

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
PROPOSED CULVERT EXTENSION
AT HIGHWAY 26 AND MOSLEY STREET,
WASAGA BEACH, ONTARIO
G.W.P. 630-91-00, SITE 30-520C
GEOCRES 41A-214**

Delcan Corporation

TRANETOB01232AA-AC
February 18, 2010

February 18, 2010

Delcan Coperation
625 Cochrane Drive, Suite 500
Markham, Ontario
L3R 9R9

Attention: Mr. Sam Dinatolo, P. Eng.

Dear Sir:

RE: Foundation Investigation and Design Report, Proposed Mosley Street Culvert Extension at Highway 26, Wasaga Beach, Ontario, G.W.P. 630-91-00, SITE 30-520C, GEOCRETS 41A-214

Please find attached the Foundation Investigation and Design Reports 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 CULVERT EXTENSION
AT HIGHWAY 26 AND MOSLEY STREET,
WASAGA BEACH, ONTARIO
G.W.P. 630-91-00, SITE 30-520C
GEOCRE 41A-214**

Delcan Coperation

Project: TRANETOB01232AA-AC
February 18, 2010

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**FOUNDATION INVESTIGATION REPORT
PROPOSED MOSLEY STREET CULVERT EXTENSION AT HIGHWAY 26
WASAGA BEACH, ONTARIO
G.W.P. 630-91-00, SITE 30-520C**

1 INTRODUCTION

As part of the realignment of Highway 26, from the Town of Wasaga Beach to Collingwood, Coffey Geotechnics Inc. (Coffey) was retained by Delcan Corporation (Delcan) to carry out a foundation investigation at the site of proposed Mosley Street culvert extension (Site 30-520C) and the associated retaining wall in the Town of Wasaga Beach, Ontario.

The existing structure is an about 71.9 m long concrete box culvert (4267x2438 mm inner dimension) under the junction of the existing Highway 26 and Mosley Street. The 24.5 m long culvert extension is planned toward the south end (inlet) of the culvert.

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 location of the Mosley Street Culvert falls within Nottawasaga Basin which belongs to the physiographic region of Simcoe Lowlands (The Physiography of Southern Ontario, Chapman and Putnam, 1984). Nottawasaga Basin is named for the river which drains it. For the most part, the basin was one time part of the floor of Lake Algonquin, therefore, its surface beds are therefore deposits of deltaic lacustrine origin and not glacial outwash.

According to the Quaternary Geology of Ontario, Collingwood-Nottawasaga Area (Map P.919) and Quaternary Geology of Ontario, Southern Sheet (Map P.2715), the project area lies between two major units of paleozoic deposits, one of which, on the west side, is a glaciolacustrine deposit of gravel, sand, silt and clay of post-Nippissing Age. Till deposit that correlates with the Newmarket Till deposited during the Port Bruce Stadial was mapped on the east side. This till deposit, which consists predominantly sand silt to silt, is rich in clasts and often high in total matrix carbonate content.

Bedrock from the Middle and Upper Ordovician and Lower and Middle Silurian ages underlie the project area (Bedrock Geology of Ontario, Southern Sheet, Map 2544). Bedrock may consists of limestone dolostone, shale, arkose and sandstone. The topography of bedrock is said to dip approximately 8 m per kilometer southwesterly.

The topography at the site is relatively flat. No significant signs of instability or erosion of the existing embankment at culvert location were identified at the time of our investigation.

3 FIELD AND LABORATORY WORK

The fieldwork for this project was conducted on September 24 and 25, 2009. Four (4) boreholes (Boreholes 101, 102, 103 and 104) were drilled and sampled for the proposed culvert extension at the existing culvert inlet area (south end of the culvert) to the proposed depth of 10.5 m. The locations of the boreholes at the sites are given on the Borehole Location Plan in Drawing No 1.

Table 3.1: Borehole Locations and Drilling Depths

Borehole No.	Coordinate (Northing / Easting)	Existing Ground Surface Elevation (m)	Depth of Borehole Below Existing Ground Surface (m)	Piezometer
101	4925343.8/256431.0	183.8	10.5	No
102	4925329.0/256439.0	183.7	10.5	Yes
103	4925327.7/256427.1	183.5	10.5	No
104	4925320.9/256440.4	183.5	10.5	No

The boreholes were advanced using a track-mounted drilling rig owned and operated by Walker Drilling Limited of Barrie, Ontario, under the full-time supervision of a technical staff (Mr. Gem Jiang, EIT) from Coffey. These boreholes were advanced using continuous flight hollow-stem augers.

Sampling in the boreholes 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).

Dynamic Cone Penetration Tests (DCPT) were performed from the ground surface adjacent to Boreholes 101, 102, and 103, as well as from the bottom of Boreholes 102 and 104 to refusal. 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 cases of soil back up at the bottom of the boreholes during the drilling) affect the SPT results.

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 by Delcan.

Water level observations in the open boreholes were made during the drilling and at completion of each borehole.

A piezometer was installed to a depth of 10.0 m in Borehole 102 to determine the groundwater levels over a prolonged period of time, without interference from surface water.

Upon completion, each borehole was backfilled with bentonite/cement mixture, as per MTO procedures. The piezometer in Borehole 102 was not decommissioned, as it may provide useful information prior and/or during the construction of the culvert extension. The decommissioning should be carried out during the construction.

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, Atterberg Limits tests and grain size analyses, was performed on selected representative samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets.

In 2002 – 2003, Golder Associates Limited (Golder) carried out a geotechnical investigation at the site of the proposed culvert extension. The findings of the investigation were presented in a report entitled "Foundation Investigation Report, Proposed New Culverts, Highway 26, G.W.P. 630-91-00, Agreement Number 3005-A-000164", dated February 2006. The investigation included two boreholes at the proposed Mosley Street Culvert Extension site (Boreholes 7 and 8). The boreholes put down by Golder at the site were used to supplement the boreholes by Coffey at this site. The locations of these boreholes are shown on the Borehole Location Plan, Drawing No 1.

4 SUMMARIZED SUBSURFACE CONDITIONS

Boreholes 101, 102, 103 and 104 were advanced at the proposed culvert extension site, adjacent to the south end of the exiting Mosley Street Culvert. Boreholes 101 and 102 were put down in the middle of the proposed extension (about 6 and 11 m away from the existing culvert inlet southerly on each side of the proposed culvert extension) from the existing ground surface (El. 183.7 and 183.8 m) level, while Boreholes 103 and 104 were drilled at the end of the proposed extension (about 25 m away from the existing culvert inlet southerly on each side of the proposed culvert extension) also from the existing ground surface (El. 183.5 m) level.

All boreholes drilled at the site encountered a 0.2 to 0.3 m thick topsoil at the ground surface. Underlying the topsoil, Boreholes 101, 102 and 103 contacted a 0.7 to 1.1 m thick surficial silty sand. Below this silty sand cap, Boreholes 101, 102 and 103 encountered a 0.4 to 1.0 m thick silty clay layer. Below this silty clay in Boreholes 101, 102 and 103 and underlying the topsoil in Borehole 104, all boreholes contacted a sandy silt to silty sand glacial till deposit at depths of 0.3 to 2.0 m (or El. 181.7 to 183.2 m). The glacial till deposit was found to extend to a depth of 4.4 m or El. 179.1 to 179.4 m and is further underlain by a silty fine sand deposit. The boreholes were terminated within this deposit at the proposed borehole depth of 10.5 m (El. 173.0 to 173.3 m). Some soil back-up (about 0.2 to 0.3 m) due to the hydrostatic uplift within the silty fine sand deposit was noted during the drilling.

Dynamic Cone Penetration tests (DCPT) were performed from the bottom of Boreholes 102 and 104 and refusal was encountered at 14.2 m (or El. 169.5 m) and 11.3 m (or El. 172.2 m), respectively. DCPT from the original ground surface adjacent to the drilled boreholes was also carried out in Boreholes 101, 102 and 103 and refusal was encountered at depths of 3.3 m, 5.8 m and 4.9 m or El., 180.5, 177.9 and 178.6 m, respectively.

Subsurface conditions at the site are discussed in the following sections. Details of the stratigraphy encountered in the boreholes are presented on the Records of Borehole Sheets (including boreholes by Golder Associates Limited). The locations of the boreholes along with an inferred subsurface profile (based on Coffey boreholes) are given in Drawing No. 1. Photographs of the proposed culvert extension site are included in Appendix C. The following paragraphs are only meant to complement these data. Appropriate portions of the previous investigation report for this proposed culvert extension (prepared by Golder in 2006) is also included in Appendix D of this report.

4.1 Topsoil

A layer of topsoil ranging from 0.2 to 0.3 m in thickness was contacted in all boreholes at ground surface.

4.2 Surficial Silty Sand

Underlying the topsoil in Boreholes 101, 102 and 103, a 0.7 to 1.1 m thick surficial silty sand was encountered at depths of 0.2 to 0.3 m below the ground surface and found to extend to depths of 0.9 to 1.4 m or El. 182.1 to 182.9 m. This deposit contains traces of rootlets, as well as clay and gravel size particles.

The surficial silty sand is a granular (non-cohesive) soil. The grain-size distribution of a sample recovered from the deposit is presented in Figure B-1, in Appendix B which indicates following grain-size distribution:

Gravel:	0 %
Sand:	63 %
Silt:	29 %
Clay:	8 %

Standard Penetration Tests conducted in the silty sand yielded N-values between 4 to 14 blows/0.3 m. These results indicate that the relative density of the silty sand can be described as very loose to compact.

4.3 Silty Clay

Below the surficial silty sand, Boreholes 101, 102 and 103 contacted a 0.4 to 1.0 m thick silty clay deposit, extending to El. 181.7 to 181.9 m. This deposit contains traces to some sand and gravel, traces of rootlets, occasional silt pockets and sand seams.

The grain-size distribution of two samples from this deposit was determined in the laboratory and the resulting curves are given in Figure B-2 in Appendix B. The following grain-size distribution is indicated.

Gravel:	6-15 %
Sand:	5-17 %
Silt:	36-38 %
Clay:	41-42 %

The results of Atterberg Limits tests performed on two samples recovered from this deposit are given on the individual Record of Borehole Sheets and also on the plasticity chart presented in Figure B-3 in Appendix B. The following index values were obtained:

Liquid Limit:	35-36 %
Plastic Limit:	17-18 %
Plasticity Index:	17-18

These results are characteristic of cohesive soils of low to intermediate plasticity.

Standard Penetration tests, performed in this cohesive deposit, yielded N-values of between 3 and 13 blows/0.3 m, indicating soft to stiff consistency.

4.4 Sandy Silt to Silty Sand Till

Underlying the silty clay in Boreholes 101, 102 and 103, and the topsoil in Borehole 104, a glacial deposit consisting of a heterogeneous mixture of sandy silt to silty sand with traces to some gravel and clay was encountered at depths ranging from 0.3 to 2.0 m or El. 181.7 to 183.2 m. This sandy silt to silty sand till deposit was found to extend to a depth of 4.4 m below the ground surface or El. 179.1 to 179.4 m. Grain-size analysis conducted on five samples, retrieved from this deposit, gave the following grain size distribution (see Figure B-4 in Appendix B):

Gravel:	3-11 %
Sand:	36-49 %
Silt:	34-52 %
Clay:	7-10 %

N-values obtained from Standard Penetration Tests performed in this granular (i.e. non-cohesive) deposit are between 3 and 54 blows/0.3 m. This indicates that the deposit is in a very loose to very dense compactness condition, but typically compact to dense.

It should also be mentioned that owing to their mode of deposition, the presence of cobbles and boulders should always be anticipated in the glacial till deposits.

4.5 Silty Fine Sand

All boreholes contacted a silty fine sand deposit below the glacial till at a depth of 4.4 m or at El. 179.1 to 179.4 m. The boreholes were terminated within this deposit at a depth of 10.5 m or El. 173.0 to 173.3 m.

The silty fine sand was identified as a dilatant material.

The grain-size distribution of four samples from this deposit was determined in the laboratory and the resulting curves are given in Figure B-5 in Appendix B. The following grain-size distribution is indicated.

Gravel:	0 %
---------	-----

Sand:	57-89 %
Silt:	10-40 %
Clay:	1-5 %

N-values obtained from Standard Penetration Tests performed in this granular (i.e. non-cohesive) deposit are between 4 and 55 blows/0.3 m. This indicates that the deposit is in a very loose to very dense compactness condition, but typically compact to dense.

It is noted that an about 0.2 to 0.3 m soil back-up into the hollow stem augers was recorded during the investigation within the deposit where very low N-values were recorded, due to the hydrostatic uplift and therefore, these recorded low N-values may not be reliable.

4.6 Groundwater Conditions

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of each borehole. One piezometer was installed in Borehole 102 to a depth of 10.0 m. The observations made in the boreholes are summarized in Table 4.5.1 and presented on the Record of Borehole Sheets in Appendix A.

Table 4.6.1.: Groundwater Conditions

Borehole	Ground Elevation (m)	Depth / Elevation of the Tip of Piezometer (m)	Date	Water Level Depth / Elevation (m)
101	183.8	-	Upon completion	4.6/179.2
102	183.7	10.0/173.7	A day after installation	4.4/179.3
103	183.5	-	Upon completion	4.0/179.5
104	183.5	-	Upon completion	4.2/179.3

In Boreholes 7 and 8 advanced by Golder Associates, the water level was measured in July and August 2003 at about El. 179 m.

Based on these results, the ground water level at the time of our investigation and in July and August 2003 was between El. 179.0 and 179.5 m, at the site.

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. In addition a perched water table may occur due to accumulation of the surface water in the upper more pervious zones of the soil overlying the less pervious silty clay and the dense to very dense till. As well, the groundwater can be expected to be influenced by the water level in the existing creek.

For and on behalf of Coffey Geotechnics Inc.

Gwangha Roh, Ph.D.

Ramon Miranda, P.Eng
Manager, Transportation Division



Zuhtu Ozden, P.Eng.
Senior Principal



Drawing

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: 630-91-00

HIGHWAY 26 REALIGNMENT
MOSLEY ST. CULVERT EXTENSION
BOREHOLE LOCATION PLAN
AND SOIL STRATA



SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

- Borehole & Cone (Coffey)
- Borehole (Golder)
- 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.	ELEVATION	NORTHING	EASTING
101	183.8	4925343.8	256431.1
102	183.7	4925329.0	256439.0
103	183.5	4925327.7	256427.1
104	183.5	4925321.0	256440.4
7	183.7	4925346.6	256425.5
8	184.5	4925353.1	256449.3

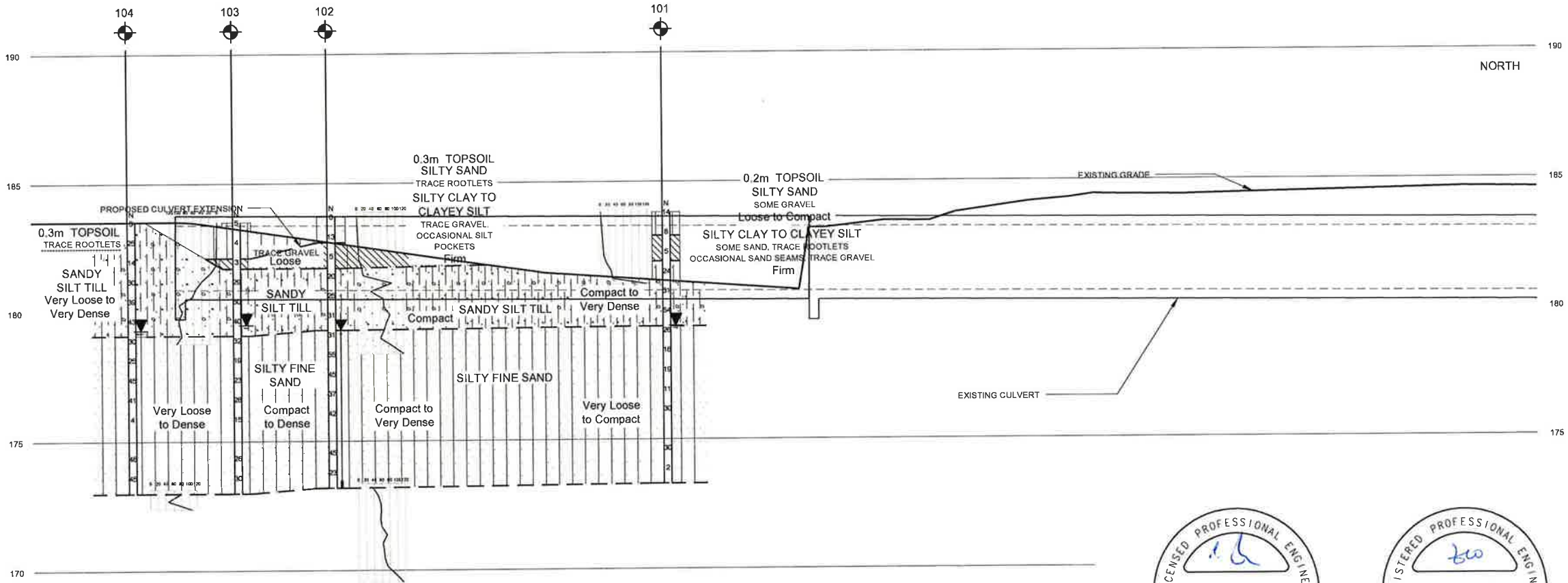
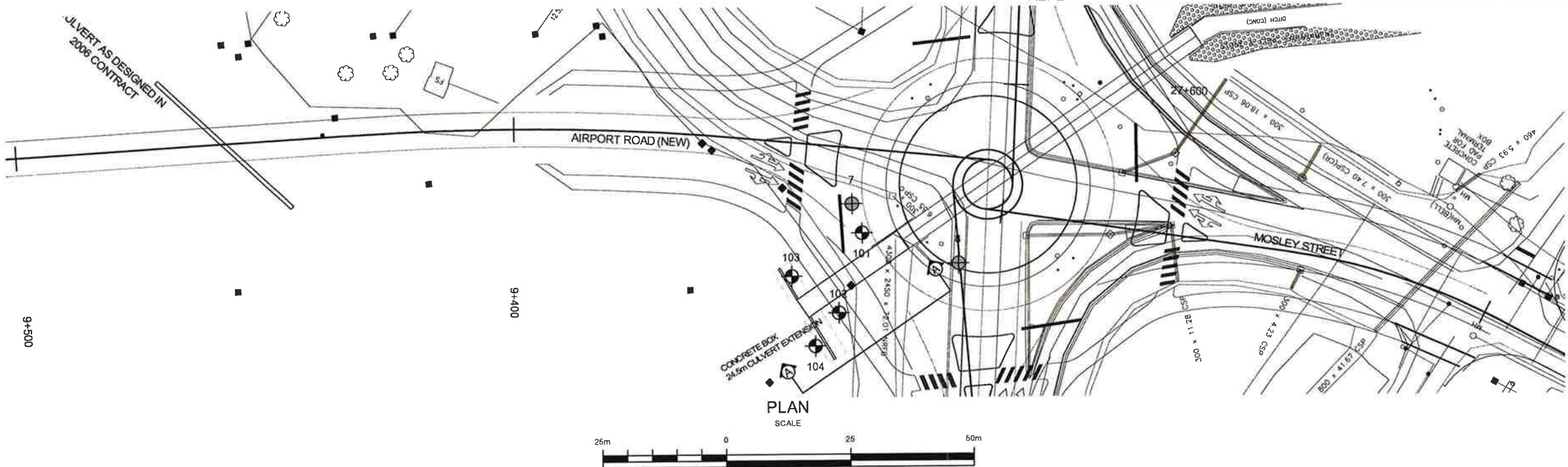
-NOTE-

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. 41A-214	TRANETO01232AA	DIST
SUBMD	CHECKED	DATE Jan. 27, 2010
DRAWN	PHK	CHECKED RM
APPROVED	ZO	DWG
1		



SECTION A-A
HORIZONTAL SCALE



Appendix A

Borehole Logs

TRANETOBO1232AA

RECORD OF BOREHOLE No 101

1 OF 1

METRIC

GWP 630-91-00 LOCATION Intersection of Hwy 26 and Mosley Street ORIGINATED BY G.J.
DIST HWY HWY 26 BOREHOLE TYPE Hollow Stem Augers, Dynamic Cone Penetration Test (DCPT) COMPILED BY W.C.
DATUM Geodetic DATE 9/24/2009 9/25/2009 CHECKED BY Z.O.

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						
183.8 0.0	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100					
	0.2 m TOPSOIL SILTY SAND some gravel, brown loose to compact, moist		1	SS	14									
182.9 0.9	SILTY CLAY some sand, tr. gravel, tr. rootlets occ. sand seams, brown, firm		2	SS	8									
181.9 1.9	SANDY SILT TO SILTY SAND TILL grey, compact to v. dense		3	SS	5									15 5 38 42
			4	SS	24									
			5	SS	31									
			6	SS	54									11 42 39 8
179.4 4.4	SILTY FINE SAND grey, loose to compact, dilatant, wet		7	SS	26									0.2 m soil back up (N-value may not be reliable) wet spoon 0 57 40 3
			8	SS	18									
			9	SS	19									
			10	SS	11									
			11	SS	30									
			12	SS	30									
			13	SS	2*									*0.3 m soil back up (N-value is not reliable)
173.3 10.5	End of Borehole. Borehole caved in @ 4.4 m. Water level @ 4.6 m (not stabilized)* upon completion. Dynamic Cone Penetration Test (DCPT) performed adjacent to the borehole from ground surface to 3.3 m.													

+ 3, X 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

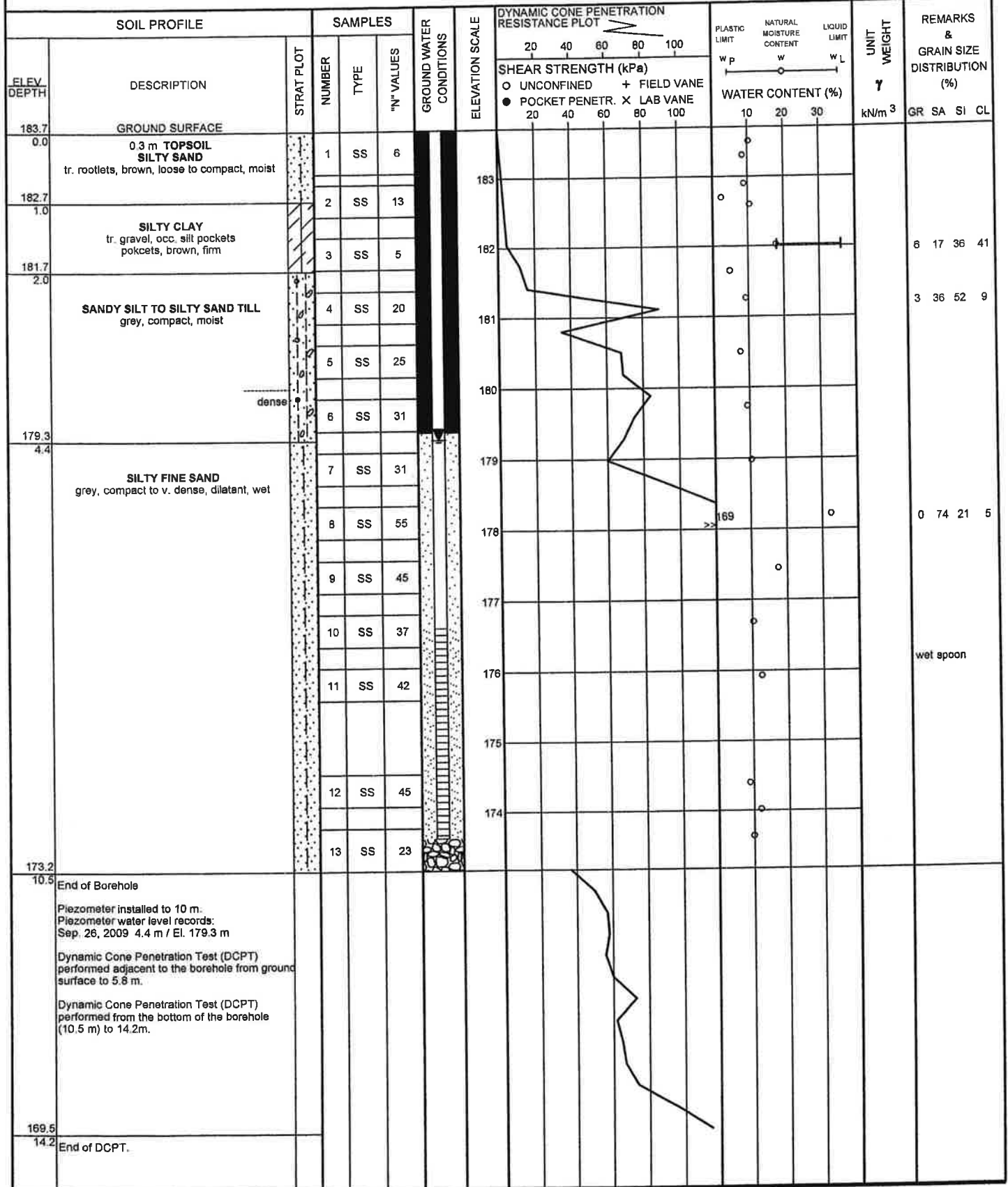
TRANETOB01232AA

RECORD OF BOREHOLE No 102

1 OF 1

METRIC

GWP 630-91-00 LOCATION Intersection of Hwy 26 and Mosley Street ORIGINATED BY G.J.
DIST HWY HWY 26 BOREHOLE TYPE Hollow Stem Augers, Dynamic Cone Penetration Test (DCPT) COMPILED BY W.C.
DATUM Geodetic DATE 9/25/2009 CHECKED BY Z.O.



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

TRANETO801232AA

RECORD OF BOREHOLE No 103

1 OF 1

METRIC

GWP 630-91-00 LOCATION Intersection of Hwy 26 and Mosley Street ORIGINATED BY G.J.
 DIST HWY HWY 26 BOREHOLE TYPE Hollow Stem Augers, Dynamic Cone Penetration Test (DCPT) COMPILED BY W.C.
 DATUM Geodetic DATE 9/24/2009 9/25/2009 CHECKED BY Z.O.

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)							
								○ UNCONFINED + FIELD VANE							
								● POCKET PENETR. X LAB VANE							
								20 40 60 80 100							
								20 40 60 80 100							
183.5	GROUND SURFACE														
0.0	0.3 m TOPSOIL SILTY SAND tr. rootlets, tr. gravel brown, v. loose, moist		1	SS	5		183								0 63 29 8
			2	SS	4										
182.1							182								
1.4	SILTY CLAY tr. gravel, brown, soft, moist		3	SS	3										
181.7															
1.8	SANDY SILT TO SILTY SAND TILL grey, v. loose to dense, dilatant		4	SS	29		181								auger grinding @ 3.0 m
			5	SS	50										
			6	SS	40		180								10 49 34 7
179.1							179								
4.4			7	SS	32										
	SILTY FINE SAND grey, compact to dense, dilatant, wet		8	SS	19		178								wet spoon 0.2 m soil back up (N-value may not be reliable)
			9	SS	23										
			10	SS	28		177								
			11	SS	15		176								
							175								
			12	SS	26		174								0 89 10 1
			13	SS	30										
173.0							173								
10.5	End of borehole Borehole caved in @ 4.0 m. Water level @ 4.0 m upon completion (not stabilized)*. Dynamic Cone Penetration Test (DCPT) performed adjacent to the borehole from ground surface up to 4.9 m.														

TRANETOB01232AA

RECORD OF BOREHOLE No 104

1 OF 1

METRIC

GWP 630-91-00 LOCATION Intersection of Hwy 26 and Mosley Street ORIGINATED BY G.J.
DIST HWY HWY 26 BOREHOLE TYPE Hollow Stem Augers COMPILED BY W.C.
DATUM Geodetic DATE 9/24/2009 CHECKED BY Z.O.

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)							
183.5 0.0	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30					
	0.3 m TOPSOIL	tr. rootlets	1	SS	5		183								
	SANDY SILT TO SILTY SAND TILL grey, compact to dense		2	SS	25		182								9 45 36 10
			3	SS	14		181								
			4	SS	30		180								
			5	SS	39		179								
			6	SS	43		178								4 48 40 8
179.1 4.4	SILTY FINE SAND grey, loose to dense, dilatant, wet	moist wet	7	SS	30		177								0 83 15 2
			8	SS	25		176								wet spoon *0.3 m soil back up (N-value may not be reliable)
			9	SS	45		175								
			10	SS	41		174								
			11	SS	4*		173								
			12	SS	48										
			13	SS	45										
173.0 10.5	End of borehole. Water level @ 4.2 m upon completion (not stabilized)*.														
172.2 11.3	Dynamic Cone Penetration Test (DCPT) performed from the bottom of the borehole (10.5 m) to 11.3 m.														
	End of DCPT														

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

PROJECT 001-3232-4-4

RECORD OF BOREHOLE No 7

1 OF 1

METRIC

G.W.P. 630-91-00

LOCATION

N 4925346.6 E 256425.5

ORIGINATED BY MR

DIST 30 HWY 26

BOREHOLE TYPE POWER AUGER (HOLLOW STEM)

COMPILED BY WDF

DATUM GEODETIC

DATE

8 July 2003

CHECKED BY AMH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								UNCONFINED + FIELD VANE								
								QUICK TRIAXIAL x LAB VANE								
183.69						20	40	60	80	100	W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L			
0.00	TOPSOIL, silty Brown															
0.23	FILL, sandy silt with topsoil layers Loose Brown															
182.78																
0.91	SILTY CLAY, trace sand, trace gravel Stiff Mottled brown and grey		1	SS	8											
181.86																
1.83	SANDY SILT, trace gravel, trace clay, with cobbles (TILL) Compact Brown to Grey at 2.1m		2	SS	11											
180.79																
2.90	SILTY FINE SAND, Very dense Grey															
180.49																
3.20	SANDY SILT, trace gravel, trace clay, with cobbles (TILL) Very dense Grey		3	SS	28											
			4	SS	76											
			5	SS	62											4 45 45 6
179.27																
4.42	SANDY SILT, with silty fine sand layers Dense Grey		6	SS	42											
178.51																
5.18	SILTY FINE SAND, Very dense Grey															
			7	SS	62											
			8	SS	59											0 65 35 0
			9	SS	39											
176.37																
7.32	END OF BOREHOLE															
	Groundwater encountered at elev. 179.12m during drilling July 8, 2003.															
	Groundwater measured at elev. 179.15m July 10, 2003.															
	Groundwater measured at elev. 178.81m July 14, 2003.															
	Groundwater measured at elev. 178.81m July 17, 2003.															
	Groundwater measured at elev. 178.72m Aug. 20, 2003															

ON_MTO 001-3232-4-4.GPJ ON_MOT.GOT 26/11/03 DATA INPUT:

RECORD OF BOREHOLE No 8

1 OF 1

METRIC

PROJECT 001-3232-4-4

G.W.P. 630-91-00

LOCATION

N 4925353.1 ; E 256449.3

ORIGINATED BY MR

DIST 30

HWY 26

BOREHOLE TYPE

POWER AUGER (HOLLOW STEM)

COMPILED BY WDF

DATUM GEODETIC

DATE

10 July 2003

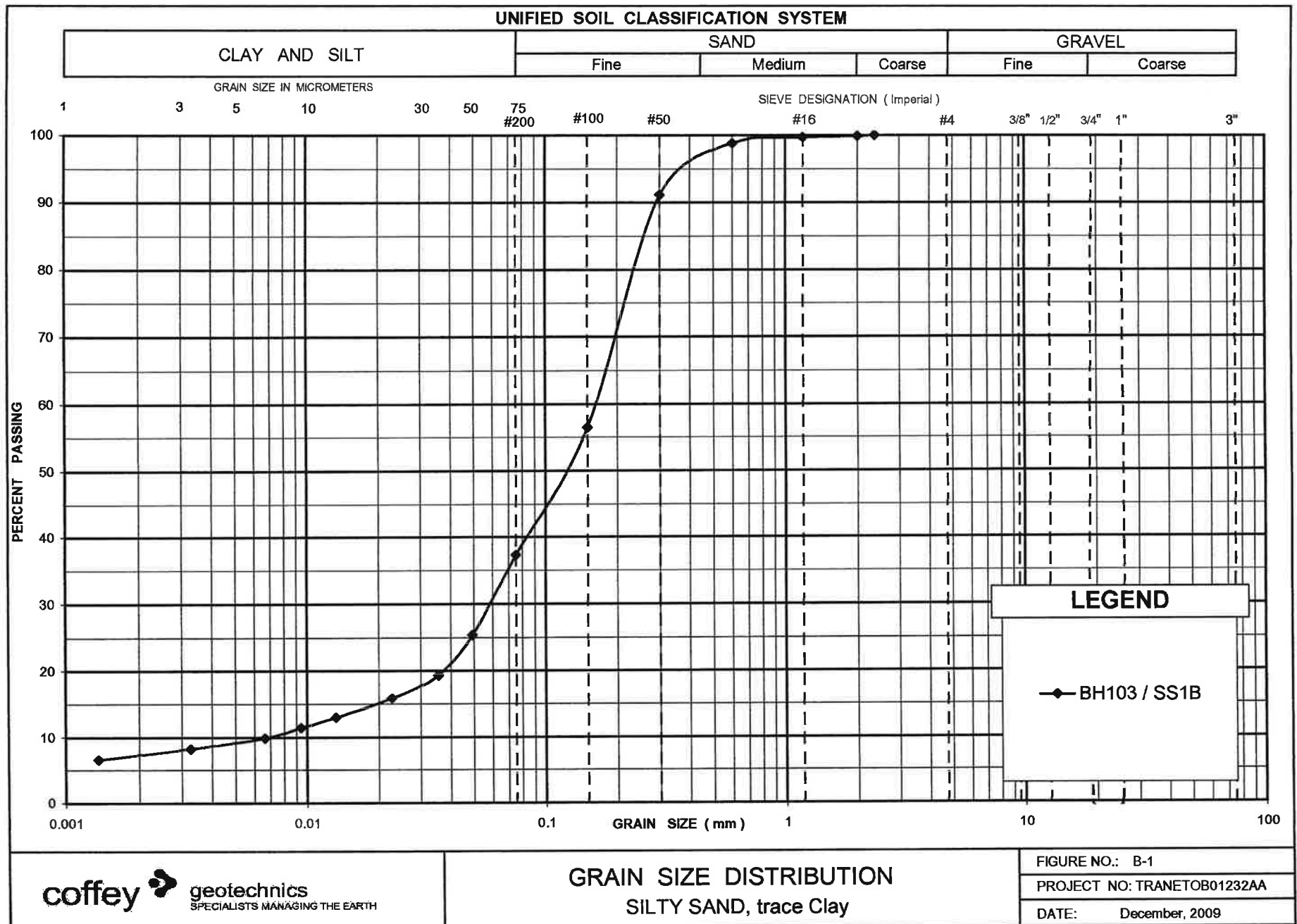
CHECKED BY AMH

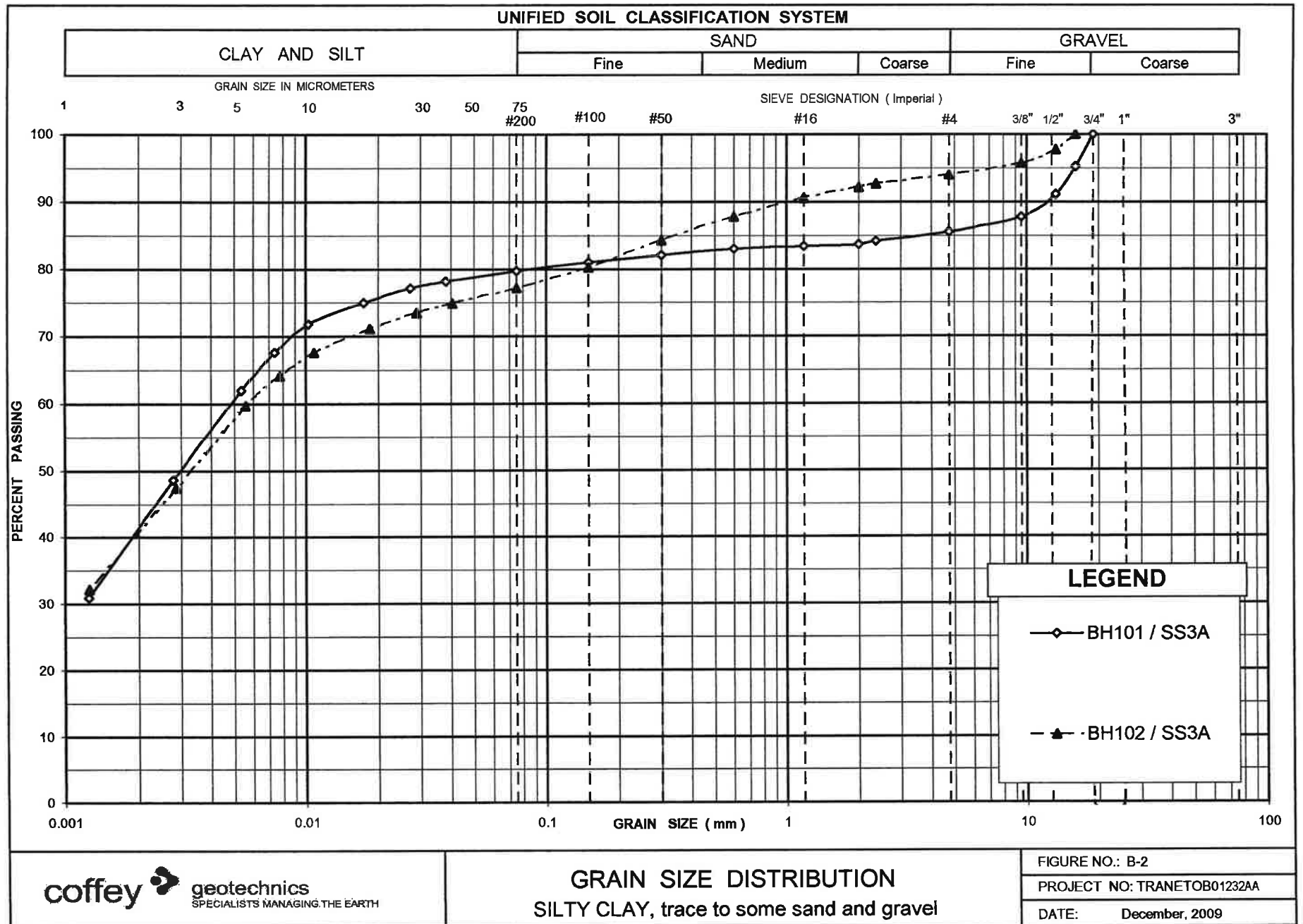
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
184.47								20 40 60 80 100	20 40 60 80 100	10 20 30				
0.05	FILL, sand, medium to coarse (25mm)													
183.91	Brown													
0.56	ASPHALT (25mm)													
	FILL, sand and gravel													
	Brown													
	SANDY SILT, trace gravel, trace clay, with cobbles (TILL)		1	SS	17									
	Compact to very dense													
	Brown to Grey at 2.9m													
			2	SS	16									
			3	SS	90									
			4	SS	55									
			5	SS	100/225mm									
180.36														
4.11	SILTY FINE SAND, Very dense													
180.05	Grey													
4.42	SANDY SILT, trace gravel, with silty fine sand layers, with cobbles (TILL)		6	SS	64									
179.29	Very dense													
	Grey													
5.18	SAND, fine to medium, trace to some silt		7	SS	89									
	Dense to very dense													
	Grey													
			8	SS	53									
			9	SS	42									
177.31														
7.16	SILT, trace fine sand													
7.32	Dense													
	Grey													
	END OF BOREHOLE													
	Groundwater encountered at elev. 179.14m during drilling July 10, 2003.													
	Groundwater measured at elev. 178.89m July 14, 2003.													
	Groundwater measured at elev. 178.89m July 17, 2003.													
	Groundwater measured at elev. 178.56m Aug. 20, 2003													

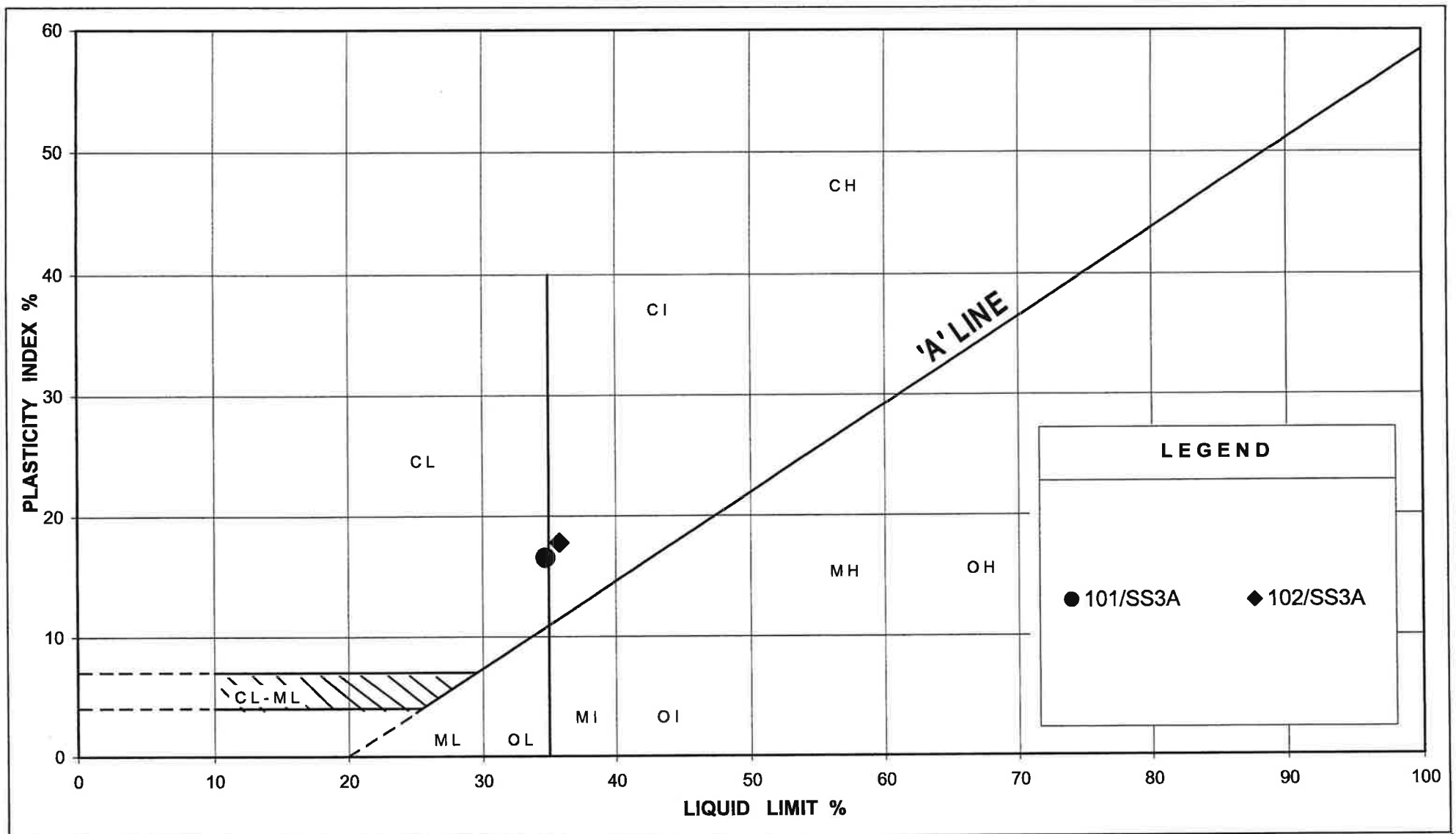
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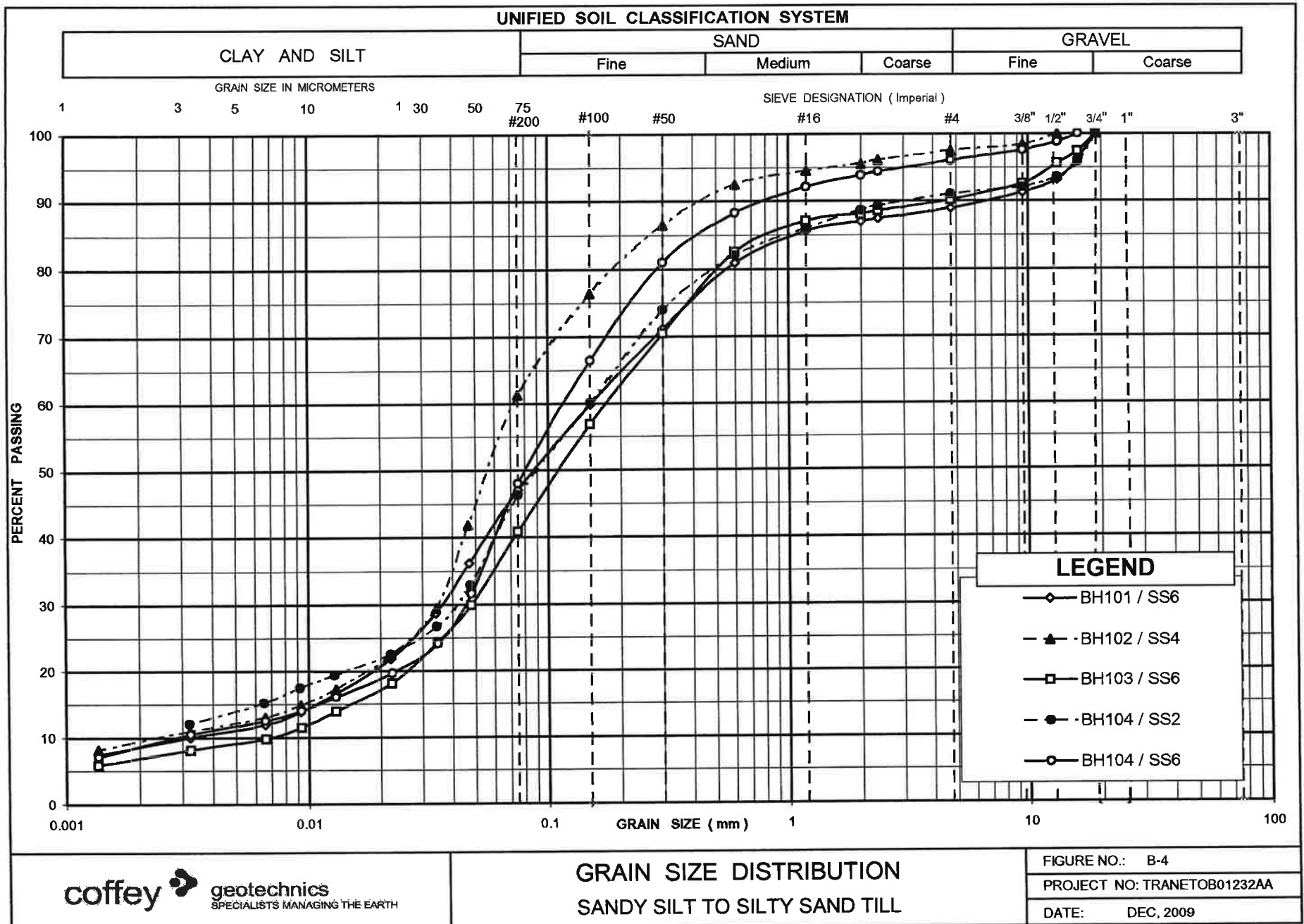
Appendix B

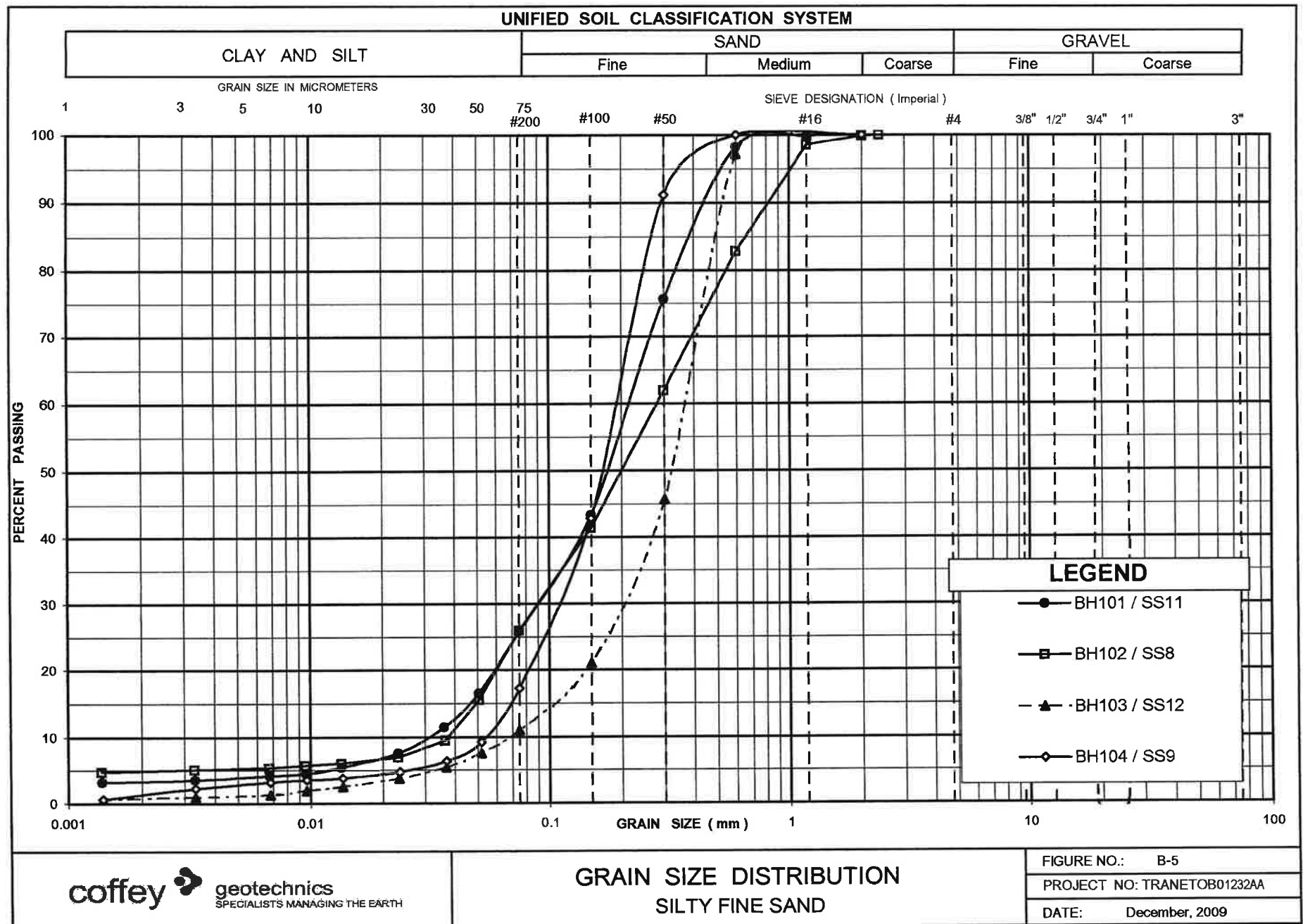
Test Results











Appendix C

Site Photographs



Photograph 1. Retaining wall at the existing culvert inlet and embankment



Photograph 2. Existing culvert inlet and concrete slab



Photograph 3. Mosley Street Culvert (South End), Looking Towards South



Photograph 4. Mosley Street Culvert (South End), Looking Towards East

Appendix D

Golder Associates Foundation Investigation Report

4.1.4 Mosley Street Culvert

Topsoil and Fill

Borehole 7, drilled near the southeast end of the existing Mosley Street culvert, encountered 230 millimetres of silty topsoil over 0.7 metres of loose sandy silt fill materials. The fill had a standard penetration test N value of 8 blows per 300 millimetres penetration and the water content of the fill sample collected from the borehole was about 7 per cent.

Borehole 8 was drilled at the edge of Airport Road near the southwest end of the existing Mosley Street culvert and encountered 25 millimetres of sand over a 25 millimetre thick layer of asphalt. The asphalt was underlain by 510 millimetres of sand and gravel roadbase.

Silty Clay

Borehole 7 encountered a deposit of stiff silty clay beneath the fill materials at elevation 182.8 metres. The deposit was about 0.9 metres thick and contained varying amounts of sand and gravel. The silty clay deposit had standard penetration test N values of 8 and 11 blows per 300 millimetres penetration and water contents of about 22 to 29 per cent.

Sandy Silt Till

Beneath the silty clay in borehole 7 and the roadbase materials in borehole 8, a deposit of compact to very dense sandy silt till was encountered at elevations 181.9 to 183.9 metres. The till deposit was about 2.6 to 4.6 metres thick and extended to between elevations 183.9 and 179.3 metres. A 0.3 metre thick layer of silty fine sand material was encountered within the till in both boreholes. The till had standard penetration test N values between 11 blows per 300 millimetres penetration and 100 blows per 225 millimetres penetration. The water contents of the till samples collected from the boreholes were between about 4 and 6 per cent, with an average of about 10 per cent.

Figure A-3 in Appendix A shows gradation curves for samples recovered from the sandy silt till deposit in boreholes 7 and 8. The deposit consists mainly of sand and silt size material with a trace of gravel and clay size particles. Cobbles and boulders should be expected in the till.

Sandy Silt

A 0.8 metre thick pocket of dense sandy silt was encountered beneath the till deposit in borehole 7. The sandy silt pocket had a standard penetration test N value of 42 blows per 300 millimetres penetration and a water content of about 19 per cent.

Sands

Layers of very dense silty fine sand were encountered within and beneath the till deposit in boreholes 7 and 8. Where fully penetrated, the sand layers were between 0.3 and 2.0 metres thick. Borehole 7 was terminated in the sand deposit at a depth of 7.3 metres, or elevation 176.4 metres, after exploring it for some 2.1 metres. The sand layers had standard penetration test N values of 39 blows per 300 millimetres penetration to 100 blows per 225 millimetres penetration. The water contents of the sand samples ranged between about 7 and 24 per cent, with an average of about 16 per cent.

Figure A-4 in Appendix A shows a gradation curve for a sample recovered from the sand deposit in borehole 7.

A 2 metre thick layer of fine to medium sand was encountered beneath the sandy silt till at elevation 179.3 metres with borehole 8. The sand had standard penetration test N values of 53 and 89 blows per 300 millimetres and water contents of 7 to 18 per cent.

Silt

Borehole 8 encountered and was terminated in a silt deposit. The deposit was explored for 0.1 metres before terminating the borehole at a depth of 7.3 metres below ground surface, or elevation 177.2 metres. The silt had a standard penetration test N value of 42 blows per 300 millimetres penetration and a water content of about 10 per cent.

4.2 Groundwater Conditions

Water levels were noted in the open boreholes during drilling and piezometers were installed in selected boreholes upon completion of the drilling operations. The water levels encountered in the boreholes during drilling and measured in August 2002 and July and August 2003 are summarized in the following table together with the relevant surface water elevation at the existing Service Road and Mosley Street culverts. There are no existing water courses at the other two culvert locations and it should be noted that the groundwater levels are subject to seasonal fluctuations.

BOREHOLE	GROUND	SURFACE	ENCOUNTERED	<u>MEASURED GROUNDWATER ELEVATION (m)</u>				
	SURFACE	WATER	GROUNDWATER	2002		2003		
	<u>ELEVATION</u>	<u>ELEVATION</u>	<u>ELEVATION</u>	<u>August 22</u>	<u>August 27</u>	<u>July 14</u>	<u>July 17</u>	<u>August 20</u>
	(m)	(m)	(m)					
Service Road								
1	194.31	191.47	Dry	185.47	189.86	-	-	190.41
2	193.77		Dry	-	-	-	-	-
28+420								
3	190.37	N/A*	183.36	182.69	182.45	-	-	-
4	190.38		181.85	182.91	182.88	-	-	-
28+050								
5	188.75	N/A*	180.22	-	181.37	-	-	181.50
6	188.95		181.74	-	-	-	-	181.33
Mosley Street								
7	183.69	180.77	179.12	-	-	178.81	178.81	178.72
8	184.47		179.14	-	-	178.89	178.89	178.56

* No existing channel at proposed culvert location.

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 STRUCTURAL FEATURES AND/OR STRENGTH.

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_l	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
γ_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
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_p) / I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(w_L - w) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
PROPOSED CULVERT EXTENSION
AT HIGHWAY 26 AND MOSLEY STREET,
WASAGA BEACH, ONTARIO
G.W.P. 630-91-00, SITE 30-520C
GEOCRES 41A-214**

Delcan Cooperation

Project: TRANETOB01232AA-AC
February 18, 2010

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Appendices

Appendix F: OPSD

Appendix G: Limitations of Report

**FOUNDATION DESIGN REPORT
PROPOSED MOSLEY STREET CULVERT EXTENSION AT HIGHWAY 26
WASAGA BEACH, ONTARIO
G.W.P. 630-91-00, SITE 30-520C**

5 DISCUSSION AND RECOMMENDATIONS

The existing creek at the site flows northerly towards Wasaga Beach across the junction of the existing Highway 26 with Mosley Street (east side) and Airport Road (west side) through the existing 71.9 m long, 4.27 x 2.44 m concrete box culvert (at Station 27+818). The existing culvert, located in between Airport Road and Highway 26, will be extended at the south end by another 24.5 m, bringing the total culvert length to 96.4 m.

Four boreholes (Boreholes 101 through 104) were drilled at the culvert extension site. Boreholes 101, 102 and 103 showed below some topsoil, and a 0.7 to 1.1 m thick surficial silty sand, the presence of a 0.4 to 1.0 m thick silty clay deposit at depths of 0.9 to 1.4 m or El. 182.1 to 182.9 m. Underlying the surficial soils in Boreholes 101, 102 and 103 at El. 181.9 to 181.7 m and immediate below the veneer of topsoil in Borehole 104 at a depth of 0.3 m below the ground surface or at El. 183.4 m, a major glacial deposit consisting of sandy silt to silty sand till was contacted. The till was found to extend to a depth of 4.4 m or to El. 179.4 to 179.1 m and is underlain by a deposit of silty fine sand. The boreholes were terminated in this unit at a depth of 10.5 m or El. 173.3 to 173.0 m. The groundwater table at the time of our investigation was measured at about El. 179.5 m and 179.2 m, but would be subject to fluctuations.

We understand that the invert of the concrete box culvert will be at about 180.5 m at the existing inlet, rising to about El. 181.1 m at the new inlet (i.e. 24.5 m further south of the existing inlet). At these elevations all available boreholes (including Golder's boreholes) show the presence of sandy silt to silt sand till. Golder boreholes also indicate the presence of silty fine sand layers within the till deposit. From the recorded N-values (i.e. 20 to in excess of 50 blows/0.3 m) the sandy silt to silty sand till is considered to be in a compact to very dense condition.

5.1 Available Options

The existing culvert is a closed bottom 4.27 x 2.44 m concrete box type structure.

In general, the choice of culvert type depends on such parameters as the initial cost, service life, maintenance costs, hydraulic performance, ease of construction, salvageability and structural strength of material used, fish passage, etc. The material choice includes corrugated steel, concrete and plastic, while the Ministry may entertain alternative materials, as well. Some of the factors considered in making an appropriate choice of material type include:

- Steel and plastic have the advantage of simpler and quicker construction, especially in remote areas, while steel has the added advantage of often being at least partly salvageable after being washed out.

- A well designed concrete culvert is extremely durable under a wide range of conditions.
- Precast concrete and smooth walled plastic pipes provide more efficient inlets than sharp edged inlets on metal culverts.
- The greater roughness of corrugated interiors may be an advantage for fish passage and for other situations where barrel or outlet velocities must be reduced.
- Flexible pipe culverts may have an advantage over concrete box culverts in certain unfavourable foundation soil conditions.

As a material choice in the present case, a concrete culvert is considered more appropriate under a primary highway (i.e. Highway 26), for the reasons cited above (i.e. its durability), as well as the existing favourable foundation conditions as revealed by the boreholes. The following concrete culvert options may be considered.

- Extending the existing reinforced concrete closed bottom box culvert with a similar (i.e. closed bottom concrete box culvert) structure.
- Extending the existing culvert with an open bottom (i.e. open-invert) of similar geometry as the existing culvert.

Both of the above options are feasible.

A disadvantage of open bottom culverts is that they are vulnerable to failure caused by scour, degradation or artificial deepening. As well excavations for the footings of an open bottom culvert increase the possibility of encountering groundwater. We understand that the existing closed bottom culvert has performed satisfactorily. For these reasons, it is our opinion that matching the existing culvert for the extension is the better choice (i.e. the use of a box culvert is recommended).

5.2 Structure Foundations

The proposed culvert extension can be founded on the base slab (box culvert) or on normal strip foundations (open bottom culvert). The use of deep foundations is neither necessary nor recommended for the following reasons. The soils underlying the site, below the foundation levels, are competent materials. Therefore, the use of deep foundations will be much more costly in comparison to normal strip footing foundations. Driven piles will require pile tip reinforcement (i.e. OPSD 3000.010) due to the anticipated heavy driving condition through glacial till deposit to attain sufficient penetration depths, increasing their cost. Furthermore, vibrations induced while driving the piles may be detrimental to the integrity of the existing structure. Cast-in-place concrete (caisson) foundations are not recommended as they will extend to below the water table in cohesionless soils. The use of deep foundations at this site is, therefore, not recommended.

5.2.1 Strip Footing Foundations

The culvert bottom elevation will be at about El. 180.2 to 180.7 m. Reference to the boreholes indicates that the culvert extension can be founded on strip footings in the undisturbed competent sandy silt to silty sand till. For frost protection, the footings should have a permanent minimum earth cover of at least 1.5 m

or its thermal equivalent. The following geotechnical resistances are recommended for the culvert to be placed on undisturbed competent sandy silt to silty sand till at or below about El. 180.5 m.

Bearing Resistance at U.L.S. = 500 kPa

Factored Geotechnical Resistance at S.L.S. = 350 kPa

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

When making the recommendations a minimum 2.0 m wide footing was assumed.

Higher geotechnical bearing resistances are available at greater depths within the silty sand to sandy silt till deposit but are not believed to be necessary for the relatively light structure proposed. In addition, extensive dewatering will be required, as the water table at the time of our investigation was at about El. 179.5 m and could be higher during wetter seasons. For this reason, the excavations should be kept to a minimum to avoid extensive dewatering. To reduce the possibility of extensive dewatering and to match the existing structure we recommend a closed bottom culvert extension be provided.

5.2.2 Foundation Support for Closed Bottom Concrete Box Culvert Structure

As was mentioned before, the design invert (i.e. bottom of concrete box) elevation ranges from 180.5 m at the existing inlet, rising to 181.1 m at the new proposed inlet. At these elevations all boreholes show the presence of compact to very dense but typically compact sandy silt to silty sand till. As per MTO practice, a well compacted granular bedding material may be placed between the bottom of concrete and the silty sand to sandy silt till, for uniform support.

The sandy silt to silty sand till, if undisturbed, will provide good foundation support for the structure. The following geotechnical resistances are recommended for the proposed 4.27 x 2.44 m structure.

Bearing Resistance at U.L.S. = 500 kPa

Factored Geotechnical Resistance at S.L.S. = 350 kPa

Since the imposed loads due to the structure will be partially counter-balanced by the removal of the existing overburden pressure, there should theoretically be no major problems associated with settlements, provided that the bearing soil is undisturbed during the construction. However, an allowance of 25 mm of possible total settlement should be made including possible rebound during construction due to stress relief. It should be pointed out that this settlement will lead to a differential settlement between the existing culvert and the proposed culvert extension where highest additional loads can be expected (i.e. least amount of overburden removal). This aspect should be taken into consideration in the design and the construction. 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. 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. Depending on the site conditions, the placement of 75 mm thick layer of lean concrete (mud mat) on the bearing surface may be necessary as directed by the QVE. This should be placed as soon as possible after the bearing surface is

exposed and approved. We recommend that an allowance for this be made in the contract in case the QVE decides that it is necessary to proceed in this manner.

5.3 Bedding

For precast concrete box culvert, we recommend that a minimum 200 (can be 150) mm thick bedding material be placed beneath the concrete box culvert slab to provide uniform support. This can consist of a well-graded material such as Granular 'A'. If it is necessary to use a bedding material which is not well graded, the bedding should be protected against the migration of the silt subgrade into the bedding material by placing a suitable geotextile against the subgrade soil. The geotextile (OPSS 1860) should be a Class II non woven type of filter cloth with Filtering Opening Size (F.O.S.) not larger than 115 micron (such as Terrafix 400R, or approved equivalent). We also recommend that the compatibility of the geotextile with the exposed silty subgrade be reviewed and approved during the construction.

The unfactored horizontal resistance against sliding between approved till surface and the bedding can be calculated using a friction angle of 28 degrees. The same value can be used if a geotextile is utilized in conjunction with the bedding (i.e. if a poorly graded material is used as a bedding material), as well as with clean concrete skim coat. It is, however, believed that sliding will not present a problem.

5.4 Backfilling

Backfilling for the culvert construction should consist of select, suitable materials, compacted in accordance with the MTO standards and conform to OPSD-803.010. For fills immediately below any roadway, it is recommended that Granular 'A' or 'B' aggregates be used. Where necessary, proper tapering as per standards should be provided. Below a depth of about 1.5 m from any finished road grade, approved compactable fill, such as select subgrade materials (SSM) can be used.

In any case, the backfill around the culvert should be compacted in shallow lifts, not exceeding 300 mm loose thickness, to at least 95% of the material's Standard Proctor Maximum Dry Density (SPMDD). The Granular 'A' or 'B' materials should be compacted to not less than 98% of their SPMDD's. To avoid damaging or laterally dislocating it, care should be exercised when compacting fill adjacent to and immediately on top of the culvert structure and compaction equipment should be restricted in size as per MTO convention. The backfilling operation should be carried out simultaneously on both sides of the culvert as per MTO specifications.

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

Computation of earth pressures acting against rigid culvert walls and any wing walls should be in accordance with the Canadian Highway Bridge Design Code, (CHBDC) 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 $\phi=30^\circ$ (unfactored)

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

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

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

Where K_b is the 'intermediate' earth pressure coefficient for a partially restrained structure.

K^* is the earth pressure coefficient for a soil loading a fully-restrained structure, including compaction surcharge effects.

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

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

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

5.5 Approach Embankments

The grade over the proposed culvert will be raised to match the existing highway grade to about 4 m above existing grades (o.g.) at the existing culvert inlet and gradually decrease to zero at the proposed culvert inlet location.

Boreholes of our investigation and Boreholes 7 and 8 of the previous investigation by Golder Associates show that after the removal of the surficial topsoil and any other organic or otherwise unsuitable soils, the subgrade is suitable to support up to about 4 m high embankments.

In Boreholes 101 and 102, a surficial silty sand cap was contacted to depths of 0.9 m and 1.0 m or to El. 182.9 m and 182.7 m, respectively. The recorded N-values in the surficial silty sand ranged from 6 to 14 blows/0.3 m. Underlying the surficial silty sand, Boreholes 101 and 102 contacted a 1.0 m thick silty clay layer, in which has the recorded N-values were 5 to 13. The silty clay in Boreholes 101 and 102 is further underlain by a sandy silt to silty sand till. From these conditions, the subgrade below the topsoil (or any other unsuitable soil) is considered suitable to support up to about 4 m high embankments, after stripping all the topsoil and any underlying unsuitable soils as per MTO procedures. If unsuitable materials are encountered during the construction of approach embankments, unsuitable materials have to be removed to a sufficient depth and subgrade should be inspected and approved. It would then have to be placed in shallow layers and properly compacted. With this procedure, conventional 2H:1V slopes or flatter should not cause foundation instability.

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

Proper benching of the existing embankment slope should be implemented if and where abutting into the existing embankments, as per MTO procedures and in accordance with OPSD 208.01.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. select subgrade materials or Granular 'B' – OPSS 1010). The embankment fill should be placed on the approved and properly rolled subgrade in lifts not exceeding 300 mm when loosely placed and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. Embankment construction should be carried out in conformance with SP206S03.

Embankment loadings would likely result in a settlement of the order of 25 mm due to the settlement of natural foundation soils. About one-third of this settlement should take place within one month, with the majority of the remaining within the next two years.

In addition, the settlement of the new embankment fills under their own weight can be expected to occur. This should not exceed 25 mm. The time rate of the settlement will depend on the material used for construction. However, if SSM or granular soils are used, about half of this settlement should be completed within two months and the remaining half substantially completed within one year.

As these settlements are not excessive, neither surcharging nor preloading is considered necessary. However, consideration may be given to delaying any paving by about three weeks after the completion of the embankment to its full height.

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

5.6 Proposed Retaining Walls

Typically, in Ontario, wing walls consists of reinforced concrete retaining walls supported on normal strip footing foundations placed on undisturbed competent natural soils.

Reinforced Soil System (RSS) is also sometimes used. Gabion type walls (or similar crib-type gravity walls) or geoweb type walls are also occasionally used.

We understand that the present design incorporates reinforced concrete wing walls at the proposed culvert inlet.

Conventional reinforced concrete type retaining walls are supported on normal strip footing foundations, placed on undisturbed competent natural soils. Boreholes drilled at the site show the presence of dense to very dense sandy silt to silty sand till at or below the anticipated founding elevation of 180.5 m, underlain by compact to very dense silty fine sand. Based on this, the following geotechnical resistances can be used for footings at least 2.0 m wide.

Factored Geotechnical Resistance at U.L.S. = 500 kPa

Geotechnical Resistance at S.L.S. = 350 kPa

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

The structure should be checked against overturning and sliding, with an appropriate factor of safety. The unfactored horizontal resistance against sliding between poured concrete and approved sandy silt to silty sand till subgrade surface can be calculated using a friction angle of 28 degrees. Additional resistance can be provided by keying into the founding soil, if necessary.

The lateral earth pressures acting on retaining walls will depend on the type and the method of placement of the backfill materials and on the subsequent lateral movements of the structure. The backfill properties given in Section 5.4 can be used for design purposes. In addition, traffic loads may need to be taken into consideration.

As mentioned in Section 5.2.2 of this report, after excavating to the proposed founding level, the exposed subgrade should be inspected, evaluated and approved by the Geotechnical Engineer appointed by the QVE. It is recommended that an allowance be made to place a 75 to 100 mm thick layer of skim coat of lean concrete on the foundation bearing surface, as rapidly as possible after the excavation and the approval.

Frost and scour should be taken into consideration when choosing the founding depths. As well the position of the groundwater table should be considered, as extensive dewatering may be required for excavations extending below the groundwater table, as discussed in Section 5.7 of this report.

RSS type walls at both culvert locations can be utilized after the removal of any underlying topsoil, fill, weak or otherwise unsuitable natural soils and their replacement with properly compacted, acceptable engineered fills. Scour will need to be considered.

If feasible (i.e. depending on the site conditions at the time of construction) for RSS construction, the exposed surface should be rolled from the surface. The grade can then be raised using engineered fill placed in thin layers (i.e. not exceeding 0.3 m when loosely placement) and each layer should be properly compacted to at least 98% of its Standard Proctor Maximum dry density. The fill should consist of a clean, compactable soil, which is free of organics, boulders, frozen soils and other deleterious materials. The first 0.6 m (i.e. immediately above the exposed acceptable subgrade) of the fill may need to consist of granular material such as Granular 'A' or 'B' Type II materials, to provide a suitable base upon which the required degree of compaction can be attained with other soil types.

While no major problems are anticipated, the RSS is typically a patented method and the provider of the system normally guarantees its stability. This aspect should be looked into after the details are known.

Gabion type walls (or similar crib-type gravity walls) or geoweb would be suitable after the removal of all unsuitable soils. These types of walls may undergo vertical or horizontal movements and are seldom used for primary highways. Scour will need to be considered in this case, as well.

5.7 Construction Comments

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

SP 105 S19 – Protection Systems

SP 902 S01 – Excavation and Backfilling to Structures

The boreholes show that the excavations can be expected to extend through topsoil, fill materials into silty clay and then into sandy silt to silty sand till. At the time of our investigation, the groundwater table was found at about El. 179.5 m.

The soils can be classified as follows for temporary excavations.

Topsoil and Fill	Type 3 soil
Silty Clay	Type 2 soil above groundwater level and Type 3 soil below the groundwater level
Sandy Silt to Silty Sand Till	Type 3 soil above groundwater level (i.e. if properly dewatered) Type 4 soil below groundwater level
Silty Fine Sand	Type 4 soil below groundwater level

Depending on the site conditions and the position of the groundwater at the time of construction, dewatering may be required to lower the groundwater table for excavations extending below Elevation 180 m, to facilitate the construction and to stabilize the soil. The water level in the existing creek will also play a role. We recommend therefore that the construction of the proposed extension be carried out during

a dry period since the till below groundwater can be easily disturbed, as well, if possible, foundation excavations should not extend below El. 180.0 m. If excavations need to be extended to below the groundwater table, however, it is our opinion that the groundwater table can be controlled by means of gravity drainage and pumping from strategically located filtered sumps. If the groundwater level is to be lowered by more than about 0.7 m, dewatering by means of well points will likely be required. In this case the wells will need to extend into the silty fine sand underlying the till. As well, vacuum may need to be applied.

If the culvert and/or the wing walls is/are founded on unduly disturbed soil, excessive settlements can ensue after the structures are constructed and backfilled. For this reason, careful construction techniques should be followed. In addition, care should be taken to avoid disturbing the foundation soils.

To facilitate the construction, water flow in the existing culvert will need to be controlled so that the construction can be carried out in sufficiently dry conditions. The construction of a temporary culvert will likely be difficult under the primary highway and, as well, since the existing culvert is quite long. Consideration may be given to divert the flow into a pipe, placed within the existing culvert, starting near the inlet of the existing culvert. This may be possible if the construction is carried out during a dry period and if hydraulics permit this approach.

All bearing surfaces should be inspected and approved by the Geotechnical Engineer appointed by the QVE. Consideration can be given to an NSSP for proper diversion of the creek flow and the dewatering of the foundation excavations (if it is required), with the responsibility assigned to the Contractor. The Contractor should also be "red flagged" (i.e. warned) for the possible presence of cobbles and boulders, especially in the glacial till.

It is our understanding that temporary shoring will be required for this extension of the existing culverts. Shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this case, the required performance level is considered 2. The shoring system should be designed by a Professional Engineer, experienced in this type of work and the shoring should be in accordance with SP 105 S19. The coefficient of lateral earth pressures given in Table 5.7.1 can be used for the design of the temporary shoring system.

Table 5.7.1 Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	Unit Weight (kN/m ³)
Topsoil	0.41	0.58	2.4	15.0
Embankment Fill	0.35	0.52	2.9	20.0
Silty Clay	0.38	0.55	2.7	18.0
Sandy Silt Till	0.29	0.45	3.5	21.5
Silty Fine Sand	0.32	0.48	3.1	20.5

5.8 Erosion Protection

Erosion and scour protection should be provided at the culvert outlet including the side slopes, if necessary. The erosion/scour protection should be designed by a specialist River Engineer/Scientist who is familiar with the findings of this investigation.

The boreholes indicate that at the invert level, the predominant soil type at the site consists of sandy silt to silty sand till. The soil on the sides may consist of the sandy silt to silty sand till overlain by silty clay and possibly by surficial silty sand and fill. In addition to the soil types, the particular design depends on other considerations such as water velocity in the Creek at the culvert locations, fisheries, etc.

We recommend that concrete cut-off (apron) and head walls 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).

In addition to cut-off and head walls, consideration may be given to erosion/scour protection at the inlet and the outlet.

At the inlet, consideration may also be given, as an alternative to concrete head walls, 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 at least 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. The clay seal should be protected by laying a 0.6 m thick rock protection over it. The clay seal should be extended at least 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 head walls or in conjunction with, a 0.6 m thick rock protection consisting of 300 mm size rock can be considered, overlying a 200 mm thick layer of granular filter material. This should extend at least 6 m along the channel and the sides (to at least 0.3 m above the high water level). The granular filter material underlying the rock protection should consist of a suitable granular material such as Granular 'A'. Alternatively, a suitable geotextile can be used underneath the rock fill, in lieu of the granular filter material.

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

These are suggestions and, as mentioned before, the erosion and scour protection should be designed by a specialist.

5.9 Frost Protection

Design frost protection for the general area is 1.5 m. A permanent soil cover of at least 1.5 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

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

For and on behalf of Coffey Geotechnics Inc.



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Senior Principal



Appendix F

OPSD

List of Standard Specifications

OPSD

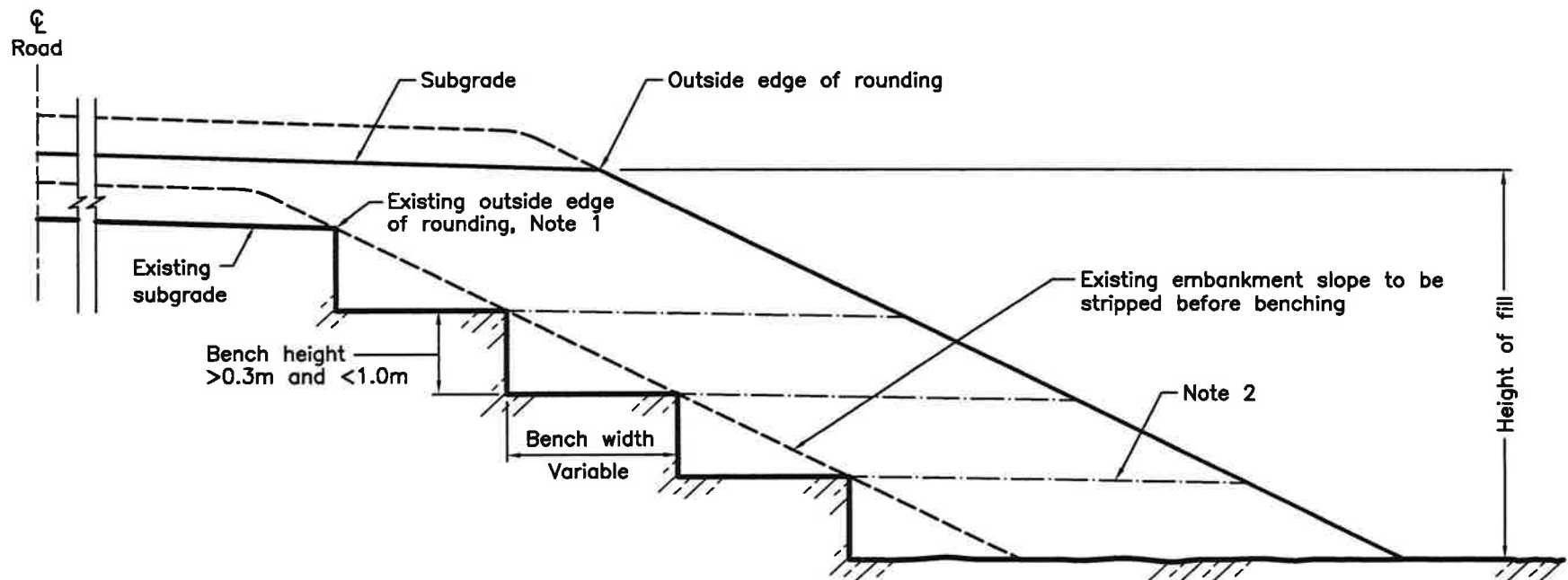
- 208.010 Benching on Earth Slope
- 803.010 Backfill and Cover for Concrete Culverts
- 810.010 Riprap Treatment for Sewer and Culvert Outlets

OPSS

- 571 Construction Specification for Sodding
- 572 Construction Specification for Seed and Cover
- 1003 Material Specification for Aggregates – Hot Mix Asphalt
- 1010 Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
- 1205 Material Specification for Clay Seal
- 1860 Material Specification for Geotextiles

SP

- 105S19 Amendment to OPSS 539, November 2003
- 206S03 Amendment to OPSS 206, December 1993
- 902S01 Excavation and Backfilling - Structures



NOTES:

- 1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.
- 2 Benches are to be excavated one level at a time and the fill placed and compacted before the next bench is excavated.
- A Benching is not required on existing slopes flatter than 3H:1V.
- B All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2008

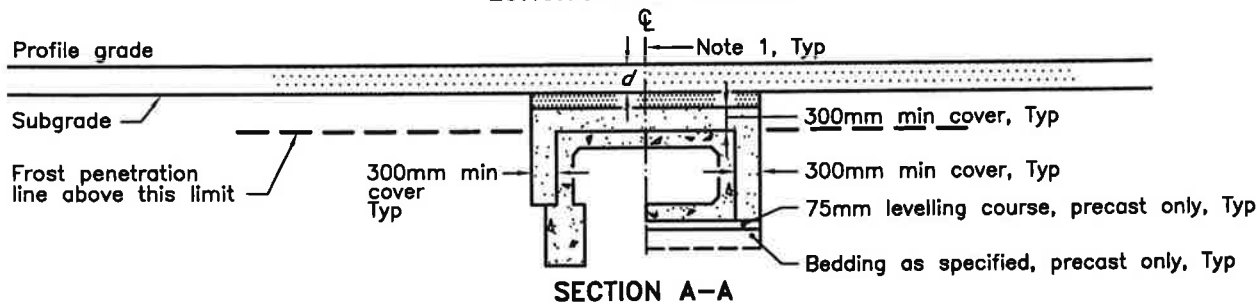
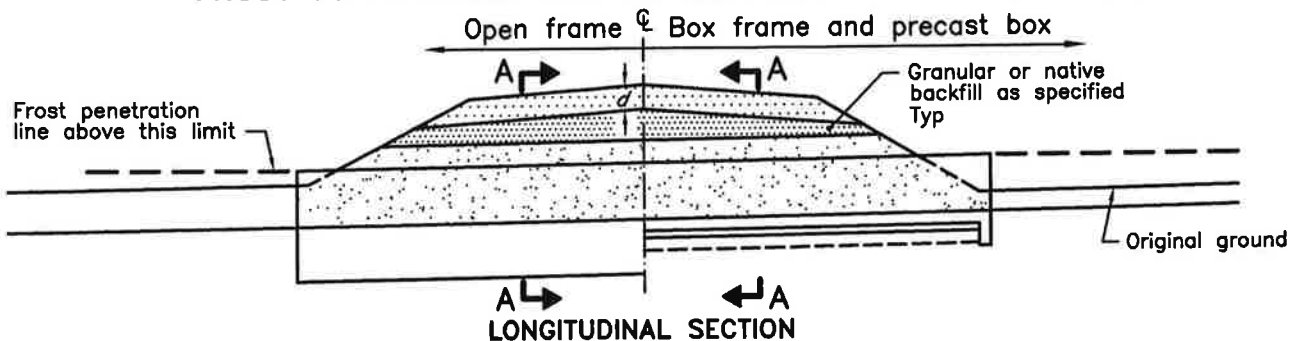
Rev 2

BENCHING OF EARTH SLOPES

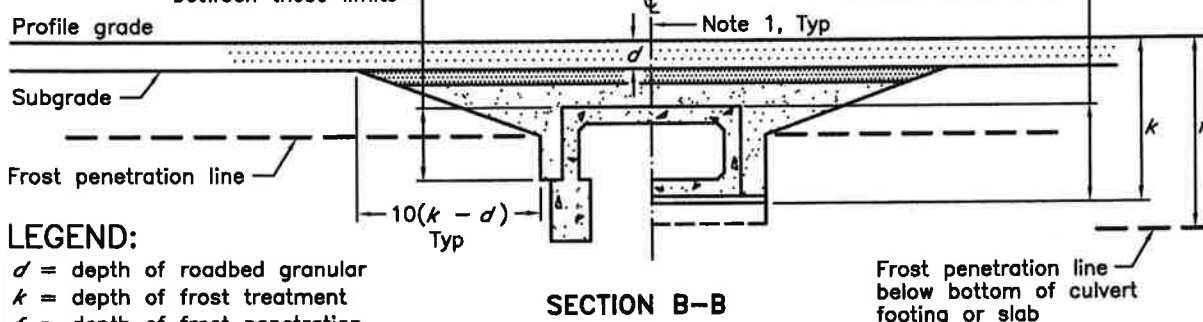
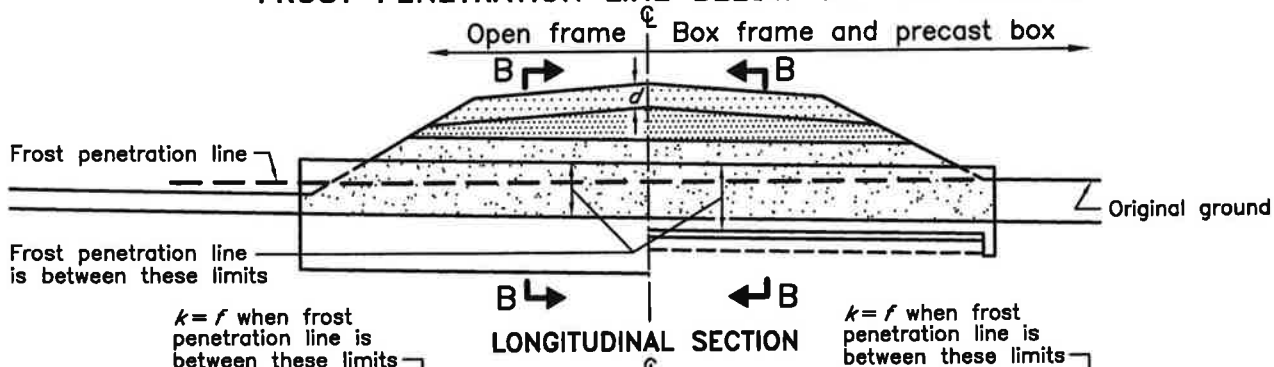
OPSD 208.010



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.

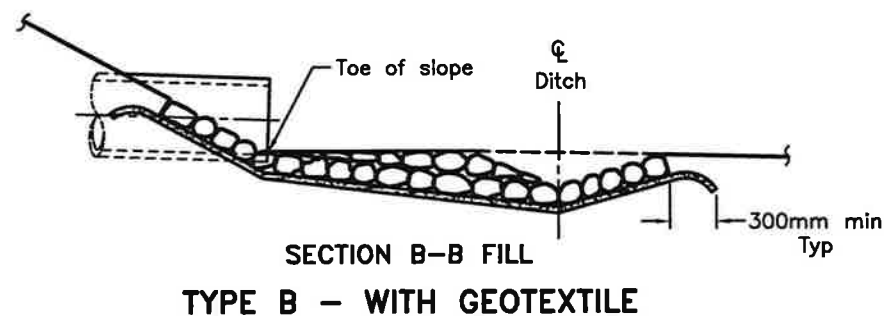
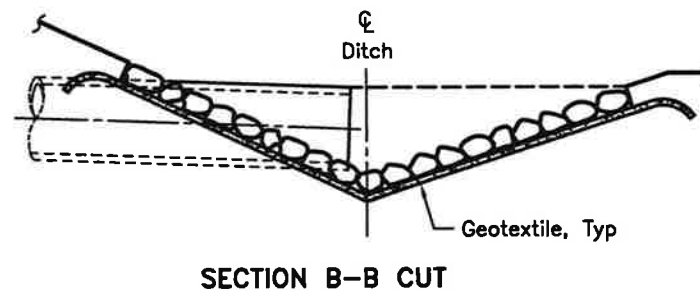
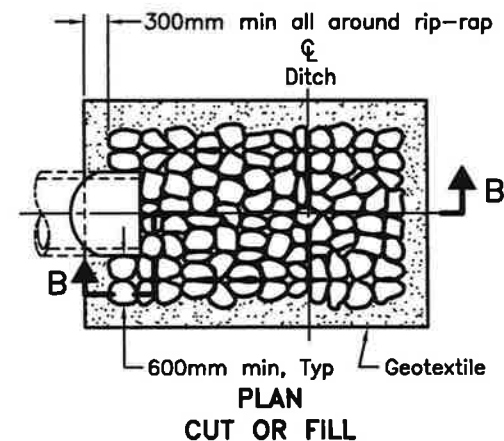
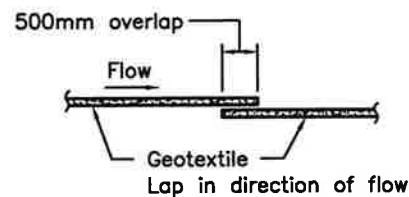
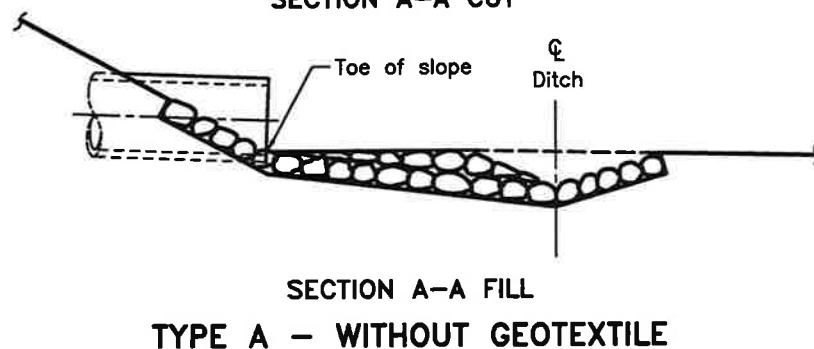
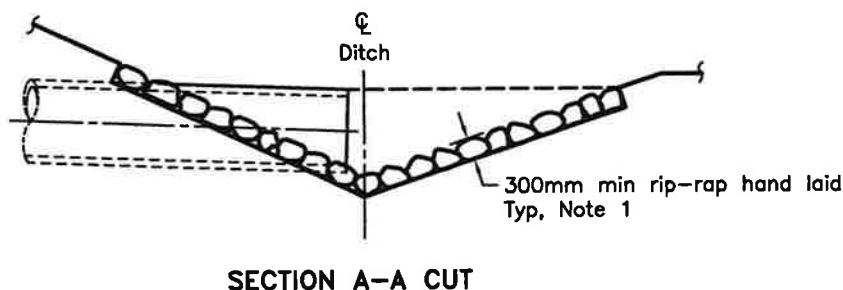
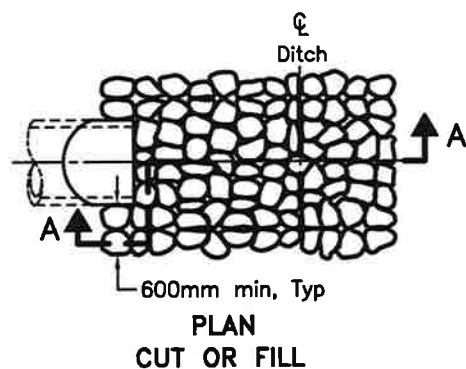
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2006 Rev 1

**BACKFILL AND COVER
FOR CONCRETE CULVERTS**

OPSD 803.010





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 Reports

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.