectively.

The upper portion of the embankment fill to a depth of about 0.7 m consists of granular pavement fill (gravelly sand). Standard Penetration tests performed in the granular pavement fill yielded N-values of 14 and 20 blows/0.3 m, indicating a compact condition.

In general, underlying the granular pavement fill, the embankment fill was found to consist of sandy silt with some clay and gravel, which appears to have been derived from local sandy silt to silt and clayey silt glacial till deposits. Traces of organic soil were also found to be intermixed with the fill. In Borehole 2, a 0.7 m thick layer/pocket of sand and gravel was found at 1.4 m below the ground surface.

To within about 2.5 to 3.0 m below the ground surface, the recorded N-values range from 12 to 74 blows/0.3 m indicating that some systematic compaction was applied when the fill was first placed. Below this depth, to the bottom of the fill (i.e. to 4.6 m in Borehole 1 and 3.9 m in Borehole 2), the recorded values range from 2 to 9 blows/0.3 m, indicating that proper compaction was not applied. This may be because of difficulties that may have been experienced when building the highway while applying compaction over not so firm subgrade below or near the groundwater table.

4.2 PEAT, TOPSOIL, ORGANIC SILT

Up to about 0.6 m thick organic rich soil, consisting of peat, topsoil and organic silt was encountered at the ground surface level in Boreholes HD1 and HD2, which were drilled from the o.g. level, near the toe of the embankment. From recorded equivalent N-values of 1 blow/0.3 m, these soils are described as very soft.

Similar organic soils were found in Borehole 1, immediately below the embankment fill, from a depth of 4.6 m (El. 198.2 m) to 5.4 m (El. 197.4 m). An N-value of 5 blows/0.3 m was recorded indicating a firm to loose condition. The relatively higher N-value in this borehole

(in comparison with 1 blow/0.3 m in Boreholes HD1 and HD2) reflects the strength gain due to compression of the soil beneath the embankment fill.

4.3 CLAYEY SILT WITH SILT ZONES

Underlying the embankment fill or the organic rich soils, all four boreholes contacted a cohesive deposit consisting of clayey silt with silt zones, below El. 198.0 and 198.1 m in Boreholes HD1 and HD2, El. 198.3 m in Borehole 2 and El. 197.4 m in Borehole 1. In Boreholes 1 and 2, the thickness of the deposit was found to be 1.6 m and 1.4 m respectively (i.e. extended to El. 195.8 m and 196.9 m, respectively, while Boreholes HD1 and HD2 were terminated in this deposit at depths of 1.4 and 1.2 m below the ground surface or at El. 197.4 and 197.5 m.

Atterberg Limits tests performed in the laboratory on two samples from the deposit gave the following index values, as shown in Figure B-1 in Appendix B.

Liquid Limit:	26 - 32%
Plastic Limit:	17 - 22%
Plasticity Index:	9 - 10%

These values are characteristic of clayey soils of low plasticity. The measured natural moisture contents range from 18 to 27%.

N-values recorded in the deposit range from 4 to 20 blows/0.3 m, indicating a soft to very stiff consistency. Typically, the lower N-values were recorded in the upper zones and the consistency appears to be stiffer with increased depth.

From a visual examination of the soil samples the material is described as wet and dilatant.

4.4 SILT TO SANDY SILT TILL

The clayey silt to silt deposit, described in the preceding section, is underlain at 7.0 m (El. 195.8 m – BH1) and 5.3 m (El. 196.9 m – BH2) below the road embankment level by a glacial deposit consisting of silt to sandy silt till. Both boreholes were terminated in these till deposits at depths of 9.2 to 11.0 m, after penetrating it by about 4 m.

In general the deposit attains a somewhat coarser texture with depth. The upper 1.5 ± m of the deposit is described as a basically cohesive soil (i.e. silt till) and based on N-values of 11 and 18 blows/0.3 m, its consistency is described as stiff to very stiff. The deposit was noted to be wet and dilatant.

Atterberg Limits tests performed on a sample from this upper zone of the deposit yielded the following index values:

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Atterberg Limits tests performed on a sample from this upper zone of the deposit yielded the following index values:

Liquid Limit:	13%
Plastic Limit:	9%
Plasticity Index:	4%

These results are characteristic of cohesive soils of low plasticity, as shown in Figure B-2 in Appendix B.

Figure B-3 shows the grain-size distribution of a sample from the deposit, as follows:

Gravel:	13%
Sand:	37%
Silt:	37%
Clay:	13%

The lower portion of the deposit, which is somewhat coarser in texture (i.e. more sandy), is described as sandy silt till and is a basically granular material.

N-values recorded in this deposit are all in excess of 100 blows/0.3 m, indicating a very dense condition.

4.5 GROUNDWATER CONDITIONS

Upon their completion water levels in Boreholes 1 and 2 were recorded at 9.7 m (El. 193.1 m) and 8.1 m (El. 194.1 m), respectively. These levels, however, do not represent stabilized groundwater levels. Based on moisture contents of the samples, however, it is our opinion that the groundwater table at the time of our investigation was near the o.g. level or at about El. 198 \pm m.

It should be pointed out that the groundwater level at the site would be subject to seasonal fluctuations and fluctuations in response to major weather events. In addition the water table would be influenced by the water level in the existing water course.

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.

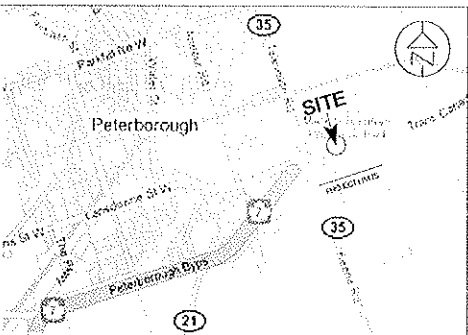
CONT No 2007-4005

WP No 581-93-00

HIGHWAY 7, PETERBOROUGH
CULVERT @ 10+012
BOREHOLE LOCATIONS



SHEET

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S.

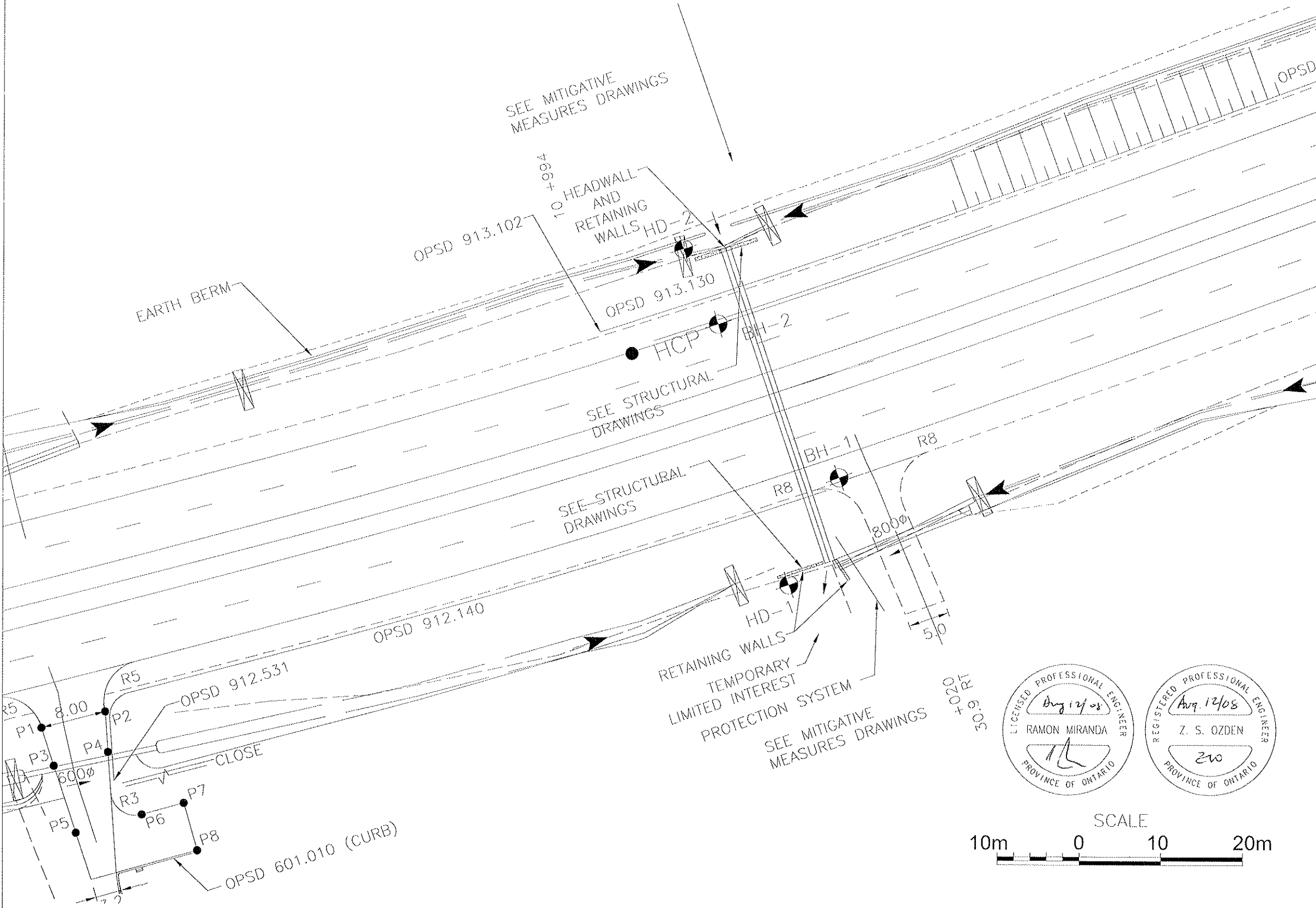
LEGEND

LEGEND		
 Borehole		
No.	ELEV.	STATION NO
BH-1	202.83	10+016
BH-2	202.23	10+008
HD-1	198.58	10+007
HD-2	198.89	10+006

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 0-443			
SP1 1182 A-01			DIST
SUBM'D	CHECKED	DATE JUN 2008	SHE
DRAWN PK	CHECKED RM	APPROVED ZO	DWG 1



BOREHOLE LOCATION PLAN

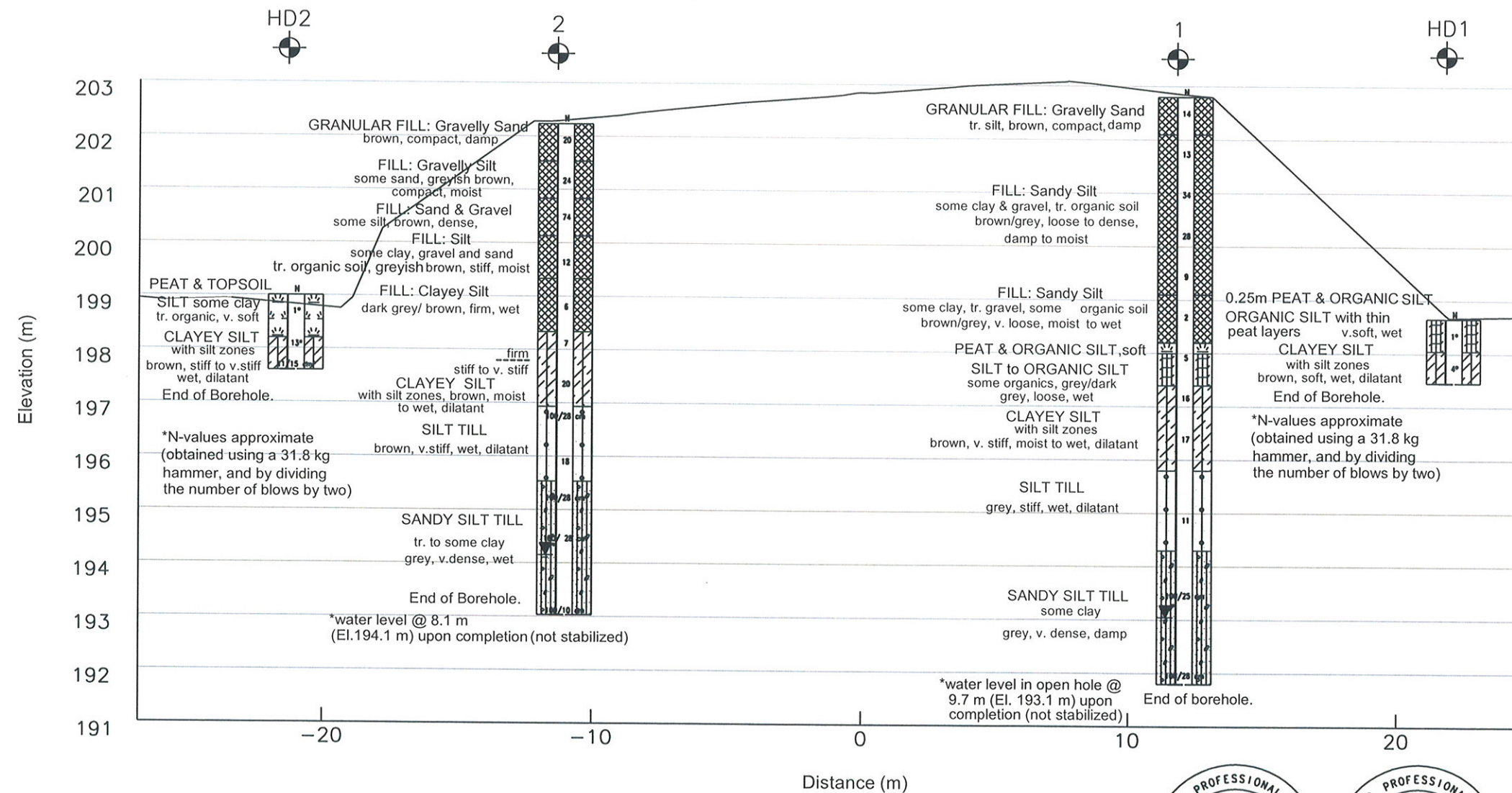
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
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NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.


HIGHWAY 7, PETERBOROUGH
Culvert @ 10+012
STRATIGRAPHY



SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND		
 Borehole		
No.	ELEV.	STATION NO
BH-1	202.83	10+016
BH-2	202.23	10+008
HD-1	198.58	10+007
HD-2	198.89	10+006

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-443				
SPT 11B2 A-01				DIST
SUBM'D	CHECKED	DATE JUN 2008	SITE	
DRAWN PK	CHECKED RM	APPROVED ZO	DWG 2	

STRATIGRAPHY



Appendix A

Record of Borehole Sheets

SPT1182A-01: Highway 7 (Peterborough)

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

GWP G.W.P. 173-98-00 LOCATION (Sta : 10+016) 12.5 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 6/11/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								WATER CONTENT (%)						
202.8 0.0	GROUND SURFACE													
202.1 0.7	GRANULAR FILL: Gravelly Sand tr. silt, brown, compact, damp		1	SS	14									
			2	SS	13									
	FILL: Sandy Silt some clay & gravel, tr. organic soil brown/grey, loose to dense, damp to moist		3	SS	34									
			4	SS	28									
199.1 3.7	FILL: Sandy Silt some clay, tr. gravel, some organic soil brown/grey, v. loose, moist to wet		5	SS	9									
198.2 4.6	PEAT & ORGANIC SILT, soft		6	SS	2									
198.0 4.8	SILT to ORGANIC SILT some organics, grey/dark grey, loose, wet		7	SS	5									
197.4 5.4	CLAYEY SILT with silt zones brown, v. stiff, moist to wet, dilatant		8	SS	16									
			9	SS	17									
195.8 7.0	SILT TILL grey, stiff, wet, dilatant		10	SS	11									
194.3 8.5	SANDY SILT TILL some clay grey, v. dense, damp		11	SS	100/25 cm									
191.8 11.0	End of borehole. *water level in open hole @ 9.7 m (El. 193.1 m) upon completion (not stabilized)		12	SS	108/28 cm									

+ 3 x 3. Numbers refer to
Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

SPT1182A-01: Highway 7 (Peterborough)

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

GWP G.W.P. 173-98-00 LOCATION (Sta : 10+008) 10.0 m LL C/L of Hwy 7, Peterborough ORIGINATED BY SK
 DIST HWY 7 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
 DATUM Geodetic DATE 6/12/2008 CHECKED BY ZO

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									
								○ UNCONFINED	+ FIELD VANE	● POCKET PENETR.	× LAB VANE	WATER CONTENT (%)					
							20	40	60	80	100		10	20	30		GR SA SI CL
202.2	GROUND SURFACE																
0.0	GRANULAR FILL: Gravelly Sand brown, compact, damp		1	SS	20												
201.5																	
0.7	FILL: Gravelly Silt some sand, greyish brown, compact, moist		2	SS	24												
200.8																	
1.4	FILL: Sand & Gravel some silt, brown, dense, damp		3	SS	74												
200.1																	
2.1	FILL: Silt some clay, gravel and sand tr. organic soil, greyish brown, stiff, moist		4	SS	12												
199.3																	
2.9	FILL: Clayey Silt tr. gravel, some organics soil mixture dark grey/ brown, firm, wet		5	SS	6												
198.3																	
3.9			6	SS	7												
			7	SS	20												
196.9	CLAYEY SILT with silt zones, brown, moist to wet, dilatant																
5.3																	
	SILT TILL brown, v. stiff, wet, dilatant		8	SS	100/28 cm												
195.5																	
6.7			9	SS	18												
	SANDY SILT TILL tr. to some clay grey, v. dense, wet		10	SS	100/28 cm												
193.0																	
9.2			11	SS	100/28 cm												
			12	SS	100/10 cm												
	End of Borehole. *water level @ 8.1 m (El. 194.1 m) upon completion (not stabilized)																

SPT1182A-01: Highway 7 (Peterborough)

RECORD OF BOREHOLE No HD1

1 OF 1

METRIC

GWP G.W.P. 173-98-00 LOCATION (Sta : 10+007) 23.0 m RI C/L of Hwy 7, Peterborough ORIGINATED BY SK
 DIST HWY 7 BOREHOLE TYPE Manual COMPILED BY SS
 DATUM Geodetic DATE 6/11/2008 CHECKED BY ZO

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						
198.6	GROUND SURFACE							20	40	60	80	100		
0.0	0.25m PEAT & ORGANIC SILT ORGANIC SILT with thin peat layers v. soft, wet		1	SS	1*									
198.0														
0.6	CLAYEY SILT with silt zones brown, soft, wet, dilatant		2	SS	4*									
197.4														
1.2	End of Borehole *N-values approximate (obtained using a 31.6 kg hammer, and by dividing the number of blows by two)													

+ 3 . X 3 . Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT1182A-01: Highway 7 (Peterborough)

RECORD OF BOREHOLE No HD2

1 OF 1

METRIC

GWP G.W.P. 173-98-00 LOCATION (Sta : 10+006) 20.0 m Lt C/L of Hwy 7, Peterborough ORIGINATED BY SK
 DIST HWY 7 BOREHOLE TYPE Manual COMPILED BY SS
 DATUM Geodetic DATE 6/11/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
198.9	GROUND SURFACE							20	40	60	80	100					
0.0	PEAT & TOPSOIL SILT some clay tr. organic, v. soft		1	SS	1*												
198.1																	
0.8	CLAYEY SILT with silt zones		2	SS	13*		198										
197.5	brown, stiff to v. stiff, wet, dilatant		3	SS	11/15 cm*												
1.4	End of Borehole. End of Borehole *N-values approximate (obtained using a 31.8 kg hammer, and by dividing the number of blows by two)																

+ 3. X 3

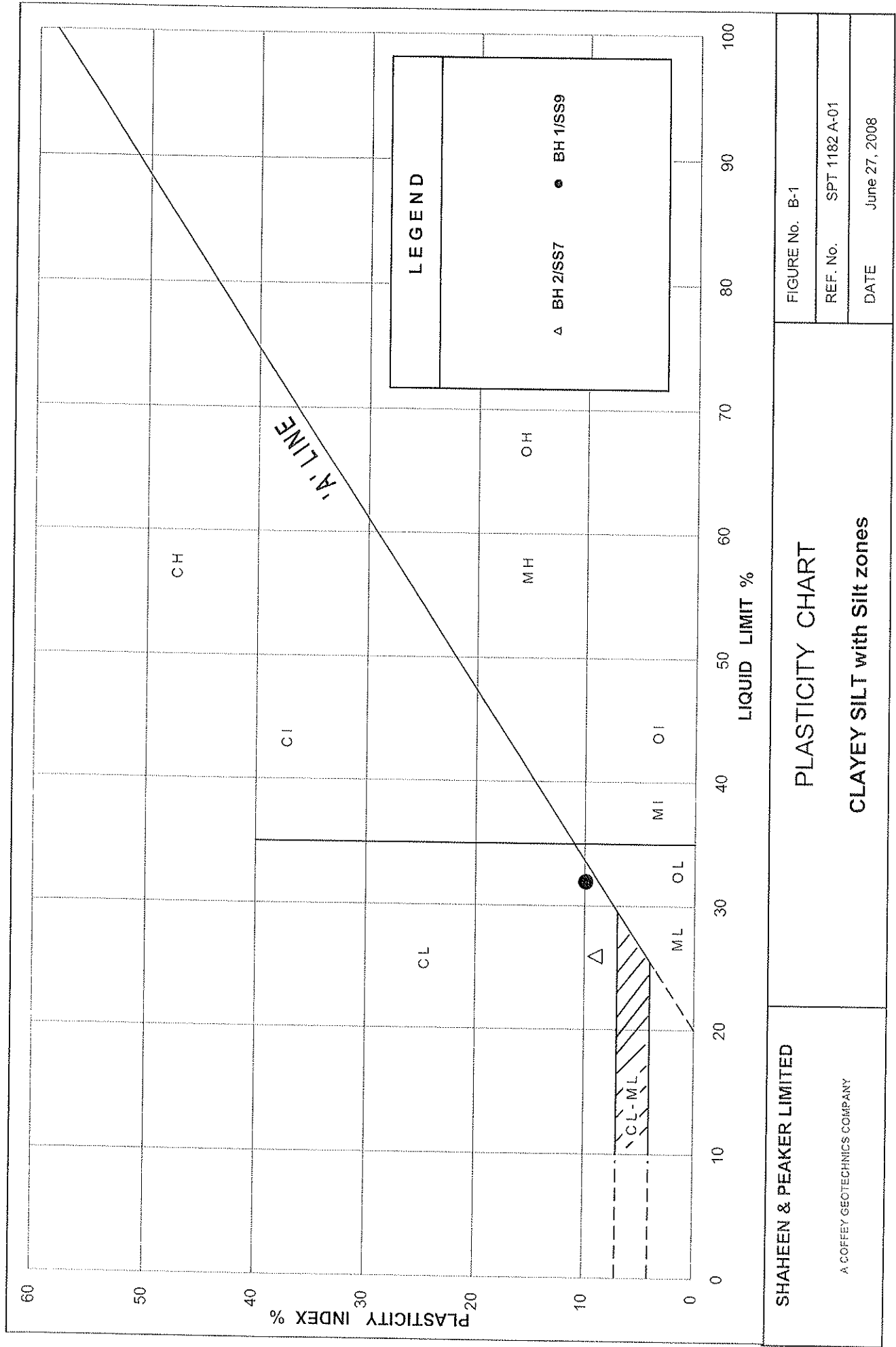
Numbers refer to
Sensitivity

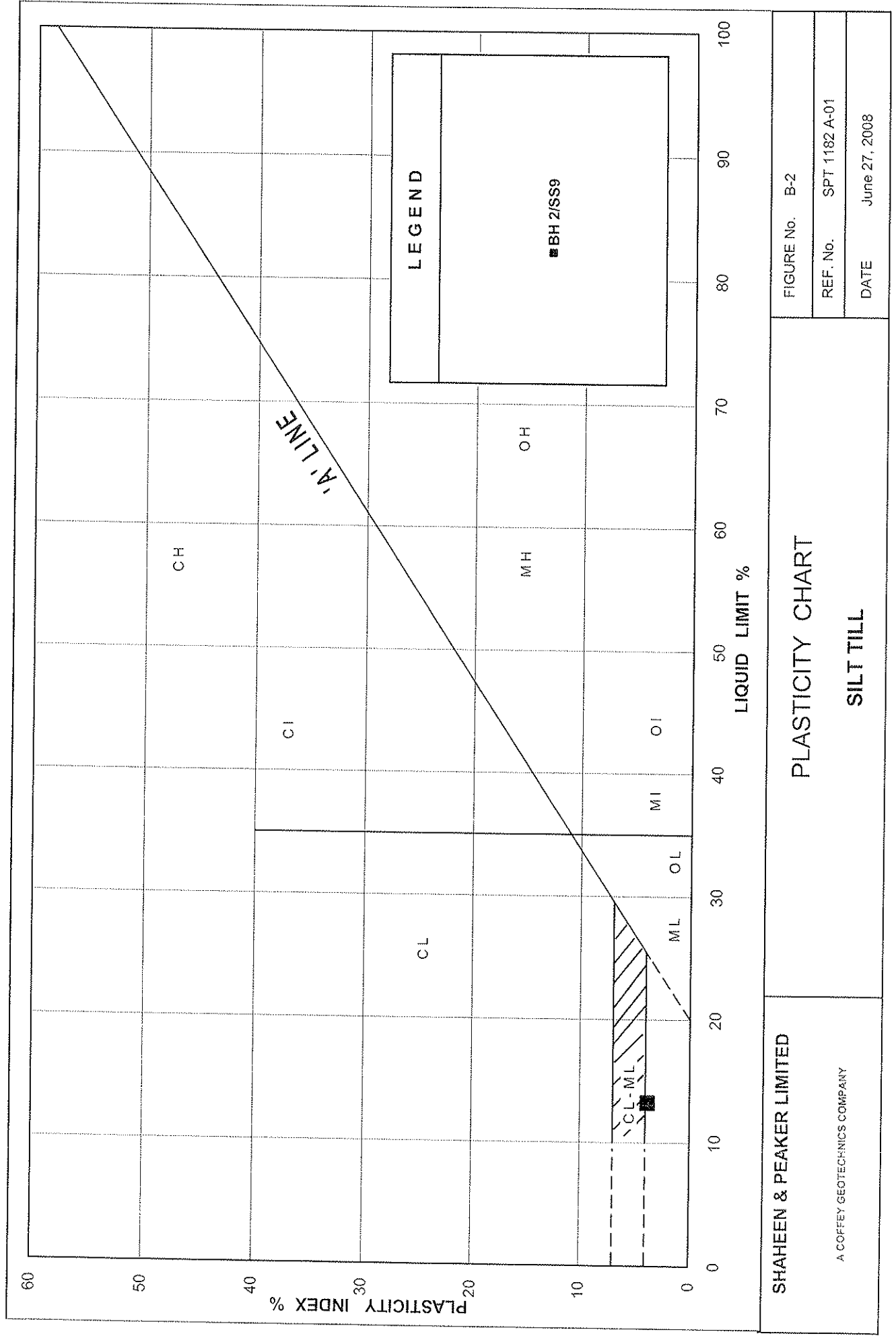
20
15
10

(%) STRAIN AT FAILURE

Appendix B

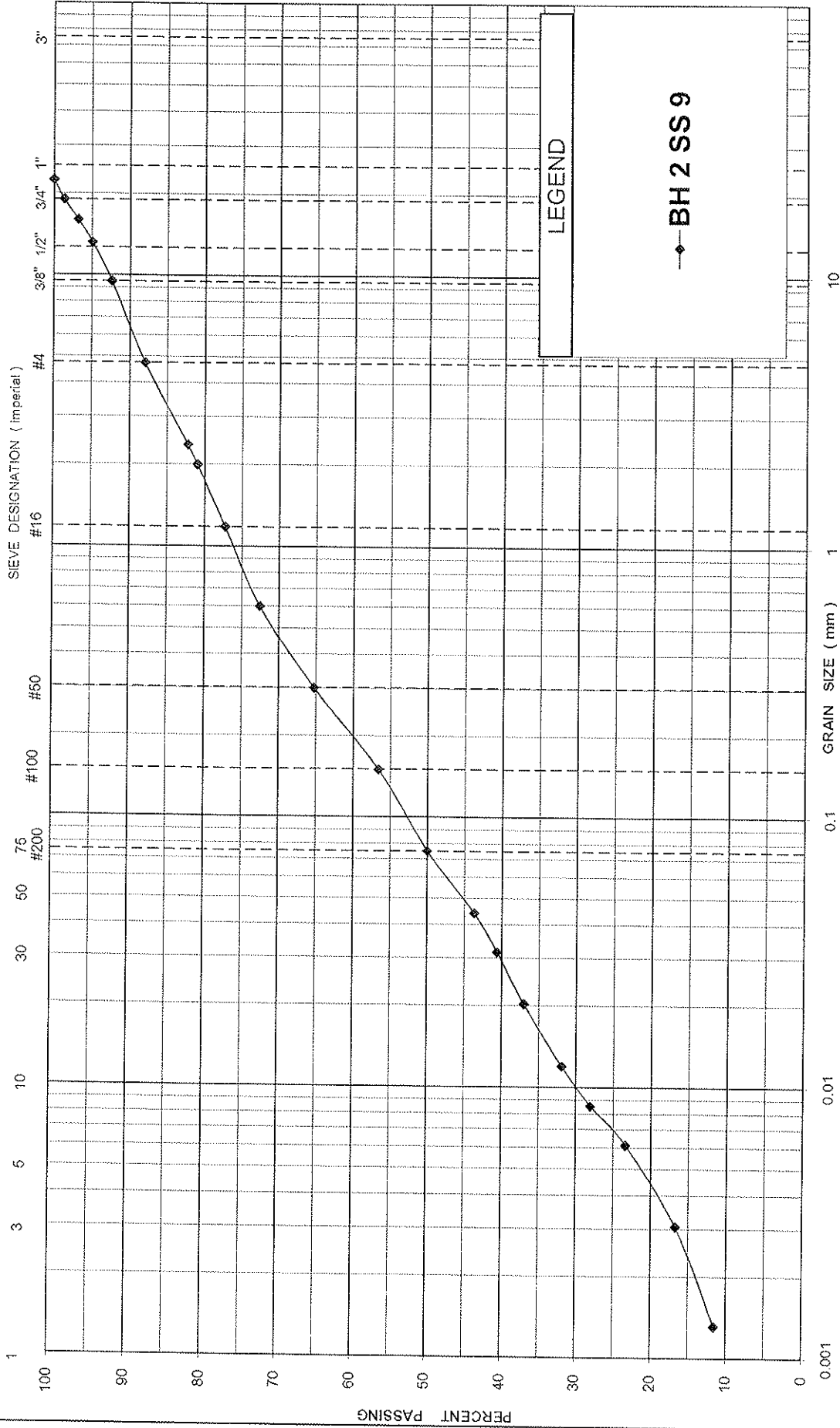
Laboratory Test Results





UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS		Fine	Medium	Coarse	Fine	Coarse	



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A COFFEY GEOTECHNICS COMPANY

GRAIN SIZE DISTRIBUTION SILT TILL

FIGURE No. B-3

REF. No. SPT 1182A-01

DATE JUNE 2008

Appendix C

Site Photographs



Photograph 1. Borehole 1



Photograph 2. Borehole 2



Photograph 3. Borehole HD2

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 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 (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
WS WASH SAMPLE
ST SLOTTED TUBE SAMPLE
BS BLOCK SAMPLE
CS CHUNK SAMPLE
TW THINWALL OPEN

TP THINWALL PISTON
OS OSTERBERG SAMPLE
RC ROCK CORE
PH TW ADVANCED HYDRAULICALLY
PM TW ADVANCED MANUALLY
FS FOIL SAMPLE

STRESS AND STRAIN

u_w kPa PORE WATER PRESSURE
 r_u 1 PORE PRESSURE RATIO
 σ kPa TOTAL NORMAL STRESS
 σ' kPa EFFECTIVE NORMAL STRESS
 Ω 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
 f 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
 Ω kPa SHEAR STRENGTH
 c' kPa EFFECTIVE COHESION INTERCEPT
 i' ° EFFECTIVE ANGLE OF INTERNAL FRICTION
 c_u kPa APPARENT COHESION INTERCEPT
 i_u ° APPARENT ANGLE OF INTERNAL FRICTION
 Ω kPa RESIDUAL SHEAR STRENGTH
 $\hat{\Omega}$ kPa REMOULDED SHEAR STRENGTH
 S_t 1 SENSITIVITY = $c_u / \hat{\Omega}$

PHYSICAL PROPERTIES OF SOIL

P_s kg/m^3 DENSITY OF SOLID PARTICLES	e 1.0% VOID RATIO	e_{min} 1.0% VOID RATIO IN DENSEST STATE
γ_s kN/m^3 UNIT WEIGHT OF SOLID PARTICLES	n 1.0% POROSITY	i_D 1 DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w kg/m^3 DENSITY OF WATER	w 1.0% WATER CONTENT	D mm GRAIN DIAMETER
γ_w kN/m^3 UNIT WEIGHT OF WATER	s_r % DEGREE OF SATURATION	D_n mm N PERCENT – DIAMETER
P kg/m^3 DENSITY OF SOIL	w_L % LIQUID LIMIT	C_u 1 UNIFORMITY COEFFICIENT
γ' kN/m^3 UNIT WEIGHT OF SOIL	w_p % PLASTIC LIMIT	h m HYDRAULIC HEAD OR POTENTIAL
P_d kg/m^3 DENSITY OF DRY SOIL	w_s % SHRINKAGE LIMIT	q m^2/s RATE OF DISCHARGE
γ_d kN/m^3 UNIT WEIGHT OF DRY SOIL	I_p % PLASTICITY INDEX = $(w_L - w_p)$	v m/s DISCHARGE VELOCITY
P_{sat} kg/m^3 DENSITY OF SATURATED SOIL	i_L 1 LIQUIDITY INDEX = $(w - w_p) / I_p$	i 1 HYDRAULIC GRADIENT
γ_{sat} kN/m^3 UNIT WEIGHT OF SATURATED SOIL	I_c 1 CONSISTENCY INDEX = $(w_L - w) / I_p$	k m/s HYDRAULIC CONDUCTIVITY
P' kg/m^3 DENSITY OF SUBMERGED SOIL	e_{max} 1.0% VOID RATIO IN LOOSEST STATE	j kN/m^2 SEEPAGE FORCE
γ' kN/m^3 UNIT WEIGHT OF SUBMERGED SOIL		

**FOUNDATION DESIGN REPORT
PROPOSED HEADWALLS FOR
EXISTING CULVERT AT STATION 10+012
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET
PETERBOROUGH, ONTARIO
W.P. 581-93-00 CONTRACT NO. 2007-4005**

GEOCRES NO. 31D-443

Prepared For:

UMA/AECOM ENGINEERING LIMITED

Prepared by:

SHAHEEN & PEAKER

**Project: SPT1182A-01
August 12, 2008**



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APPENDIX E: LIMITATIONS OF REPORT

FOUNDATION DESIGN REPORT
PROPOSED HEADWALLS FOR EXISTING CULVERT AT STATION 10+012
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET, PETERBOROUGH
W. P. NO. 581-93-00; CONTRACT NO. 2007-4005

5. DISCUSSION AND RECOMMENDATIONS

We understand that the project consists of the construction of head walls at or near the inlet and outlet of the existing culvert. The anticipated founding level for the retaining/head walls are 197.4 and 197.1 m on the north and south sides of the road embankments, near the culvert.

The boreholes show, below some embankment fill and organic rich soils, the presence of natural inorganic soils below elevations ranging from 198.3 m and 197.4 m. The natural soils below these elevations consist of clayey silt with silt zones. The consistency of this deposit is typically firm near its surface, becoming very stiff below. Underlying the clayey silt, Boreholes 1 and 2 contacted a stiff to very stiff silt till followed by a very dense sandy silt till. The groundwater table is believed to be at about El. 198± m, or at about o.g. level.

5.1 FOUNDATIONS

The borehole data show that the use of normal spread footing is feasible, as well as the use of deep foundations. The following table summarizes the foundation alternatives.

Summary of Foundation Alternatives

Foundation Option	Comments	Recommendations
Conventional Spread Footings Founded on Natural Clayey Silt	Suitable bearing resistance, more cost-effective than deep foundations but will require careful dewatering to maintain the bearing resistance of the silty subgrade, due to high water table at the site.	Recommended
Driven piles	Suitable support (i.e. very dense stratum) was found at about 4 m below the proposed wall base but driven piles may be objectionable due to the vibrations created during their installation, as well as their short lengths.	Driven piles are not recommended due to reliability and cost
Caissons	Short caissons would be suitable.	Caissons will be very reliable but will be less cost effective than normal spread footing foundations. May be considered if combined with other retaining walls to be constructed under the same contract.

5.1.1 SPREAD FOOTING FOUNDATIONS

As the groundwater table at the site is high and the soil types revealed by the boreholes are dilatent (i.e. can easily be disturbed and dilate), it is recommended that footings be placed as high as possible, taking into consideration frost and scour.

The recommended bearing surface depths/elevations and geotechnical resistances are tabulated below.

Table 5.1.1

Borehole No./ Elevation	Recommended Highest Founding Level (Bottom) of Footing Depth/Elevation (m)	Recommended Factored Bearing Resistance at ULS (kPa)	Recommended Bearing Resistance at SLS (kPa)	Subgrade Material
BH1/202.8	5.4/197.4	240	150	Clayey silt with silt zones
BH2/202.2	4.8/197.4	280	180	Clayey silt with silt zones

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 clayey silt to silt subgrade surface can be calculated using a friction angle of 25 degrees. If necessary, additional resistance can be obtained by keying-in the foundation of the structure. In this case the unfactored cohesion (c) can be taken as 100 kPa.

All footing excavations will need to be inspected, evaluated and approved by the Geotechnical Engineer appointed by QVE, who is familiar with the findings of this investigation, as per SP 902S01.

Upon completion of 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.

It should be pointed out considerable dewatering effort would be required to effect the construction and to prevent the disturbance of the founding soils.

5.1.2 DEEP FOUNDATIONS

The very dense sandy silt till encountered below El. 194.3 m and 195.5 m in Boreholes 1 and 2, respectively would be suitable to support deep foundations.

The use of driven piles is considered unsuitable for this purpose since 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 combined with other sites within the present contract (i.e., equipment and mobilization costs would be shared).

Caisson foundations socketed at least 1.2 m into the very dense soil (e.g. El. 193.1 m at Borehole 1 and 194.3 m at Borehole 2) 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. The steel casing would be carefully withdrawn as the concrete is poured. QVE inspection should be provided in accordance with SP 903S01.

Higher bearing resistances are available if the caissons are socketed further into the very dense soil. This, however, is unlikely to be necessary and furthermore it 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.

5.1.3 PERMANENT SOLDIER PILE AND LAGGING SYSTEM

A permanent soldier pile and lagging system would present another alternative. Since temporary shoring would likely be required during the construction of the reinforced concrete wall supported on spread footings or deep foundations a permanent soldier pile and lagging type wall may present a cost effective alternative, especially if it will be used at other locations within this contract and if they all can be built simultaneously.

We will be pleased to discuss such a system, if you wish us to do so.

5.2 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 and wingwalls should conform to the minimum requirements illustrated in OPSD 3101.150. The granular backfill should conform to OPSS 1010 for either Granular 'A' or '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 μ 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/m³

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/m³

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 and municipal practice. Vibration generated by traffic should also be considered in the selection of appropriate earth pressure coefficients.

5.3 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 excavations for the construction of the retaining wall can be expected to go through side slopes (near toe) of the existing embankment into possibly some organic rich soils (if they were not properly stripped during the construction of the embankment) and finally into the clayey silt deposit with silt zones. These soils can be classified as follows:

Granular Pavement Fill:	Type 3 soil
Silt to Sandy Silt Embankment Fill:	Type 3 soil above water level Type 4 soil below water level (if the soil was not properly dewatered)
Organic soils	Type 4 soil
Clayey silt with silt zones	Type 3 soil above water level Type 4 soil below water level (if the soil was not properly dewatered)

We recommend that the flow of water in the watercourse and in the culvert be properly diverted so that the construction can be carried out in sufficiently dry conditions. It may be possible to achieve this by diverting the water by means of a cofferdam.

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 1.2 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. We recommend that the contractor asked to submit their dewatering and excavation proposal to CA for their information and approval.

We would once again like to point out that the clayey silt deposit and especially the interbedded silt zones are dilatant materials, which can dilate and easily be disturbed, especially in the presence of water, a condition which can be recognized by the jelly-like, liverish appearance of the soil. By means of careful construction and dewatering techniques the disturbance of this subgrade soil should be prevented. We recommend that the Contractor be alerted of the dilatant behavior of the founding soils and that special care should be taken to avoid disturbing the founding soils.

Temporary shoring may be required to support the excavations. 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 5.3.1 can be used for the design of the temporary shoring system.

Table 5.3.1
Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	Unit Weight (kN/m^3)
Embankment Fill	0.45	0.55	2.0	19.0
Organic Rich Soils	0.6	0.8	1.0	14.0
Clayey Silt with Silt Zones	0.46	0.56	2.8	18.5
Silt Till	0.42	0.50	3.0	20.5
Sandy Silt Till	0.30	0.45	3.5	21.5

5.4 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


Ramon Miranda, P.Eng.


Zuhtu S. Ozden, P.Eng.


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

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.