

GEOCRES No:
31L-130

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
RESMER CREEK CULVERT (C28), STATION 26+500
HIGHWAY 17, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00
AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-130**

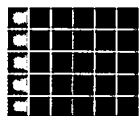
Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

**SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.**

**Project: SPT1211F
January 8, 2009**



shaheen & peaker

**20 Meteor Drive
Toronto, Ontario
M9W 1A4
Tel: (416) 213-1255
Fax: (416) 213-1260
EMAIL: Info@shaheenpeaker.ca**

**FOUNDATION INVESTIGATION REPORT
RESMER CREEK CULVERT (C28), STATION 26+500
HIGHWAY 17, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00
AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-130**

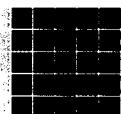
Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

**SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.**

**Project: SPT1211F
January 8, 2009**



shaheen peaker

**20 Meteor Drive
Toronto, Ontario
M9W 1A4
Tel: (416) 213-1255
Fax: (416) 213-1260
EMAIL: Info@shaheenpeaker.ca**

Table of Contents

1. INTRODUCTION	1
2. SITE DESCRIPTION AND PHYSIOGRAPHY	1
3. INVESTIGATION PROCEDURES	2
4. SUBSURFACE CONDITIONS	4
4.1 Embankment Fill.....	4
4.2 Organic Soil	5
4.3 Upper Sand	5
4.4 Layered Silt/Silty Fine Sand/Sandy Silt/Clayey Silt	6
4.5 Basal Granular Soils.....	6
4.6 Bedrock	7
4.7 Groundwater Conditions.....	8

DRAWING	DRAWING No.
BOREHOLE LOCATIONS & SOIL STRATA	1
APPENDIX A: RECORD OF BOREHOLE SHEETS	
APPENDIX B: LABORATORY TEST RESULTS	
APPENDIX C: ROCK CORE PHOTOGRAPHS	
APPENDIX D: SITE PHOTOGRAPHS	
APPENDIX E: EXPLANATION OF TERMS USED IN REPORT	

**FOUNDATION INVESTIGATION REPORT
RESMER CREEK CULVERT (C28)
STATION 26+500, HIGHWAY 17
CAMERON TOWNSHIP, MATTAWA, ONTARIO
G.W.P. 173-98-00; AGREEMENT NO. 5006-E-0040**

1. INTRODUCTION

As part of the rehabilitation of Highway 17 (from 9.5 km east of Highway 533, near Mattawa, easterly for 14.9 km) it is also planned to rehabilitate the existing culvert carrying Resmer Creek under Highway 17. This culvert is located at Station 26+500 and was assigned the number C28 for the purposes of this project.

Shaheen & Peaker (S&P), A Division of Coffey Geotechnics Inc. was retained by D.M. Wills Associates Limited (WILLS) to carry out a foundation investigation at the site of the proposed rehabilitation of the existing Resmer Creek culvert (C28) under Highway 17 at Station 26+500 in Cameron Township, near Mattawa, Ontario.

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

The report presents the findings of the investigation.

2. SITE DESCRIPTION AND PHYSIOGRAPHY

The project site is located on Highway 17, between Mattawa and Deux-Rivieres, some 70 km east of North Bay, Ontario.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the site is located within the Physiographic Region known as the Algonquin Highlands. Much of this region is underlain by Precambrian granitic bedrock. Locally, relief is rough, rounded knobs and ridges standing up, usually 15 to 60 m but occasionally up to 150 m high. The overburden is generally shallow but its thickness over the bedrock varies greatly over short distances. Many of the valleys are floored with outwashes of sand and gravel, with frequent swamp and bogs in the hollows. The northern part of Algonquin lake plain, that extends east to near Mattawa, shows the presence of silty clay, silt and sand deposits. In general, the highway in the project area appears to be built along spillways and shallow rock ridges, along with shallow till deposits.

According to Bedrock Geology of Ontario Map 2544 (Ministry of Northern Development and Mines, Ontario), the bedrock underlying the site are Mesoproterozoic Precambrian rocks

(i.e. approximately 900 million years old), which consist of primarily felsic igneous tonalite, granodiorite, monzonite, granite, syenite and derived gneisses.

The site is located in between two rock knobs, with a grade sharply falling towards a creek valley (Resmer Creek) in between knobs. Starting from the west side, the grade at the o.g. level falls sharply towards the east (i.e. towards the project site), to about El. 208 m at about Station 26+050. The grade continues to fall to about El. 200 m at Station 26+250. From thereon, the grade falls more gradually towards the Resmer Creek at about Station 26+500 to about El. 192 m. Beyond the creek, the grade rises easterly to about 204 m at about Station 26+800, and finally, to a rock outcrop (i.e. top of the second knob) at about Station 27+100.

3. INVESTIGATION PROCEDURES

The fieldwork at this project site was performed on April 24, April 25, April 27 and May 13, 2008. The field investigation consisted of drilling and sampling three boreholes (Boreholes C28-1, C28-2 and C28-3) to depths of between 7.8 m and 13.7 m below the ground surface, as follows:

Borehole C28-1 (Station 26+475),	5.4 m Rt of CL	7.8 m deep
Borehole C28-2 (Station 26+509),	5.0 m Rt of CL	13.7 m deep
Borehole C28-3 (Station 26+510),	18.5 m Rt of CL	11.0 m deep

In addition, a Dynamic Cone Penetration test was performed at Borehole C28-4 (Station 26+490).

Landcore Drilling Inc. of Chelmsford, Ontario, drilling contractor, carried out the drilling, testing and sampling under the supervision and direction of a Professional Engineer from S&P.

The locations of the boreholes in the field are given on the Borehole Location Plan, Drawing No. 1.

Additional boreholes were drilled to the east and west of the culvert location. These boreholes were reported under separate cover. Reference can be made to our report entitled "Foundation Investigation Report, Proposed Widening of Highway 17 from 9.5 km East of Highway 533, Easterly 14.9 km, Mattawa, Ontario, G.W.P. 173-98-00; Agreement No. 5006-E-0040" Ref. No. SPT1211B, for additional borehole data.

Samples of the soil in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test Method (SPT), in general accordance with ASTM D1586. This test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter O.D. split barrel (SS 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 which is indicative of the compactness condition of granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

Borehole C28-1 terminated upon encountering refusal to further sampling (i.e. split-spoon sampler bouncing without any penetration) and refusal to further augering at 7.8 m below the ground surface. In the remaining two boreholes (i.e. Boreholes C28-2 and C28-3), after encountering practical refusal on the augers, the bedrock was proven by diamond drilling methods whereby NQ size rock cores were obtained. The length of coring by diamond drilling was 3.8 m and 3.5 m in Boreholes C28-2 and C28-3, respectively.

As mentioned before, a Dynamic Cone Penetration Test was performed at Station 26+490 (Borehole C28-4). 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. In the present case, the DCPT was terminated when the number of blows to drive the cone/rod assembly by 0.05 m exceeded 100. At Station 26+490, due to drill inaccessibility, the DCPT was conducted manually using a 31.8 kg hammer (instead of 63.5 kg) similar to SPT method. In this case, the recorded number of blows was divided by two to obtain an approximate equivalent resistance value.

Groundwater conditions in the boreholes were observed during the drilling in the open boreholes. Upon their completion, the boreholes were grouted using a cement/bentonite mixture as per MTO procedures.

The details of the drilling, sampling, field testing and soil conditions encountered are given on the Record of Borehole Sheets in Appendix A. An inferred subsurface profile is presented in Drawing No. 1.

A laboratory testing programme, consisting of natural moisture content measurements and grain-size analyses was performed on selected soil samples. The results of laboratory tests are presented on the appropriate Record of Borehole Sheets and also in Appendix B.

The ground surface elevations at the borehole locations were provided to us by our client (D. M. Wills Associates Limited). We understand that the elevations are related to the Geodetic Datum.

4. SUBSURFACE CONDITIONS

This investigation consisted of drilling and sampling three boreholes. As well, a dynamic cone penetration test was performed at Station 26+490.

At Station 26+295 (i.e. about 200 m west of the creek location) the top of road elevation is about 201.0 m, dropping to about El. 198.3 m at Station 26+500 (i.e. creek location). From thereon, the top of road elevation rises easterly to El. 198.7 m at Station 26+575 and continues to rise more steeply in the easterly direction. The original grades (o.g.) in this stretch are between about 199.6 and 200.8 m between Stations 26+295 and 26+450 and falls sharply thereafter to Resmer Creek at Station 26+510 area to about El. 191 m and rising sharply to about El. 197.6 m at Station 26+575. The height of the embankment varies from about zero (i.e. almost at grade) to about 7 m at Resmer Creek.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A, while a stratigraphic profile is given in Drawing No. 1. The various soil strata encountered in the boreholes and their geotechnical properties are briefly described in the following paragraphs. It should be noted that the soil and groundwater conditions may vary in between and beyond the borehole locations.

4.1 EMBANKMENT FILL

Boreholes C28-1 and C28-2 were drilled from the top of the road embankment (shoulder area) and consequently encountered embankment fill materials to depths of 2.3 m (Borehole C28-1) and 5.4 m (Borehole C28-2).

Granular pavement (i.e. gravelly sand) fill was contacted to depths of 0.7 m (Borehole C28-2) and 0.9 m (Borehole C28-1). Standard Penetration tests yielded N-values of 19 and 26 blows/0.3 m in the granular fill indicating a compact condition.

Below this upper granular fill, the embankment fill was found to consist typically of fine sand with traces to some silt and occasional gravel, which is a basically granular (i.e. non-cohesive) material. The grain-size distribution samples from this fill was determined on three samples and these show the following grain-size distribution, as presented in Figure B-1 in Appendix B.

Gravel:	0-3%
Sand:	87-92%
Silt & Clay:	8-10%

Standard Penetration tests performed in the embankment fill yielded N-values of between 7 and 20 blows/0.3 m, indicating a loose to compact relative density.

Fill was also contacted in Borehole C28-3, which was drilled near the toe of the embankment. In this borehole, the fill was found to extend to a depth of 2.3 m or to El. 191.5 m. The upper 0.3 m of this fill was found to consist of topsoil underlain by sand fill with traces of gravel. The sand fill is in turn underlain by a rather random type fill consisting of silty fine sand with traces of gravel, asphalt pieces and organics. This is a basically granular (i.e. non-cohesive) soil. N-values of 2, 8 and 2 blows/0.3 m were recorded in this fill indicating that it has not received compaction.

It should be pointed out that in our experience the thickness of fill deposits can vary in between and beyond borehole locations.

4.2 ORGANIC SOIL

As discussed in Section 4.1, Borehole C28-3, which was located near the bottom of the valley, contacted a 2.3 m thick random, loosely placed mixed fill at El. 191.5 m. This fill was found to be underlain by a 0.3 m thick sandy organic silt layer. It appears that at this location organic soil was not stripped before the random fill was dumped.

We would like to point out that in our experience the thickness and extent of organic soils can vary in between and beyond borehole locations. The presence of thicker organic soils can be expected in depressed areas and in particular near watercourses (e.g. Resmer Creek valley).

4.3 UPPER SAND

A gravelly sand deposit was encountered in Borehole C28-2, immediately underlying the embankment fill. The grain-size distribution of a sample of the gravelly sand from this borehole is given in Figure B-2 in Appendix B. The curve indicates the following grain-size distribution:

Gravel:	28%
Sand:	60%
Silt & Clay:	12%

N-values recorded in this granular deposit range from 6 to 19 blows/0.3 m. These results indicate a loose to compact condition.

A finer grained upper granular deposit was contacted in Borehole C28-3. This material, which consists of silty fine sand with traces of gravel, was contacted below the sandy organic silt layer (underlying fill) at a depth of 2.6 m and extended to 3.7 m (i.e. 1.1 m thick layer). N-values of 2 and 4 blows/0.3 m were recorded indicating a very loose condition.

4.4 LAYERED SILT/SILTY FINE SAND/SANDY SILT/CLAYEY SILT

Underlying the fill in Borehole C28-1 at 2.3 m (El. 196.0 m), gravelly sand in Borehole C28-2 at 8.4 m (El. 189.8 m) and the fine sand in Borehole C28-3 at 3.7 m (El. 190.1 m), all three boreholes contacted a layered deposit consisting of silt, silty fine sand, sandy silt and clayey silt with occasional very thin, clay interbeds. The thickness of this deposit was recorded to be 2.0 m, 1.3 m and 2.2 m and the deposit was found to extend to Elevations 194.0, 188.5 and 187.9 m, respectively at Boreholes C28-1, C28-2, and C28-3, respectively.

The deposit is a basically fine-grained granular (i.e. non-cohesive) soil type with some cohesive interbeds.

The grain-size distribution of two samples from the deposit from Borehole C28-1 is given in Figure B-3, in Appendix B. the curves indicate the following grain-size distribution.

Gravel:	0-1%
Sand:	35-49%
Silt :	40-53%
Clay:	10-12%

When analyzing these results, it should be kept in mind that these results represent an amalgamation of a number of interbeds, each with different grain-size characteristics and that a good portion of the clay size particle percentage would likely be due to the presence of clayey silt and thin clay seams.

Standard Penetration tests performed in the deposit yielded N-values which range from 8 to 14 blows/0.3 m in Borehole C28-1, indicating a loose to compact condition. In Boreholes C28-2 and C28-3, located closer to the existing culvert, the recorded N-values are between 3 and 4 blows/0.3 m, indicating a typically very loose material with some soft zones. It should also be pointed out that these results reflect some strength grain under the weight of the fill at the borehole locations (i.e. prior to the placement of the fill materials, the N-values would probably have been somewhat lower).

4.5 BASAL GRANULAR SOILS

In Boreholes C28-1 and C28-3 the interbedded silt deposit, described in Section 4.4 of this report, is underlain by granular soils ranging from silty fine sand with traces of gravel (Borehole C28-1) to sand with some gravel (Borehole C28-3). These granular soils were encountered at depths of 4.3 m/El. 194.0 m at Borehole C28-1 and 5.9 m/El. 187.9 m in Borehole C28-3 and extended to 7.8 m/El. 190.5 m (auger refusal) and 7.3 m/El. 186.5 m, respectively.

The grain-size distribution of a sample of the silty fine sand from Borehole C28-1 is given in Figure B-4. The following grain-size distribution is indicated.

Gravel:	2%
Sand:	70%
Silt & Clay:	28%

N-values recorded in these deposits typically range from 10 to 25 blows/0.3 m which indicate a generally compact condition.

In Borehole C28-2, the basal granular deposit was found to be a very coarse grained material, consisting of cobbles/boulders and rock fragments with some sand and gravel infill immediately above the bedrock. The presence of a similar bouldery zone with shattered rock was also found in the lower zone of the sand deposit in Borehole C28-3 immediately above the bedrock.

4.6 BEDROCK

The presence of bedrock was proven by rock coring in Boreholes C28-2 and C28-3. The surface of the rock was found at about 5.3 m and 5.6 m below the o.g. levels or at El. 187.5 m and 185.9 m, respectively.

These two boreholes are located about 1 m apart in the longitudinal direction along the highway and about 13.5 m apart in the transverse direction (i.e. perpendicular to the highway). It appears therefore that the bedrock surface dips by 1.6 m in the transverse direction within a distance of 13.5 m or at a gradient of about 12%, from about 5 m from the centerline of the highway to 18.5 m from the centerline, in the southerly direction near the Resmer Creek Culvert location at about Station 26+510.

The rock was cored for a vertical distance of about 3.0 m. Visual examination of the rock cores showed that the bedrock consists of a granitic rock formation with some metamorphosed magmatic rock intrusions. It is mainly pinkish grey. In addition to horizontal and nearly horizontal fractures, the presence of vertical and subvertical fractures with some slight sediment inclusions and oxidization was noted. The percentage of recovery was 100% while the RQD values ranged from zero to typically 50 to 65%. These results show that within the depths cored, the upper 1.5 m of the rock in Borehole C28-2 is highly fractured while the rest of rock is considered to be relatively sound.

The photographs of the rock cores are given in Appendix C.

In Borehole C28-1, practical refusal to augering was encountered at a depth of 7.8 m or El. 190.5 m, probably on the surface of the bedrock or close to it.

4.7 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling and upon completion of each borehole.

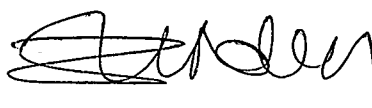
The details of the observations are shown on the Record of Borehole Sheets. These indicate that the groundwater levels at the time of our investigation were close to the o.g. (original ground) levels.

It should, however, be pointed out that the groundwater table can be expected to undergo seasonal fluctuations as well as fluctuations in response to major weather events.

SHAHEEN & PEAKER

A Division of Coffey Geotechnics Inc.


Ramon Miranda, P.Eng.



Z.S. Ozden, P.Eng.



ZO:tr/idrive

Drawing

METRIC

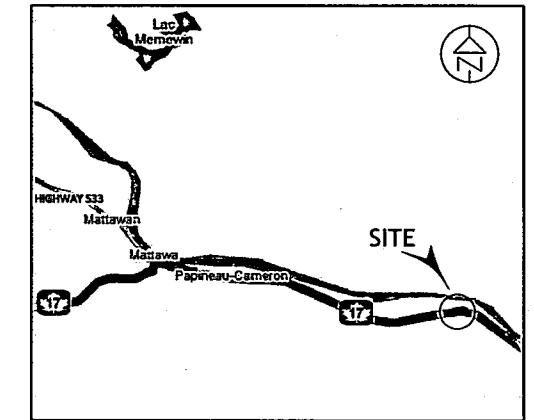
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No. 5006-E-0040

GWP: 173-98-00

Highway 17 Mattawa
BOREHOLE LOCATION PLAN &
STRATIGRAPHY (Culvert 28 @ 26+500)

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Dynamic Cone Penetration Test
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	STATION NO.	OFFSET
C28-1	198.3	26+475	5.4 m Rt
C28-2	198.2	26+509	5.0 m Rt
C28-3	193.8	26+510	18.5 m Rt
C28-4	191.5	26+490	21.5 m Rt

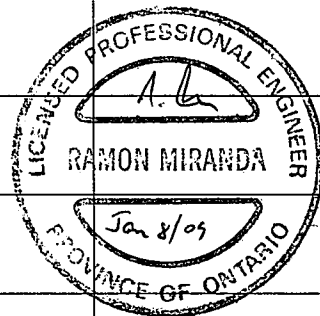
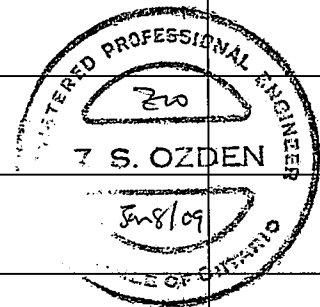
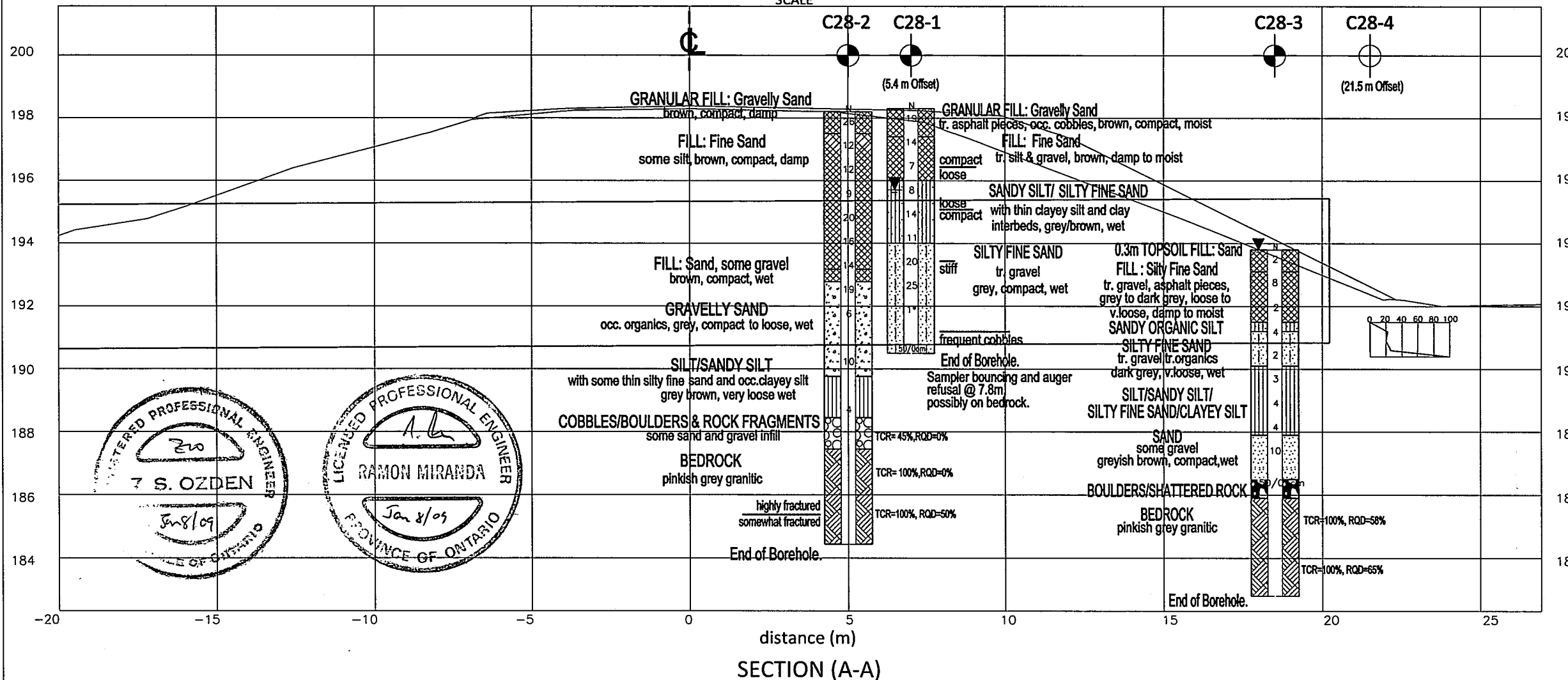
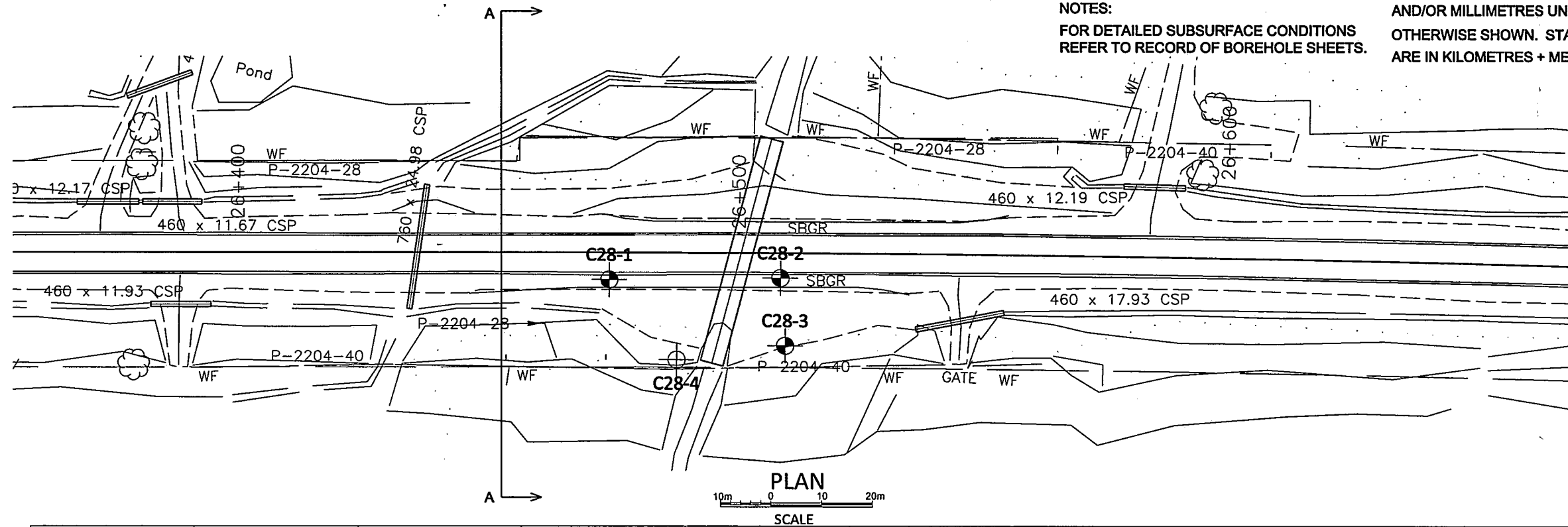
=NOTE=
The boundaries between soil strata have been established only at Borehole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

REV.	DATE	BY	DESCRIPTION

Geocres No. 31L-130

SUBM'D	CHECKED	DATE	SITE
		Oct. 2008	
DRAWN	PHK	CHECKED	RM
		APPROVED	ZO
		DWG	1



Appendix A

Record of Borehole Sheets

SPT 1211F : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C28-1

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 26+475 : 5.4m Rt C/L of Hwy 17 (Shoulder) (D-0.1m) ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 4/25/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
198.3	GROUND SURFACE							20 40 60 80 100	10 20 30					
0.0	GRANULAR FILL: Gravelly Sand tr. asphalt pieces, occ. cobbles brown, compact, moist		1	SS	19	G.W. TABLE	198							
197.4														
0.9	FILL: Fine Sand tr. silt & gravel compact loose brown, damp to moist		2	SS	14		197							
			3	SS	7									
196.0	SANDY SILT/SILTY FINE SAND loose compact with thin clayey silt and clay interbeds, grey/brown, wet stiff		4	SS	8		196							
2.3			5	SS	14		195							
			6	SS	11		194							
194.0			7	SS	20		193							
4.3	SILTY FINE SAND tr. gravel grey, compact, wet Frequent cobbles		8	SS	25		192							
			9	SS	1*		191							
190.5			10	SS	50/0cm									
7.8	End of Borehole. Sampler bouncing and auger refusal @ 7.8 m, possibly on bedrock. Water level in open hole @ 2.6 m upon completion.													

+³, X³: Numbers refer to
Sensitivity

20
15-5
10 (%) STRAIN AT FAILURE

SPT 1211F : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C28-2

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 26+509 : 5m Rt C/L of Hwy 17 (Shoulder) (D-0.2m) ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NQ Coring COMPILED BY SS
DATUM Geodetic DATE 4/24/2008 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 (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					WATER CONTENT (%)		
								20 40 60 80 100							
								20 40 60 80 100							
					○ UNCONFINED	+ FIELD VANE				W P	W	W L			
					● POCKET PENETR.	X LAB VANE									
198.2	GROUND SURFACE														
0.0	GRANULAR FILL: Gravelly Sand brown, compact, damp		1	SS	26		198								
197.5															
0.7	FILL: Fine Sand some silt brown, compact, damp		2	SS	12		197						0 92 (8)		
			3	SS	12		196								
			4	SS	9		195								
			5	SS	20		194								
			6	SS	16		193						0 90 (10)		
193.2		wet	7	SS	14		192								
5.0	FILL: Sand, some gravel brown, compact, wet														
192.8															
5.4	GRAVELLY SAND occ. organics, grey, compact to loose, wet		8	SS	19		191								
			9	SS	6		190								
			10	SS	10		189								
189.8															
8.4	SILT/SANDY SILT with some thin silty fine sand and occ. clayey silt and clay interbeds grey brown, very loose wet														
			11	SS	4		188								
188.5															
9.7	COBBLES/BOULDERS & ROCK FRAGMENTS some sand and gravel infill														
187.5			12	RC	TCR= 45% RQD=0%		187								
10.7	BEDROCK pinkish grey granitic														
		highly fractured	13	RC	TCR= 100% RQD=0%		186								
		somewhat fractured													
			14	RC	TCR=100% RQD=50%		185								
184.5															
13.7	End of Borehole.														

+ 3. x 3. Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1211F : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C28-4 (DCPT)

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 26+490: 21.5m Rt of C/L of Hwy 17 ORIGINATED BY GH
DIST HWY 17 BOREHOLE TYPE DCPT COMPILED BY SS
DATUM Geodetic DATE 5/13/2008 CHECKED BY ZO

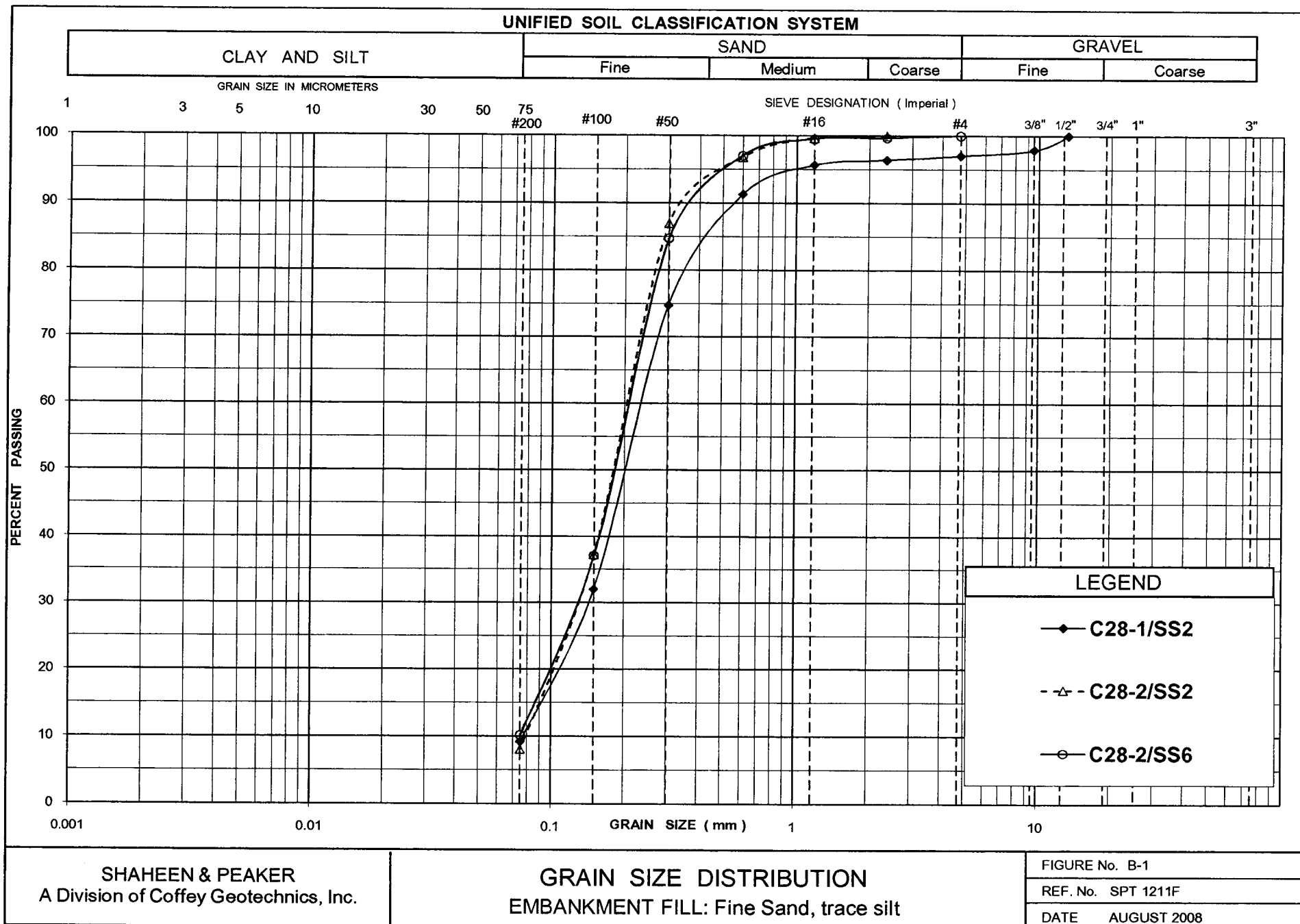
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELFV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
191.5	GROUND SURFACE												
0.0							191						
190.1	Dynamic Cone Penetration Test (DCPT) performed from 0 to 1.4 m												DCPT hole moved 3 times due to shallow refusal.
1.4	End of DCPT.												

+³ . X³ : Numbers refer to
Sensitivity

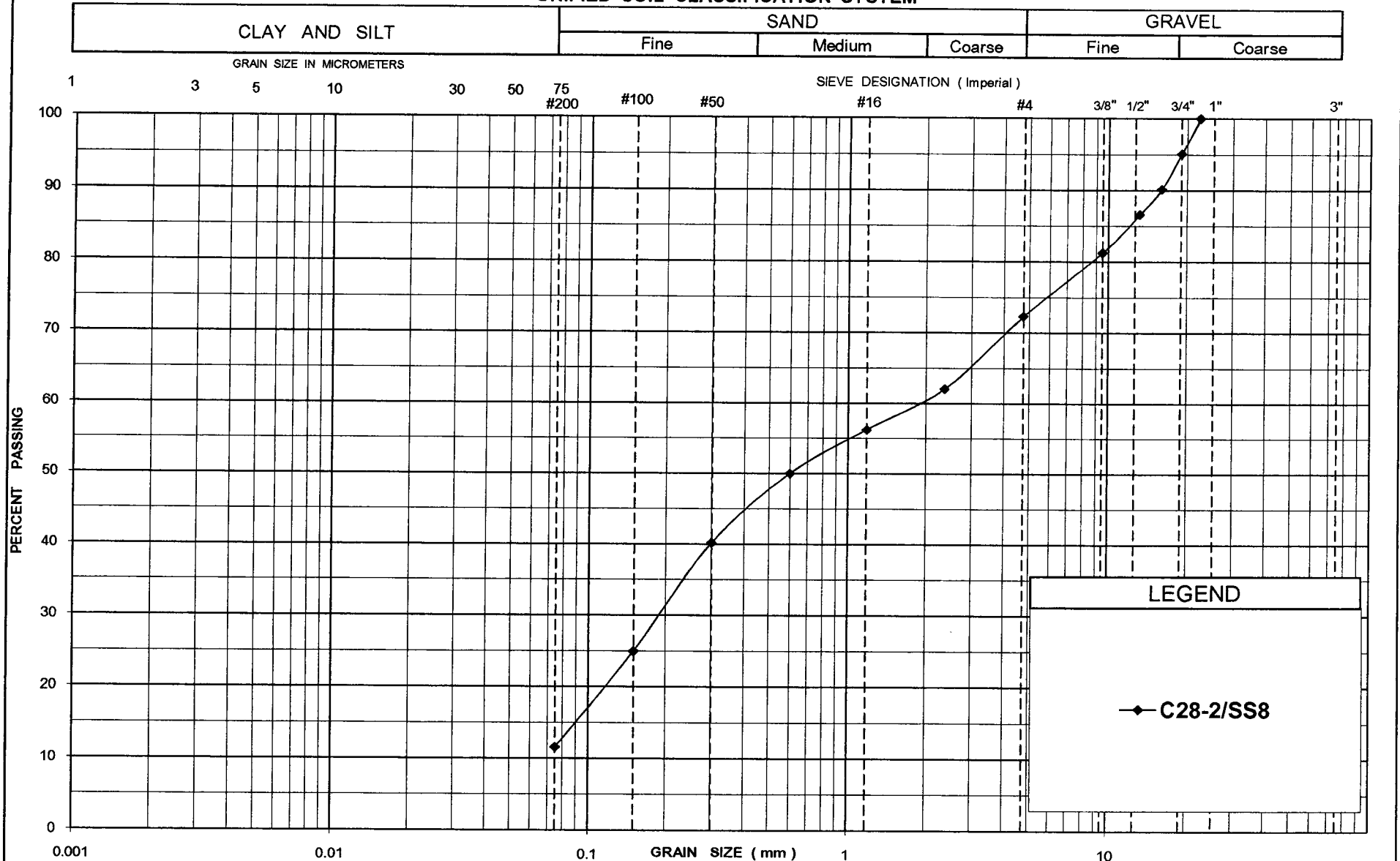
20
15 5
10 (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results



UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

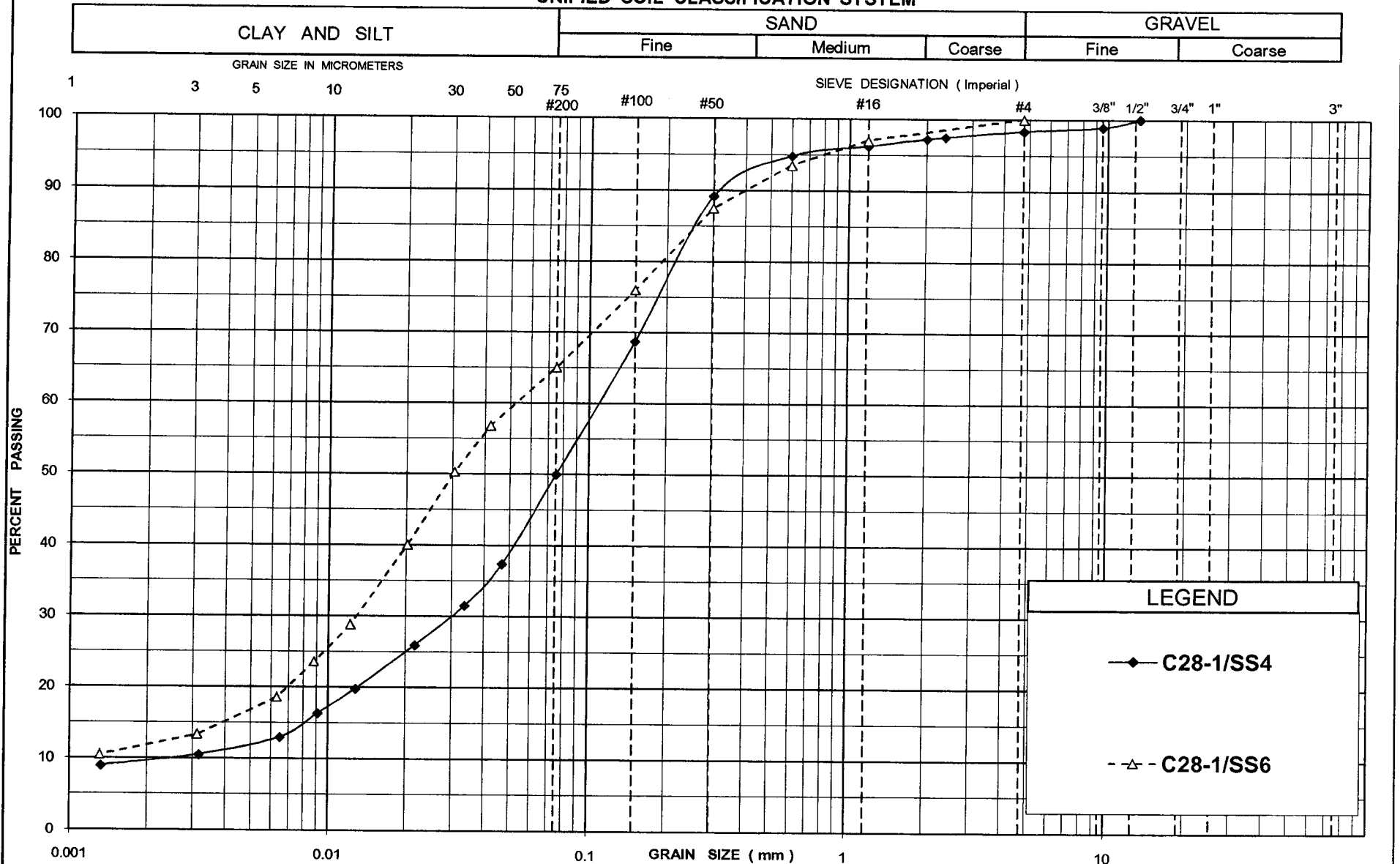
GRAIN SIZE DISTRIBUTION
GRAVELLY SAND, trace silt

FIGURE No. B-2

REF. No. SPT 1211F

DATE AUGUST 2008

UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

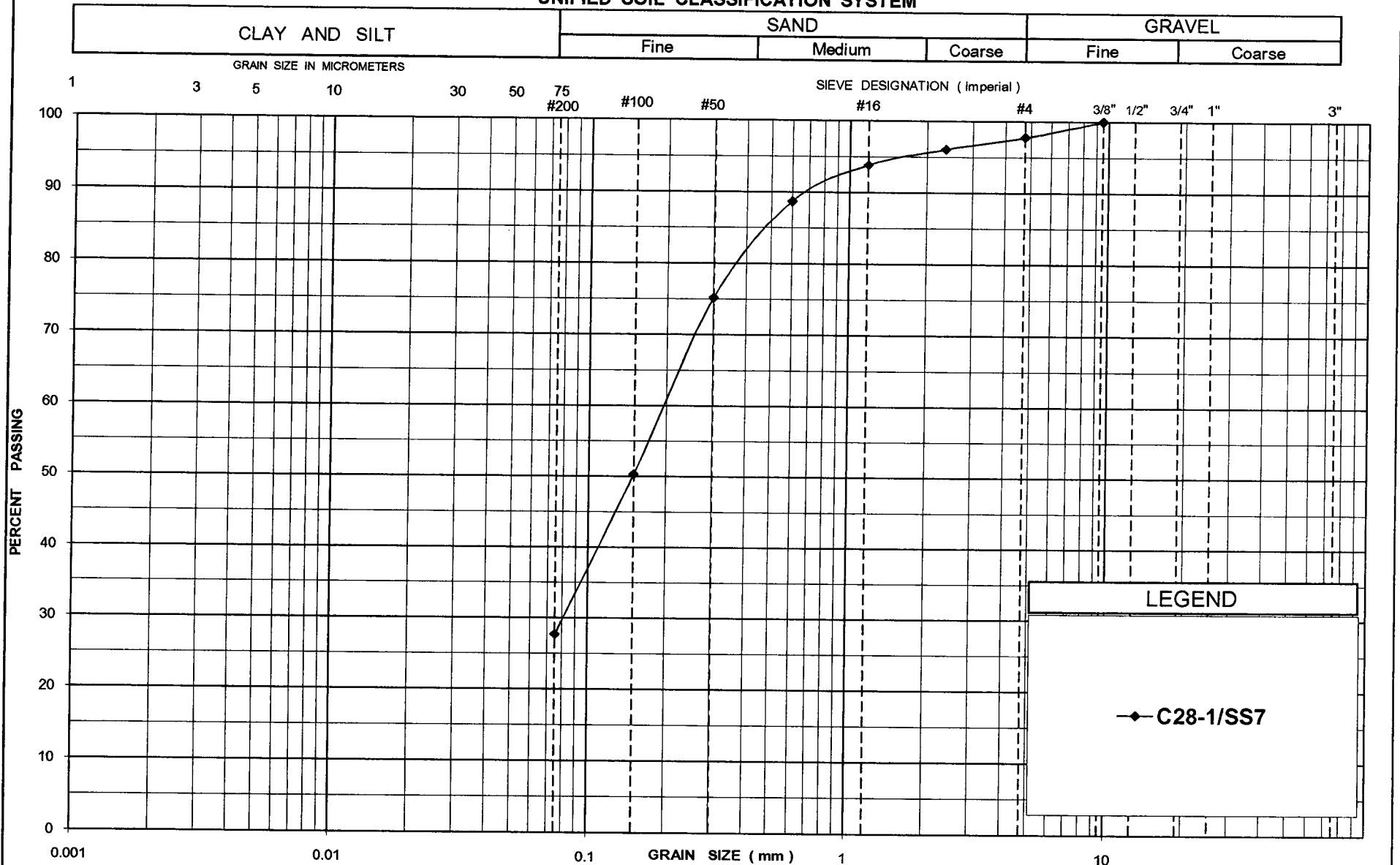
GRAIN SIZE DISTRIBUTION
LAYERED SILTY FINE SAND / SILT / CLAYEY SILT

FIGURE No. B-3

REF. No. SPT 1211F

DATE AUGUST 2008

UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION
LOWER SAND: SILTY FINE SAND

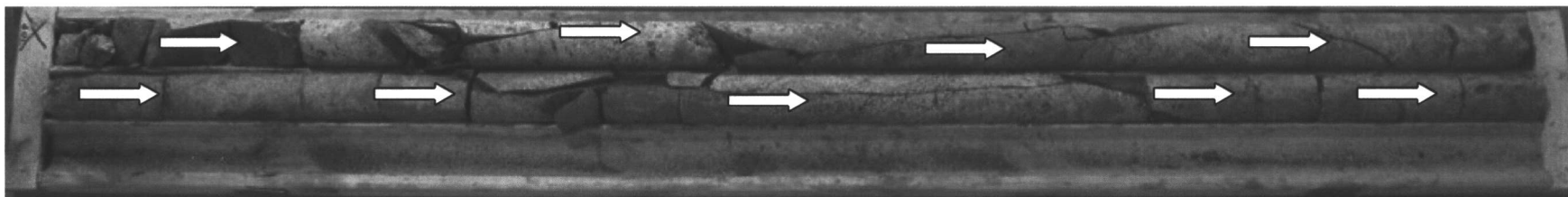
FIGURE No. B-4

REF. No. SPT 1211F

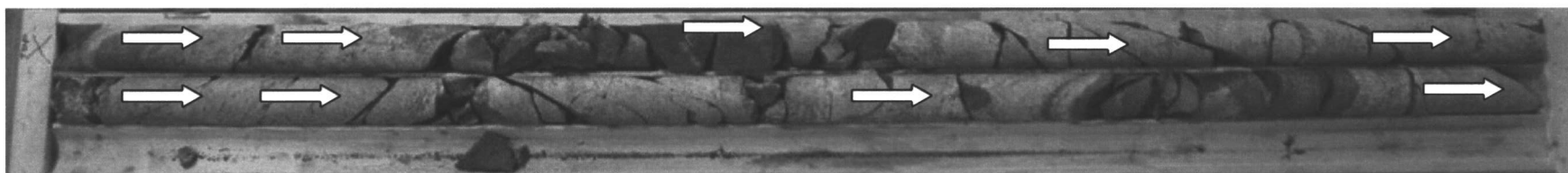
DATE AUGUST 2008

Appendix C

Rock Core Photographs



Borehole C28-2 (Station 26+509 Rt)



Borehole C28-3 (Station 26+510 Rt)

Appendix D

Site Photographs



Photograph1 Culvert C 28, Station 26+500, north side (looking east)



Photograph2 Culvert C 28, Station 26+500, north side



Photograph3 Culvert C 28, Station 26+500, south side (looking west)



Photograph4 Culvert C 28, Station 26+500, south side

Appendix E

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

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

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	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_c	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_c

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1.0	VOID RATIO	e_{min}	1.0	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1.0	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}}$
P_w	kg/m^3	DENSITY OF WATER	w	1.0	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_n	%	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 = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1.0	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
RESMER CREEK CULVERT (C28), STATION 26+500
HIGHWAY 17, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00
AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-130**

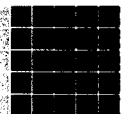
Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

**SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.**

**Project: SPT1211F
January 8, 2009**



shaheen peaker

**20 Meteor Drive
Toronto, Ontario
M9W 1A4
Tel: (416) 213-1255
Fax: (416) 213-1260
EMAIL: Info@shaheenpeaker.ca**

Table of Contents

5. DISCUSSION AND RECOMMENDATIONS	9
6. CLOSURE	13

APPENDIX F: OPSD

APPENDIX G: LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT
RESMER CREEK CULVERT (C28)
STATION 26+500, HIGHWAY 17
CAMERON TOWNSHIP, MATTAWA, ONTARIO
G.W.P. 173-98-00; AGREEMENT NO. 5006-E-0040**

5. DISCUSSION AND RECOMMENDATIONS

The improvements to Highway 17, as part of Agreement No. 5006-E-0040, will involve the rehabilitation of the existing culvert which drains Resmer Creek under the Highway at Station 26+500. The existing culvert is a 4610 mm diameter, 43.9 m long steel plate corrugated steel pipe (SPCSP). We understand that the invert of the pipe is at El. 190.8 m at the inlet (south side) and at 190.6 m at the outlet (north side) and the water level in the Creek on the inlet side was measured at El. 191.1 m in September 2007.

We understand that the bottom of the steel culvert is corroded and it is proposed to rehabilitate it by installing cast-in-place concrete invert paving with wire mesh reinforcing. The concrete paving is proposed to extend across the bottom and up the sides to about 1850 mm above the invert (i.e. slightly less than the culvert radius). The thickness of the cast-in-place concrete paving is expected to be minimum 150 mm, with an average thickness of about 200 mm.

Originally nine boreholes were drilled between Stations 26+295 and 26+575 on the south side of the highway from the south shoulder and along the toe of the road embankment (from the o.g. level) for a passing lane, (our Report No. SPT1211B). Subsequently, were requested to prepare a separate report, using the information obtained from that investigation. The closest boreholes to culvert C28 are Boreholes 26+475 (Borehole C28-1), 26+509 (Borehole C28-2) and 26+510 (Borehole C28-3) along with a dynamic cone test at Station 26+490 (Borehole C28-4), as shown on the Borehole Location Plan.

Boreholes C28-2 and C28-3 are located close to existing pipe and these boreholes show, below about El. 190.6 m (i.e. below the invert of the pipe and underlying granular bedding which should be present under the pipe), the presence of a compact (N-value=10) gravelly sand and very loose silty fine sand (N-value =2), respectively. These deposits extend to El. 189.8 m and 190.1 m, respectively and are underlain by a layered silt deposit, with silty sand, sandy silt, clayey silt and occasional thin clay interbeds. The deposit is 1.3 m and 2.2 m thick in Boreholes C28-2 and C28-3, extending to El. 188.5 and 187.9 m, respectively. Based on recorded N-values of typically 4 blows/0.3 m, it is considered to be very loose.

In Borehole C28-3, the stratified silt deposit is underlain by a basal sand layer with some gravel content. From a recorded N-value of 10 blows/0.3 m, the relative density of this 1.4 m thick deposit is described as compact.

Below these deposits, both boreholes contacted at El. 188.5 m (Borehole C28-2) and at El. 186.5 m (Borehole C28-3) a layer of boulders and shattered rock fragments extending to the surface of granitic bedrock at El. 187.5 m and 185.9 m, respectively.

Based on an average thickness of about 200 mm concrete paving to be placed on the bottom portion of the pipe, the estimated additional stresses due to the rehabilitation on the natural subgrade supporting the pipe and the underlying granular bedding, including a 0.1 m possible grade raise for the rehabilitation of the pavement, are expected to be of the order of 8 kPa.

Based on these assumptions and the data obtained from the boreholes, the foundation settlements due to the 8 kPa additional stresses is about 10 to 12 mm. This amount of settlement is considered acceptable from geotechnical engineering point of view.

It is our opinion that the 'invert paving' process will need to take place in sufficiently dry conditions which will necessitate the diversion of the water flow in the culvert for the duration of the construction and until the concrete has sufficiently set. As well, depending on the site conditions at the time of the construction, some dewatering of the subgrade soils will likely be required to facilitate the construction. For these reasons, we recommend that the construction be carried out during a dry season.

Construction of a temporary culvert would be a way to divert the water flow in the Creek from the existing culvert. This would however require either a complete traffic closure to implement the installation of the temporary culvert (probably unacceptable to MTO) or a staged construction by shoring (i.e. one half of the temporary culvert would be installed while the traffic is diverted to the other side of the road and then traffic is reversed and the remaining half of the temporary culvert is installed). Both methods are considered to be rather expensive.

A more practical and cost-effective alternative would be to divert the water flowing in the Creek away from the existing culvert and pump the water across the highway at a suitable location to the downstream side (north) of the highway. Such a scheme would however only be feasible when the water in the creek is manageably small.

It is normally up to the Contractor to choose a suitable diversion scheme.

In addition to diversion, as mentioned before, dewatering of the subgrade will likely be necessary. The method to be used to achieve sufficient dewatering to facilitate proper

access to the site will depend on the site conditions at the time of construction. For dewatering, which will require a draw-down of less than about 0.6 m, gravity drainage and pumping from strategically placed filtered sumps will suffice. For more effective draw-down, the use of deep wells and/or well points will likely be required. When devising such a system, the position of the bedrock and that of the bouldery layer immediately above it should be kept in mind. The choice of a proper dewatering method to achieve suitable working conditions would also be up to the Contractor. We recommend however that the Contractor be asked to submit their proposed diversion and dewatering methods to the contract CA for information purposes.

We also recommend that the Contractor be warned of the possible presence of cobbles and boulders in the embankment fill, as well as possible rock fill. Similarly, cobbles and boulders may occur in the overburden, especially immediately above the bedrock. It may furthermore be prudent to make the contractor aware that the stratified silt deposit which was encountered at El. 189.8 m and 190.1 m at Boreholes 28+509 and 28+510, respectively, is a dilatant material, which can easily be disturbed in the presence of water. Such disturbed and dilated soils can undergo settlements. For this reason, if the silt deposit is encountered during the construction extra care would be required. In particular vibrations which could disturb the soil should be kept to a practical minimum. The Contractor may choose to dig test pits to investigate the prevailing site conditions.

We recommend that the existing culvert be evaluated for the sufficiency of the existing erosion and scour measures and if observations show that they are deficient or if the concrete paving is expected to adversely affect erosion and scour potentials, further measures may be necessary. The following is a discussion of possible erosion measures.

Erosion and scour protection should be provided at the culvert inlet and outlet (including the slopes and sides). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. The following are some general suggestions, considering that below some probable organic and alluvial deposits at the creek level, the boreholes indicate that the native soils can be expected to consist of fine sands, gravelly sand possibly silts. The fine sands and particularly the silt are erodible soil types.

We recommend that concrete cut-off (apron) wall be constructed both at the inlet and the outlet to prevent seepage beneath and around the culvert, especially through the granular bedding the underlying granular soils (particularly the gravelly sand) as well as the granular backfill around the culvert, if such cut-off walls are not present beneath and around the existing culvert. Beneath the culvert, the concrete cut-off wall should extend to a suitable

depth (e.g. below any possible scour depth). Consideration may also be given to an impervious seal at the inlet and outlet.

At the inlet, consideration may also be given to the use of a clay seal. The purpose of the clay seal is to ensure that water flow is channeled through the culvert and does not seep through the backfill around the structure and from beneath the structure. The clay seal should therefore be continuous and is typically 0.6 m thick. It should comply with the material specifications given in OPSS 1205. It should be extended around the culvert from at least 0.5 m above the high water level in the creek down to the channel bed and up the other side in a continuous manner. It should be ensured that it extends to cover all the granular backfill materials to prevent any seepage through them. Typically, the clay seal is protected by laying a 0.6 m thick rock protection over it. The clay seal would generally be extended at about 8 m beyond the inlet.

At the outlet as well as at the inlet (if clay seal is not used), in addition to the concrete cut-off wall and/or impervious seal or in conjunction with these, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered. As the subgrade may consist of silty sand soils, a layer of granular or man-made filter material should be used. This would generally be extended about 8 m along the channel and the sides (to at least 0.3 m above the high water). The granular filter material (where necessary) underlying the rock protection can consist of a suitable granular material such as Granular 'A'. Alternatively, a suitable geotextile can be used underneath the rock fill, in lieu of the granular filter material.

Another reference for consideration is OPSD 810.010 Rip-Rap Treatment for Culvert Outlets.

Design frost protection for the general area is 2.0 m. A permanent soil cover of at least 2.0 m or its thermal equivalent is therefore required for frost protection. In case of riprap (rock fill), only one half of the rock fill thickness should be assumed to be effective in providing frost protection.

6. CLOSURE


We recommend that once the details of the project are finalized, our recommendations be reviewed for their specific applicability.


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

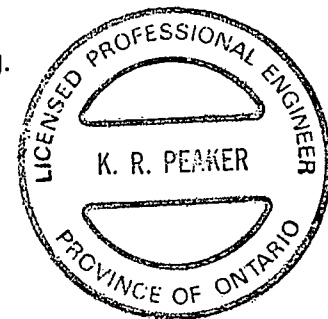
SHAHEEN & PEAKER

A Division of Coffey Geotechnics Inc.


Ramon Miranda, P.Eng.


Zuhtu Ozden, P.Eng.

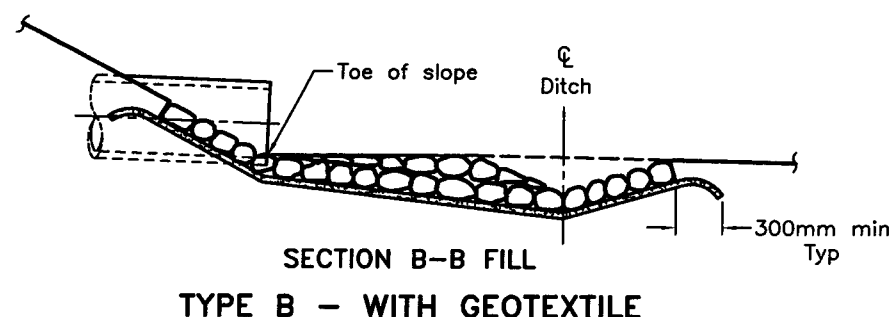
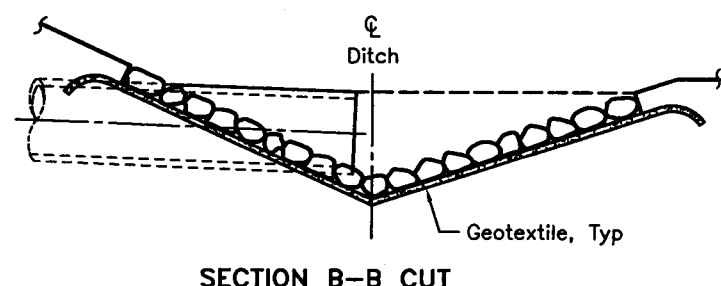
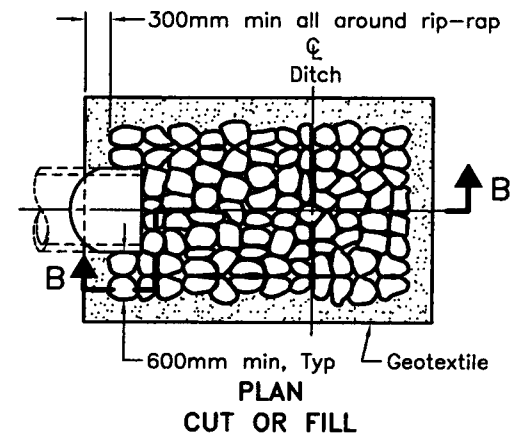
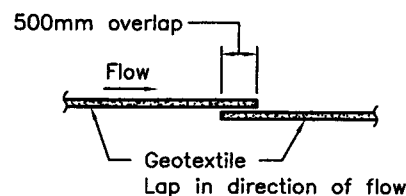
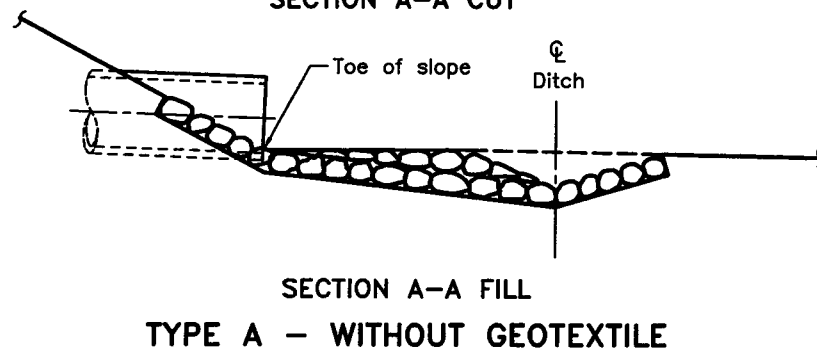
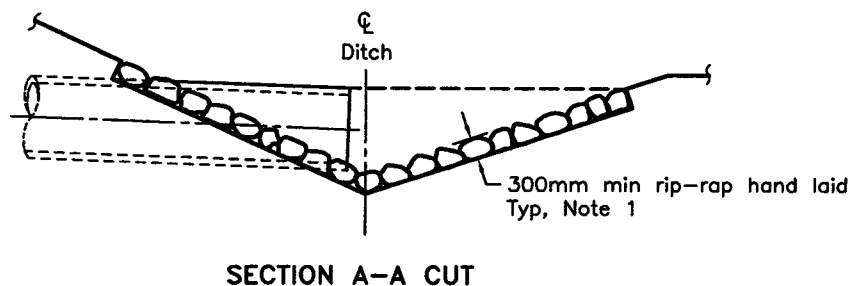
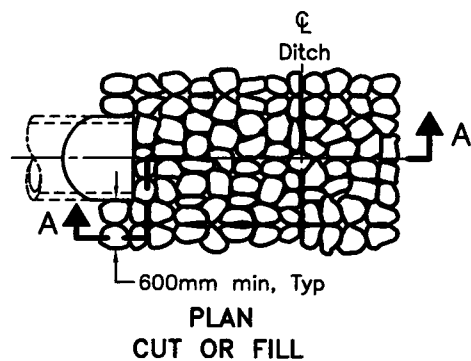

K. R. Peaker, Ph.D., P.Eng.



ZO:tr/ldrive

Appendix F

OPSD



NOTES:

1 The thickness of the rip-rap layer shall be at least 1.5 times the rip-rap mean diameter.

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

**RIP-RAP TREATMENT
FOR SEWER AND CULVERT OUTLETS**

Nov 2007 Rev 1



OPSD 810.010

Appendix G

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 Coffey Geotechnics Inc. (Coffey) at the time of preparation. Unless otherwise agreed in writing by Coffey, 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. Coffey accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

