FOUNDATION INVESTIGATION AND
DESIGN REPORTS
PROPOSED RETAINING WALLS FOR
EXISTING CULVERT AT STATION 10+763
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET
PETERBOROUGH, ONTARIO
W.P. 581-93-00 CONTRACT NO. 2007-4005

GEOCRES NO. 31D-445

Prepared For:

UMA/AECOM ENGINEERING LIMITED

Prepared by:

SHAHEEN & PEAKER

Project: SPT1182A-05
August 12, 2008

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1. INTRODUCTION

Shaheen & Peaker Limited (S&P) was retained by UMA/AECOM Engineering Limited (UMA) to carry out a foundation investigation at the existing culvert location at Station 10+763, Highway 7. The site is located to the east of the junction of Highway 7 with Highway 7/115, east of Peterborough, Ontario

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

The findings of the investigation are presented in the report.

2. PHYSIOGRAPHY

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

Based on the Physiograpy 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 on June 17, 2008 and consisted of putting down two boreholes (Boreholes 9 and 10) at the locations shown on the Borehole Location Plan, Drawing No.1.

The boreholes were put down from the highway embankment, using a drilling rig, equipped with hollow-stem augers. The boreholes were extended to depths of 7.5 and 7.3 m below the ground surface. In the boreholes sampling was effected 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 relatively undisturbed ground in the borehole by a distance of 0.30 m was recorded. The number represents the Standard Penetration Resistance or the N-value of the soil and provides an indication of the consistency or the compactness condition of the soil.

In cohesive (clayey) deposits, where the consistency of the soil permitted, the undrained shear strength of the soil was measured by means of field vane tests.

Adjacent to Borehole 10, a Dynamic Cone Penetration test was performed. For this purpose, augering was effected to depth of 1.5 m below the top of the embankment (to reduce friction) and then the 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 investigation was carried out under the supervision and direction of a Geotechnical Engineer from Shaheen & Peaker Limited.

Groundwater observations were made in the open borehole during the drilling and at the completion of the boreholes. Upon completion, Borehole 9 was sealed using a cement/ bentonite mixture as per MTO procedures, while in Borehole 10, a piezometer was installed to enable us to monitor the groundwater level over a prolonged period of time, without interference from surface water.

The borehole locations were determined by Shaheen & Peaker Limited field staff in relation to station markers and the centerline of the ongoing construction along the highway. The geodetic ground elevations at the borehole locations were provided to us by Greer Galloway Group Inc. staff who were at the site, filling the CA role.

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 an approximately 3.3 m and 2.2 m of embankment fill, Boreholes 9 and 10 show the presence of a sandy silt to silty sand deposit to a depth of 5.2 m or to El. 195.8 m. Underlying this surficial fine grained granular deposit, the boreholes contacted a cohesive clayey silt deposit. The clayey silt is in turn underlain by a possible very dense/ hard soil or bedrock at a depth of about 7.1 m or at Elevation 193.9 m

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 9 and 10 were drilled from the top of the highway embankment (which was undergoing widening at the time) and these boreholes contacted embankment fill to depths of 3.3 m and 2.2 m (El. 197.7 m and 198.8 m), respectively.

The upper 1.2 m to 1.3 m of the fill consisted of granular pavement fill (gravelly sand or sand & gravel). Based on N-values of between 18 and 41 blows/0.3 m, which were recorded in the granular pavement fill, the relative density of the granular pavement fill is described as compact to dense.

Underneath the granular fill, the remaining portion of the embankment fill was found to consist of sandy silt, silty sand or fine sand, containing in some cases traces of gravel and/or silt pockets. In Borehole 10, the embankment fill was found to be intermixed with some clayey silt and organic soil.

These embankment fill soils are basically fine grained granular soils and based on Standard Penetration Test results, which yielded N-values of between 6 and 20 blows/0.3 m, the relative density is described as loose to compact.

4.2 SANDY SILT/ SILTY SAND

The embankment fill is underlain in both boreholes by a fine grained granular deposit, ranging in composition from sandy silt to silty fine sand. The deposit was found to extend to a depth of 5.2 m below the ground surface or to El. 195.8 m. In Borehole 9 the presence of some organics was noted in the upper zones of the deposit while in Borehole 10, the presence of some silt and clayey silt seams/lenses was noted.

The grain-size distribution of samples from the deposit is given in figures B-1 and B-2 in Appendix B.

N-values recorded in the deposit range from 4 to 13 blows/0.3 m, indicating a very loose to compact condition, but typically N-values of 7 to 9 blows/0.3 m, indicate a loose relative density.

4.3 CLAYEY SILT

At a depth of 5.2 m or El. 195.8 m, both boreholes contacted a cohesive deposit consisting of clayey silt with a clay layers.

Atterberg Limits tests performed on the four samples recovered from the deposit gave the following index values (See Figure B-3 in Appendix B):

Liquid Limit:	25 - 33%
Plastic Limit	17 - 22%
Plasticity Index:	8 - 11%

These results are characteristic of clayey soils of low plasticity. The measured natural moisture contents are at or in excess of the measured liquid limit values, which indicates a weak soil.

N – Values recorded in the deposit range from 4 to 9 blows/0.3 m and the field vane tests performed yielded undrained in-situ shear strengths of between 24 and 36 kPa. Based on these field and laboratory test results, the consistency of the deposit is described as soft to stiff but typically firm.

From the behavior of the augers the presence of a hard / very dense material was inferred at a depth of 7.1 m (El. 193.9 m) in both boreholes. Standard Penetration Tests showed no penetration for 100 blows at 7.5 m and 7.3 m depths, as well as auger refusal, where the boreholes were terminated. This may represent a bouldery till or bedrock.

4.4 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of each borehole. In addition a piezometer was installed in Borehole 10, to enable us to monitor the groundwater table over a prolonged period of time without interference from surface water. The observations are shown on the Record of Boreholes Sheets in Appendix A.

As shown on the individual borehole logs, the groundwater level in the boreholes was recorded within or immediately below the embankment fill at elevations 198.6 m and 199.5 m in boreholes 9 and 10, respectively. Based on this it is our opinion that the groundwater table at the time of our investigation was near the o.g. level or between El. 199.5 m and 198.6 m.

It should however be pointed out that the groundwater table 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.

SHAHEEN & PEAKER LIMITED

Ramon Miranda, P.Eng.

Z.S. Ozden, P.Eng.

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

ZO:tr/idrive

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K. R. PEAKER
PROVINCE OF ONTARIO

Drawings

METRIC

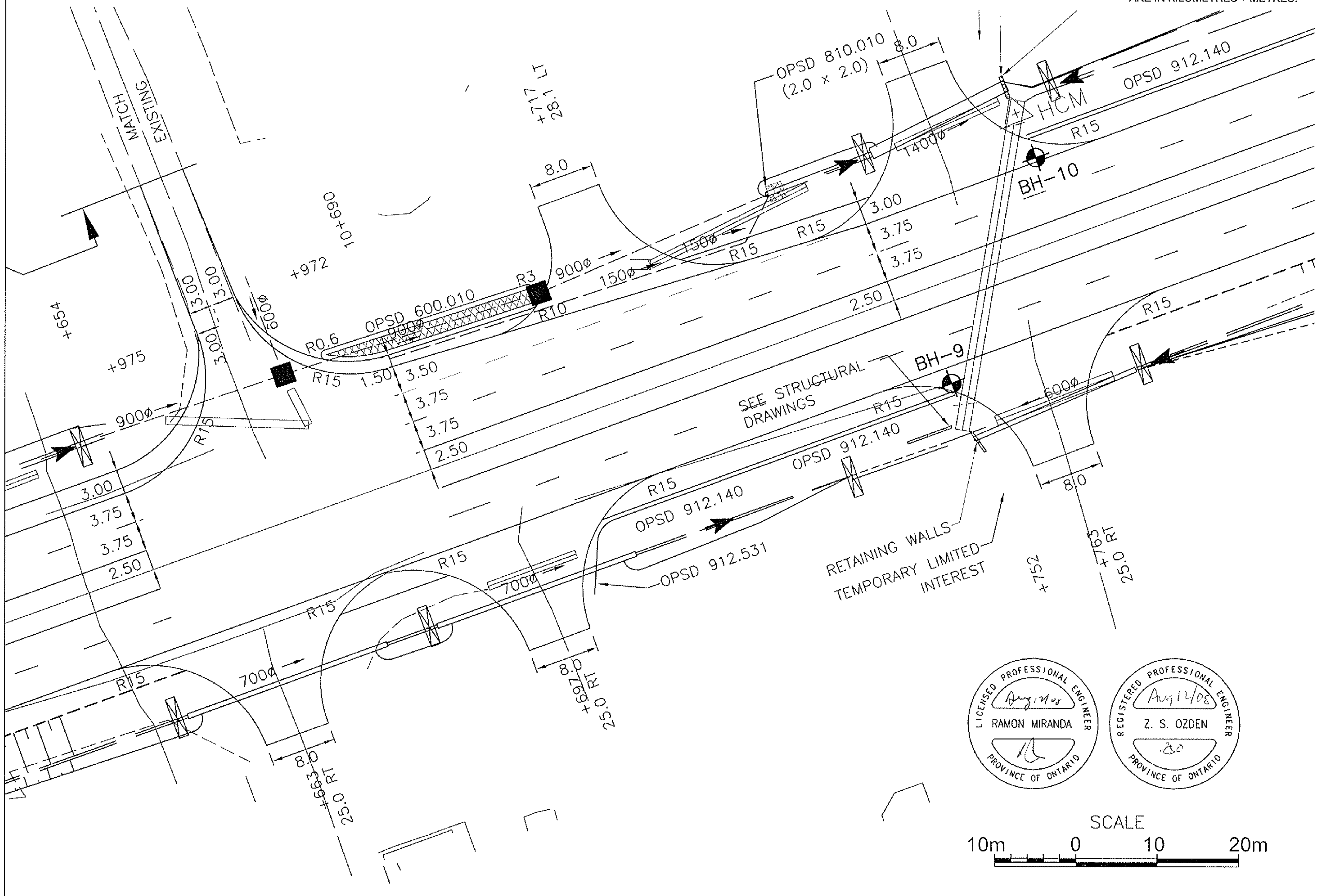
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

NOTES:

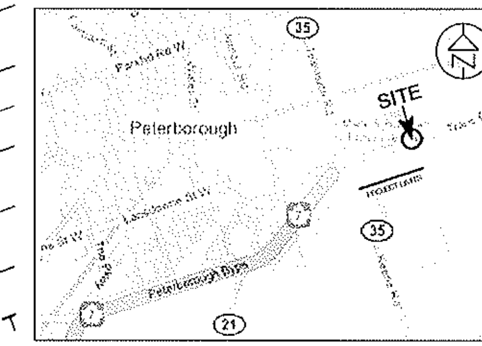
FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No 2007-4005	
WP No 581-93-00	
HIGHWAY 7, PETERBOROUGH CULVERT @ 10+763 BOREHOLE LOCATIONS	SHEET

SHAHEEN & PEAKER LIMITED



BOREHOLE LOCATION PLAN



KEY PLAN

N.T.S.

LEGEND

Borehole

No.	ELEV.	STATION NO
BH-9	200.97	10+753
BH-10	200.99	10+773

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-445

SPT 1182 A-05			DIST
SUBM'D	CHECKED	DATE JUN 2008	SITE
DRAWN PK	CHECKED RM	APPROVED ZO	DWG 1

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

METRIC

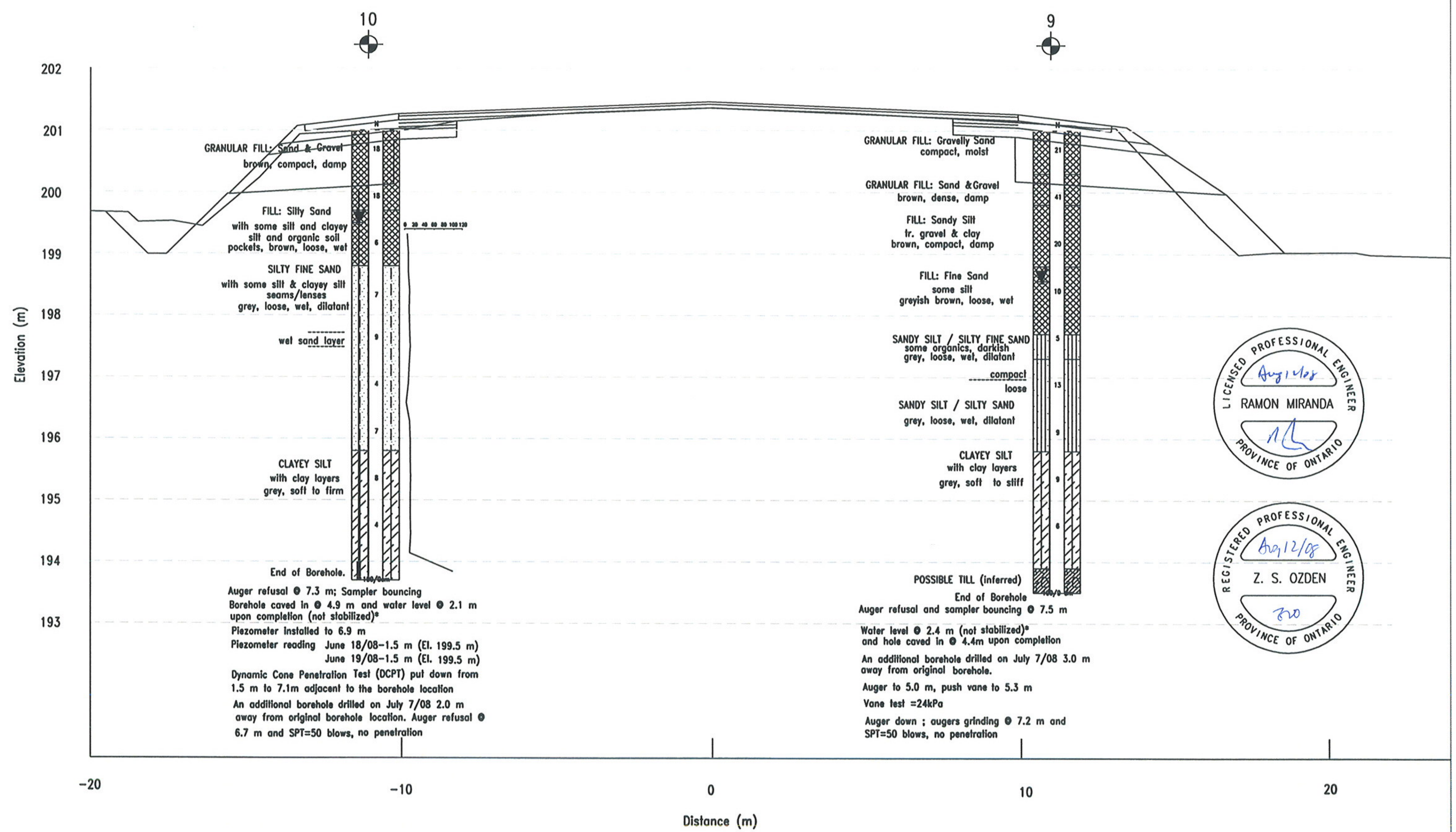
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

CONT No 2007-4005
WP No 581-93-00

HIGHWAY 7, PETERBOROUGH
CULVERT @ 10+763
STRATIGRAPHY

SHEET

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND		
Borehole		
No.	ELEV.	STATION NO
BH-9	200.97	10+753
BH-10	200.99	10+773

=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-445

SPT 1182 A-05			DIST
SUBM'D	CHECKED	DATE JUN 2008	SITE
DRAWN PK	CHECKED RM	APPROVED ZO	DWG 2

STRATIGRAPHY

Appendix A

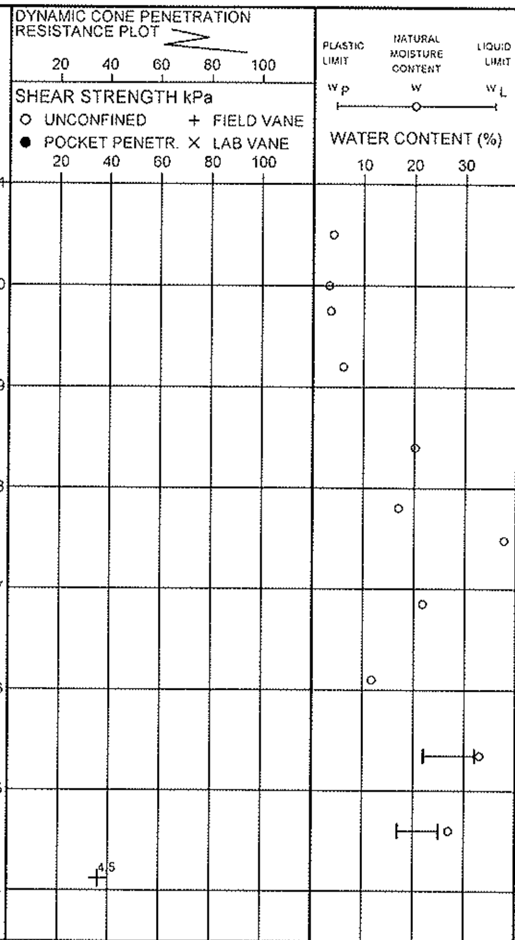
Record of Borehole Sheets

RECORD OF BOREHOLE No 9

METRIC

GWP G.W.P. 173-98-00	LOCATION (Sta : 10+753) 12.0 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 6/17/2008		CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
201.0	GROUND SURFACE					
0.0	**GRANULAR FILL: Gravelly Sand** compact, moist	1	SS	21		
200.3 / 0.7	**GRANULAR FILL: Sand & Gravel** brown, dense, damp	2	SS	41		
199.8 / 1.2	**FILL: Sandy Silt** tr. gravel & clay brown, compact, damp	3	SS	20		
198.8 / 2.2	**FILL: Fine Sand** some silt greyish brown, loose, wet	4	SS	10	▼*	
197.7 / 3.3	**SANDY SILT / SILTY FINE SAND** some organics, darkish grey, loose, wet, dilatant	5	SS	5		
197.3 / 3.7	**SANDY SILT / SILTY SAND** compact / loose grey, loose, wet, dilatant	6	SS	13		1 28 (71)
		7	SS	9		
195.8 / 5.2	**CLAYEY SILT** with clay layers grey, soft to stiff	8	SS	9		
		9	SS	6		
193.9 / 7.1	**POSSIBLE TILL** (inferred)	~~10~~	~~SS~~	~~100/0 cm~~		hard augering below 7.1 m ~~No recovery~~
193.5 / 7.5	End of Borehole					



Auger refusal and sampler bouncing @ 7.5 m
Water level @ 2.4 m (not stabilized)*
and hole caved in @ 4.4m upon completion

An additional borehole drilled on July 7/08 3.0 m away from original borehole.

Auger to 5.0 m, push vane to 5.3 m
Vane test =24kPa

Auger down ; augers grinding @ 7.2 m and SPT=50 blows, no penetration

$+^3, \times^3$: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 10

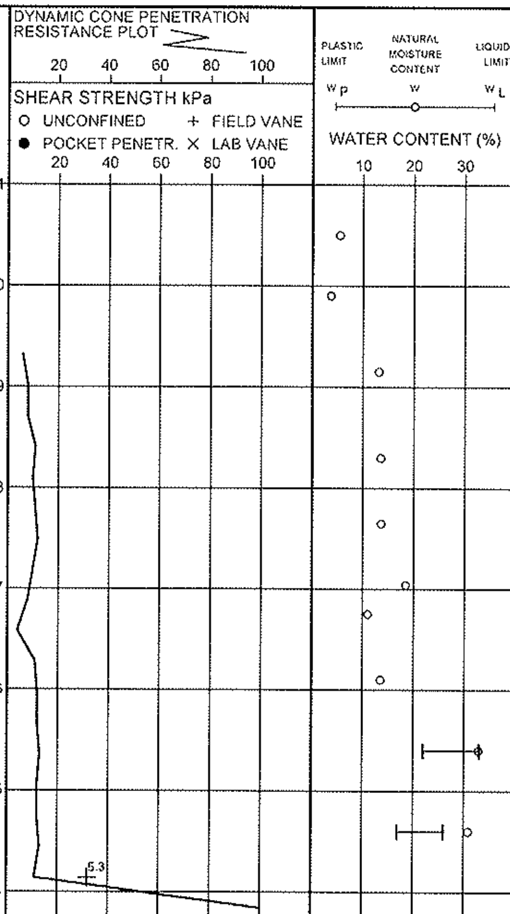
METRIC

GWP G.W.P. 173-98-00 LOCATION (Sta.: 10+773) 10.5 m Lt C/L of Hwy 7, Peterborough ORIGINATED BY SK

DIST HWY 7 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS

DATUM Geodetic DATE 6/17/2008 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
201.0	GROUND SURFACE				
0.0	**GRANULAR FILL: Sand & Gravel** brown, compact, damp	1	SS	18	
		2	SS	18	
199.7 / 1.3	**FILL: Silty Sand** with some silt and clayey silt and organic soil pockets, brown, loose, wet	3	SS	6	
198.8 / 2.2	**SILTY FINE SAND** with some silt & clayey silt seams/lenses grey, loose, wet, dilatant	4	SS	7	
		5	SS	9	0 56 (44)
	wet sand layer	6	SS	4	0.1 m sand back-up in hollow stem auger
		7	SS	7	
195.8 / 5.2	**CLAYEY SILT** with clay layers grey, soft to firm	8	SS	8	0 73 (27)
		9	SS	4	
193.7 / 7.3		10	SS	100/0cm	No Recovery



UNIT WEIGHT γ kN/m³

End of Borehole.

Auger refusal @ 7.3 m, Sampler bouncing

Borehole caved in @ 4.9 m and water level @ 2.1 m upon completion (not stabilized)*

Piezometer installed to 6.9 m
Piezometer reading
June 18/08-1.5 m (El. 199.5 m)
June 19/08-1.5 m (El. 199.5 m)

Dynamic Cone Penetration Test (DCPT) put down from 1.5 m to 7.1m adjacent to the borehole location

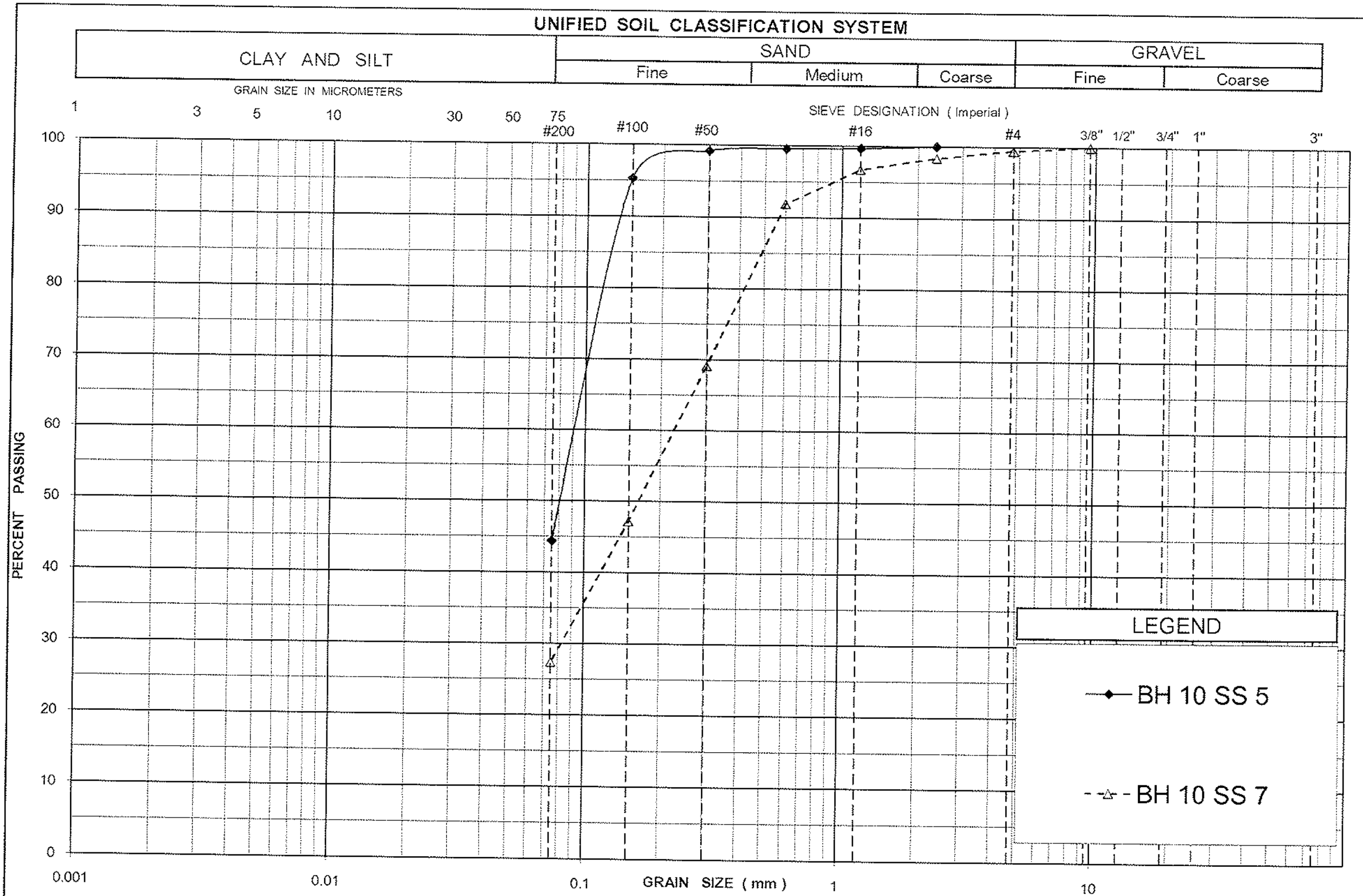
An additional borehole drilled on July 7/08 2.0 m away from original borehole location. Auger refusal @ 6.7 m and SPT=50 blows, no penetration

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results



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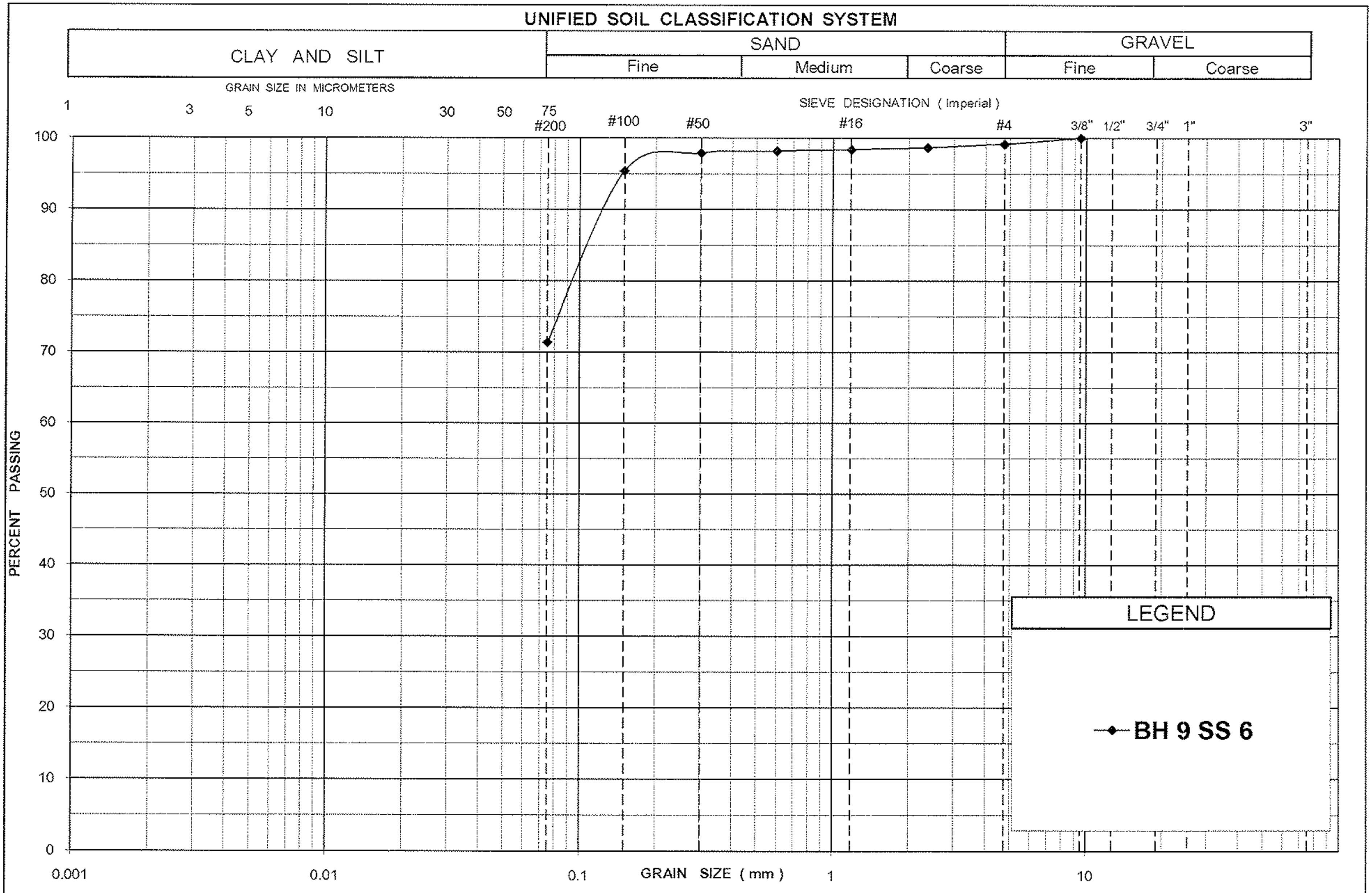
GRAIN SIZE DISTRIBUTION

SILTY FINE SAND with some silt & clayey silt seams

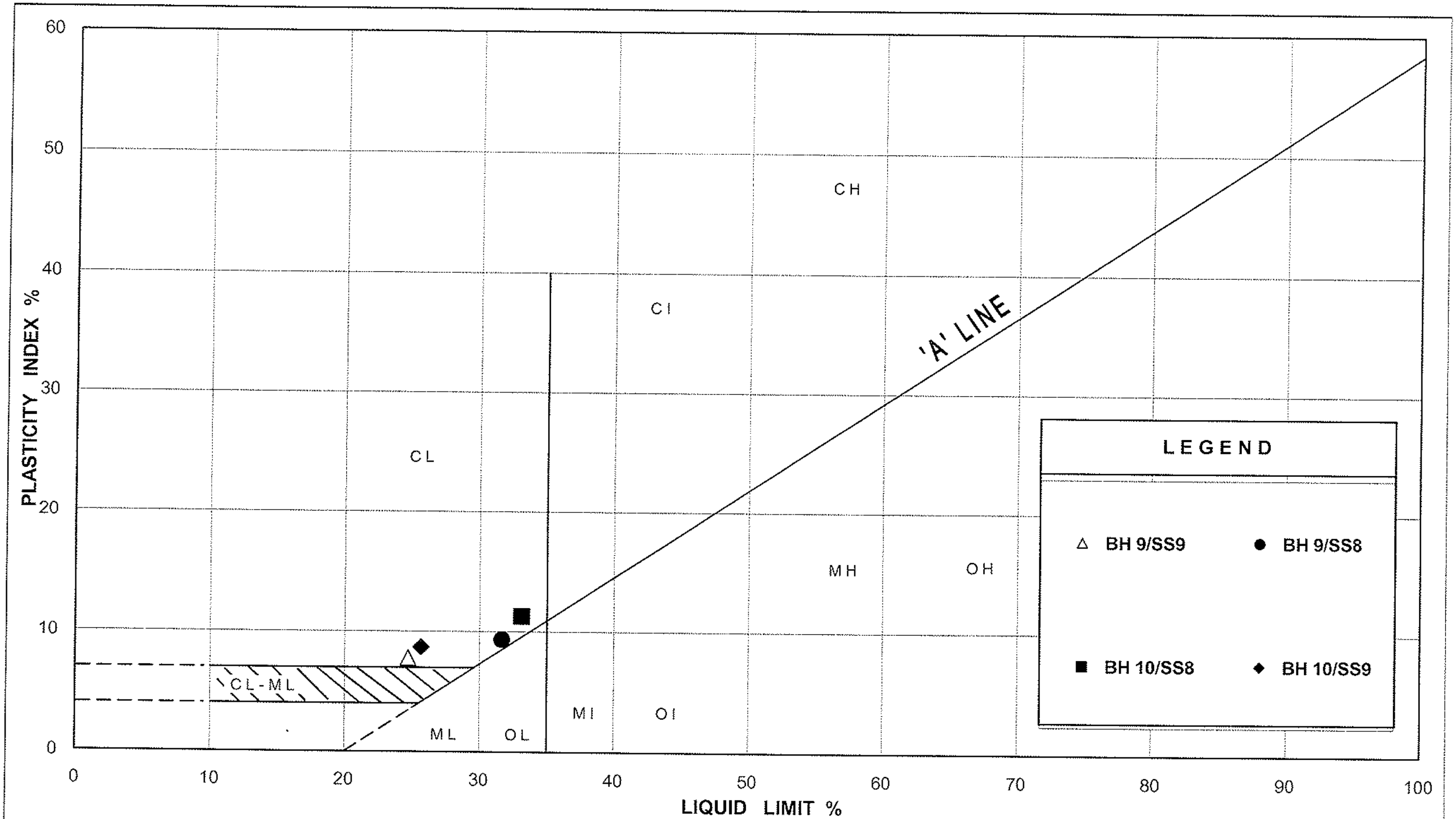
FIGURE No. B-1

REF. No. SPT 1182 A-05

DATE July 2008



SHAHEEN & PEAKER LIMITED A Coffey Geotechnics Company	GRAIN SIZE DISTRIBUTION SANDY SILT /SILTY SAND	FIGURE No. B-2 REF. No. SPT 1182 -05A DATE July 2008



SHAHEEN & PEAKER LIMITED A COFFEY GEOTECHNICS COMPANY	PLASTICITY CHART **CLAYEY SILT with clay layers**	FIGURE No. B-3 REF. No. SPT 1182 A-05 DATE July 02, 2008

Appendix C

Site Photographs

Photograph 1. Borehole 9 (looking east)

Photograph 2. Borehole 10 (looking south)

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 $\overline{N}$.

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 COMPOSION 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
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	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'	$-^o$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$-^o$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES
i_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
P_w	kg/m^3	DENSITY OF WATER
i_w	kN/m^3	UNIT WEIGHT OF WATER
P	kg/m^3	DENSITY OF SOIL
i	kN/m^3	UNIT WEIGHT OF SOIL
P_d	kg/m^3	DENSITY OF DRY SOIL
i_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
i_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
P'	kg/m^3	DENSITY OF SUBMERED SOIL
i'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
s_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $(W_L – W_L)$
I_L	1	LIQUIDITY INDEX = $(W – W_P)/ I_P$
I_C	1	CONSISTENCY INDEX = $(W_L – W) / 1_P$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} – e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	N PERCENT – DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

FOUNDATION DESIGN REPORT
PROPOSED RETAINING WALLS FOR
EXISTING CULVERT AT STATION 10+763
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET
PETERBOROUGH, ONTARIO
W.P. 581-93-00 CONTRACT NO. 2007-4005

GEOCRES NO. 31D-445

Prepared For:

UMA/AECOM ENGINEERING LIMITED

Prepared by:

SHAHEEN & PEAKER

Project: SPT1182A-05
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FOUNDATION DESIGN REPORT
PROPOSED RETAINING WALLS FOR EXISTING CULVERT AT STATION 10+763, 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, will entail the construction of new retaining walls adjacent to the existing culvert at Station 10+763. The height of the retaining walls will be less than 2.0 m.

Boreholes 9 and 10, which were put down on the south and north sides of the roadway embankment, showed below 2.2 and 3.3 m embankment fill, the presence of fine-grained granular soils to a depth of 5.2 m or El. 195.8 m. These consist of sandy silt to silty fine sand and based on N-values which range from 4 to 13 blows/0.3 m, they are considered to be in a very loose to compact condition. Underlying these fine-grained granular soils at a depth of 5.2 m, the boreholes contacted a weak and compressible cohesive soil deposit which consists of clayey silt with clay layers/seams to a depth of 7.1 m or El. 193.9 m. At this depth, a hard or very dense material was inferred with auger refusal immediately below depths of 7.5 and 7.3 m or El. 193.5 and 193.7 m, respectively.

In the clayey silt deposit the recorded N-values range from 4 to 9 blows/0.3 m and field vane tests gave in-situ undrained shear strength values of 24 kPa to 36 kPa.

The groundwater level was measured at about the o.g. level or at El. 198.6 m and 199.5 m in Boreholes 9 and 10, respectively. The groundwater table would, however, be subject to fluctuations.

We understand that the height of the retaining walls will be less than 2.0 m. The retained soil immediately behind the wall rises at 2H:1V slope by another 0.5 m ± to the top (i.e. shoulder of the highway), giving a total height for the retained soil of 2.5 m or less.

We also understand for spread footing foundations the anticipated underside footing elevation is 197.7 m and 197.6 m on the north and south sides, respectively.

The following table summarizes support systems which may be considered for this project.

Wall Type

Type of Wall	Comments	Recommendations
Reinforced Concrete Retaining Wall	Relatively unfavorable soil conditions for the use of normal spread footing foundations. They may not be economical due to their width and dewatering requirements. As well, higher than usual settlements can be expected (i.e. 30 to 40 mm).	Shallow spread footings do not present a very expedient and reliable solution due to poor soil conditions and high water table.
	Caisson foundations are likely to be a better choice than spread footing foundations, due to their short length (not very costly), less rigorous dewatering requirements and more expedient (i.e. probably less time loss during construction) in comparison with spread footing foundation.	More reliable than spread footing foundations.
Contiguous Caisson Wall	Reliable but uneconomical.	Not recommended based on cost factor.
Permanent Soldier Pile and Lagging	Can be an economical solution since tie-backs are unlikely to be necessary for a 2.5 m high retained soil. Will require less strict dewatering than most other methods. Reliable.	Can be a more economical solution in comparison with other systems where a temporary shoring system is required.
Armour Stone / Gabion Wall	Can be considered if some distortions and lateral yield can be tolerated. Scour should be checked.	Not recommended based on reliability.
RSS (Retained Soil System)	Scour may be a problem. Will require dewatering.	Unlikely to be economical if a temporary shoring system is required.

5.1 REINFORCED CONCRETE RETANING WALL

The borehole data show that to about 5 m below the o.g. level (i.e. to about El. 193.9 m), the existing overburden contacted in the boreholes is weak and compressible. The actual conditions beyond the fill height embankment footprint probably present a weaker picture, as these soils would not have had the benefit of surcharging by the stresses imposed by the embankment. As well, the groundwater table at the site is high.

5.1.1 SPREAD FOOTING FOUNDATIONS

As the groundwater table at the site is at about o.g. level, we recommend that the footings be placed as high as possible, taking into consideration frost and if applicable, scour depths, as well as the loose nature of the sandy silt/silty fine sand and the underlying weak and compressible clayey silt.

The following geotechnical resistances are available for footings placed on undisturbed natural soils at or above El. 197.6 m.

Factored Bearing Resistance at U.L.S. = 130 kPa*
Bearing Resistance at S.L.S. = = 80 kPa**

* Based on a minimum footing width of 1.2 m and a minimum earth cover of 1.6 m.
** Anticipated maximum settlement is 30 mm for footing sizes less than 1.8 m; anticipated maximum settlement for footing sizes between 1.8 and 2.6 m is 40 mm.

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

The design of the structure should include overturning and sliding considerations. The unfactored horizontal resistance against sliding between poured concrete and approved sandy silt/silty fine sand surface can be calculated using a friction angle of 27 degrees.

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

Upon completion of excavation, inspection and approval a 120 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 the disturbance and dilatation of the bearing surface.

It is expected that the construction of normal spread footings will be difficult due to high water table and dewatering that will be required.

5.1.2 DEEP FOUNDATIONS

If the bearing resistances quoted for normal (shallow) spread footing foundations are insufficient, or the quoted settlements can not be tolerated, considerations can be given to the use of deep foundations.

Boreholes 9 and 10 show the presence of a hard/very dense layer, or possibly bedrock at a depth of about 7.1 m or El. 193.9 m. The following are some possible alternatives which can be considered for deep foundations, however, it would be prudent to verify the presence of the bearing zone, by coring if deep foundations are to be utilized.

The use of driven piles may be objectionable for this project as the vibrations induced during pile driving may create problems. However, the use of drilled and poured in place concrete (caisson) foundations could be a viable alternative, especially if the use of such foundations are being considered for other sites along the highway within the contract being presently implemented (i.e. reduced equipment mobilization sites for site and economics of scale).

For design purposes, the following geotechnical resistances can be used for caissons socketed at least 0.6 m into the very dense or sufficiently hard soil (i.e. shear strength of at least 400 kPa), or at least 0.1 m into the bedrock.

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

Higher bearing resistances are available for caissons socketed into the competent soil/bedrock, but such resistances are unlikely to be required.

During their installation, the caissons would require the use of temporary employment of steel casings to enable the bases to be inspected and properly cleaned of debris and loosened material. The minimum caisson diameter will therefore need to be 0.76 m. The temporary steel casing would be carefully withdrawn as the concrete is poured, ensuring no 'necking.' QVE inspection should be implemented in accordance with SP 903S01.

It is expected that caissons will require some dewatering but not to the same extent as normal shallow spread footing foundations.

5.2 CONTIGUOUS CAISSON WALL

The embankment can be retained by means of a contiguous (interlocking) caisson (augered and cast-in-place concrete) wall. Tiebacks are unlikely to be necessary, since the height of the retained soil is less than about 2.5 m.

A difficulty that may arise during the installation of the caisson is the presence of relatively pervious sands. As well, the presence of cobbles and boulders should be anticipated in the glacial till deposit (inferred) and/or advancing into the bedrock would be costly.

Another disadvantage of the system is its high cost as well as the esthetically unpleasing appearance of the exposed face. This can be rectified by providing a facing but this will add to the cost. Support may also be required during the setting of the concrete. As well, the concrete used may need to be erosion and pollutant resistant. For these reasons, this type of wall is not recommended (i.e. high cost, unsuitable soil conditions, etc.).

5.3 SOLDIER PILE AND LAGGING SYSTEM

A soldier pile and lagging system is similar to a contiguous caisson wall, except that a caisson type hole together with an I-beam reinforcing steel is provided at about every 2.5 to 3.0 ±m spacing and as such it is more cost-effective than a contiguous wall. 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 resistant against elements and corrosive materials such as salt. The exposed steel I-beam will also need to be protected from the elements (e.g. against rusting). 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.

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. One problem that may arise in advancing the caisson holes into the hard/very dense soil or the bedrock (as evidenced by the refusals encountered on the augers when advancing the boreholes). As well the design should be cognizant of the fact that the clayey silt deposit is weak and will provide only minimal toe resistance (i.e. kick-out) should be checked if sufficient advance into the underlying hard layer becomes a problem.

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 is that temporary shoring will not be required which maybe required for some of the other types of wall structures. In other words, during its installation, the structure will also serve as the temporary support system. As well, available information leads us to believe that the caissons will be very short. For these reasons, in our opinion, this method presents an attractive 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 extend about 0.5 m beyond the perimeter of the footprint of the wall (0.3 m at the back of the wall) and would include the placement of a biaxial reinforcing geo-grid such as Terrafix BX 1200 or equivalent. The purpose of the biaxial geo-grid is to distribute the loads more effectively in both directions. The thickness of the granular pad should be 350 mm and the subgrade should consist of 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	Highest Suitable Subgrade Elevation (m)
9	201.0	197.5
10	201.0	198.8

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.

As well precautions may need to be taken by placing a suitable geotextile to prevent the loss of fine grained granular soils from the back and the front of the wall.

These support systems are unlikely to be acceptable due to scour considerations and lateral yield but we will be pleased to discuss further details, if you require us to do so.

5.5 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.

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

A MESA type wall (provided by Tensar) would likely be suitable or its equivalent (must be on MTO's approved list). Depending on the details, this type of wall would likely be placed on a reinforced granular pad similar to that discussed in the previous section of this report (i.e. Section 5.4).

This system would require less rigorous construction dewatering than if spread footing foundations are used to support a reinforced concrete retaining wall, but more rigorous than deep foundations and/or a permanent soldier pile and lagging system.

Scour considerations may make the system unsuitable for the project. We recommend that this aspect be given consideration and be discussed with a specialized contractor before the system can be evaluated.

We will be pleased to discuss this system further if scour is not a problem.

5.6 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 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 µm) 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 ϕ=35° (unfactored)
Unit weight = 22 kN/m^3
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 ϕ=32° (unfactored)
Unit Weight = 21 kN/m^3
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.

5.7 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 to silty sand deposits which are in turn underlain by clayey silt with clay layers. These deposits are underlain by possible bouldery till or bedrock. 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, Silty Sand/Fine Sand Embankment Fill	Type 3 soil above water level Type 4 soil below water level
Sandy Silt, Silty Sand	Type 3 soil above water table Type 4 soil below water table (if the soil was not dewatered)
Clayey Silt	Type 3 soil above water table Type 4 soil below water table
Glacial Till (inferred)	Type 2 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 the subgrade soil should be prevented, especially if spread footing foundations are employed. The Contractor should be alerted that special care shall be taken to avoid disturbing the founding soils. In addition, we recommend that Contractor should be asked to submit their dewatering and excavation proposal to the CA for information purposes.

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 5.7.1 can be used for the design of the shoring system. The shoring system should be designed by a Professional Engineer who is experienced in this type of work.

Table 5.7.1
Recommended Unfactored Parameters for Shoring Design

Soil Type	**K_a**	**K_o**	**K_p**	**Unit Weight (kN/m^3)**
Granular Fill	0.30	0.50	3.3	21.5
Sandy Silt/Silty Sand/Sand Embankment fill	0.40	0.56	2.5	18.0
Organic Rich Soils	0.60	0.80	1.0	18.0
Sandy Silt/Silty Sand	0.40	0.56	2.5	18.0
Clayey Silt	0.48	0.66	2.0	16.5
Glacial Till (inferred)	0.30	0.45	3.6	22.0

5.8 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.

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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.