FOUNDATION INVESTIGATION AND DESIGN REPORTS
CULVERT C8, STATION 17+448
HIGHWAY 17, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00
AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-129

Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.

Project: SPT1211C
November 24, 2008

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1. INTRODUCTION

As part of the rehabilitation of Highway 17, from 9.5 km east of Highway 533 (Mattawa) easterly for 14.9 km, it is proposed to rehabilitate several existing culverts.

Shaheen & Peaker, A Division of Coffey Geotechnics Inc.(S&P) was retained by D.M. Wills Associates Limited to carry out a foundation investigation at the site of a proposed rehabilitation of the existing culvert under Highway 17 at Station 17+448 in Cameron Township, near Mattawa, Ontario.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes, and to determine the engineering characteristics of the subsurface soils by means of field and laboratory tests.

The findings of the investigation are presented in this report.

2. SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located approximately 13 km east from the junction of Highway 533 with Highway 17 in Cameron Township as shown in Drawing No. 1.

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 the 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 consists of Mesoproterozoic Precambrian rocks (i.e. approximately 900 million years old), primarily felsic igneous tonalite, granodiorite, monzonite, granite, syenite and derived gneisses.

3. PROCEDURES

The fieldwork for this project was performed on May 26 and on June 4 to 5, 2008 and consisted of drilling and sampling three boreholes (C1-1, C1-2 and C1-3) to depths of 3.3 to 8.7m below existing grades at the existing culvert location, and five boreholes (C1-D1 to C1-D5) for the proposed road detour to depths of 0.9 to 6.3m below existing grades. The locations of the boreholes at the site are given on the Borehole Location Plan, Drawing No. 1.

The boreholes were advanced using a track-mounted drilling rig owned and operated by Landcore Drilling of Chelmsford, Ontario, under the full-time supervision of technical personnel from S&P. The boreholes were advanced using three different methods (i.e. continuous flight hollow-stem augers, wash boring in the overburden and rock coring) depending on the ground conditions.

Samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. The test 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 (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).

In some cases, auger refusal was encountered within the borehole due to the presence of cobbles or boulders in the overburden. This necessitated wash boring with N-type casing. The bedrock was cored at two locations by NQ rock coring method.

The borehole locations were established in the field by S&P engineering staff, in relation to the existing features. The locations were then tied in and the geodetic elevations of the ground at the borehole locations were determined by the client's surveyors. This survey information was provided to us.

Groundwater conditions in the boreholes were observed during and on completion of drilling in the open boreholes. Upon their completion, the boreholes were grouted using a cement/bentonite mixture as per MTO procedures. A standpipe piezometer was installed in Borehole C1-1 on completion.

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

4. SUMMARIZED SUBSURFACE CONDITIONS

The ground surface elevations range from 190.3 to 196.6 m for the boreholes. The existing top of highway embankment at the culvert location has an elevation 195.0m, while the ground surface elevation at Boreholes C1-1 and C1-3, advanced beyond the embankment (i.e. near the toe of the embankment) adjacent to the culvert are 190.3 and 191.9 m.

The culvert boreholes were advanced from 20.0 m south (Borehole C1-1) and 20.0 m north (Borehole C1-3) of the existing road centreline of Highway 17. Borehole C1-2 was advanced from the existing Highway 17 shoulder. Boreholes C1-1 and C1-3 show, below an about 0.1 m thick layer of topsoil, the presence of granular (i.e. non-cohesive) soils to depths/Elevations of 2.2 m/188.1 m and 3.3 m/ 188.6 m, respectively, while in Borehole C1-3, a granular embankment fill was contacted to 4.3 m or to El. 190.7 m, underlain by gravelly sand with some silt to 5.7 m (El. 189.3 m). Below these elevations, Boreholes C1-1 and C1-2 encountered gneissic bedrock at depths/Elevations 2.2 m /El. 188.1 m and 5.7 m /El. 189.3 m. In Borehole C1-3 refusal to augering was contacted at a depth/Elevation of 3.3 m/188.6 m probably on or near the surface of the bedrock.

The boreholes advanced for the possible detour embankment (C1-D1 to C1-D5) were extended 6.5 to 11 m north of the existing road centerline. Boreholes C1-D2 to C1-D5 encountered refusal to further augering at depths of 0.9 to 2.3 m or elevations 192.1 to 192.9 m, probably on the surface of the bedrock or close to it, while Borehole C1-D1 encountered auger refusal at a depth of 6.3 m or elevation 190.3 m, also probably on the surface of bedrock or close to it. The overburden contacted in the boreholes consisted of basically granular soils which range in composition from silty sand with some gravel, sand to gravelly sand, except in Borehole C1-D1, where a 1.6 m thick basically cohesive silt with thin clay and clayey silt seams was encountered at 3.7 m depth.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. An inferred stratigraphic section is shown in Drawings No. 1 and 2. The following description of the individual soil strata is to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site. It should be noted that the surface conditions may vary in between and beyond borehole locations.

4.1 CULVERT SITE

Boreholes C1-1, C1-2 and C1-3 were put down at the site of Culvert C8.

4.1.1 TOPSOIL

Boreholes C1-1 and C1-3, which were advanced from the near the toe of the highway embankment, at the original ground (o.g.) level, contacted a 0.05 m and 0.08 m thick topsoil layer, respectively.

It should, however, be pointed out that, in our experience, the thickness of topsoil and other organic rich soils frequently varies in between and beyond borehole locations. In particular, thicker organic soils frequently occur in depressed areas and within watercourse valleys.

4.1.2 EMBANKMENT FILL

Fill materials were contacted in Borehole C1-2 which was drilled from the top of the road embankment. The recorded thickness of the fill in this borehole was approximately 4.3 m.

The upper 0.2 m of the fill consisted of granular pavement fill (i.e. gravelly sand). Below this and extending to a depth of 1.4 m, the fill was found to consist of sand with some gravel and silt content. The grain-size distribution of a sample from this fill material is given in Figure B-1 in Appendix B. The following grain-size distribution is indicated.

Gravel:	13%
Sand:	69%
Silt & Clay	18%

Standard Penetration tests performed in this granular (i.e. non-cohesive) fill yielded N-values of 13 and 17 blows/0.3 m, which indicate a compact condition.

Below 1.4 m and extending to about 4.3 m (El. 190.7 m), the fill was found to consist of a coarser material ranging from gravelly sand to sandy gravel with frequent cobbles and boulders. The grain-size distribution of a sample from this relatively coarse granular soil is given in Figure B-2 in Appendix B. The curve indicates:

Gravel:	39%
Sand:	50%
Silt & Clay	11%

size particles. N-values recorded in the fill ranged from 19 to 88 blows/0.3 m. These results indicate a compact to very dense relative density.

4.1.3 GRANULAR OVERBURDEN

Underlying the topsoil in Boreholes C1-1 and C1-3 and the embankment fill in Borehole C1-2, the natural overburden soils were found to extend to depths ranging from about 1.4 m in Borehole C1-2 to 3.3 m in Borehole C1-3, below the o.g. levels.

The composition of the natural overburden soils was found to range from silty sand with some gravel to gravelly sand.

The grain-size distribution of a sample from the deposit was determined in the laboratory and the results are given in Figure B-3 in Appendix B. The following grain-size distribution is indicated.

Gravel:	32%
Sand:	59%
Silt & Clay:	9%

The natural overburden soils are considered to have a relatively high permeability.

The recorded N-values in these granular soils range from 4 to in excess of 50 blows/0.3 m indicating a very loose to very dense condition. Two lowest N-values of 4 and 12 blows/0.3 m were however recorded immediately at the o.g. levels. Below this, the recorded N-values are generally 29 to 42 blows/0.2 m.

4.1.4 BEDROCK

Bedrock was cored at two borehole locations (C1-1 and C1-2), as follows:

Borehole No.	Ground Elevation (m)	Overburden Depth to the Surface of Bedrock (m)	Elevation of the Surface of Bedrock (m)
C1-1	190.3	2.2	188.1
C1-2	195.0	5.7	189.3

Borehole C1-3 encountered auger refusal on possible bedrock at a depth of 3.3m, or elevation 188.6m.

From the cores, the bedrock was identified as a gneiss. Its colour is generally pinkish grey. The formation belongs to the Pre-Cambrian Era.

The Total Core Recovery (TCR) in the bedrock was 100% for all coring runs, with Rock Quality Designation (R.Q.D.) values generally between 69 and 100% except for the upper 1.3 m of the rock core obtained from Borehole C1-2 where the R.Q.D. was measured as 39%. These results indicate a relatively sound rock with a fair to excellent rock quality except for the upper 1.3 m of the bedrock in Borehole C1-2 where poor rock quality is inferred.

From the table presented above the surface of the bedrock appears to be fairly flat, with a slight rise below the road embankment at the culvert location.

4.1.5 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of each borehole. One standpipe piezometer was installed at Borehole C1-1. The observations are shown on the individual Record of Borehole sheets.

Due to the fact that wash boring and coring was used in Boreholes C1-1 and C1-2, these observations may not represent stabilized conditions.

Based on the moisture contents of the soil samples, it is our opinion that at the time of our investigation, the groundwater level at the site was generally at about elevation 189.8 to 191.0 m.

It should, however, be pointed out that the groundwater at the site would be subject to seasonal fluctuations as well as fluctuations due to weather events and the water level in the water course.

4.2 DETOUR EMBANKMENT

As shown on Borehole Location Plan, Drawing No. 1, Boreholes C1-D1, C1-D2, C1-D3, C1-D4 and C1-D5 were put down on the north side of the existing highway embankment, for the construction of a possible temporary detour embankment which may be utilized during the rehabilitation of the existing culvert pipe. These boreholes, together with Borehole C1-3 which was advanced on the north side of the existing roadway embankment at the culvert site, encountered practical refusal to further augering at depths of 0.9 m (Borehole C1-D4) to 6.3 m (Borehole C1-D1) or between Elevations 192.9 m (Borehole C1-D2) to 188.6 m (Borehole C1-3).

4.2.1. TOPSOIL

All the boreholes except for Borehole C1-D5, contacted a 0.08 to 0.13 m thick veneer of topsoil.

It should however be pointed out that, in our experience, the thickness of organic rich soils frequently varies in between and beyond borehole locations.

4.2.2. FILL

Borehole C1-D5 was drilled on the shoulder of the highway at Station 17+543 and as such granular pavement fill was contacted to a depth of 1.0 m or to El. 192.1 m. At this elevation, practical refusal to further augering was contacted, which is believed to represent the surface of the bedrock.

4.2.3. GRANULAR DEPOSITS

Underlying the topsoil, Borehole C1-D1 through C1-D4 and C1-3 contacted granular deposits. The composition of these deposits range from gravelly sand & silt, sand to gravelly sand.

Grain-size distribution curves for four selected samples from these granular overburden soils are given in Figures B-3, B-4 and B-5. These indicate the following grain-size distribution range:

Gravel:	0 - 32%
Sand:	59 - 96%
Silt & Clay:	3 - 9%

N-values recorded in these granular overburden soils range from 5 to 44 blows/0.3 m indicating a loose to dense but typically compact relative density.

4.2.4 SILT WITH CLAY & CLAYEY SILT SEAMS

In Borehole C1-D1 a silt deposit with clay and clayey silt interbeds was contacted at a depth of 3.7 m or El. 192.9 m. The thickness of this unit was found to be 1.6 m and the deposit extended to 5.3 m or to El. 191.3 m.

The material consists of cohesive to non-cohesive silt with thin cohesive clay and clayey silt seams. The overall behavior of the soil is believed to be basically cohesive.

The grain-size distribution of a sample from the deposit is given in Figure B-6. When analyzing the results, it should be kept in mind that the relatively high percentage of clay size particles (i.e. 28%) is due to the clay seams in the deposit. This deposit is considered to be considerably less pervious than the other overburden soils encountered at the site.

Standard Penetration tests performed in the deposit yielded N-values of 8 and 10 blows/0.3 m, which indicate a stiff consistency.

4.2.5 INFERRED BEDROCK

As mentioned before, the boreholes contacted practical refusal to augering at depths ranging from 0.9 m (C1-D4) to 6.3 m (C1-D1) below the ground surface or at between El. 192.9 (C1-D2) and 188.6 m (C1-3 near the existing culvert and the water course). These refusal elevations are believed to represent the surface of the bedrock or elevations close to it.

4.2.6 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were recorded during the drilling and at the completion of each borehole, as shown on the appropriate Record of Borehole sheets.

Based on the observations made, it is our opinion that the groundwater table at the time of our investigation at the site was at about El. 193 m on the west side (at Boreholes C1-D1 and C1-D2), dropping to about El. 191.3 m at Boreholes C1-D3, C1-3 and C1-D4 and rising to El. 192 m at Borehole C1-D5 to the east.

It should however be pointed out that the groundwater table would be subject to seasonal variations and fluctuations in response to major weather events.

SHAHEEN & PEAKER

A Division of Coffey Geotechnics Inc.

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Drawings

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

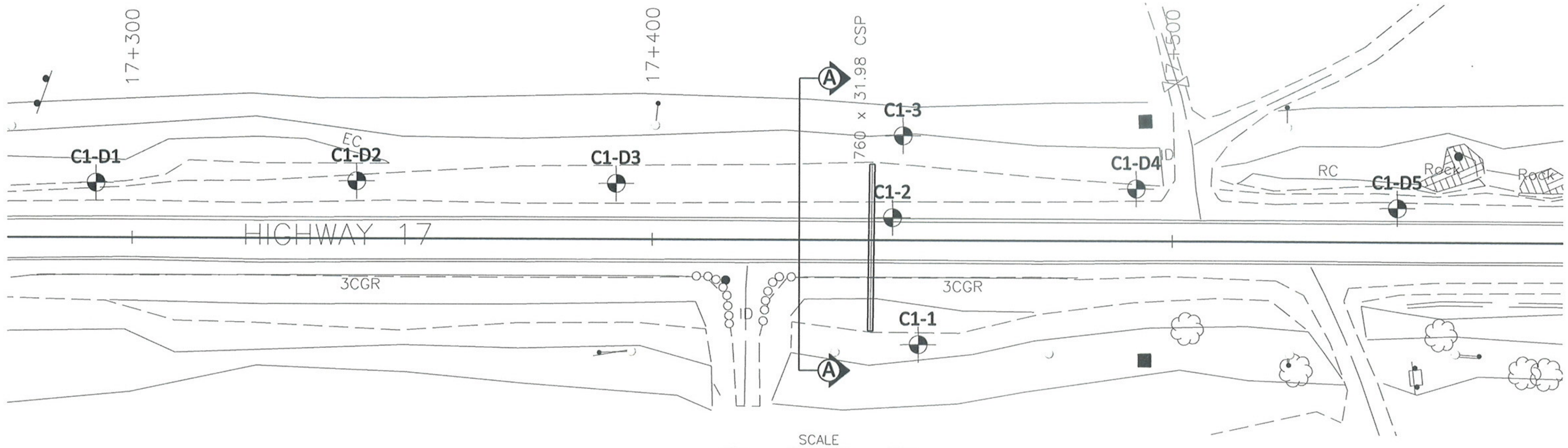
METRIC

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

CONT No. 5006-E-0040
WP: 173-98-00

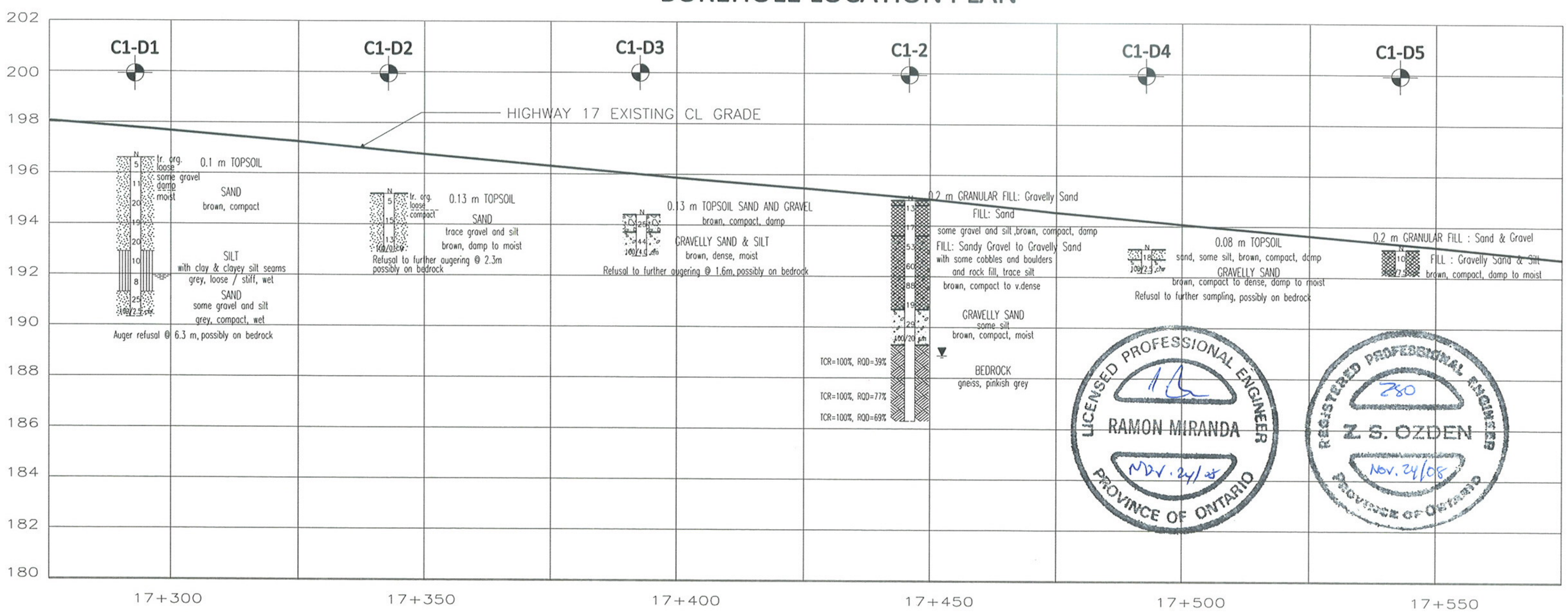
Highway 17 Mattawa
BORHOLE LOCATION PLAN & STRATIGRAPHY (Culvert 8 @ 17+448)

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.



BOREHOLE LOCATION PLAN

KEY PLAN
N.T.S



Profile (along Highway 17 centerline)

LEGEND

- Borehole
- N 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
C1-1	190.3	17+451	20.0m Rt
C1-2	195.0	17+446	4.3m Lt
C1-3	191.9	17+448	20.0m Lt
C1-D1	196.6	17+293	10.5m Lt
C1-D2	195.2	17+343	11.0m Lt
C1-D3	194.4	17+393	10.5m Lt
C1-D4	193.1	17+493	10.0m Lt
C1-D5	193.1	17+543	6.5m Lt

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-129

SPT 1211C			DIST
SUBM'D	CHECKED	DATE Sept.2008	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1

METRIC

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

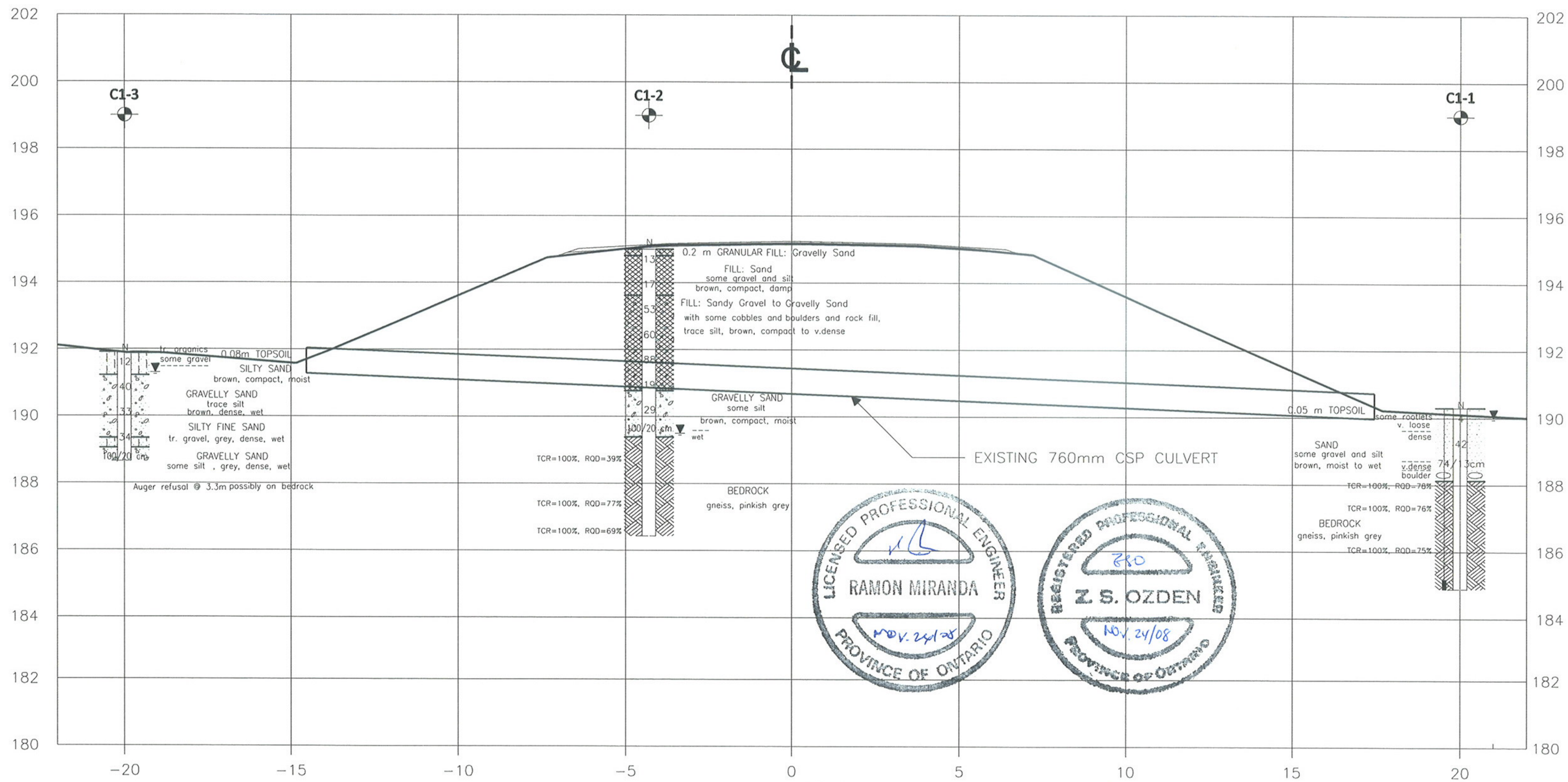
NOTES:
FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No. 5006-E-0040
WP: 173-98-00

Highway 17 Mattawa
STRATIGRAPHY (Culvert 8 @ 17+448)

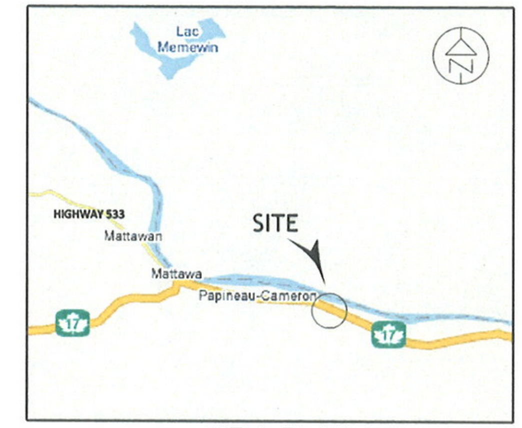
SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.

HIGHWAY 17



17+443

Cross Section (A-A)



KEY PLAN
N.T.S

LEGEND

- Borehole
- N 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
C1-1	190.3	17+451	20.0m Rt
C1-2	195.0	17+446	4.3m Lt
C1-3	191.9	17+448	20.0m Lt

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-129

SPT 1211C			DIST
SUBM'D	CHECKED	DATE Sept.2008	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 2

Appendix A

Records of Borehole Sheets

SPT1211C : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C1-1

METRIC

GWP 173-98-00 LOCATION Sta. 17+451 ;20.0m Rt of C/L of Hwy 17 ORIGINATED BY SK

DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NW Casing COMPILED BY SS

DATUM Geodetic DATE 6/4/2008 6/5/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
190.3	GROUND SURFACE								
0.0	0.05 m TOPSOIL								
	SAND, some gravel and silt, brown, moist to wet; some rootlets, v. loose		1	SS	4		190		
	dense		2	SS	42		189		
	v.dense		3	SS	74/13cm				Spoon bouncing on boulder
188.1 / 2.2	boulder		4	RC			188		
	BEDROCK, gneiss, pinkish grey		5	RC	TCR=100% RQD=78%				
			6	RC	TCR=100% RQD=76%		187		
			7	RC	TCR=100% RQD=75%		186		
184.8 / 5.5			8	RC	TCR=100% RQD=100%		185		

End of Borehole
Water level in open borehole @ 0.3 m (not stabilized)* upon completion.
Piezometer installed to 5.5 m
Water level in piezometer @ 0.4 m (El. 189.9 m) on June 06/2008

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE; ● POCKET PENETR. X LAB VANE: 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³
GR SA SI CL

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

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

SPT1211C : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C1-2

METRIC

GWP 173-98-00	LOCATION Sta. 17+446 :4.3m Lt of C/L of Hwy 17	ORIGINATED BY SK
DIST HWY 17	BOREHOLE TYPE Hollow Stem Auger & NW Casing	COMPILED BY SS
DATUM Geodetic	DATE 6/5/2008	CHECKED BY ZO

ELEV. DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
195.0	GROUND SURFACE					
194.8 / 0.2	0.2 m **GRANULAR FILL:** Gravelly Sand					
	FILL: Sand some gravel and silt brown, compact, damp	1	SS	13		
193.6 / 1.4		2	SS	17		13 69 (18)
	FILL: Sandy Gravel to Gravelly Sand with some cobbles and boulders and rock fill, trace silt brown, compact to v.dense	3	SS	53		39 50 (11)
	damp	4	SS	60		Auger grinding on cobbles and boulders between 2.2 and 5.7 m
	moist	5	SS	88		Spoon bouncing on a cobble
		6	RC			
190.7 / 4.3		7	SS	19		
	GRAVELLY SAND some silt brown, compact, moist	8	SS	29		
189.3 / 5.7	wet	9	SS	100/20 cm	▼*	Spoon bouncing on bedrock
	BEDROCK gneiss, pinkish grey	10	RC	TCR=100% RQD=39%		
		11	RC	TCR=100% RQD=77%		
186.3 / 8.7		12	RC	TCR=100% RQD=69%		
	End of Borehole. Water level in open borehole @ 5.5 m (not stabilized)* upon completion.					

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; ELEVATION SCALE 195 194 193 192 191 190 189 188 187

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

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

RECORD OF BOREHOLE No C1-3

METRIC

GWP 173-98-00 LOCATION Sta. 17+448 :20.0m Lt of C/L of Hwy 17 ORIGINATED BY SK

DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS

DATUM Geodetic DATE 6/5/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
191.9	GROUND SURFACE							
0.0	0.08m **TOPSOIL** **SILTY SAND** tr. organics, some gravel, brown, compact, moist	1	SS	12	Water level 0.6 m*			
191.2 / 0.7	**GRAVELLY SAND** trace silt, brown, dense, wet	2	SS	40		191		
		3	SS	33		190		32 59 (9)
189.3 / 2.6	**SILTY FINE SAND** tr. gravel, grey, dense, wet	4	SS	34				
189.0 / 2.9	**GRAVELLY SAND** some silt, grey, dense, wet					189		
188.6 / 3.3		5	SS	100/20cm				Spoon bouncing

End of Borehole.
Auger refusal @ 3.3 m,
possibly on bedrock
Water level in open borehole @ 0.6 m
upon completion (not stabilized)*

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

+ 3 , × 3 : Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

Foundation Design

SPT1211C : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C1-D1

METRIC

GWP 173-98-00	LOCATION Sta. 17+293 :10.5m Lt of C/L of Hwy 17		ORIGINATED BY SK
DIST	HWY 17	BOREHOLE TYPE Hollow Stem Auger	COMPILED BY SS
DATUM Geodetic	DATE 5/26/2008		CHECKED BY ZO

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
196.6	GROUND SURFACE										
0.0	0.1 m TOPSOIL; SAND brown, compact; tr. org. loose	1	SS	5		196					
	some gravel damp	2	SS	11				10	87	(3)	
	moist	3	SS	20		195					
		4	SS	19		194					
		5	SS	20				0	96	(4)	
192.9 / 3.7	SILT with clay & clayey silt seams grey, loose / stiff, wet	6	SS	10		193					
		7	SS	8	▽	192		2	5	65	28
191.3 / 5.3	SAND some gravel and silt grey, compact, wet	8	SS	25		191					
190.3		9	SS	100/2.5 cm							
6.3	End of Borehole. Auger refusal @ 6.3 m, possibly on bedrock. Borehole caved-in @ 4.5 m upon completion										

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE; ● POCKET PENETR. × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+³, ×³: Numbers refer to Sensitivity

15-⊕-5 (20, 10) (%) STRAIN AT FAILURE

SPT1211C : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C1-D2

METRIC

GWP	173-98-00	LOCATION	Sta. 17+343 :11.0m Lt of C/L of Hwy 17	ORIGINATED BY	SK
DIST	HWY 17	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	SS
DATUM	Geodetic	DATE	5/26/2008	CHECKED BY	ZO

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
195.2	GROUND SURFACE										
0.0	0.13 m **TOPSOIL**						195				
	SAND trace gravel and silt, brown, damp to moist; tr org, loose		1	SS	5				o (~ 5)		
	compact		2	SS	15		194		o (~ 9)		2 92 (5)
			3	SS	13				o (~ 10)		
192.9			4	SS	100/0 CM		193				No Recovery
2.3	End of Borehole. Refusal to further augering @ 2.3 m, possibly on bedrock. Borehole dry on completion (not stabilized)										

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

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

SPT1211C : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C1-D3

METRIC

GWP 173-98-00 LOCATION Sta. 17+393 :10.5m Lt of C/L of Hwy 17 ORIGINATED BY SK

DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS

DATUM Geodetic DATE 5/26/2008 CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
194.4	GROUND SURFACE										
0.0	0.13 m TOPSOIL **SAND AND GRAVEL** brown, compact, damp		1	SS	25		194				
193.7 0.7	**GRAVELLY SAND & SILT** brown, dense, moist		2	SS	44						
192.8			3	SS	100/4.0 cm		193				Spoon bouncing
1.6	End of Borehole. Refusal to further augering @ 1.6 m, possibly on bedrock Borehole dry on completion (not stabilized)										

+ ³, × ³: Numbers refer to Sensitivity

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

SPT1211C : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C1-D4

METRIC

GWP 173-98-00 LOCATION Sta. 17+493 :10.0m Lt of C/L of Hwy 17 ORIGINATED BY SK

DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS

DATUM Geodetic DATE 5/26/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
193.1	GROUND SURFACE										
0.0	0.08 m **TOPSOIL**						193				
192.7 0.4	**SAND** some silt, brown, compact, damp		1	SS	18						
192.2 0.9	**GRAVELLY SAND** brown, compact to dense, damp to moist		2	SS	100/2.5 cm						
	End of Borehole. Refusal to further sampling, possibly on bedrock. Borehole dry on completion (not stabilized)										

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

+³, ×³: Numbers refer to Sensitivity

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

SPT1211C : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C1-D5

METRIC

GWP 173-98-00 LOCATION Sta. 17+543 :6.5m Lt of C/L of Hwy 17 ORIGINATED BY SK

DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS

DATUM Geodetic DATE 5/26/2008 CHECKED BY ZO

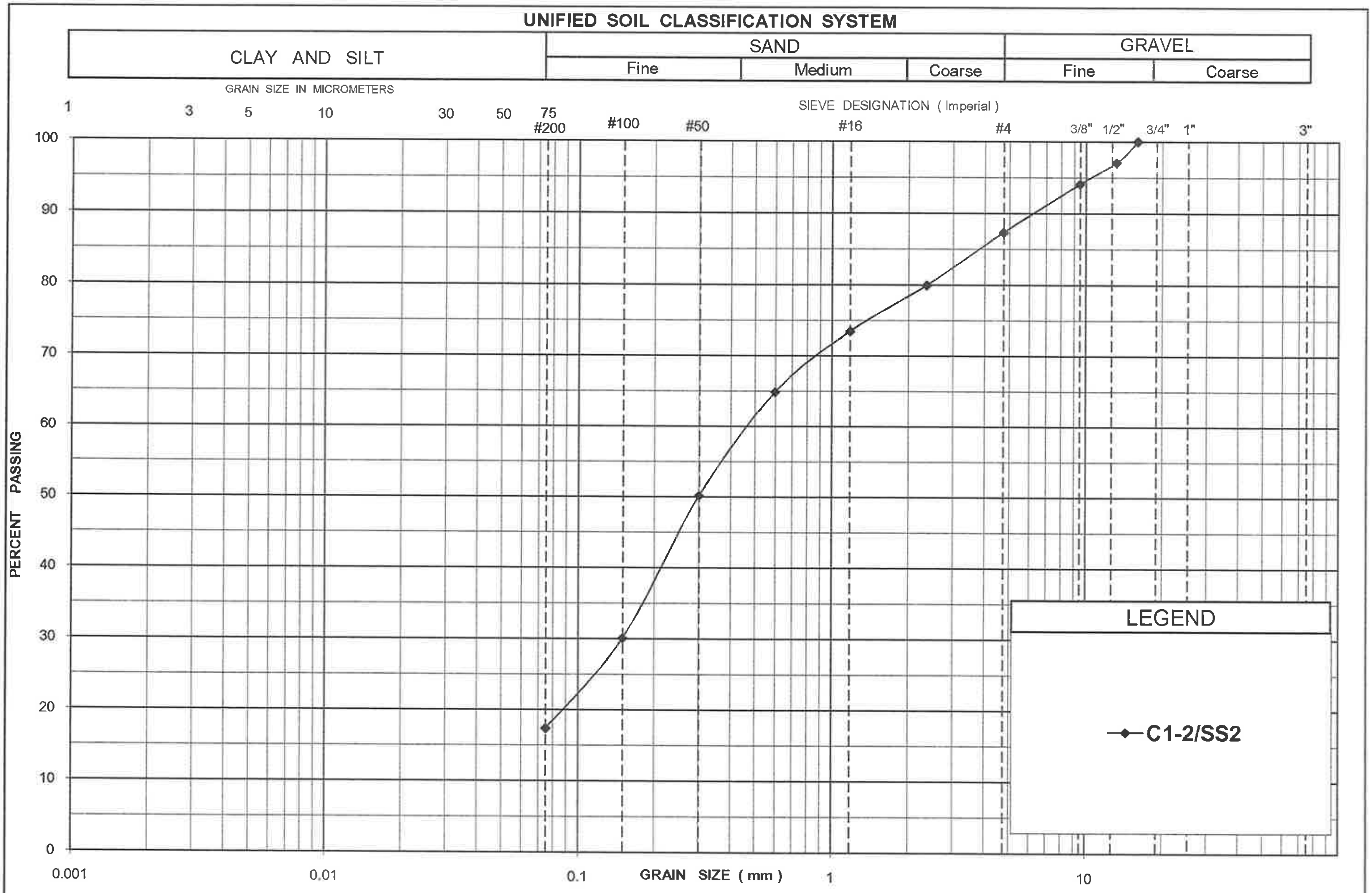
ELEV. / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED + FIELD VANE; ● POCKET PENETR. X LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
193.1	GROUND SURFACE										
0.0	0.2 m GRANULAR FILL : Sand & Gravel						193				
	FILL : Gravelly Sand & Silt brown, compact, damp to moist		1	SS	10				o		
192.1			2	SS	100/7.5 cm				o		
1.0	End of Borehole. Borehole dry on completion (not stabilized)										

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

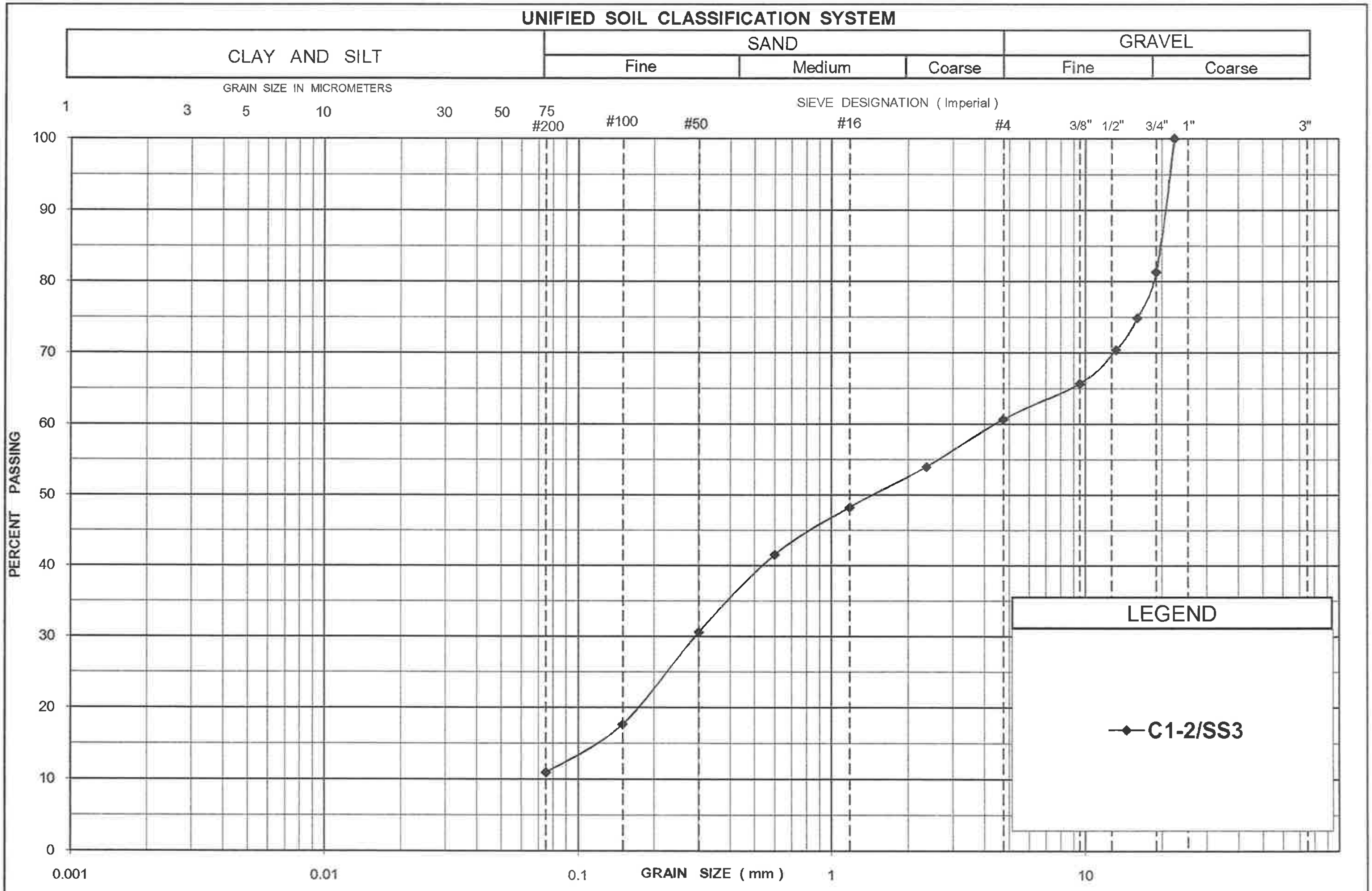
(%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	**GRAIN SIZE DISTRIBUTION** EMBANKMENT FILL: SAND, some gravel and silt	FIGURE No. B-1 REF. No. SPT 1211C DATE SEPTEMBER 2008

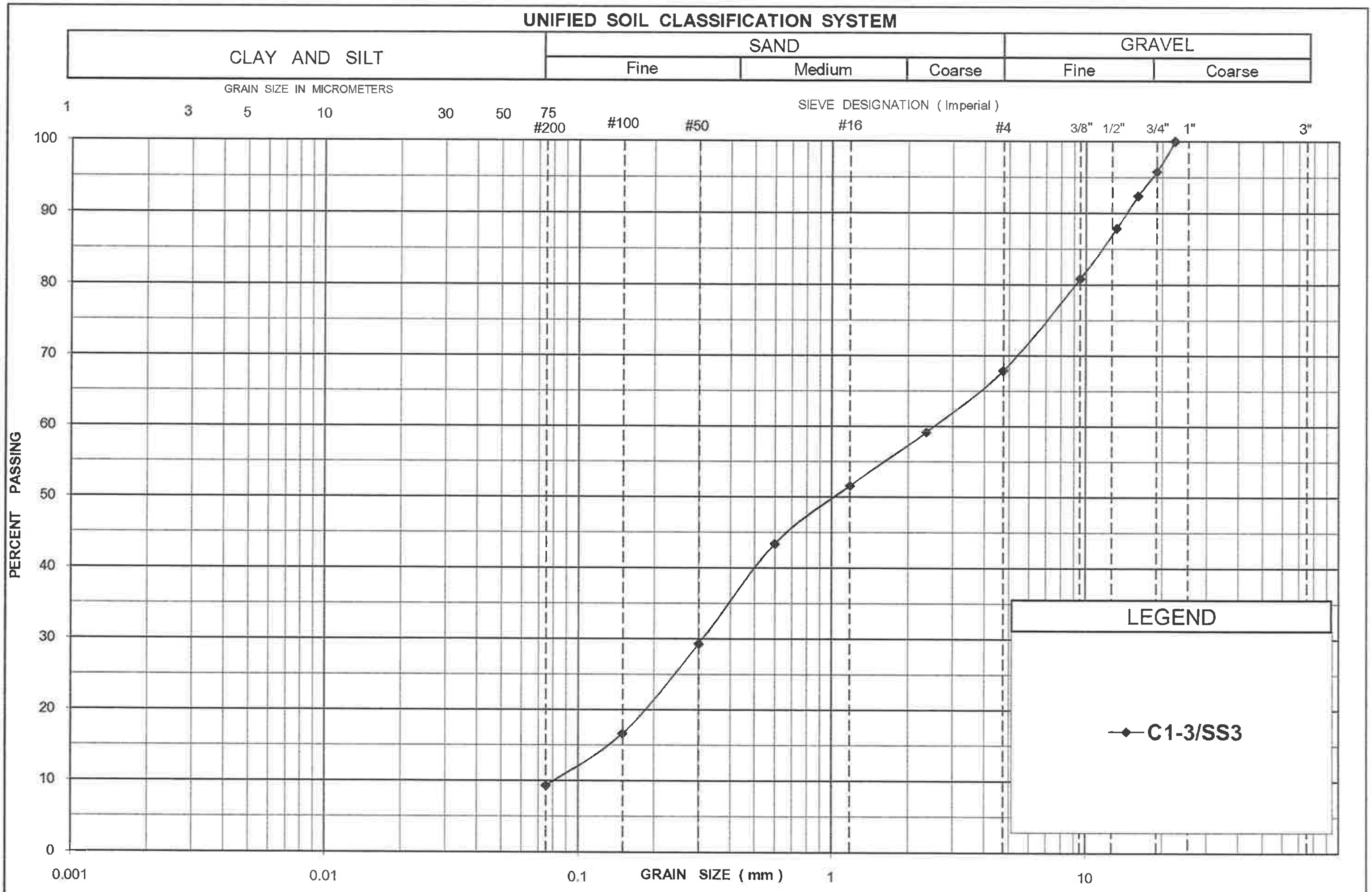


SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

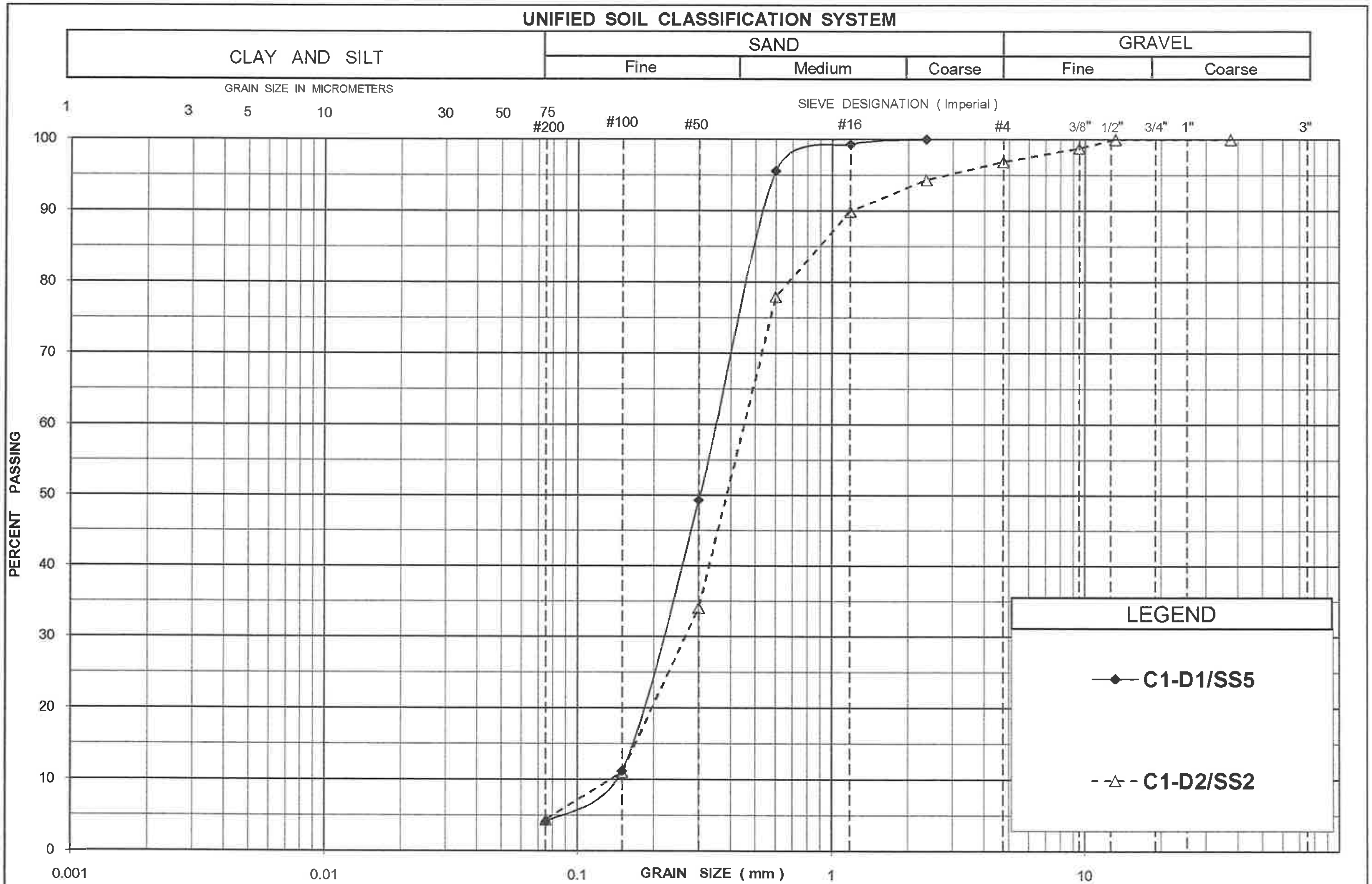
GRAIN SIZE DISTRIBUTION

EMBANKMENT FILL:GRAVELLY SAND, trace silt

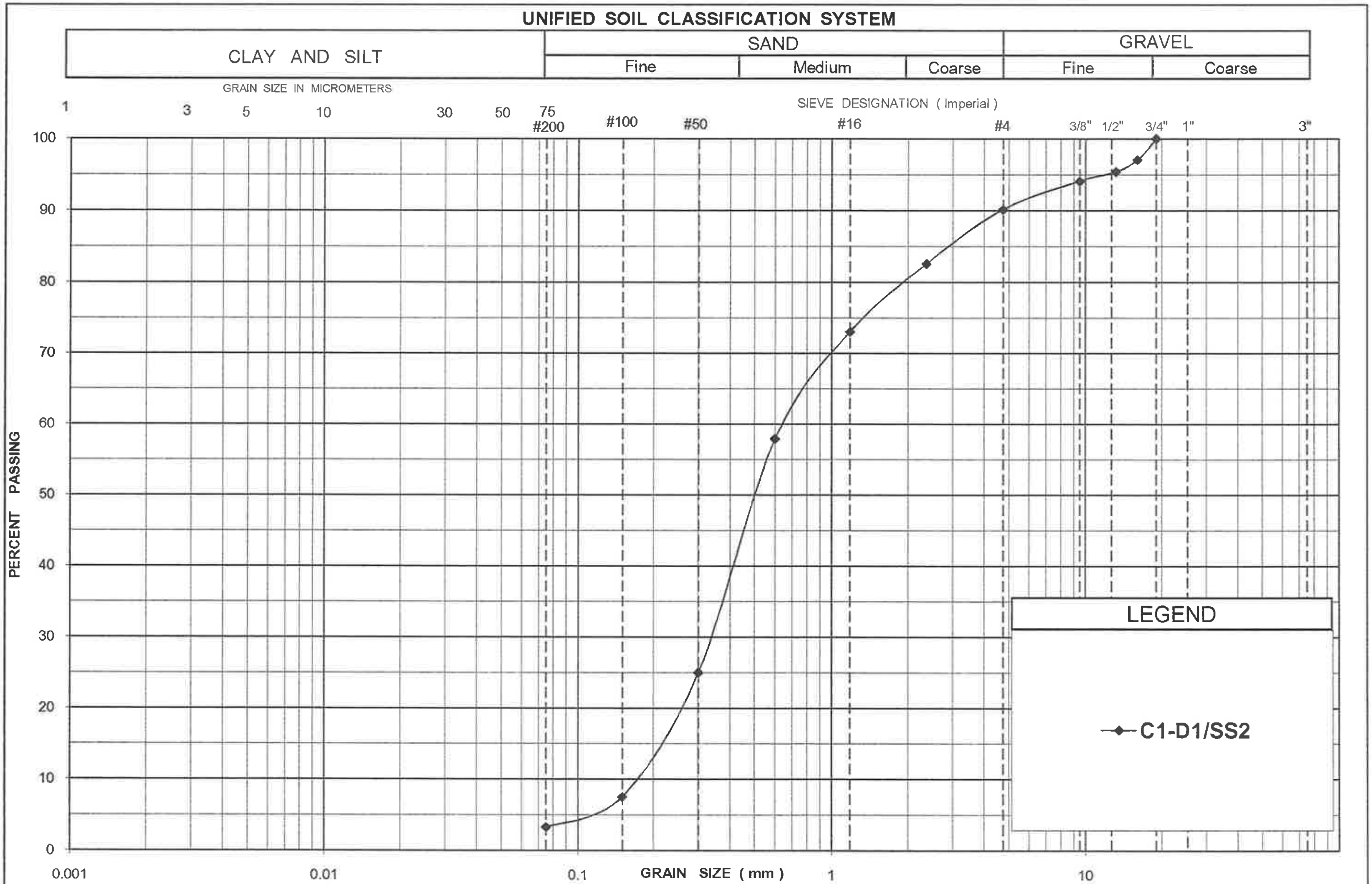
FIGURE No. B-2	
REF. No.	SPT 1211C
DATE	SEPTEMBER 2008



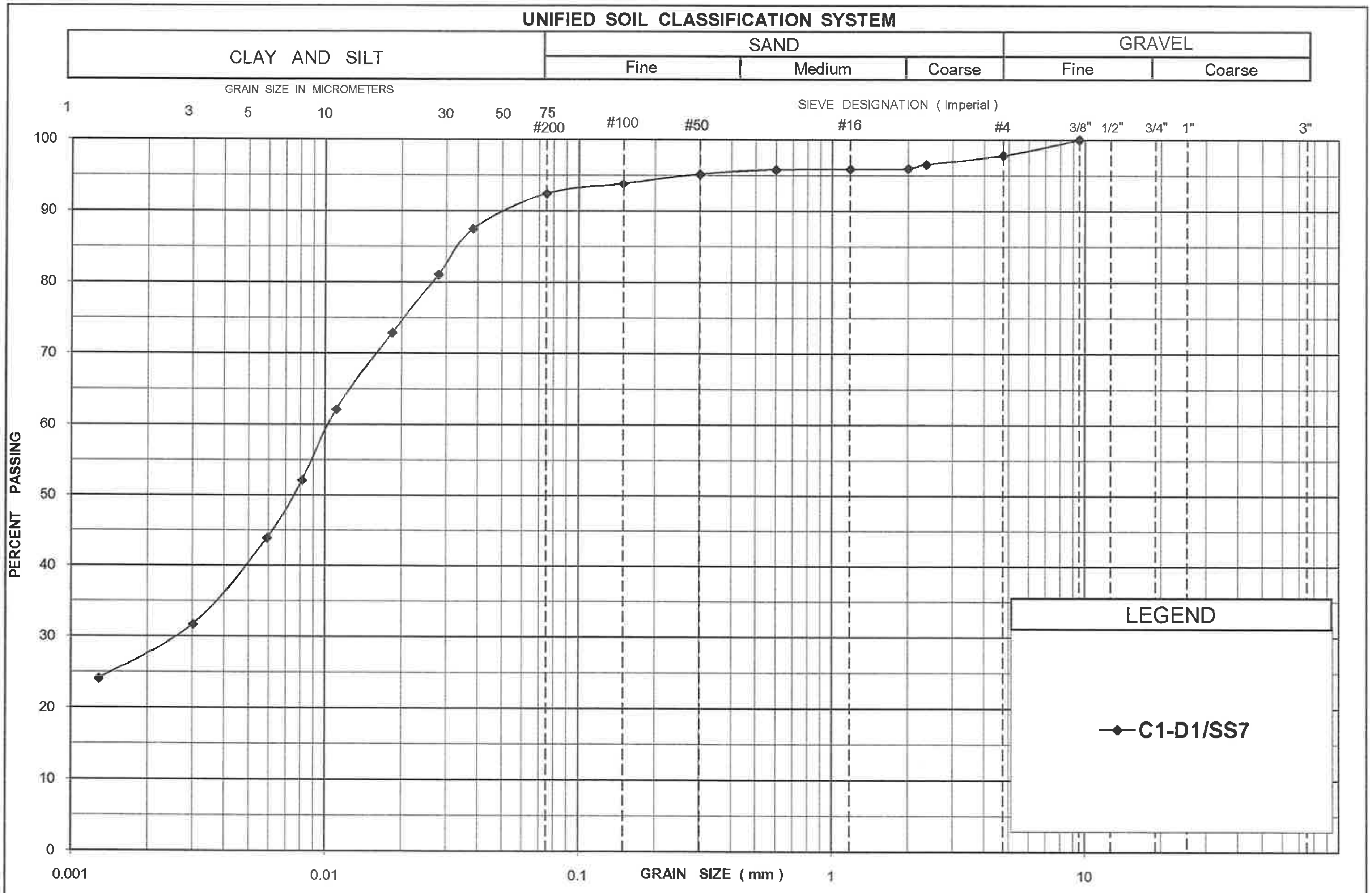
SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION GRAVELLY SAND, trace silt	FIGURE No. B-3 REF. No. SPT 1211C DATE SEPTEMBER 2008



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION SAND	FIGURE No. B-4 REF. No. SPT 1211C DATE OCTOBER 2008



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION SAND, some gravel	FIGURE No. B-5 REF. No. SPT 1211C DATE OCTOBER 2008



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	**GRAIN SIZE DISTRIBUTION** SILT with clay & clayey silt seams	FIGURE No. B-6 REF. No. SPT 1211C DATE OCTOBER 2008

Appendix C

Site Photographs

Photograph 1 Culvert C8 North side

Photograph 2 Culvert C8 North side

Photograph 3 Culvert C8 South side

Photograph 4 Culvert C8 South side

Appendix D

Rock Core Photographs

Rock Core C1-1

Rock Core C1-2

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 $\overline{N}$.

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	$-^o$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$-^o$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
i_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
i_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
i	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
i_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L - W_L)$	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	HYDAULIC GRADIENT
i_{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 SUBMERED SOIL	e_{max}	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
i'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

FOUNDATION DESIGN REPORT
CULVERT C8, STATION 17+448
HIGHWAY 17, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00
AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-129

Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.

Project: SPT1211C
November 24, 2008

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FOUNDATION DESIGN REPORT
CULVERT C8, STATION 17+448
HIGHWAY 17, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00; AGREEMENT NO. 5006-E-0040

5. DISCUSSION AND RECOMMENDATIONS

The watercourse at Station 17+448 flows southerly through an existing Culvert C8 under Highway 17, and is located about 13 km east of the junction of Highway 533 with Highway 17. The Culvert C8 structure is a 32.0 m long corrugated steel pipe culvert (i.e. CSP) with a diameter of 760 mm.

According to data supplied by D.M. Wills Associates Limited, the invert of the existing CSP is at about elevation 191.3 m at the inlet on the north side of the Highway, dropping to about elevation 189.95 m at the outlet on the south side.

5.1 REHABILITATION OF THE EXISTING CULVERT

Boreholes C1-1, C1-2 and C1-3, which were drilled in the immediate vicinity of the existing culvert show, underlying the culvert, the presence of granular overburden soils overlying bedrock at about elevation 188.1 to 189.3 m. Standard Penetration tests performed in the natural overburden soils yielded N-values which range from 4 to greater than 50 blows/0.3 m, but are typically between 19 to greater than 50 blows/0.3 m. From this, the relative density of the soils can be described as compact to very dense, with very loose to compact soils above the culvert invert.

We understand that the existing 760 mm diameter CSP may be relined to rehabilitate it. For this purpose, a smaller diameter pipe will be placed inside the existing pipe and the space between the two pipes will be grouted. It is anticipated that the thickness of the grout to fill the space in between the pipes will be of the order of 100 to 150 mm. Based on this information, the additional stresses on the surface of the subgrade supporting the pipe would be less than 15 kPa. Using this figure and the subsurface data obtained from the boreholes, the settlement due to the rehabilitation should not exceed 5 mm (i.e. the settlements due to the additional stresses should be negligibly small).

We also understand that the pavement may be rehabilitated without widening the embankment. The rehabilitation of the pavement may involve up to 100 mm grade raise. This may induce additional settlements but the aggregate settlements (i.e. due to culvert rehabilitation and the minor grade raise) should not exceed 10 mm. This is considered acceptable.

We recommend that the grouting be carried out in a manner so as not to cause an uplift of the liner pipe and also of the road surface.

We also recommend that during the construction the amount of grout pumped be checked and compared with calculated volumes and in the event of a discrepancy, the construction will need to be halted and the reason(s) for the discrepancy will need to be investigated.

It is furthermore recommended that the ground surface be observed (especially in areas where the soil cover is limited e.g. side slopes) during grouting for signs of uplift.

To provide sufficiently dry conditions during the rehabilitation of the culvert, it will be necessary to divert the water flowing in the watercourse. This can consist of the construction of a temporary cofferdam, such as pre-cast concrete barrier (i.e. jersey barrier), impervious soil cofferdam barrier, sand bags, etc to divert the water away from the culvert to be relined, or the construction of a temporary culvert, although the latter is believed to be impractical and not cost-effective. In the case of diversion, one solution would be, after the diversion the water can be collected in a temporary pond area and promptly pumped away to where it can be disposed (e.g. downstream of the water course). In addition to diverting, depending on the site conditions at the time of construction, some dewatering may be necessary. This is likely to consist of gravity drainage in shallow perimeter ditches and pumping from strategically placed filtered sumps. It is, however, normally up to the contractor to come up with a plan to achieve a suitable diversion and dewatering (if necessary). We recommend however that the contractor be asked to submit their diversion and dewatering method to the CA for information purposes. We also recommend that in order to minimize dewatering, the construction be carried out during a dry period, if possible.

We furthermore recommend that the contractor be made aware the presence of cobbles and boulders in the existing embankment fill and the possibility of this in the overburden, as well as the possibility of rock fill in the make-up of the existing embankment fill.

5.1.1 EROSION PROTECTION

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 relining 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 watercourse level the boreholes indicate that the native soils can be expected to consist of granular soils ranging in composition from gravelly sand to silty sand. The silty sand would be an erodible soil type.

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 and granular backfill around the 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 watercourse 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 6 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 6 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.

5.2 DETOUR EMBANKMENT

If the construction of a detour embankment becomes necessary, the height of the new embankment will probably match the existing Highway 17 embankment elevation (i.e. about elevation 193 to 196 m) and can therefore be expected to be 1 to 4 m high.

The boreholes show, after stripping the existing organic soils (about 0.1 to 0.2 m can be used for preliminary calculating purposes), the exposed subgrade soils can be expected to consist of sandy soils. Based on the borehole data, no foundation failures are anticipated for up to about 4 m embankments. Normal 2H:1V side slopes can be used.

All organic and otherwise unsuitable soils should be removed within an envelope given by an imaginary slope no steeper than 1:1 from the toe of the proposed embankment. After stripping, the exposed subgrade should be inspected and approved. It should then be compacted, where feasible, from the surface using a suitable compactor.

Proper benching of the existing embankment slope should be implemented where the new abutments abut into existing embankments, as per MTO procedures and in accordance with OPSD 208.010.

The materials used for the construction of the embankment should consist of approved, acceptable earth fill. The embankment fill should be placed on the approved and properly rolled (where feasible) subgrade in lifts not exceeding 300 mm when loosely placed and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. Embankment construction should be carried out in conformance with SP206S03.

Embankment loadings would likely result in a settlement of less than 20 mm (for an embankment height of up to 4 m) due to the settlement of natural foundation soils. About one-third of this settlement should take place within one month, with the majority of the remaining settlements within the next three months.

In addition, the settlement of the new embankment fills under their own weight can be expected to occur. If the embankment is constructed to MTO standards, this should not exceed 20 mm. The time rate will depend on the material used for construction. However, if SSM or granular soils are used, about half of this settlement should be completed within one month and the remaining half substantially completed within one year.

As the combination of these settlements (i.e. 20+20=40 mm) is not excessive, neither surcharging nor preloading is considered necessary for a detour embankment.

Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

5.3 FROST PROTECTION

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

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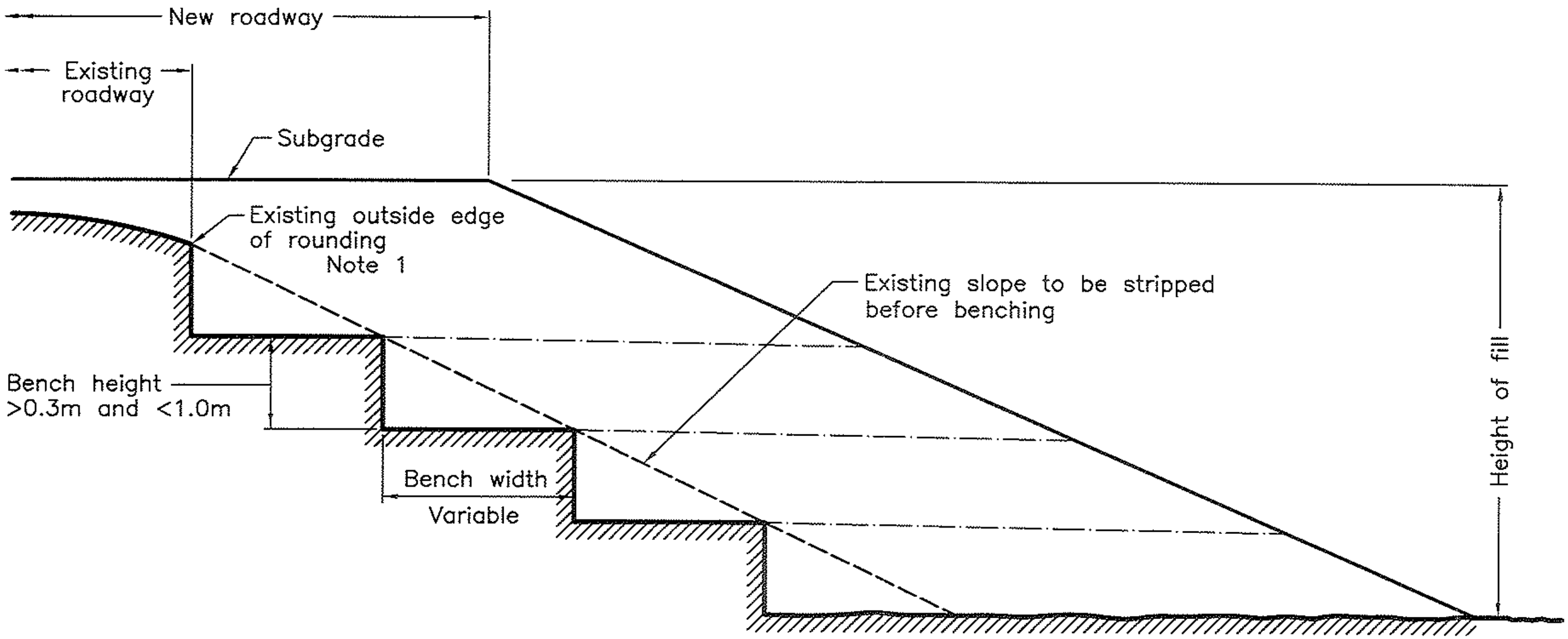
ZO:tr/idrive

Appendix F

OPSD



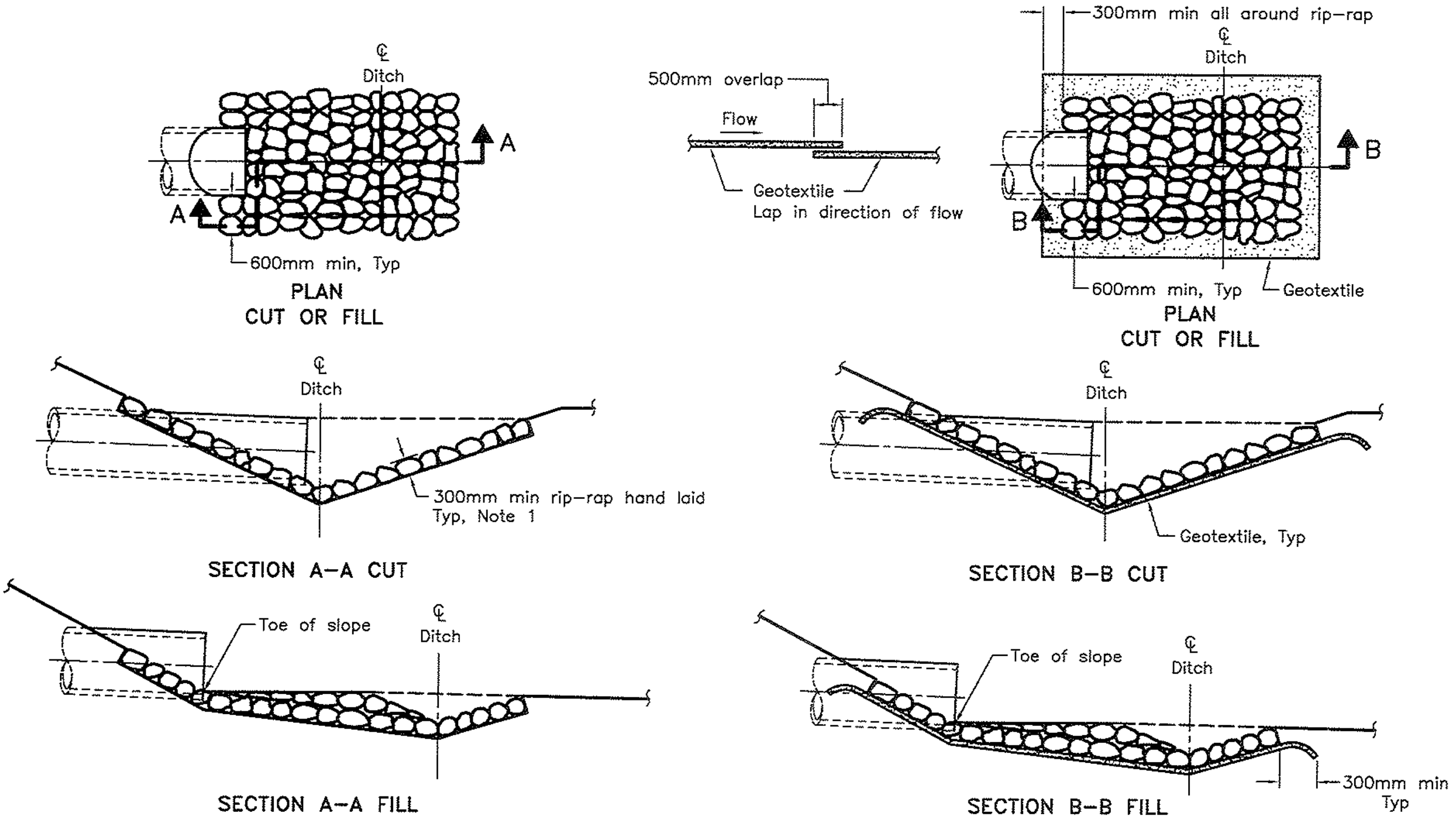
NOTES:

1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.

A Benching is not required on existing slopes flatter than 3H:1V.

B Benches are to be excavated one level at a time and the compacted fill brought up before the next benching level is excavated.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2003	Rev 1
BENCHING OF EARTH SLOPES		
	OPSD – 208.010	



TYPE A – WITHOUT GEOTEXTILE

TYPE B – WITH GEOTEXTILE

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	Nov 2007	Rev 1
RIP-RAP TREATMENT FOR SEWER AND CULVERT OUTLETS		
	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 Shaheen & Peaker, A Division of Coffey Geotechnics Inc. at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

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

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

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

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