

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
PROPOSED REPLACEMENT OF BEAUDRY
CREEK CULVERT ON HIGHWAY 534,
TOWNSHIP OF GURD, ONTARIO.
DISTRICT 54, SITE # 44-263/C,
G.W.P # 5053-05-00,
GEOCRES NO. 31L-136**

D.M. Wills Associates Limited

TRANETOB01238AA
December 17, 2009

December 17, 2009

D.M. Wills Associates Limited
452 Charlotte Street
Peterborough, Ontario
K9J 2W3

Attention: Mr. Michael Lang, P.Eng.

Dear Sir:

**RE: Foundation Investigation and Design Report, Proposed Replacement of Beaudry Creek
Culvert on Highway 534, Township of Gurd, Ontario, District 54, Site # 44-263/C,
G.W.P #5053-05-00, Geocres No. 31L-136**

Please find attached the results of our draft geotechnical investigation and design report relating to the above noted site.

If you have any comments or enquiries please contact the undersigned.

For and on behalf of Coffey Geotechnics Inc.



Ramon Miranda, P.Eng.
Manager, Transportation Division

Attachment A: Attachments

**FOUNDATION INVESTIGATION REPORT
PROPOSED REPLACEMENT OF BEAUDRY
CREEK CULVERT ON HIGHWAY 534,
TOWNSHIP OF GURD, ONTARIO.
DISTRICT 54, SITE # 44-263/C,
G.W.P # 5053-05-00,
GEOCRES NO. 31L-136**

D.M. Wills Associates Limited

Project: TRANETOB01238AA
December 17, 2009

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**FOUNDATION INVESTIGATION REPORT
BEAUDRY CREEK CULVERT REPLACEMENT
TOWNSHIP OF GURD, ONTARIO
G.W.P. # 5053-05-00**

1 INTRODUCTION

Coffey Geotechnics Inc (Coffey) was retained by D.M. Wills Associates Limited (Wills) to carry out a foundation investigation at the site of the Beaudry Creek culvert replacement (Site # 44-263/C) under Highway 534, in Township of Gurd, 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 on Highway 534, approximately 1.8 km east of Highway 524 in Township of Gurd as shown in Drawing No. 1. Topography at the culvert site is gently rolling.

According to the Quaternary Geology of Ontario Map M2556 (Ministry of Northern Development and Mines, Ontario), and the Quaternary Geology of South River Area Map P3160 (Ontario Geological Survey, 1990), the site is in an area of glaciofluvial outwash deposits, consisting of gravel, sand, minor till, esker, kame, moraine, ice-marginal delta and subaqueous fan deposits. Generally, after the last glacial withdrawal, glacial Lake Algonquin deposited glaciolacustrine (silts and clays) over the sandy glacial till. The glaciolacustrine deposits were then overlain by glaciofluvial sediments (sands, gravels and silts). Local creeks and rivers then bisected the glaciofluvial soils depositing sands, gravels, organic materials and muck.

According to the Bedrock Geology of Ontario Map 2544 (Ministry of Northern Development and Mines, Ontario), the bedrock underlying the site consists of strongly foliated gneisses to migmatic rocks of the Central Gneiss Belt, which is part of the Grenville Province (a structural subdivision of the Canadian Shield).

The existing embankments close to the existing twin culverts, do not exhibit any apparent signs of slope instability or excessive erosion. As well, in the immediate vicinity of the existing twin culverts, there are no signs of excessive settlements/unusual cracking or deformations in the pavement.

3 FIELD AND LABORATORY WORK

The fieldwork for this project was performed on April 28, 29, 30 and May 6, 2009 and consisted of drilling and sampling three boreholes (Boreholes C1, C2 and C3), to depths of 7.8 to 16.8 m below the existing grades. The locations of the boreholes at the site are given on the Borehole Location Plan, Drawing No. 1.

The boreholes were advanced using a track-mounted drilling rig owned and operated by Landcore Drilling of Chelmsford, Ontario, under the full-time supervision of technical personnel from Coffey. The boreholes

were advanced using three different methods (i.e. continuous flight hollow-stem augers, wash boring in the overburden and rock coring) depending on the ground conditions.

Samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. The test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS-split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils). Several thin walled Shelby tube samples were also obtained in the cohesive soils.

In addition to SPT, where the consistency permitted, field vane tests were performed using MTO type vanes to measure the undrained shear strength of the soil in-situ.

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

The borehole locations were established in the field by Coffey 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/casings. Upon their completion, the boreholes were grouted using a cement/bentonite mixture as per MTO procedures. A standpipe piezometer was installed in Borehole C2 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

Boreholes C1, C2 and C3 were advanced at the culvert site. Borehole C2 was put down 10.5 m left (outlet, west) of the centerline of the Highway, near the toe of the embankment from the o.g. (original ground) level (El. 226.4 m), while Borehole C3 was advanced from the other (inlet, east) side of the road, 14.5 m right of the centerline from o.g. level at El. 226.7 m. Borehole C1 was drilled from the shoulder of the roadway, 5.0 m right of the centerline (El. 228.8 m). The height of the embankment at this location is about 2 to 3.5 m.

Borehole C1, drilled from the top of road embankment, contacted embankment fill to a depth of 3.5 m to El. 225.3 m, underlain by a 1.1 m thick silt deposit. In Boreholes C2 and C3, a similar silt deposit was contacted immediately below the topsoil (El. 226.2 and 226.5 m). The silt was found to extend to El. 221.2-224.6 m (i.e. 1.1 to 5.0 m thick) and are underlain in all boreholes by a clayey silt deposit. This clayey silt was found to be 4.2 to 4.6 m thick and is in turn underlain by a sand deposit to the surface of the bedrock in Boreholes C1 and C2 at depths of 13.8 and 12.8 m or at El. 215.0 and 213.6 m. Borehole C3 was terminated in the sand deposit.

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 Drawing No. 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 Fill

Borehole C1 drilled from the shoulder of Highway 534 contacted an about 0.2 m sand and gravel pavement shoulder granular fill, followed by an about 3.3 m thick sand embankment fill. The top 1.9 m of the sand fill contains traces of gravel.

The grain-size distribution of two samples from the fill is given in Figure B-1, in Appendix B. This indicates the following grain-size distribution.

Gravel: 4 – 12 %

Sand: 73 – 81 %

Silt and Clay: 15%

This fill is a granular (i.e. non cohesive) soil. Standard Penetration Tests performed the fill yielded N-values of 2 to 10 blows/0.3 m. These results indicate that the relative density of the fill can be described to a depth of 2.1 m as loose, followed by a very loose condition below this depth. These results indicate that the embankment fill has not received a systematic compaction when it was first placed.

4.2 Topsoil

A layer of topsoil, about 0.2 m thick, was encountered in Boreholes C2 and C3 at ground surface.

4.3 Silt

Underlying the embankment fill in Borehole C1 and the topsoil in Boreholes C2 and C3, a silt layer containing some clay and traces of sand, organics and peat was encountered. The silt deposit extended to depths of 2.1 to 5.2 m or to El. 224.6 to 221.2 m.

The grain-size distribution of five samples from the deposit is given in Figure B-2. The curves show the following grain-size distribution:

Gravel: 0 %

Sand: 4 – 21%

Silt: 70 – 85%

Clay: 8 – 18%

Atterberg limits tests were carried out on five samples from this deposit. Three sample were identified as non-plastic. The results are given on the individual Record of Borehole Sheets and the test results for the

two samples which showed some plasticity are also given in Figure B-3 in Appendix B. These show the following index values:

Liquid Limit: 18 – 20%

Plastic Limit: 15 – 16%

Plasticity Index: 2 – 5%

This soil unit can be mostly considered as non-plastic and a basically non-cohesive (i.e. fine grained granular) material.

Standard Penetration Tests performed in this basically cohesionless soil deposit yielded N-values of between 0 and 6 blows/0.3 m. These results indicate that the relative density of the deposit can be described as very loose to loose, but typically very loose.

4.4 Clayey Silt

Underlying the silt layer, all boreholes contacted a major clayey silt deposit at depths of 2.1 to 5.2 m or El. 224.6 to 221.2 m. The deposit extended to depths of 6.7 to 9.4 m below the ground surface or to El. 220.0 to 217.0 m. A visual excavation of the soil samples showed that the material has a complex structure with frequent silty clay to silt and fine sand seams. It should be also noted that the relatively coarser zones of the deposit (i.e. silt, fine sand and clayey silt) exhibit a dilatent behaviour.

The grain-size distribution of two samples from the top portion of this deposit is given in Figure B-4, in Appendix B. This indicates the following grain-size distribution.

Sand: 3 – 7%

Silt: 63 – 70%

Clay: 27 – 30 %

Atterberg limits tests were performed on five samples from this deposit which yielded the following index values (see Figure B-5, Appendix B):

Liquid Limit: 28 – 32%

Plastic Limit: 16 – 18 %

Plasticity Index: 12 – 15 %

These values are the characteristic of clayey soils of low plasticity, as shown in the Figure B-5 in Appendix B. In most cases, the measured natural moisture contents are in excess of the measured liquid limit values indicating a relatively weak material.

Standard Penetration tests performed in this basically cohesive soil deposit yielded N-values of 0 to 6 blows/0.3 m. The undrained in-situ shear strength of the deposit was measured in the field by means of field vane tests, using MTO type field vanes due to the predominant cohesive nature of this deposit. The measured values range from 20 to 65 kPa. The field test results indicate a soft to stiff consistency, but typically soft to firm.

4.5 Basal Sand Deposit

Underlying the clayey silt, all three boreholes contacted a basal granular deposit. Boreholes C1 and C2 encountered a 4.6 to 3.4 m thick sand deposit at depths of 9.2 and 9.4 m or El. 219.6 and 217.0 m. Borehole C1 was advanced by wash boring from a depth of 10.7 m due to auger jamming and further advanced by rock coring from a depth of 11.1 m due to the presence of boulders in this layer. After the borehole advanced through the boulders, an about 1.2 to 1.5 m soil back up at the bottom of the casing was observed due to a possible hydrostatic uplift. In Borehole C2, auger refusal was encountered at a depth of 11.4 m and hole was further advanced using NW casing to the surface of the bedrock.

Underlying the clayey silt, Borehole C3 encountered at depth/elevation of 6.7 m/220.0 m, a sand deposit containing frequent cobbles/gravel and the borehole was terminated in this unit at a depth of 7.8 m or El. 218.9 m due to auger refusal, possibly on rock fragments or a boulder.

Grain size analysis was performed from a sample with frequent cobbles/gravel obtained from Borehole C3. The results are presented in Figure B-6 in Appendix B.

Gravel: 31 %

Sand: 52 %

Silt: 13 %

Clay: 4 %

The grain-size distribution of a more typical sample from the deposit from Borehole C2 is given in Figure B-7 which shows the following grain-size distribution:

Gravel: 7%

Sand: 69%

Silt: 21%

Clay: 3%

Standard Penetration tests performed in this basically granular soil deposit yielded N-values of 6 to in excess of 100 blows/0.3 m. These results indicate that the relative density of the soil can be described as loose to very dense (typically very dense close to the bedrock surface).

4.6 Bedrock

A pinkish grey gneiss bedrock was encountered in Boreholes C1 and C2 and was proven by NQ coring.

Table 4.6.1 - Bedrock elevation and condition

Borehole	Ground Elevation (m)	Bedrock Depth/Elevation (m)	T.C.R (%) *	R.Q.D. (%) **
C1	228.8	13.8/215.0	89 – 100	71 – 100
C2	226.4	12.8/213.6	95 – 100	81 – 88

*T.C.R. = Total Core Recovery

**R.Q.D = Rock Quality Designation

The Boreholes were advanced into the bedrock for a vertical distance of about 3.0 m by NQ coring. The percentage of recovery was 71 to 100% while the RQD values vary from 89% to 100%. These results indicate a rock quality of good to excellent.

4.7 Groundwater Condition

Groundwater conditions were observed in the open boreholes while drilling and upon completion of each borehole. In the deep boreholes, where NQ coring and wash boring were used (i.e. water introduced into the boreholes) the on-completion water levels may not be reliable. In addition, a piezometer was installed in Borehole C2. The observations made in the open boreholes and in the piezometer installed in Borehole C2 are shown on the individual Record of Borehole Sheets in Appendix A and summarized in Table 4.7.1.

Table 4.7.1: Water Level Observations

Borehole	Ground Elevation (m)	Depth / Elevation of the Tip of Piezometer (m)	Date	Water Level Depth / Elevation (m)
C1	228.8		April 29, 2009	1.8* / 227.0
C2	226.4	15.8 / 210.6	April 30, 2009 April 30, 2009 May 6, 2009	Ground Surface*/226.4 +0.3 / 226.7 (Artesian) +0.4 / 226.8 (Artesian)
C3	226.7		May 6, 2009	0.9* / 225.8

*Observation in open borehole, upon completion. Not stabilized.

The results indicate that the water levels in the open boreholes were recorded at or near the o.g. elevations. A mild artesian condition was observed at Borehole C2 emanating from the granular soil confined between relatively impervious materials (i.e. clayey silt at top and bedrock at the bottom).

It should be pointed out that the observed water levels represent the conditions at the time of our investigation and that the groundwater level at the site would be subject to fluctuations, both seasonally and in response to major weather events.

For and on behalf of Coffey Geotechnics Inc.

Gwangha Roh, Ph.D.

Ramon Miranda, P.Eng.
Manager, Transportation Division



Z.S. Ozden, P.Eng.
Senior Principal



Drawings

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

METRIC

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

CONT No.
GWP: 5053-05-00

HIGHWAY 534, BEAUDRY CREEK
PROPOSED CULVERT REPLACEMENT
BOREHOLE LOCATION PLAN

SHEET



LEGEND

Borehole

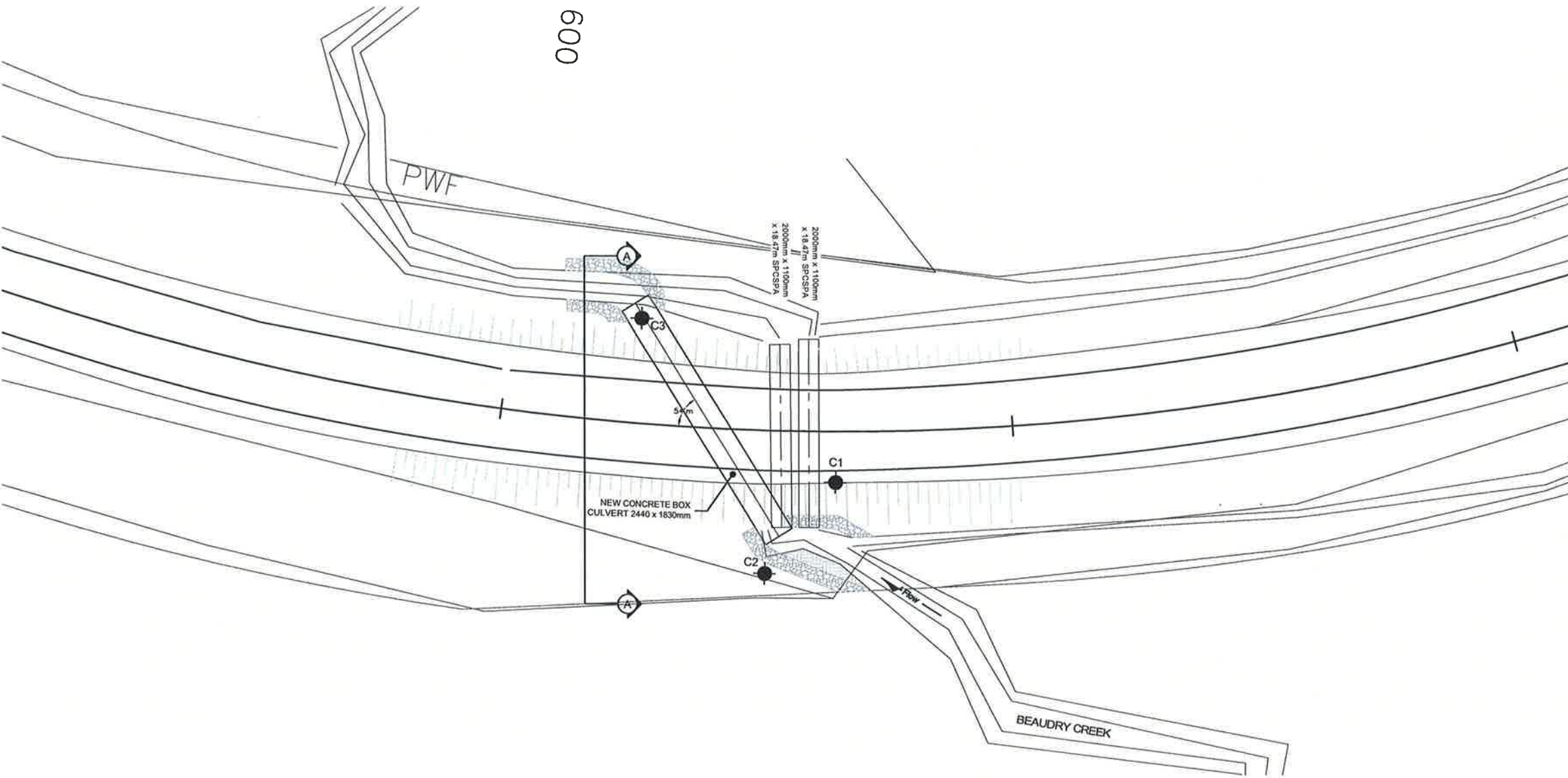
No.	ELEVATION	STATION	OFFSET
C1	228.8	10+632.5	5.0m Rt C/L
C2	226.4	10+614.2	10.5m Lt C/L
C3	226.7	10+625	14.5m Rt C/L

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

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31L-136			
TRANETO01238AA			
SUBMD	CHECKED	DATE Dec. 17, 2009	SITE 44-263/C
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1



METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.
GWP: 5053-05-00

HIGHWAY 534, BEAUDRY CREEK
PROPOSED CULVERT REPLACEMENT
SOIL STRATA

SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



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
- ARTESIAN WATER
- Head
- Encountered

No.	ELEVATION	STATION	OFFSET
C1	228.8	10+632.5	5.0m Rt C/L
C2	226.4	10+614.2	10.5m Lt C/L
C3	226.7	10+625	14.5m Rt C/L

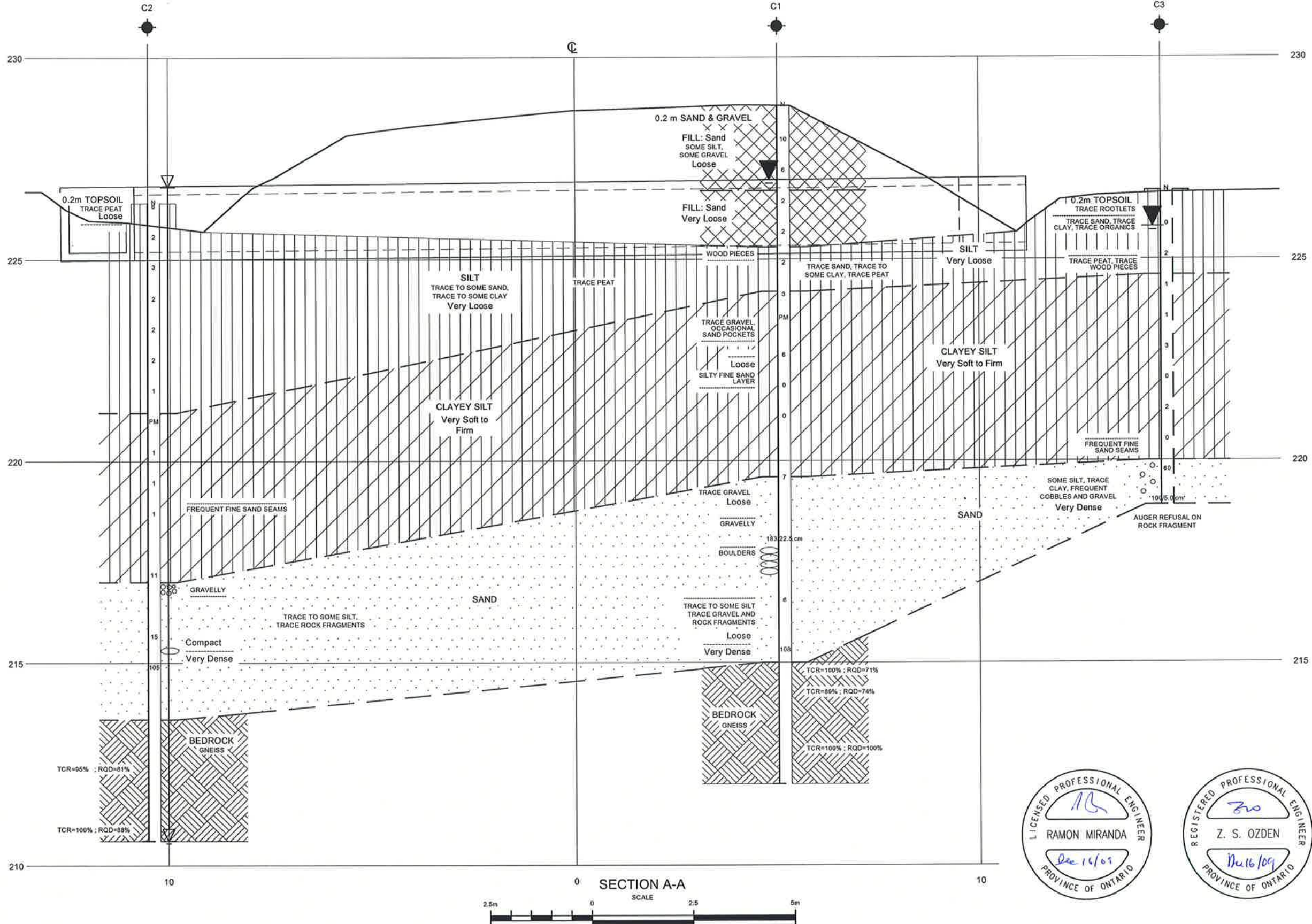
-NOTE-

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

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31L-136		TRANETOB01238AA		DIST	54
SUBMD	CHECKED	DATE	Dec. 17, 2009	SITE	44-263/C
DRAWN	PHK	CHECKED	RM	APPROVED	ZO
				DWG	2



Appendix A

Borehole Logs

TRANETO01238AA

RECORD OF BOREHOLE No C1

1 OF 2

METRIC

GWP 5053-05-00 LOCATION (Beaudry Creek Culvert) Sta: 10+632.5 ; 5.0 m Rt C/L (Sh) Hwy 534 ORIGINATED BY ZI
DIST 54 HWY HWY 534 BOREHOLE TYPE Hollow Stem Augers, Wash Boring & NQ Coring COMPILED BY WC
DATUM Geodetic DATE 4/28/2009 4/29/2009 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							WATER CONTENT (%)
							20 40 60 80 100	○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE						GR SA SI CL	
228.8	GROUND SURFACE														
0.0	0.2 m SAND & GRAVEL		1	AS										4 81 (15)	
	FILL: Sand some silt and gravel, brown loose, damp		2	AS											
			3	SS	10										
			4	SS	6										12 73 (15)
226.7															
2.1	FILL: Sand brown, v. loose, wet		5	SS	2										
225.3			6	SS	2										
3.5	wood pieces														
	SILT tr. sand, tr. to some clay, tr. peat brown to grey, v. loose, wet, dilatant		7	SS	2										0 8 79 13
224.2								4.5							
4.6			8	SS	3										0 7 63 30
	tr. gravel, occ. sand pockets dilatant		9	TW	PM										
	CLAYEY SILT grey, v. soft to firm, wet							4.0							spoon wet below
	loose		10	SS	6										
	silty fine sand layer, dilatant														
			11	SS	0										
			12	SS	0			6.0							
219.6	tr. gravel, loose			13	SS	7									
9.2	brown														
	grey, gravelly														
	SAND wet		14	SS	163/22.5 cm										wash boring from 10.7 m
	boulders														spoon bouncing
			15	RC	TCR=66% ; RQD=0%										NQ coring from 11.1 m to 12.2 m
															due to boulders
	tr. to some silt tr. gravel and rock fragments		16	RC	TCR=25% ; RQD=0%										soil back up 1.2 m
	loose		17	SS	6										
	v. dense														
215.0				18	SS	108									soil back up 1.5 m
13.8	BEDROCK pinkish grey gneiss			19	RC	TCR=100% ; RQD=71%									NQ coring from 13.8 m

Continued Next Page

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


TRANETOB01238AA

RECORD OF BOREHOLE No C1

2 OF 2

METRIC

GWP 5053-05-00 LOCATION (Beaudry Creek Culvert) Sta: 10+632.5 ; 5.0 m Rt C/L (Sh) Hwy 534 ORIGINATED BY ZI
DIST 54 HWY HWY 534 BOREHOLE TYPE Hollow Stem Augers, Wash Boring & NQ Coring COMPILED BY WC
DATUM Geodetic DATE 4/28/2009 4/29/2009 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									
213.8								20	40	60	80	100					
	BEDROCK pinkish grey gneiss		20	RC	TCR=89% ; RQD=74%		213										
			21	RC													
212.0					TCR=100% ; RQD=100%		212										
16.8	End of Borehole Water level @ 1.8 m (not stabilized)* upon completion Hole caved-in @ 5.8 m upon completion																

TRANETO01238AA

RECORD OF BOREHOLE No C2

1 OF 2

METRIC

GWP 5053-05-00

LOCATION (Beaudry Creek Culvert) Sta: 10+614.2 ; 10.5 m Lt C/L Hwy 534

ORIGINATED BY ZI

DIST 54

HWY HWY 534

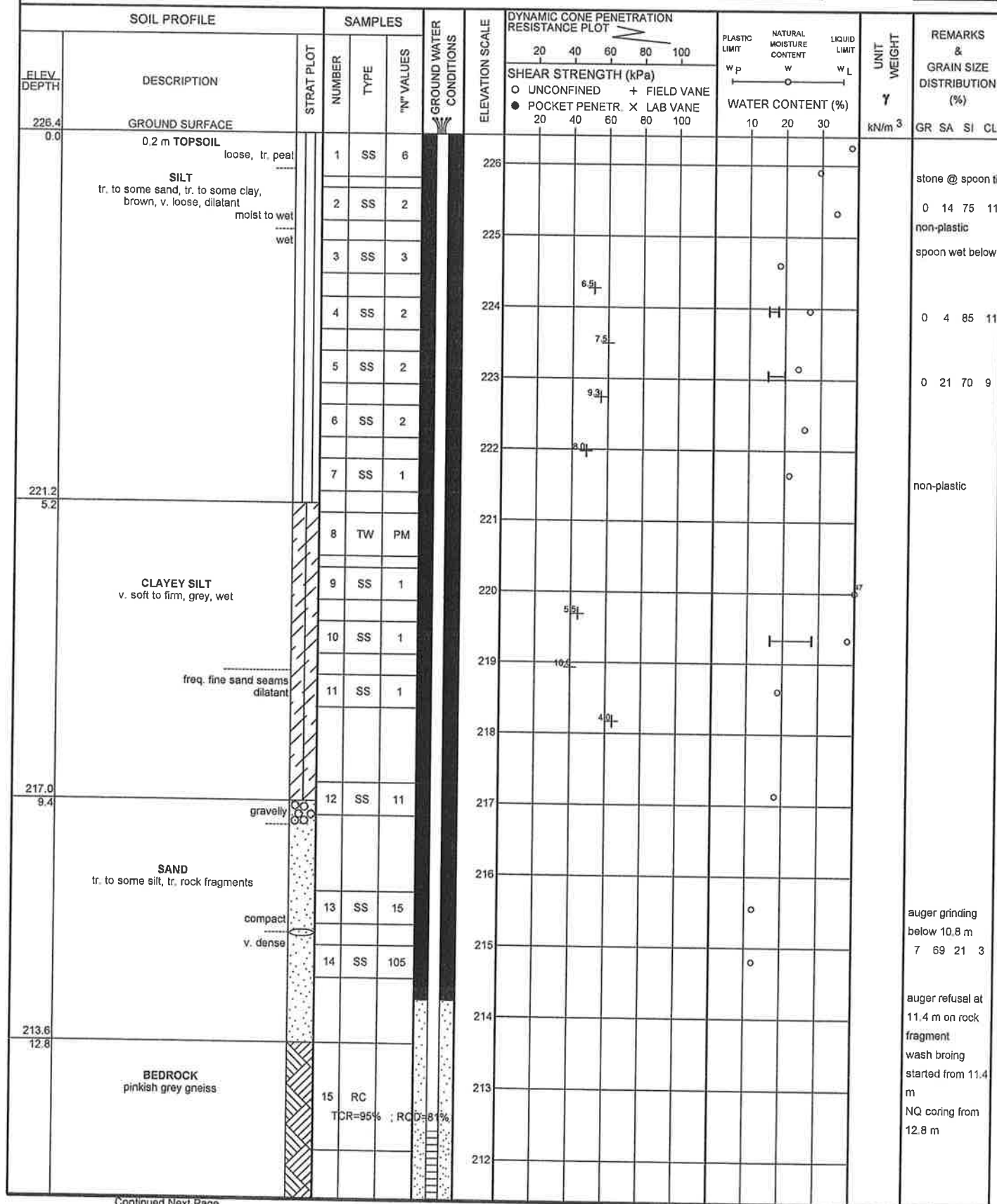
BOREHOLE TYPE Hollow Stem Augers, Wash Boring & NQ Coring

COMPILED BY WC

DATUM Geodetic

DATE 4/29/2009 4/30/2009

CHECKED BY ZO



Continued Next Page

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

TRANETO01238AA

2 OF 2

METRIC

GWP	5053-05-00	LOCATION	(Beaudry Creek Culvert) Sta: 10+614.2 ; 10.5 m Lt C/L Hwy 534	ORIGINATED BY	ZI
DIST	54	HWY	HWY 534	BOREHOLE TYPE	Hollow Stem Augers, Wash Boring & NQ Coring
DATUM	Geodetic	DATE	4/29/2009 4/30/2009	COMPILED BY	WC
				CHECKED BY	ZO

[illegible]

+³, ×³: Numbers refer to Sensitivity

TRANETOB01238AA

1 OF 1

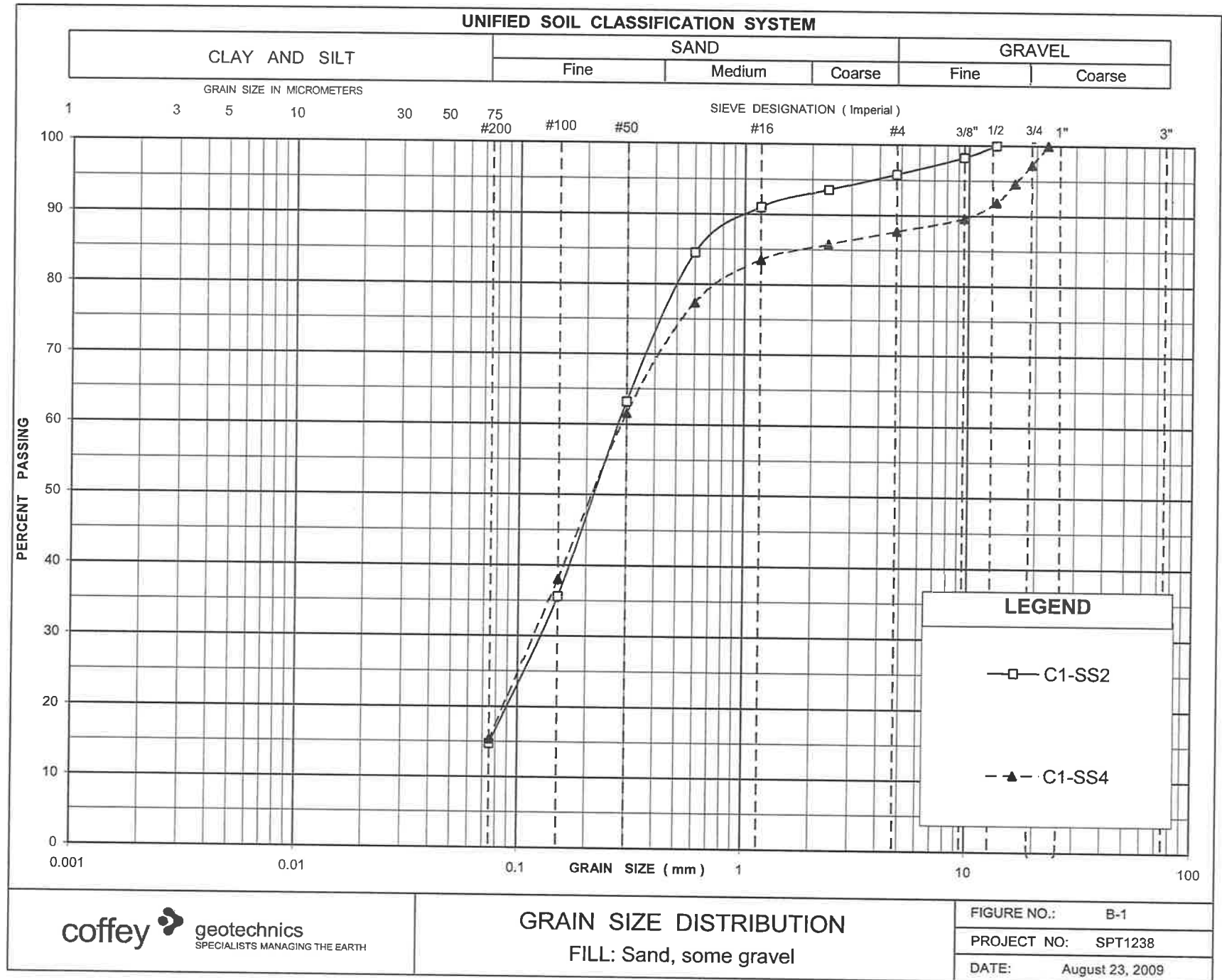
METRIC

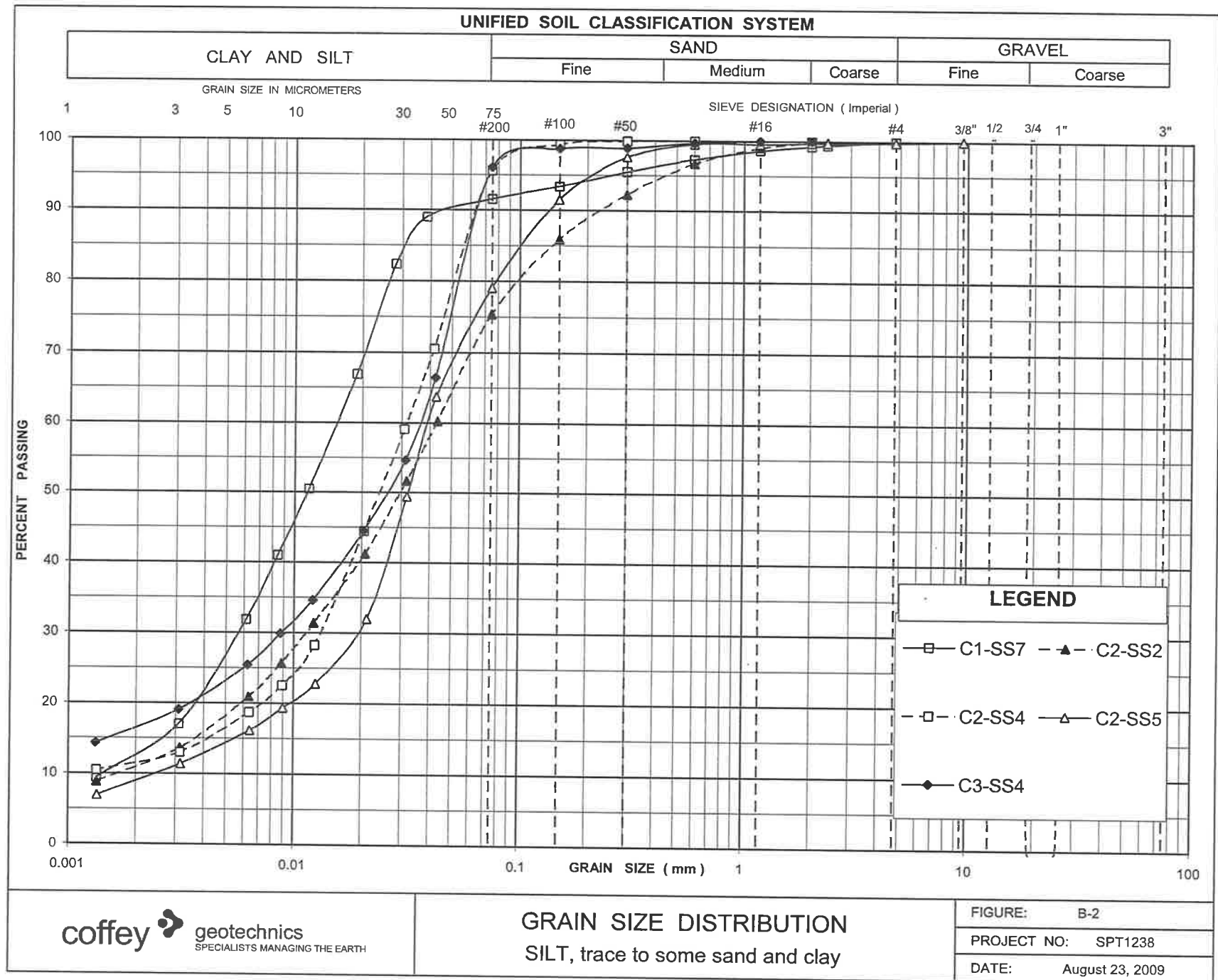
GWP	<u>5053-05-00</u>	LOCATION	<u>(Baudry Creek Culvert) Sta: 10+825 ; 14.5 m Rt C/L Hwy 534</u>	ORIGINATED BY	<u>ZI</u>
DIST	<u>54</u>	HWY	<u>HWY 534</u>	BOREHOLE TYPE	<u>Hollow Stem Auger</u>
DATUM	<u>Geodetic</u>	DATE	<u>5/6/2009</u>	COMPILED BY	<u>WC</u>
				CHECKED BY	<u>ZO</u>

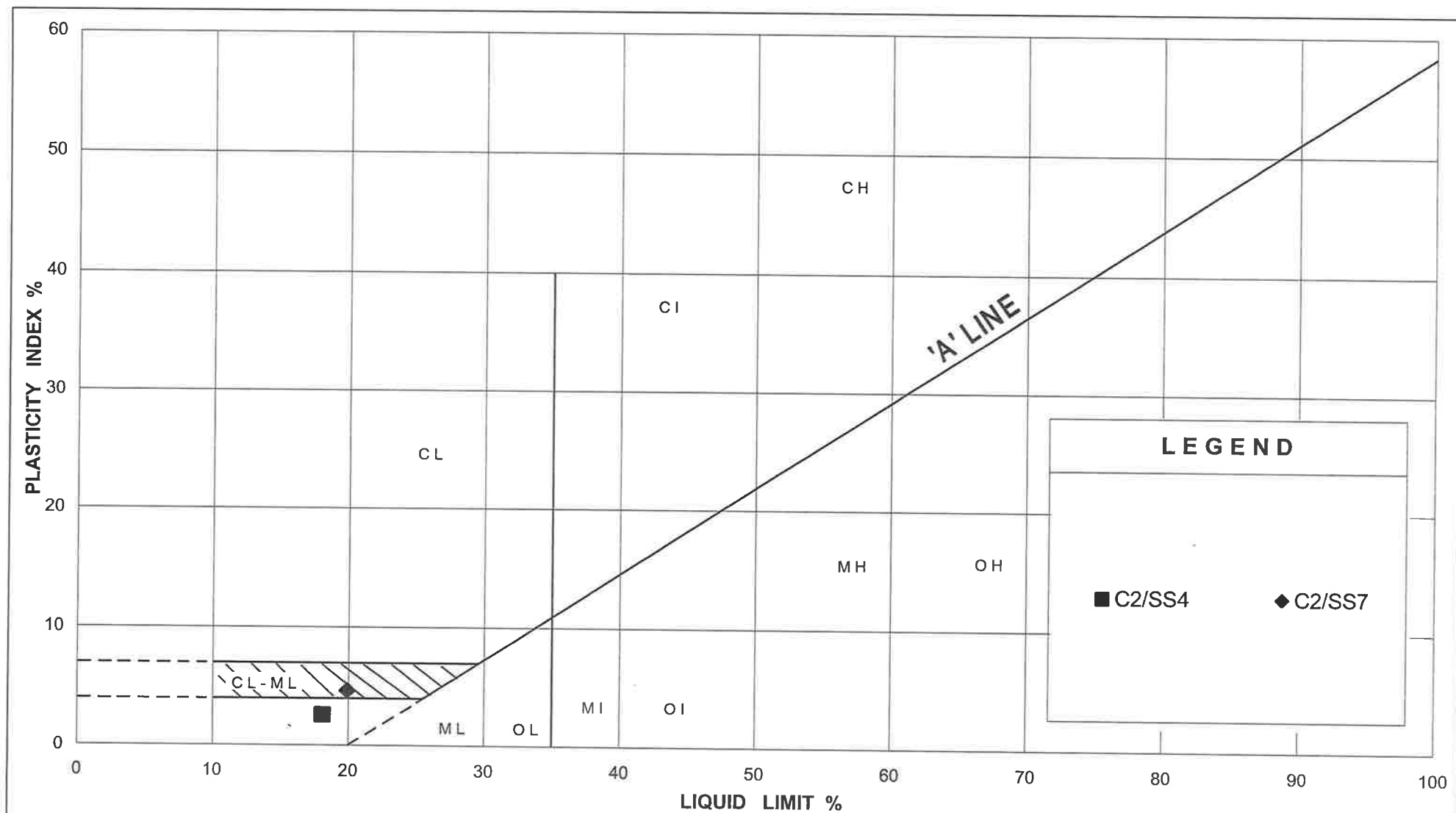
+³, ×³: Numbers refer to Sensitivity

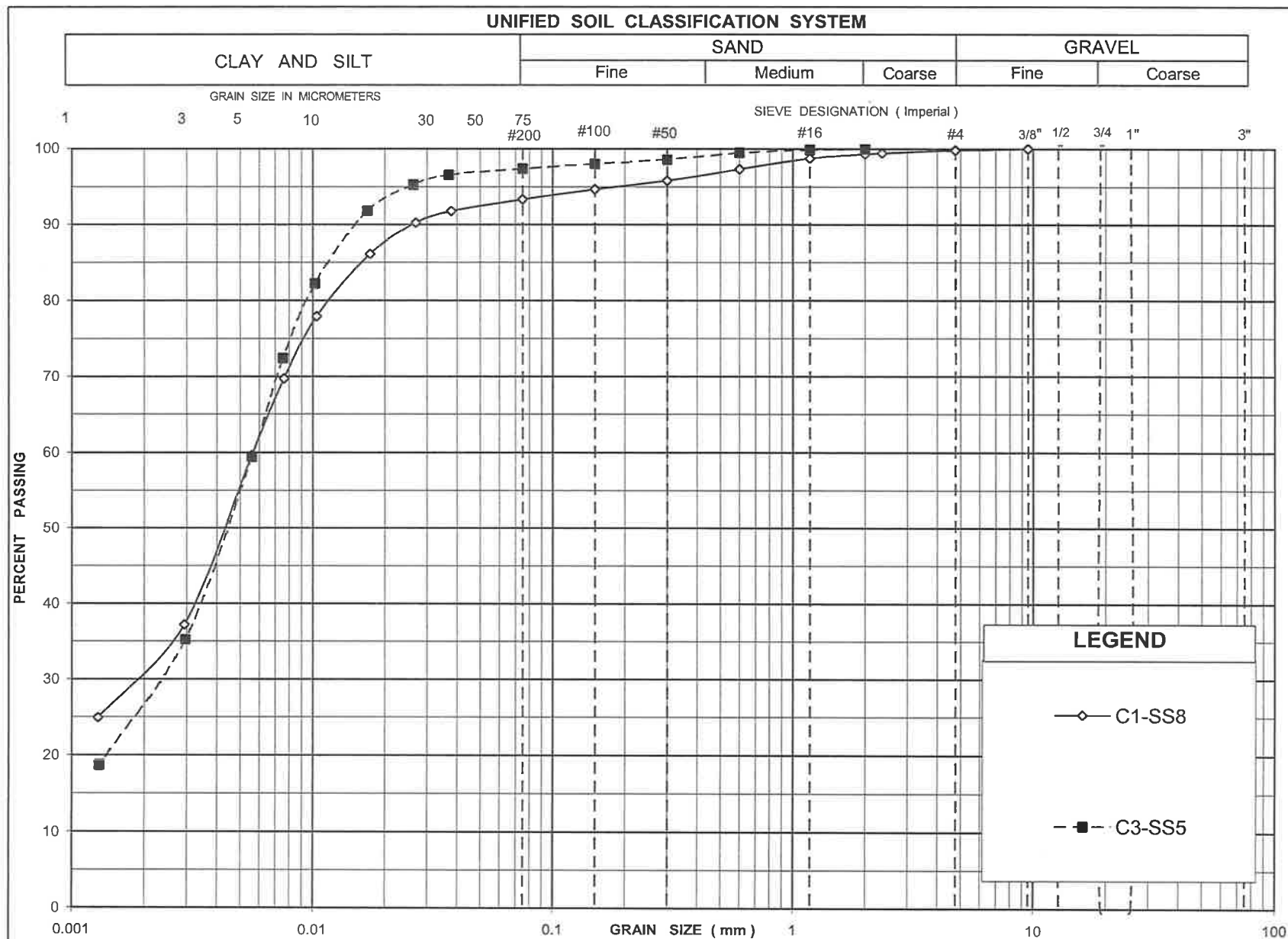
Appendix B

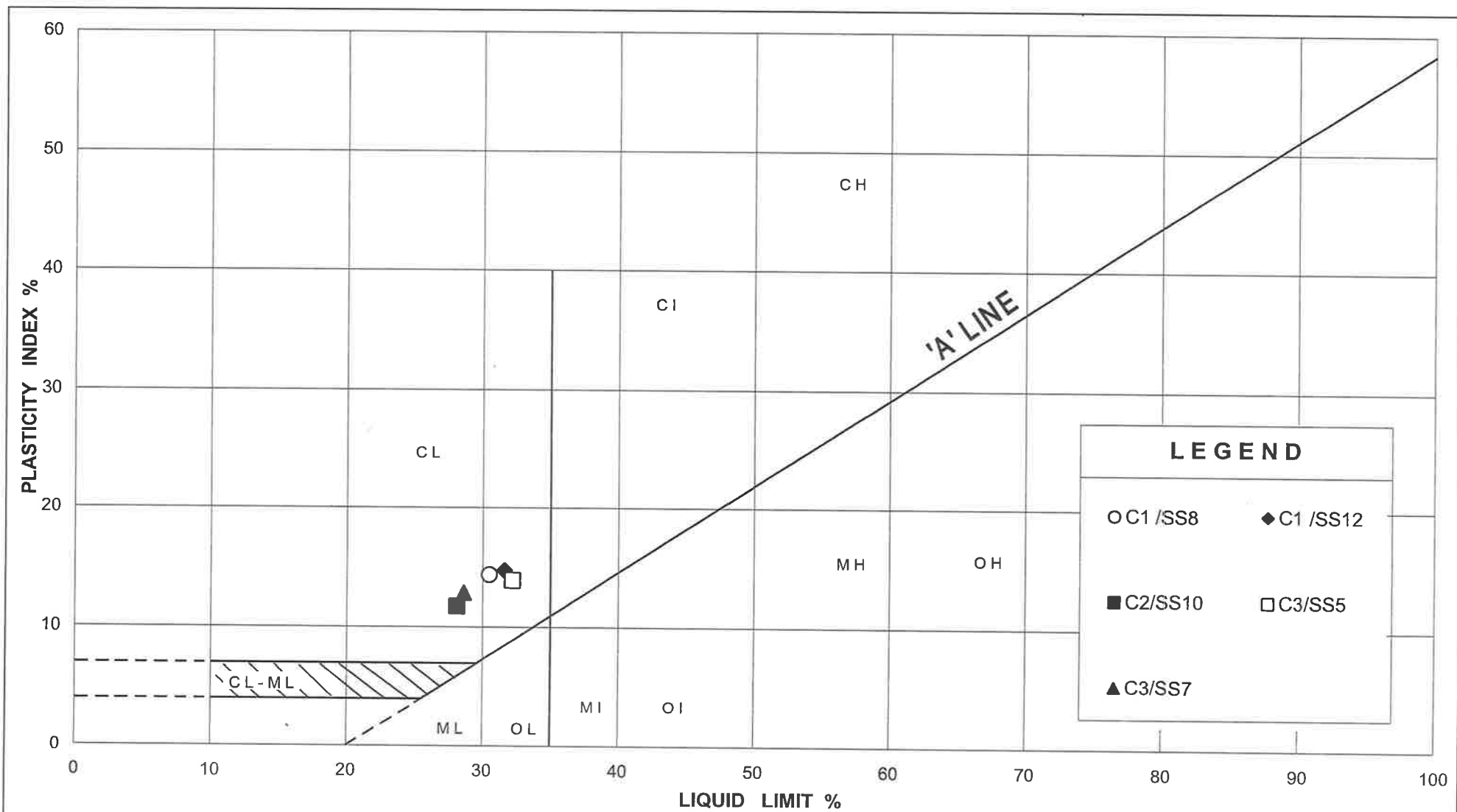
Test Results

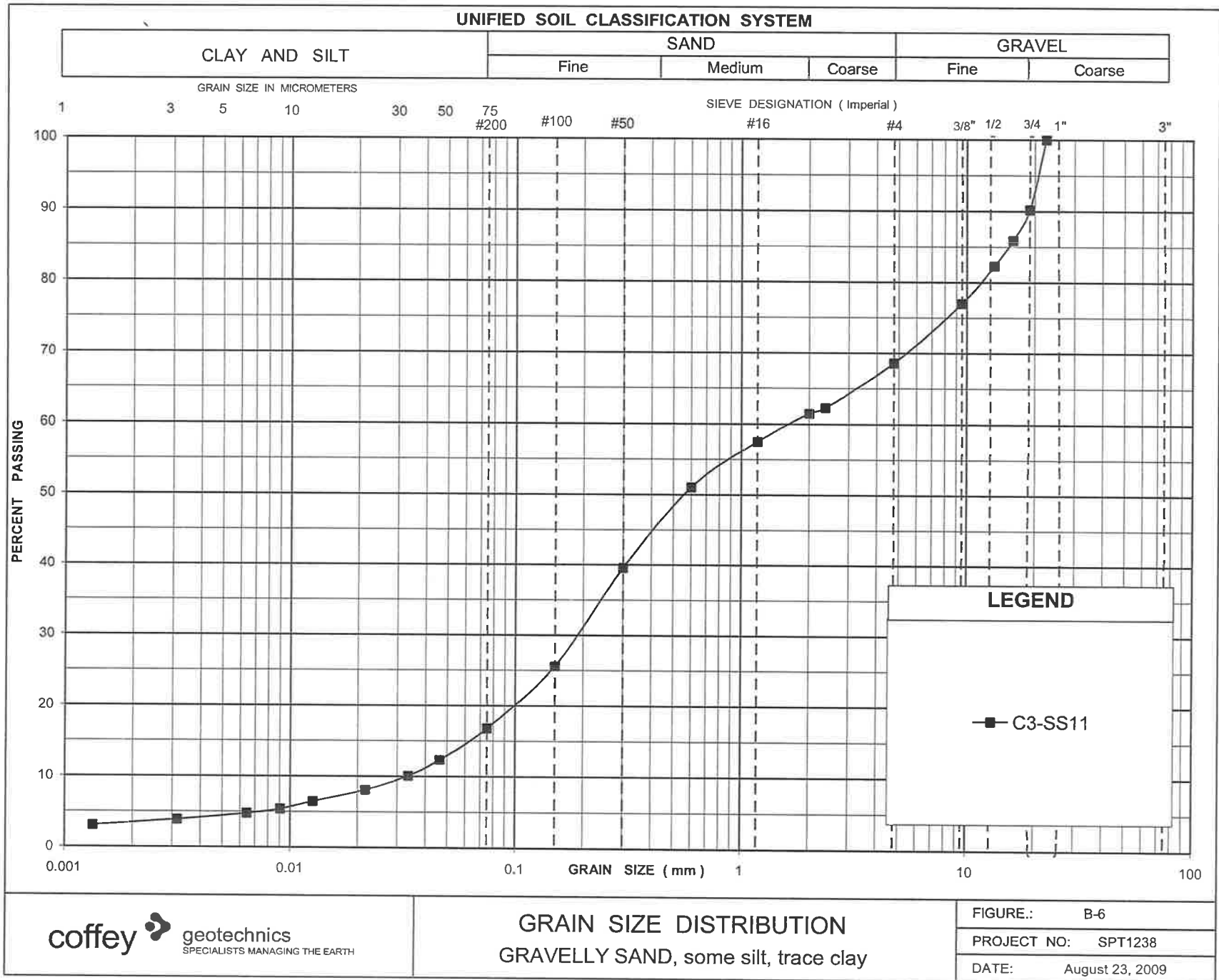


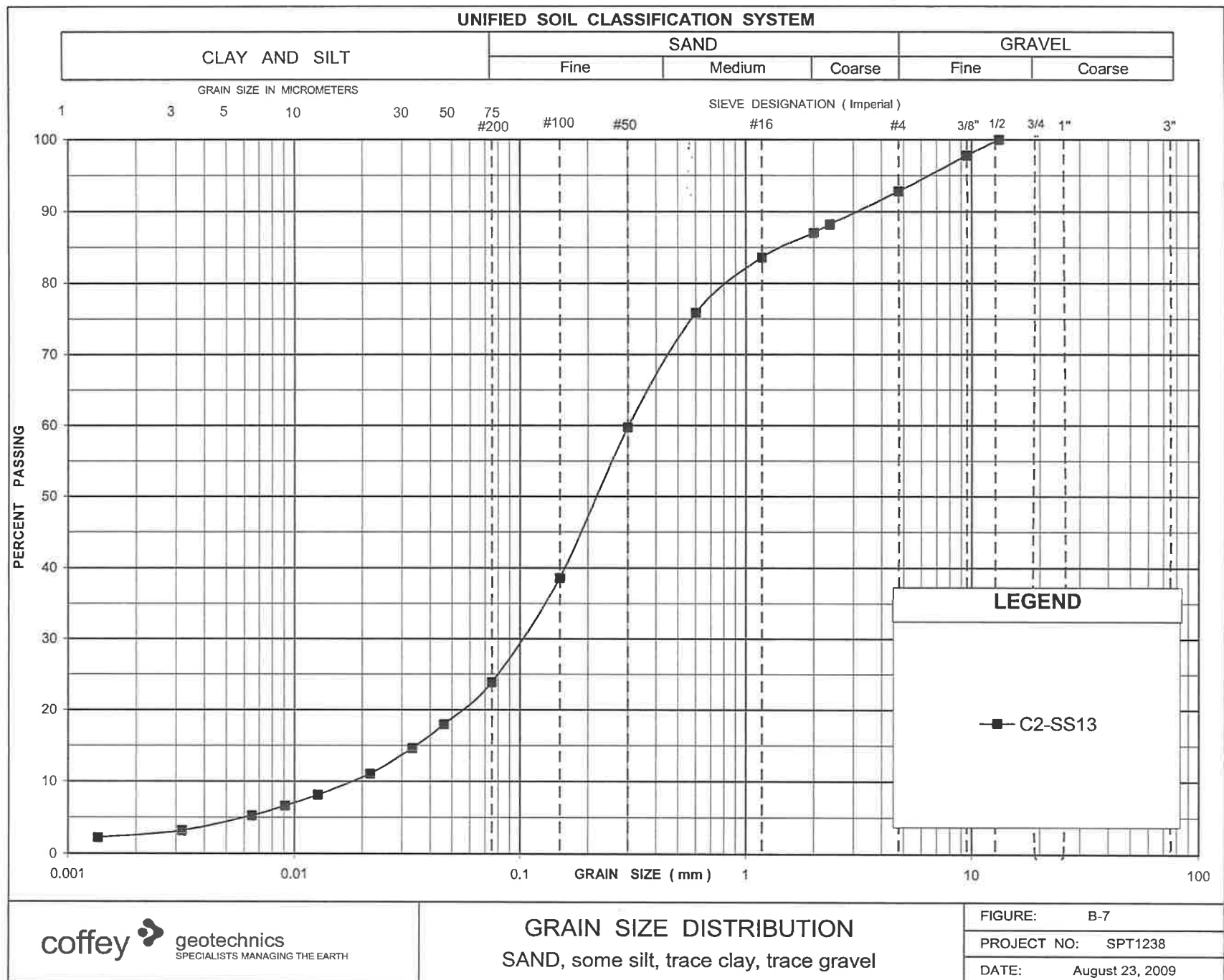












Appendix C

Site Photographs



Figure C-1 – View of the inlet of Beaudry Creek Culvert



Figure C-2 – View of the outlet of Beaudry Creek Culvert

Appendix D

Rock Core Photographs



Figure D-1 -- Rock core retrieved at Borehole C1



Figure D-2 -- Rock core retrieved at Borehole C2

Appendix E

Explanation of Terms Used in the 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
PROPOSED REPLACEMENT OF BEAUDRY
CREEK CULVERT ON HIGHWAY 534,
TOWNSHIP OF GURD, ONTARIO.
DISTRICT 54, SITE # 44-263/C
G.W.P # 5053-05-00,
GEOCRES NO: 31L-136**

D.M. Wills Associates Limited

TRANETOB01238AA
December 17, 2009

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Appendix F: OPSD

Appendix G: Limitations of Report

**FOUNDATION DESIGN REPORT
REPLACEMENT OF BEAUDRY CREEK CULVERT
HIGHWAY 534, TOWNSHIP OF GURD, ONTARIO
G.W.P. 5053-05-00, SITE 44-263/C**

5 DISCUSSION AND RECOMMENDATIONS

The existing culvert beneath Highway 534, about 1.8 km east of Highway 524, provides water flow in the Beaudry Creek from east (right) to west (left). The existing 1880 x 1260 mm, approximately 18 m long, twin CSP culverts, which are showing signs of excessive deformations (visible flattening of both culverts at inlet), will be replaced with a concrete box culvert. There is approximately 2.0 m of cover over the crown of the roadway. The existing twin culverts are located perpendicular to the Highway 534, but the proposed new culvert will be constructed on a skew angle of 54° to the existing Highway 534 centreline to redirect the creek water flow back to its original flow direction to prevent the water accumulation (i.e. ponding) at the inlet of the culvert which is currently present and allow for a smoother flow of the creek (see Borehole Location Plan, Dwg No. 1).

In accordance with general practice, the following replacement alternatives were considered by D.M. Wills Associates Limited (Wills), TPM consultant for the project.

- Do nothing
- Precast concrete box culvert
- Cast-in-place concrete box culvert
- Corrugated steel pipe culvert
- Open bottom culvert

Based on the prevailing subsurface conditions from a geotechnical point of view, a flexible pipe culvert is preferred to replace the existing twin culverts. However, concrete structures are typically favoured in northern Ontario given their potential to provide the greatest longevity in aggressive environments, including soft water conditions at the present site. Precast concrete is generally considered more durable than cast-in place concrete due to the controlled fabrication environment. A precast structure would also be preferential to cast-in-place due to ease of installation and faster implementation. As well, a precast concrete culvert is better suited (i.e. more flexible) than a cast-in-place concrete culvert with the prevailing unfavourable subsurface conditions.

In addition, open bottom culvert at the site is not recommended due to the prevailing highly erodible soil at the culvert location.

Based on the drawing provided to us by Wills, the new culvert will be about 27 m long, 2440 (span) x 1830 (rise) mm concrete box culvert. The proposed culvert invert elevations will be about 225.8 m (inlet) and 225.7 m (outlet).

Three boreholes were advanced for this investigation, one from the top of highway embankment (Borehole C1 from the shoulder) and two from the bottom of the embankment close to the existing culverts (Boreholes

C2 and C3). Since the orientation and location of proposed culvert was not decided prior to our investigation, boreholes were drilled adjacent to the existing culverts. Typically, the water elevation in the creek is about 226.5 m based on the drawing provided to us by Wills.

In general, at the culvert site, Borehole C1 drilled from the top of road embankment contacted embankment fill to a depth of 3.5 m to El. 225.3 m, underlain by a 1.1 m thick silt deposit. In Boreholes C2 and C3, a similar silt deposit was contacted immediately below the topsoil (El. 226.2 and 226.5 m). The silt deposit was found to extend to El. 221.2-224.6 m (i.e. 1.1 to 5.0 m thick) and is underlain in all three boreholes by a clayey silt deposit. The clayey silt was found to be 4.2 to 4.6 m thick and is in turn underlain by lower granular deposits (i.e. sand, containing in places some gravel, cobbles and boulders) to the surface of gneiss bedrock in Boreholes C1 and C2 at depths of 13.8 and 12.8 m or at El. 215.0 and 213.6 m respectively. Borehole C3 was terminated in the lower granular deposit possibly on rock fragments or on a boulder.

The groundwater table at the time of our investigation was found at near the o.g. levels. The piezometer installed in Borehole C2 recorded a mild artesian condition (i.e. 0.4 m above ground surface), emanating from the sand deposit underlying the bedrock. The groundwater level can be expected to fluctuate in response to major weather events and seasonally.

5.1 Foundations

The proposed culvert will be constructed at a skew to the roadway to coincide with the original creek bed, as shown on the Borehole Location Plan, Drawing No. 1. Only the inlet of the new culvert will be located close to the inlet of the existing culvert. It is our understanding that there will be no grade raise in the highway and thus there are no anticipated additional stresses induced by the new culvert installation. The information provided to us by Wills indicate that the invert elevation at the inlet will be at about 225.8 m and at the outlet it will be 225.7 m.

The boreholes show below the topsoil in Boreholes C2 and C3 and the embankment fill in Borehole C1, the presence of a silt deposit overlying a major clayey silt deposit. The silt at the site is found to be non-plastic and can dilate in the presence of water if it is disturbed during the construction. The clayey silt is a cohesive material but some fine sand and silt seams within this clayey silt deposit also show a dilatant behaviour.

Based on the findings of our investigation, we have considered a number of foundation options varying from normal spread footings to deep foundations. The following table provides a summary of foundation alternatives.

TABLE 5.1.1 Foundation Type Summary

Foundation Type	Comments	Recommendations
A CSP type flexible culvert supported on thick granular bedding	Represents the best alternative from a geotechnical point of view, due to the presence of very weak and dilatant silts	Recommended alternative from geotechnical engineering perspective
Precast Concrete Box Culvert supported on thick bedding	Feasible if a thick granular fill is provided over the weak subgrade soils	Recommended alternative if the construction of a CSP type flexible culvert is not feasible at this site
Normal Spread Footings	Founding on the dilatant silt will require careful dewatering and vibration-free construction. Very low geotechnical resistance.	Not recommended based on reliability
Deep foundations (driven piles, drilled caissons, etc.)	Feasible option, but rather costly and time consuming to construct	Not recommended based on cost and anticipated long construction schedule.

As the table presented indicates, the preferred method from a geotechnical engineering point of view is to use a flexible pipe supported on a relatively thick bedding, with the prevailing unfavourable subsurface conditions. If a CSP type culvert is unacceptable, then a precast concrete structure can be considered. A cast-in-place concrete structure supported on deep foundations can also be considered but this will not be cost effective, as well as requiring a considerably long time to construct.

Based on the information provided to us by Wills, the new culvert invert inlet and outlet elevations will be about 225.8 and 225.7 m, respectively. The present investigation findings indicate that after sub-excavation for bedding, the exposed subgrade can be expected to consist of silt, assuming that the borehole results are representative of the entire site. The silt at the site is very loose, the groundwater table is very high (a slight artesian condition) and the silt is prone to disturbance. It can easily dilate, especially in the presence of water.

We understand that the elevation of the existing road will remain the same and therefore there will be no stress increase. In fact, since the culvert will weigh less than the soil removed there will be a stress

decrease but this will be a somewhat compensated by the soil exchange (i.e. very loose silt will be removed and replaced with heavier granular soils) for the construction of the bedding, especially in the case of a precast concrete culvert.

5.1.1 Corrugated Steel Pipe (CSP) type Culvert

A CSP type culvert is the preferred option with the prevailing unfavourable subsurface conditions as revealed by the boreholes.

We recommend that an at least 0.7 m of granular bedding should be provided beneath the CSP pipe in view of the very loose condition of the underlying silt.

The following procedure is recommended for this purpose.

We recommend that the site be dewatered to ensure that the groundwater level be lowered to at least 0.3 m below the anticipated excavation level (e.g. for an average invert elevation of say 225.7 m the excavation would be carried to El. 225.0 m less 0.3 m bringing the effective dewatering level to El. 224.7 m). This can probably be achieved by perimeter ditches, gravity drainage and rigorous pumping from properly filtered and strategically placed sumps, depending on the conditions at the time of construction. If this proves to be ineffective then a vacuum well pointing system may also be required. The presence of the practically impervious nature of the clayey silt underlying the silt will need to be taken into consideration when designing the sump system and particularly the well points. It should be noted that the interface of the silt with the underlying clayey silt appears to be variable across the site. This interface was contacted at El. 224.6 m at Borehole C3 and at El. 224.2 m at Borehole C1, whereas at Borehole C2 it was encountered at El. 221.2 m.

Due to the observed artesian condition and the very loose nature of the silt it is recommended that the excavation be carried out to the required level (e.g. where the invert of the pipe is 225.7 m, to 0.7 m below this or to El. 225.0 m) in narrow widths in the perpendicular direction to the pipe. The recommended width of the excavation is 2 to 3 m but this may need to be adjusted depending on the site conditions, if necessary. As the excavation progresses, it should be ensured that the subgrade consists of inorganic soils. This can be achieved by a visual and tactile examination of the soil excavated from the bottom of the trench. The excavated soil should immediately be replaced with Granular 'B' Type II soil to about 0.2 m below the invert level. The granular 'B' Type II backfill should be compacted as much as possible during placement by pushing in with the bucket of the backhoe, as well after the construction of the entire Granular 'B' Type II portion of the bedding by operating track-mounted construction equipment, if site conditions permit and the silt subgrade does not appear to be disturbed. No heavy construction equipment should be operated at the culvert area unless base is considered firm enough and the silt subgrade would not be disturbed. The upper 0.2 m should consist of Granular 'A' type soil which should be placed when the pipe is ready to be installed. One problem with this approach is that the uncompacted Granular 'B' Type II material may cave-in when excavating for the next (adjacent) strip. To prevent this, good workmanship would be required and the width of the strips may need to be adjusted.

Pumping from filtered sumps within the granular soil may be necessary if and where water collect within these granular soils. Any water collected would be discharged in order to stabilize the granular soils and to effect some compaction.

The excavation and soil replacement will need to extend at least 1.2 m beyond the perimeter of the foot print of the pipe.

After the excavation to 0.7 m below the invert of the pipe and its replacement with Granular 'B' Type II material to 0.5 m (i.e. to 0.2 m below the invert) is completed, the upper 0.2 m of the bedding can be placed. This should consist of Granular 'A' Type material and should be compacted from the surface to not less than 93% of its Standard Proctor Maximum Dry Density (SPMDD), immediately before placing the pipe, using a suitably small compactor in order not to disturb the underlying dilatent silt soil. For the same reason, depending on the site conditions, vibration should be applied very sparingly or not at all.

The dewatering of the site should be continued (i.e. dewatering to at least 0.3 m below the bottom of the granular soil and the surface of the underlying silt) until the pipe is placed and sufficiently backfilled. To avoid unbalanced loading on the culvert, the height of the backfill around the culvert should be maintained equal on both sides throughout the construction, as much as practically possible. The backfill should be placed in suitably thin layers and each layer should be compacted to MTO standards.

The entire construction, including excavation, backfilling of the bedding underneath the culvert as well as the placement and compaction of the backfill around the culvert structure should be carried out under the supervision of the Quality Verification Engineer (QVE).

Based on the above, a Factored Bearing Resistance at U.L.S of 90 kPa and a value of 30 kPa at S.L.S. can be assigned for design purposes. As was mentioned before, imposed loads due to the culvert will be less than the existing, especially near the ends. Therefore, in theory, there should be no problem associated with settlements, provided that the bearing soil is undisturbed during the construction. However, an allowance of 40 mm of possible total settlement should be made due to possible rebound during construction due to stress relief. With this amount of settlement cambering is not believed to be necessary. We would like to point out that the quoted settlement is based on undisturbed founding subgrade. If the subgrade is unduly disturbed during construction, greater settlements could occur. To prevent this, adequate dewatering needs to be applied, as discussed before.

We recommend that all bearing surfaces be inspected and approved by a qualified Geotechnical Engineer who is familiar with the findings of this investigation and is appointed by the QVE.

5.1.2 Precast Concrete Culvert

We understand that the construction of a precast concrete culvert is planned at this site. As mentioned before, with the prevailing subsurface conditions the use of a CSP type culvert is the preferred option at the site. However the use of a relatively flexible precast concrete culvert is also feasible, as discussed below.

The procedure is basically similar to the construction method discussed in the previous section for the CSP type culvert. In this case, however, the minimum thickness of the bedding beneath the culvert would be increased from 0.7 to 0.9 m. In this case since the bottom of the concrete culvert will be at El. 225.6 m at the inlet and 225.5 m at the outlet, the soil exchange will be carried out to 0.9 m below these elevations (i.e. 224.7 m at the inlet and 224.6 m at the outlet). In addition, the excavation and soil exchange would extend at least 1.8 m beyond the foot-print of the structure.

The site would be dewatered to ensure the groundwater level is depressed to not less than 0.2 m below the bottom of excavation (i.e. to at least El. 224.5 m at the inlet and 224.4 m at the outlet and to not less than 224.4 m in between), as discussed before.

The excavation would be carried out in narrow widths and immediately backfilled with Granular 'B' Type II material to within 0.2 m below the proposed bottom elevation of the box culvert. In this event since the unsupported height of the uncompacted Granular 'B' Type II material would be even higher (i.e. 0.7 m), good workmanship and careful construction techniques would be required to prevent a cave-in condition.

Following the completion of soil replacement to 0.2 m below the bottom of the concrete panel elevations (i.e. to the top of the Granular 'B' Type II), some compaction of the granular material can be effected (e.g. operating track-mounted construction equipment), being cognizant of not disturbing the silt subgrade. The grade can then be raised to the underside elevation of the concrete pre-cast panels by placing a 0.2 m thick Granular 'A' material. This Granular 'A' layer would also be lightly compacted by the operation of track-mounted equipment followed by, if site conditions permit, operating a light compactor with little or no vibration. It is recommended that if feasible the Granular 'A' be compacted to not less than 93% of the Standard Proctor Maximum Dry Density for the material.

Assuming that the above recommendations are followed, the following geotechnical resistances can be assigned:

Factored Bearing Resistance at U.L.S = 95 kPa

Geotechnical Resistance at S.L.S = 35 kPa

The transportation and placement of the precast concrete panels will need to proceed with caution such that the weight of the adjacent embankment and that of the construction equipment including the loaded crane will not cause disturbance and/or failure of the newly constructed bedding and/or the underlying weak soil. The panels and the crane should not be brought to the site, until the site was fully backfilled by Granular 'B' Type II and Granular 'A' bedding material. The suitability of the existing embankment to carry the loaded crane will need to be determined. This is contractor's responsibility.

5.1.3 Deep Foundations

If a cast-in-place concrete culvert is required then deep foundation will be required. Since however, this option is not favoured by Wills (due to the very costly and time consuming nature of this alternative), the following are some brief comments.

As was mentioned in the previous section of this report, allowance should be made for up to 40 mm settlement. As these could possibly translate into differential settlements in between individual panels, this aspect should be discussed with the supplier. In our experience, however, a 40 mm differential settlement does typically not pose a threat to the integrity of a precast concrete culvert.

The use of deep foundations is feasible. The salient features of this approach are the presence of bedrock (encountered at El. 215.0 m and 213.6 m at Boreholes C1 and C2, respectively or about 13.8 to 12.8 m below the o.g. levels) and the presence of a basal sand deposit (overlying the bedrock). This sand deposit is under excessive upward hydrostatic pressure and has a variable relative density throughout its depth (ranging from loose to very dense) and as well, contains cobbles and boulders in a random fashion.

Drilled and cast-in-place concrete piles (caissons) are not recommended due to the anticipated installation problems when drilling through the water bearing sand under upward excess hydrostatic pressure, and the presence of cobbles and boulders in the stratum. As well the caissons will have to be socketed into the bedrock. All these will render the installation of caissons very costly for a project of this nature and as such their use is not recommended.

Timber piles may be damaged when driving into the basal sand deposit with variable relative density, due to the presence of cobbles and boulders. The use of timber piles is not recommended, based on reliability.

Driven steel (i.e. steel H or tube) piles may be considered. These would be driven to refusal on the surface of the bedrock or in the overburden immediately above it. Some problems may occur due to the presence of cobbles and boulders. However, driven steel piles would be most suitable choice for the prevailing subsurface conditions. Another problem with driven piles is the presence of the artesian condition which will need to be taken into consideration in the design.

For all these reasons, based on cost factor, the use of deep foundations and hence a cast-in-place concrete structure is not recommended.

5.2 Backfilling

The bedding and embedment material should be extended along the sides to cover the culvert. The selection and placing of the backfill should be in accordance with OPSD-803.010. 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 in loose condition) 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. 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 culverts, in particular the CSP's, 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.

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) S6-06. 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 $\phi=32^\circ$ (unfactored)

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

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

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 should be restricted in size as per current MTO practice.

5.3 Construction Comments

We understand that the new culvert will be located beyond the existing twin culverts location. As such, the existing culverts can maintain the creek flow during the construction.

As the natural, inorganic subgrade is expected to consist of primarily silt, the water level will need to be lowered at least 0.2 to 0.3 m below bottom of excavation as was discussed in Sections 5.1.1 and 5.1.2. It should be noted that the clayey silt deposit underlying the silt may cause some clogging problems during the dewatering and this should be taken into consideration when designing the dewatering method.

We recommend that the contractor be asked to submit their method of dewatering to the CA for information purposes and the appropriate Non Standard Special Provision (NSSP) should be included in the contract documents.

In addition, if at all possible, the construction should be carried out during a dry season.

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.

Embankment Fill	Type 3 soil above water level*
	Type 4 soil below water level
Topsoil	Type 3 soil above water level
	Type 4 soil below water level
Silt	Type 4 soil
Clayey Silt	Type 4 soil
Basal Granular Soils	Type 4 soil

*Even if embankment fill is Type 3 soil above groundwater level, we recommend 2H:1V temporary excavation slope due to very loose to loose conditions of fill at the site.

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

An NSSP should be included in the Contract Documents alerting the Contractor of the subsurface and groundwater conditions and that the groundwater control requirements should be planned accordingly by the Contractor prior to construction.

The open-cut construction, as presently planned, can be carried out at 2H:1V side slopes or flatter, including excavations extending into the existing embankment, provided that the site is fully dewatered.

We understand that the construction will be carried out with full road closure and shoring will not be required. This is the preferred choice in our opinion since shoring can be expected to be costly. However, for the sake of completeness the following parameters are provided for the soil types encountered in the boreholes.

Table 5.3.1: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m ³)
Embankment fill	0.33	0.50	3.0	20.5
Silt	0.42	0.60	2.4	17.0
Clayey Silt	0.42	0.60	2.4	16.5
Lower sand	0.29	0.45	3.4	20.5

Typically, temporary support (shoring) in Ontario is provided in the form of soldier piles and lagging. Depending on the depth of excavation, tie-backs may also be required (e.g. excavations in excess of about 3 m). The piles can be expected to extend to the surface of the bedrock or close to it. Some problems may be encountered while driving the soldier piles, due to the presence of cobbles and boulders in the basal sand deposit.

Shoring system should be designed so that the lateral movement of the 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 design should be carried out by a Professional Engineer experienced in this type work.

It is our opinion that depths of piles and requirement of dead-man and anchor system should be decided when shoring detail is available. If roadway protection system is required, we recommend an additional investigation may need to be carried out to decide the design parameters of the shoring system.

5.4 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. It should be noted that the silt deposit overlying the clayey silt is a highly erodible and highly frost susceptible material.

The following are some general suggestions.

We recommend that a concrete cut-off (apron) be constructed both at the inlet and outlet to prevent seepage beneath and around the culvert, especially through the granular bedding and granular backfill around the culvert. Beneath the culvert, the concrete cut-off wall should extend to a suitable depth (e.g. below any possible scour depth). Consideration may also be given to an impervious seal at the inlet and outlet.

At the inlet, consideration may also be given to the use of a clay seal. The purpose of the clay seal is to ensure that water flow is channeled through the culvert and does not seep through the backfill around the structure and from beneath the structure. The clay seal should therefore be continuous and 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 to 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. This would generally be extended about 8 m along the channel and the sides (to at least 0.3 m above the high water).

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

5.5 Frost Protection

Design frost protection for the general area is 1.9 m. A permanent soil cover of at least 1.9 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

Consideration may be given to advance an additional borehole, near the central portion of the new culvert, since the surface of the clayey silt deposit is quite variable from inlet to outlet, and to confirm the condition of the existing embankment fill.

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

For and on behalf of Coffey Geotechnics Inc.



Gwangha Roh, Ph.D.



Ramon Miranda, P.Eng.



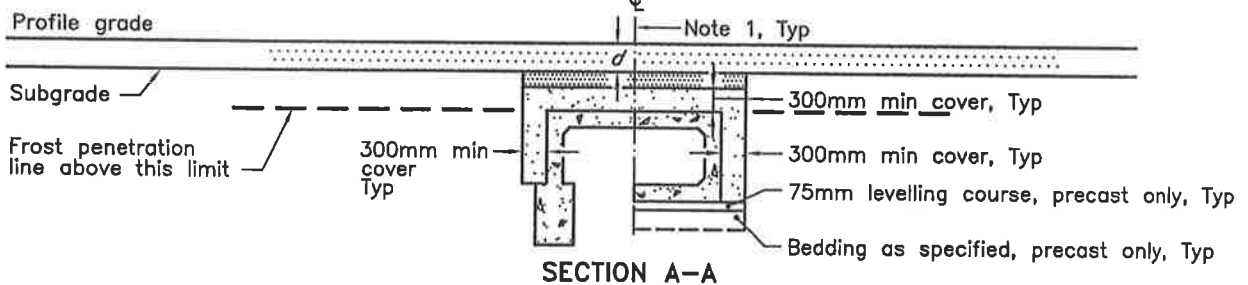
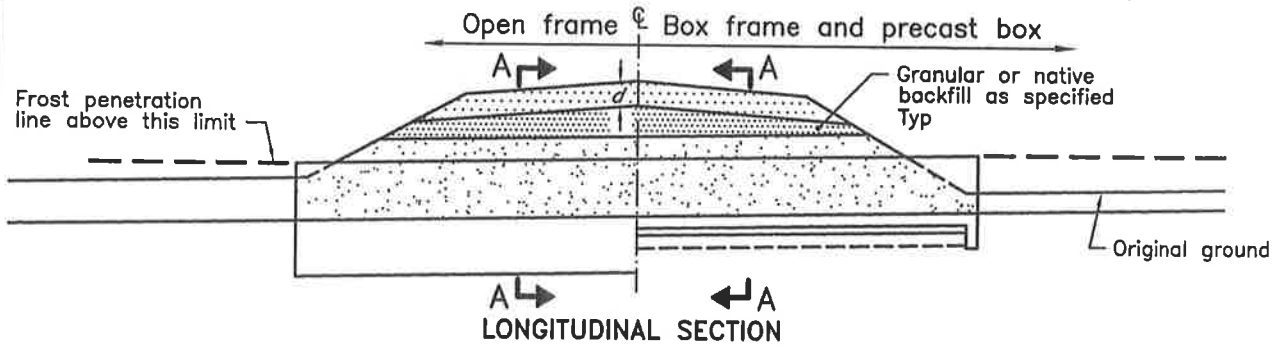
Zuhtu Ozden, P.Eng.



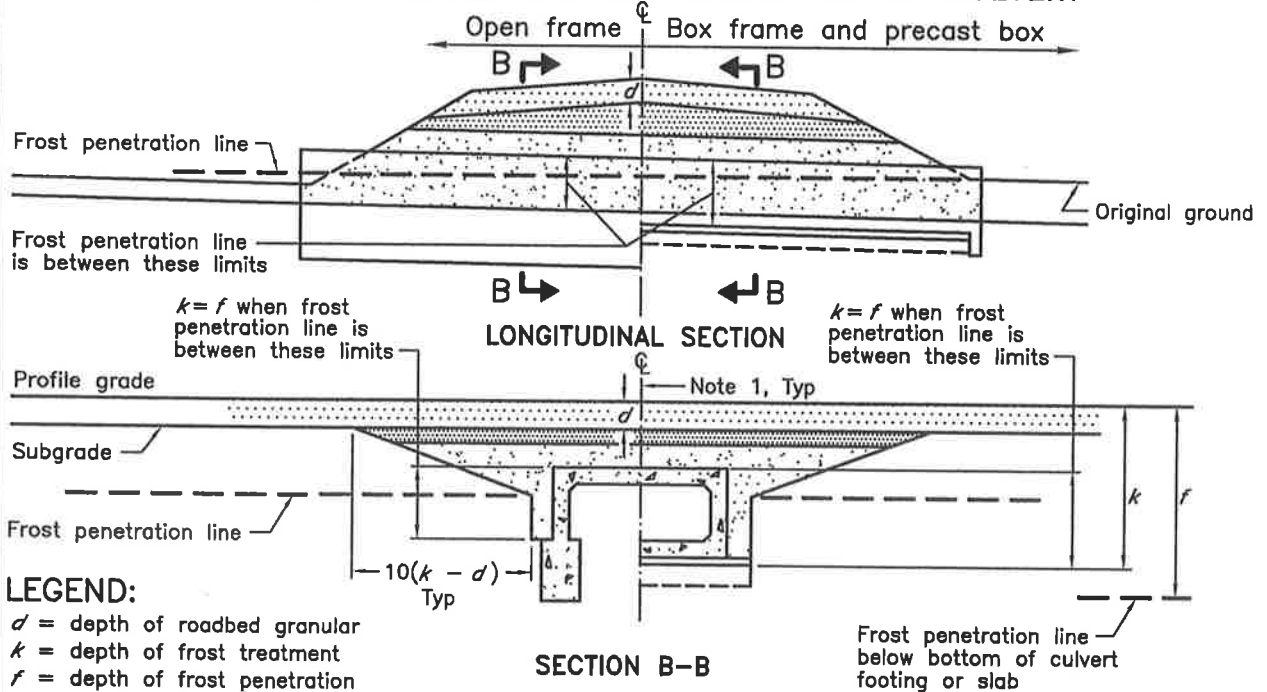
Appendix F

OPSD

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.

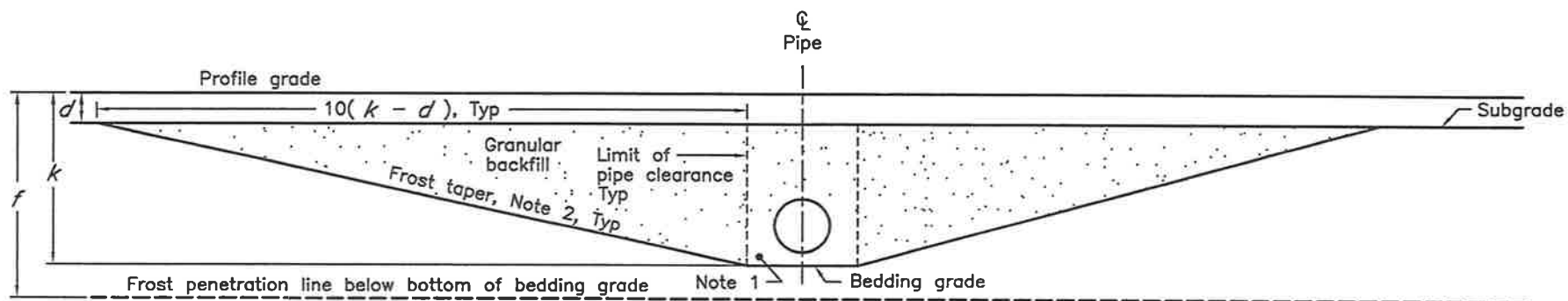
ONTARIO PROVINCIAL STANDARD DRAWING

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:

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

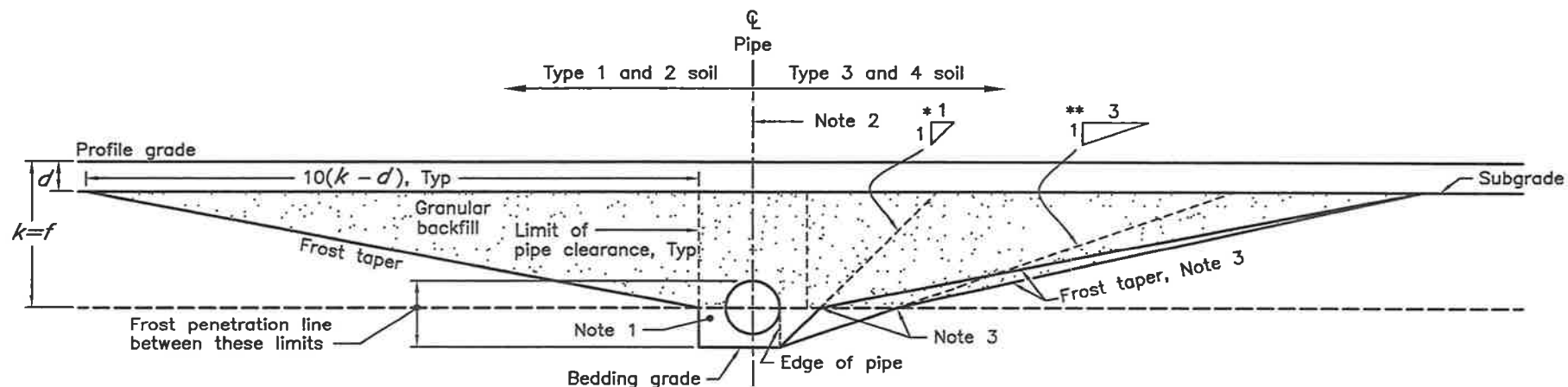
ONTARIO PROVINCIAL STANDARD DRAWING

FROST TREATMENT – PIPE CULVERTS
FROST PENETRATION LINE BELOW
BEDDING GRADE

Nov 2005 Rev 1



OPSD – 803.030



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

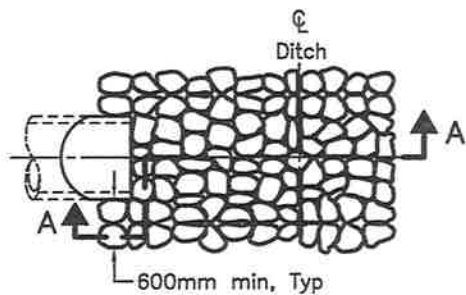
ONTARIO PROVINCIAL STANDARD DRAWING

FROST TREATMENT – PIPE CULVERTS
FROST PENETRATION LINE BETWEEN
TOP OF PIPE AND BEDDING GRADE

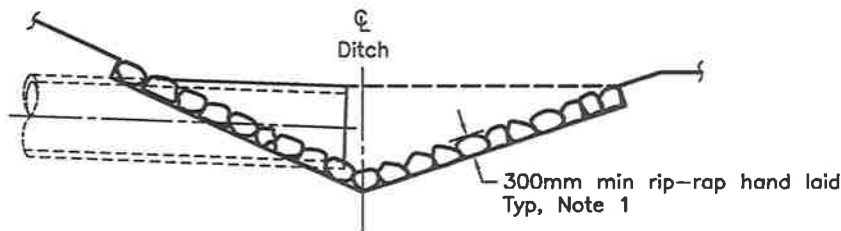
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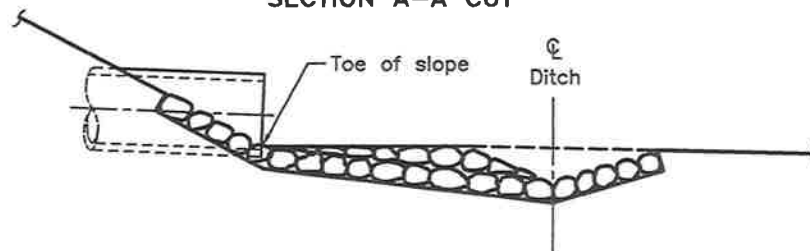
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PLAN
CUT OR FILL

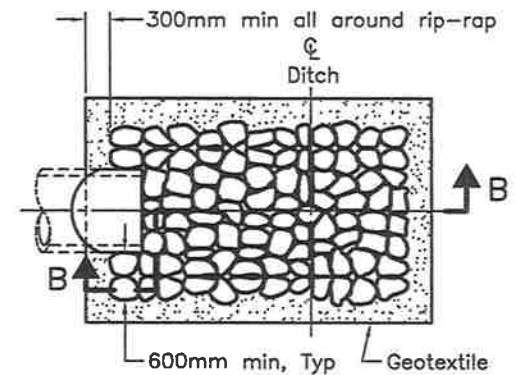
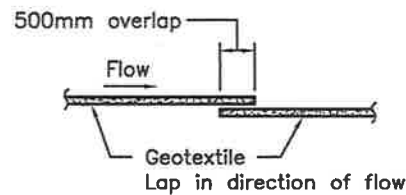


SECTION A-A CUT

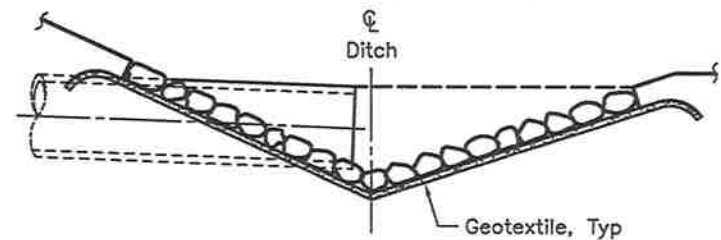


SECTION A-A FILL

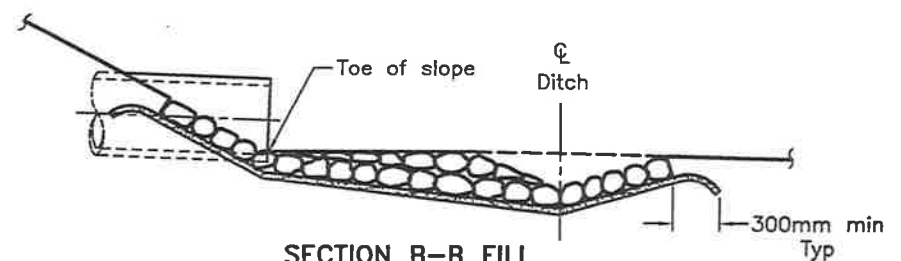
TYPE A - WITHOUT GEOTEXTILE



PLAN
CUT OR FILL



SECTION B-B CUT



SECTION B-B FILL

TYPE B - WITH GEOTEXTILE

NOTES:

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

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2007

Rev 1

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

OPSD 810.010



Appendix G

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Coffey Geotechnics Inc. (Coffey) at the time of preparation. Unless otherwise agreed in writing by Coffey, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

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

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

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

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