

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
CULVERT (C21), STATION 24+215
HIGHWAY 17, FROM 9.5 KM EAST OF HIGHWAY 533
EASTERLY 14.9 KM, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00
AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-131**

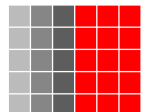
Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

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

**Project: SPT1211E
November 28, 2008**



shaheen & peaker



**20 Meteor Drive
Toronto, Ontario
M9W 1A4**

Tel: (416) 213-1255

Fax: (416) 213-1260

EMAIL: Info@shaheenpeaker.ca

**FOUNDATION INVESTIGATION REPORT
CULVERT (C21), STATION 24+215
HIGHWAY 17, FROM 9.5 KM EAST OF HIGHWAY 533
EASTERLY 14.9 KM, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00
AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-131**

Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

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

**Project: SPT1211E
November 28, 2008**



**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. PROCEDURES	2
4. SUMMARIZED SUBSURFACE CONDITIONS	3
4.1 Topsoil.....	3
4.2 Fill	4
4.2.1. Granular Pavement Fill	4
4.2.2. Embankment Fill	4
4.3 Peat and Organic Silt	5
4.4 Sand.....	6
4.5 Clayey Silt	6
4.6 Sand Till	7
4.7 Bedrock	7
4.8 Groundwater Conditions.....	8

DRAWINGS

DRAWING No.

SITE PLAN, BOREHOLE LOCATION PLAN & SOIL STRATA

1 & 2

APPENDICES

APPENDIX A: RECORDS OF BOREHOLE SHEETS

APPENDIX B: LABORATORY TEST RESULTS

APPENDIX C: SITE PHOTOGRAPHS

APPENDIX D: ROCK CORE PHOTOGRAPHS

APPENDIX E: EXPLANATION OF TERMS USED IN REPORT

**FOUNDATION INVESTIGATION REPORT
CULVERT (C21), STATION 24+215
HIGHWAY 17, FROM 9.5 KM EAST OF HIGHWAY 533
EASTERLY 14.9 KM, CAMERON TOWNSHIP MATTAWA, ONTARIO
G.W.P. 173-98-00; AGREEMENT NO. 5006-E-0040**

1. INTRODUCTION

The rehabilitation of Highway 17, from 9.5 km east of Highway 533 (Mattawa) easterly for 14.9 km, includes the rehabilitation or replacement of a number of existing culverts.

Shaheen & Peaker (S&P), A Division of Coffey Geotechnics Inc., was retained by D.M. Wills Associates Limited to carry out a foundation investigation at the site of the proposed replacement of Culvert C21 under Highway 17 at Station 24+215. The site is located 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 20 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 during the period from May 6 to May 14, 2008 and consisted of drilling and sampling five boreholes to depths of 9.9 to 15.1m below grade. Three of the boreholes (C3-1, C3-2 and C3-3) were advanced adjacent to the existing culvert to depths of 9.9 to 14.0 m below existing grades. Two boreholes (C3-RP1 and C3-RP2) were advanced some 77 m west and 74 m east of the culvert, to 12.3 and 15.1 m below the paved road surface. The locations of the boreholes are shown 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 overburden 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 natural overburden or rock fill and boulders in the embankment fill. 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 C3-3 on completion.

A laboratory testing programme, consisting of natural moisture content determinations, Atterberg Limits tests and grain size analyses, was 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 existing top of highway embankment at the borehole locations has an elevation 218.0 to 220.1 m at this location, while the ground surface elevations at the boreholes advanced beyond the embankment are 213.5 and 214.6m.

At the culvert location the boreholes were advanced from 19.0 m north (Borehole C3-1) and 18.0 m south (Borehole C3-3) of the centerline of Highway 17, while Borehole C3-2 was advanced from the existing Highway 17 shoulder. Borehole C3-1 encountered 0.1m topsoil over a fill deposit to a depth of 1.4 m. Below the fill in Borehole C3-1 and at the surface of Borehole C3-3, a peat deposit was encountered to a depth of 3.2 to 4.1 m, or elevation 210.3 to 210.5 m. Below the peat, sand was encountered, underlain by a rather thick sand till in which the boreholes were terminated. Borehole C3-2 which was drilled from the top of the embankment contacted fill to a depth of 7.5 m or to El. 211.1 m, followed by a 0.4 m thick organic silt layer, underlain by the sand till.

Boreholes C3-RP1 and C3-RP2 were advanced 2.0 to 2.5 m north (left) of the existing road centerline of Highway 17 from the top of the roadway. The existing pavement structure and fill was encountered to depths of 2.4 to 3.7 m. Below the pavement fill, Borehole C3-RP1 encountered a 1.7 m thick clayey silt deposit underlain by an approximately 3 m thick silty fine sand which is in turn underlain by silty sand till. Borehole C3-RP2 contacted silty sand till immediately below the embankment fill at 3.7 m depth. In both boreholes, the till extends to the surface of the bedrock at depths/elevations of 11.9 m/ 208.2 m and 9.5 m/ 208.5 m, respectively.

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 soil and groundwater conditions may vary in between and beyond borehole locations.

4.1 TOPSOIL

A 0.1 m thick topsoil layer was encountered in Borehole C3-1.

4.2 FILL

4.2.1. GRANULAR PAVEMENT FILL

Borehole C3-2 was advanced from the existing road shoulder and contacted a 2.1m thick sand fill deposit. Boreholes C3-RP1 and C3-RP2 were advanced through the existing road and encountered 150 to 250 mm asphaltic concrete underlain by gravelly sand pavement fill to a depth of 0.6 m. The grain-size distribution of a sample from the 2.1 m thick pavement fill in Borehole C3-2 is given in Figure B-1 in Appendix B. The following grain-size distribution is indicated.

Gravel:	16%
Sand:	63%
Silt & Clay:	21%

Standard Penetration tests performed in the granular pavement fill yielded N-values which range from 9 to 38 blows/0.3 m indicating a loose to dense condition.

4.2.2. EMBANKMENT FILL

In Borehole C3-2, the shoulder pavement fill is underlain at 2.1 m by a coarse fill material, to a depth of 3.0 m. This material is likely to be rock fill or possibly a very bouldery granular fill. This rock fill is underlain to a depth of 5.3 m by a granular embankment fill consisting of sand with some gravel and cobbles & boulders (as inferred during the drilling operations). N-values in this granular fill material range from 4 to 18 blows/0.3 m. These values indicate a very loose to compact condition and that a systematic compaction was not applied when the fill was first placed.

At a depth of 5.3 m (El. 213.3 m) the borehole contacted another fill consisting of sandy silt to silty sand fill with gravel and some clayey soil lenses. This may be on site material mixed with granular soils brought to the site during the construction of the culvert. This soil is classified as a granular (i.e. non-cohesive) material and was found to extend to 7.5 m depth or to El. 211.1 m. The recorded N-values in this lower fill range from 10 to 60 blows/0.3 m, indicating a compact to very dense relative density.

In Boreholes C3-RP1 and C3-RP2, the granular pavement fill is underlain by granular embankment fill to depths/elevations of 2.4 m/217.7 m and 3.7 m/214.3 m, respectively. This granular embankment fill can be classified as a granular soil and was found to consist of sand with some gravel and some silt along with occasional cobbles and boulders. The grain size distribution of a sample from Borehole C3-RP2 is given in Figure B-2 in Appendix B, which indicates the following particle size distribution.

Gravel:	24%
Sand:	54%
Silt & Clay:	22%

N-values recorded in these two boreholes range from 6 to 104 blows/0.3 m. These results indicate a loose to very dense condition.

Fill was also contacted in Borehole C3-1, underlying a 0.1 m thick topsoil layer. This fill which was found to extend to a depth of 1.4m consisted of gravelly sand with traces of silt and organics. Standard Penetration tests yielded N-values of 7 and 1 blows/0.3 m, indicating a loose to very loose condition (i.e. no systematic compaction).

4.3 PEAT AND ORGANIC SILT

Boreholes C3-1 and C3-3 were drilled near the toe of the road embankment at the culvert site. Borehole C3-1 which was drilled on the north side of the highway contacted at 1.4 m below the fill deposit, a 1.8 m thick peat layer extending to a depth of 3.2 m or El. 210.3 m. Borehole C3-3 which was also drilled near the toe of the road embankment but on the opposite side (i.e. south side) contacted peat immediately at the ground (o.g.) level. In this borehole the peat deposit was found to extend to a depth of 4.1 m or to El. 210.5 m. In this borehole, the presence of gravel, along with occasional cobbles and sand lenses, was noted in the peat.

The measured natural moisture contents of the peat samples ranged from 44 to 258%. The deposit can be considered a cohesive soil and based on recorded N-values which range from zero to 4 blows/0.3 m (typically 1 to 2 blows) the consistency of the material can be described as very soft to soft but typically very soft.

Owing to its organic origin, the peat can be expected to be a weak and highly compressible soil which can undergo long-term settlements due to a phenomenon known as 'secondary consolidation.'

Underlying the fill in Borehole C3-2, a 0.4 m thick layer of organic silt was contacted between elevations of 211.1 and 210.7 m. The natural moisture content for this material was measured to be 122%.

It should be pointed out that while the thick peat layer contacted in Boreholes C3-1 and C3-3 was not encountered in Borehole C3-2, it is likely that it was stripped when the culvert was first constructed. This opinion is based on the premise that it is unlikely that the peat would not have existed in between the two boreholes drilled 37 m apart only, within a swampy area.

4.4 SAND

In Borehole C3-1, the peat deposit is underlain by a 0.2 m thick silt & sand layer and a 0.4 m thick sand layer to a depth of 3.8 m or to El. 209.7 m. Based on an N-value of 2 blows/0.3 m the relative density of these granular soils is described as very loose.

At Borehole C3-3, the peat is underlain by a 1.9 m thick sand layer in between depths/Elevations 4.1/210.5 m and 6.0/208.6 m. The grain-size distribution of a sample from the deposit is given in Figure B-3 in Appendix B. The following grain-size distribution is indicated.

Gravel:	11%
Sand:	78%
Silt & Clay:	11%

In this granular (i.e. non-cohesive) deposit an N-value of 1 blow/0.3m was recorded near the top (indicating a very loose condition) followed by 22 and 42 blows/0.3 m (indicating a compact and then a dense condition).

A 2.9 m thick silty fine sand deposit was contacted in Borehole C3-RP1 at 4.1 m depth or at El. 216.0 m. This granular soil was found to extend to a depth of 7.0 m or to El. 213.1 m. In this deposit the recorded N-values ranged from 5 to 40 blows/0.3 m, which indicate a loose to dense condition.

4.5 CLAYEY SILT

Underlying the embankment at 2.4 m below the paved road surface or at El. 217.7 m, Borehole C3-RP1 contacted a deposit of clayey silt with silt, sandy silt, silty very fine sand and thin clay interbeds. The thickness of this layered deposit was found to be 1.7 m and it extended to 4.1 m or to El. 216.0 m.

The grain-size distribution of a sample from the lower, more sandy portion of the deposit is given in Figure B-4 which shows the following overall grain-size distribution:

Gravel:	0%
Sand:	28%
Silt :	57%
Clay:	15%

An Atterberg limits test yielded the following index values (see Figure B-5, Appendix B):

Liquid Limit:	31%
Plastic Limit:	22%
Plasticity Index:	9

Natural Moisture Content 27%

These results are characteristic of clayey soils of low plasticity.

Based on N-values of 11 and 20 blows/0.3 m, the consistency of this basically cohesive material is considered stiff to very stiff.

4.6 SAND TILL

Underlying the fill (Borehole C3-RP2), sand deposits (Boreholes C3-1, C3-3 and C3-RP1) and organic silt (Borehole C3-2), all boreholes contacted a basal granular glacial till deposit consisting of sand to silty sand with traces to considerable gravel content and traces of clay size particles. The presence of cobbles and boulders was also noted in the deposit.

This deposit was contacted at depths ranging between 3.7 and 7.9 m below the ground surface or below Elevations 214.3 to 208.6 m. Boreholes C3-1, C3-2 and C3-3 were terminated in this deposit at depths/elevations of 9.9/203.6 m, 14.0/204.6 m and 10.8/203.8 m, respectively. In Boreholes C3-RP1 and C3-RP2, the deposit extended to depths/elevations 11.9/208.2 m and 9.5/208.5 m, respectively, to the surface of bedrock.

The results of three grain-size analyses, presented in Figure B-6, in Appendix B, yielded to the following results:

Gravel:	8-15%
Sand:	46-59%
Silt :	21-25%
Clay:	11-14%

As was mentioned before, the presence of cobbles and boulders was noted during the drilling, which necessitated advancing holes in this deposit by coring.

4.7 BEDROCK

Bedrock was encountered and cored at two borehole locations (C3-RP1 and C3-RP2), as follows:

Borehole No.	Ground Elevation (m)	Overburden Depth to the Surface of Bedrock (m)	Elevation of the Surface of Bedrock (m)
C3-RP1	220.1	11.9	208.2
C3-RP2	218.0	9.5	208.5

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 (T.C.R.) in the bedrock was 91 to 99% for all rock core runs, with Rock Quality Designation (R.Q.D.) values generally between 76 to 81%, with lower values of 20 to 46% within the upper 2m of the rock in Borehole C3-RP1. These results indicate a relatively sound rock, typically with a good rock quality, except in the upper 2m of Borehole C3-RP1 where very poor to poor rock was encountered.

Photographs of the rock cores are shown in Appendix C.

4.8 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 C3-3. The observations are shown on the individual Record of Borehole sheets.

Due to the fact that wash boring and coring was used in the boreholes, most of these observations may not represent stabilized conditions. However, in the piezometer installed in Borehole C3-3, the groundwater level was observed at 0.5 m below the o.g. level or at El. 214.1 m, which is believed to accurately represent the ground water level at the borehole location.

Based on the observations and the moisture contents of the soil samples, it is our opinion that at the time of our investigation the groundwater level adjacent to the culvert was at about El. 214.1 and 212.0 m, while at Boreholes C3-RP1 and C3-RP2 it was at about El. 217-216 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.

SHAHEEN & PEAKER

A Division of Coffey Geotechnics Inc.



Ramon Miranda, P.Eng.

A handwritten signature in black ink, appearing to read 'Z. S. Ozden'.

Z. S. Ozden, P.Eng.



ZO:tr/idrive

Drawings

METRIC

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

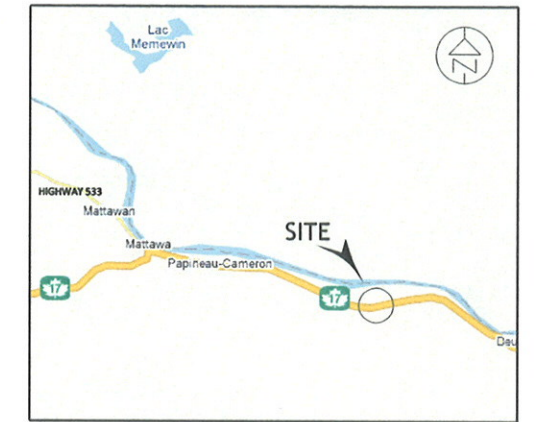
NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

CONT No. 5006-E-0040

GWP: 173-98-00

Highway 17 Mattawa
BOREHOLE LOCATION PLAN &
STRATIGRAPHY (Culvert 21 @ 24+215)

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.



KEY PLAN
N.T.S

LEGEND

- Borehole
- 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
C3-1	213.5	24+212	19.0 m Lt
C3-2	218.6	24+215	4.5 m Lt
C3-3	214.6	24+213	18.0 m Rt
C3RP1	220.1	24+138	2.0 m Lt
C3RP2	218.0	24+289	2.5 m Lt

NOTE

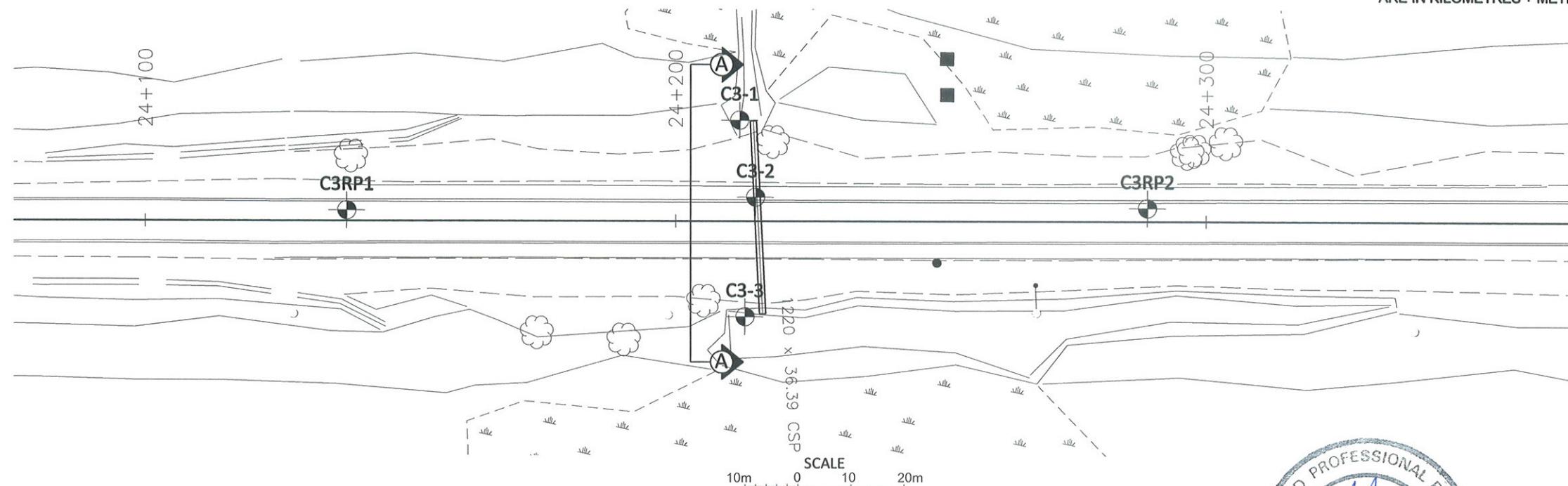
The boundaries between soil strata have been established only at Borehole locations. Between Borehole the boundaries are assumed from geological evidence.

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

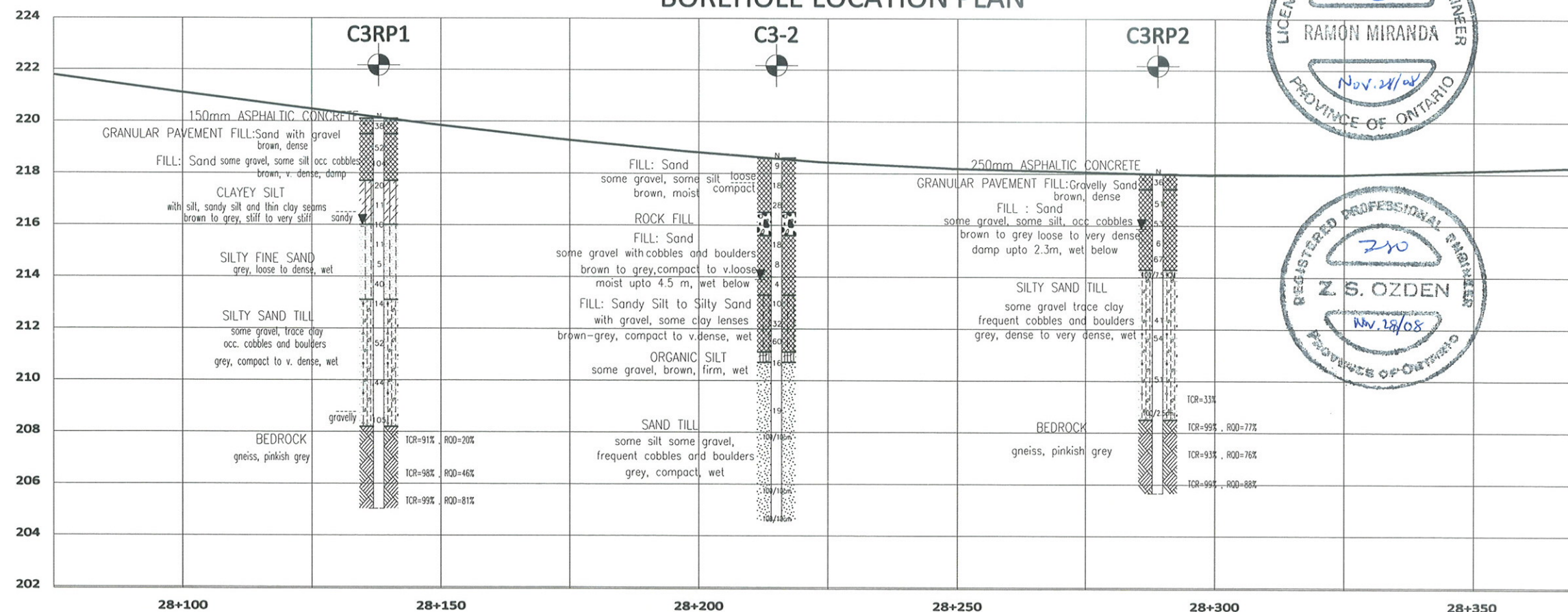
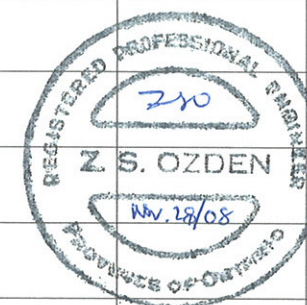
REV.	DATE	BY	DESCRIPTION

Geocres No. 31L-131

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



BOREHOLE LOCATION PLAN



Profile (along Highway 17 centerline)

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

GWP: 173-98-00

Highway 17 Mattawa
STRATIGRAPHY (Culvert 21 @ 24+215)

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.



KEY PLAN
N.T.S.

LEGEND

- Borehole
- 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
C3-1	218.53	24+212	19.0 m Lt
C3-2	218.55	24+215	4.5 m Lt
C3-3	214.56	24+213	18.0 m Rt

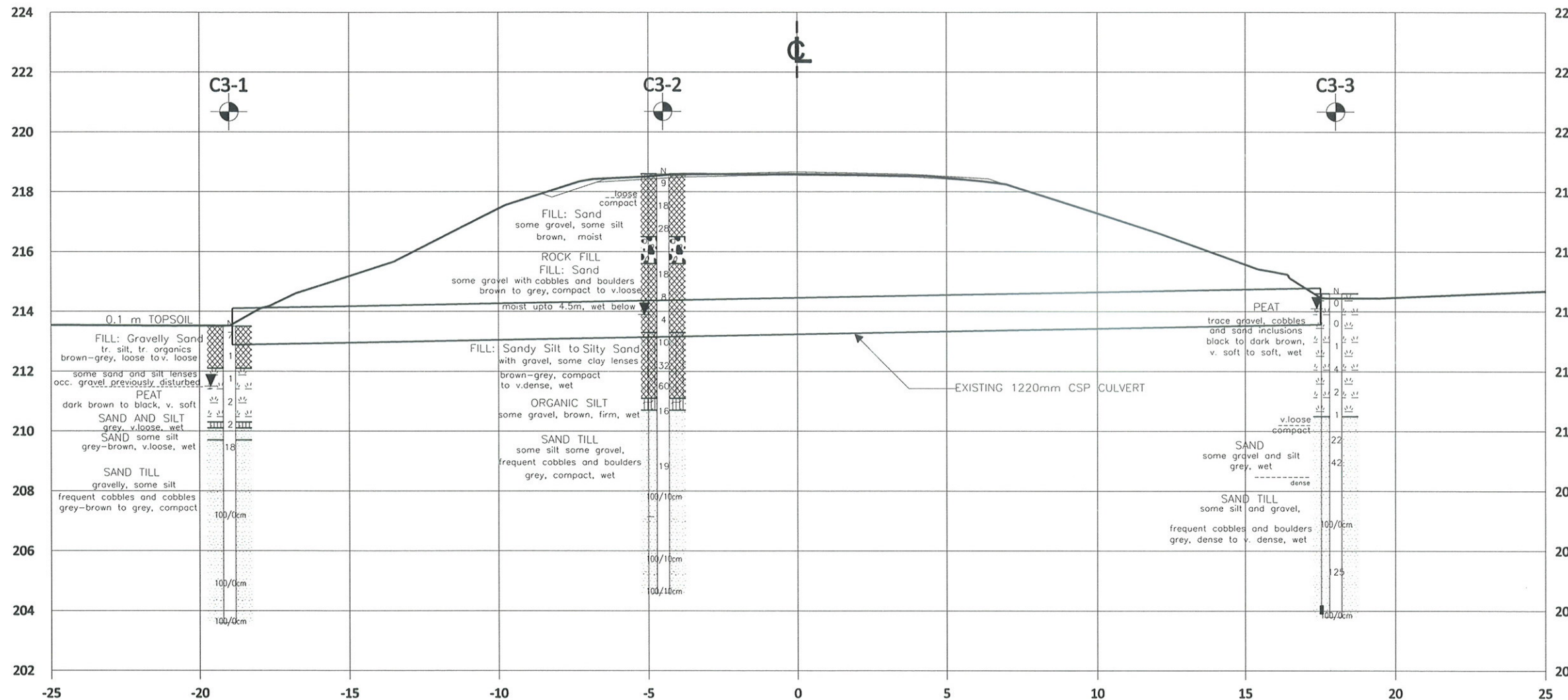
NOTE

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-131			
SPT 1211E			DIST
SUBM'D	CHECKED	DATE Nov. 2008	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 2

HIGHWAY 17



Cross Section (A-A)



Appendix A

Records of Borehole Sheets

SPT 1211E : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C3-1

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 24+212 :19.0m Lt of C/L of Hwy 17 ORIGINATED BY GH
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & Wash Boring & NQ Coring COMPILED BY SS
DATUM Geodetic DATE 5/6/2008 5/7/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)											
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. x LAB VANE							WATER CONTENT (%)										
213.5 0.0	GROUND SURFACE							20 40 60 80 100		10 20 30				GR SA SI CL											
	0.1 m TOPSOIL		1	SS	7	g ^W	213							44 42 (14) Auger refusal @ 4.3 m, advance borehole by Wash boring. Sand back up below 5.0 m due to hydrostatic uplift in borehole. Spoon bouncing on a boulder											
	FILL: Gravelly Sand tr. silt, tr. organics brown-grey, loose to v. loose, moist		2	SS	1										Spoon bouncing on a boulder										
212.1 1.4	some sand & silt lenses occ. gravel previously disturbed		3	SS	1		212									Spoon bouncing on a boulder									
	PEAT dark brown to black, v. soft		4	SS	2		211										Spoon bouncing on a boulder								
210.3 3.2	SAND & SILT grey, v. loose, wet		5	SS	2		210											Spoon bouncing on a boulder							
210.1 3.4	SAND some silt grey-brown, v. loose, wet		6	SS	18														Spoon bouncing on a boulder						
209.7 3.8			7	RC			209													Spoon bouncing on a boulder					
	SAND TILL gravelly, some silt frequent cobbles & boulders grey-brown to grey, compact		8	RC			208														Spoon bouncing on a boulder				
			9	SS	1000000		207															Spoon bouncing on a boulder			
			10	RC			206																Spoon bouncing on a boulder		
			11	SS	1000000		205																	Spoon bouncing on a boulder	
			12	RC			204																		Spoon bouncing on a boulder
203.6 9.9	End of Borehole. Water level @ 2.0 m (not stabilized)* upon completion		13	SS	1000000																				

+³, ×³ Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1211E : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C3-2

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 24+215 : 4.5m Lt of C/L of Hwy 17 ORIGINATED BY GH
 DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NQ Coring COMPILED BY SS
 DATUM Geodetic DATE 5/8/2008 5/12/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE	WATER CONTENT (%)					
218.6 0.0	GROUND SURFACE		1	SS	9									
		loose compact	2	SS	18									
216.5 2.1	ROCK FILL		3	SS	28									
215.6 3.0			4	RC										
			5	SS	18									
			6	SS	8									
			7	SS	4									
213.3 5.3			8	SS	10									
			9	SS	32									
			10	SS	60									
211.1 7.5	ORGANIC SILT		11	SS	16									
210.7 7.9			12	RC										
			13	RC										
			14	SS	19									
			15	RC										
			16	SS	100/10cm									
			17	RC										
			18	SS	100/10cm									
			19	SS	100/10cm									
204.6 14.0	End of Borehole. Water level in open borehole @ 4.7 m (not stabilized)* on May 08, 2008.													

+ 3, X 3: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

METRIC

+ 3, × 3 Numbers refer to Sensitivity

SPT 1211E : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C3-RP1

1 OF 2

METRIC

GWP 173-98-00 LOCATION Sta. 24+138 :2.0m Lt of C/L of Hwy 17 ORIGINATED BY GH
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NQ Coring 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			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
220.1 0.0	GROUND SURFACE											
219.5 0.6	150mm ASPHALTIC CONCRETE GRANULAR PAVEMENT FILL: Sand with gravel, brown, dense		1	SS	38		220					
	FILL : Sand		2	SS	52		219					
	some gravel, some silt occ cobbles brown, v. dense, damp		3	SS	104		218					
217.7 2.4	CLAYEY SILT with silt, sandy silt and thin clay seams brown to grey, stiff to very stiff		4	SS	20		217					
			5	SS	11		216					
216.0 4.1	sandy		6	SS	10		215					
	SILTY FINE SAND grey, loose to dense, wet		7	SS	11		214					
			8	SS	5		213					
			9	SS	40		212					
213.1 7.0	SILTY SAND TILL some gravel, trace clay occ. cobbles and boulders grey, compact to v. dense, wet		10	SS	14		211					
			11	SS	52		210					
			12	SS	44		209					
	gravelly		13	SS	105		208					
208.2 11.9	BEDROCK gneiss, pinkish grey		14	RC	TCR=91% ROD=20%		207					
			15	RC	TCR=98% ROD=48%		206					
			16	RC	TCR=99%							

Continued Next Page

+³ . X³ : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1211E : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C3-RP1

2 OF 2

METRIC

GWP 173-98-00 LOCATION Sta. 24+136 :2.0m Lt of C/L of Hwy 17 ORIGINATED BY GH
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NQ Coring 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
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
205.1							20 40 60 80 100						
206.1							20 40 60 80 100						
15.1	End of Borehole. Water level @ 4.1 m upon completion (not stabilized)*						20 40 60 80 100						

+ 3 . X 3 Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1211E : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C3-RP2

1 OF 1

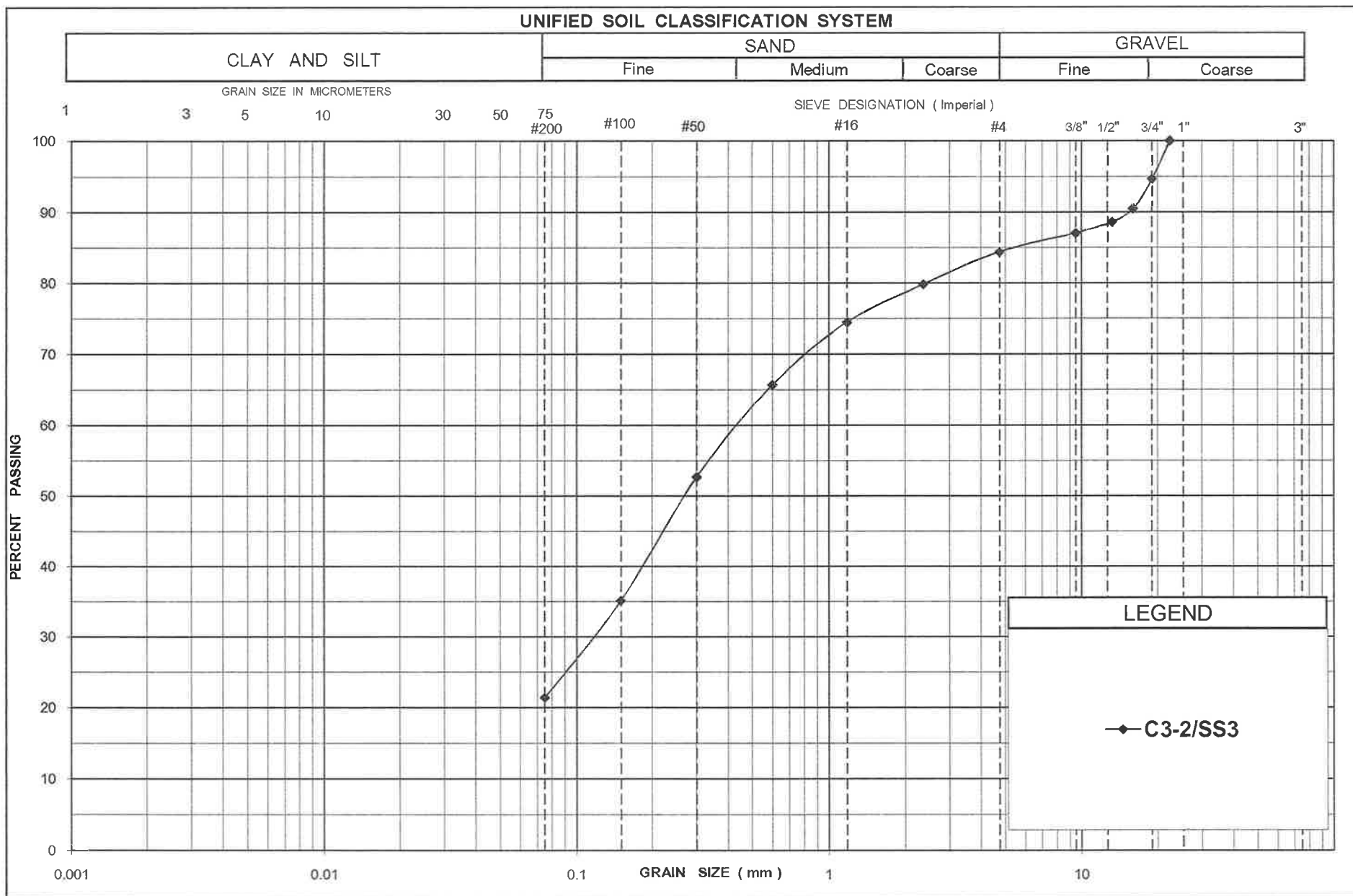
METRIC

GWP 173-98-00 LOCATION Sta. 24+289 :2.5m Lt of C/L of Hwy 17 (WBL) ORIGINATED BY GH
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NQ Coring COMPILED BY SS
DATUM Geodetic DATE 5/14/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
218.0	GROUND SURFACE						218							GR SA SI CL
0.0	250mm ASPHALTIC CONCRETE		1	SS	36		218							
217.4	GRANULAR PAVEMENT FILL: Gravelly Sand brown, dense						217							
0.6			2	SS	51		217							
	FILL : Sand some gravel, some silt		3	SS	53		216							24 54 (22)
	occ cobbles brown to grey loose to very dense, damp upto 2.3m, wet below		4	SS	6		215							
			5	SS	67		215							
214.3			6	SS	100/7.5 CM		214							
3.7							213							
	SILTY SAND TILL		7	SS	41		212							
	some gravel trace clay		8	SS	54		211							
	frequent cobbles and boulders		9	SS	51		210							12 56 21 11
	grey, dense to very dense, wet		10	RC	TGR=33%		209							
			11	SS	100/2.5 cm		208							15 46 25 14
208.5			12	RC	TCR=99% ROD=77%		207							
9.5			13	RC	TCR=93% ROD=76%		206							
	BEDROCK		14	RC	TCR=99% ROD=88%									
	gneiss, pinkish grey													
205.7														
12.3	End of Borehole. Water level @ 2.1 m upon completion (not stabilized)*													

Appendix B

Laboratory Test Results



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

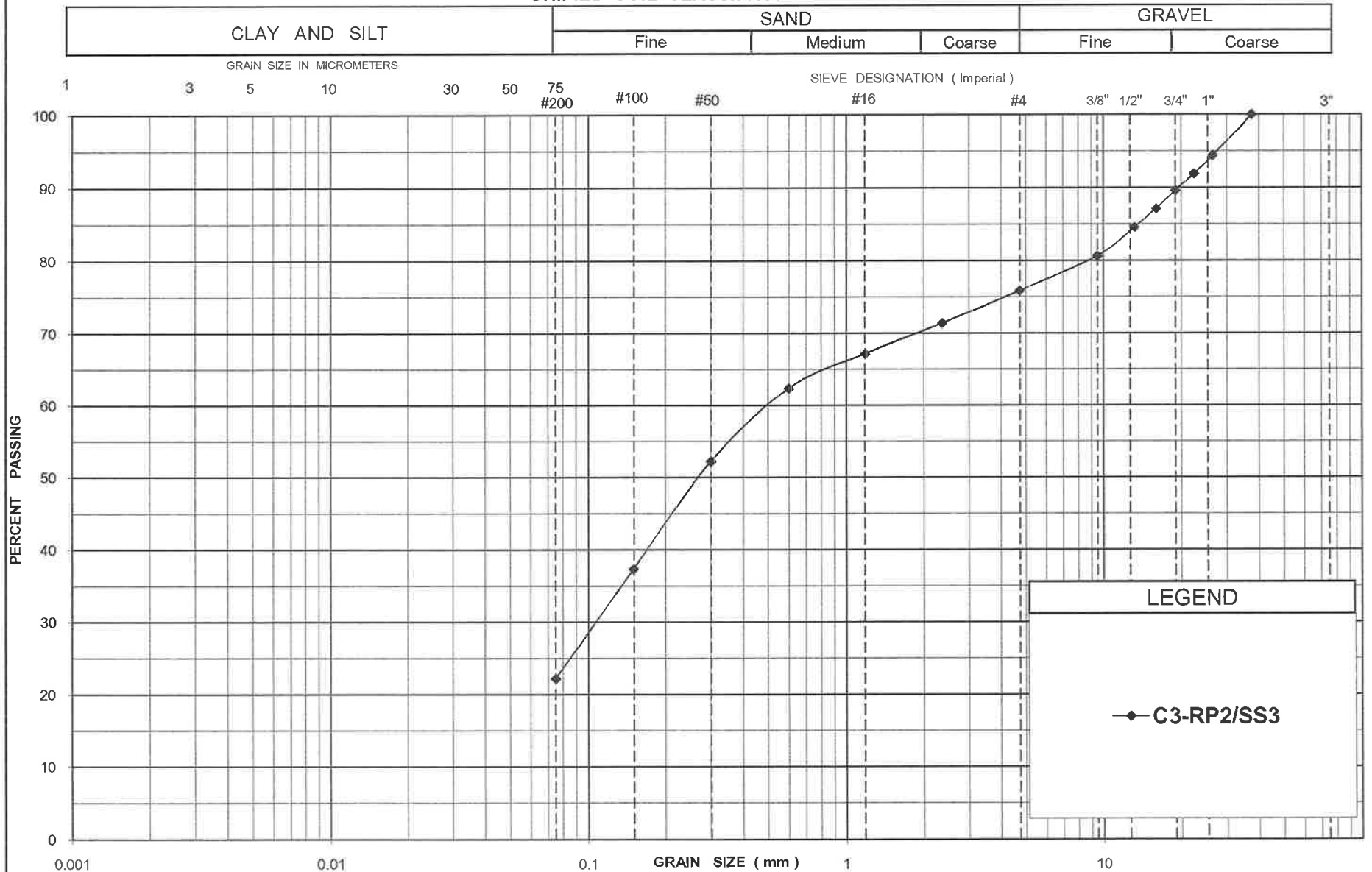
GRAIN SIZE DISTRIBUTION
FILL: Sand, some gravel, some silt

FIGURE No. B-1

REF. No. SPT 1211E

DATE SEPTEMBER 2008

UNIFIED SOIL CLASSIFICATION SYSTEM

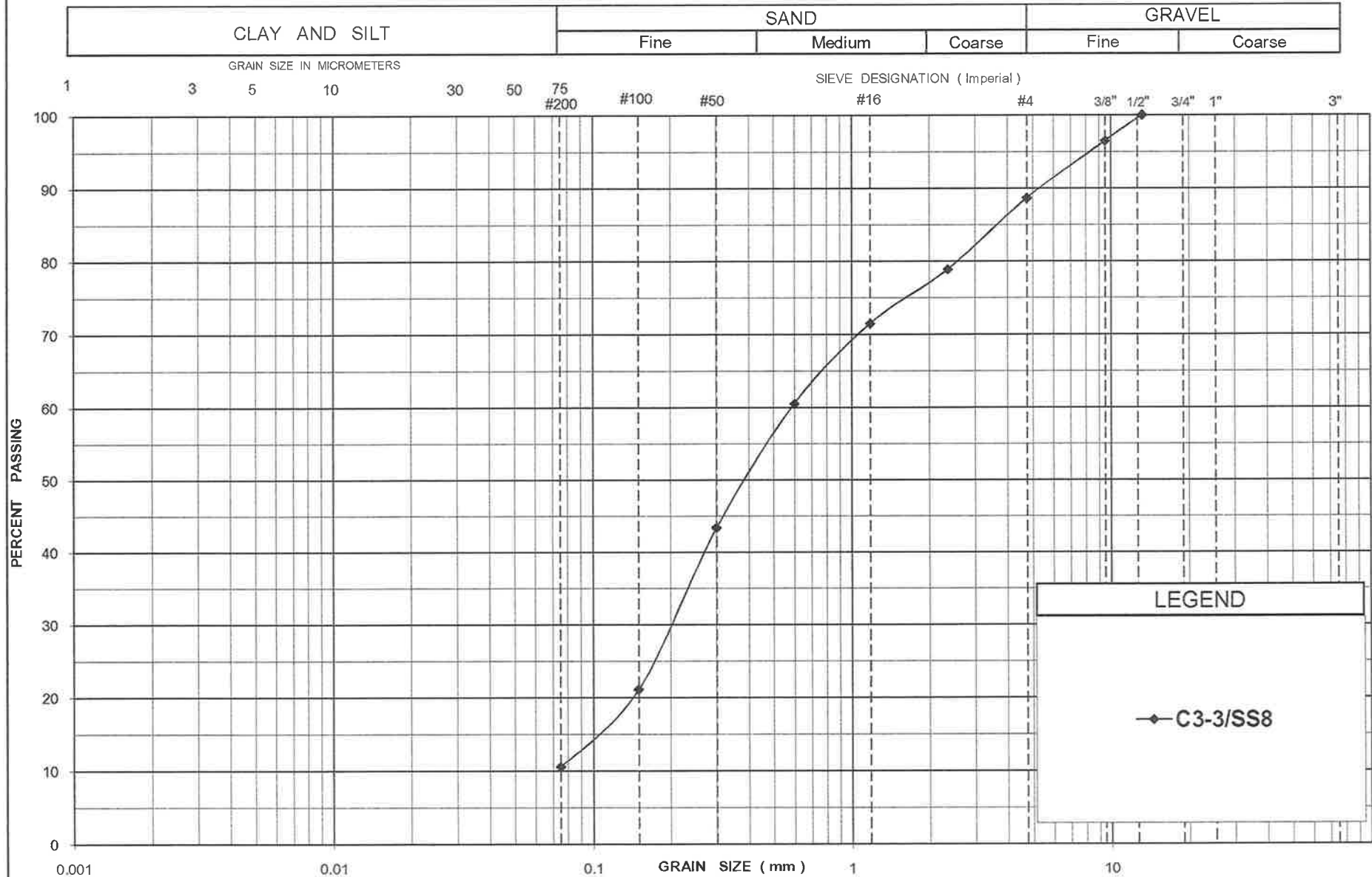


SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION
FILL: Sand, some gravel, some silt

FIGURE No. B-2
REF. No. SPT 1211E
DATE OCTOBER 2008

UNIFIED SOIL CLASSIFICATION SYSTEM



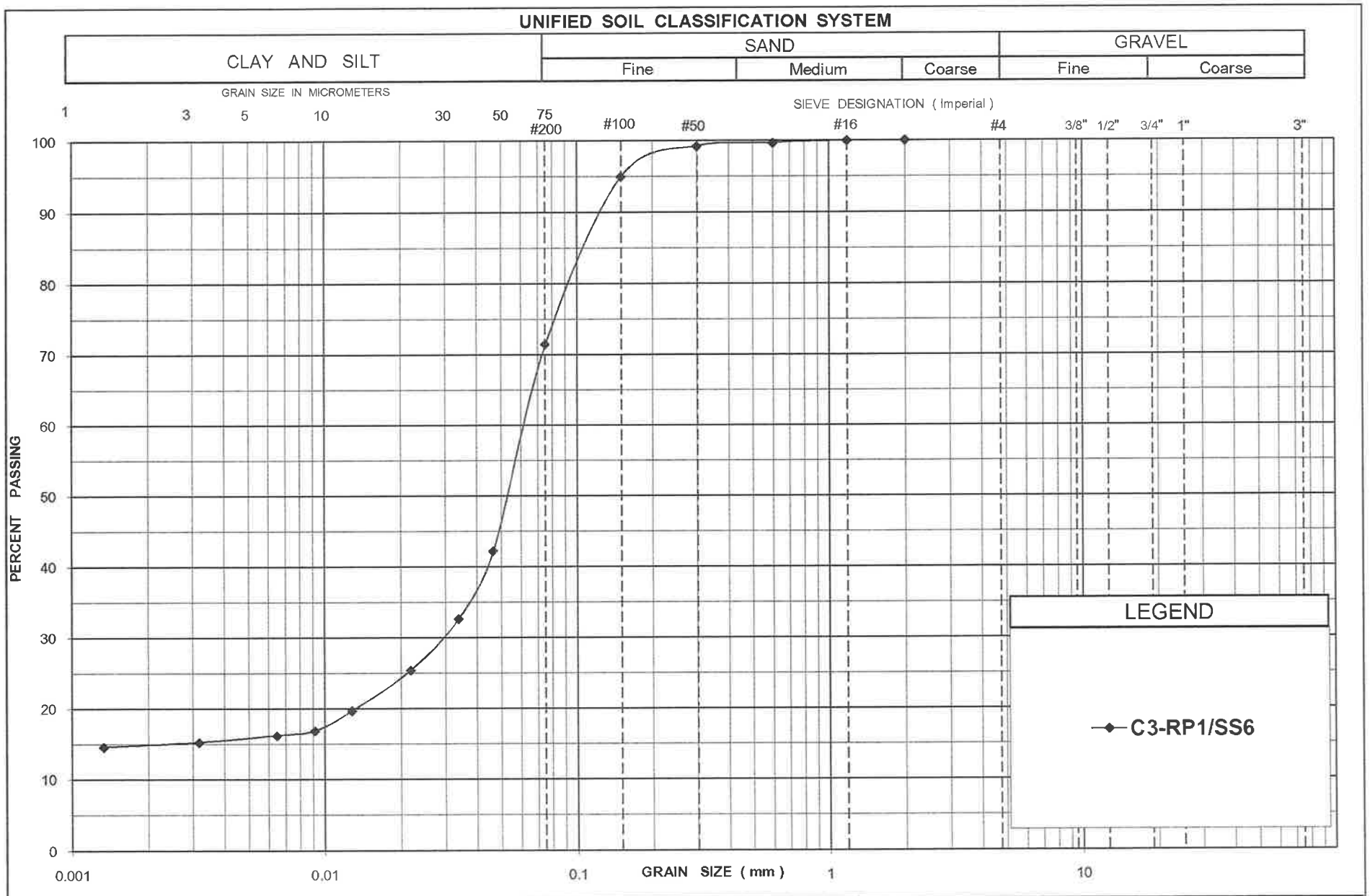
SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION
SAND, some gravel, some silt

FIGURE No. B-3

REF. No. SPT 1211E

DATE SEPTEMBER 2008



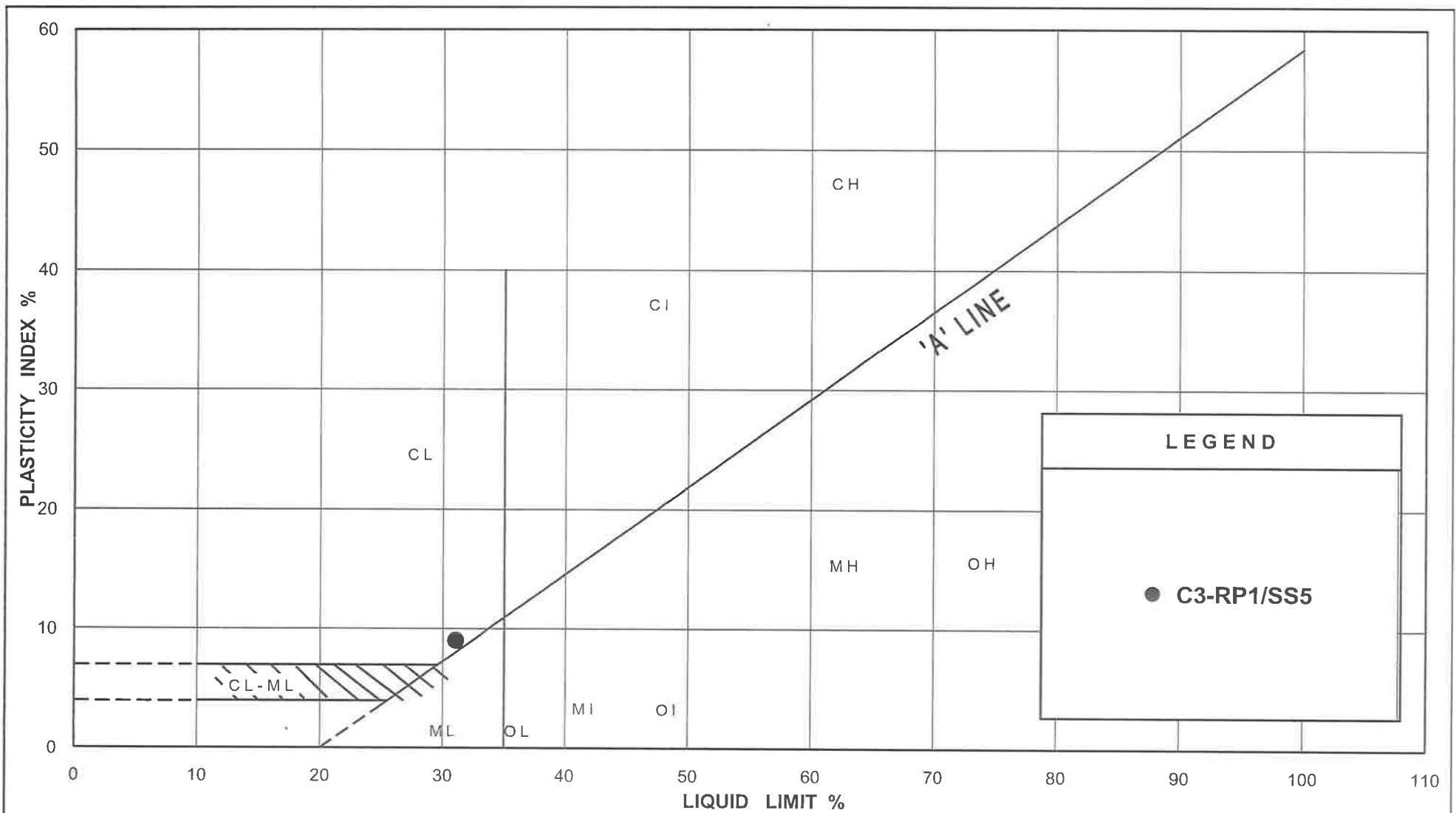
SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION
CLAYEY SILT with silt, sandy silt, silty sand and thin clay interbeds

FIGURE No. B-4

REF. No. SPT 1211E

DATE OCTOBER 2008



SHAHEEN & PEAKER

A Division of Coffey Geotechnics, Inc.

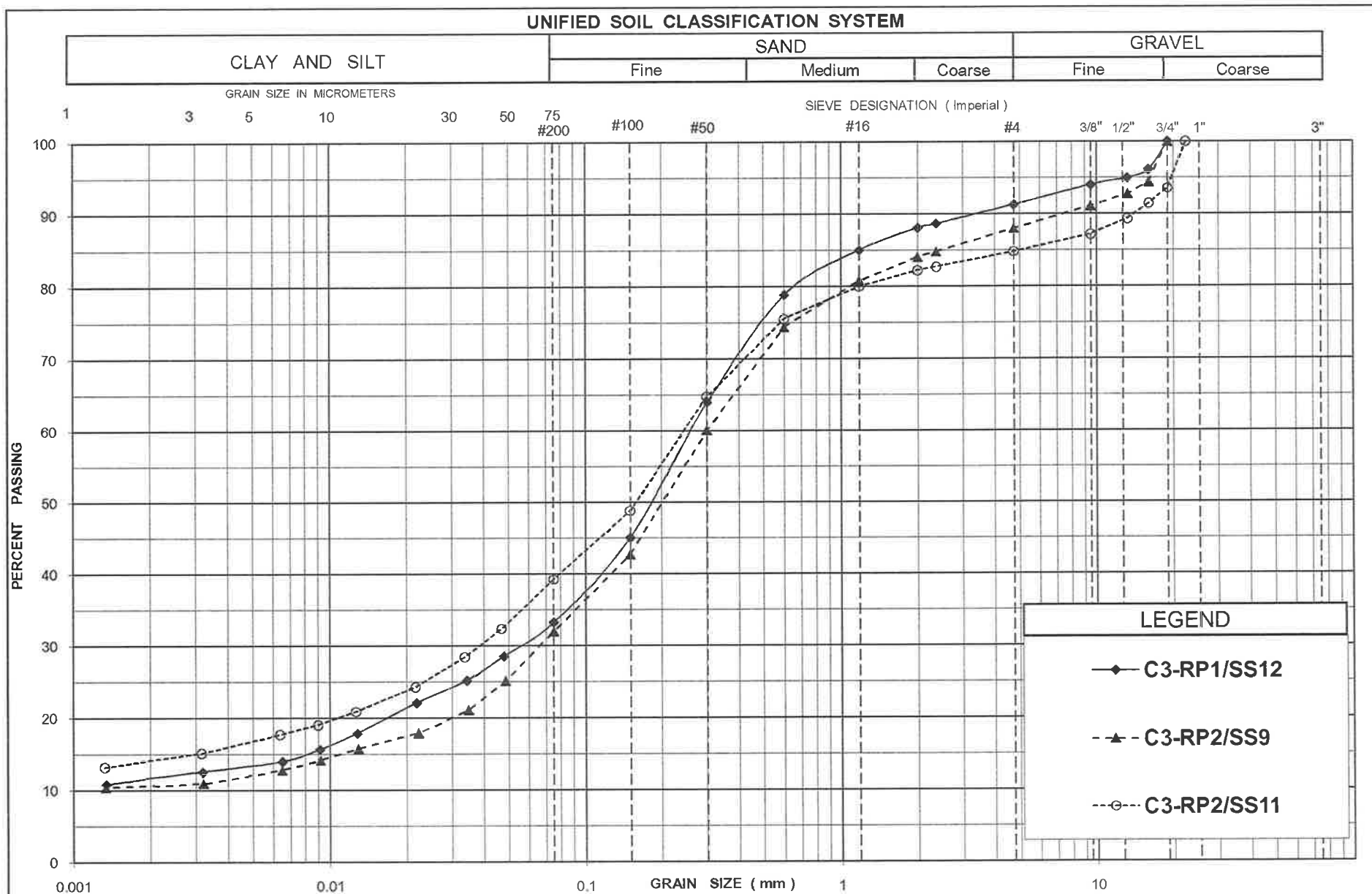
PLASTICITY CHART

CLAYEY SILT with silt, sandy silt, silty sand and thin clay interbeds

FIGURE No. B-5

REF. No. SPT 1211E

DATE October 2008



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

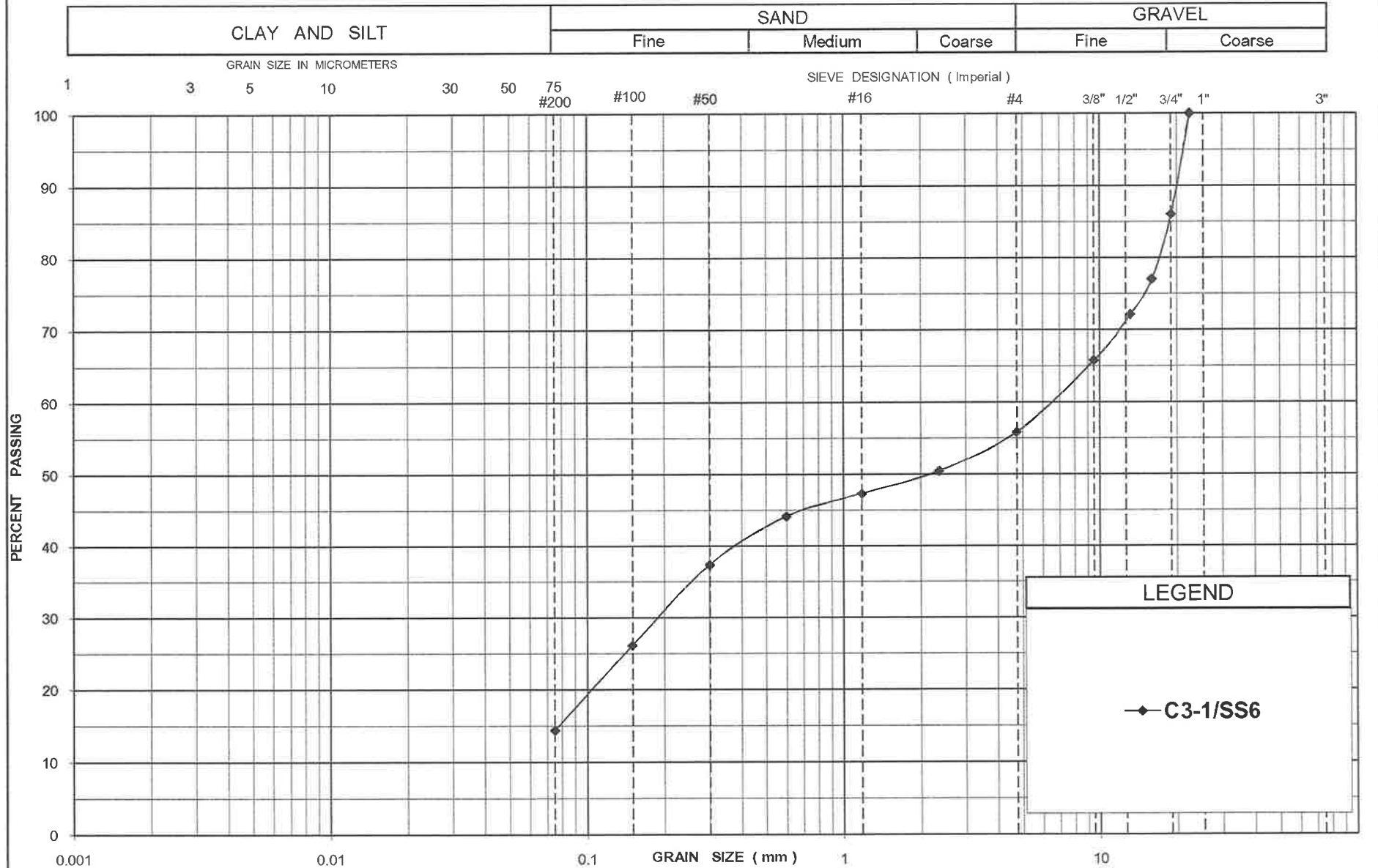
GRAIN SIZE DISTRIBUTION
SILTY SAND TILL, trace clay

FIGURE No. B-6

REF. No. SPT 1211E

DATE OCTOBER 2008

UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION
SAND TILL, gravelly, some silt

FIGURE No. B-7

REF. No. SPT 1211E

DATE SEPTEMBER 2008

Appendix C

Site Photographs



Photograph 1 Borehole C3-1 location (North side)



Photograph 2 North side of embankment at culvert C 21 location



Photograph 3 Borehole C3-3 location (South side)



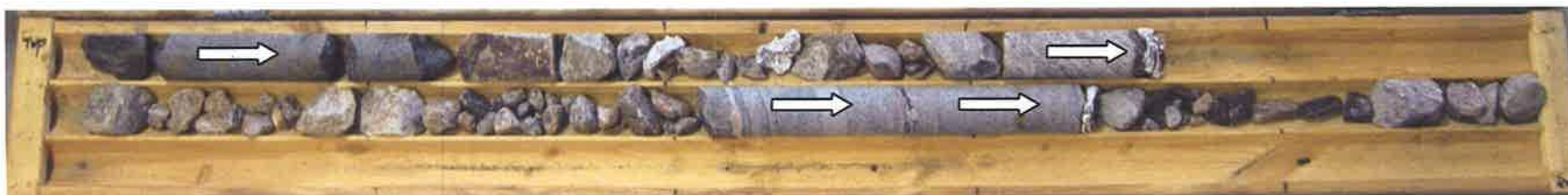
Photograph 4 South side of embankment at culvert C 21 location

Appendix D

Rock Core Photographs



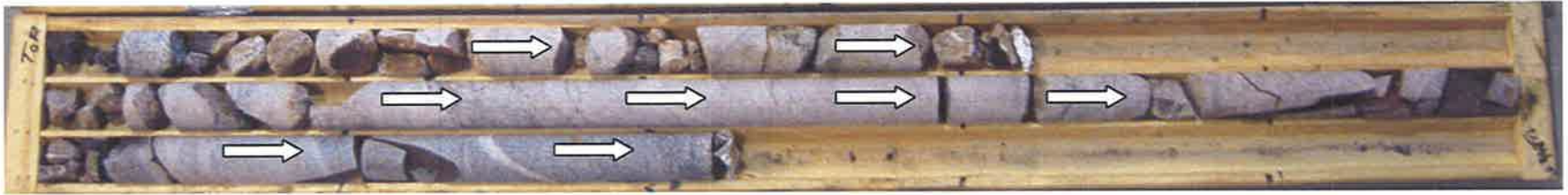
BH C3-1



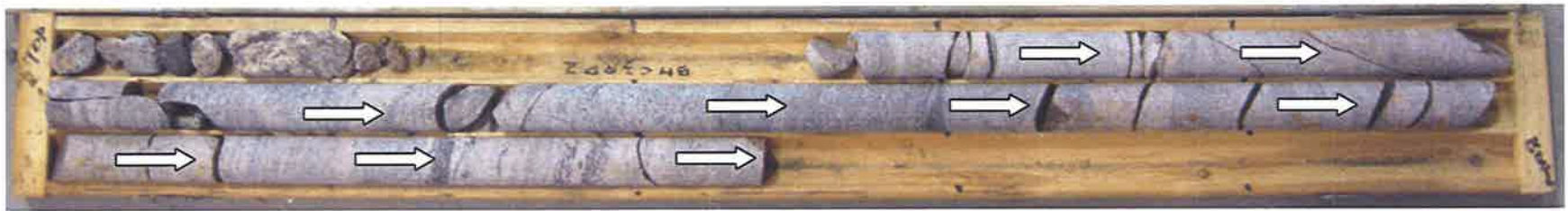
BH C3-2



BH C3-3



BH C3 RP1



BH C3 RP2

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 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
$\sigma_1, \sigma_2, \sigma_3$	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

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'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
j_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
j_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
j	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
j_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_p) / I_p$	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	HYDAULIC GRADIENT
j_{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
j'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
CULVERT (C21), STATION 24+215
HIGHWAY 17, FROM 9.5 KM EAST OF HIGHWAY 533
EASTERLY 14.9 KM, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00
AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-131**

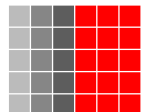
Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

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

**Project: SPT1211E
November 28, 2008**



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
5.1 Culvert Foundation Support	10
5.1.1 Spread Footing Foundations	10
5.1.2 Engineered Fill	11
5.2 Bedding	12
5.3 Backfilling	13
5.4 Construction Comments	14
5.5 Erosion Protection	17
5.6 Retaining Walls	17
5.7 Frost Protection	19
6. CLOSURE	19

APPENDIX F: OPSD

APPENDIX G: LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT
CULVERT (C21), STATION 24+215
HIGHWAY 17, FROM 9.5 KM EAST OF HIGHWAY 533
EASTERLY 14.9 KM, CAMERON TOWNSHIP MATTAWA, ONTARIO
G.W.P. 173-98-00; AGREEMENT NO. 5006-E-0040**

5. DISCUSSION AND RECOMMENDATIONS

Highway 17 crosses a swamp at the existing culvert location at Station 24+215. The ground surface elevations at the borehole locations range from 213.5 to 220.1 m. The existing top of highway embankment has an elevation 218.0 to 220.2m at this location, while the ground surface elevation at the boreholes advanced beyond the embankment range from 213.5 to 214.6 m.

The existing culvert is a 36.3 m long corrugated steel pipe culvert (i.e. CSP) which has a diameter of 1220 mm. The cover over the pipe is about 4 m.

According to data supplied by D.M. Wills Associates Limited, the invert of the existing CSP is at about elevation 213.6m at the inlet on the south side of the Highway, dropping to about elevation 212.9m at the outlet on the north side. We understand that the pipe has a collapsed joint at 6 m from the right (south) end. There is a visible settlement above this section of the pipe (see photographs in Appendix C). We also understand that the existing culvert will be replaced with a new culvert of similar diameter and invert elevations.

Boreholes C3-1, C3-2, and C3-3, which were drilled in the immediate vicinity of the existing culvert show the presence of a peat deposit to a depth of about 3.2 and 4.1 m (or elevation 210.3 and 210.5 m) at the ends of the culvert (i.e. at Boreholes C3-1 and C3-3). In Borehole C3-2, the embankment fill extends to a depth of about 7.5 m, or to elevation 211.1 m. No peat was encountered in this borehole except for a 0.4 m thick organic silt layer; it is therefore likely that the peat may have been stripped from under the pipe area (i.e. beneath the bedding area) when the culvert was first placed, but not near the ends of the culvert. The peat may however been replaced by a sandy silt to silty sand material with gravel rather than a Granular A or Granular B type soil (see type of fill in Borehole C3-2 below 5.3 m or below El. 215.6 m). In Boreholes C3-1 and C3-3, the peat is underlain by sand deposits. Underlying the sand in Boreholes C3-1 and C3-3 and the fill and thin organic clayey silt in Borehole C3-2, the site is underlain by sand till to the termination of boreholes at depths of about 9 to 11 m below the o.g. levels. The relative density of this sand till deposit ranges from compact to very dense. Cobbles and boulders were encountered within this basal till deposit.

Boreholes C3-RP1 and C3-RP2 encountered bedrock at depths of 11.8 and 12.2 m below the road surface or at El. 208.5 and 208.2 m.

At the time of our investigation, the groundwater level at the culvert site in Boreholes C3-1, C3-2 and C3-3 was encountered at about 0.5 m below the o.g. levels, but would be subject to fluctuations. The groundwater level can also be expected to be controlled by the water level in the creek.

The boreholes show, below the proposed invert levels, the presence of unsuitable soils, which will require excavation and replacement in order to support the proposed new culvert. Due to the presence of the organic soils, the use of relatively deep foundations or the removal and replacement of the unsuitable soils with engineered fill will be required.

In our opinion, the engineered fill option would provide a more cost-effective solution than extending the culvert foundation to suitable inorganic native soils below groundwater level. In this case the use of a CSP culvert would be a better choice in comparison with more rigid concrete structures, since the CSP would be more flexible and therefore less sensitive to total and differential settlements.

5.1 CULVERT FOUNDATION SUPPORT

5.1.1 SPREAD FOOTING FOUNDATIONS

The native compact to very dense sands or the sand till, in their undisturbed state, are suitable to support spread footing foundations. The following geotechnical resistances are available, provided that the culvert footings are placed on undisturbed compact to very dense native, inorganic soils.

Borehole No.	Existing Ground Elevation (m)	Recommended Highest Footing Base (Bottom) Level Below Existing Ground Surface (m)	Recommended Footing Base (Bottom) Elevation (m)	Recommended Factored Geotechnical Resistance at U.L.S. (kPa)	Recommended Geotechnical Resistance at S.L.S. (kPa)	Subgrade Material
C3-1	213.5	3.6	209.9	300	200	Compact sand
C3-2	218.6	8.0	210.6	300	200	Compact Sand till
C3-3	214.6	4.6	210.0	300	200	Compact sand

All footing excavations should be carefully inspected, evaluated and approved by the Geotechnical Engineer appointed by the QVE, who is familiar with the findings of this investigation.

Due to the high water table the site must be carefully dewatered prior to the excavation, in order to preserve the load carrying capacity of the cohesionless soils underlying the site, and to facilitate the construction. A rather extensive dewatering effort will be required, as discussed later in this report.

5.1.2 ENGINEERED FILL

Engineered fill option would involve the sub-excavation of the fill, peat and the organic silt and replacing them with properly compacted engineered fill. The anticipated excavation and replacement with engineered fill depths are given below.

Borehole No.	Existing Ground Elevation at Borehole (m)	Recommended Depth/Elevation of Excavation (m)
C3-1	213.5	3.2/210.3
C3-2	218.7	7.9/210.7
C3-3	214.6	4.2/210.4

As mentioned before, extensive dewatering will be required to stabilize the subgrade soils but to a somewhat lesser degree in comparison with spread footing option on natural soil. In either case, the site dewatering will be preceded with diversion of the water in the existing culvert or alternatively the existing culvert can be used to maintain the flow of the creek water while constructing the new culvert at a location close to it. In this event after the construction of the new culvert, the existing culvert would need to be removed or grouted. If grout is used for this purpose, we recommend a foam type grout to avoid additional settlement. In case of removal, during the removal of the existing culvert and fill placement, it must be ensured that no lateral yielding of the new culvert and of the soils supporting it should be allowed to take place, otherwise excessive settlements would likely ensue.

We recommend that a granular fill be used for the construction of the engineered fill. It is also recommended that the granular fill consist of a well-graded soil such as Granular 'A' or Granular 'B' Type I or II. The use of poorly graded granular fill (such as clear limestone) may allow fines and sand particles to infiltrate into the fill causing settlements. If poorly graded fill is used, the exposed subgrade (bottom) and the sides as well as the top must be protected with a suitable geotextile. The placement of a geotextile may, however, in practice, be impractical to implement.

All organic soils should be removed within a sufficient zone supporting the pipe. This can consist of a horizontal distance of at least one diameter of pipe on each side, extending to the bottom of the organic soil at a slope no steeper than 1H:1V. For example, if the organic soil (e.g. peat) extends 2.0 m below the invert of the pipe, the excavation width for a 1.2 m diameter pipe would be at least 2.0 m+1.2 m+1.2 m+1.2 m+2.0 m= 7.6 m.

After the excavation and removal of all unsuitable soils to a sufficient depth, the granular fill should be placed in shallow lifts not exceeding 300 mm before compaction. An exception to this may be the bottom layer immediately above the exposed and approved subgrade. The thickness of this layer may need to be increased to 0.5 to 0.6 m, if the bottom of the excavation is not sufficiently dry and stable to effect proper compaction. Each layer of the engineered fill should be uniformly compacted to not less than 95% of the Standard Proctor Maximum Dry Density (SPMDD) of the material. The degree of compaction of the upper 0.5 m of the granular fill (which would receive the bedding material) should be increased to at least 98% of the SPMDD. A Bearing Resistance at ULS of 240 kPa and an SLS value equal to 150 kPa can be assigned to the engineered fill placed in this manner. The SLS value is based on the premise that the settlements will not exceed 25 mm. This value includes settlements due to stress increase arising from soil exchange (i.e. engineered fill will weigh more than in-situ soils) as well as a grade raise of up to 100 mm due to possible pavement rehabilitation (i.e. up to 100 mm of extra asphaltic concrete placement).

As mentioned before, with this approach a flexible CSP culvert is the preferred option rather than a concrete culvert. If a flexible CSP culvert is used another approach could be to effect the excavation in the vicinity of Borehole C3-2 to El. 212.6 m rather than El. 210.7 m given in the above table and below this elevation to check the subgrade by digging test pits to evaluate the competence of the subgrade and the presence of organic soils, to decide the depth of stripping. This is because in Borehole C3-2 the fill below El. 212.6 m appear to be well-compacted (i.e. N-values=32 and 60 blows/0.3 m) and the thickness of the organic soil underlying the fill at El. 211.1 m was only 0.4 m. If conditions are found to be similar or better than those found in Borehole C3-2, then the excavation depth can be limited to El. 212.6 m under this portion of the roadway. It is anticipated that with this approach the settlements would be somewhat greater (i.e. by about 10 mm or for a total of about 35 mm rather than 25 mm) due to the secondary consolidation of the organic silt over a prolonged period of time. However, this settlement should not be excessive for a CSP (i.e. flexible) culvert. An at least 250 mm thick well compacted granular bedding (i.e. Granular Type A or Type B II) material should be placed under the pipe. With this approach a Bearing Resistance at ULS of 170 kPa and an SLS value of 100 kPa can be used. As mentioned before the SLS value with this approach assumes a settlement of up to 35 mm.

5.2 BEDDING

The bedding material should be placed as soon as practicable after the preparation of the subgrade, as discussed, its inspection and approval. The bedding should be in accordance with the appropriate standards (e.g. OPSD-802.010 and 802.014) and should consist of not less than a 250 mm thick layer (after compaction) of approved granular material, such as Granular 'A'. The thickness of the bedding material may need to be increased depending on

the site conditions at the time of construction. The bedding material should be compacted to at least 98% of the material's SPMDD. If the bedding is to consist of a poorly graded material such as clear crushed stone, a suitable geotextile should be placed as a separator at the bottom and sides of the excavation, as well as the top.

5.3 BACKFILLING

The bedding and embedment material should be extended along the sides to cover the pipe. The selection and placing of the backfill should be in accordance with OPSD-802.010 and OPSD-802.014. The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular 'A' or 'B' (OPSS-1010). All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted to at least 96% of the material's SPMDD. The Granular 'A' and Granular 'B' sub-base courses should be compacted to 100% of the material's SPMDD.

We would like to point out that the performance of flexible pipe culverts is largely dependent on the side support provided by the backfill and the adjacent soils. The use of adequate backfill material and especially good compaction are, therefore, necessary for proper side support. For the same reason, the organic soils should be removed within a suitable distance from the footprint of the culvert. This distance could be minimum one culvert diameter on each side plus daylighting up at no steeper than 1H:1V. For example, if the peat or other organic soil extends to 1.0 m above the invert of the pipe, this distance would be $1.0 + 1.2 + 1.2 + 1.2 + 1.0 = 5.6$ m. The use of heavy compaction equipment should, however, be avoided immediately adjacent and above the pipe, as per MTO practice. During backfill placement, the height of the backfill should be maintained at approximately same level on both sides of the pipe, to avoid lateral displacement of the pipe.

Proper frost treatment is required in accordance with OPSD-803.030 or 803.031, whichever is applicable.

Backfilling behind any retaining (wing) walls, if any, should consist of granular materials in accordance with the MTO standards. Free draining backfill materials, weepholes, etc. should be provided in order to prevent hydrostatic pressure build-up.

Computation of earth pressures acting against rigid culvert walls and any wing walls should be in accordance with the Canadian Highway Bridge Design Code (CHDBC). For design purposes, the following properties can be assumed for backfill.

Compacted Granular 'A' or Granular 'B' Type II

Angle of Internal Friction $\phi=35^\circ$ (unfactored)

Unit weight = 22 kN/m³

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.27$	$K_a=0.34$	$K_a=0.40$
$K_b=0.35$	$K_b=0.44$	$K_b=0.50$
$K_o=0.43$	$K_o=0.56$	$K_o=0.62$
$K^*=0.45$	$K^*=0.60$	$K^*=0.66$

Compacted Granular 'B' Type I

Angle of Internal Friction $\phi=30^\circ$ (unfactored)

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.33$	$K_a=0.42$	$K_a=0.54$
$K_b=0.41$	$K_b=0.52$	$K_b=0.64$
$K_o=0.50$	$K_o=0.66$	$K_o=0.76$
$K^*=0.57$	$K^*=0.74$	$K^*=0.86$

Note:

- K_a is the coefficient of active earth pressure
- K_b is the backfill earth pressure coefficient for an unrestrained structure including compaction efforts
- K_o is the coefficient of earth pressure at rest
- K^* is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

These values are based on the assumption that the backfill behind the retaining structure is free-draining granular material and adequate drainage is provided.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients. The use of vibratory compaction equipment behind the culvert and the retaining walls should be restricted in size as per current MTO practice.

5.4 CONSTRUCTION COMMENTS

The flow in the existing watercourse will need to be maintained during construction. This can be achieved by placing a temporary pipe for the construction period or using the

existing culvert for this purpose. The flow would then be diverted to the completed new culvert. Alternatively, if the excavation is carried out during a dry season the water can be diverted to a holding area and pumped downstream across the highway.

Due to the high groundwater level encountered during the investigation, extensive dewatering will be likely required to facilitate the construction and to preserve the load carrying capacity of the founding soils. Dewatering in the form of deep wells and/or well points will be required. It should be pointed out it may not be feasible to place well points across the highway unless traffic is totally diverted. As well, the presence of cobbles and boulders in the overburden may render the installation of well points/deep wells difficult.

We recommend that the Contractor be made aware of dewatering requirements to facilitate the construction. In this respect, the Contractor may choose to dig some test pits to investigate conditions at the time of construction and the methods that may be required for this purpose.

The Contractor should also be made aware of the presence of rock fill, cobbles and boulders in the embankment fill (See photograph in Appendix C) and possibly within the peat and the underlying soils.

We recommend that the Contractor be asked to submit their method of diversion of water and that of dewatering to the CA for information purposes.

The construction of the culverts should be in accordance with SP421S01.

All excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA), Regulation 213/91, as well as the following specifications:

SP 105 S19 – Protection Systems

SP 902 S01 – Excavation and Backfilling to Structures

In accordance with the Province's Safety Regulations, the following soil classification would be applicable.

Fill	:	Type 3 soil above groundwater level and Type 4 soil below groundwater level.
Sand	:	Type 3 soil above groundwater level (or if the soil is dewatered) and Type 4 soil below groundwater level
Peat and organic Silt	:	Type 4 soil

Clayey Silt : Type 3 soil above groundwater level and Type 4 soil below groundwater level

Sand Till : Type 4 soil

All bearing surfaces should be carefully evaluated and approved by the Geotechnical Engineer appointed by the QEV. As well any engineered fill should be carried out under the full time supervision of the Geotechnical Engineer.

In case of spread footings on natural soils, allowance should be made to place a skim-coat of concrete (mud-slab) once the excavation is completed, inspected and approved, without any delay.

It is expected that temporary shoring will be required to support the excavations. In Ontario shoring typically consists of soldier pile and timber lagging. But tight interlocking steel sheet piling system may also be considered within the peat area, with internal bracing, deadman anchors, tiebacks or raker footings whichever ones are appropriate. The shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this case, the required performance level is considered 2. The shoring system should be designed by a Professional Engineer, experienced in this type of work.

The coefficient of lateral earth pressures given in Table 5. 4.1 can be used for the design of the temporary shoring system, based on the borehole results.

Table 5.4.1
Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m ³)
Granular Fill and Embankment Fill	0.33	0.50	3.0	21.5
Peat/organic silt	0.45	0.80	1.0	11.0
Sand	0.36	0.53	2.8	18.0
Clayey Silt	0.38	0.55	2.7	18.5
Sand/Silty Sand Till	0.31	0.47	3.3	21.5
Bedrock	0.12	0.15	5.0	24.0

It should be pointed out that the cobbles and boulders within the underlying native soils and in the embankment fill can be expected to cause problems during the installation of the caisson holes and during the driving of steel sheet piling.

As well, there may be some difficulty in advancing the caisson holes into the rather hard Precambrian bedrock, if this is required to effect shoring.

5.5 EROSION PROTECTION

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 the invert level may consist of erodible soils.

We recommend that concrete cut-off (apron) be constructed both at the inlet and outlet to prevent seepage beneath and around the culvert, especially through the granular engineered fill, 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, if peat is not the prevalent underlying soil. 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 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 0.3 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 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 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 can be expected to consist of organic or sandy 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 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.

5.6 RETAINING WALLS

It is unlikely that new wing walls will be required for the proposed new culvert but the following are provided for completeness.

Backfilling for any retaining (wing) walls should consist of suitable free-draining granular materials, compacted in accordance with the MTO standards and should conform to the applicable OPSD such as OPSD-803.010. For fills below the groundwater level or immediately below the roadway, it is recommended that Granular 'A' or 'B' materials be used. Where necessary, proper tapering as per MTO standards should be provided. The fill should be compacted in shallow lifts, not exceeding 200 mm loose thickness, to at least 97% of the material's Standard Proctor Maximum Dry Density (SPMDD). To avoid damaging or laterally dislocating the structure, care should be exercised when compacting fill adjacent to and immediately on top of the retaining wall structures. Compaction equipment should be restricted in size as per Ontario Ministry of Transportation (MTO) convention to prevent structural damage to the culvert.

Backfill behind any retaining (wing) walls should consist of Granular 'B' type materials (with fines content less than 6 %) and in accordance with the MTO Standards. Free draining backfill materials, weepholes, etc. should be provided in order to prevent hydrostatic build-up, as shown on OPSD-3101.150.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients. The use of vibratory compaction equipment behind the culvert and the retaining walls should be restricted in size as per current MTO practice.

As an alternative to conventional retaining walls, consideration could be given to MTO's Retained Soil System in which case the designer will have to include the geometric, performance and appearance requirements (i.e: medium performance and low to medium appearance).

Based on the results of the boreholes, strip footing foundations to support reinforced concrete retaining walls can be designed for the geotechnical resistances outlined in Section 5.1.

All footing excavations should be carefully inspected, evaluated and approved by the Geotechnical Engineer appointed by the QVE, who is familiar with the findings of this investigation.

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

The structure will need to be checked against overturning and sliding, with an appropriate factor of safety. The unfactored horizontal resistance against sliding between poured

concrete and approved native sand or granular engineered fill subgrade surface can be calculated using a friction angle of 30 degrees.

Due to the presence of the high ground water table, the high permeability of the sandy subgrade and the required dewatering needed in order to construct the footings, the site is not well suited for the construction of a reinforced concrete wall, especially a high wall (i.e. more than about 2 m high).

Consideration can be given to other wall types including RSS (Reinforced Soil System), etc. Gabion type or crib type walls may also be suitable if some lateral yielding would not be objectionable. These aspects can be discussed with us, if desired, once the details of the project are known.

5.7 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

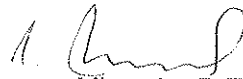
We recommend that once the details of the project are finalized, our recommendations be reviewed for their specific applicability.

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

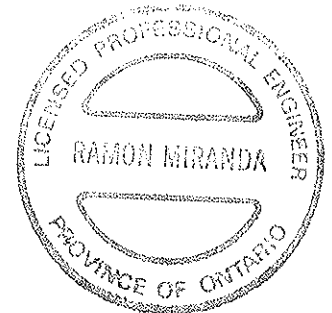
SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.



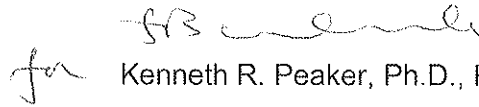
Zuhtu Ozden, P.Eng.



Ramon Miranda, P.Eng.



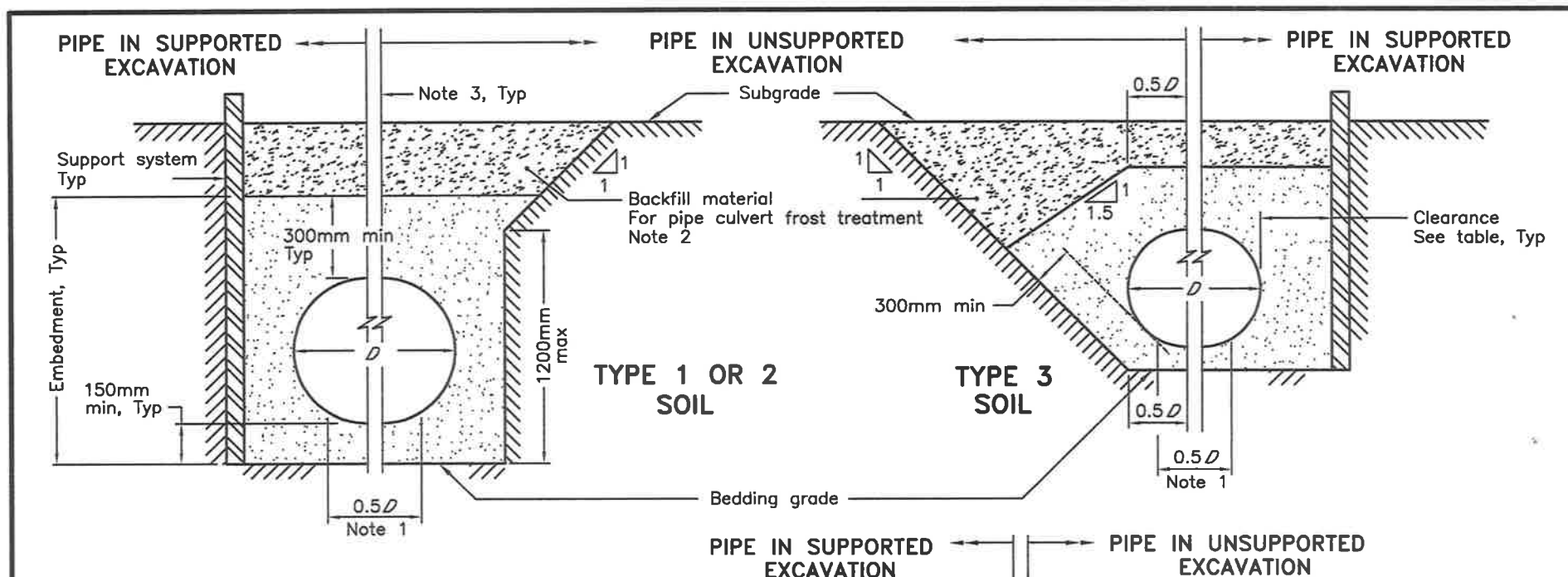
ZO:tr/ldrive



for Kenneth R. Peaker, Ph.D., P.Eng.

Appendix F

OPSD



LEGEND:

D - Inside diameter

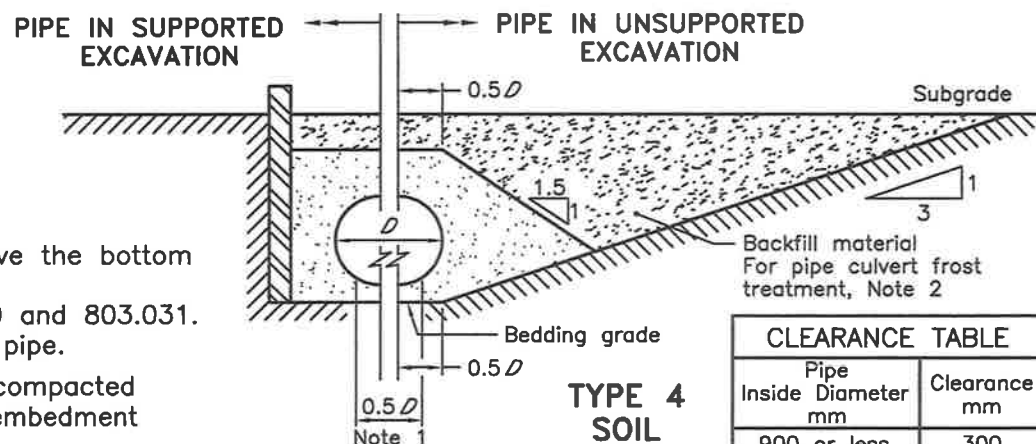
NOTES:

- 1 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
- 2 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
- 3 Condition of trench is symmetrical about centreline of pipe.

A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.

B Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.

C All dimensions are in metres unless otherwise shown.



CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

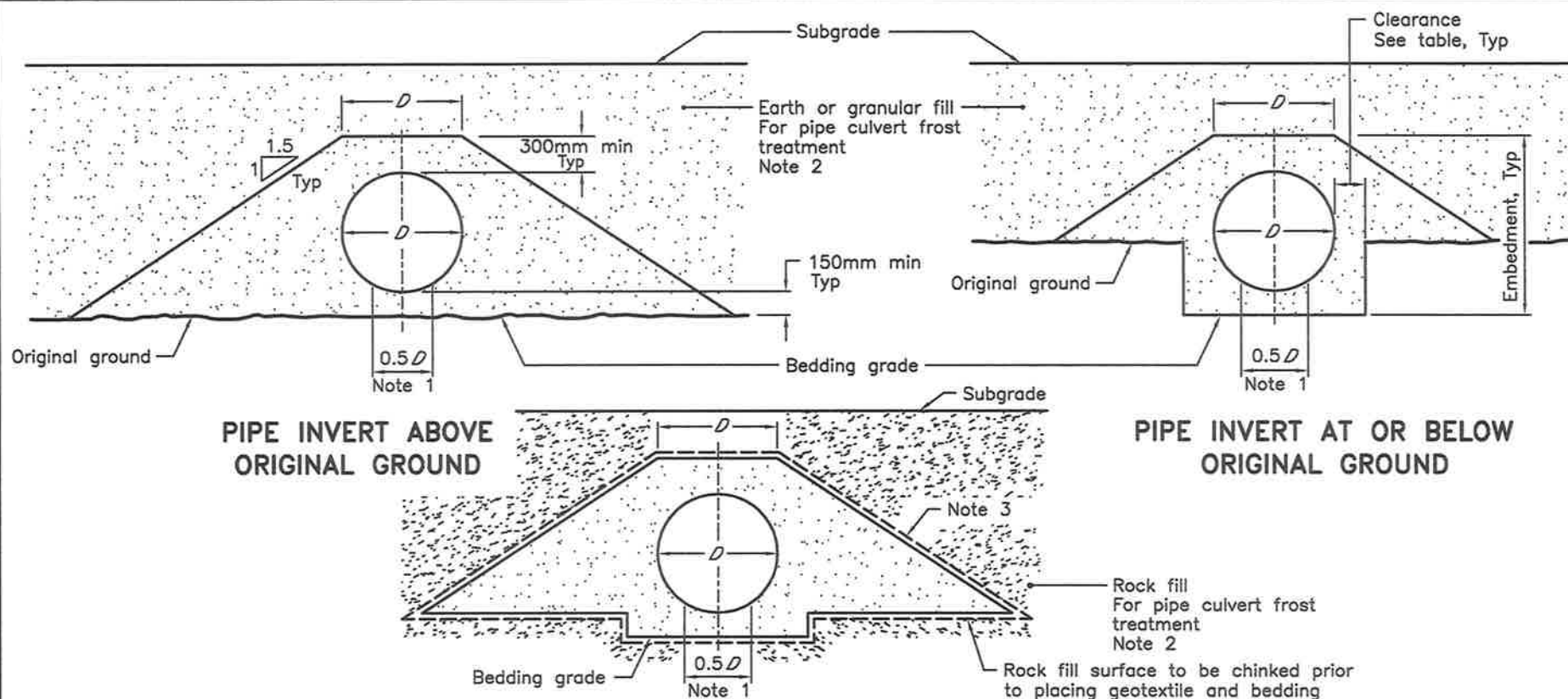
Nov 2005

Rev 1

FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION



OPSD - 802.010



LEGEND:

D - Inside diameter

NOTES:

- 1 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 2 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
 - 3 Embedment material to be wrapped in non-woven geotextile when specified.
- A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
- B All dimensions are in metres unless otherwise shown.

PIPE EMBEDMENT WITH ROCK FILL UNDER AND OVER THE PIPE

CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

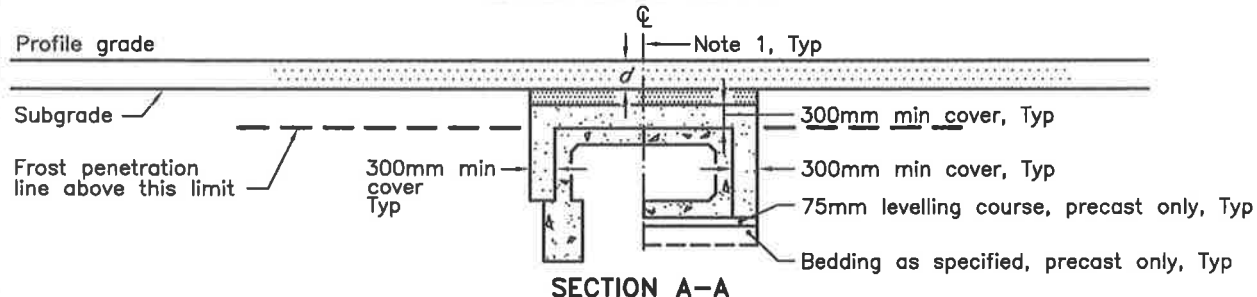
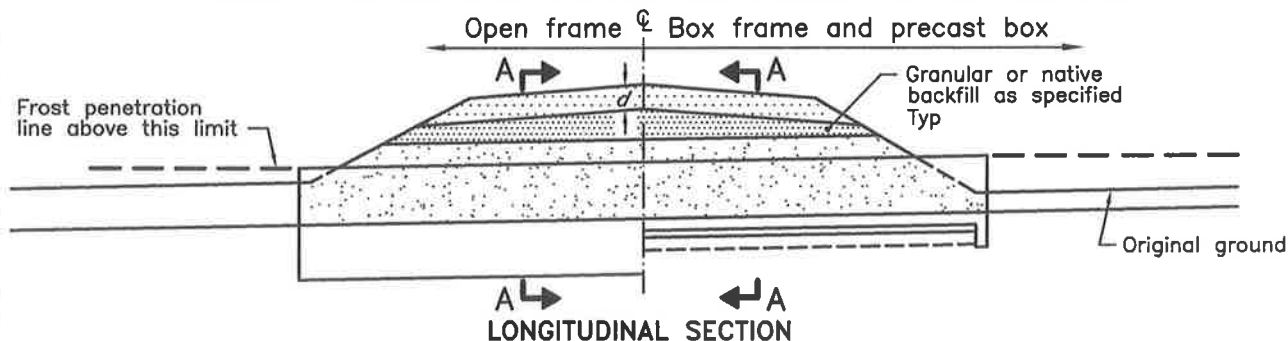
Nov 2005 Rev 1

FLEXIBLE PIPE EMBEDMENT
IN EMBANKMENT
ORIGINAL GROUND: EARTH OR ROCK

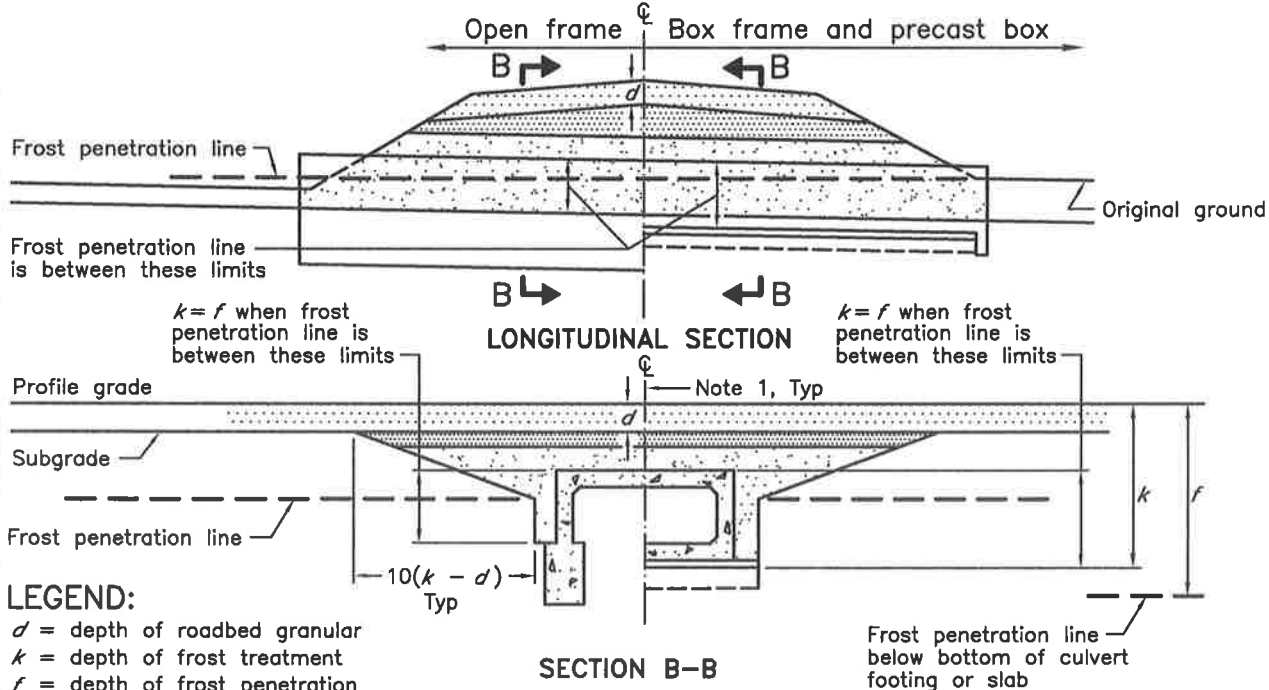
OPSD - 802.014



FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



FROST PENETRATION LINE BELOW TOP OF CULVERT



LEGEND:

d = depth of roadbed granular
 k = depth of frost treatment
 f = depth of frost penetration

NOTES:

- 1 Condition of frost treatment symmetrical about centreline of culvert.
- A Bedding, levelling, and cover material to be granular as specified.
- B This standard applies to cast-in-place and precast concrete culverts with spans less than or equal to 3.0m.
- C The depth of roadbed granular to be 600mm minimum.
- D The maximum depth of frost treatment to be bottom of box frame or top of footing.
- E All dimensions are in millimetres unless otherwise shown.

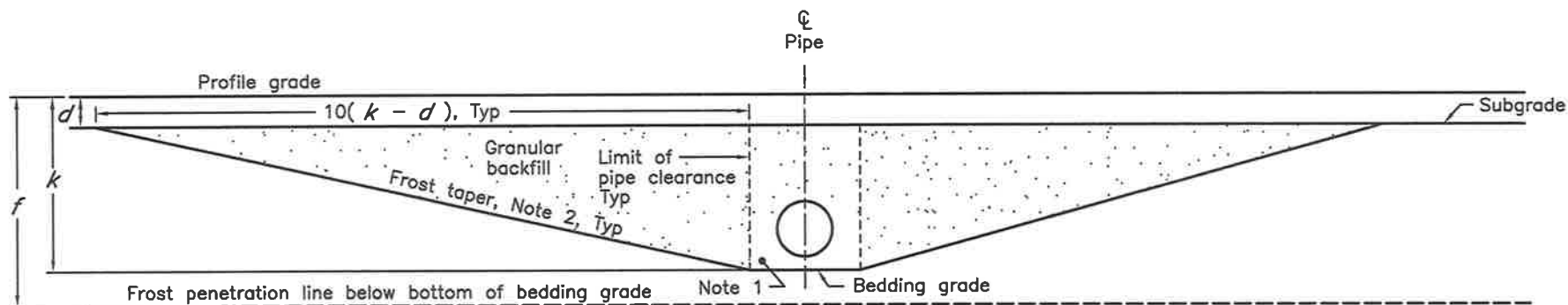
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2006 Rev 1



BACKFILL AND COVER
 FOR CONCRETE CULVERTS

OPSD 803.010



FROST TREATMENT – RIGID AND FLEXIBLE PIPE

NOTES:

- 1 Pipe embedment or bedding, cover, and backfill according to:
 - a) Flexible – OPSD–802.010, 802.013, 802.014, 802.020, 802.023, and 802.024
 - b) Rigid – OPSD–802.030, 802.031, 802.032, 802.033, 802.034, 802.050, 802.051, 802.052, 802.053, and 802.054.
- 2 Frost tapers start at bedding grade.

A Frost tapers are not required in rock embankment.

LEGEND:

- κ –depth of roadbed granular
 k –depth of frost treatment
 f –depth of frost penetration

ONTARIO PROVINCIAL STANDARD DRAWING

FROST TREATMENT – PIPE CULVERTS
 FROST PENETRATION LINE BELOW
 BEDDING GRADE

Nov 2005

Rev 1



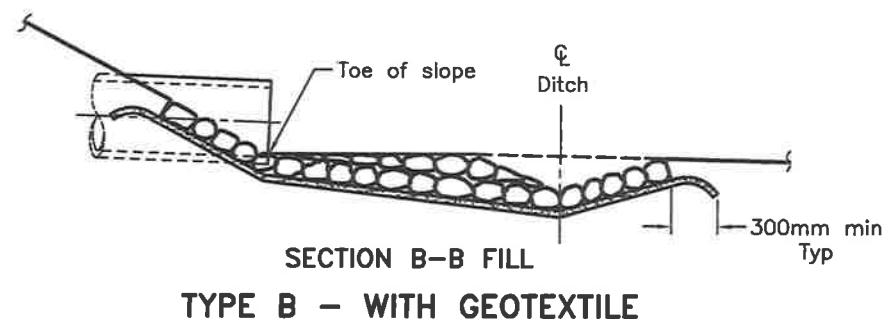
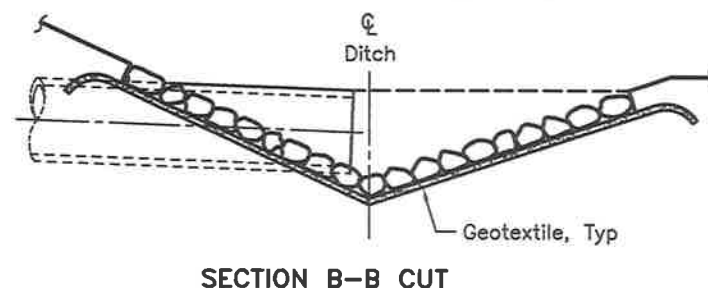
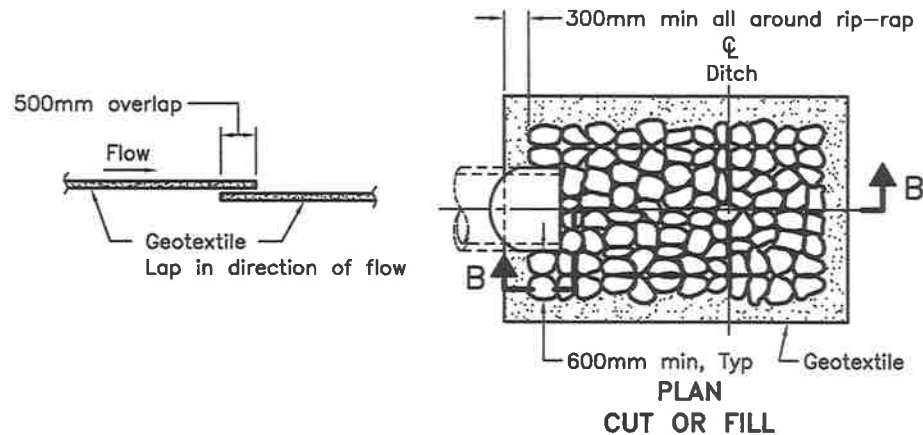
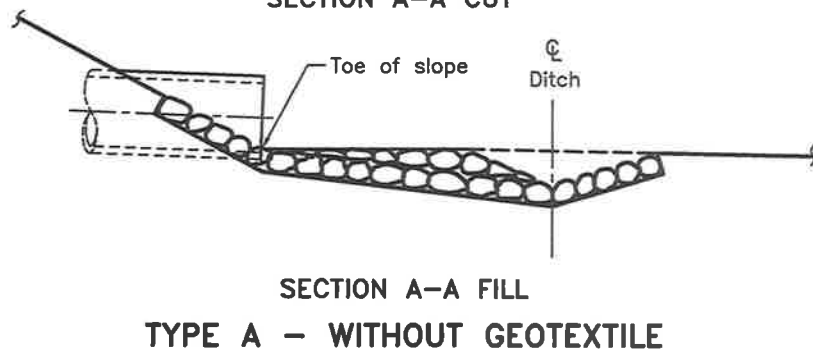
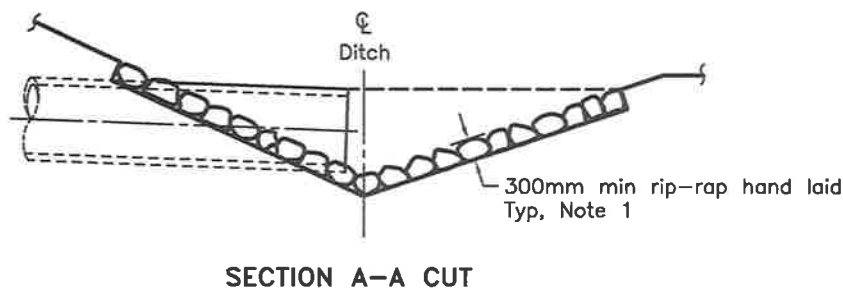
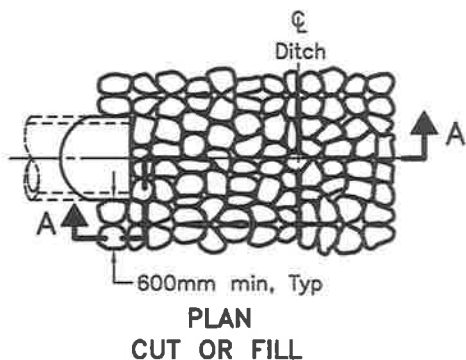
OPSD – 803.030



- 1 Pipe embedment or bedding, cover, and backfill according to:
 - a) Flexible – OPSD-802.010, 802.013, 802.014, 802.020, 802.023 and 802.024
 - b) Rigid – OPSD-802.030, 802.031, 802.032, 802.033, 802.034, 802.050, 802.051, 802.052, 802.053, and 802.054

d — depth of roadbed granular
 k — depth of frost treatment
 f — depth of frost penetration
 $*$ — Type 3 soil
 $**$ — Type 4 soil

- OPSD – 803.031**



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.

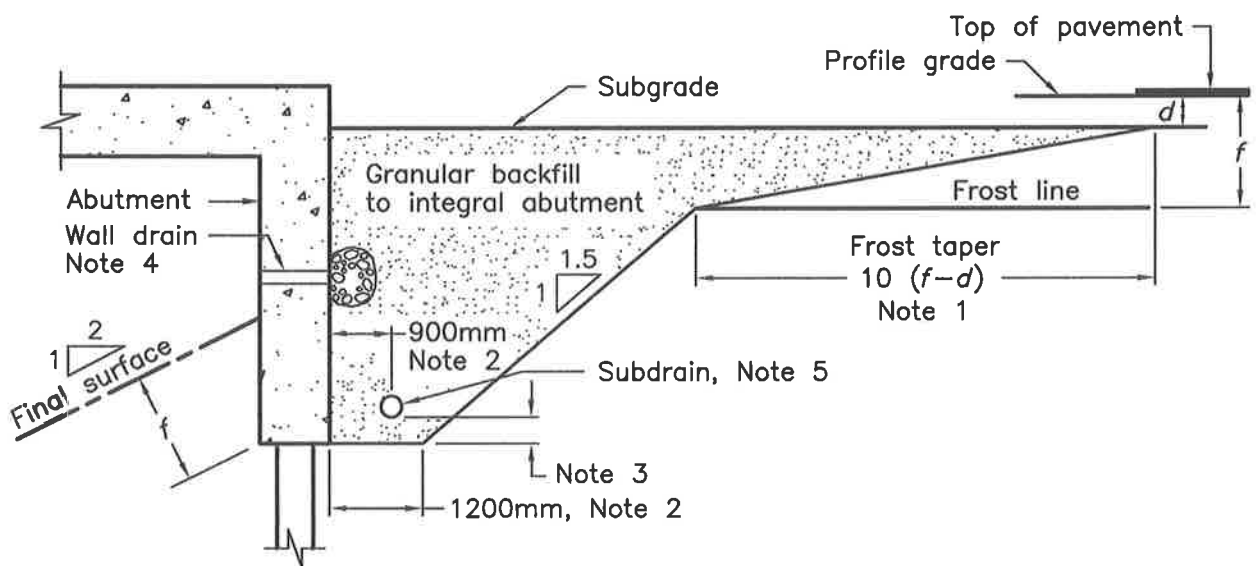
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2007 Rev 1

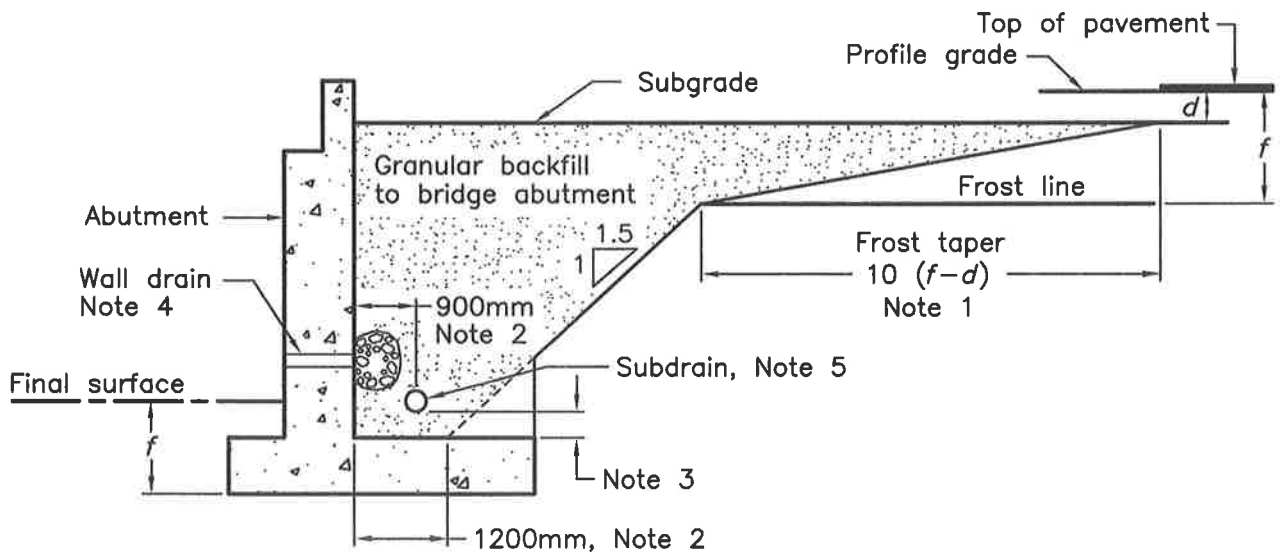
RIP-RAP TREATMENT FOR SEWER AND CULVERT OUTLETS



OPSD 810.010



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

- 1 d = depth of combined base and subbase courses.
 f = roadbed depth of frost penetration as specified.
- 2 Dimensions perpendicular to back face of abutment.
- 3 Height to be consistent with positive drainage of subdrain as specified.
- 4 Where specified, wall drains shall be installed according to OPSD-3190.100.
- 5 150mm dia perforated pipe subdrain wrapped with geotextile.
- A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the fill unless interrupted by the retaining wall or wingwall.
- B Sections shown are parallel to centreline of roadway.
- C Subdrain to be installed with a 2% gradient behind wall.
- D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 Rev 0



WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

OPSD - 3101.150

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.