**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
PROPOSED CULVERT REPLACEMENT (C7) AT
STATION 11+824, HIGHWAY 522 REHABILITATION,
FROM 32.2 KM WEST OF HIGHWAY 524 EASTERLY 6 KM
G.W.P. 480-98-00, DISTRICT 54, SUDBURY
GEOCRES NO. 31E-280**

Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

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

**Project: SPT1218B
November 27, 2008**

shaheen & peaker

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

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1. INTRODUCTION

Shaheen & Peaker (S&P), A Division of Coffey Geotechnics Inc., was retained by D.M. Wills Associates, to carry out a foundation investigation at the site of the proposed replacement of the existing culvert (Culvert 7 at Station 11+824) under Highway 522 near Port Loring, Ontario. This site is located about 0.3 km north of the junction of Highway 522 with Wilson Lake Crescent in Port Loring, Ontario.

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

The findings of the investigation are presented in this report.

2. SITE DESCRIPTION AND PHYSIOGRAPHY

Port Loring is located about 60 km west of Trout Creek which is located at the junction of Highway 522 and Highway 11. The topography near the site is of a rolling nature, with occasional knobs resulting from bedrock outcrops.

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. The Quaternary deposits found in this area are quite complex, having resulted from a variety of geological processes associated with glacial, glaciofluvial, and glaciolacustrine conditions. A large proportion of the area consists of bare bedrock with thin drift. Much of this region is underlain by Precambrian rocks of the Grenville structural Province. These rocks have been strongly metamorphosed, folded, and then intruded by igneous rock.

According to Bedrock Geology of Ontario Map 2544, the bedrock underlying the site consists of Mesoproterozoic Precambrian rocks (i.e. approximately 900 million years old), primarily felsic igneous tonalite, granodionte, monzonite, granite, syenite and derived gneisses.

3. FIELD AND LABORATORY WORK

The fieldwork for this project was performed on June 19, June 25 and June 26, 2008 and consisted of drilling and sampling three boreholes to depths ranging from 7.4 to 11.9 m below the ground surface. 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 continuous flight hollow-stem augers. The boreholes were extended by augering to depths ranging from 4.3 to 11.9 m below the ground surface, to refusal depths on the augers. Within these depths, the sampling was effected 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 (cohesionless) soils (gravels, sands and coarse silts) or the consistency of cohesive soils (clays and clayey silts).

In Boreholes C2-1 and C2-3, after encountering practical refusal on the augers, the bedrock was proven by diamond drilling and NQ size rock cores were obtained. The lengths of the coring were about 3.1 m.

Where the consistency of the soil permitted in the cohesive deposits, the undrained shear strength of the soil was measured in-situ by means of MTO field vane tests. As well, a relatively undisturbed sample was taken in Borehole C2-2 by means of a thin-walled Shelby tube sampler.

Dynamic Cone Penetration Tests (DCPT) were performed from the ground surface adjacent to all three boreholes. In this test, a 51 mm diameter, 60-degree apex cone, screw attached to the tip of an A-size rod, is driven into the ground, using the same driving energy as the SPT method. By recording the number of blows of the hammer to drive the cone/rod assembly, into the soil every 0.3 m, a qualitative record of soil compactness condition is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic force effects which in some cases (such as the present case) affect the SPT results.

The borehole locations were established in the field by S&P engineering staff, in relation to the existing features. The borehole geodetic elevations were provided by D.M. Wills Associates.

Water level observations in the open boreholes were made during drilling and at completion. In addition, a piezometer was installed at Borehole C2-1 to enable us to monitor the ground water level over a prolonged period of time, without interference fro surface water. Water level observations in the piezometer were made by subsequent site visits.

Upon their completion, the boreholes were grouted using quick grout slurry.

The soil samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations, grain size analyses and Atterberg Limits tests, was performed on selected representative 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

Boreholes C2-1 and C2-3, which were drilled at the toe of the embankment, contacted topsoil to a depth of about 0.1 to 0.25 m, underlain by silt to a depth of 0.6 m.

Borehole C2-2 was drilled from the top of the highway embankment and therefore contacted embankment fill materials. The depth of the fill was found to be 5.5 m at the borehole location.

Underlying the embankment fill at Borehole C2-2 location and the surficial silt in the others, all three boreholes contacted an approximately 2 m thick deposit of silty clay. The silty clay is underlain by granular soils to the surface of the bedrock which was contacted at about 4 to 9 m below the original ground (o.g.) levels.

Details of the stratigraphy encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The following paragraphs are only meant to complement and amplify these data. An inferred subsurface cross-section is given in Drawing No. 1.

4.1 TOPSOIL

In the boreholes drilled from the o.g. level, a 0.1 m and 0.25 m thick topsoil layer was encountered in Borehole C2-1 and C2-3, 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.

4.2 FILL

Fill materials were contacted in Borehole C2-2 (drilled from the top of the road embankment) to a depth/elevation of 5.5 m/227.4 m.

Below a 38 mm thick asphalt layer, granular pavement fill (i.e. gravelly sand) was contacted to a depth of 0.5 m. Based on an N-value of 29 blows/0.3 m, the relative density of this pavement fill is described as compact. Underlying the pavement fill, Borehole C2-2 contacted embankment fill materials.

The composition of the embankment fill was found to be very variable ranging from granular sand & gravel, sand with some silt & gravel to silty sand to basically cohesive silty clay materials. But in general, the fill at the borehole location was found to be primarily of a granular nature. The presence of occasional slag and organic mixtures was also noted. As well, the presence of boulders was inferred at 2.1 and 3.6 m depths.

The grain-size distribution of samples from the embankment fill is given in Figures B-1 and B-2 in Appendix B.

The bottom 0.6 m of the fill was found to consist of silty clay with some organic clay and topsoil mixture. This probably represents the original soil which was disturbed and intermixed with the overlying topsoil and organic rich soils due to construction activities when the existing culvert was first built.

Standard Penetration tests recorded in the upper 3.5 m yielded N-values which range from 14 to 30 blows/0.3 m, indicating a generally compact relative density with a stiff zone at a depth of about 2.1 to 2.6 m depth. These results indicate that the embankment fill within the upper 3.5 ±m has received some systematic compaction when it was first constructed. Below 3.5 m the recorded N-values are 5 and 6 blows/0.3 m indicating little or no compaction.

It should also be pointed out that the make-up and degree of compaction of the embankment fill can be expected to be variable across the highway.

4.3 SURFICIAL SILT

In Boreholes C2-1 and C2-3, which were drilled from the o.g. level, immediately beyond the toe of the embankment, a silt deposit was contacted, immediately beneath the topsoil. This deposit was found to extend to a depth of 0.6 m below the ground surface.

Standard Penetration tests in this surficial silt deposit yielded N-values of 1 and 3 blows/0.3 m which indicate a very loose condition.

4.4 SILTY CLAY

Underlying the surficial silt (Boreholes C2-1 and C2-3) and the embankment fill (Borehole C2-2), the boreholes contacted a 1.6 to 2.1 m thick silty clay deposit. This deposit was contacted at El. 228.6 m (Borehole C2-1) to 227.4 m (Borehole C2-2) and extended to El. 226.9 m (Borehole C2-1) to 225.3 m (Borehole C2-2).

The grain-size distribution of three samples from this cohesive deposit is given in Figure B-3 in Appendix B. This indicates the following grain-size distribution:

Gravel:	0-1%
Sand:	4-22 %
Silt:	36-52%
Clay:	41-52%

Based on these results, the deposit is considered to be practically impervious material with a coefficient of permeability (*k*) of less than 10^{-6} cm/s.

Atterberg limits tests (four samples) gave the following index values:

Liquid Limit:	24 – 49%
Plastic Limit:	16 – 27%
Plasticity Index:	8 - 22

As shown in the Plasticity Chart, Figure B-4, in Appendix B, these results are characteristic of clayey soils of low to medium but generally low plasticity. The measured natural moisture contents are typically near or above the measured liquid limit values, which indicate a relatively weak consistency for the deposit.

The recorded N-values in the deposit range from 5 to 15 blows/0.3 m, and a field vane test yielded an undrained in-situ shear strength of 68 kPa. Based on these test results, together with a visual and tactile examination of the soil samples the consistency of the material is described as firm to stiff.

4.5 BASAL GRANULAR DEPOSITS

The silty clay is underlain by basal granular soils. The grain-size distribution of these granular soils generally range from silty fine sand with traces to some clay to sand & gravel.

These basal granular soils were contacted at depths/elevations of 2.3 m/226.9 m, 7.6 m/225.3 m and 2.2 m/226.3 m in Boreholes C2-1, C2-2 and C2-3, respectively. In Boreholes C2-1 and C2-3, these soils were found to extend to 4.3 m/El. 224.9 m and 8.6 m/219.9 m, respectively to the surface of the bedrock. Borehole C2-2 was terminated at El. 221.0 m upon encountering refusal to sampling and to further augering, probably on or near the surface of the bedrock.

The grain-size distribution of three samples from the deposit is given in Figure B-5.

These granular soils are considered to be considerably more pervious than the overlying silty clay deposit.

N-values recorded in these deposits range widely from 1 to 29 blows/0.3 m, indicating a very loose to compact condition. It should also be pointed out that in Borehole C2-3, two N-values of 1 blow/0.3 m each were recorded. This may be due to soil disturbance while sampling, due to upward hydrostatic pressure and the actual (more realistic) N-values may be somewhat higher, as evidenced by the results of the Dynamic Cone Penetration tests (DCPT).

It should also be pointed out that in Borehole C2-3, a 0.7 m thick silt deposit was contacted, above the bedrock surface at a depth of 7.9 m or at El. 220.6 m. The presence of broken rock pieces was noted immediately above the bedrock surface.

4.6 BEDROCK

Based on the behaviour of the augers during the drilling process, the samples recovered from SPT and on the DCPT results, as well as the coring results (where coring was carried out), the surface of the bedrock in the boreholes was inferred/encountered at the following depths/elevations.

Table 4.6.1
Inferred Bedrock Surface

Borehole No.	Existing Ground Surface Elevation (m)	Inferred Bedrock Surface Below Existing Ground Surface (m)	Bedrock Surface Elevation (m)	Coring Carried Out
C2-1	229.2	4.3	224.9	Yes
C2-2	232.9	12.0*	220.9±*	No
C2-3	228.5	8.6	219.9	Yes

*In Borehole C2-2, refusal to further advancing the augers was encountered at a depth of 11.9 m below the road level or at Elevation 221.0 m. As well, a DCPT was performed adjacent to the borehole and this test encountered refusal at about El. 221.1 m. From this and the findings of the other two boreholes, El. 220.9 ±m probably represents the surface of the bedrock or a bouldery layer or broken rock.

In Borehole C2-1, the surface of the bedrock was contacted at El. 224.9 m. In Borehole C2-2, some 13.2 m away, the surface of the rock was estimated at about El. 220.9 ±m. From this, the surface of the bedrock appears to dip down from Borehole C2-1 towards Borehole C2-2 at a slope of 4.0 m/13.2 m or at a steep slope of about 30%. Beyond Borehole C2-2 location to Borehole C2-3 location, the estimated elevation difference is only about 1.0 m or a rate of about 1/21 (i.e. relatively flat).

From a visual examination of the rock cores, the bedrock appears to be a light to dark grey coloured gneiss, with some pinkish grey bands typically at a 30 degree inclination to the horizontal.

The following table presents the percentage of recovery and R.Q.D. values measured on the rock cores obtained in Boreholes C2-1 and C2-3.

Table 4.6.2
Rock Core Information

Borehole No.	**Rock Core No.**	**Percentage of Recovery (%)**	**Measured R.Q.D.* Value (%)**
C2-1	7	100	80
	8	100	0
	9	100	77
	10	100	78
C2-2	12	100	85
	13	100	100
	14	100	100

*R.Q.D. = Rock Quality Designation is the sum of those intact core pieces, 100 mm+ in length, expressed as a percentage of the total length of coring run.

From the above table, it is noted that the total recovery (i.e. percentage of recovery) of the rock cores from the bedrock is consistently 100 %. R.Q.D. values range from 0 to 100% but are typically 77 to 100%. Based on these results, the rock mass quality is described as poor to excellent but generally good to excellent.

4.7 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of each borehole. The observations are shown on the individual Record of Borehole sheets.

A piezometer was installed in Boreholes C2-1 at a depth of 7.3 m. The water levels encountered in the piezometer upon completion and after several days of installation were 0.3 and 0.45 m below ground surface.

Based on the information, the groundwater table at the time of our investigation was at about o.g. elevation.

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

Ramon Miranda, P. Eng.

Zuhtu S. Ozden, P.Eng.

ZO:tr/idrive

Drawings

LICENSED PROFESSIONAL ENGINEER RAMON MIRANDA NOV. 27/08 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER Z. S. OZDEN NOV. 27/08 PROVINCE OF ONTARIO

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.
WP: 480-98-00

Highway 522 Port Loring
BOREHOLE LOCATION PLAN & STRATIGRAPHY (Culvert 7 @ STA. 11+824)

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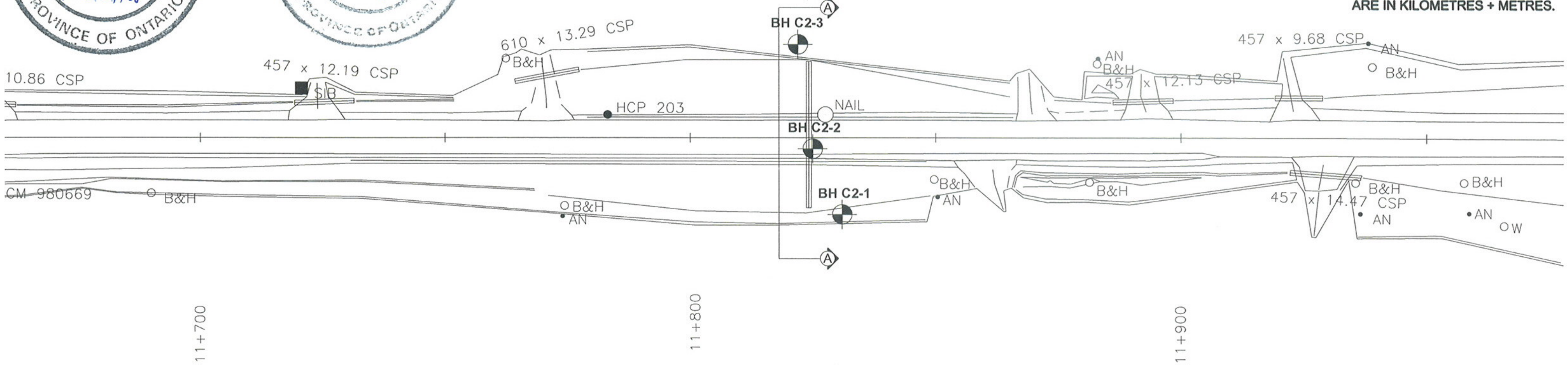
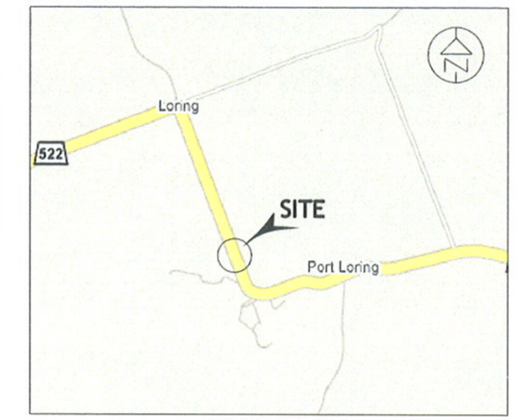
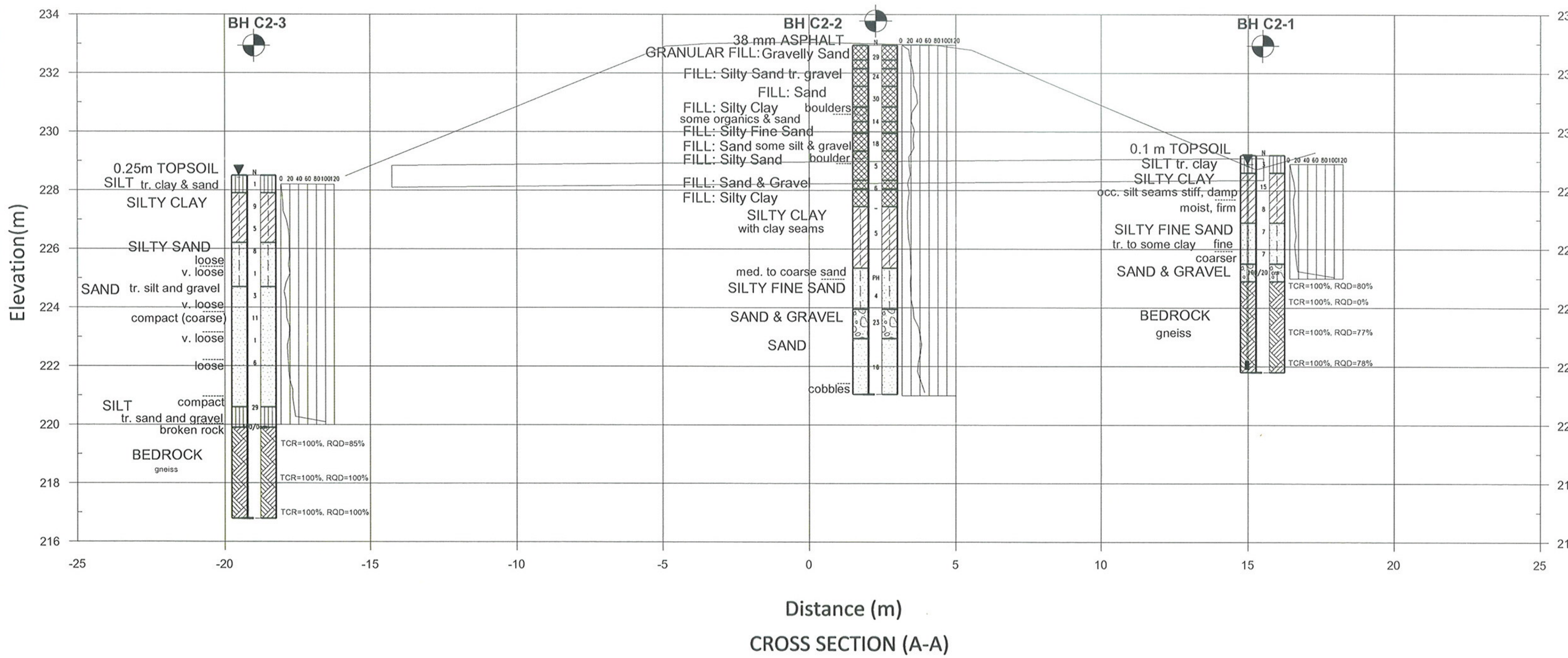
A Division of Coffey Geotechnics Inc.



BOREHOLE LOCATION PLAN



KEY PLAN
N.T.S



CROSS SECTION (A-A)

LEGEND

Borehole & Cone

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
C2-1	229.2 m	11+831	15.5 Rt C/L
C2-2	232.9 m	11+825	2.25 Rt C/L
C2-3	228.5 m	11+822	19.0 Rt C/L

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. 31E-280

SPT 1218B			DIST
SUBM'D	CHECKED	DATE Sept.2008	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1

Appendix A

Records of Borehole Sheets

RECORD OF BOREHOLE No C2-1

METRIC

GWP 480-98-00	LOCATION Sta 11+831, 15.5 m Rt C/L of Hwy 522		ORIGINATED BY RK
DIST ___ HWY 522	BOREHOLE TYPE Hollow Stem Auger		COMPILED BY SS
DATUM Geodetic	DATE 6/24/2008	6/26/2008	CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
229.2	GROUND SURFACE					
0.0 / 228.6	0.1 m TOPSOIL **SILT** tr. clay, v.loose, moist, dilatant	1	SS	3	229	
0.6	**SILTY CLAY** mottled brown, occ.clay seams occ. silt seams stiff, damp	2	SS	15	228	0 5 43 52
226.9	moist, firm	3	SS	8	227	
2.3	**SILTY FINE SAND** tr. to some clay, brown, wet, dilatant, loose fine	4	SS	7		0 46 37 17
225.5	coarser	5	SS	7	226	
3.7 / 224.9	**SAND & GRAVEL** brown, wet, v. dense	6	SS	100/20 c	225	
4.3	**BEDROCK** gneiss light to dark grey with pinkish grey bands sound	7	RC	TCR=100% RQD=80%		
		8	RC	TCR=100% RQD=0%	224	
		9	RC	TCR=100% RQD=77%	223	
221.8 / 7.4		10	RC	TCR=100% RQD=78%	222	

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

End of borehole
Water level in open hole @ 0.30 m upon completion (not stabilized)*
Piezometer installed to 7.3 m and
Piezometer reading
0.45 m - June 25, 2008
0.3 m - June 25, 2008
0.3 m - July 02, 2008
Dynamic Cone Penetration Test (DCPT) performed adjacent to the borehole from 0 to 4.2 m.

$+^3, \times^3$ Numbers refer to Sensitivity   15-5 (20/10) (%) STRAIN AT FAILURE

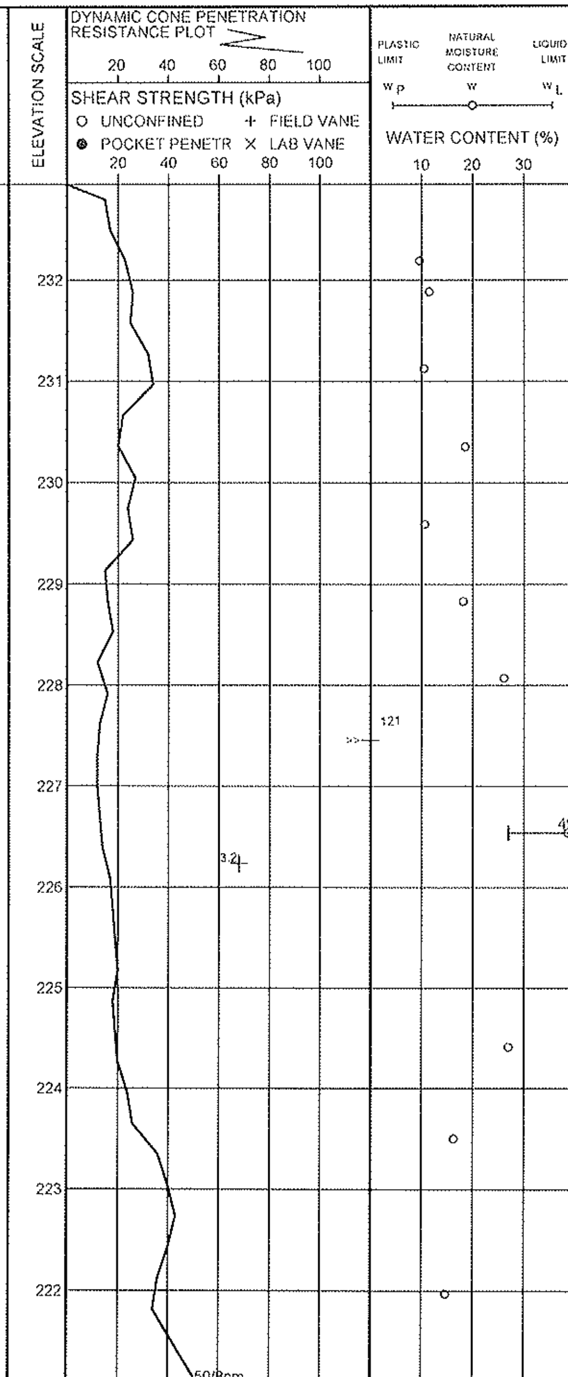
SPT1218B: Highway 522 (Port Loring)

RECORD OF BOREHOLE No C2-2

METRIC

GWP 480-98-00	LOCATION Sta : 11+825, 2.25 m Rt C/L of Hwy 522		ORIGINATED BY RK
DIST	HWY 522	BOREHOLE TYPE Hollow Stem Auger	COMPILED BY SS
DATUM Geodetic	DATE 6/19/2008		CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
232.9	GROUND SURFACE				
0.0	38 mm **ASPHALT** **GRANULAR FILL:** Gravelly Sand brown, compact, moist	1	SS	29	
232.4 0.5	**FILL:** Silty Sand tr gravel, brown, compact, moist				
232.1 0.8	**FILL:** Sand some silt with clay & clayey silt pockets/layers occ.clay, borwn, compact, damp	2	SS	24	Auger refusal @ 2.1 m, relocate borehole refusal again @ 2 1 m. Advance by wash boring.
231.5 1.4	**FILL:** Sand some silt & gravel, brown, compact, damp	3	SS	30	
230.8 2 1	**FILL:** Silty Clay boulders some organics & sand brown/grey stiff				
230.3 2.6	**FILL:** Silty Fine Sand brown, compact, moist	4	SS	14	1 72 (27)
229.9 3.0	**FILL:** Sand some silt & gravel brown, compact, moist	5	SS	18	
229.3 3.6	**FILL:** Silty Sand boulder tr. gravel, brown&reddish brown, loose, moist	6	SS	5	
228.3 4.6	**FILL:** Sand & Gravel	7	SS	6	45 47 (8)
228.0 4.9	**FILL:** Silty Clay mixed with organic clay to topsoil, tr. gravel brown/dark brown/black, firm				
227.4 5.5	**SILTY CLAY** with clay seams grey with reddish grey layers firm to stiff	8	AS	-	
		9	SS	5	0 4 52 44
225.3 7 6	med. to coarse sand **SILTY FINE SAND** brown, v loose, wet	10	TW	PH	
		11	SS	4	
223.9 9.0	**SAND & GRAVEL** brown, compact, wet	12	SS	23	
222.9 10.0	**SAND** brown, compact, wet cobbles	13	SS	16	
221.0 11 9	End of borehole. Possibly on bedrock.				



RECORD OF BOREHOLE No C2-3

METRIC

GWP 480-98-00 | LOCATION Sta : 11+822, 19.0 m Lt C/L of Hwy 522 | ORIGINATED BY SK

DIST ___ HWY 522 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY SS

DATUM Geodetic | DATE 6/25/2008 | CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
228.5	GROUND SURFACE					
0.0	0.25m **TOPSOIL** **SILT** tr. clay & sand brown, v. loose moist	1	SS	1		no sample recovery
227.9 / 0.6	**SILTY CLAY** greyish brown, firm to stiff, moist	2	SS	9	228	1 22 36 41
		3	SS	5	227	
226.3 / 2.2	**SILTY SAND** brown, wet; loose	4	SS	8	226	
	v. loose	5	SS	1	225	0 61 26 13
224.7 / 3.8	**SAND** tr. silt and gravel brown, wet; v. loose	6	SS	3	224	
	compact (coarse)	7	SS	11		1 96 (3)
	v. loose	8	SS	1	223	
	loose	9	SS	6	222	
	compact				221	
220.6 / 7.9	**SILT** tr. sand and gravel brown, compact, wet	10	SS	29		
	broken rock	~~11~~	~~SS~~	~~100/0cm~~	220	
219.9 / 8.6	**BEDROCK** gneiss light to dark grey with pinkish grey bands sound	12	RC	TCR=100% RQD=85%	219	
		13	RC	TCR=100% RQD=100%	218	
216.8 / 11.7		14	RC	TCR=100% RQD=100%	217	

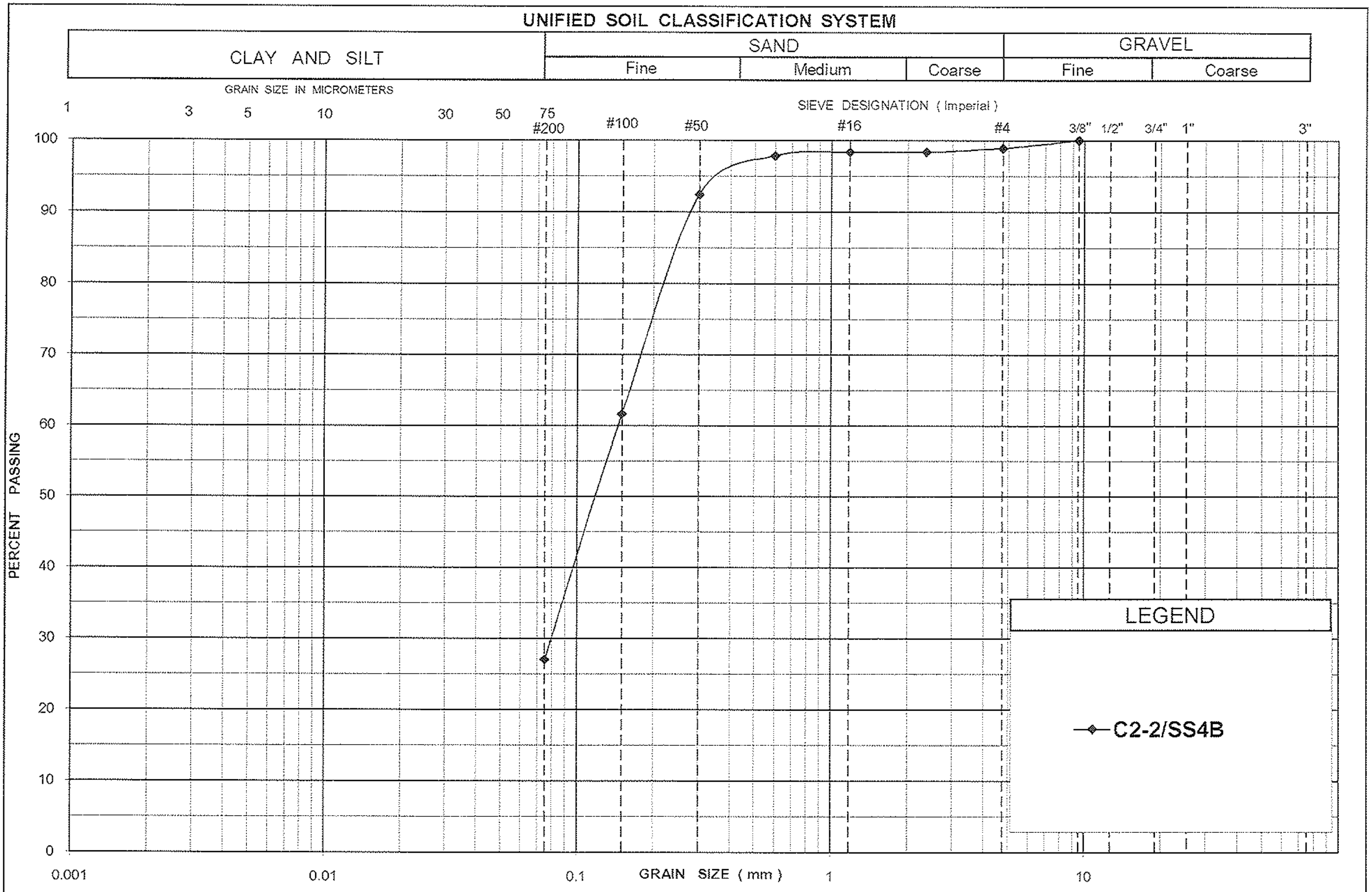
End of borehole.
*Water level at ground level upon completion.
Dynamic Cone Penetration Test (DCPT) performed adjacent to the borehole from 0 to 8.4 m.

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; 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

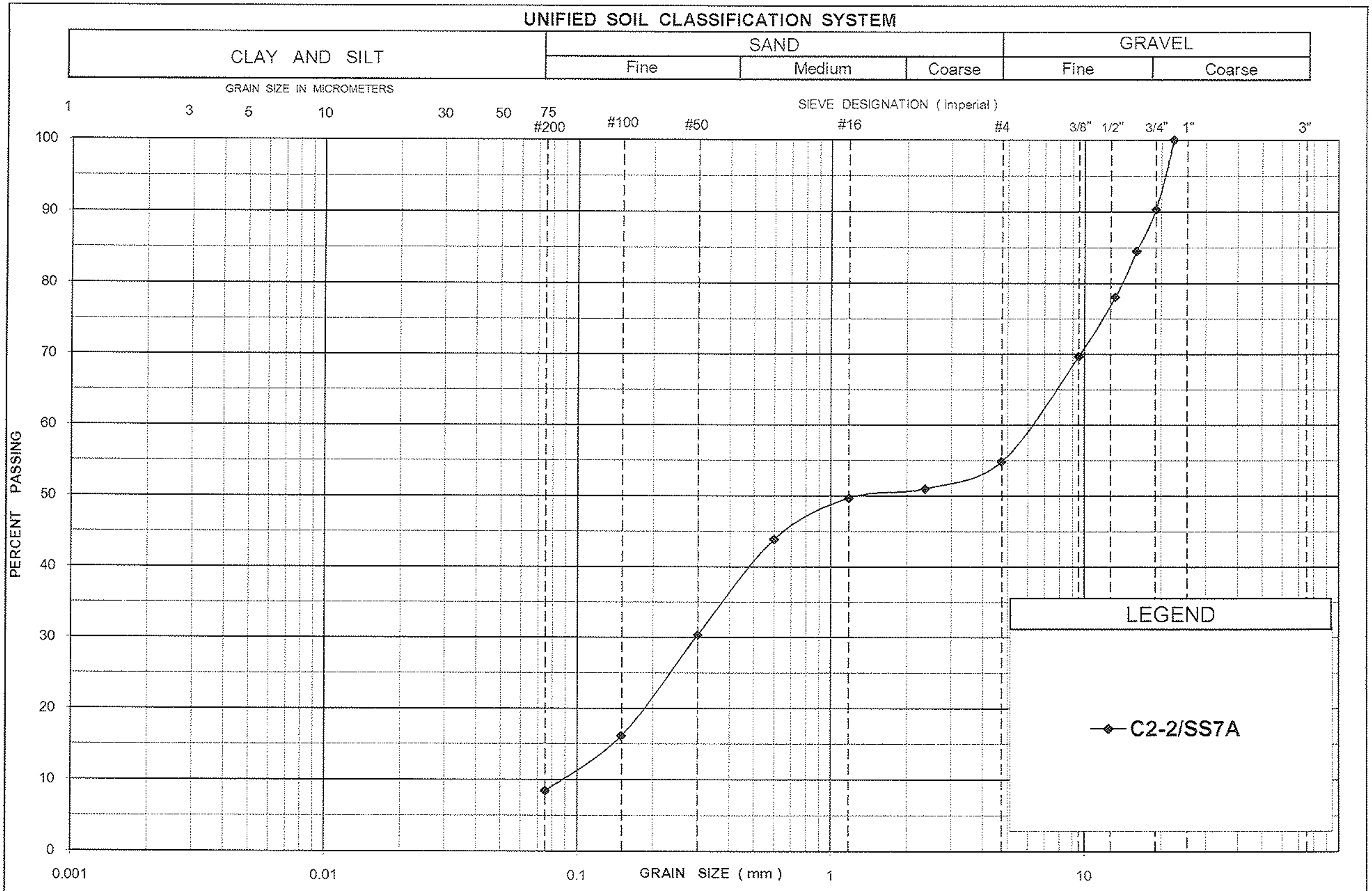
$+^3, \times^3$: Numbers refer to Sensitivity — (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results



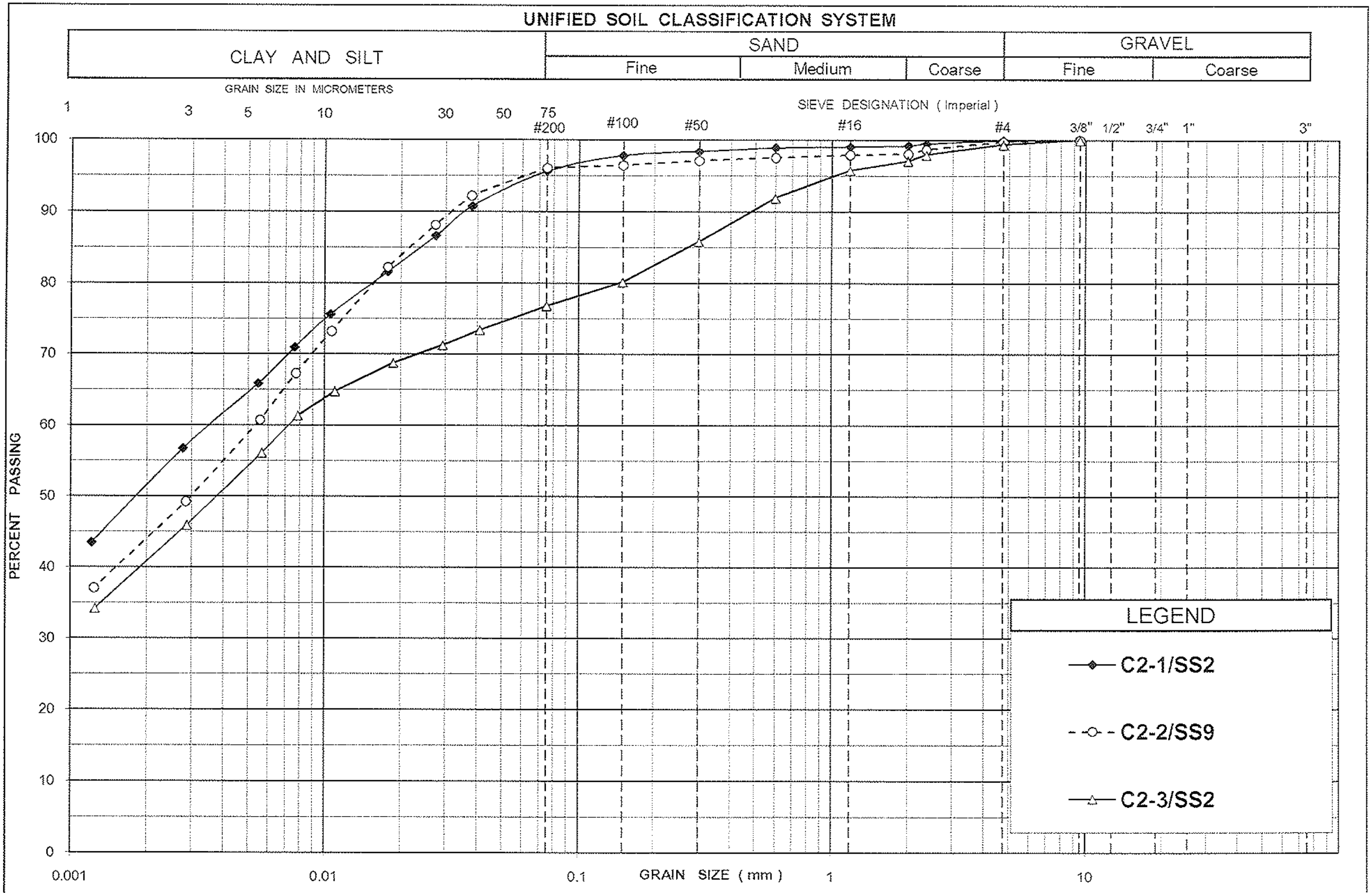
SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION FILL: Silty Fine Sand	FIGURE No. B-1 REF. No. SPT 1218B DATE SEPTEMBER 2008



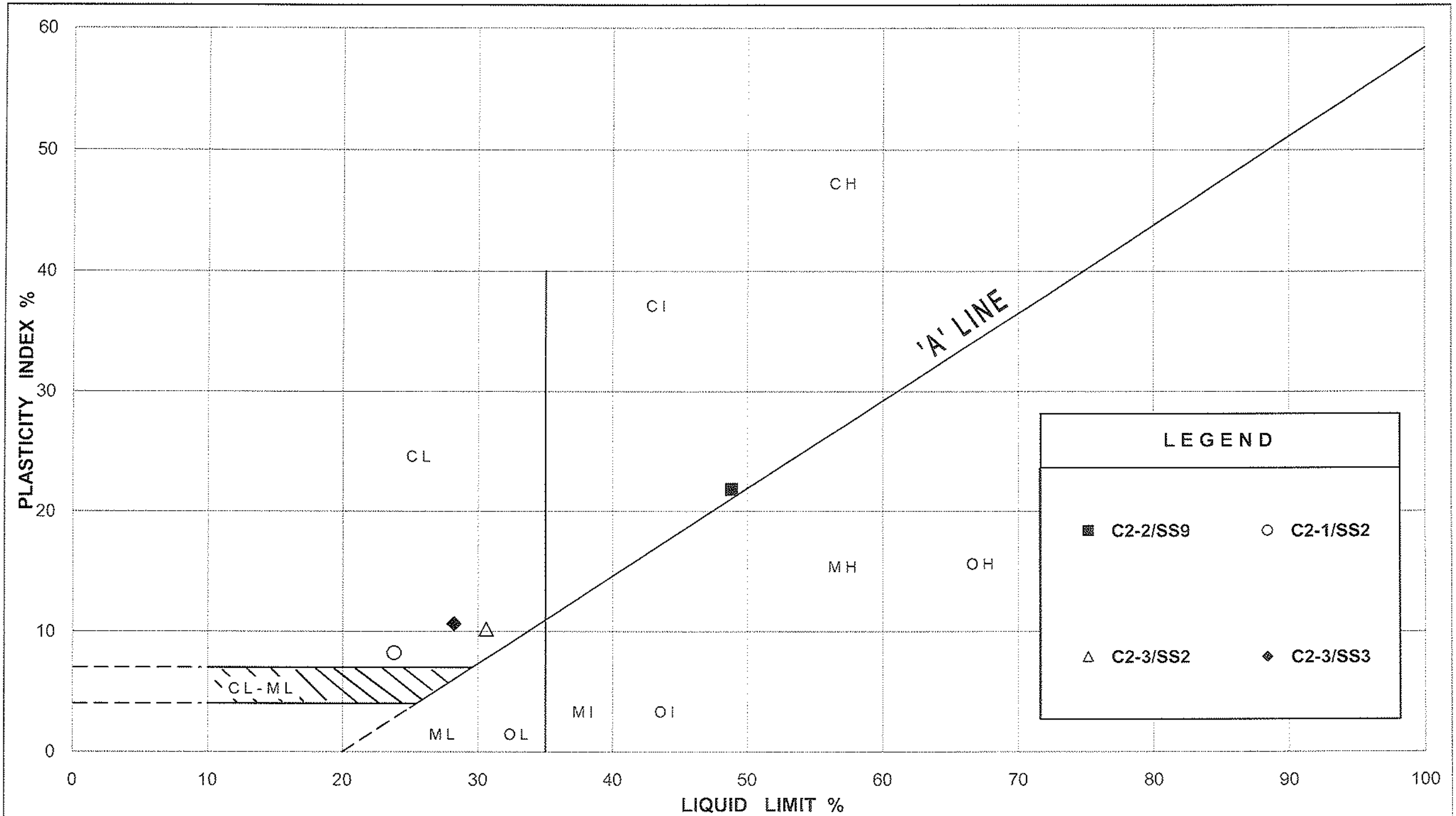
SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION
FILL: Sand & Gravel

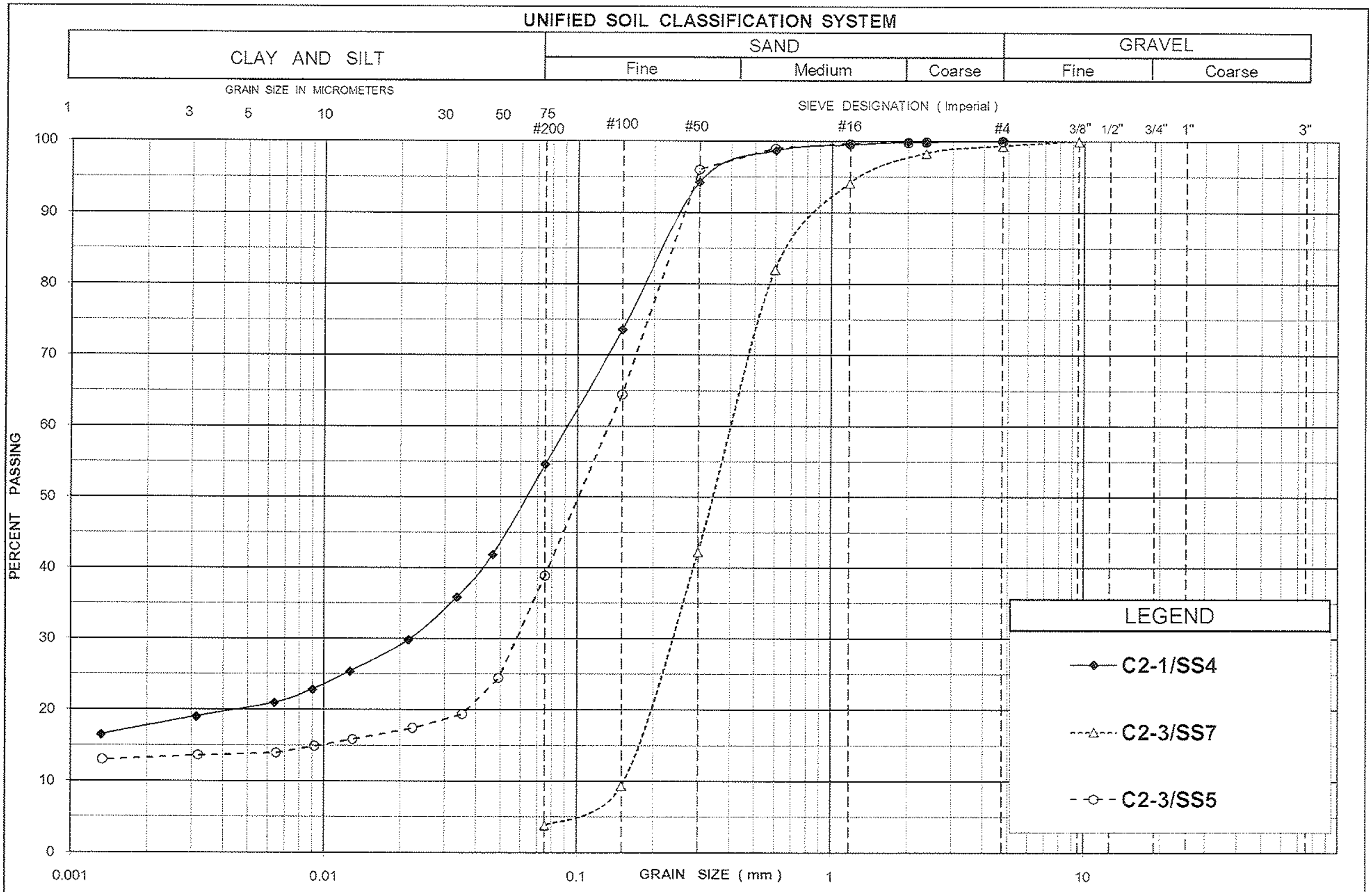
FIGURE No. B-2
REF. No. SPT 1218B
DATE SEPTEMBER 2008



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION SILTY CLAY	FIGURE No. B-3 REF. No. SPT 1218B DATE SEPTEMBER 2008



SHAHEEN & PEAKER	PLASTICITY CHART	FIGURE No. B-4
		REF. No. SPT 1218B
A Division of Coffey Geotechnics, Inc.	**SILTY CLAY**	DATE SEPTEMBER 2008



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION SILTY FINE SAND	FIGURE No. B-5 REF. No. SPT 1218B DATE SEPTEMBER 2008

Appendix C

Site Photographs

Photograph 1. Top of the embankment at culvert 7 location (looking south)

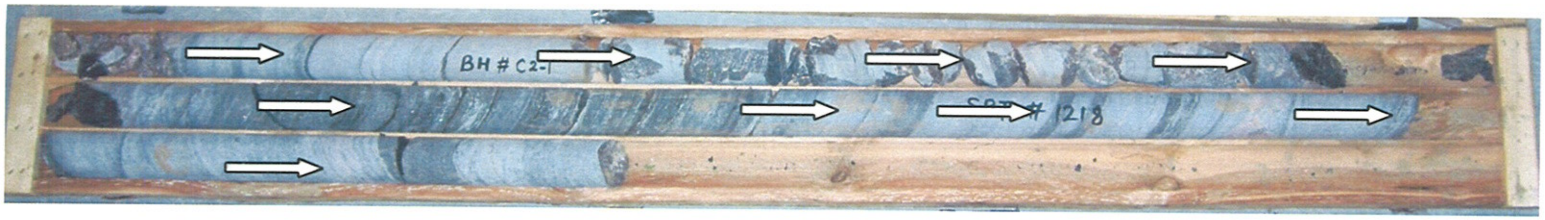
Photograph 2. Toe area of the embankment at culvert 7 location (east side)

Photograph 3. East side of embankment at culvert 7 location (looking south)

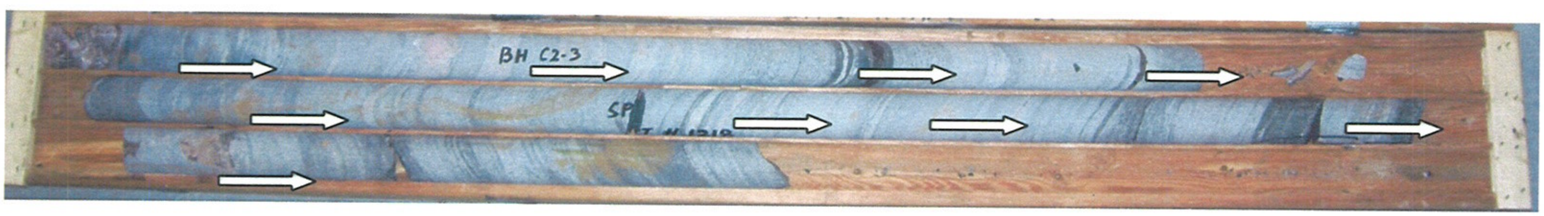
Photograph 4. Toe area of the embankment at culvert 7 location (west side)

Appendix D

Rock Core Photographs



C2-1



C2-3

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
PROPOSED CULVERT REPLACEMENT (C7) AT
STATION 11+824, HIGHWAY 522 REHABILITATION,
FROM 32.2 KM WEST OF HIGHWAY 524 EASTERLY 6 KM
G.W.P. 480-98-00, DISTRICT 54, SUDBURY
GEOCRES NO. 31E-280

Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.

Project: SPT1218B
November 27, 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

FOUNDATION DESIGN REPORT
PROPOSED CULVERT REPLACEMENT (C7) AT
STATION 11+824 ON HIGHWAY 522 REHABILITATION, FROM 32.2 KM.
WEST OF HIGHWAY 524 EASTERLY 6 KM.
G.W.P. 480-98-00, DISTRICT 54, SUDBURY

5. DISCUSSION AND RECOMMENDATIONS

5.1 PROJECT DESCRIPTION

The existing culvert under Highway 522 at Station 11+824 is a 762 mm x 29.79 m CSP. The invert of the pipe is at El. 228.4 m at the inlet and at El. 228.1 m at the outlet. It is our understanding that the existing culvert will be replaced with a new culvert matching the diameter and the invert elevations of the existing one while the length of the pipe will be about 30.7 m. The length of the pipe will be about 30.7 m. The road grade may be raised by up to about 100 mm due to pavement rehabilitation and there will be no widening of the roadway embankment.

The investigation at the site consisted of three boreholes. Boreholes C2-1 and C2-3 were put down at the inlet and outlet areas, respectively and Borehole C2-2 was drilled from the shoulder of the highway, immediately adjacent to the existing culvert.

Boreholes drilled from the toe of the embankment showed, underlying a thin veneer of topsoil, the presence of a surficial silt deposit to a depth of 0.6 m. The borehole drilled from the top of the highway embankment showed the presence of an embankment fill to a depth of 5.5 m. Underlying the surficial silt or the embankment fill all three boreholes contacted a 1.6 to 2.1 m thick deposit of firm to stiff silty clay. The silty clay is underlain by basal granular soils which extend to the surface of the bedrock/inferred bedrock at depths of about 4 to 9 m below the o.g. levels or at between El. 224.9 and 219.9 m.

At the time of our investigation, the groundwater level was recorded close to the o.g. levels, but can be expected to be subject to fluctuations.

5.2 CULVERT FOUNDATION SUPPORT

The pipe invert elevations for this project will be 228.4 m on the right side (inlet) and 228.1 m on the left (outlet). With an allowance of about 0.2 m for the granular bedding the anticipated subgrade soil elevations range from 228.2 to 227.9 m. The following table summarizes suitable subgrade elevations at the borehole locations, based on the borehole data.

Table 5.2.1
Highest Suitable Culvert Support Elevations

Borehole No.	Existing Ground Elevation (m)	Approximate Design Subgrade Elevation at Borehole Location (m)	Approximate Highest Suitable Subgrade Elevation at Borehole Location (m)	Anticipated Approximate Over-excavation
C2-1	229.2	228.2	228.4	None
C2-2	232.9	228.0	227.3	0.7
C2-3	228.5	227.9	227.7	0.2

From the above table, it appears that some sub-excavation of the unsuitable soils from under the pipe invert level and the replacement of the excavated soils with suitable granular soils will likely be required. The boreholes also indicate that the granular bedding materials will likely be placed on a relatively uniform subgrade of firm to stiff silty clay.

After sub-excavation to the suitable subgrade level or to the proposed (design) subgrade level, the exposed subgrade should be inspected, evaluated and approved by the Geotechnical Engineer appointed by the QVE.

As the silty clay is a relatively weak material, the use of a flexible culvert (i.e. CSP) is better suited for this project, rather than a rigid concrete structure. A flexible concrete pipe with short sections can also be considered but would, in our opinion, be less suitable than a CSP for the prevailing conditions.

Provided that all the unsuitable soils are removed, and where necessary replaced with suitable granular soils (where the grade needs to be raised after sub-excavation, e.g. probably at Borehole C2-2 and possibly at Borehole C2-3 location), there should be no problems with bearing resistance and settlements, since there will virtually be no load increases over and above the existing (i.e. no widening and only up to 100 mm grade raise of the road). However, for completeness the following geotechnical resistances are suggested for the undisturbed silty clay.

Bearing Resistance at U.L.S.	=	120 kPa
Factored Geotechnical Resistance at S.L.S.	=	70 kPa

Under the embankment, the value at SLS is less than the existing embankment loading. This however is not considered to be a problem since the overburden under the existing embankment would have fully consolidated/settled under the existing embankment loads. As in this present case, there will be little or no additional loading, there should be negligible additional settlements. However, a settlement of about 25 mm should be allowed for, due to slight increased load for pavement rehabilitation and soil exchange as well as for rebound during construction (i.e. the embankment will be excavated) and re-settlement after backfilling. Based on this, it is our opinion that cambering is not required.

5.3 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) for flexible pipes or OPSD-802.030, 031, 032 or 034 (for rigid pipes) and should consist of not less than 250 mm thick layer (after compaction) of approved granular material, such as Granular 'B' Type II or Granular 'A' (Granular 'B' Type II is preferred under the pipe.) Under the pipe, 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 95% of the material's Standard Proctor Maximum Dry Density (SPMDD) using a suitably light compactor to ensure that the underlying subgrade is undisturbed. 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 sides of the excavation, as well as the top of the bedding material. However, the use of a poorly graded bedding is not recommended for this project.

5.4 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 for flexible pipes or appropriate standards for rigid pipes. 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 should be compacted to at least 96% of the material's SPMDD. The Granular 'A' base and the Granular 'B' sub-base courses should be compacted to 100% of the SPMDD. The fill should be placed simultaneously on each side of the pipe to prevent lateral dislocation of the pipe. Uplift of the pipe must be prevented by means of dewatering and/or placing sufficient fill above it.

We would like to point out that the performance of flexible pipe culverts is largely dependent on the side support provided by the bedding and the adjacent soils. The use of proper bedding and backfill materials and especially good compaction are, therefore, necessary for proper side support. The use of heavy compaction equipment should, however, be avoided immediately adjacent and above the pipes, 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.

The use of vibratory compaction equipment behind the culvert should be restricted in size as per current MTO practice.

5.5 CONSTRUCTION

Based on the information provided to us by D.M. Wills Associates, Highway 522 at the project site will be completely closed without any detour or roadway protection during the culvert replacement. The construction will be carried out without shoring.

The flow of water in the existing watercourse will need to be maintained during the construction. This can be achieved by placing a temporary pipe for the construction period or using the existing culvert for this purpose until the new culvert is built.

Depending on the groundwater level encountered at the time of the construction, some form of dewatering will likely be required to facilitate the construction and to preserve the load carrying capability of the founding soils. Based on the borehole data, excavations to the bottom of the bedding for the pipe will extend into the silty clay deposit. Since the silty clay is a relatively impervious material, it is believed that a sufficiently dry working condition can be created by means of gravity drainage and pumping from strategically placed sumps.

We recommend that the contractor be made aware of possible dewaterinig 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 necessity for dewatering, and the methods that may be required for this purpose.

Care should be taken to avoid disturbing subgrade soils by minimizing construction traffic (including foot traffic) and minimizing vibrations. As well, stripping should be carried out under geotechnical supervision to acceptable subgrade level. The bedding material and/or the granular soil to raise the grade should be placed immediately after exposing the suitable subgrade, its inspection and approval. We recommend that the material placed above the approved subgrade to raise the grade and/or as bedding consist of Granular ‘B’ Type II material. Where the subgrade is relatively weak, we recommend that the Granular ‘B’ Type II be pushed into the inorganic subgrade, if necessary, in order to improve the subgrade to make it firmer. As well, where subgrade is relatively weak, the first lift of backfill may need to be up to 0.6 m thick.

The contractor should also be made aware of the presence of cobbles and boulders in the embankment fill and their possible presence in the underlying overburden.

We recommend that the contractor be alerted by means of an NSSP that special care is needed to avoid disturbing the founding soils. As well, the contractor should be required to submit their dewatering and excavation proposal to the CA for information purposes.

The construction of the culvert should be in accordance with OPSS 421.

All excavations should be carried out in accordance with the Province’s Occupational Health and Safety Act (OHSA), O. Reg. 213/91, as well as the following:

SP 105 S19 – Protection Systems
SP 902 S01 – Excavation and Backfilling - Structures

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

Granular Pavement Fill	Type 3 soil
Embankment Fill	Type 3 soil above water level
	Type 4 soil below water level
Topsoil	Type 4 soil below water level
Silty Clay	Type 3 soil above water level
	Type 4 soil below water level
Basal Granular Soils	Type 4 soil below water level

Regardless of the classification given above, we recommend that side slopes above water level for temporary excavations with unsupported side slopes should be no steeper than 2H:1V. This can be steepened, if approved by the QEV, but no steeper than 1 1/2H:1V. Below water level (e.g. if the site was not properly dewatered), flatter side slopes would be required.

5.6 EROSION PROTECTION

Erosion and scour protection should be provided at the culvert inlet and outlet (including the side slopes). 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.

We recommend that a 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 cut-off walls should extend to a suitable depth (i.e. below any possible scour depth).

Based on the available borehole data, the soil at both the inlet and outlet will likely consist of highly erodible silt and silty clay (which is not highly erodible. – i.e. less erodible than the overlying silt).

5.7 BEARING SURFACES

We recommend that all bearing surfaces should be inspected and approved by a qualified Geotechnical Engineer (QVE).

5.8 FROST PROTECTION

Design frost protection for the project area is 1.8 m. Therefore, a permanent soil cover of 1.8 m or its thermal equivalent of artificial insulation is required for frost protection of foundations. 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 culvert 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

Zuhtu S. Ozden, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Ramon Miranda, P. Eng.

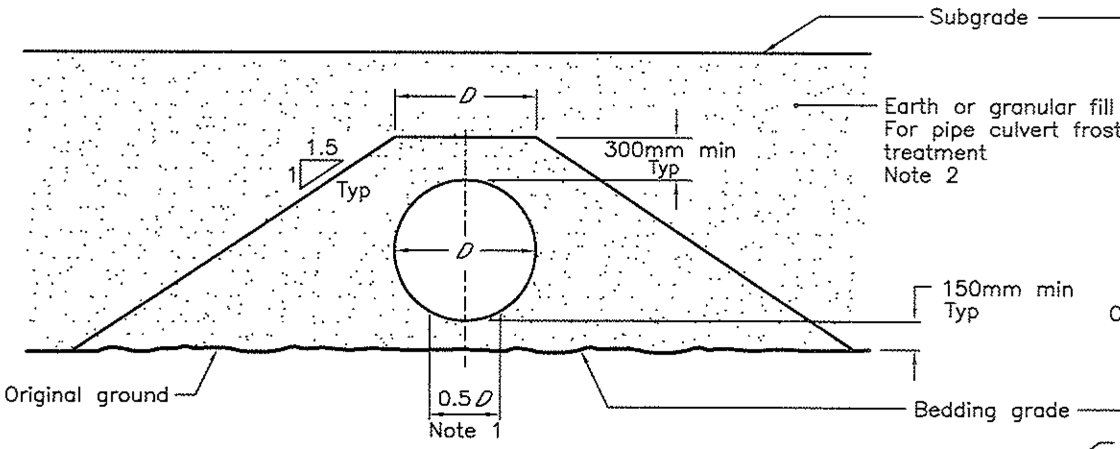
ZO:tr/idrive

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

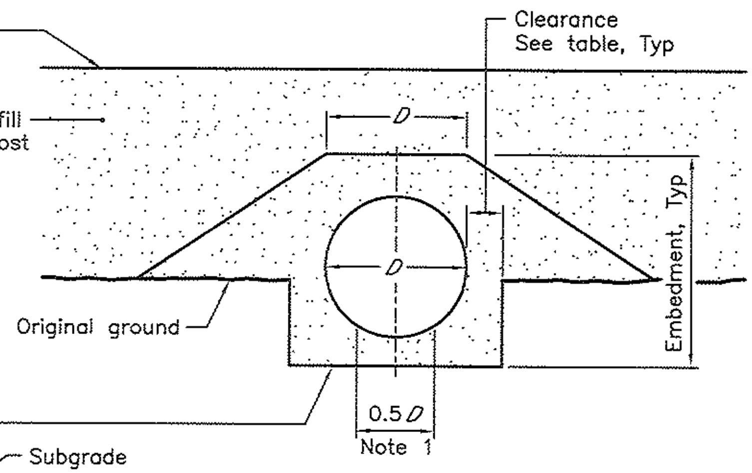
LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

Appendix F

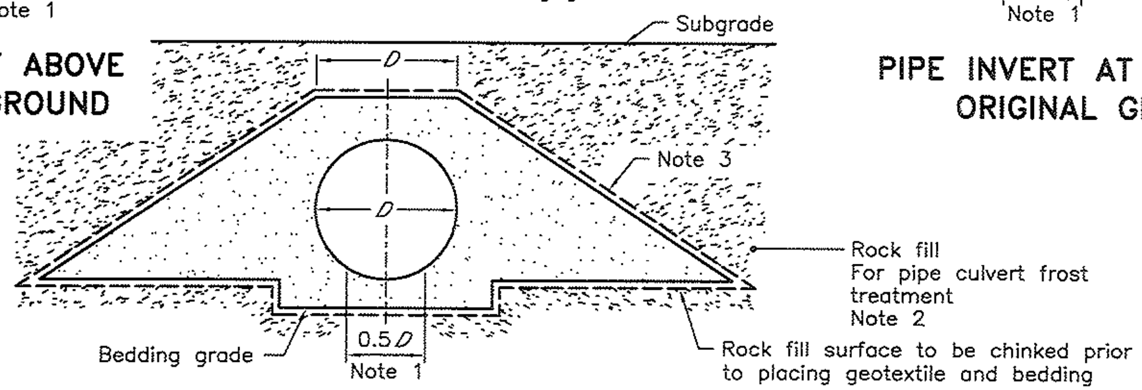
OPSD



PIPE INVERT ABOVE
ORIGINAL GROUND



PIPE INVERT AT OR BELOW
ORIGINAL GROUND



PIPE EMBEDMENT
WITH ROCK FILL UNDER AND OVER THE PIPE

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.

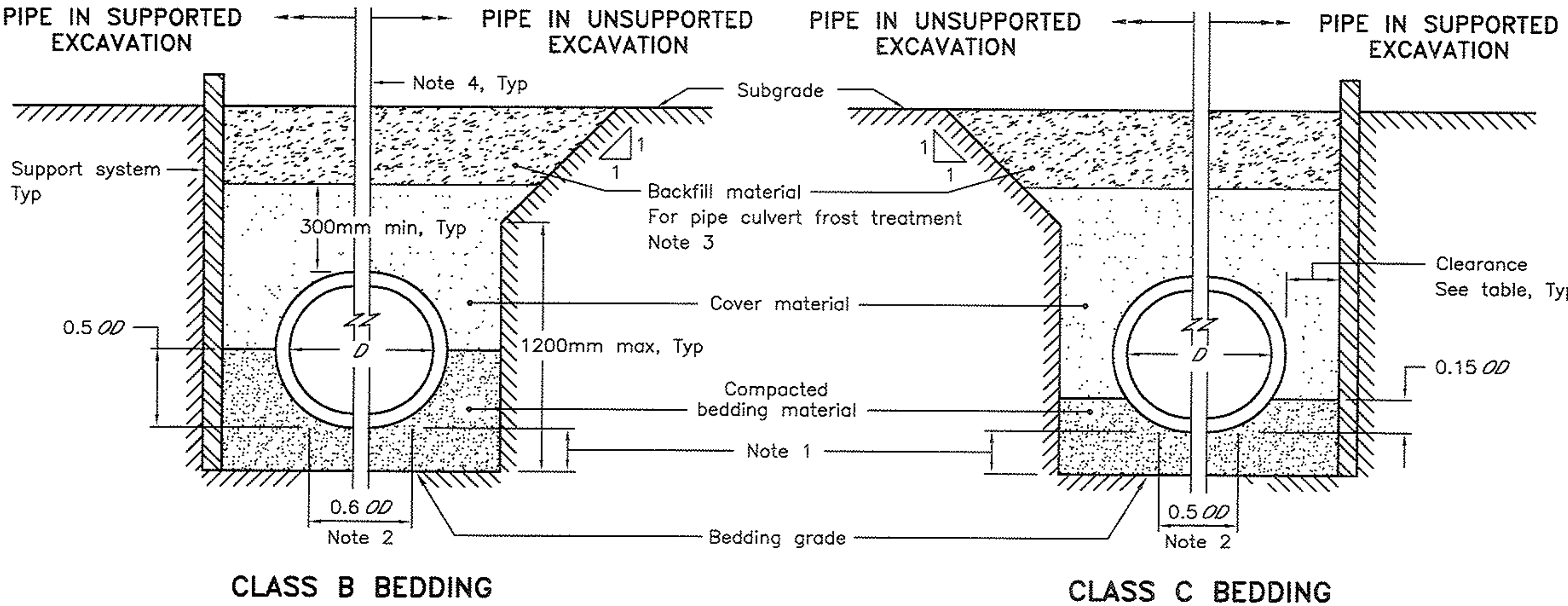
CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 Rev 1

FLEXIBLE PIPE EMBEDMENT IN EMBANKMENT ORIGINAL GROUND: EARTH OR ROCK

OPSD – 802.014



NOTES:

1 The minimum bedding depth below the pipe shall be 0.15*D*. In no case shall this dimension be less than 150mm or greater than 300mm.

2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.

3 Pipe culvert frost treatment according to OPSD–803.030 and 803.031.

4 Condition of trench is symmetrical about centreline of pipe.

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

B All dimensions are in metres unless otherwise shown.

LEGEND:

D – Inside diameter

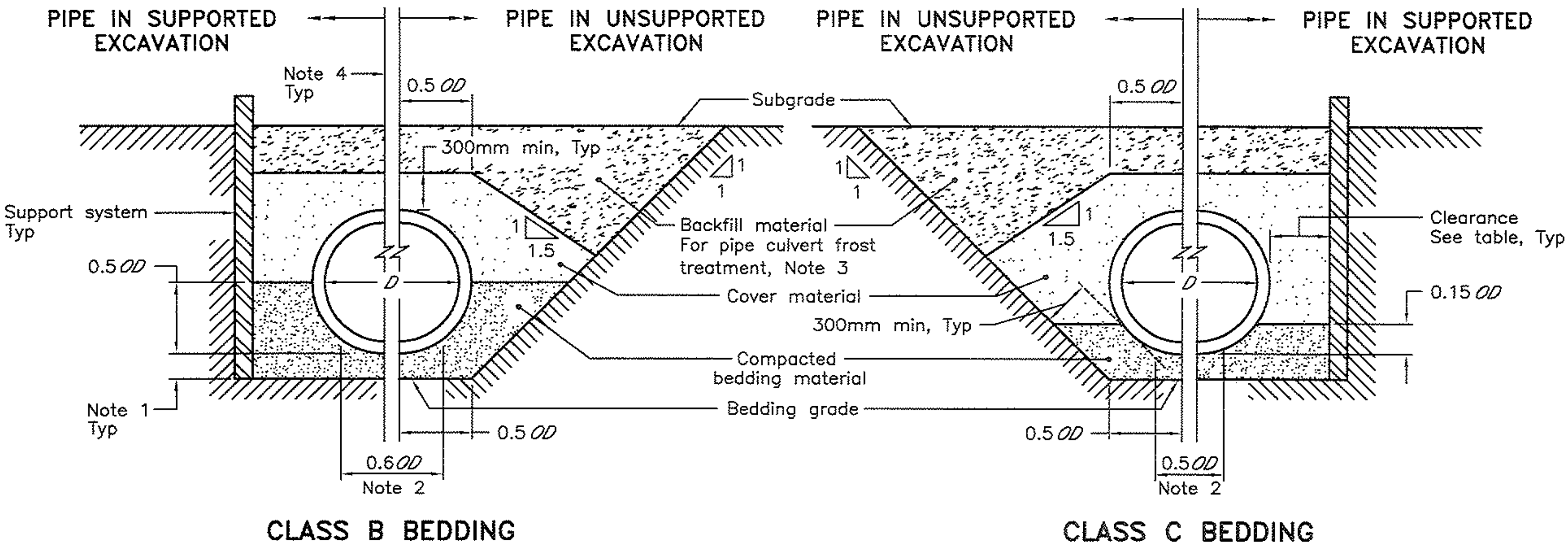
OD – Outside diameter

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

ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2005 | Rev 1

RIGID PIPE BEDDING, COVER, AND BACKFILL TYPE 1 OR 2 SOIL – EARTH EXCAVATION

OPSD – 802.030



NOTES:

1 The minimum bedding depth below the pipe shall be 0.15*D*. In no case shall this dimension be less than 150mm or greater than 300mm.

2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.

3 Pipe culvert frost treatment according to OPSD–803.030 and 803.031.

4 Condition of trench is symmetrical about centreline of pipe.

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

B All dimensions are in metres unless otherwise shown.

LEGEND:

D – Inside diameter

OD – Outside diameter

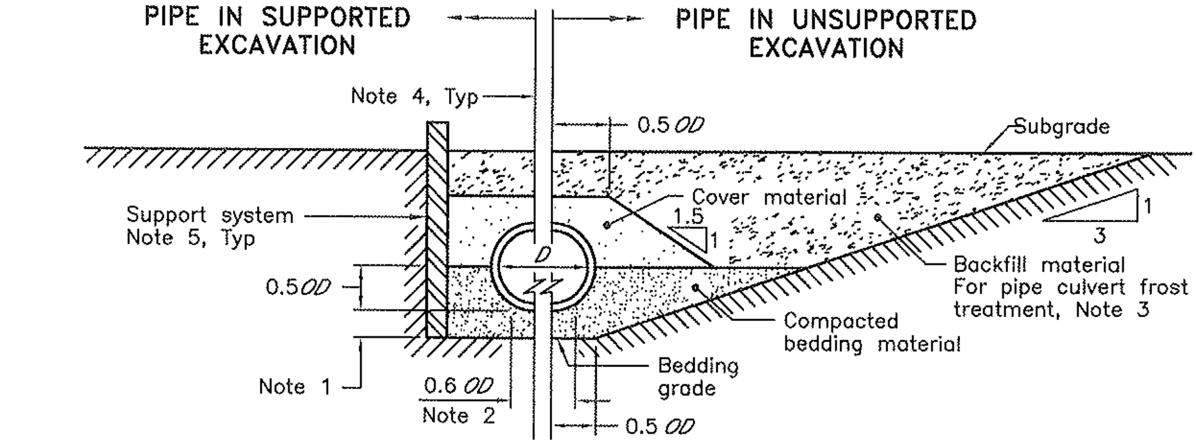
CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

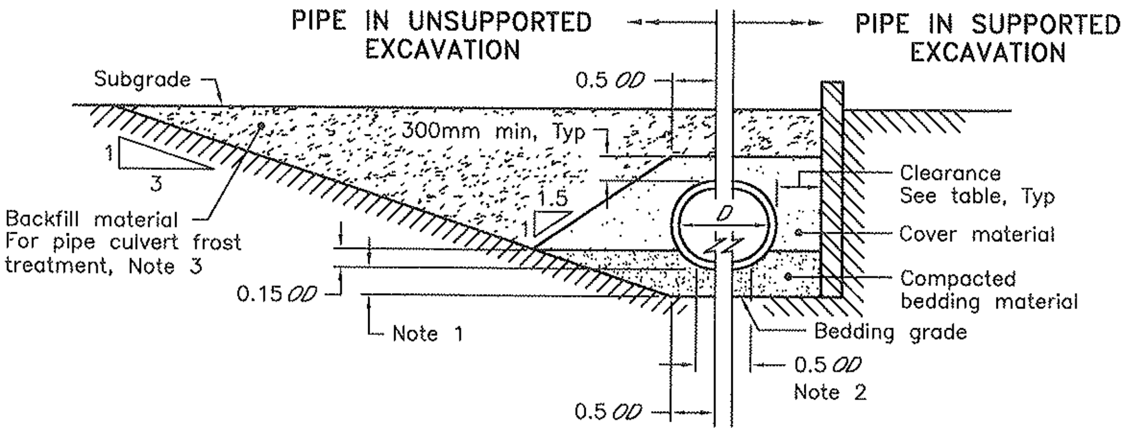
Nov 2005 Rev 1

RIGID PIPE BEDDING, COVER, AND BACKFILL TYPE 3 SOIL – EARTH EXCAVATION

OPSD – 802.031



CLASS B BEDDING



CLASS C BEDDING

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

LEGEND:

D – Inside diameter

OD – Outside diameter

NOTES:

1 The minimum bedding depth below the pipe shall be 0.15*D*. In no case shall this dimension be less than 150mm or greater than 300mm.

2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.

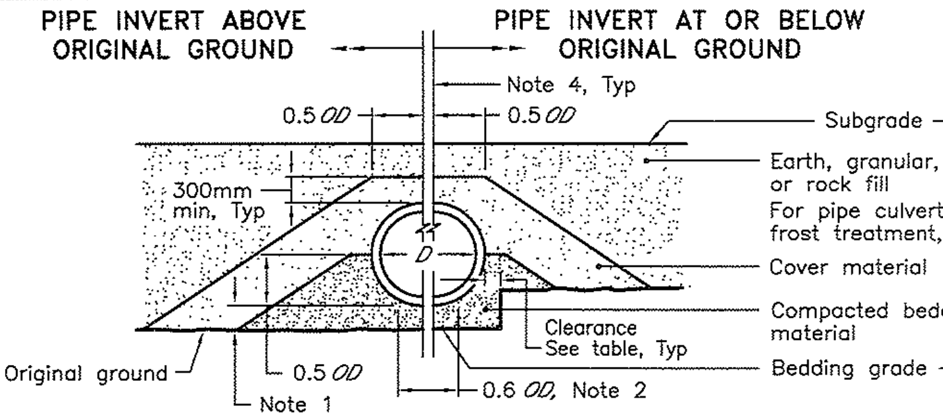
3 Pipe culvert frost treatment according to OPSD–803.030 and 803.031.

4 Condition of trench is symmetrical about centreline of pipe.

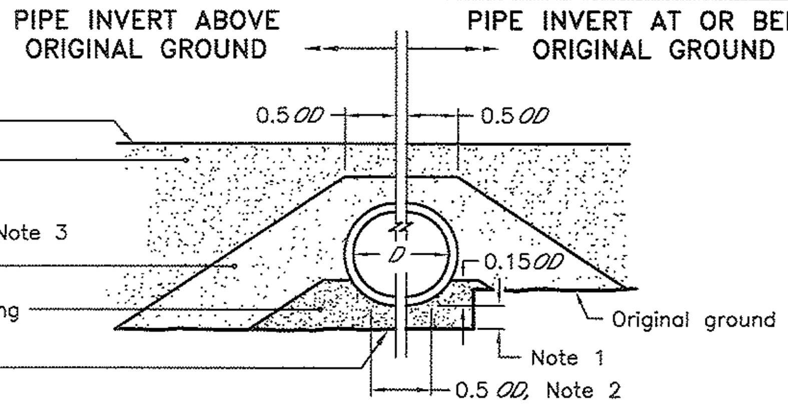
A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.

B All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2005	Rev 1
RIGID PIPE BEDDING, COVER, AND BACKFILL TYPE 4 SOIL – EARTH EXCAVATION	OPSD – 802.032	

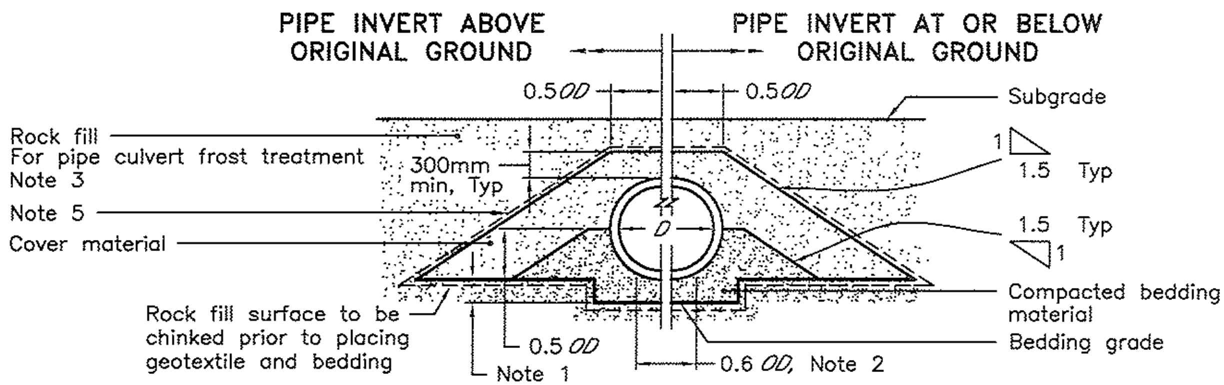


CLASS B BEDDING



CLASS C BEDDING

EARTH AND ROCK EXCAVATION



PIPE BEDDING AND COVER
WITH ROCK FILL UNDER AND OVER THE PIPE

NOTES:

1 The minimum bedding depth below the pipe shall be $0.15D$, except on a rock foundation where the minimum bedding depth shall be $0.25D$. In no case shall the minimum dimension be less than 150mm or the maximum dimension exceed 300mm.
2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
3 Pipe culvert frost treatment according to OPSD–803.030 and 803.031.
4 Condition of trench is symmetrical about centreline of pipe.
5 Bedding and cover material to be wrapped in non–woven geotextile when specified.

A All dimensions are in metres unless otherwise shown.

LEGEND:

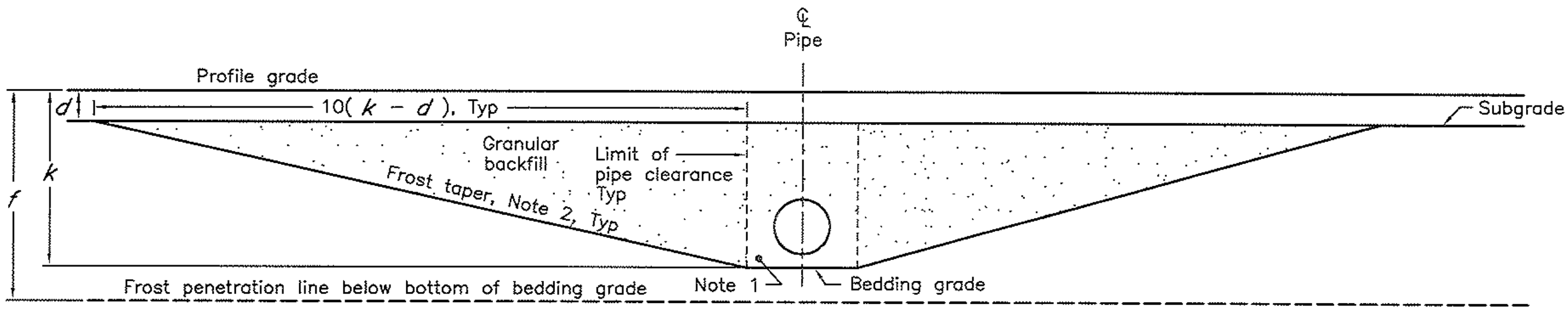
D – Inside diameter
OD – Outside diameter

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

ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2005 | Rev 1

RIGID PIPE BEDDING AND COVER IN EMBANKMENT ORIGINAL GROUND: EARTH OR ROCK

OPSD – 802.034



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.

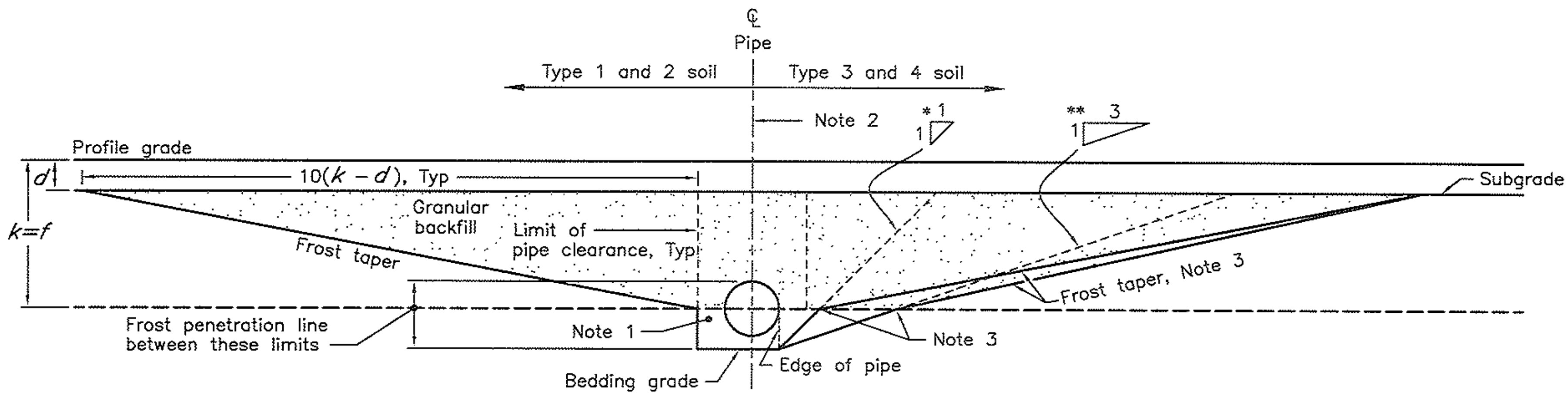
2 Frost tapers start at bedding grade.

A Frost tapers are not required in rock embankment.

LEGEND:

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

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2005	Rev 1
FROST TREATMENT – PIPE CULVERTS FROST PENETRATION LINE BELOW BEDDING GRADE		
	OPSD – 803.030	



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 Condition of frost treatment symmetrical about centreline of pipe.

3 Frost tapers start at the intersection of the 1H:1V or 3H:1V slope and the frost penetration line.

A Frost tapers are not required in rock embankment.

B Frost tapers not required when frost line is above the top of pipe.

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

LEGEND:

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

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2005	Rev 2
FROST TREATMENT – PIPE CULVERTS FROST PENETRATION LINE BETWEEN TOP OF PIPE AND BEDDING GRADE		
	OPSD – 803.031	

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.