

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
PROPOSED RETAINING WALLS AT
STATION 10+850-10+880
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET
PETERBOROUGH, ONTARIO
W.P. 581-93-00 CONTRACT NO. 2007-4005**

GEOCRES NO. 31D-442

Prepared For:

UMA/AECOM ENGINEERING LIMITED

Prepared by:

SHAHEEN & PEAKER

**Project: SPT1182A-02
August 12, 2008**



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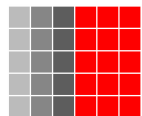
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Table of Contents

1. INTRODUCTION	1
2. PHYSIOGRAPHY	1
3. INVESTIGATION PROCEDURES	2
4. SUBSURFACE CONDITIONS	3
4.1 Embankment Fill	3
4.2 Peat and Topsoil	3
4.3 Silty Fine Sand to Sand	3
4.4 Clayey Silt	4
4.5 Clayey Silt Till	5
4.6 Sandy Silt Till	5
4.7 Groundwater Conditions.....	5

DRAWINGS

DRAWING No.

BOREHOLE LOCATIONS & SOIL STRATA

1 & 2

APPENDIX A: RECORD OF BOREHOLE SHEETS

APPENDIX B: LABORATORY TEST RESULTS

APPENDIX C: SITE PHOTOGRAPH

APPENDIX D: EXPLANATION OF TERMS USED IN REPORT

**FOUNDATION INVESTIGATION REPORT
PROPOSED RETAINING WALLS AT STATION 10+850-10+880
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET, PETERBOROUGH
W. P. NO. 581-93-00 CONTRACT NO. 2007-4005**

1. INTRODUCTION

As part of on-going construction along Highway 7, immediately east of Peterborough, retaining (toe) walls will be constructed on the north side of the highway between approximate Stations 10+850 and 10+880. Shaheen & Peaker Limited (S&P) was retained by UMA/AECOM Engineering Limited (UMA) to conduct a foundation investigation for the proposed toe walls.

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

This findings of the investigation are presented in this report.

2. PHYSIOGRAPHY

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

Based on the Physiography of Southern Ontario (by Putnam & Chapman), the proposed 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 0.6 to 0.9 m 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 can easily disintegrate.

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

3. INVESTIGATION PROCEDURES

The field investigation at the site was carried out on June 11, 12 and 13, 2008 and, as shown on Drawing No. 1, consisted of putting down three boreholes (Boreholes 3, 4 and HD3).

Boreholes 3 and 4 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. Both boreholes were extended to a depth of 7.3 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 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.

Borehole HD3 was put down from the o.g. level, near the toe of the Highway embankment, using manual methods, to a depth of 1.4 m below the o.g. level. When advancing the borehole, 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 obtain an approximate (equivalent) SPT 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, Borehole 3 was grouted and sealed using a cement/bentonite mixture. In Borehole 4, a piezometer was installed to enable longer term water level observations, without interference from surface water.

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. Site photographs are presented in Appendix C.

4. SUBSURFACE CONDITIONS

In general below some embankment fill (to depths of 2.2 – 2.3 m), Boreholes 3 and 4 show the presence of approximately 1.9 to 3.7 m thick silty sand to sand, underlain by a 0.8 to 1.9 m thick clayey silt which is in turn underlain by glacial till deposits.

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 3 and 4, drilled from the roadway, which at the time of our fieldwork was under construction (i.e. being widened), encountered embankment fill, to depths of 2.3 m (El. 198.5 m) and 2.2 m (El. 198.6 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 30 and 32 blows/0.3 m, indicating a compact to dense condition.

The grain-size distribution of a sample from the granular pavement fill is given in Figure B-1 in Appendix B.

In general, underlying the granular pavement fill, the embankment fill was found to consist of sandy silt with traces to some gravel, which appears to have been derived from local sandy silt glacial till deposits. Traces of organic soil were also found to be intermixed with the fill.

The recorded N-values within the embankment fill range from 17 to 36 blows/0.3 m indicating that a systematic compaction was applied when the fill was first placed. Based on these blow counts the compactness condition of the embankment fill at the borehole locations is described as compact to dense.

4.2 PEAT AND TOPSOIL

In Borehole HD3, which was put down from the o.g. level (El. 199.6 m), a 0.2 m thick organic layer was contacted at the ground surface level. This organic soil was found to consist of peat and topsoil.

4.3 SILTY FINE SAND TO SAND

The embankment fill at Boreholes 3 and 4 and the organic soil at Borehole HD3 are underlain by granular (i.e. non-cohesive) soils which range from silty fine sand to fine to

medium sand. Borehole HD3 was terminated in this deposit at a depth of 1.4 m (El. 198.2 m) while in Boreholes 3 and 4, these basically granular soils extended to 6.0 m and 4.1 m below the ground surface or to El. 194.8 m and 196.7 m, respectively.

The grain-size distribution of two samples from Borehole 3, from the somewhat coarser zones of the deposit (i.e. fine to medium sand), is given in Figure B-2, in Appendix B. The curves show the following grain-size distribution:

Gravel:	0 - 1%
Sand:	95 - 98%
Silt & Clay:	2 - 4%

The presence of some silt zones and thin clay seams was noted within these deposits.

These soils were found to be wet and water-bearing. Standard Penetration tests performed yielded N-values which generally range from 12 to 20 blows/0.3 m. These results indicate a generally compact relative density.

4.4 CLAYEY SILT

At depths of 6.0 m (El. 194.8 m) and 4.1 m (El. 196.7 m), Boreholes 3 and 4 show the presence of a clayey silt deposit with some silt zones and occasional thin clay seams. In Borehole 3 this cohesive deposit was found to be 0.8 m thick and extended to El. 194.0 m, while in Borehole 4, it was found to be 1.9 m thick and extended to El. 194.8 m.

Atterberg Limits tests performed on two samples from Borehole 4 gave the following index values (see Plasticity Chart, Figure B-3 in Appendix B):

Liquid Limit:	25 - 27%
Plastic Limit:	16 - 19%
Plasticity Index:	8 - 9%

These results are characteristics of clayey soils of low plasticity. The measured natural moisture contents are close to or in excess of the measured Liquid Limit values which indicate that the material is likely to be weak and compressible.

N-values recorded in the deposit range from 6 to 9 blows/0.3 m, indicating a firm to stiff consistency.

This deposit is considered to be significantly less pervious than the overlying silty sand to sand deposits.

4.5 CLAYEY SILT TILL

Underlying the clayey silt, Borehole 4 contacted at a depth of 6.0 m (El. 194.8 m) a glacial till deposit which consists of a heterogeneous mixture of clayey silt with some sand and gravel. This clayey silt till deposit was found to be 1.0 m thick and extended to a depth of 7.0 m below the ground surface to El. 193.8 m.

From a recorded N-value of 7 blows/0.3 m, the consistency of this cohesive till deposit is described as firm.

4.6 SANDY SILT TILL

Underlying the clayey silt till, a non-cohesive (i.e. granular) glacial deposit, consisting of a heterogeneous mixture of sandy silt with some gravel and traces of clay size particles, was contacted in Borehole 4, at a depth of 7.0 m or El. 193.8 m. Based on N-values which are in excess of 100 blows/0.3 m, this deposit is described as very dense. Auger refusal was encountered in this borehole at 7.3 m (El. 193.5 m).

In Borehole 3, underlying the clayey silt deposit at 6.8 m (El. 194.0 m), a similarly very dense material was contacted but the sampler could not penetrate this deposit despite 100 blow of the hammer in two attempts. This material was inferred to be a very dense sandy silt till, similar to that encountered in Borehole 4. In this borehole (i.e. Borehole 3), refusal to further augering was encountered at 7.3 m (El. 193.5 m). On July 7, 2008 another borehole was drilled 1.5 m from the original borehole and augering was effected to 7.6 m, where a Standard Penetration test showed no penetration.

Based on these observations, the presence of a bouldery till or bedrock can be inferred at a depth of about 7.3 m below the ground surface or below about El. 193.5 m, at both borehole locations.

4.7 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during drilling and at the completion of each borehole, as shown on the individual Record of Borehole Sheets. Upon their completion, water levels in Boreholes 3 and 4 were found at 2.1 m (El. 198.7 m). A piezometer was installed in Borehole 4 to facilitate longer terms water level measurements and on July 7, 2008, the water level in the piezometer was measured at 1.4 m below the ground surface (i.e. shoulder elevation) or at El. 199.4 m.


Based on the observations made and moisture contents of the soil samples, the groundwater table at the time of our investigation was at about El. 198.7 – 199.4 m. It should, however, be pointed out that the groundwater level at the site would be subject to seasonal fluctuations and fluctuations in response to major weather events.

It should also be pointed out that while drilling a back-up of up to 0.5 m was observed in the sand deposit into the hollow-stem augers, which indicates an upward hydrostatic gradient in the sand deposits.

SHAHEEN & PEAKER LIMITED


Ramon Miranda, P.Eng.

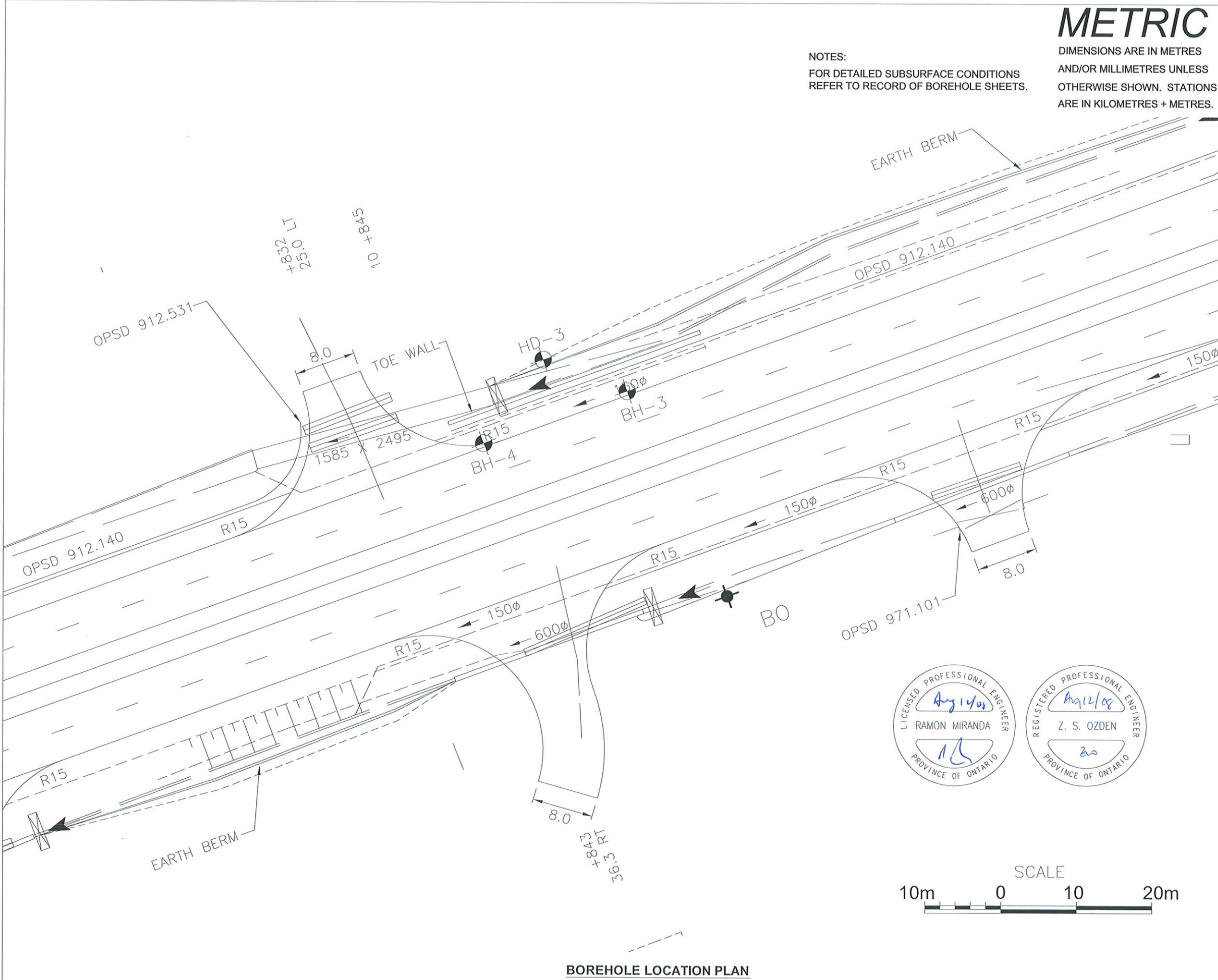

Z.S. Ozden, P.Eng.


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

ZO:tr/idrive



Drawings



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
TOE WALL @ 10+850 -10+880
BOREHOLE LOCATIONS



SHEET

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

Borehole

No.	ELEV.	STATION NO
BH-3	200.82	10+870
BH-4	200.81	10+850
HD-3	199.56	10+861

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.



BOREHOLE LOCATION PLAN

REV.	DATE	BY	DESCRIPTION
Geocres No. 31D-442			
SPT 1182 A-02			DIST
SUBM'D	CHECKED	DATE JUN 2008	SITE
DRAWN PK	CHECKED RM	APPROVED ZO	DWG 1

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
Toe Wall @ 10+850-10+880
STRATIGRAPHY



SHEET

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S.

LEGEND

Borehole

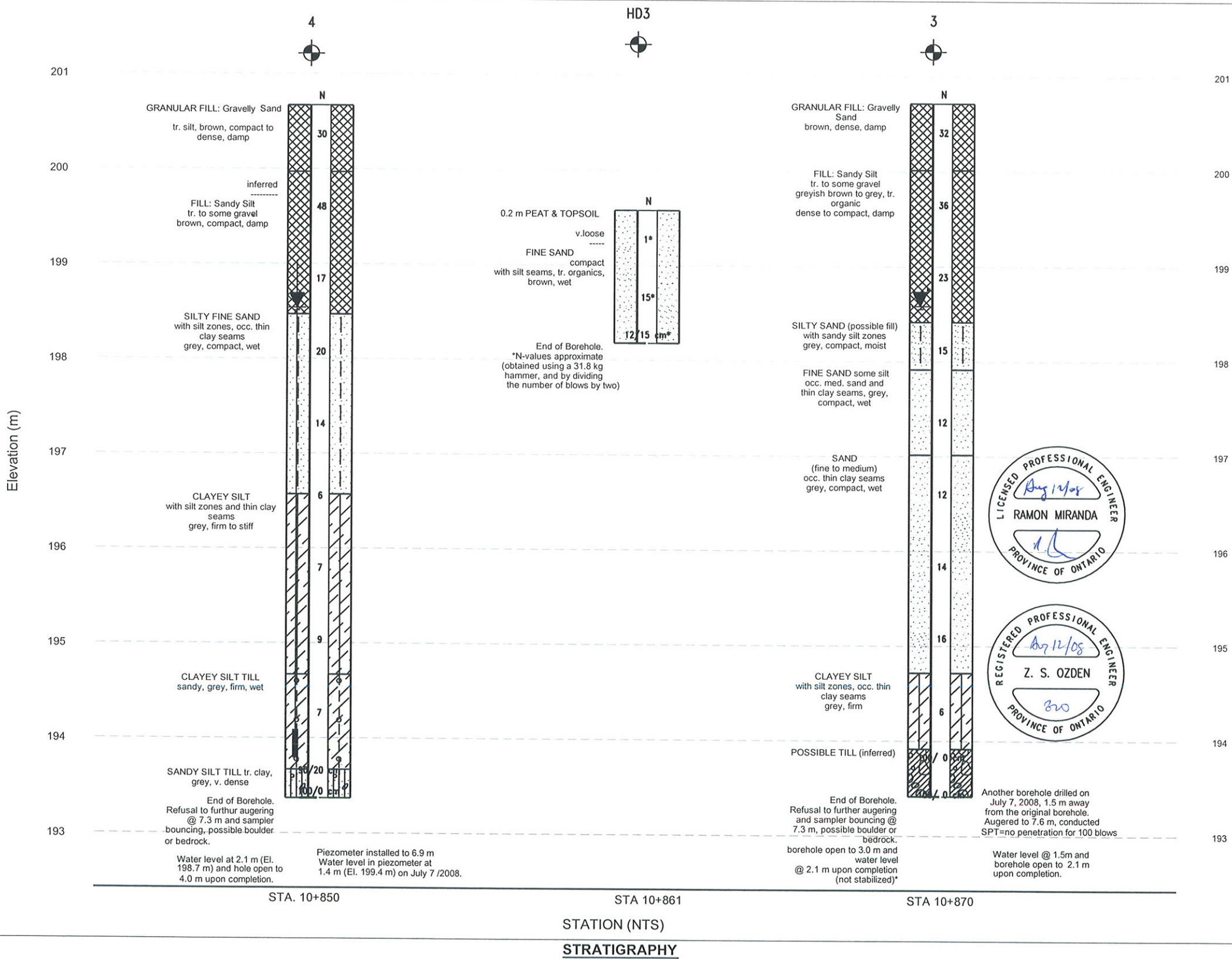
No.	ELEV.	STATION NO
BH-3	200.82	10+870
BH-4	200.81	10+850
HD-3	199.56	10+861

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
1			
Geocres No. 31 D-442			
SPT 1182 A-02			DIST
SUBM'D	CHECKED	DATE JUN 2008	SITE
DRAWN PK	CHECKED RM	APPROVED ZO	DWG 2



Appendix A

Record of Borehole Sheets

SPT1182A-02: Highway 7 (Peterborough)

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

GWP: G.W.P. 173-98-00 LOCATION: (Sta : 10+870) 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: 6/12/2008 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				
200.8	GROUND SURFACE							20 40 60 80 100				
0.0	GRANULAR FILL: Gravelly Sand brown, dense, damp		1	SS	32		200					
200.1												
0.7	FILL: Sandy Silt tr. to some gravel greyish brown to grey, tr. organic dense to compact, damp		2	SS	36		199					
198.5			3	SS	23							
2.3	SILTY SAND (possible fill) with sandy silt zones grey, compact, moist		4	SS	15		198					
198.0												
2.8	FINE SAND some silt occ. med. sand and thin clay seams, grey, compact, wet		5	SS	12		197					
197.1												
3.7	SAND (fine to medium) occ. thin clay seams grey, compact, wet		6	SS	12		196					
			7	SS	14							
194.8			8	SS	16		195					
6.0	CLAYEY SILT with silt zones, occ. thin clay seams grey, firm		9	SS	6							
194.0												
6.8	POSSIBLE TILL (inferred)		10	SS	100.0 cm		194					
193.5			11	SS	100.0 cm							
7.3	End of Borehole. Refusal to further augering and sampler bouncing @ 7.3 m, possible boulder or bedrock. borehole open to 3.0 m and water level @ 2.1 m upon completion (not stabilized)* Another borehole drilled on July 7, 2008, 1.5 m away from the original borehole. Augered to 7.6 m, conducted SPT=no penetration for 100 blows. Water level @ 1.5m and borehole open to 2.1 m upon completion.											SS10+11 no penetration No Recovery 40 minutes to auger from 6.8 m to 7.3 m No Recovery

SPT1182A-02: Highway 7 (Peterborough)

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

GWP G.W.P. 173-98-00 LOCATION (Sta : 10+850) 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 6/13/2008 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 (%)
								20 40 60 80 100					
								20 40 60 80 100					
200.8	GROUND SURFACE												
0.0	GRANULAR FILL: Gravelly Sand tr. silt, brown, compact to dense, damp		1	SS	30							40 50 (10)	
200.1												SS2: sampler pushing gravel	
0.7	FILL: Sandy Silt tr. to some gravel brown, compact, damp	inferred	2	SS	48		200					no sample recovery	
			3	SS	17		199					N-value not reliable	
198.6	SILTY FINE SAND with silt zones, occ. thin clay seams grey, compact, wet		4	SS	20		198						
2.2			5	SS	14								
			6	SS	6		197						
196.7	CLAYEY SILT with silt zones and thin clay seams grey, firm to stiff		7	SS	7		196						
4.1			8	SS	9		195						
194.8	CLAYEY SILT TILL sandy, grey, firm, wet		9	SS	7		194						
6.0			10	SS	90/20 cm								
193.8	SANDY SILT TILL tr. clay, grey, v. dense		11	SS	100/0 cm							No Recovery	
7.0	End of Borehole												
193.5	Refusal to further augering @ 7.3 m and sampler bouncing, possible boulder or bedrock.												
7.3	Water level at 2.1 m (El. 198.7 m) and hole open to 4.0 m upon completion.												
	Piezometer installed to 6.9 m. Water level in piezometer at 1.4 m (El. 199.4 m) on July 7 /2008.												

+ 3, X 3 Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT1182A-02: Highway 7 (Peterborough)

RECORD OF BOREHOLE No HD3

1 OF 1

METRIC

GWP G.W.P. 173-98-00 LOCATION (Sta : 10+861) 18.0 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 w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
199.8	GROUND SURFACE							20	40	60	80	100				
0.0	0.2 m PEAT & TOPSOIL		1	SS	1*		199								132	
	FINE SAND		2	SS	15*											
198.2	with silt seams, tr organics, brown, wet		3	SS	12/15 cm*											
1.4	End of Borehole															
	*N-values approximate (obtained using a 31.8 kg hammer, and by dividing the number of blows by two)															

+³, ×³

Numbers refer to
Sensitivity

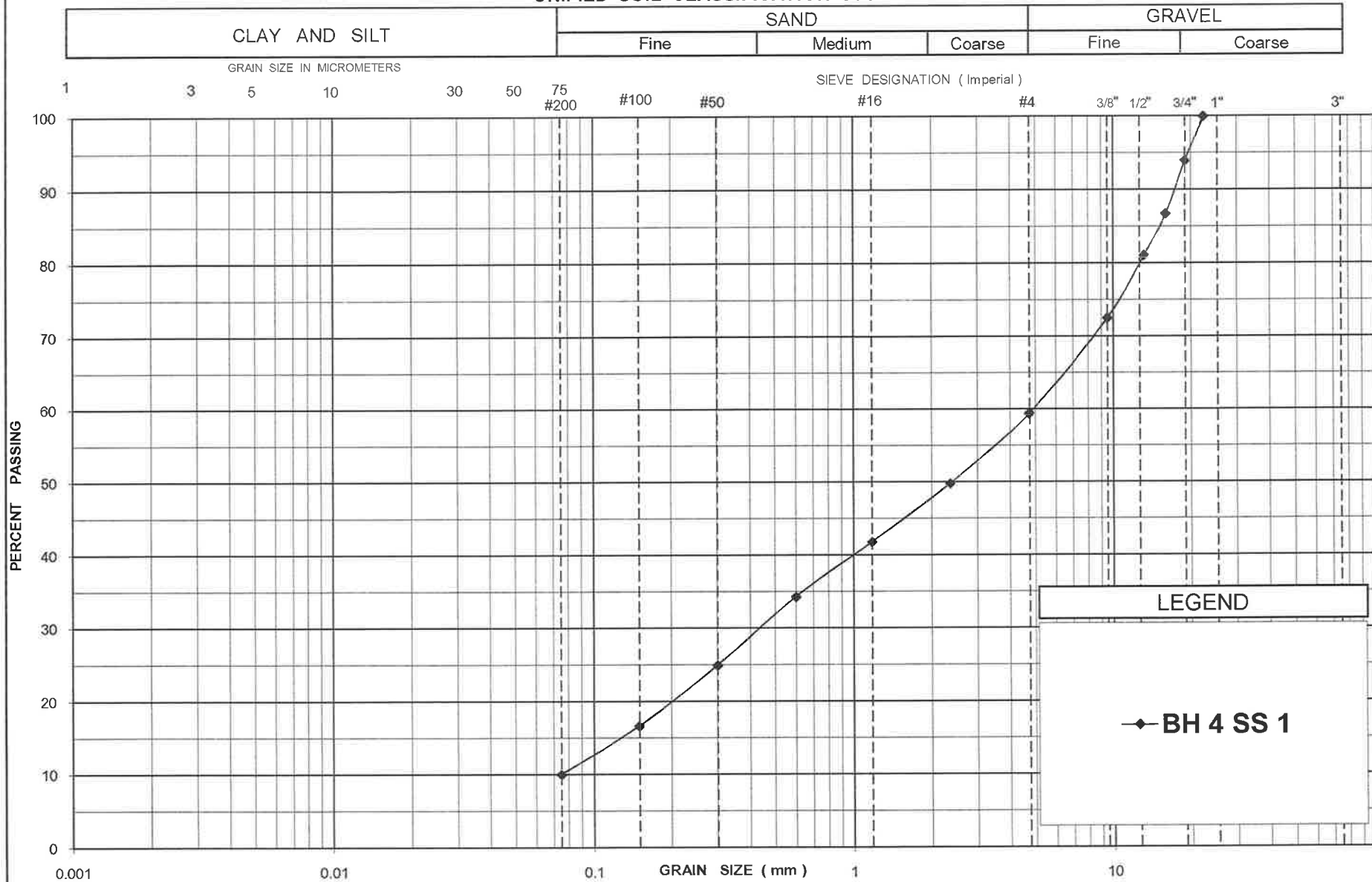
20
15
10

(%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER LIMITED
A Coffey Geotechnics Company

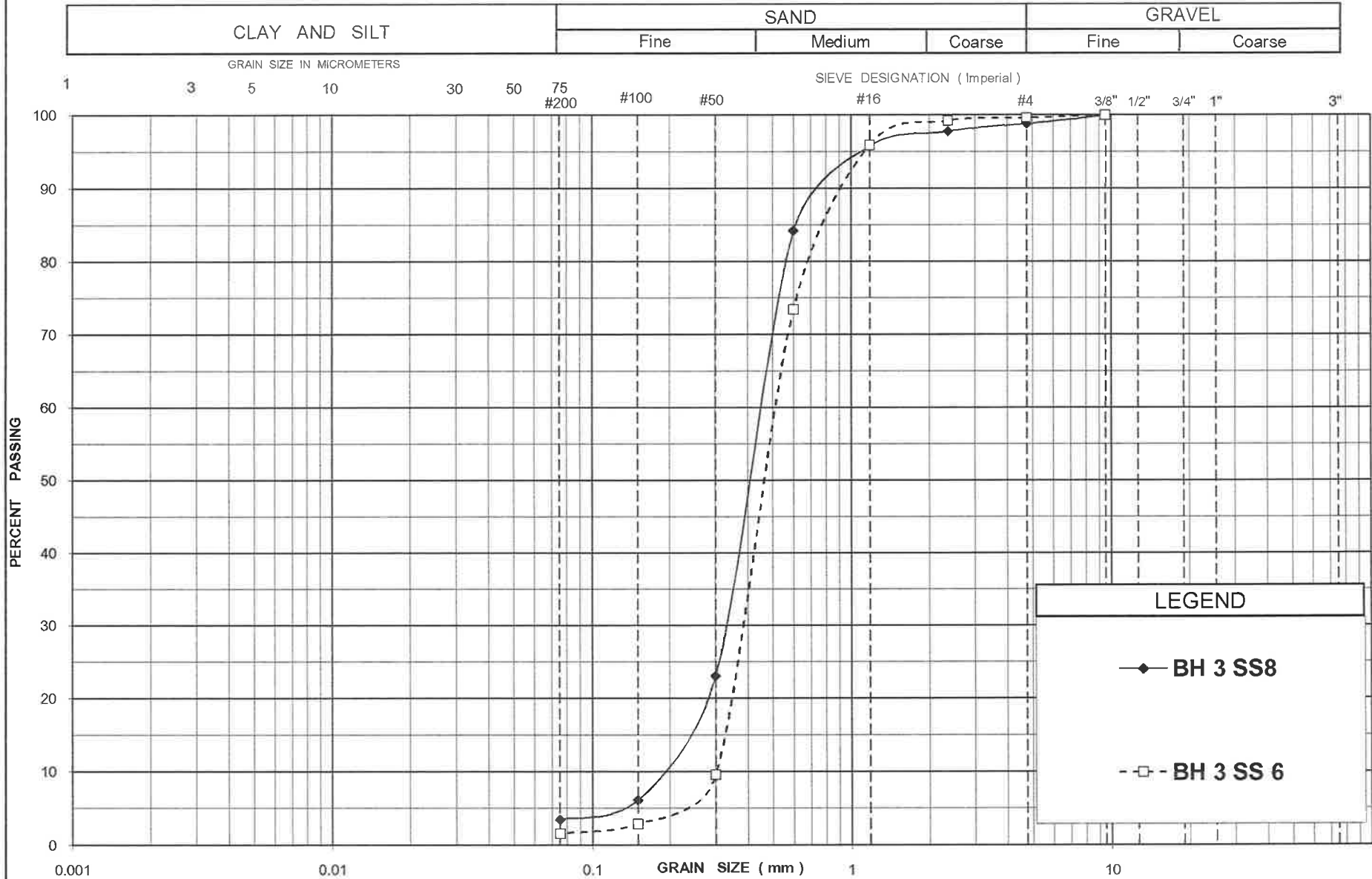
GRAIN SIZE DISTRIBUTION
GRANULAR FILL: GRAVELLY SAND, tr. silt

FIGURE No. B-1

REF. No. SPT 1182 A -02

DATE JUNE 2008

UNIFIED SOIL CLASSIFICATION SYSTEM



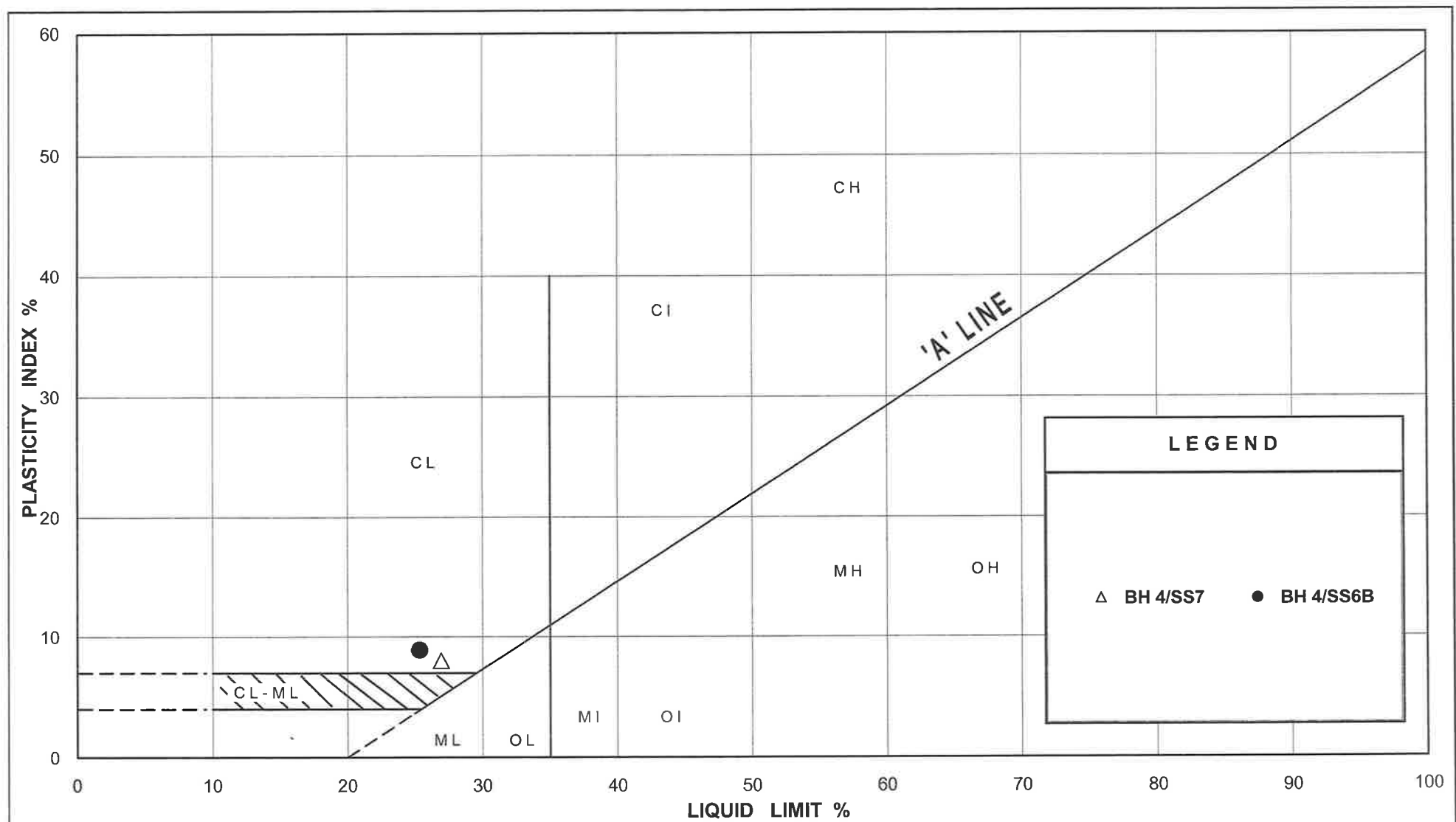
SHAHEEN & PEAKER LIMITED
A Coffey Geotechnics Company

GRAIN SIZE DISTRIBUTION
SAND (FINE TO MEDIUM)

FIGURE No. B-2

REF. No. SPT 1182 A-02

DATE JUNE 2008



SHAHEEN & PEAKER LIMITED A COFFEY GEOTECHNICS COMPANY	PLASTICITY CHART CLAYEY SILT, some silt zones and thin clay seams	FIGURE No. B-3
		REF. No. SPT 1182 A-02
		DATE June 27, 2008

Appendix C

Site Photograph



Photograph 1. Borehole 4

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 63.5 kg, 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 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	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
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_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	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	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^3	DENSITY OF WATER	w	1, %	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^3/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 = $(w_L - w) / 1_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
PROPOSED RETAINING WALLS AT
STATION 10+850-10+880
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET
PETERBOROUGH, ONTARIO
W.P. 581-93-00 CONTRACT NO. 2007-4005**

GEOCRES NO. 31D-442

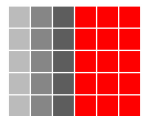
Prepared For:

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Prepared by:

SHAHEEN & PEAKER

**Project: SPT1182A-02
August 12, 2008**



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Table of Contents

5. DISCUSSION AND RECOMMENDATIONS	7
5.1 Reinforced Concrete Retaining Wall	8
5.1.1 Spread Footing Foundations.....	8
5.1.2 Deep Foundations.....	9
5.2 Contiguous Caisson Wall	10
5.3 Soldier Pile and Lagging System	11
5.4 Gabion Wall/Armour Stone Wall	11
5.5 Retained Soil System (RSS)	12
5.6 Lateral Earth Pressures	13
5.7 Construction	15
5.8 Frost Protection	16
6. CLOSURE	17

APPENDIX E: TYPICAL CROSS SECTION OF TOE WALL

APPENDIX F: LIMITATIONS OF REPORT

FOUNDATION DESIGN REPORT
PROPOSED RETAINING WALLS AT STATIONS 10+850-10+880
HIGHWAY 7, EASTERLY FROM LANSDOWNE STREET, PETERBOROUGH
W. P. NO. 581-93-00; CONTRACT NO. 2007-4005

5. DISCUSSION AND RECOMMENDATIONS

Due to property restrictions, a new retaining wall (toe wall) will be constructed along the north side of Highway 7, between approximate Stations 10+850 and 10+880. The retaining wall will be adjacent to a proposed roadside drainage ditch and therefore the wall and its foundation may be subject to erosion. The maximum elevation difference between the bottom of the ditch to top of the Highway will be about 2.5 m. The height of the wall will be about 1.8 m from the bottom of the ditch and beyond the top of the wall towards the highway normal embankment slope of 2H:1V will be constructed (see Figure E-1, in Appendix E).

Boreholes 3 and 4 which were put down from the top of the embankment showed the presence of embankment fill to about El. 2.3 m (El. 198.5 m), underlain by compact silty fine sand and sand deposits to 4.1 and 6.0 m (El. 196.7 and 194.8 m). These deposits are in turn underlain by firm to stiff clayey silt and clayey silt till to about 6.8 to 7.0 m (El. 194.0 and 193.8 m). The presence of very dense sandy silt till was found or inferred below these depths. The groundwater table at the time of our investigation was found at about El. 198.7-199.4 m, but would be subject to fluctuations.

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

Support Systems

Type of Support	Comments	Recommendations
Reinforced Concrete Retaining Wall	The bottom of foundations need to be below frost and scour depths. Subsurface conditions are not favourable for the use of normal spread footing foundations due to high water table and the weak nature of the soil deposits, especially at Borehole 4 location.	Structure supported on short caissons would be reliable, but relatively costly. Will require some dewatering and possibly temporary shoring.
Contiguous Caisson Wall	Reliable but uneconomical.	Not Recommended based on cost factor.
Permanent Soldier Pile & Lagging System	Will be an economical solution since tiebacks are unlikely to be required due to low height of retained embankment (i.e. about 2.5 m high)	Reliable. Less sensitive to dewatering in comparison with spread footing foundations but the cost should be checked. Will likely be the most attractive and expedient solution since temporary shoring will not be required.

Type of Support	Comments	Recommendations
Gabion Wall	Can be considered if some distortions and lateral yield can be tolerated. May not be acceptable due to scour.	Economical but may not be reliable, due to possible yielding which may induce some cracking in the roadway
Armour Stone Wall	Can be considered if some distortions and lateral yield can be tolerated. May not be acceptable due to scour.	Economical but may not be reliable due to possible lateral yield which may induce some cracking in the roadway.
Retained Soil System (RSS)	Can be considered but scour may be a problem. May require the expense of temporary shoring.	More costly than gabion or armour stone walls but more reliable. Less sensitive to dewatering in comparison with spread footing foundation. May not be feasible due to scour. Will probably require temporary shoring.

In addition to the above wall systems given in the above table, consideration can be given to modifying the embankment geometry such that the provision of retaining walls is eliminated altogether, if and where possible. Consideration can also be given to the use of a gabion mat or gabion revetment system. Rock slope protection can also be considered, where feasible.

5.1 REINFORCED CONCRETE RETAINING WALL

The borehole data show that a reinforced concrete retaining wall can be supported on spread footing foundations, as well as on deep foundations.

It is recommended that the position of the groundwater be considered when selecting the foundation support and the elevation for the foundations. In addition, the weak and compressible clayey silt layer, which was contacted at El. 194.8 m and 196.7 m, respectively in Boreholes 3 and 4, will need to be considered when selecting spread footing foundations. Below about El. 193.8-194.0 m a competent stratum exists which would likely be suitable to support deep foundations.

5.1.1 SPREAD FOOTING FOUNDATIONS

Bottom of the ditch elevations given to us are about El. 199.3 m at Borehole 4 location (Station 10+850) and El. 199.4 m at Borehole 3 location (Station 10+870). For frost protection the bottom of the footings will likely be at 1.6 m below these elevations or El. 197.8 m at Borehole 3 location and El. 197.7 m at Borehole 4 location, assuming this satisfies scour criterion for design.

At Borehole 3 location the following geotechnical (bearing) resistances are available between El. 198.0-197.0 m for footings placed on natural undisturbed sand subgrade.

Factored Bearing Resistance at U.L.S. = 150 kPa
Bearing Resistance at S.L.S. = 100 kPa

With this serviceability value, the maximum total and differential settlements should not exceed 25 mm and 20 mm, respectively.

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 silty fine sand surface can be calculated using a friction angle of 28 degrees.

All footing excavations should be certified as per 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 dilation and disturbance of the approved bearing subgrade.

At Borehole 4 findings are less favourable than Borehole 3 for spread footing foundations (i.e. the weak clayey silt layer is at a higher elevation or closer to the bottom of the footing). The following geotechnical resistances are based on Borehole 4 data, assuming that the footings will be placed on undisturbed natural soil.

Bearing Elevation (m) (Underside Footing)	Footing Size (m)	Recommended Factored Bearing Resistance at ULS (kPa)	Recommended Factored Bearing Resistance at SLS (kPa)	Estimated Maximum Settlement (mm)
198.6-197.7	1.5-1.9	140	80	35
	2.0-2.4	130	80	40
	2.5-2.7	120	80	50

As these values are not favourable (i.e. will result in large footing sizes and over excavation) and the location of the change from more favourable conditions at Borehole 3 location is unclear, the use of a reinforced concrete structure supported on spread footing foundations is not recommended. When making this recommendation the dewatering that would be required in order to preserve the bearing resistance of the soil and to facilitate construction along with the cost of temporary shoring which will likely be required were taken into consideration.

5.1.2 DEEP FOUNDATIONS

As the conditions for the use of normal (shallow) spread footing foundations are unfavorable, consideration can be given to the use of deep foundations.

Boreholes 3 and 4 show the presence of a hard layer, or possibly bedrock at depths of 6.8 and 7.0 m or El. 194.0-193.8 m, respectively. The following are some possible alternatives

which can be considered, 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 the pile driving may be objectionable. However, the use of drilled and poured in placed concrete (caisson) foundations could be a viable alternative, since the anticipated caisson lengths are rather short and 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). Furthermore, the dewatering requirements would be much less rigorous in comparison with shallow foundations.

For preliminary estimating and 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 properly cleaned of debris and loosened material inspected and approved. For this reason the minimum caisson diameter should be 0.76 m. The temporary steel casing would be carefully withdrawn as the concrete is pouring, ensuring no 'necking.' QVE inspection should be provided in accordance with SP 903S01.

Some dewatering may be required during the installation of the caisson, especially through the water-bearing, non cohesive (granular) soils (i.e. silty fine sand and sand deposits), to facilitate the installation of the caissons. As well, the cleaning of the base, inspection/approval and the pouring of the concrete should be carried out rapidly so as to prevent disturbing the caisson base due to upward hydrostatic pressure.

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 only 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. These conditions may make it difficult to achieve a contiguous system.

As well, the geometry of the side slope may not present the opportunity to maintain the concrete while it is curing.

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. As well, the concrete used may need to be erosion and pollutant resistant. For these reasons, this type of wall is not recommended.

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 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 scour and pollution (e.g. acid water flowing in the ditch, salt, elements, etc) resistant. 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 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 is that temporary shoring will not be required. 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 caisson 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 typically extend about 0.6 m beyond the perimeter of the footprint of the wall (this distance can be reduced to 0.3 m at the back of the wall, where stresses would be lower) 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 to distribute stresses over a larger area in both directions under the wall, thus reducing distortions. 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 2 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)
3	200.8	198.5
4	200.8	198.6
HD3	199.6	198.9

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.

These support systems may not be acceptable due to scour considerations 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 a 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).

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. 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 or OPSD 3120.100 (whichever is applicable) 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.

5.7 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 silty sand to silt deposits followed by clayey silt and clayey silt till (Borehole 4 only) which are in turn underlain by very dense sandy silt till (inferred in Borehole 3). 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 Embankment Fill:	Type 3 soil above water level Type 4 soil below water level
Silty Fine Sand & Sand	Type 3 soil above water table (if the soil was not dewatered) Type 4 soil below water table
Clayey Silt	Type 3 soil above water table Type 4 soil below water table
Sandy Silt Till	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 0.8 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 1.0 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 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 system should be designed by a Professional Engineer, with experience 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 5.7.1 can be used for the design of the shoring system.

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
Organic Rich Soils	0.6	0.8	1.0	14.0
Silty Sand to Sand	0.36	0.53	2.8	19.0
Clayey Silt	0.48	0.66	2.0	16.5
Sandy Silt Till	0.30	0.45	3.6	22.0

Alternatively, to avoid shoring, a staged excavation/construction in controlled lengths can be considered, if geometry permits this approach.

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 F, are an integral part of this report.

SHAHEEN & PEAKER LIMITED


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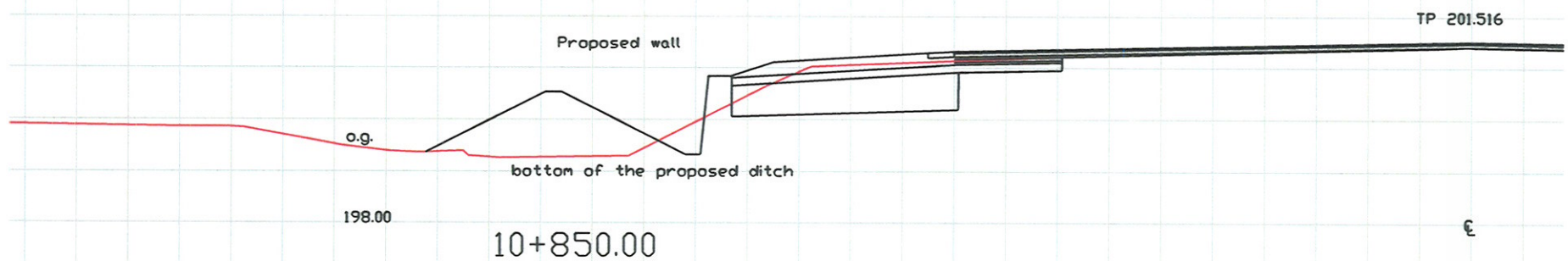
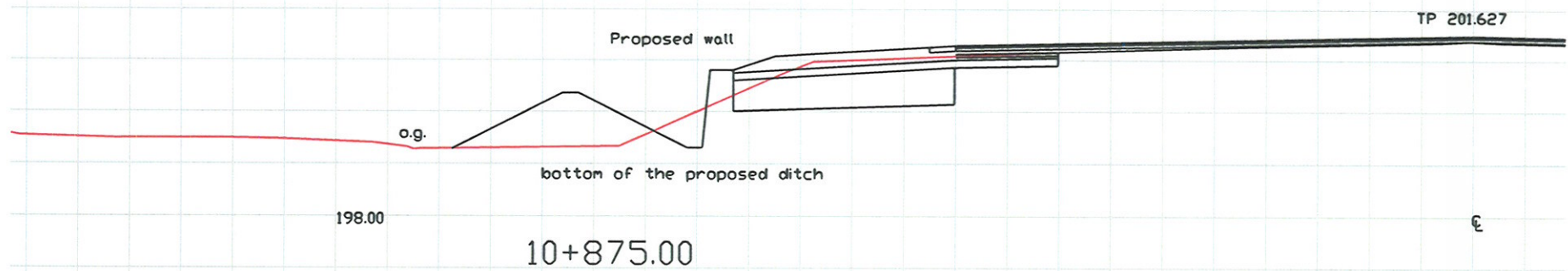

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Appendix E

Typical Cross Section of Toe Wall



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A COFFEY GEOTECHNICS COMPANY

TYP. CROSS SECTION OF TOE WALL @ STA. 10+850 to 10+870

SPT 1182 A-02			DIST
SUBM'D	CHECKED	DATE JULY 2008	SITE
DRAWN SS	CHECKED RM	APPROVED ZO	DWG E-1

Appendix F

Limitations of Report

LIMITATIONS OF REPORT

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.