

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
BURNTWOOD CREEK CULVERT,
SITE NO. 39E-247/C, HIGHWAY 634
AVON TOWNSHIP, ONTARIO
GEOCRES NO. 42H-36 W.P. 5121-05-00**

D. M. Wills Associates Limited

Project: SPT1233
April 6, 2009

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

Attention: Mr. Andy Staszak, P.Eng.

Dear Sirs:

**RE: Foundation Investigation and Design Reports, Burntwood Creek Culvert, Site No. 39E-247/C,
Highway 634, Avon Township, Ontario**

Please find attached the draft Foundation Investigation and Design Reports relating to the above noted site.

For and on behalf of Coffey Geotechnics Inc.



Ramon Miranda, P.Eng.
Manager, Transportation Division

Attachment A: Attachments

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**FOUNDATION INVESTIGATION
BURNTWOOD CREEK CULVERT
HIGHWAY 634, SITE NO. 39E-247/C
AVON TOWNSHIP, ONTARIO**

1 INTRODUCTION

Coffey Geotechnics inc. (Coffey), formerly Shaheen & Peaker, was retained by D.M. Wills Associates Limited (Wills) to carry out a foundation investigation at the site of the proposed replacement of Burntwood Creek culvert under Highway 634 at Station 17+240 in Avon Township, near Fraserdale, Ontario.

The existing culvert consists of 3.2 m diameter twin structural plate corrugated steel pipes, 48.5 m in length.

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

The findings of the investigation are presented in this report.

2 SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located approximately 65km north of the junction of Highways 11 and 634 in Avon Township, north of Smooth Rock Falls, Ontario, as shown on the Key Map in Drawing No. 1.

According to the Ontario Geological Survey Paper 93 (1984), the Surficial Geology of Northern Ontario Map 2518 (by the Ministry of Northern Development and Mines), and the Northern Ontario Engineering Geology Terrain Study Map 5033 (Ministry of Natural Resources), the site is located within the Physiographic Region known as the Canadian Shield. During the last ice age, glacial advance and recession shaped the major land features. At the end of the last ice age a glacial re-advance deposited glacial till over the bedrock surface, as well as eskers and ice-contact sediments. A large lake, known as Lake Barlow-Ojibway, later formed after retreat of the glaciers depositing thick layers of clay over a large area. Glaciolacustrine silts were also formed as part of the glacial lake and isolated outwash sand deposits as part of the glacial retreat. Bogs have formed over the poorly drained clays over a large area and recent rivers and creeks have deposited muck, sand and gravel along their alignments. The site is mapped as an isolated area of glacial till surrounded by peat bogs and Barlow-Ojibway clays.

According to the Bedrock Geology of Ontario Map 2543 (Ministry of Northern Development and Mines, Ontario), the bedrock underlying the site consists of Neo to Meso-Archean period, Precambrian age metasedimentary rocks, such as granulite grade wacke, arkose, argillite, slate, marble, chert, and minor metavolcanic rocks.

3 PROCEDURES

The fieldwork for this project was performed on October 8 and 9, 2008 and consisted of drilling and sampling three boreholes (FC1, FC2 and FC3) to depths of 4.7 to 11.1m below existing grades at the existing culvert location, and five boreholes (P1 to P5) for the proposed road regrading (staging) to depths of 3.5 to 9.0m below existing grades. The locations of the boreholes at the site are given on the Borehole Location Plan Drawing No. 1.

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

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).

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. Upon their completion, the boreholes were grouted using a cement/bentonite mixture as per MTO procedures.

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

4 SUMMARIZED SUBSURFACE CONDITIONS

The ground surface elevations at the borehole locations range from 214.8 to 225.1 m. The existing top of highway embankment at the culvert location has an elevation of about 220.0m, while the ground surface elevation at the boreholes advanced at the toe of the embankment adjacent to the culvert range from 214.8 to 215.6m. According to data supplied by Wills, the invert of the existing culverts are at about Elevation 211.5 to 212.1m at the outlet on the east side of the highway, rising to the inlet on the west side of the highway.

The boreholes at the culvert ends were advanced from 23.5 m west (Borehole FC1) and 25.5 m east (Borehole FC3) of the existing road centreline of Highway 634. Borehole FC2 was advanced from the existing Highway 634 shoulder. Boreholes FC1 and FC3 show, below a 0.15 to 0.25 m thick layer of topsoil, very loose to loose sandy silt to silty sand fill, and loose/soft organic silt with peat in Borehole FC1, the presence of loose to dense silt. Similarly, below the sandy silt to silty sand embankment fill, Borehole FC2 encountered a compact to dense silt deposit to the termination depth of 11.1m. Below the silt deposit at a depth of 3.8 m (El. 211.0m) in Borehole FC1, a very stiff to hard silt glacial till was encountered over the remaining depth of the borehole to a depth of 4.7m/elevation 210.1m. Practical refusal to augering was contacted in three boreholes at depths of 4.7 to 11.1m, or elevations 205.5 to 210.1 m on boulders or possibly bedrock.

The boreholes advanced for the proposed road regrading (Boreholes P1 to P5) were extended along the highway shoulder and traveling lane. Below the pavement structure the boreholes encountered the very loose to compact sandy silt to silty sand embankment fill to depths of 2.2 to 7.5 m or elevation 221.4 to 212.3 m. The fill embankment is underlain by a 0.3 to 0.6 m thick topsoil layer in Boreholes P1 and P3. The embankment fill and topsoil are in turn underlain by a loose to dense silt deposit in Boreholes P1, P3 and P4, and hard silt to clayey till in Boreholes P2 and P5.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. Stratigraphic profile and section are shown in Drawing Nos. 2 and 3. The following description of the individual soil strata is to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site. It should be noted that the soil and groundwater conditions may vary in between and beyond borehole locations.

4.1 Topsoil

A 0.15 to 0.25 m thick topsoil layer was encountered in Boreholes FC1 and FC3, which were put down from the toe of the embankment, near the culvert location. Buried topsoil was encountered below the embankment fill in Boreholes P1 and P3 and its thickness was found to vary from about 0.3 m in Borehole P1 to about 0.6 m in Borehole P3.

4.2 Fill

4.2.1 Granular Pavement Fill

Boreholes P1 to P5 and FC2 were drilled from the existing road or shoulder and were advanced through the pavement structure. Boreholes P1, P2, P4 and P5 encountered an 18 to 25mm surface treated layer. Underneath the surface treatment from the ground surface in the shoulders, a granular base course was encountered which ranged in thickness from 0.13 to 0.25 m and consisted of sand and gravel fill. Below the granular base, a sub-base layer was encountered ranging in thickness from 0.25 to 0.45 m in thickness and consisted of sand with some gravel and silt.

Measured N-values in the granular fill range from 6 to 48 blows/0.3 m which indicates a loose to dense, but generally compact to dense condition. It appears that the pavement fill at the borehole locations generally received systematic compaction when it was first placed. Measured moisture contents range from 5 to 7%.

4.2.2 Embankment Fill

Underlying the pavement fill in Boreholes P1 to P5 and FC2, and the topsoil in Boreholes FC1 and FC3, a sandy silt to silty sand fill layer (i.e. embankment fill) was contacted to depths of 2.2 to 7.7 m or elevation 221.4 to 212.3 m. The sandy silt to silty sand fill contains traces to some clay, occasional gravel, asphalt fragments and silt inclusions. Measured N-values range from 2 to 30 blows/0.3 m, indicating a very loose to compact condition. The fill appears to have been systematically compacted except where the embankment overlies the culverts and above the base of the embankment where some of the fill appears to have not received systematic compaction. Measured natural moisture contents range from 10 to 22%.

Seven grain size analyses were carried out on representative samples of the embankment fill. The results are presented on the Record of Borehole sheets in Appendix A, and the grain size curves are presented in Figure B-1 in Appendix B. The results indicate 0% gravel, 18 to 55% sand, 35 to 73% silt and 10 to 15% clay size particles.

4.3 Organic Silt

Below the fill, Borehole FC1 encountered a 1.2 m thick organic silt with peat inclusions. The organic soil layer extended to a depth of 2.3 m below the ground level or to El. 212.5 m and it was found to confirm cohesive and non-cohesive zones. Measured natural moisture contents of 44 and 45% were obtained within the organic silt. Measured N-values of 4 and 6 blows per 0.3m per obtained within these deposits indicating a loose compactness condition and a soft/firm consistency.

4.4 Silt

Underlying the topsoil, organic silt and the embankment fill, all the boreholes except Boreholes P2 and P5 encountered a silt deposit at depths of 1.4 to 8.3 m below the ground surface or below elevations ranging from 221.1 to 211.7 m. Boreholes P1, P3, P4, FC2 and FC3 were terminated within this deposit at depths of 4.4 to 11.1 m or at Elevations 205.5 to 220.7 m. The silt contains trace to some sand and occasional clay seams or layers, gravel and silty sand layers. In Borehole FC2 the silt contains clay and fine sand interbeds.

Four grain size analyses were carried out on representative samples of the silt. The results are presented on the Record of Borehole sheets in Appendix A, and the grain size curves are presented in Figure B-2 in Appendix B. The results indicate 0 to 6% gravel, 1 to 4% sand, 71 to 87% silt and 12 to 19% clay size particles. This is a predominantly non-cohesive (i.e. fine-grained granular soil with some plastic (cohesive) zones).

Atterberg Limits tests were carried out on a sample of the silt. A Liquid Limit of 23%, a Plastic Limit of 17% and plasticity index of 6 were obtained, indicating a low plasticity material (see Figure B-3 in Appendix B).

Measured N-values range from 8 to 41 blows per 0.3 m indicating a loose to dense relative density with occasional silt to hard zones. Measured moisture contents range from 13 to 27%.

4.5 Glacial Till

Below the fill in Boreholes P2 and P5 and the silt deposit in Borehole FC1, a silt to clayey silt glacial till was contacted. This deposit is considered a cohesive soil. These boreholes were terminated within the glacial till at depths of 3.5 to 4.9 m, or Elevations 210.1 to 218.4 m. Auger refusal was obtained in Borehole FC1 at depth of 4.7 m, or elevation 210.1 m, probably on a boulder or possibly on bedrock. The glacial till is generally a heterogeneous mixture of clayey silt with varying amounts of sand and gravel. Two grain size analyses were carried out on representative samples of the glacial till. The results are presented on the Record of Borehole sheets in Appendix A, and the grain size curves are presented in Figure B-4 in Appendix B. The results indicate 0 to 1% gravel, 8 to 16% sand, 58 to 73% silt and 19 to 25% clay size particles. Atterberg Limits tests performed on a sample from the deposit and as shown on the Plasticity Chart (Figure B-5, Appendix B) the following index values were obtained.

Liquid Limit	=	22%
Plasticity Limit	=	15%
Plasticity Index	=	7

These values are characteristic of clayey soils of low plasticity. Although not encountered in the boreholes, cobbles and boulders should always within the glacial till deposits due to their nature of deposition.

Measured N-values ranged from 25 to greater than 100 blows per 0.3 m indicating a very stiff to hard consistency. Measured moisture contents range from 10 to 20%.

4.6 Groundwater Conditions

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

Based on the moisture contents of the soil samples, and the observations from the boreholes, it is our opinion that the groundwater level at the site was generally at about Elevation 212 to 213 m at the culvert location. At the time of our investigation in Boreholes P1, P2, P4 and P5 drilled beyond Burntwood Creek, it is our opinion that the groundwater level was generally at or near the o.g. levels or between Elevations ranging from 221.1 m (Borehole P1) and 215 m (Borehole P4) ranges from about Elevation 218 to 221 m.

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

For and on behalf of Coffey Geotechnics Inc.


Ramon Miranda, P.Eng.




Zuhtu Ozden, P.Eng.



Drawings

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

coffey  **geotechnics**
SPECIALISTS MANAGING THE EARTH

Plan view of Highway 634 showing proposed and existing culverts, boreholes, and stationing. The diagram includes labels for stationing (17+100, 17+200, 17+300), culvert types (1/2 Culvert 600mm, 3050 x 48-75 SPCC, 3050 x 48-72 SPCC), and borehole locations (BH P1, BH P2, BH P3, BH P4, BH P5, BH FC1, BH FC2, BH FC3). It also indicates existing culverts, proposed culverts, and various features like bush, asphalt (ASPH), and 3CGR (3rd Class Growth Rate).



LEGEND

No.	ELEV.	STATION No.	OFFSET
BH P1	225.1	17+133	2.0m Rt C/L
BH P2	222.5	17+183	2.0m Rt C/L
BH P3	220.0	17+240	4.5m Lt C/L
BH P4	220.2	17+290	2.0m Lt C/L
BH P5	221.9	17+340	2.0m Lt C/L
BH FC1	214.8	17+224	23.5m Lt C/L
BH FC2	220.1	17+233	4.0m Rt C/L
BH FC3	215.6	17+238	25.5m Rt C/L

==NOTE==

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

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

REV.			
	DATE	BY	DESCRIPTION

Geocres No. 42H-36

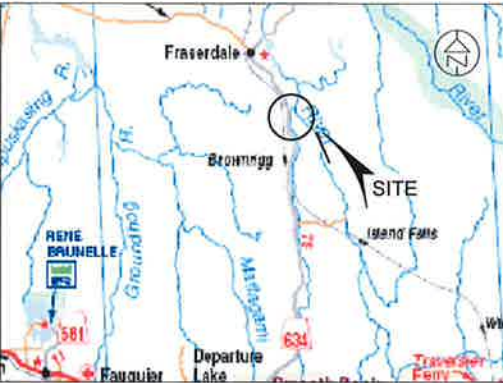
SPT 1233			DIST
SUBM'D	CHECKED	DATE Jan 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1

METRIC

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

CONT No.
WP: 5121-05-00

HIGHWAY 634, BURNTWOOD CREEK
(STATION 17+231)
CROSS SECTION



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

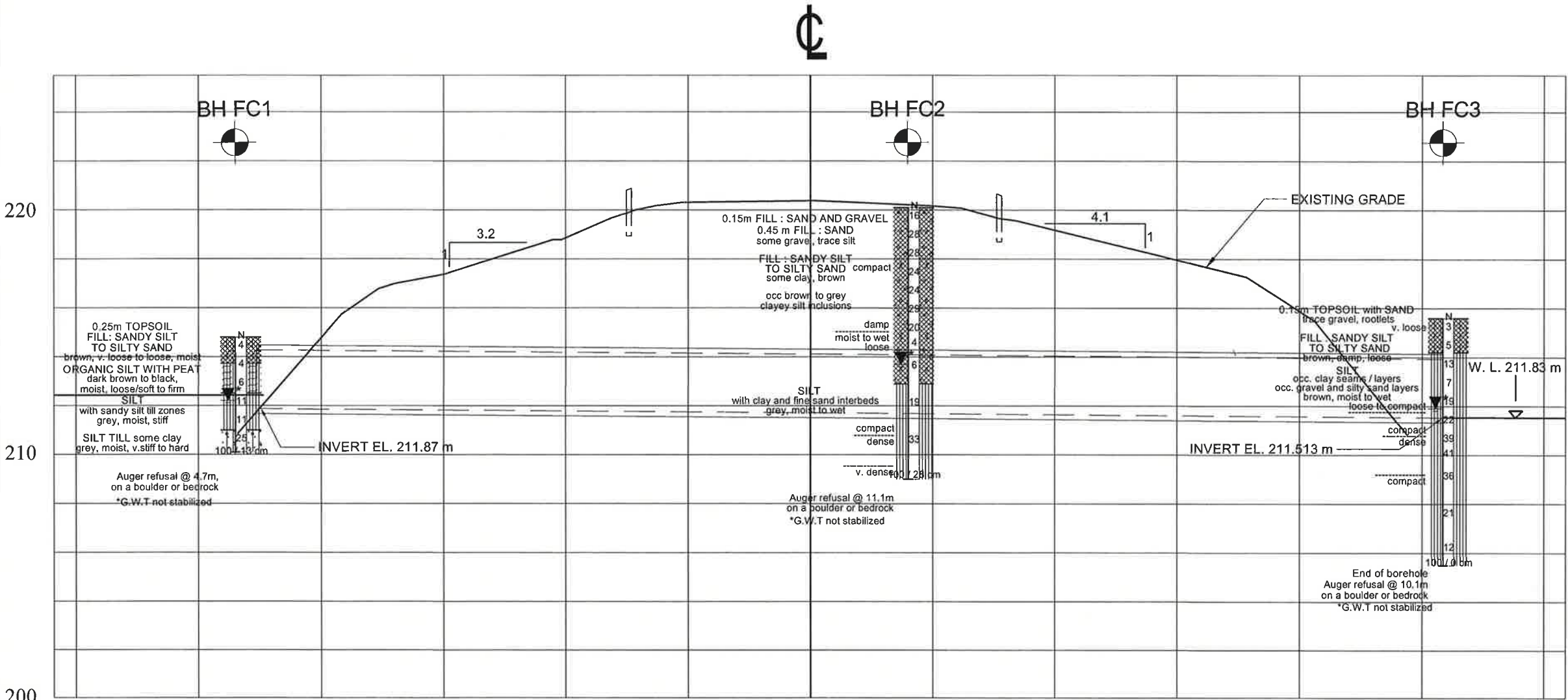
No.	ELEV.	STATION No.	OFFSET
BH FC1	214.8	17+224	23.5m Lt C/L
BH FC2	220.1	17+233	4.0m Rt C/L
BH FC3	215.6	17+238	25.5m Rt C/L

NOTE

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

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

REV.	DATE	BY	DESCRIPTION
Geocres No. 42H-36			
SPT 1233			DIST
SUBMD	CHECKED	DATE Jan 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 2



SECTION A-A



METRIC

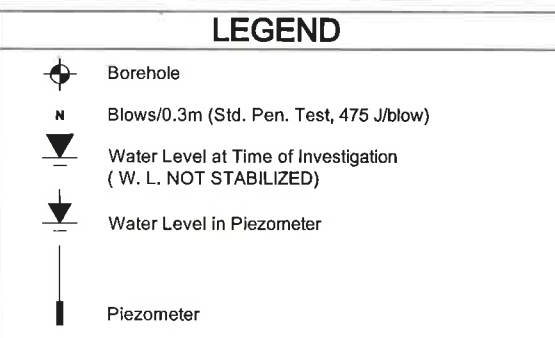
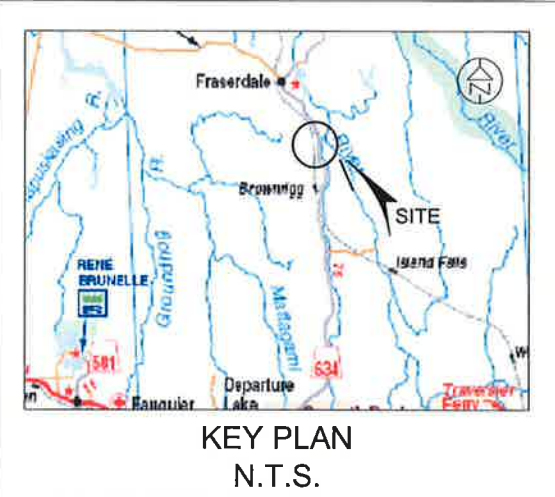
NOTES:

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.
WP: 5121-05-00
HIGHWAY 634, BURNTWOOD CREEK
PROFILE

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

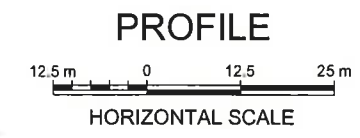
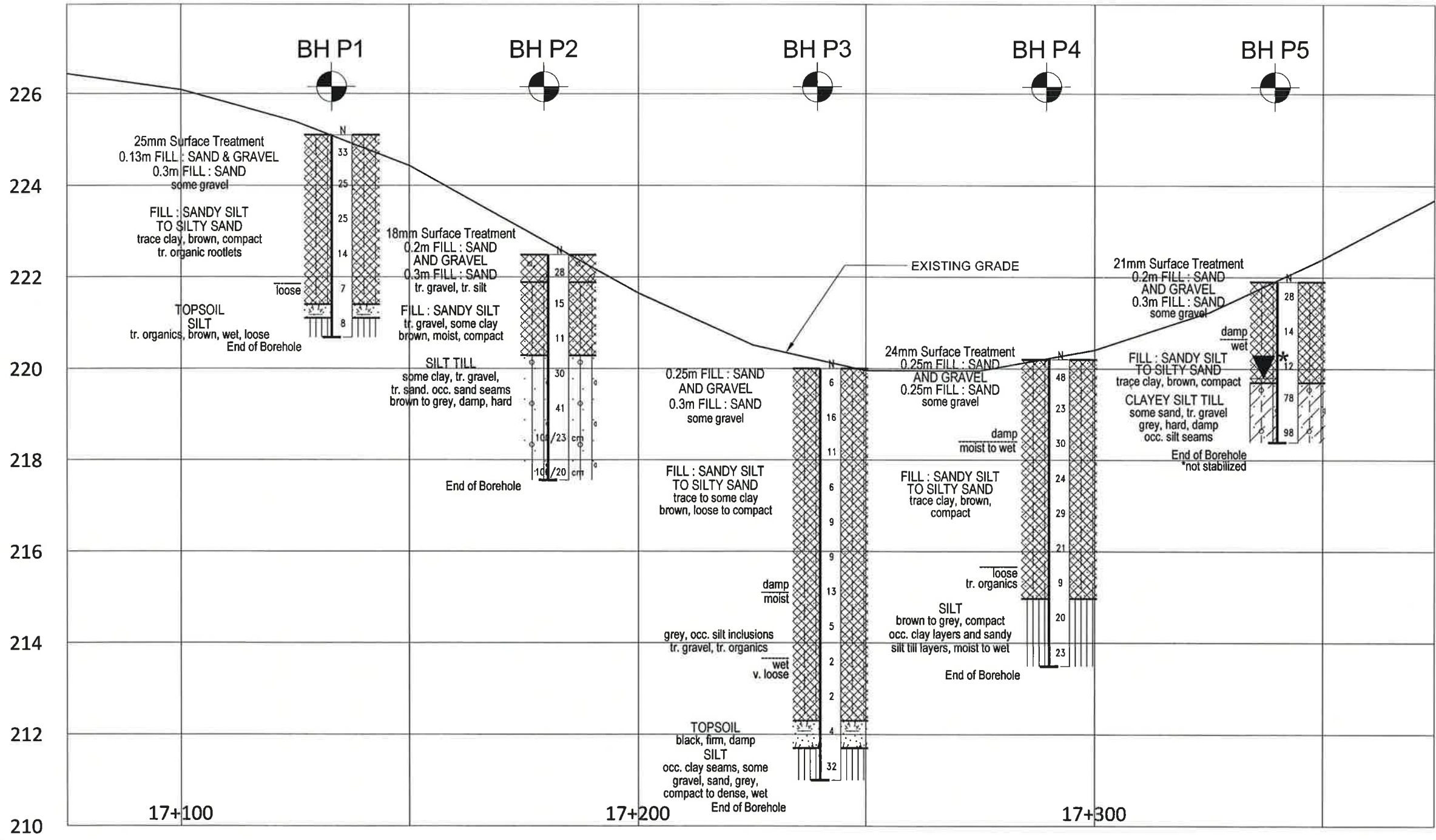


No.	ELEV.	STATION No.	OFFSET
BH P1	225.1	17+133	2.0m Rt C/L
BH P2	222.5	17+183	2.0m Rt C/L
BH P3	220.0	17+240	4.5m Lt C/L
BH P4	220.2	17+290	2.0m Lt C/L
BH P5	221.9	17+340	2.0m Lt C/L

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

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

REV.	DATE	BY	DESCRIPTION
Geocres No. 42H-36			
SPT 1233			DIST
SUBMD	CHECKED	DATE Jan 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 3



Appendix A

Record of Borehole Sheets



SPT 1233: Burntwood Creek

RECORD OF BOREHOLE No FC1

1 OF 1

METRIC

GWP 5121-05-00 LOCATION Sta: 17+224 : 23.5 m Lt. C/L of HWY 634 ORIGINATED BY SK
DIST HWY 634 BOREHOLE TYPE Hollow Stem Augering COMPILED BY SS
DATUM Geodetic DATE 10/9/2008 10/9/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
								20 40 60 80 100						
								20 40 60 80 100						
214.8	GROUND SURFACE													
0.0	0.25 m TOPSOIL		1	SS	4	☼	214							
	FILL:Sandy Silt to Silty Sand tr. asphalt fragments brown, v. loose to loose, moist		2	SS	4									
213.7														
1.1	ORGANIC SILT WITH PEAT dark brown to black,moist loose/soft to firm		3	SS	6		213							
212.5														
2.3	SILT with sandy silt till zones tr to some clay trace gravel grey, moist, stiff		4	SS	11	212								
			5	SS	11									
211.0														
3.8	SILT TILL some clay		6	SS	25	211								
210.1	grey, moist, v.stiff to hard		7	SS	100 / 13.3m									
4.7	End of borehole													
	Auger refusal @ 4.7m, on a boulder or bedrock													
	Water level @ 2.6 m upon completion, (not stabilized)*													

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1233: Burntwood Creek

RECORD OF BOREHOLE No FC2

1 OF 1

METRIC

GWP 5121-05-00 LOCATION Sta: 17+233 ; 4.0 m Rt. C/L of HWY 634 ORIGINATED BY SK
DIST HWY 634 BOREHOLE TYPE Hollow Stem Augering COMPILED BY SS
DATUM Geodetic DATE 10/8/2008 10/8/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					
								O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE					
220.1 0.0	GROUND SURFACE						20 40 60 80 100	W _p W W _L					
0.15 m FILL : Sand and Gravel 0.45 m FILL : Sand some gravel, trace silt FILL : Sandy Silt to Silty Sand brown some clay occ brown to grey clayey silt inclusions	compact		1	SS	18								
			2	SS	28								
			3	SS	28								0 35 50 15
			4	SS	24								
			5	SS	24								
			6	SS	29								0 31 54 15
			7	SS	20								
			8	SS	4								
			9	SS	6								
212.9 7.2	SILT grey with clay and fine sand interbeds moist to wet	compact dense	10	SS	19								
			11	SS	33								0 1 81 18
209.0 11.1	v. dense		12	SS	100 / 28 cm								
End of borehole Auger refusal @ 11.1 m, on a boulder or bedrock Water level @ 6.4 m (not stabilized)* and caved-in @ 6.4 m upon completion,													

+³, x³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1233: Burntwood Creek

RECORD OF BOREHOLE No FC3

1 OF 1

METRIC

GWP 5121-05-00 LOCATION Sta: 17+238 ; 25.5 m RL C/L of HWY 634 ORIGINATED BY SK
 DIST HWY 634 BOREHOLE TYPE Hollow Stem Augering COMPILED BY SS
 DATUM Geodetic DATE 10/8/2008 10/9/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE					WATER CONTENT (%) W _P W W _L					
215.8 0.0	GROUND SURFACE							20	40	60	80	100						
	0.15m TOPSOIL with sand trace gravel, rootlets FILL : Sandy Silt to Silty Sand brown, damp, loose	v. loose	1	SS	3		215											
			2	SS	5													
214.2 1.4			3	SS	13		214											
	SILT occ. clay seams / layers occ. gravel and silty sand layers brown, moist to wet		4	SS	7		213											
	loose to compact		5	SS	19		212											
			6	SS	22													
	compact		7	SS	39		211											
	dense		8	SS	41		210											
			9	SS	36		209											
	compact		10	SS	21		208											
			11	SS	12		207											
			12	SS	100 / 0 cm		206											
205.5 10.1	End of borehole Auger refusal @ 10.1 m, on a boulder or bedrock Water level @ 3.7 m and caved-in @ 4.1 m upon completion (not stabilized)*																	

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE



SPT 1233: Burntwood Creek

RECORD OF BOREHOLE No P1

1 OF 1

METRIC

GWP 5121-05-00 LOCATION Sta: 17+133, 2.0 m Rt. C/L of HWY 634 ORIGINATED BY SK
DIST HWY 634 BOREHOLE TYPE Hollow Stem Augering COMPILED BY SS
DATUM Geodetic DATE 10/8/2008 10/8/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L		
225.1 0.0	GROUND SURFACE												
	25 mm SURFACE TREATMENT 0.13 m FILL : Sand and Gravel 0.3 m FILL : Sand, some gravel		1	SS	33		225						
			2	SS	25		224						
	FILL : Sandy Silt to Silty Sand brown, compact trace clay		3	SS	25		223						
			4	SS	14		222						
	tr. organic rootlets		5	SS	7		221						
221.4 3.7	TOPSOIL												
221.1 4.0	SILT tr. organics, brown, wet, loose		6	SS	8		221						
220.7 4.4	End of Borehole Borehole dry on completion (not stabilized)												

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE



SPT 1233: Burntwood Creek

RECORD OF BOREHOLE No P2

1 OF 1

METRIC

GWP 5121-05-00 LOCATION Sta: 17+183, 2.0 m Rt. C/L of HWY 634 ORIGINATED BY SK
 DIST HWY 634 BOREHOLE TYPE Hollow Stem Augering COMPILED BY SS
 DATUM Geodetic DATE 10/8/2008 10/8/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					WATER CONTENT (%)
222.5	GROUND SURFACE							20 40 60 80 100		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
0.0	18 mm SURFACE TREATMENT		1	SS	28		222						
221.9	0.2 m FILL : Sand and Gravel		2	SS	15		221						
0.6	0.3 m FILL : Sand, tr. gravel, tr. silt		3	SS	11		220						
220.3	FILL : Sandy Silt tr. gravel, some clay brown, moist, compact		4	SS	30		219						
2.2	SILT TILL some clay, tr. gravel, tr. sand occ. sand seams brown to grey damp, hard		5	SS	41		218						
			6	SS	100/23 cm								
			7	SS	100/20 cm								
217.6	End of Borehole. Borehole dry on completion (not stabilized)												
4.9													

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1233: Burntwood Creek

1 OF 1

METRIC

GWP		5121-05-00	LOCATION	Sta: 17+240 : 4.5 m Lt. C/L of HWY 634		ORIGINATED BY	SK
DIST	HWY	634	BOREHOLE TYPE	Hollow Stem Augering		COMPILED BY	SS
DATUM	Geodetic		DATE	10/9/2008	10/9/2008	CHECKED BY	ZO

+³, ×³: Numbers refer to Sensitivity

SPT 1233: Burntwood Creek

1 OF 1

METRIC

coffey geotechnics
SPECIALISTS MANAGING THE EARTH






SPT 1233: Burntwood Creek

RECORD OF BOREHOLE No P5

1 OF 1

METRIC

GWP 5121-05-00 LOCATION Sta: 17+340, 2.0 m Lt. C/L of HWY 634 ORIGINATED BY SK
DIST HWY 634 BOREHOLE TYPE Hollow Stem Augering COMPILED BY SS
DATUM Geodetic DATE 10/9/2008 10/9/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							
221.9	GROUND SURFACE							20	40	60	80	100			
0.0	21 mm SURFACE TREATMENT 0.2 m FILL : Sand and Gravel 0.3 m FILL : Sand, some gravel		1	SS	28										
	FILL : Sandy Silt to Silty Sand trace clay brown, compact		2	SS	14										
			3	SS	12										
219.7			4	SS	78										
2.2	CLAYEY SILT TILL some sand, tr. gravel grey, hard, damp occ. silt seams		5	SS	98										
218.4															
3.6	End of Borehole Water level @ 2.1 m on completion (not stabilized)*														

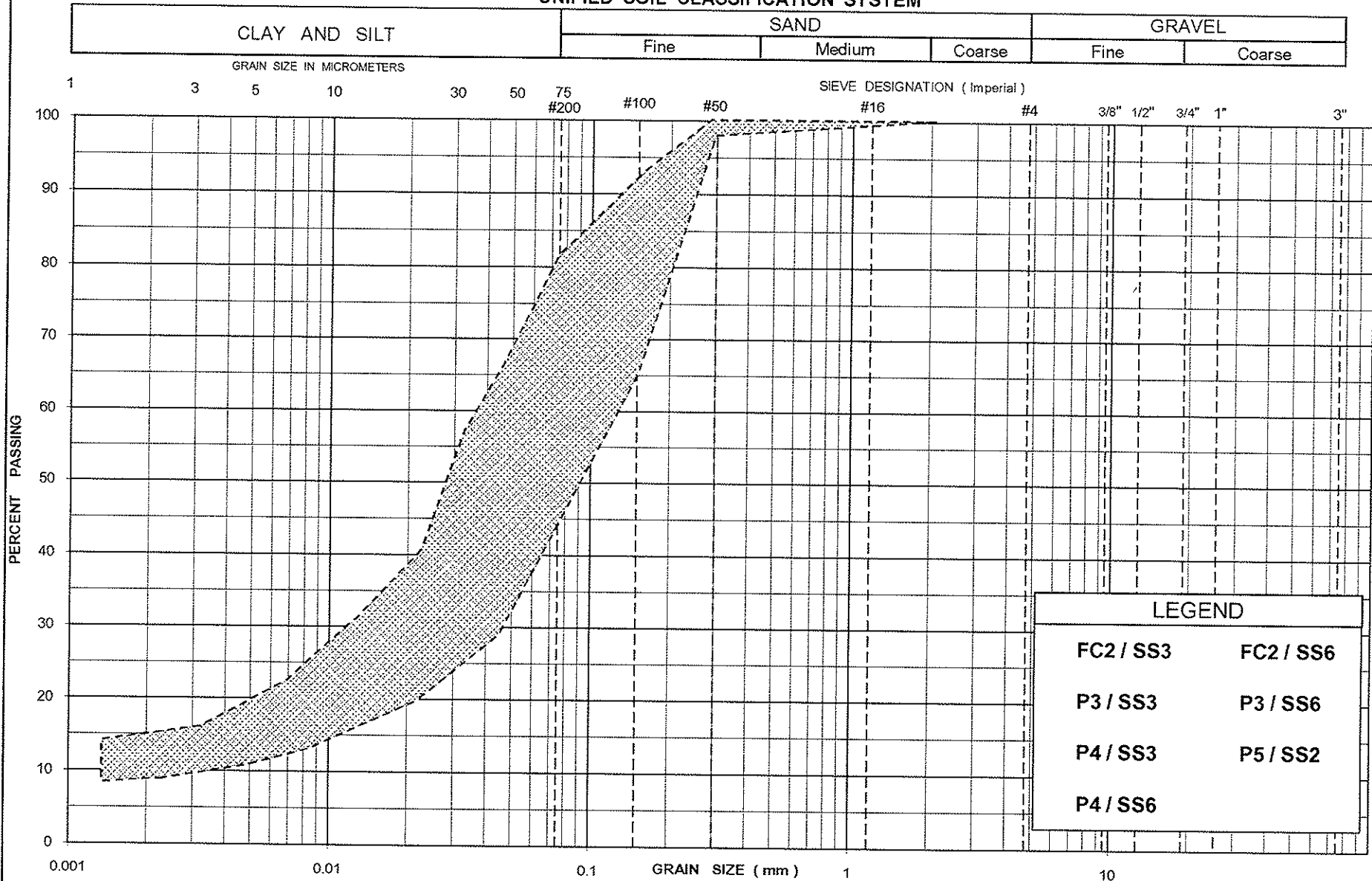
+³, ×³ : Numbers refer to
Sensitivity

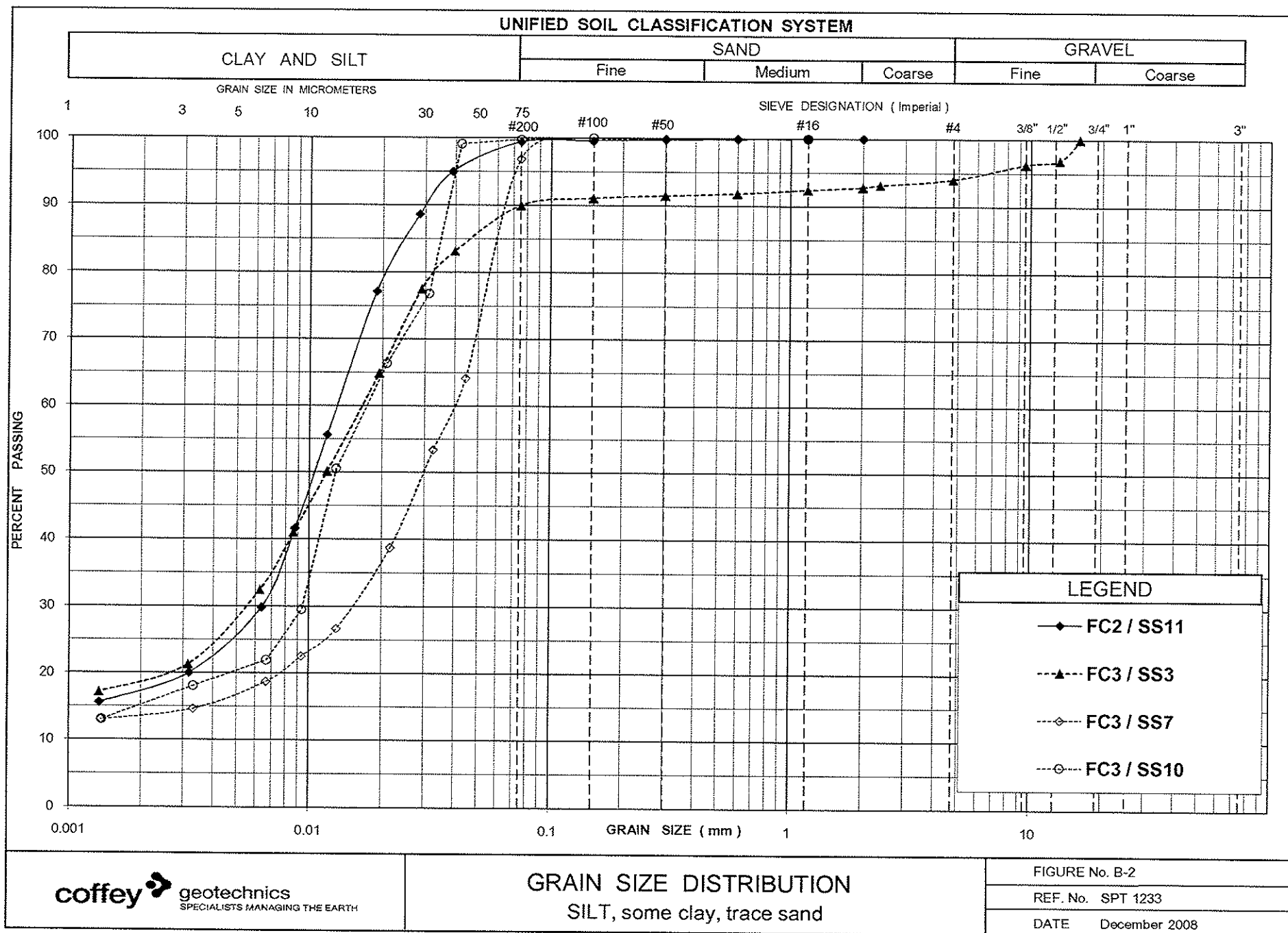
20
15 10 5
(%) STRAIN AT FAILURE

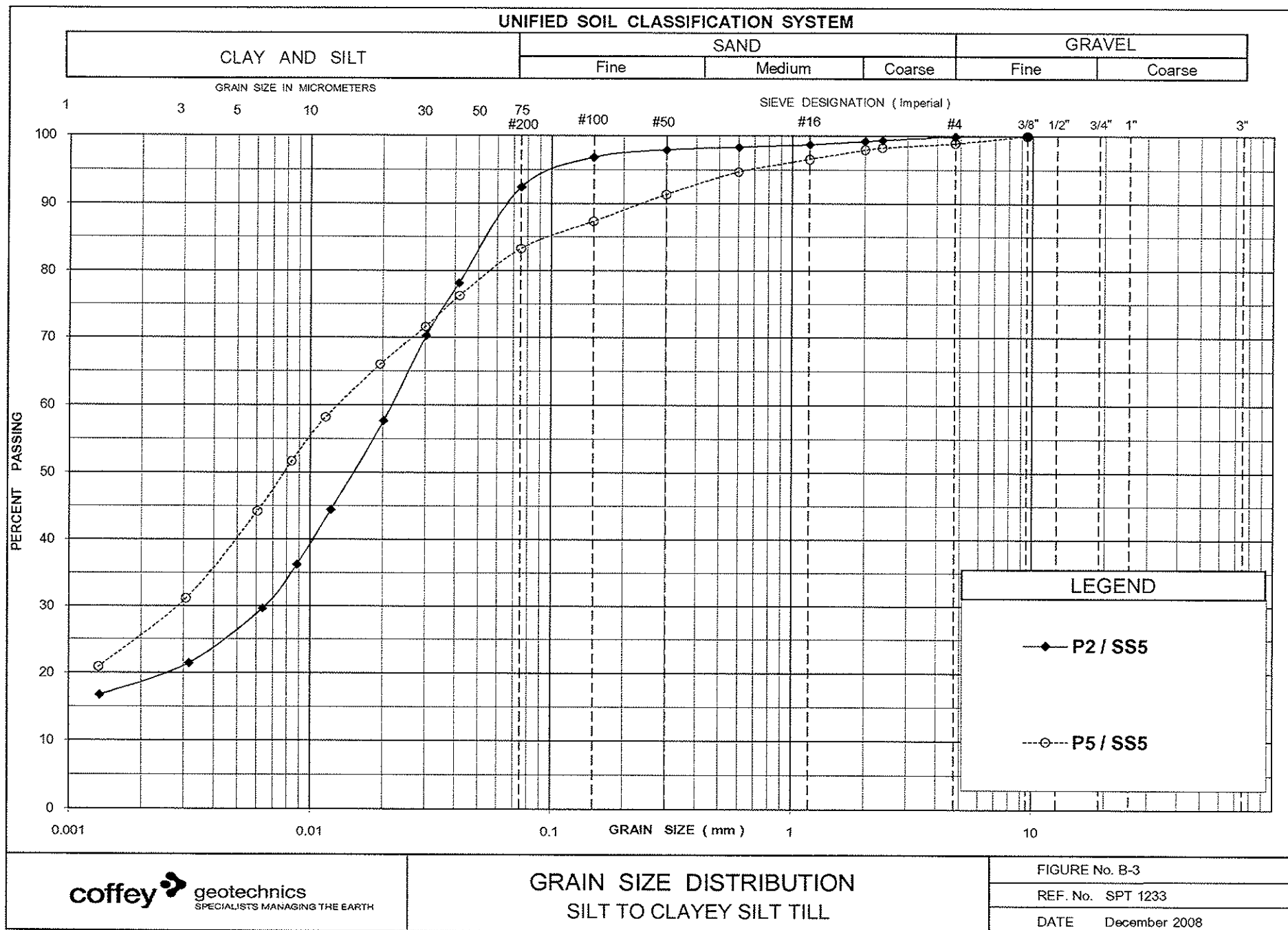
Appendix B

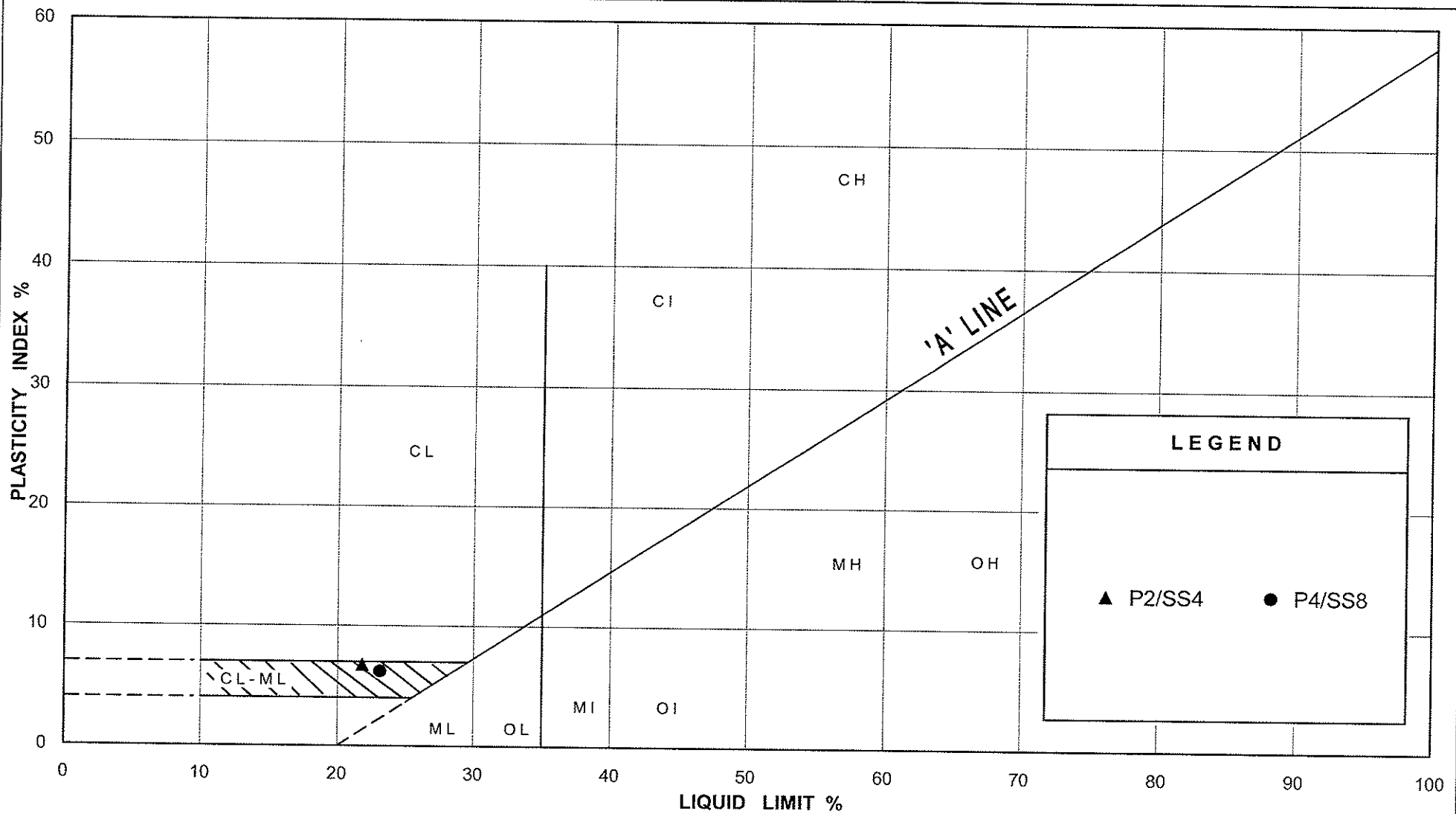
Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM









Appendix C

Site Photographs



Photograph 1 Outlet of Burntwood Creek Culvert (east side)



Photograph 2 Highway 634 over Burntwood Creek culvert (looking north)



Photograph 3 East side of Embankment showing erosion of slope



Photograph 4 Inlet of Burntwood Creek Culvert (west side)

Appendix D

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS \bar{N} .

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
j_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
j_w	kN/m ³	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
j	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
j_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_p) / I_p$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDAULIC GRADIENT
j_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(W_L - W) / 1_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m ³	DENSITY OF SUBMERED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
j'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
BURNTWOOD CREEK CULVERT,
SITE NO. 39E-247/C, HIGHWAY 634
AVON TOWNSHIP, ONTARIO
GEOCRES NO. 42H-36 W.P. 5121-05-00**

D. M. Wills Associates Limited

Project: SPT1233
April 6, 2009

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Appendix E: Proposed Staging Drawings

Appendix F: OPSD

Appendix G: Limitations of Report

**FOUNDATION DESIGN REPORT
BURNTWOOD CREEK CULVERT
HIGHWAY 634, SITE NO. 39E-247/C
AVON TOWNSHIP, ONTARIO**

5 DISCUSSION AND RECOMMENDATIONS

Burntwood Creek flows easterly under Highway 634 at Station 17+240 in Avon Township, near Fraserdale, Ontario, about 65km north of the junction of Highways 11 and 634. The existing culvert consists of twin structural plate corrugated steel pipes, 3.2 m in diameter and 48.5 m in length.

According to data supplied by D. M. Wills Associates Limited (Wills), the inverts of the existing culverts are about Elevation 211.5 to 212.1m at the outlet on the east side of the highway, rising to the inlet on the west side of the highway. The side slopes of the existing highway embankment are at about 3H:1V or flatter. The highway embankment at the culvert outlet shows signs of erosion.

The culvert boreholes were advanced from 23.5 m west (Borehole FC1) and 25.5 m east (Borehole FC3) of the existing road centreline of Highway 634. Borehole FC2 was advanced from the existing Highway 634 shoulder. Boreholes FC1 and FC3 show, below a 0.15 to 0.25 m thick layer of topsoil, the presence of very loose to loose sandy silt to silty sand fill over loose/soft organic silt with peat in Borehole FC1. Borehole FC2 encountered a loose to compact silty sand to sandy silt embankment fill to a depth/elevation of 7.2 m/212.9 m. Below these surficial soils, the boreholes encountered a compact to dense silt deposit to the termination depth of 10.1 to 11.1 m (Elevations 205.5 to 209.0 m) at Boreholes FC2 and FC3, while the base of the deposit was encountered in Borehole FC1 at a depth/elevation of 3.8 m/211.0 m. Below the silt deposit in Borehole FC1, a very stiff to hard clayey silt glacial till was encountered over the remaining depth of the borehole to a depth of 4.7 m/elevation 210.1 m. Practical refusal to further augering was obtained within the boreholes at depths of 4.7 to 11.1 m, or elevations 205.5 to 210.1 m, on boulders or bedrock.

The boreholes advanced for the proposed road regrading (P1 to P5) were extended along the highway shoulder and the traveling lane. Below the pavement structure the boreholes encountered the loose to compact sandy silt to silty sand embankment fill to depths of 2.2 to 7.7 m or elevation 221.4 to 212.3 m. The fill embankment is underlain by a 0.3 to 0.6 m thick topsoil layer in Boreholes P1 and P3. Beneath the embankment fill and the topsoil, the boreholes contacted a silt deposit or a silt to clayey silt till deposit to the full depth of the boreholes.

At the time of our investigation, the groundwater level was encountered at about Elevation 212 to 213 m adjacent to the culvert location and at about the o.g. level beyond the culvert in the P-series of boreholes, but would be subject to fluctuations. The groundwater level can also be expected to be controlled by the water level in the creek.

5.1 Replacement of the Existing Culvert

Based on the information from Wills, the existing twin culverts are to be replaced by a single 52 m long precast concrete structure which will be 3.0 m wide and 2.4 m high (inside dimensions). The culvert invert at the inlet will be at El. 211.87 m and 211.51 m at the outlet.

Assuming a bedding thickness of about 0.3 m to be placed beneath the culvert, the existing soils will be removed to about El. 211.5 and 211.2 m. At these elevations, Boreholes FC1, FC2 and FC3 show the presence of compact silt. In Borehole P3, which is also located close to the culvert location, topsoil was

contacted between El. 212.3 and 211.7 m, underlain by compact to dense silt. The silt is a dilatant material which will need to be properly stabilized by dewatering, as the water table at the site is high.

With prevailing subsurface conditions, the use of an open bottom concrete culvert is not recommended, since the silt is highly erodible and as well, since it is a dilatant material, it will require careful dewatering to effect the construction of normal spread footing foundations. A concrete box culvert would technically be more suitable in comparison with an open bottom concrete structure, as it would be more reliable. A cast-in-place concrete culvert supported on driven piles would present a reliable solution, but while technically reliable, it would present a very expensive alternative. As well, the piles may be very short or obstructions may be encountered while driving. Based on these considerations, as well as the relatively long time required to effect the construction, the use of a closed bottom, cast-in-place concrete structure, supported on deep foundations is also not recommended. Similarly, a cast-in-place concrete box culvert would likely be unsuitable due to the time and effort to effect the construction, including a lengthy dewatering period.

With the prevailing subsurface conditions (i.e. dilatant silt subgrade, coupled with high water table), the placement of a CSP type culvert would be the most suitable and cost-effective, but if necessary, a precast concrete box culvert can also be considered, as is presently planned for.

The following table presents the recommended soil resistances for these two alternatives, assuming that the existing highway grade will remain the same and that there will be no widening of the road embankment.

Table 5.1.1: Recommended Geotechnical Resistances

Borehole No.	Existing Ground Elevation (m)	Approximate Elevations of Proposed Culvert Stripping to Beneath the Granular Bedding (m)	Depth Below Existing Ground Surface (m)	Bearing Resistance at S.L.S. (kPa)	Factored Bearing Resistance at U.L.S. (kPa)	Subgrade Material
FC1	214.8	212.0	2.8	150	250	Compact silt
FC2	220.1	211.6	8.5	250	450	Compact to dense silt
FC3	215.6	211.2	4.4	300	450	Compact to dense silt
P3	220.0	211.6	8.4	250	450	Compact to dense silt

Based on the above, a uniform Bearing Resistance of S.L.S. of 150 kPa and a Factored Bearing Resistance U.L.S. of 250 kPa can be used at the site, which is considered ample for this project.

The factored bearing resistance at U.L.S. given in the above table incorporates a resistance factor of 0.5 as per the Canadian Highway Bridge Design Code, CAN/CSA-S6-00.

The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively for a precast concrete culvert and 15 mm for a CSP culvert. This can be achieved provided that the founding subgrade is undisturbed during the construction.

Cambering is not considered necessary at this site.

5.2 Corrugated Steel Pipe (CSP) Type Culvert

The native compact to dense silt, in its undisturbed state, is suitable to support a flexible structure provided a suitable bedding is placed between the properly dewatered silt subgrade and the pipe, as well as around and above the pipe. Due to high water table, considerable dewatering effort will be required to preserve the load carrying capacity of the subgrade soils and to facilitate the construction.

A minimum granular bedding thickness of 300 mm is recommended for a CSP type culvert. After stripping the site to the proposed bottom of bedding elevation, the exposed subgrade should be inspected and approved by the geotechnical engineer appointed by QVE (Quality Verification Engineer). If organic or other unsuitable soils are found (e.g. the topsoil at Borehole P3 location), these unsuitable soils should be removed and replaced with compacted granular soils.

It should also be pointed out that CSP culverts require adequate side support to maintain their structural integrity. The removal of unsuitable soils should therefore extend a suitable distance beyond the footprint of the culvert and adequate compaction should be applied to the surrounding fills.

5.3 Precast Concrete Box Culvert

We understand that at present the use of 3.0 m wide by 2.4 m high precast culvert is planned. The use of a precast concrete box culvert provides the greatest longevity given the soft water conditions that are presented at Burntwood Creek.

A minimum granular bedding of 400 mm is recommended for a precast concrete box culvert to provide a uniform working platform and support for the structure. After excavating to the proposed grade elevation (i.e. 0.4 m below the bottom of the concrete box) the exposed subgrade should be carefully inspected and approved by the geotechnical engineer appointed by QVE (Quality Verification Engineer). If organic or otherwise unsuitable soils are encountered such soils should be removed to the surface of natural, acceptable inorganic soil, and replaced with suitable compacted granular soils.

5.4 Bedding

The bedding material should consist of an approved well-graded granular material, such as Granular B Type II. The bedding material should be placed as soon as practicable after the preparation of the subgrade, its inspection and approval, as discussed in Sections 5.2 and 5.3 of this report. The bedding material should be in accordance with the appropriate standards (e.g. OPSD-802.010 and 802.014 for flexible pipe and OPSD 802.030, 802.031 and 802.033 for rigid pipe) and should consist of not less than the following thicknesses:

CSP Type Culvert	=	300 mm
Precast Concrete Box	=	400 mm
Cast-in-Place Concrete supported on deep foundations	=	200 mm

The bedding material should be compacted to MTO standards.

5.5 Backfilling

The bedding and embedment material should be extended along the sides and the top to cover the pipe. The selection and placing of the backfill should be in accordance with OPSD-802.010, OPSD-802.014 and 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 before compaction) and each lift should be compacted to at least 96% of the material's SPMDD (Standard Proctor Maximum Dry Density). The Granular 'A' and Granular 'B' sub-base courses should be compacted to 100% of the material's SPMDD.

We would like to point out that the performance of flexible pipe culverts (especially arch types) is largely dependent on the side support provided by the backfill and the adjacent soils. The use of adequate backfill material and especially good compaction are, therefore, necessary for proper side support. For the same reason, the organic soils should be removed within a suitable distance from the footprint of the culvert. The use of heavy compaction equipment should be avoided immediately adjacent and above the pipe, as per MTO practice. During backfill placement, the height of the backfill should be maintained at approximately same level on both sides of the pipe, to avoid lateral displacement of the pipe.

For precast concrete box culvert, 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 200 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.

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

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

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

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

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

Unit weight = 22 kN/m³

Coefficient of Lateral Earth Pressure:

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

Compacted Granular 'B' Type I

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

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

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

Note: K_a is the coefficient of active earth pressure

K_b is the backfill earth pressure coefficient for an unrestrained structure including compaction efforts

K_o is the coefficient of earth pressure at rest

K^* is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

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

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

5.6 Construction Comments

To maintain the flow of water in the Burntwood Creek across the highway, one of the existing culverts can be used, which would be removed after the construction of the new culvert. If this is not the preferred option, a temporary diversion with either pumping across the highway or a temporary pipe can be placed in which the flow of water across the highway can be maintained.

After making provisions for maintaining the flow of water, dewatering will be required to facilitate the construction and to preserve the load carrying capability of the silt subgrade.

Depending on the conditions at the site at the time of construction, this may be possible by pumping from strategically placed and properly filtered sumps, which will likely lower the water table by about 0.8 m, depending on the number and depth of the filtered sumps. If a greater degree of drawdown is required, vacuum well points may be necessary in addition to pumping from the sumps. It is believed that a relatively more sophisticated dewatering system will be required for the installation of a concrete culvert in comparison with a CSP type culvert.

We recommend that the contractor be warned that the subgrade silts are dilatant materials, which, in the presence of water can dilate, a condition which can be recognized by the liverish, jelly-like appearance of the soil. If the new culvert is placed on dilated soils, excessive settlements can ensue after placing the backfill, especially in the case of a precast concrete culvert. We also recommend that the contractor be asked to submit their method of diversion and dewatering to the CA for information purposes.

The construction of the culvert should be in accordance with SP421S01 or SP422S01.

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

The following soil classifications can be expected for temporary excavations in accordance with OHSA.

Fill : Type 4 soil.

Upper Zones of Embankment Fill (excluding lower 1m): Type 3 soil above water table (with side slopes no steeper than 1 1/2H:1V) and Type 4 soil below water table

Lower 1 m of Embankment Fill : Type 4 soil

Organic Soils : Type 4 soil

Silt : Type 3 soil above water table and Type 4 soil below water table

Silt Till : Type 3 soil above water table and Type 4 soil below water table

Clayey Silt Till : Type 2 soil

It should be pointed out that the presence of cobbles and boulders can be expected in the fill and overburden materials, especially in the glacial tills. You may wish to issue a reminder to the Contractor (in the Contract Documents) to this effect.

We understand that construction staging will be used for this project without the use of roadway protection. Shoring will therefore unlikely be required. We have, however, included the following table for design parameters for shoring, for the sake of completeness.

Table 5.6.1: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	(kN^2/m^3)
Granular Fill and Upper portions of Embankment Fill	0.33	0.50	3.0	20.5
Lower 1 m of Embankment Fill and Other Fill	0.36	0.53	2.8	18.5
Organic Soils/Topsoil	0.49	0.70	1.4	13.0
Silt	0.35	0.52	2.9	18.0
Silt/Clayey Silt Till	0.31	0.47	3.2	21.0

It should be pointed out that cobbles and boulders could occur in the overburden. These can be expected to cause problems during the installation of the caisson holes and/or during the driving of the sheet piling. Deeper boreholes may be required depending on the shoring design.

5.7 Scour and Erosion Protection

Erosion and scour protection should be provided at the culvert inlet and outlet (including the slopes and sides). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. The following are some general suggestions, considering that the subgrade consists of highly erodible silts, which in some cases are overlain by organic soils.

We recommend that a concrete cut-off (apron) be constructed both at the inlet and outlet to prevent seepage beneath and 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 or equivalent. 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 0.5 m above the high water level in the watercourse down to the channel bed and up the other side in a continuous manner. It should be ensured that it extends to cover all the granular backfill materials to prevent any seepage through them. Typically, the clay seal is protected by laying a 0.6 m thick rock protection over it. The clay seal would generally be extended at about 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 8m along the channel and the sides (to at least 0.3 m above high water level). A geotextile separator (non-woven, Class II FOS 75 to 150 μm) or a proper natural filter material should be placed behind beneath the protection to prevent migration of fine particles into the rockfill due to seepage pressure. A toe for the filter and rip-rap protection should be provided at the edge of the lining and protective cover to key the lining into the natural ground to provide protection to erosion and scour.

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

5.8 Construction Staging

We understand that the preferred method of replacement is staging (half and half) with single lane of traffic in a revised alignment. The proposed staging will be accommodated within the embankment platform and will be carried out by lowering the road grade temporarily by up to 3.5 m, as shown on the attached staging drawings in Appendix E, provided to us by Wills.

In accordance with the proposed staging scheme, after lowering the grade by 1.0 m over the entire width of the road, the traffic on Highway 634 will be reduced to a single lane on the east side, while the existing grade on the west side will be lowered by 2.5 m to about 216.5 m, using 3H:1V side slopes (see Pre-Stage 1 sketch on Proposed Staging Drawing No. E2 by Wills, included in Appendix E).

After lowering the grade, a single lane traffic road surface will be prepared on the west side and the traffic will shift to this side, while the east half of the new culvert is being constructed. During this time period, temporary side slopes of 2H:1V will be utilized as shown on Stage 1 sketch on the Proposed Staging Drawing in Appendix E. After installing the eastern portion of the culvert, the grade will be raised to El. 216.5 m over the completed portion of the culvert. The single lane traffic will then be diverted to this (east) side (i.e. on to the top of the completed portion of the new culvert).

The installation of the western portion of the new culvert will then be completed while a single lane traffic is maintained on the east side. The temporary slopes will be 2H:1V (see Stage 2 sketch).

Once the installation of the entire culvert is completed, the grade will be raised to 1.0 m below the final road grade of 220.0± m and the traffic will be diverted to the completed west section. As shown on the 'Final Construction' phase sketch, the grade of the east side will then be raised to the road final grade of about El. 202.0 m and thereafter the highway will be opened to normal (two-lane) traffic.

It is our opinion that this approach is feasible provided that the site was successfully unwatered. The following are a few comments from a geotechnical point of view.

We recommend that the sufficiency of the cover above the new culvert during the staging process (see Stage 2 phase) be ensured so that the integrity of the culvert under traffic loading will not be compromised.

At Pre-stage1 (i.e. advanced grade lowering by 1.0 m) and at El. 216.5 m (i.e. top of detour lane on the west side during Stage 1 phase), Boreholes FC2, P3 and P4 show the presence of loose to compact (i.e. N-values of 9-29 blows/0.3 m) sandy silt to silty sand (embankment) fill. This is considered suitable to support a detour road for a short period of time but we recommend that sufficient granular pavement material be placed to carry the traffic loads. We suggest that after stripping, the exposed subgrade be inspected and approved. The approved subgrade should be properly compacted from the surface. We suggest that a minimum of 300 mm well compacted suitable granular soil be used to sustain the traffic loads. Ideally this should consist of 150 mm of Granular 'B' Type II overlain by 150 mm of Granular 'A' material. If this is not feasible 300 mm of Granular 'B' Type II can be used. If this material is unavailable 300 mm of Granular 'A' would also be acceptable. These materials should be compacted to 100% of their respective Standard Proctor Maximum Dry Density (SPMDD). The thickness may need to be adjusted (i.e. increased depending on the inspection of the subgrade). We recommend that a provision for this be made in the contract.

It should be noted that beyond the culvert location, the revised alignment may not be entirely within the embankment. In these areas a cut of the o.g./existing side slopes may be required. In cut sections,

3H:1V side slopes are being placed and we concur with this. If in some areas, minor filling is required, the existing granular pavement fill would be suitable for filling. The underlying sandy silt to silty sand embankment fill can also be used but due to high silt content, the moisture condition at the time of the filling will need to be near the optimum and the fill will need to be placed sufficiently slowly so that construction pore pressures (i.e. bulking) will be minimized. In addition, frost susceptible nature of the soil may need to be taken into consideration, where appropriate.

During their use, the sides of the single lane traffic will need to be adequately protected (e.g. barricaded or sufficiently offset from vehicular traffic) from wheel loads to prevent a shear failure, as well as restricting the speed limit for both options. The sides of the excavation should be carefully observed for the duration of the construction for any signs of impending instability. As well, depending on the adequacy of dewatering the lower portions of the side slopes may show signs of instability, especially since the lower portion of the embankment fills do not appear to be well-compacted (e.g. below El. 214 ± m). Proper dewatering of these lower portions of the side slopes is essential. In addition, this aspect may need to be observed by the QVE, and if necessary, monitoring of the side slope should be carried out.

Proper benching should be applied during backfilling between stages.

5.9 Frost Protection

Design frost protection for the general area is 2.7 m. A permanent soil cover of at least 2.7 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 ensured to be effective in providing frost protection.

6 CLOSURE


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


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

For and on behalf of Coffey Geotechnics Inc.


Ramon Miranda, P.Eng.




Zuhtu Ozden, P.Eng.


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



Appendix E

Proposed Staging Drawings

MINISTRY OF TRANSPORTATION, ONTARIO
PROJECT 107 85-15

METRIC

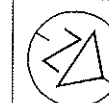
CONT
WP 5121-05-00

BURNTWOOD CREEK
STAGING PLAN

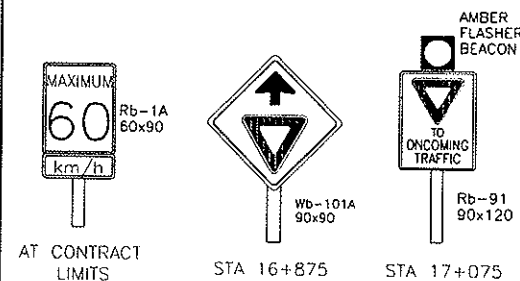
WILLS

D.M. Wills Associates Limited
Consulting Engineers

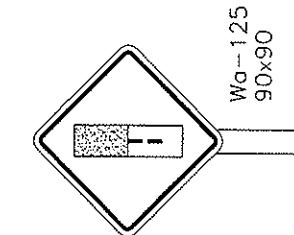
dmwllb.com



SHEET
E 1



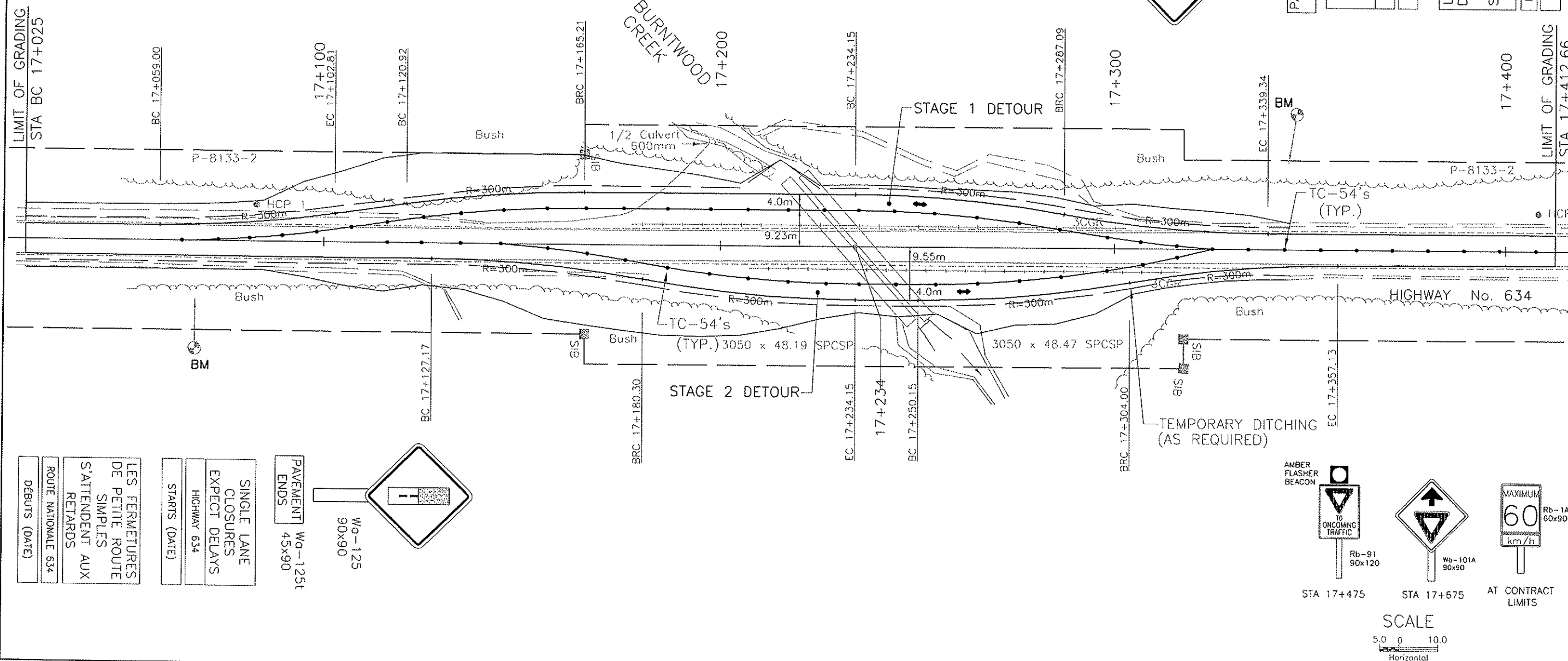
DISTRICT COCHRANE
GEOG TWP AVON
(UNSUBDIVIDED)



Wb-125
90x90
PAVEMENT
ENDS

SINGLE LANE
CLOSURES
EXPECT DELAYS
HIGHWAY 634
STARTS (DATE)

LES FERMETURES
DE PETITE ROUTE
SIMPLES
S'ATTENDENT AUX
RETARDS
ROUTE NATIONALE 634
DÉBUTS (DATE)



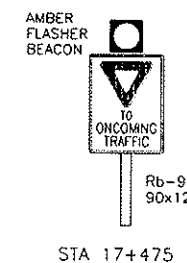
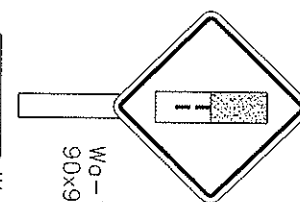
LES FERMETURES
DE PETITE ROUTE
SIMPLES
S'ATTENDENT AUX
RETARDS
ROUTE NATIONALE 634
DÉBUTS (DATE)

STARTS (DATE)
HIGHWAY 634

EXPECT DELAYS

SINGLE LANE
CLOSURES
EXPECT DELAYS

PAVEMENT
ENDS
Wb-125
90x90



SCALE

5.0 0 10.0
Horizontal

MINISTRY OF TRANSPORTATION, ONTARIO
PR-1-101 05-05

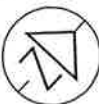
METRIC

CONT
WP 5121-05-00

BURNWOOD CREEK
SUGGESTED STAGING

WILLS
D.M. Wills Associates Limited
Consulting Engineers

dmwills.com

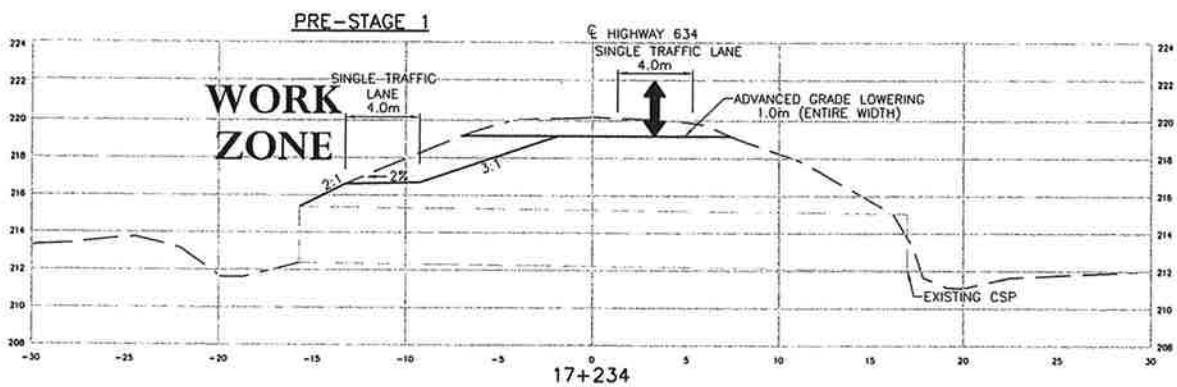


SHEET
E 2

PRE-STAGE 1

ADVANCED GRADE LOWERING OF 1.0m OVER ENTIRE WIDTH OF ROAD. MOVE TRAFFIC TO SINGLE LANE ON EXISTING EAST SIDE. LOWER EXISTING GRADE AN ADDITIONAL 2.5m± ON WEST SIDE.

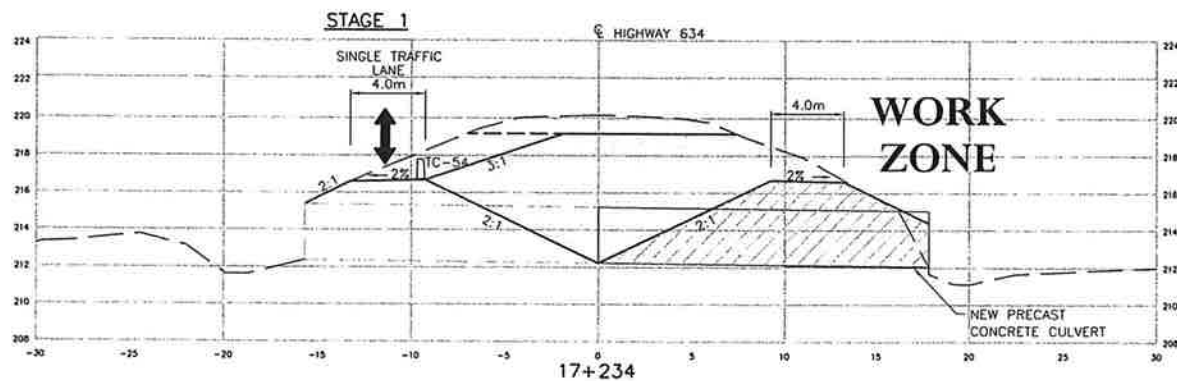
WEST



EAST

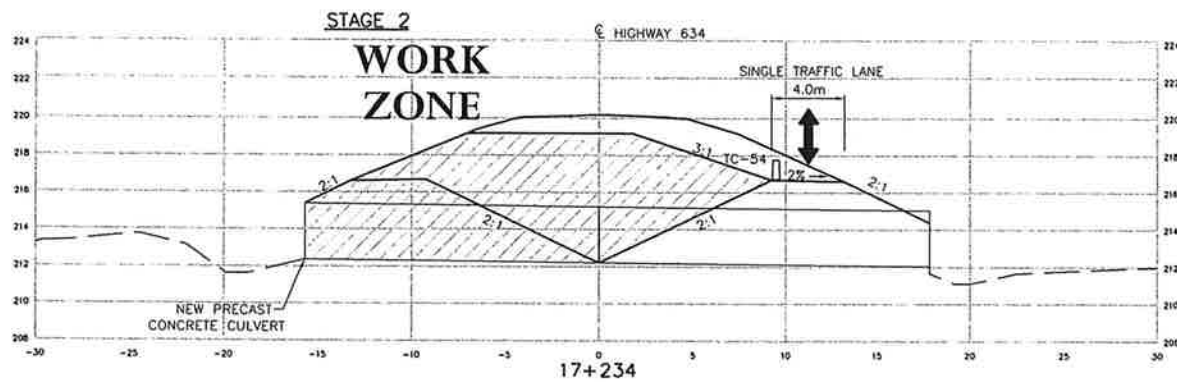
STAGE 1

SINGLE LANE TRAFFIC SHIFT TO DETOUR ON WEST SIDE, CULVERT AND DETOUR CONSTRUCTION ON EAST SIDE



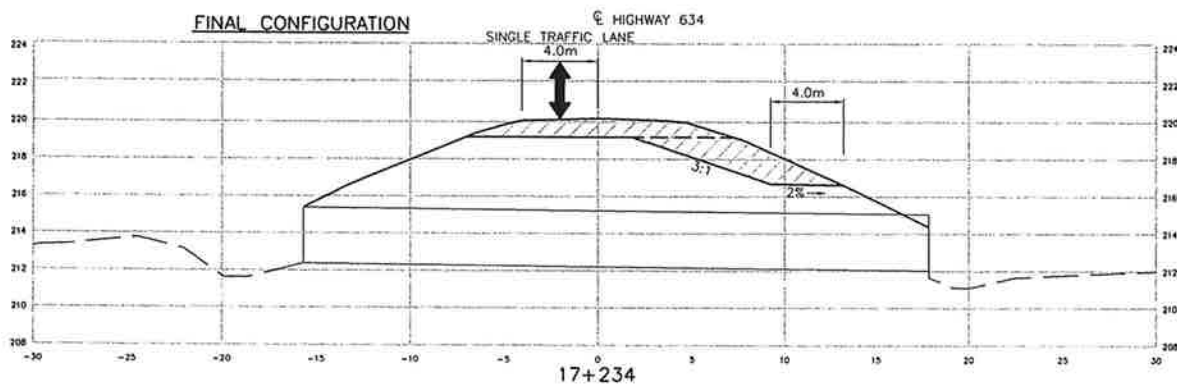
STAGE 2

SINGLE LANE TRAFFIC SHIFT TO DETOUR ON EAST SIDE, CULVERT CONSTRUCTION AND BACKFILL ON WEST SIDE TO 1.0m BELOW FINAL GRADE.



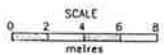
FINAL CONSTRUCTION

SINGLE LANE TRAFFIC RETURNS TO EXISTING ALIGNMENT ON WEST SIDE AND BACKFILL TO FINAL GRADES. PAVE HIGHWAY 634 AND OPEN TO TWO LANES OF TRAFFIC.



LEGEND:

- REMOVALS / EXCAVATION
- NEW CONSTRUCTION



METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

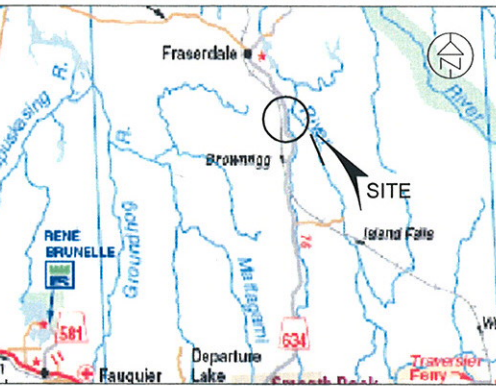
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No.

WP: 5121-05-00

HIGHWAY 634,
BURNTWOOD CREEK
BOREHOLE LOCATION PLAN

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

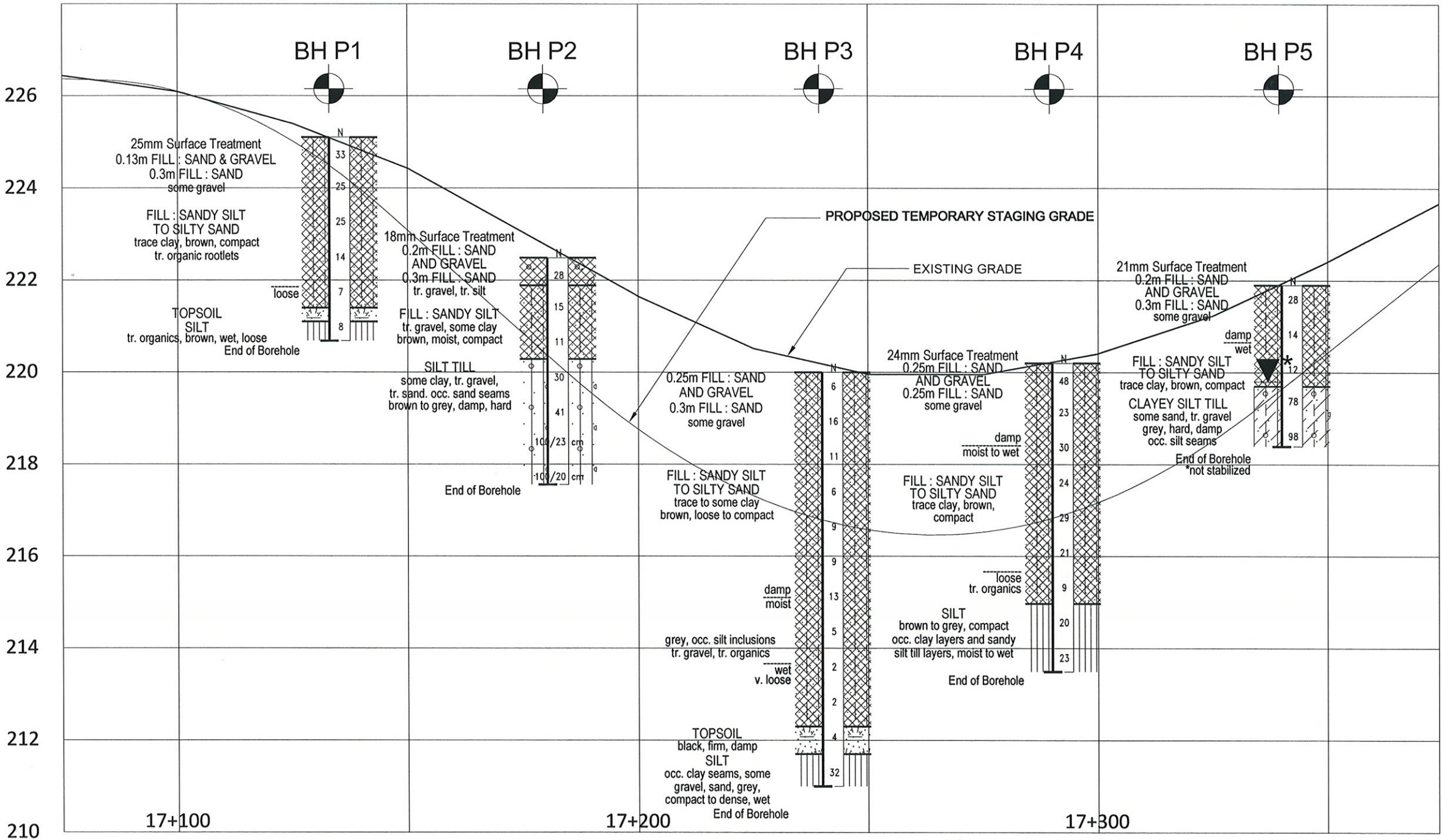
No.	ELEV.	STATION No.	OFFSET
BH P1	225.1	17+133	2.0m Rt C/L
BH P2	222.5	17+183	2.0m Rt C/L
BH P3	220.0	17+240	4.5m Lt C/L
BH P4	220.2	17+290	2.0m Lt C/L
BH P5	221.9	17+340	2.0m Lt C/L

NOTE

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

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

REV.	DATE	BY	DESCRIPTION
Geocres No. 42H-36			
SPT 1233			DIST
SUBM'D	CHECKED	DATE Jan 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG E3

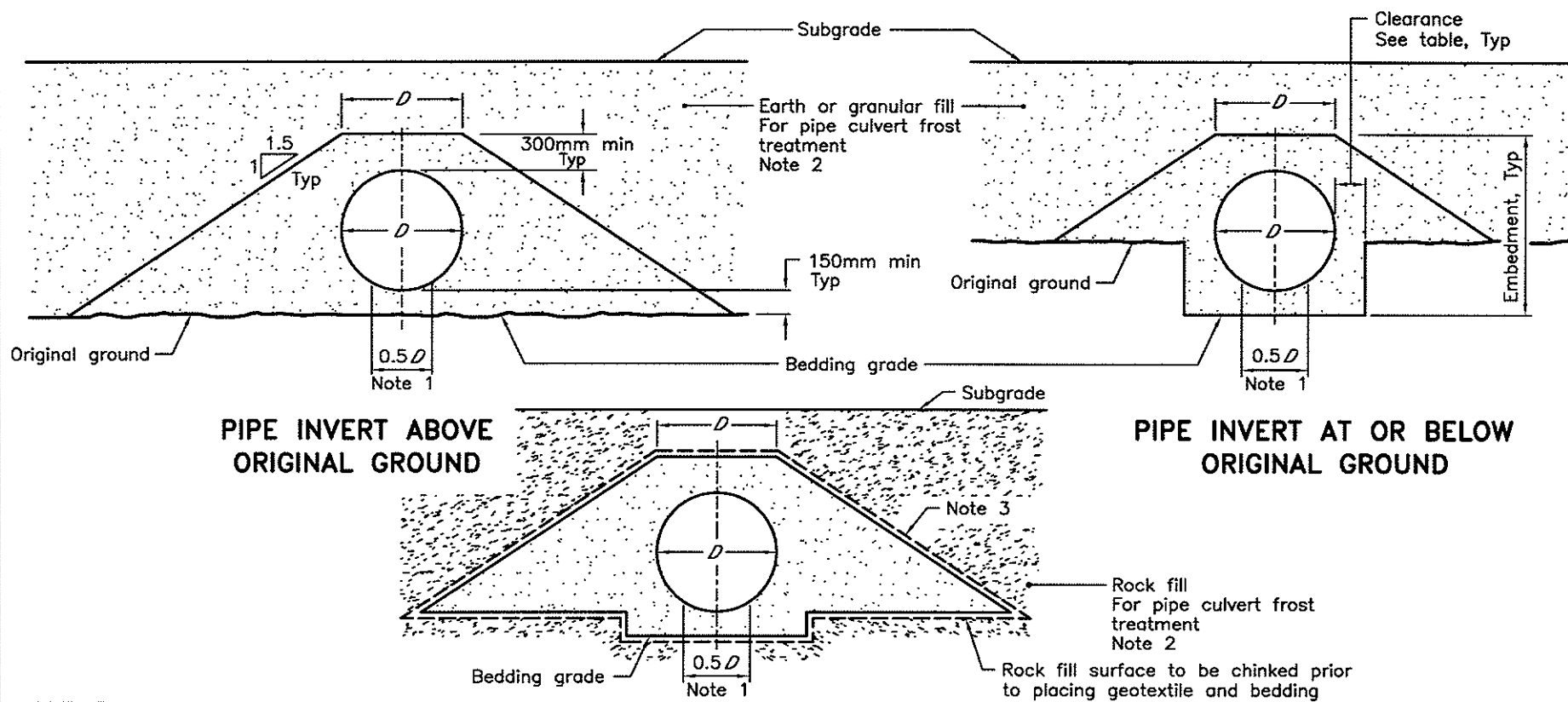


PROFILE



Appendix F

OPSD



LEGEND:

D - Inside diameter

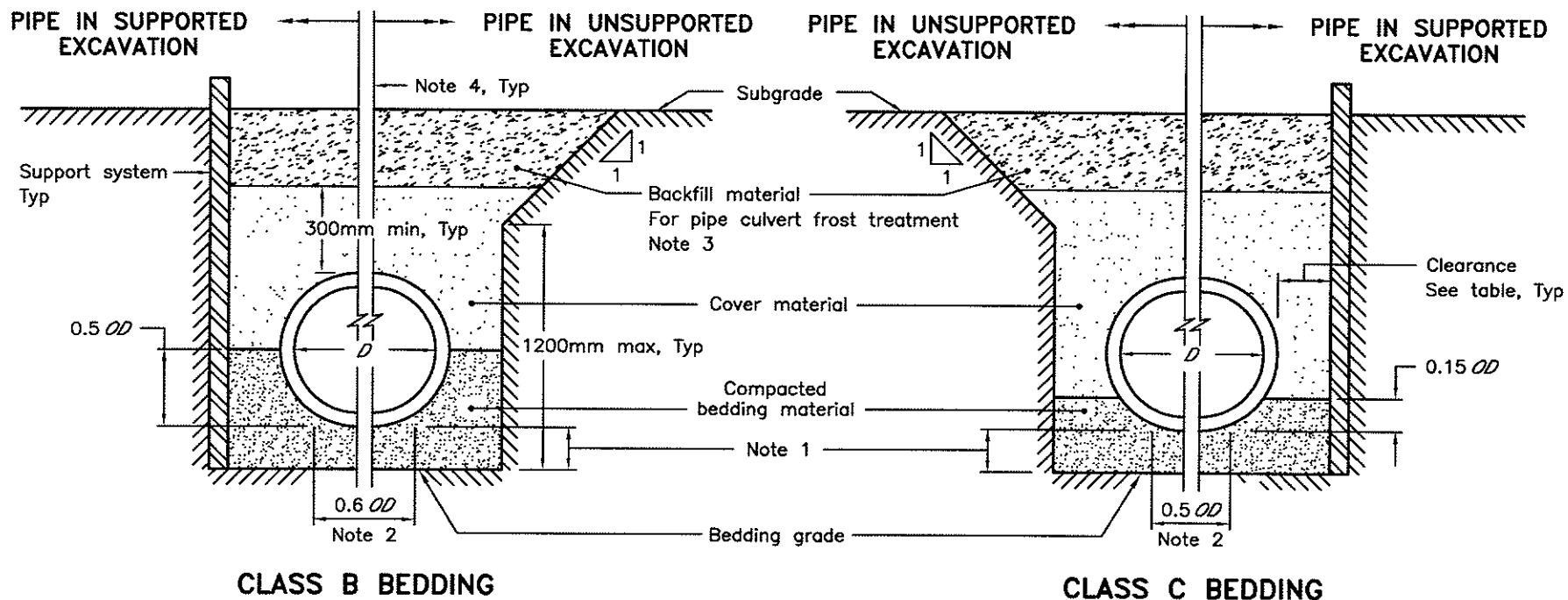
NOTES:

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

PIPE EMBEDMENT WITH ROCK FILL UNDER AND OVER THE PIPE

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

ONTARIO PROVINCIAL STANDARD DRAWING		Nov 2005	Rev 1	
FLEXIBLE PIPE EMBEDMENT IN EMBANKMENT				
ORIGINAL GROUND: EARTH OR ROCK				
OPSD - 802.014				



NOTES:

- 1 The minimum bedding depth below the pipe shall be $0.15D$. In no case shall this dimension be less than 150mm or greater than 300mm.
 - 2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 3 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
 - 4 Condition of trench is symmetrical about centreline of pipe.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- B All dimensions are in metres unless otherwise shown.

LEGEND:

D - Inside diameter
 OD - Outside diameter

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

ONTARIO PROVINCIAL STANDARD DRAWING

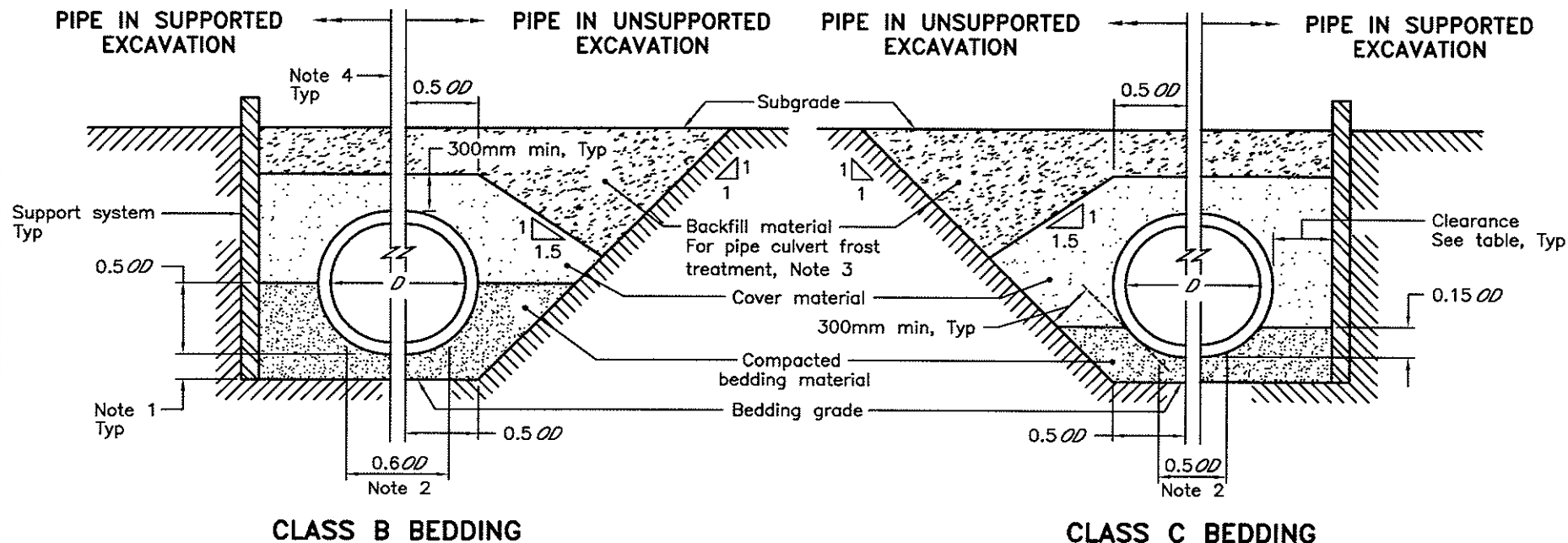
Nov 2005

Rev 1

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

OPSD - 802.030





NOTES:

- 1 The minimum bedding depth below the pipe shall be $0.15D$. In no case shall this dimension be less than 150mm or greater than 300mm.
 - 2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
 - 3 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
 - 4 Condition of trench is symmetrical about centreline of pipe.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- B All dimensions are in metres unless otherwise shown.

LEGEND:

D – Inside diameter
 OD – Outside diameter

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005

Rev 1

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

OPSD – 802.031

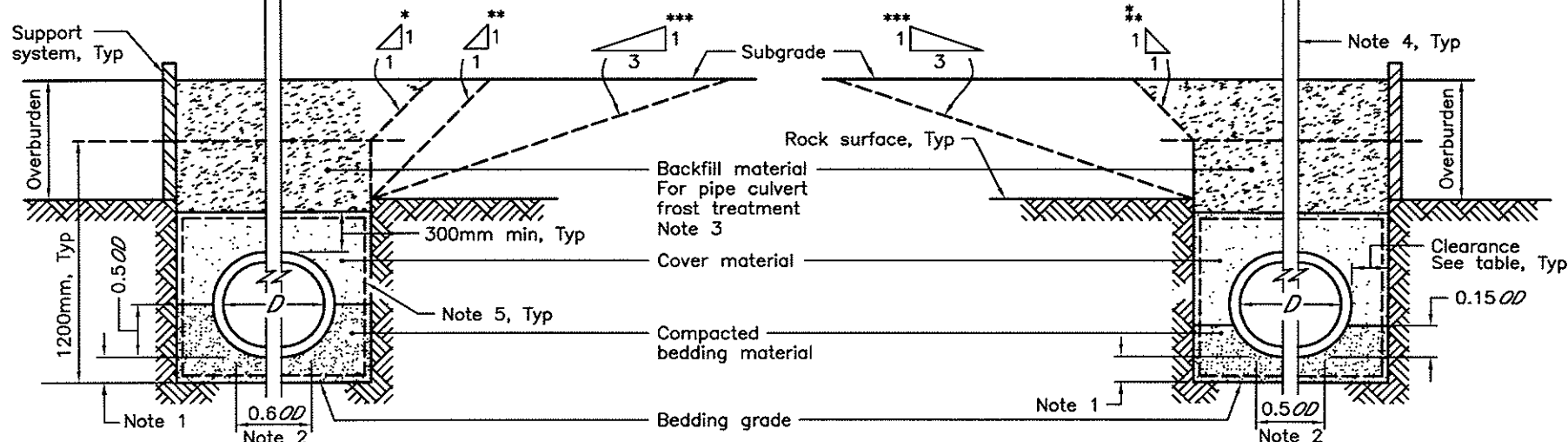


PIPE IN SUPPORTED EXCAVATION

PIPE IN UNSUPPORTED EXCAVATION

PIPE IN UNSUPPORTED EXCAVATION

PIPE IN SUPPORTED EXCAVATION



CLASS B BEDDING

CLASS C BEDDING

NOTES:

- 1 The minimum bedding depth below the pipe shall be $0.25D$. In no case shall this dimension be less than 150mm or greater than 300mm.
- 2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
- 3 Pipe culvert frost treatment according to OPSD-803.030 and 803.031.
- 4 Condition of trench is symmetrical about centreline of pipe.
- 5 Embedment material to be wrapped in non-woven geotextile when specified.
- A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
- B Fractured rock to be treated as Type 1 soil.
- C All dimensions are in metres unless otherwise shown.

LEGEND:

- D - Inside diameter
 OD - Outside diameter
 * - Type 1 or 2 soil
 ** - Type 3 soil
 *** - Type 4 soil

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

ONTARIO PROVINCIAL STANDARD DRAWING

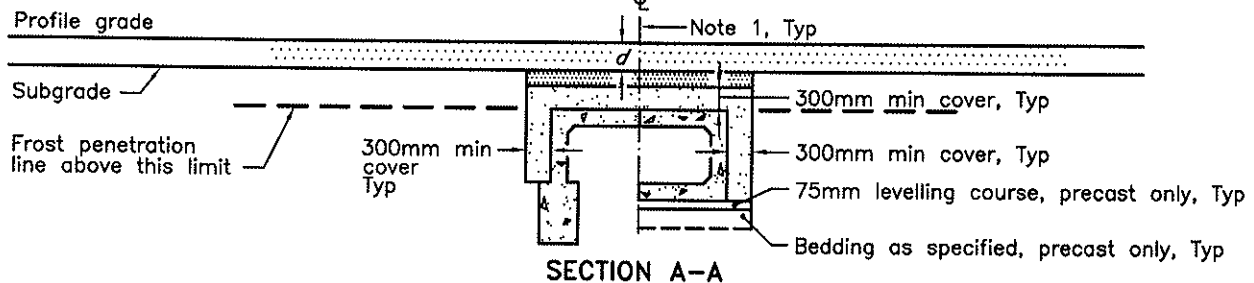
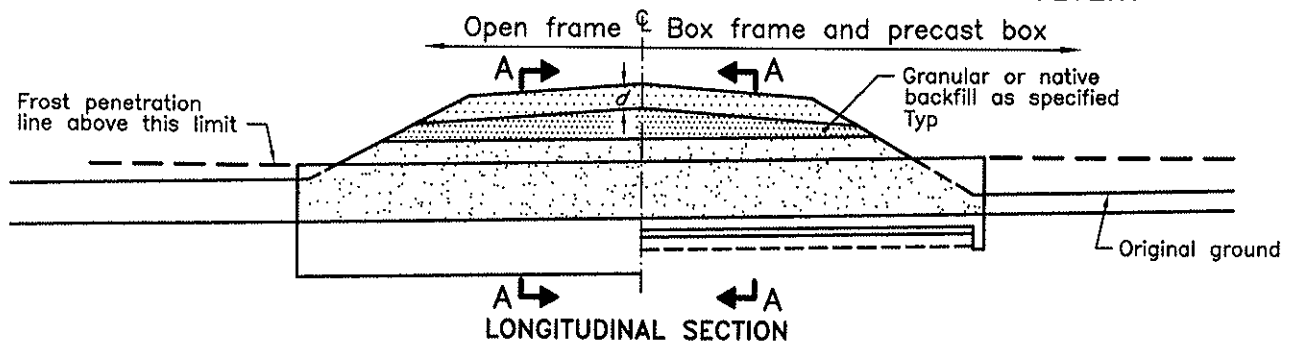
Nov 2005 Rev 1

RIGID PIPE BEDDING,
COVER, AND BACKFILL
ROCK EXCAVATION

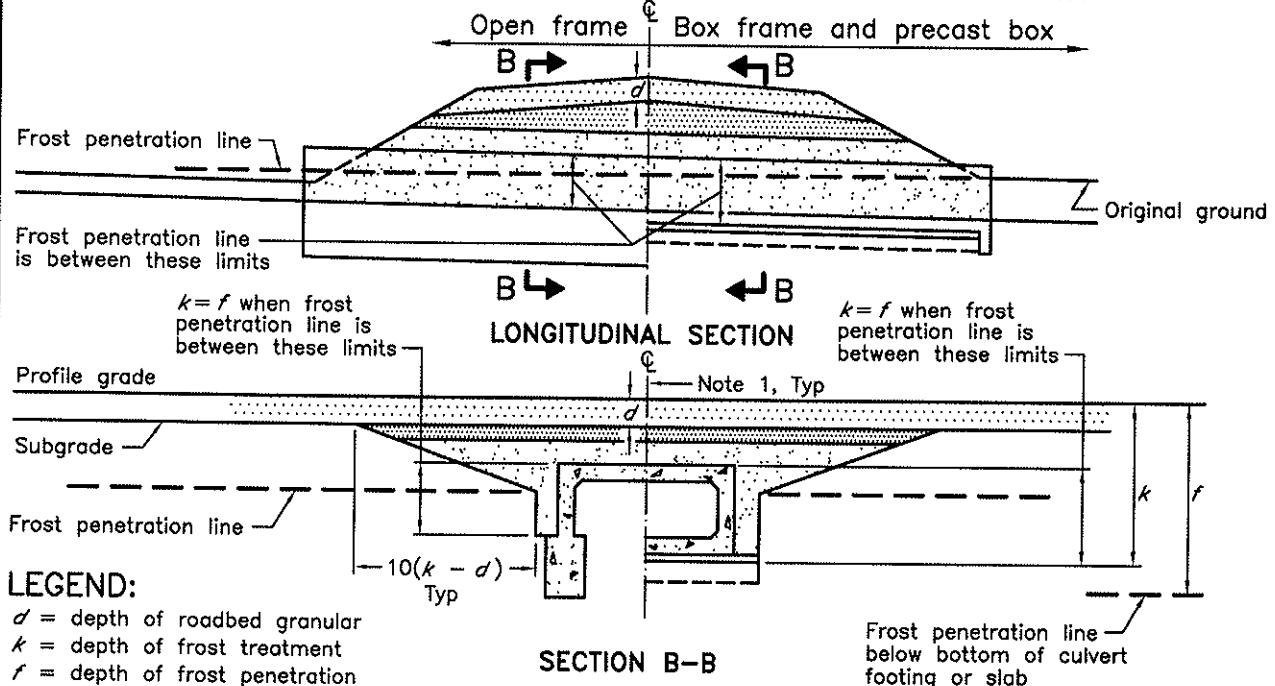
OPSD - 802.033



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.

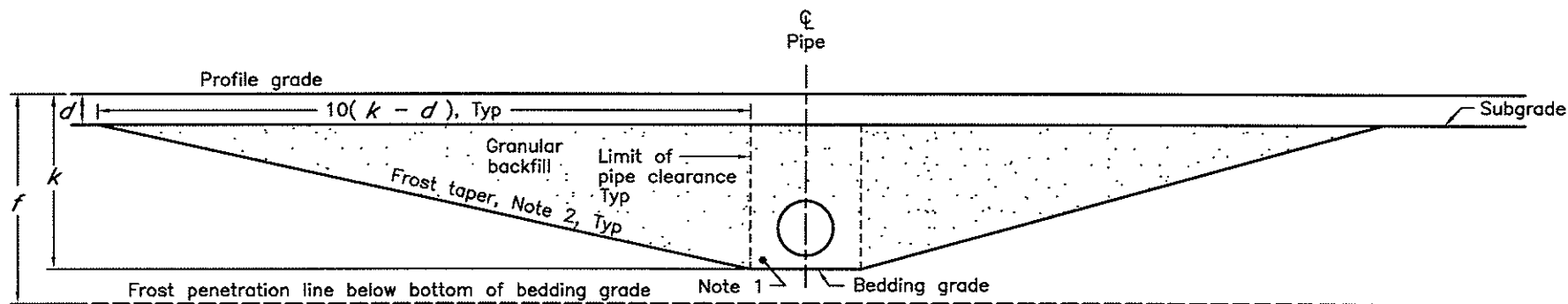
ONTARIO PROVINCIAL STANDARD DRAWING

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

Nov 2005

Rev 1

FROST TREATMENT – PIPE CULVERTS
FROST PENETRATION LINE BELOW
BEDDING GRADE

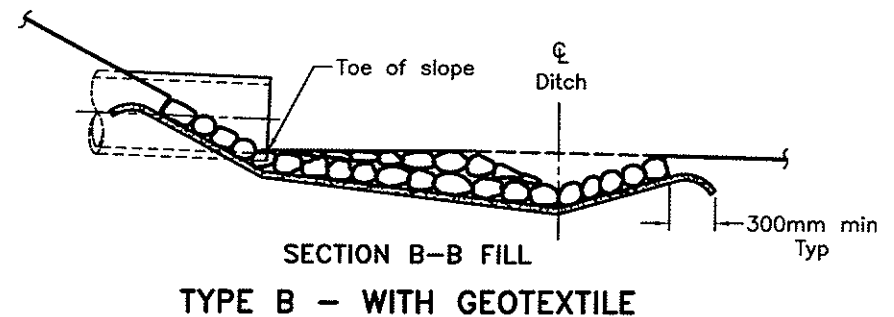
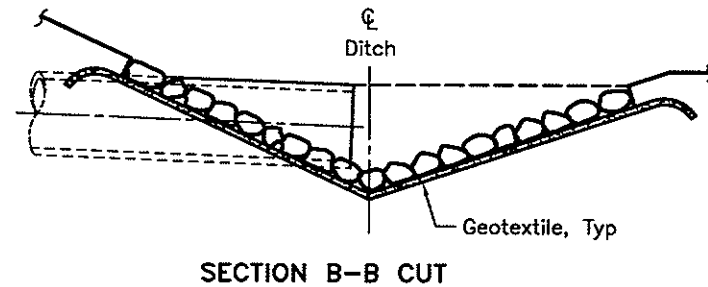
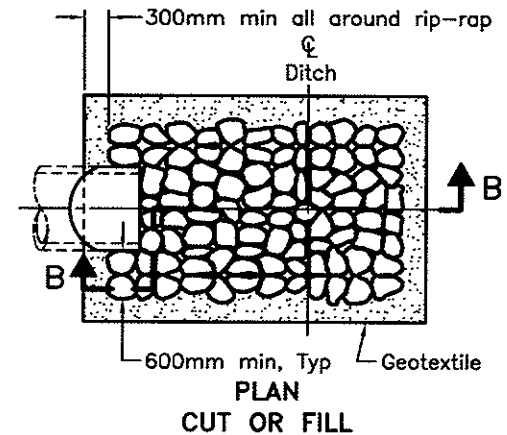
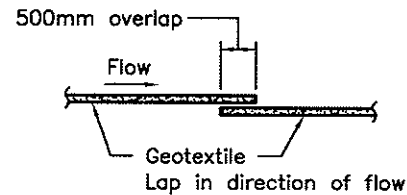
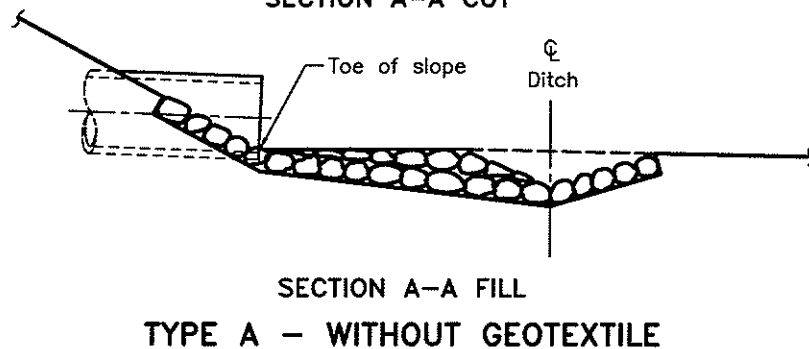
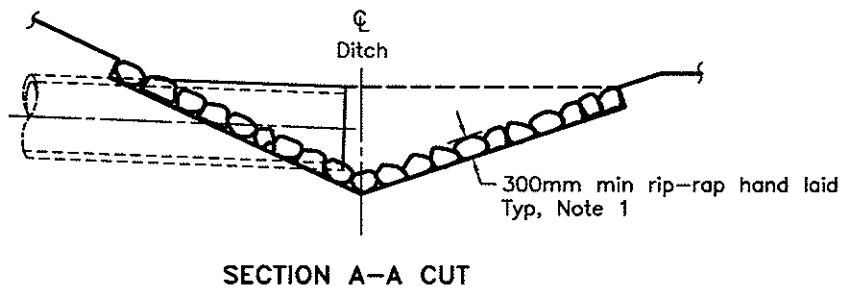
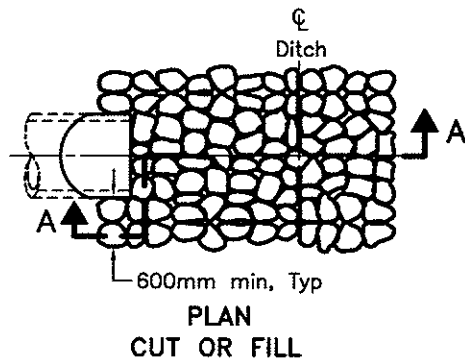
OPSD – 803.030





- 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.

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



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.