coffey geotechnics
SPECIALISTS MANAGING THE EARTH

50 YEARS AN EXTRAORDINARY JOURNEY

FOUNDATION INVESTIGATION AND DESIGN REPORTS – CULVERT REPLACEMENT AT STATION 17+625, HIGHWAY 66, TOWNSHIP OF MCGARRY, ONTARIO, G.W.P. NO. 5117-03-00 GEOCRES NO. 32D-10

Stantec Consulting Ltd

TRANETOB01241AA-AA
April 9, 2010

Coffey Geotechnics Inc.
20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

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April 9, 2010

Stantec Consulting Ltd
1400 Rymal Road East
Hamilton, Ontario
L8W 3N9

Attention: Mr. Adam Barg, P.Eng.

Dear Mr. Barg,

RE: Foundation Investigation and Design Reports, Culvert Replacement at Station 17+625 Highway 66, Township of McGarry, Ontario, G.W.P. No. 5117-03-00, Geocres No. 32D-10

Coffey Geotechnics Inc (Coffey) is pleased to present the Draft Foundation Investigation and Design Reports for the proposed culvert replacement and associated detour on Highway 66 at Station 17+625, Township of McGarry, Ontario.

Please call us on 416 213 1255 should you require further clarification on any aspects of the reports.

For and on behalf of Coffey Geotechnics Inc.

Ramon Miranda, P. Eng.

Manager, Transportation Division

Distribution: Original held by Coffey Geotechnics Inc

1 hard copy and 1 electronic copy to Stantec Consulting Ltd

5 hard copies and 1 electronic copy to MTO Project Manager

1 hard copy and 1 electronic copy to MTO Pavements and Foundation Section

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FOUNDATION INVESTIGATION REPORT CULVERT REPLACEMENT AT STATION 17+625, HIGHWAY 66, TOWNSHIP OF MCGARRY, ONTARIO G.W.P. NO. 5117-03-00 GEOCRES NO. 32D-10

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20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

CONTENTS

Tables

CONTENTS

FOUNDATION INVESTIGATION REPORT

CULVERT REPLACEMENT AT STATION 17+625

HIGHWAY 66, TOWNSHIP OF MCGARRY, ONTARIO

1 INTRODUCTION

At the request of Stantec Consulting Ltd., Coffey Geotechnics Inc. (Coffey) have prepared this foundation investigation report for the proposed replacement of the existing culvert and associated detour on Highway 66 at Station 17+625 in McGarry Township, Ontario. The foundation investigation was generally carried out in accordance with Coffey proposal (Reference PO 8968, dated 23 September 2008) and the requirements of the RFP.

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

This report provides factual information concerning subsurface conditions, in situ test results and laboratory test results, based on the foundation investigation undertaken.

2 SITE DESCRIPTION AND REGIONAL GEOLOGY

2.1 Site Description

The site is located at Station 17+625 on Highway 66 within the Township of McGarry, between Quebec-Ontario boundary and Virginiatown.

Highway 66, at the culvert location, is a 2 lane asphalt paved road, located on a fill embankment, about 5 m high with side slopes of about 2H:1V. The presence of rock fill was observed at the surface of the embankment side slopes and at the toe of the embankment. Highway 66 runs in an east-west direction and is gently sloping down to the east.

Rock cutting and rock outcrops were observed on the sides of the road.

Photographs of the site are presented in Appendix C.

2.2 Regional Geology

Based on Map 2647 “Quaternary Geology - Larder Lake Area” scale 1:50,000 published by Ontario Geological Survey, the site is underlain by Precambrian bedrock that is exposed or covered by till and glaciolacustrine coarse grained deposits composed of sand with gravel.

Based on Map 2628 “Precambrian Geology – Larder Lake Area”, scale 1:50,000 published by Ontario Geological Survey, the bedrock underlying the general area consists of the Coleman Member of the Huronian Supergroup. The Coleman Member is described to contain felspathic arenite, quartz arenite, arkose, pebbly conglomerate, argillite, paraconglomerate, pebbly agillite, feldspathic wacke, orthoconglomerate, laminated siltstone, massive siltstone, arkose and wacke.

3 METHOD OF INVESTIGATION

3.1 Fieldwork

The fieldwork for the investigation was carried out between August 20 to 22, 2009 and comprised of drilling six boreholes (C1 to C3 and D1 to D3). Drawing 1 shows the borehole locations. Table 1 below presents a summary of the borehole details.

Table 1: Summary of Borehole Details

Borehole No.	Station	Offset from road C/L	Location	Elevation (m)	Drilled depth (m)
C1	17+637	19.3 m Rt	Ditch	323.2	6.1
C2	17+623	4.6 m Rt	Shoulder	329.6	10.7
C3	17+625	15.5 m Lt	Ditch	325.9	8.7
D1	17+705	13.3 m Lt	Ditch	325.8	7.3
D2	17+665	15.1 m Lt	Ditch	325.8	8.0
D3	17+581	13.5 m Lt	Ditch	327.4	2.7

The borehole drilling was carried out by Landcore Drilling from Chelmsford, Ontario. The boreholes were drilled using a track mounted drilling rig. Each borehole was advanced using a solid flight auger until refusal was encountered between depths of about 2.6 m and 10.7 m below the ground surface. Rock coring techniques continued in Boreholes C1 and C3 to a maximum depth of 8.7 m below the ground surface. Standard Penetration Tests (SPTs) were carried out in the overburden at selected depth intervals, to assess the soil strength and obtain samples for logging and testing purposes. SPTs were carried out 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 outside diameter (OD) 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). One thin walled tube sample was obtained from Borehole D2. The rock core samples were boxed and returned to Coffey's Etobicoke Office for further visual examination and core photography.

Dynamic Cone Penetration Tests (DCPT) were carried out adjacent to Boreholes C1, C2 and C3. The DCPT consists of driving an uncased 50 mm diameter cone, attached to A-size drill rods, with a driving energy of 475 kJ (63.5 kg hammer free falling for a distance of 0.76 m) per blow, continuously, adjacent to the borehole. The number of blows for each 0.3 m of penetration is recorded, providing an indication of the relative changes in the soil density with respect to depth.

Groundwater levels and inflows observed during drilling were recorded. In Borehole C1, a piezometer was installed to enable long term groundwater level monitoring. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

The borehole locations were located on site using existing site features. The borehole ground elevations were measured by the client's surveyors and were provided to Coffey.

A Coffey representative was present during the drilling operations to direct sampling and testing, record test results and log materials encountered.

Appendix A presents the Record of Borehole Sheets.

3.2 Laboratory Testing

Soil and rock samples obtained during the investigation were taken to our Etobicoke laboratory. The following tests were performed on selected soil samples:

- Natural moisture content tests
- Grain size analyses (sieve)
- Hydrometer tests, and
- Atterberg Limits tests

Appendix B presents laboratory test results sheets.

4 SUBSURFACE CONDITIONS

A total of six boreholes was drilled within the site. Boreholes C1 to C3 were drilled along the culvert alignment at Station 17+625 and Boreholes D1 to D3 were drilled along the alignment of the Highway 66, on the north side and the bottom of the road embankment, from about Station 17+580 to 17+705. The borehole location plan is presented in Drawing 1.

Appendices A, D and E present Record of Borehole Sheets, Rock Core Photographs and Explanation of Terms Used in Report, respectively.

Soil strata along the culvert alignment at Station 17+625 and along the highway alignment are presented in Drawings 2 and 3.

A summary of strata encountered at borehole locations is presented in Table 2. The following sections present in some more detail description of the materials observed from the boreholes.

Table 2: Summary of Subsurface Conditions Observed at Borehole Locations

Unit	Depth/ Elevation to Top of Unit (m)[1]	Approximate Thickness of Unit (m)[1]	Material Description
1. Topsoil	0 / 323.2 to 327.4	0.1 to 0.2	Topsoil (Boreholes C1, C3, D1, D2 and D3)
1A. Buried Topsoil	6.2 / 323.4	0.6	Buried Topsoil (Borehole C2)
2. Embankment Fill	0 / 329.6	6.2	300mm thick sand and gravel layer overlying Gravelly Sand fill, brown to grey, with some silt and clay, compact on the upper 3m, and loose to very loose below 3m depth.
2A. Embankment Toe Fill	0 / 325.8	1.4	Gravelly Sand fill
3. Natural Soils	0.1 to 6.8 / 322.8 to 327.3	1.2 to 5.2	Silt, grey, loose to compact. (all boreholes)
	1.3 to 5.3 / 320.5 to 325.9	0.9 to 2.0	Sandy Silt / Silty Sand, fine to medium grained sand, grey, loose to compact. (all boreholes)
	2.4 to 6.7 / 319.1 to 325	0.3 to 2.0	Sandy Gravel, fine to medium grained, grey, fine to coarse grained sand, very dense. (Boreholes D1, D2 and D3)
4. Bedrock[2]	2.6 to 10.7 / 317.8 to 324.7	Not fully penetrated	Argillite, grey, slightly weathered to fresh.

Notes: 1. The depths and thicknesses are those observed in the boreholes and may not represent the range of values across the site.
2. Proven at Boreholes C1 and C3, inferred in the remaining boreholes.

4.1 Topsoil

Based on the boreholes, topsoil was encountered with thicknesses ranging from 0.1 m to 0.2 m. In Borehole C2, a 0.6 m thick topsoil that contains decayed wood cover was encountered below the embankment fill. This indicates that the topsoil layer that covered the natural soil was not removed prior to the construction of the existing road embankment fill.

Note that in our experience, the thickness of organic rich soils frequently varies in between and beyond borehole locations.

4.2 Embankment Fill

Borehole C2, drilled on the shoulder of the existing road, indicated the presence of a 300 mm thick sand and gravel. Underneath the 300 mm thick sand and gravel layer, granular fill material consisting mainly of gravelly sand with some silt and clay was encountered down to 6.2 m depth below the existing road surface or to Elevation 323.4 m. The presence of occasional silt and sand lenses/seams was noted in the embankment fill.

Borehole C3, drilled on the north side of the existing road, encountered an embankment toe fill layer to about 1.5 m depth. This embankment toe fill is described as gravelly sand with some silt and clay particles.

Grain distribution analyses carried out on embankment fill samples indicate the following distribution, as shown in Figure B1 in Appendix B.

Gravel:	8-15%
Sand:	62-68%
Silt and Clay:	20-30%

The embankment fill is considered to be a granular (non-cohesive) material.

In Borehole C2, Standard Penetration Tests yielded SPT N-values of 10 to 18 blows/0.3 m in the upper 3 m while 2 to 9 blows/0.3 m were recorded below 3 m depth. The SPT N-values showed that the upper 3 m layer is compact indicating that these materials received systematic compaction during their placement while the fill below 3 m is assessed to have a very loose to loose condition indicating that these materials were not compacted when they were first placed. It should however be pointed out that rock fill was noted on the side slopes (see site photographs in Appendix C).

In addition, boreholes advanced by Stantec for the pavement investigation of this project (typically about 4 m right of the existing centreline of the highway) encountered refusal at about 0.8 m to 1.5 m below the existing shoulder level. These observations lead us to believe that rock fill may have been used to build the existing embankment and that the gravelly sand fill materials encountered in Boreholes C2 and C3 may represent localized conditions near the existing culvert (i.e. the existing culvert may have been backfilled with soil rather than rock fill).

4.3 Natural Soils

The natural soils encountered within the boreholes include:

- Silt;
- Sandy Silt/Silty Sand; and
- Sandy Gravel Till

4.3.1 Silt

Underneath the road embankment fill and topsoil, a silt deposit was encountered at Elevations 322.8 m to 327.3 m with thicknesses of about 1.2 m to 5.2 m.

The following are the grain size distribution for the silt, as presented in Figure B2 in Appendix B.

Gravel:	0-1%
Sand:	1-11%
Silt:	82-91%
Clay:	4-17%

The results of Atterberg Limits testing carried out on a sample of silt (Borehole D2 at 3.2 m depth), provided liquid limit of 20.3 %, plastic limit of 17.2 % and a plasticity index of 3.1 %, as shown in Figure B3 in Appendix B, indicating ML classification for the silt. The silt is considered to be slightly plastic.

SPT N-values of 1 to 43 blows/0.3 m were recorded within the silt layer, indicating varying relative densities from very loose to dense. The low blow counts (1 and 4 blows/0.3 m) were recorded on the upper zone just below the topsoil. Typically SPT-N values of 6 to 14 blows/0.3 m were recorded indicating a loose to compact condition. The high SPT N-values of 16 and 43 blows/0.3 m were recorded in Borehole C2, underlying the approximately 6 m high embankment fill. These higher N-values probably reflect the densification of the silt deposit under the weight of the embankment fill at this borehole location.

4.3.2 Sandy Silt/Silty Sand

Underneath the silt deposit is a sandy silt or silty sand deposit which was encountered at Elevations 320.5 m to 325.9 m, with thicknesses ranging from 0.9 m to 2.0 m. The following are the grain size distribution for this layer, as shown in Figure B4 in Appendix B.

Gravel:	0-1%
Sand:	21-82%
Silt:	15-76%
Clay:	2-4%

This deposit is considered to be a granular (non-cohesive) material.

SPT N-values of 5 to 21 blows/0.3 m were recorded within this layer indicating a loose to compact condition. Typical N-values range from 5 to 16 blows/0.3 m. The relatively higher N-value of 21 blows/0.3 m was recorded in Borehole C2 which is believed to reflect the densification of the soil under the stresses imposed by the embankment fill. High SPT N-values (>100 to 134 blows/0.3 m) were recorded in Boreholes C1, C2 and C3 at the bottom of this layer immediately above the bedrock/inferred bedrock. The values are not considered to be representative of the compactness condition of the deposit.

4.3.3 Sandy Gravel Till

A sandy gravel till deposit was encountered in Boreholes D1, D2 and D3 at Elevations 319.1 m to 325.0 m. The thickness of this deposit at the borehole location was recorded to be of 0.3 m to 2.0 m. The deposit was found to extend to auger refusal (probably on the surface of the bedrock) at Elevation 324.7 m to 317.8 m.

The following are the grain size distribution for this deposit, as shown in Figure B5 in Appendix B.

Gravel:	41-51%
Sand:	38-42%
Silt and Clay:	11-17%

This deposit is considered to be a granular (non-cohesive) material.

SPT N-values of 50 to 59 blows/0.3 m were recorded within this deposit indicating a very dense compactness condition. Higher SPT N-values (>132 to >138) were recorded at the bottom of this deposit auger refusal was encountered, probably at or near the surface of the bedrock.

4.4 Bedrock

Bedrock was proven by coring in Boreholes C1 and C3, where the top of bedrock was encountered at Elevations 320.6 m and 320.0 m, respectively. Boreholes C2, D1, D2 and D3 encountered auger refusal on possible bedrock at depths of 2.7 m to 10.7 m below the existing ground surface or at Elevations 317.8 m to 324.7 m.

As mentioned before, the bedrock was cored in Boreholes C1 and C3. The bedrock penetrated 2.8 m to 3.5 m, respectively. Based on the rock cores recovered, the bedrock is described as argillite, grey, slightly weathered to fresh rock. The recorded total core recovery (TCR) ranges from 56 % to 100 % and Rock Quality Designation (RQD) values ranges from 0 % to 92 %. The rock is fractured with staining on the fractured surface.

As shown on the site photographs in Appendix C, rock outcrops and rock cuts were observed on the sides of the road.

4.5 Groundwater Conditions

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. In Boreholes C1 and C3, rock coring was carried out where water was introduced into the borehole. Therefore, groundwater levels measured on completion of the boreholes may not be reliable. The groundwater levels observed during the investigation are summarised in Table 3 below and are also presented on the Record of Borehole Sheets in Appendix A.

Table 3: Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation	Depth/Elevation of the Tip of Piezometer (m)	Measured Groundwater Depth/Elevation (m)	Date Measured	Piezometer Installed
C1	323.2	5.8/317.4	0.4/322.8	22 Aug 09 (two days after completion)	Yes
C2	329.6	-	None observed[1] (dry on completion, hole caved in at 5.5 m)	20 Aug 09	No
C3	325.9	-	1.2[1]/324.7 (on completion, hole caved in at 3.0 m)	21 Aug 09	No
D1	325.8	-	4.5[1]/321.3 (on completion, hole caved in at 5.5 m)	22 Aug 09	No
D2	325.8	-	3.0[1]/322.8 (on completion, hole caved in at 5.5 m)	21 Aug 09	No
D3	327.4	-	1.8[1]/325.6 (on completion, hole caved in at 5.5 m)	22 Aug 09	No

Note: 1 Groundwater level measured not stabilized.

Based on the moisture condition of the soil samples and results of the piezometer readings in Borehole C1, the site groundwater level at the time of our field program was at or near the o.g. (original ground) levels in Boreholes C1, C2 and C3. In Boreholes D1 and D2, the groundwater was at about Elevation 325.0 m, while in Borehole D3 it was at about Elevation 325.6 m.

It should be noted that groundwater levels are subject to variations due to the influence of rainfall, temperature, local drainage, seasons and other factors. There may also be potential for development of perched groundwater tables following periods of rainfall and groundwater may rise to the ground surface. In addition the water level in the watercourse would influence the groundwater level at the site.

For and on behalf of Coffey Geotechnics Inc.

Delfa Sarabia, M.Eng.
Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

Ramon Miranda, P.Eng.
Manager, Transportation

Zuhtu Ozden, P.Eng.
Senior Principal

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Drawings

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.
GWP: 5117-03-00

SHEET

HIGHWAY 66, MCGARRY TWP
CULVERT REPLACEMENT @ 17+625
BOREHOLE LOCATION PLAN

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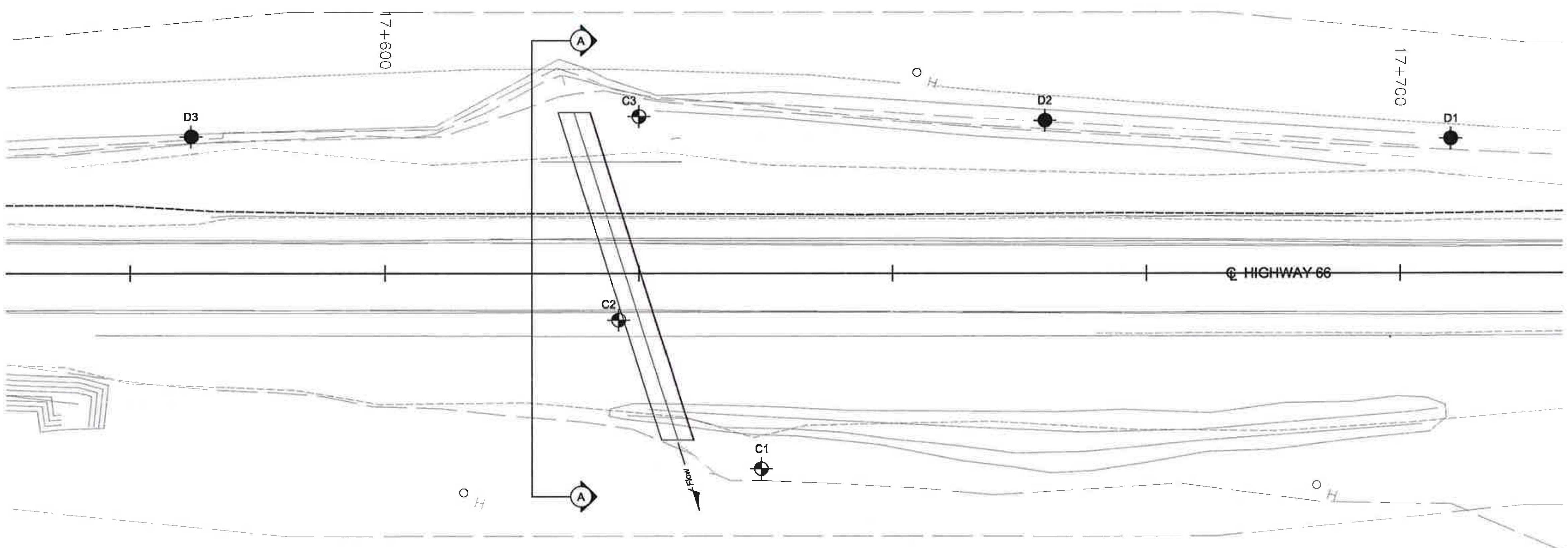
SPECIALISTS MANAGING THE EARTH



PLAN

SCALE

12.5m 0 12.5 25m

KEY PLAN
N.T.S.

LEGEND

- Borehole
- Borehole & Cone

No.	ELEVATION	STATION	OFFSET
C1	323.2	17+637	19.3m Rt C/L
C2	329.6	17+623	4.9m Rt C/L
C3	325.9	17+625	15.5m Lt C/L
D1	325.8	17+705	13.3m Lt C/L
D2	325.8	17+665	15.1m Lt C/L
D3	327.4	17+581	13.5m Lt C/L

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

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

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
Apr 9/10
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
Apr 09/10
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

Geocres No

TRANETOB01241AA			DIST
SUBM'D	CHECKED	DATE Apr. 9, 2010	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.

GWP: 5117-03-00

SHEET

HIGHWAY 66, MCGARRY TWP
CULVERT REPLACEMENT @ 17+625
SOIL STRATA (SECTION A-A)

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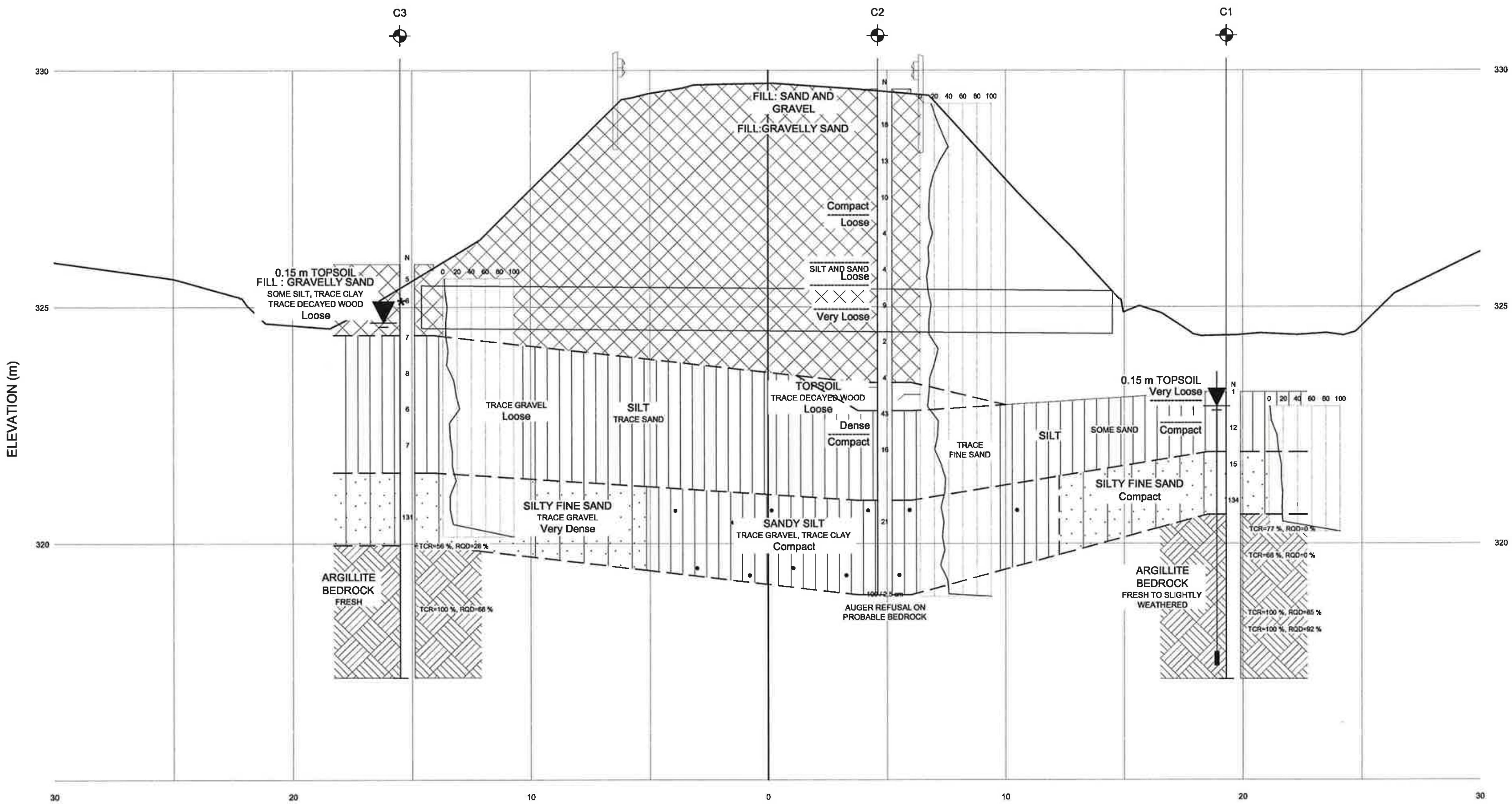
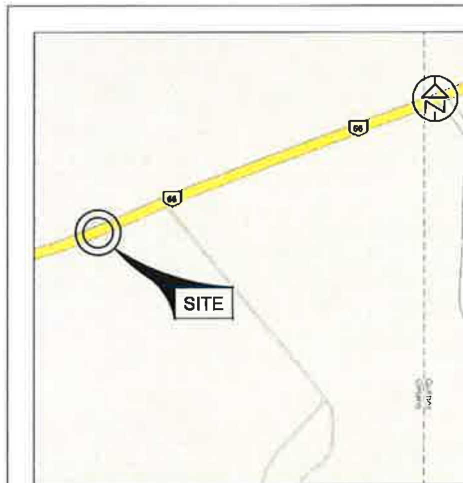


SECTION A-A

HORIZONTAL SCALE

5m 0 5 10m



KEY PLAN
N.T.S.

LEGEND

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

No.	ELEVATION	STATION	OFFSET
C1	323.2	17+637	19.3m Rt C/L
C2	329.6	17+623	4.6m Rt C/L
C3	325.9	17+625	15.5m Lt C/L

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REVISIONS		
DATE	BY	DESCRIPTION

Geocres No

TRANETOB01241AA			DIST
SUBM'D	CHECKED	DATE Apr. 9, 2010	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 2

CONT No.

GWP: 5117-03-00

SHEET

HIGHWAY 66, MCGARRY TWP
CULVERT REPLACEMENT @ 17+625
SOIL STRATA (PROFILE)

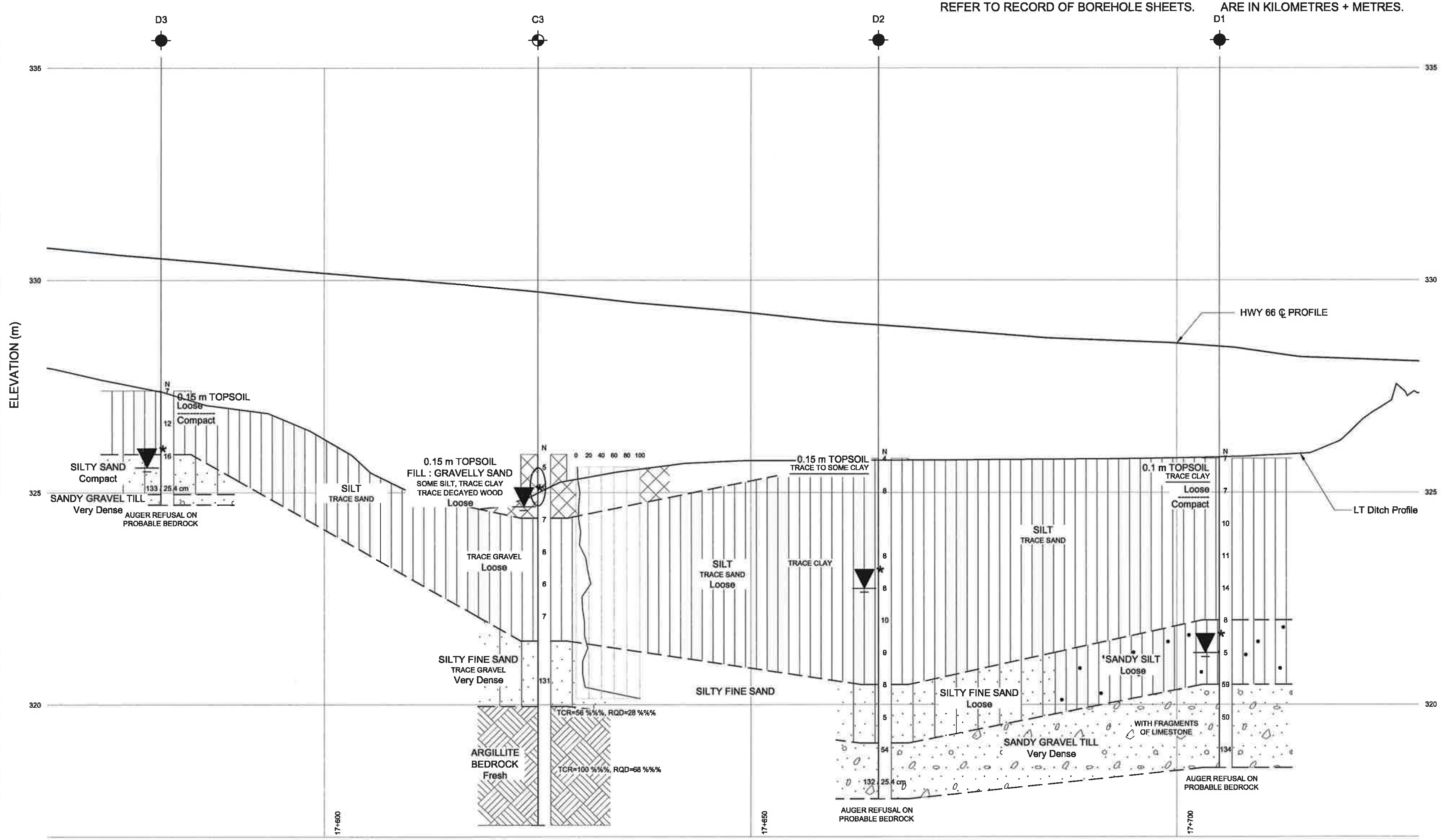
coffey geotechnics
SPECIALISTS MANAGING THE EARTH

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.



PROFILE
HORIZONTAL SCALE

12.5m 0 12.5 25m

KEY PLAN
N.T.S.

LEGEND

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

No.	ELEVATION	STATION	OFFSET
C3	325.9	17+625	15.5m Lt C/L
D1	325.8	17+705	13.3m Lt C/L
D2	325.8	17+665	15.1m Lt C/L
D3	327.4	17+581	13.5m Lt C/L

-NOTE-
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LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
Apr 9/10
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
April 9/10
PROVINCE OF ONTARIO

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No

		TRANETOB01241AA		DIST	
SUBM'D		CHECKED	DATE Apr. 9, 2010	SITE	
DRAWN PHK		CHECKED RM	APPROVED ZO	DWG	3

Appendix A

Record of Borehole Sheets

TRANETOB01241AA: Hwy 66 Culvert Replacement

RECORD OF BOREHOLE No C1

1 OF 1 **METRIC**

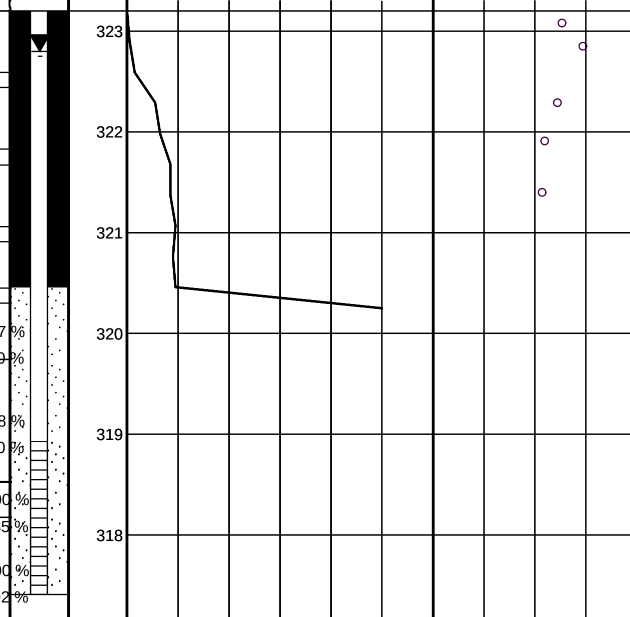
GWP 5117-03-00 LOCATION Station : 17+637, 19.3 m Rt C/L of Hwy 66 ORIGINATED BY ZI

DIST HWY 66 BOREHOLE TYPE Hollow Stem Auger & NQ Coring COMPILED BY RK

DATUM Geodetic DATE 8/20/2009 CHECKED BY RM

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
323.2	GROUND SURFACE					
0.0	0.15 m TOPSOIL; moist, v.loose	1	SS	1	323	
	SILT wet, compact; some sand, trace clay; grey	2	SS	12		0 11 83 6
321.9 / 1.3	**SILTY FINE SAND**; trace clay, grey, wet, compact	3	SS	15	322	0 82 16 2
		4	SS	134	321	cone bouncing
320.6 / 2.6	**BEDROCK: ARGILLITE**; grey, fresh to slightly weathered	5	RC	TCR=77 %, RQD=0 %	320	Borehole advanced by NQ coring
		6	RC	TCR=68 %, RQD=0 %	319	
		7	RC	TCR=100 %, RQD=85 %	318	
		8	RC	TCR=100 %, RQD=92 %		
317.1 / 6.1	End of Borehole; Piezometer installed to 5.8 m; Piezometer Water Level Records : Aug. 20, 2009 0.3 m; Aug. 22, 2009 0.4 m					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m^3



+ ^{3}, × ^{3}: Numbers refer to Sensitivity
20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

TRANETOB01241AA: Hwy 66 Culvert Replacement

RECORD OF BOREHOLE No C2

METRIC

GWP 5117-03-00 LOCATION Station : 17+623, 4.6 m Rt C/L of Hwy 66 ORIGINATED BY ZI

DIST ______ HWY 66 BOREHOLE TYPE Hollow Stem Auger COMPILED BY RK

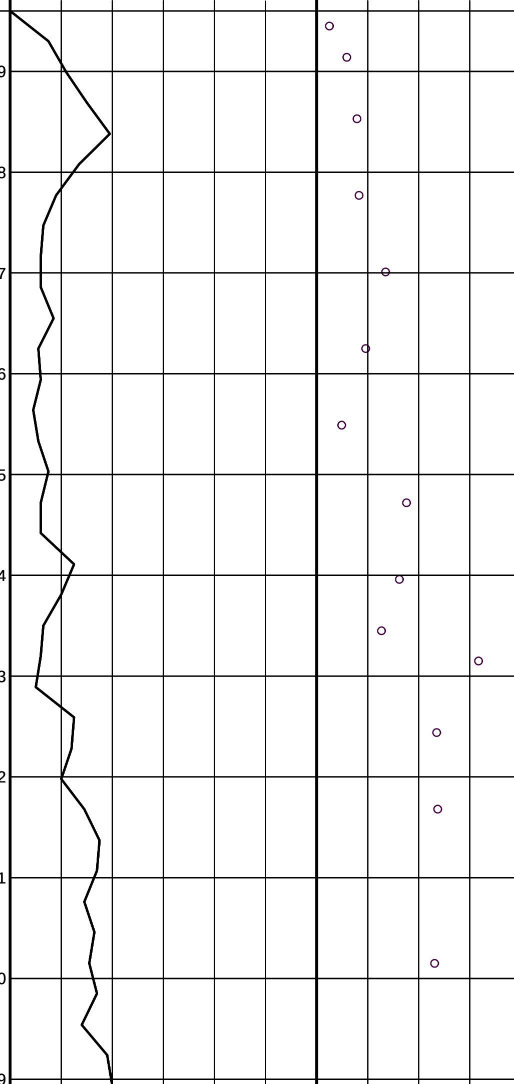
DATUM Geodetic DATE 8/20/2009 CHECKED BY RM

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
329.6	GROUND SURFACE
329.3 / 0.0 / 0.3	**FILL: SAND AND GRAVEL** brown, moist
	FILL: GRAVELLY SAND brown, damp to moist; brown, compact; gravel content decreasing, grey, loose; silt and sand, loose; v. loose
323.4 / 6.2	**TOPSOIL** trace decayed wood, loose
322.8 / 6.8	**SILT** trace fine sand and clay grey, wet; dense; compact
320.9 / 8.7	**SANDY SILT** trace gravel, trace clay grey, wet, compact
318.9 / 10.7	End of Borehole Auger refusal on probable bedrock Borehole dry (not stabilized)* and caved in @ 5.5 m upon completion

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AS		
2	AS		
3	SS	18	15 63 (22)
4	SS	13	
5	SS	10	
6	SS	4	8 62 (30)
7	SS	4	
8	SS	9	
9	SS	2	spoon wet
10	SS	4	
11	SS	43	
12	SS	16	0 3 89 8
13	SS	21	1 25 70 4
			auger grinding @ 10 and 10.6 m
14	SS	100 / 2.5 cm	



UNIT WEIGHT γ kN/m^3

+ 3, × 3: Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

TRANETOB01241AA: Hwy 66 Culvert Replacement

RECORD OF BOREHOLE No C3

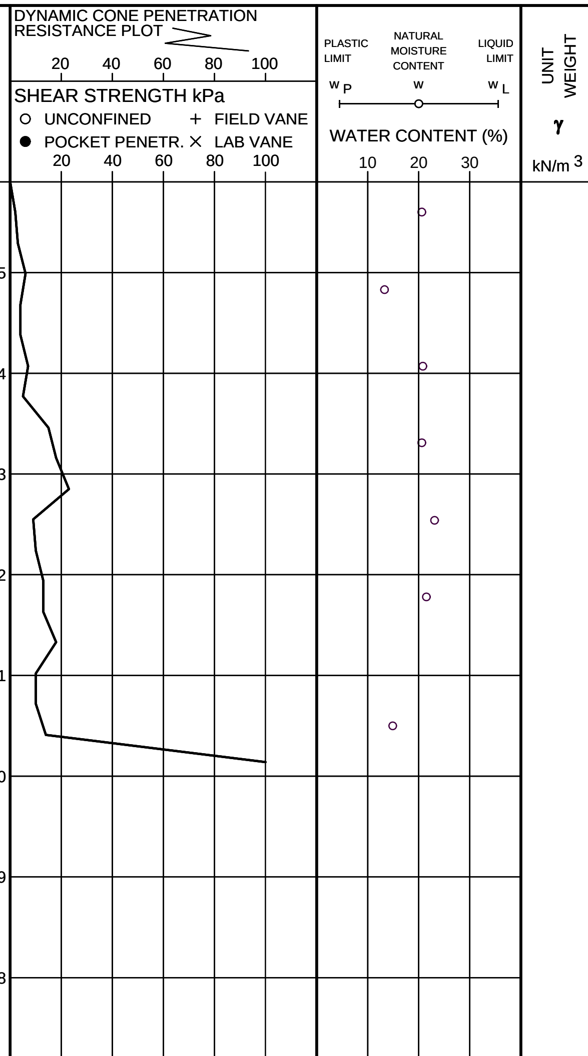
METRIC

GWP 5117-03-00 | LOCATION Station : 17+625, 15.50 m Lt C/L of Hwy 66 | ORIGINATED BY ZI

DIST | HWY 66 | BOREHOLE TYPE Hollow Stem Auger & NQ Coring | COMPILED BY RK

DATUM Geodetic | DATE 8/21/2009 | CHECKED BY RM

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
325.9	GROUND SURFACE						
0.0	0.15 m TOPSOIL **FILL: GRAVELLY SAND** some silt, trace clay trace decayed wood grey, moist. loose	1	SS	5		325	
		2	SS	6	▼*		12 68 (20)
324.4 / 1.5	**SILT** trace sand, gravel, and clay grey, wet, loose	3	SS	7		324	
		4	SS	8		323	1 4 91 4
		5	SS	6		322	
		6	SS	7			
321.5 / 4.4	**SILTY FINE SAND** tr. gravel grey, wet loose to compact ------------- v.dense	7	TW			321	
		8	SS	131			wet spoon
320.0 / 5.9	**BEDROCK: ARGILLITE** grey, fresh	9	SS	TCR=56 % RQD=28 %		320, 319	auger grinding @ 5.6 and 5.9 m auger refusal Borehole advanced with NQ casing
		10	SS	TCR=100 % RQD=68 %		318	
317.2 / 8.7	End of Borehole Water Level @ 1.2 m (not stabilized)* upon completion Hole Caved in @ 3.0 m upon completion						



+ ³, × ³ : Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

TRANETOB01241AA: Hwy 66 Culvert Replacement

RECORD OF BOREHOLE No D1

METRIC

GWP 5117-03-00 | LOCATION Station : 17+705, 13.30 m Lt C/L of Hwy 66 | ORIGINATED BY ZI

DIST | HWY 66 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY RK

DATUM Geodetic | DATE 8/22/2009 | CHECKED BY RM

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
325.8	GROUND SURFACE
0.0	0.1 m TOPSOIL trace clay brown, moist ---------------- loose --------- compact **SILT** trace sand and clay grey, moist to wet
322.0 / 3.8	**SANDY SILT** trace clay, grey, wet, loose
320.5 / 5.3	**SANDY GRAVEL TILL** some silt and clay, with fragments of argillite grey, wet, v.dense
318.5 / 7.3	End of Borehole Auger refusal on probable bedrock Water level @ 4.5 m (not stabilized)* and caved in @ 5.5 m upon completion

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7	
2	SS	7	
3	SS	10	0 2 88 10
4	SS	11	
5	SS	14	
6	SS	8	0 21 76 3
7	SS	5	wet spoon
8	SS	59	auger grinding
9	SS	50	41 42 (17)
10	SS	134	

GROUND WATER CONDITIONS | ELEVATION SCALE 325, 324, 323, 322, 321, 320, 319

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa 20 40 60 80 100
○ UNCONFINED + FIELD VANE
● POCKET PENETR. × LAB VANE

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

TRANETOB01241AA: Hwy 66 Culvert Replacement

RECORD OF BOREHOLE No D2

1 OF 1 **METRIC**

GWP 5117-03-00 LOCATION Station : 17+665 15.1 m Lt C/L of Hwy 66 ORIGINATED BY ZI

DIST HWY 66 BOREHOLE TYPE Hollow Stem Auger COMPILED BY RK

DATUM Geodetic DATE 8/21/2009 CHECKED BY RM

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
325.8	GROUND SURFACE						
0.0	0.15 m TOPSOIL tr. to some clay	1	SS	4			
	brown, moist / grey, wet	2	SS	8		325	0 2 83 15
	SILT trace sand and clay loose	3	TW			324	0 1 82 17
		4	SS	8		323	
		5	SS	8	▼*		0 2 90 8
		6	SS	10		322	
		7	SS	9		321	
320.5 5.3	**SILTY FINE SAND** grey, wet, loose	8	SS	8		320	spoon wet
		9	SS	5			spoon wet
319.1 6.7	**SANDY GRAVEL TILL** trace silt and clay grey, wet, v. dense	10	SS	54		319	51 38 (11) auger grinding at 7.3 to 8.0 m
317.8		11	SS	132 / 25.4 cm		318	
8.0	End of Borehole Auger refusal on probable bedrock Water level @ 3.05 m (not stabilized)* and caved in @ 5.5 m						auger refusal

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No D3

1 OF 1 **METRIC**

GWP 5117-03-00	LOCATION Station : 17+581, 13.5 m Lt C/L of Hwy 66		ORIGINATED BY ZI
DIST	HWY 66	BOREHOLE TYPE Hollow Stem Auger	COMPILED BY RK
DATUM Geodetic	DATE 8/22/2009		CHECKED BY RM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
327.4	GROUND SURFACE								
0.0	0.15 m TOPSOIL brown, loose		1	SS	7		327	~20	
	SILT trace sand, moist, grey, compact		2	SS	12			~21	
325.9 1.5	**SILTY SAND** some silt, grey, moist, compact		3	SS	16	▼	326	~21	0 82 15 3
325.0 2.4	**SANDY GRAVEL TILL** grey, wet, v.dense		4	SS	133 / 25.4 cm		325	~21, ~17	auger grinding at 2.1 to 2.7 m
324.7 2.7	End of Borehole Auger refusal on probable bedrock Water level @ 1.8 m (not stabilized)* and caved in @ 1.9 m upon completion								

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● POCKET PENETR.; × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity

15 ⊕ 5 (20 / 10) (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

The following tables present summaries of the results of natural moisture content, grain size an analyses, hydrometer tests and Atterberg limits tests.

Table B1: Summary of Natural Moisture Content Tests

Borehole Number	Sample Depth (Soil Description)	Natural Moisture Content (%)
C1	0.12m to 1.8m (Natural Soil)	21.4 to 29.4
C2	0.15m to 6.45m (Fill Embankment)	2.5 to 31.7
C2	7.16m to 9.45m (Natural Soil)	23.1 to 23.7
C3	0.3m to 1.07m (Fill Embankment)	13.3 to 20.6
D1	0.3m to 7.1m (Natural Soil)	6.5 to 25.3
D2	0.3m to 7.8m (Natural Soil)	9.1 to 29.2
D3	0.3m to 2.6m (Natural Soil)	17 to 22.2

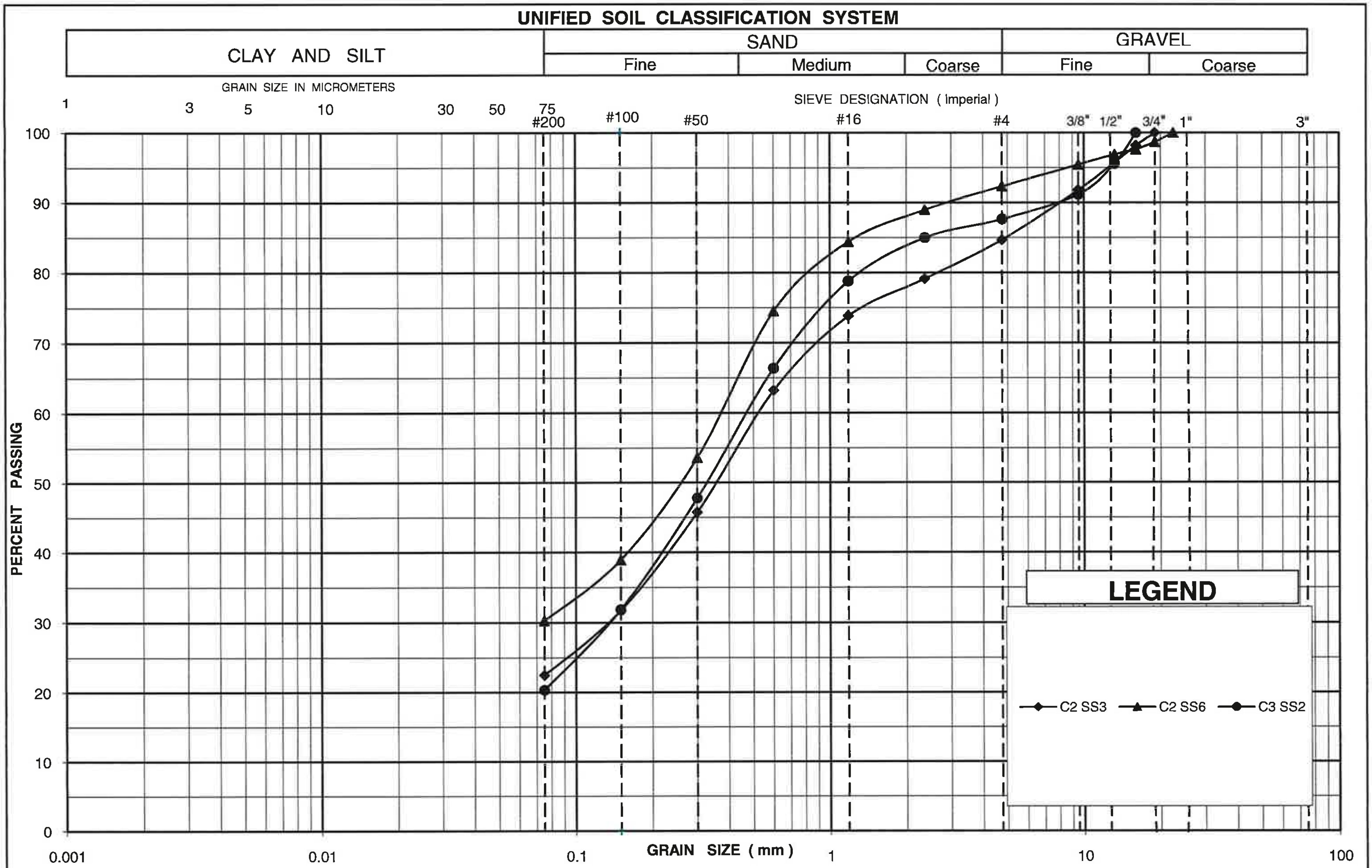
Table B2: Summary of Grain Size Distribution and Hydrometer Tests

Borehole Number	Sample ID	Percentage Retained (%)			
		Gravel	Sand	Silt	Clay
C1	SS2	0	11	83	6
C1	SS3	0	82	16	2
C2	SS3	15	63	22	
C2	SS6	8	62	30	
C2	SS12	0	3	89	8
C2	SS13	1	25	70	4
C3	SS2	12	68	20	
C3	SS4	1	4	91	4
D1	SS3	0	2	88	10
D1	SS6	0	21	76	3
D1	SS9	41	42	17	

Borehole Number	Sample ID	Percentage Retained (%)			
		Gravel	Sand	Silt	Clay
D2	SS2	0	2	83	15
D2	TW3	0	1	82	17
D2	SS5	0	2	90	8
D2	SS10	51	38	11	
D3	SS3	0	82	15	3

Table B3: Summary of Atterberg Limits Tests

Borehole Number	Sample ID	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
D2	TW3	20.3	17.2	3.1



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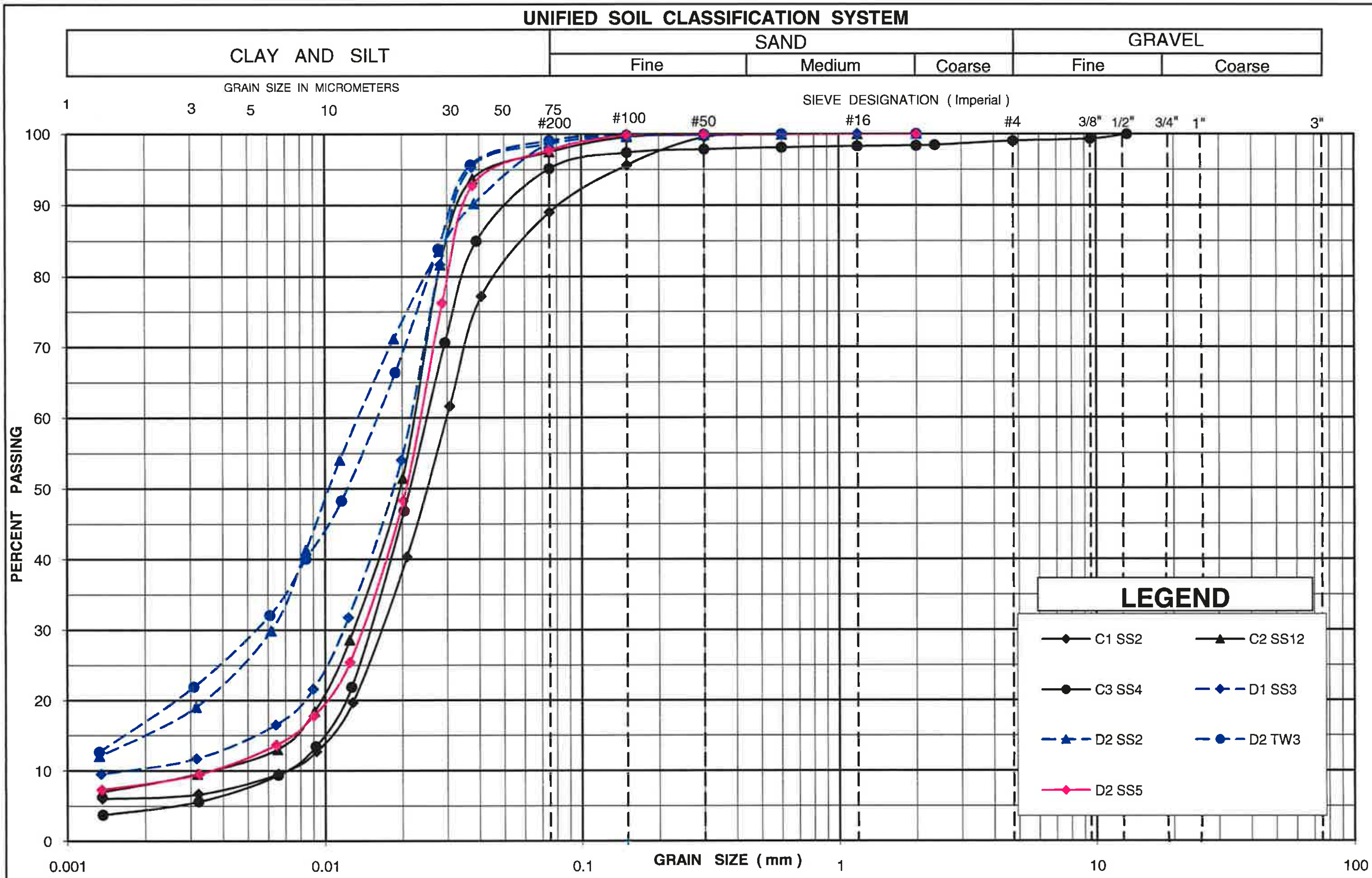
GRAIN SIZE DISTRIBUTION

EMBANKMENT FILL - Gravelly Sand with some silt and clay

FIGURE NO.:	B1
PROJECT NO:	TRANETOB01241AA
DATE:	NOV. 2009



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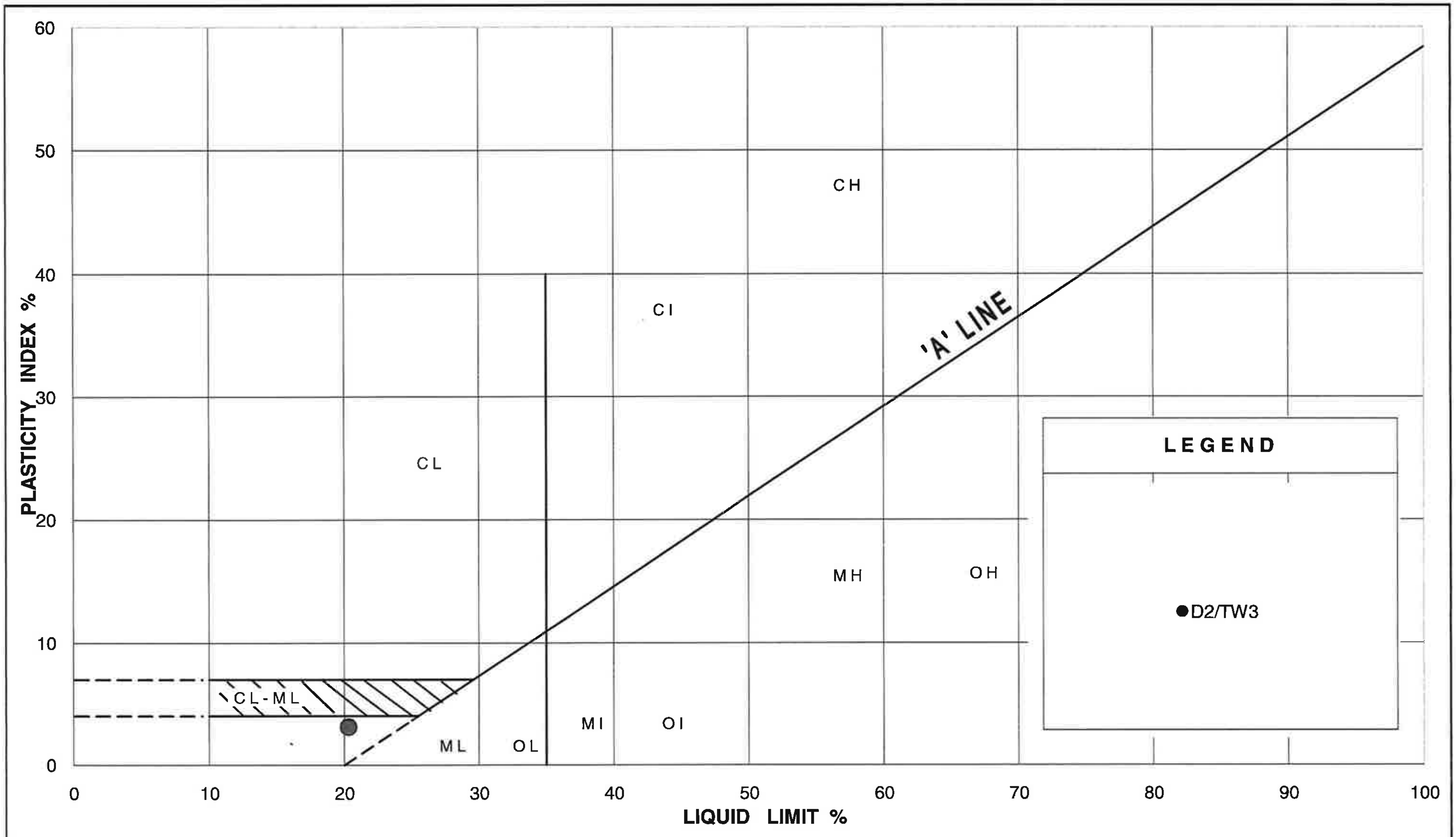
GRAIN SIZE DISTRIBUTION
SILT

FIGURE NO.: B2
PROJECT NO: TRANETOB01241AA
DATE: NOV. 2009



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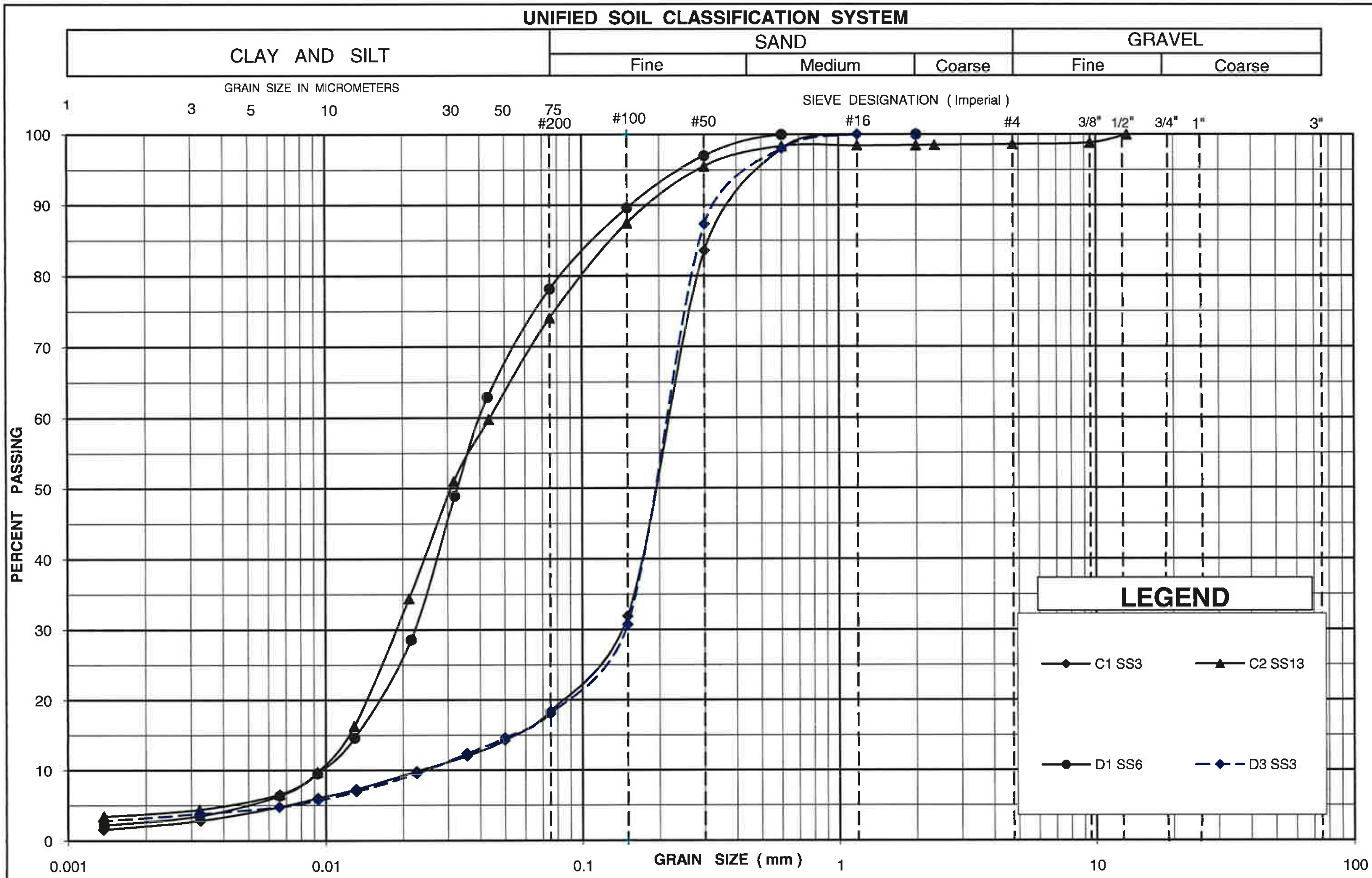
PLASTICITY CHART

SILT

FIGURE NO.: B3

PROJECT NO.: TRANETOB01241AA

DATE NOV, 2009



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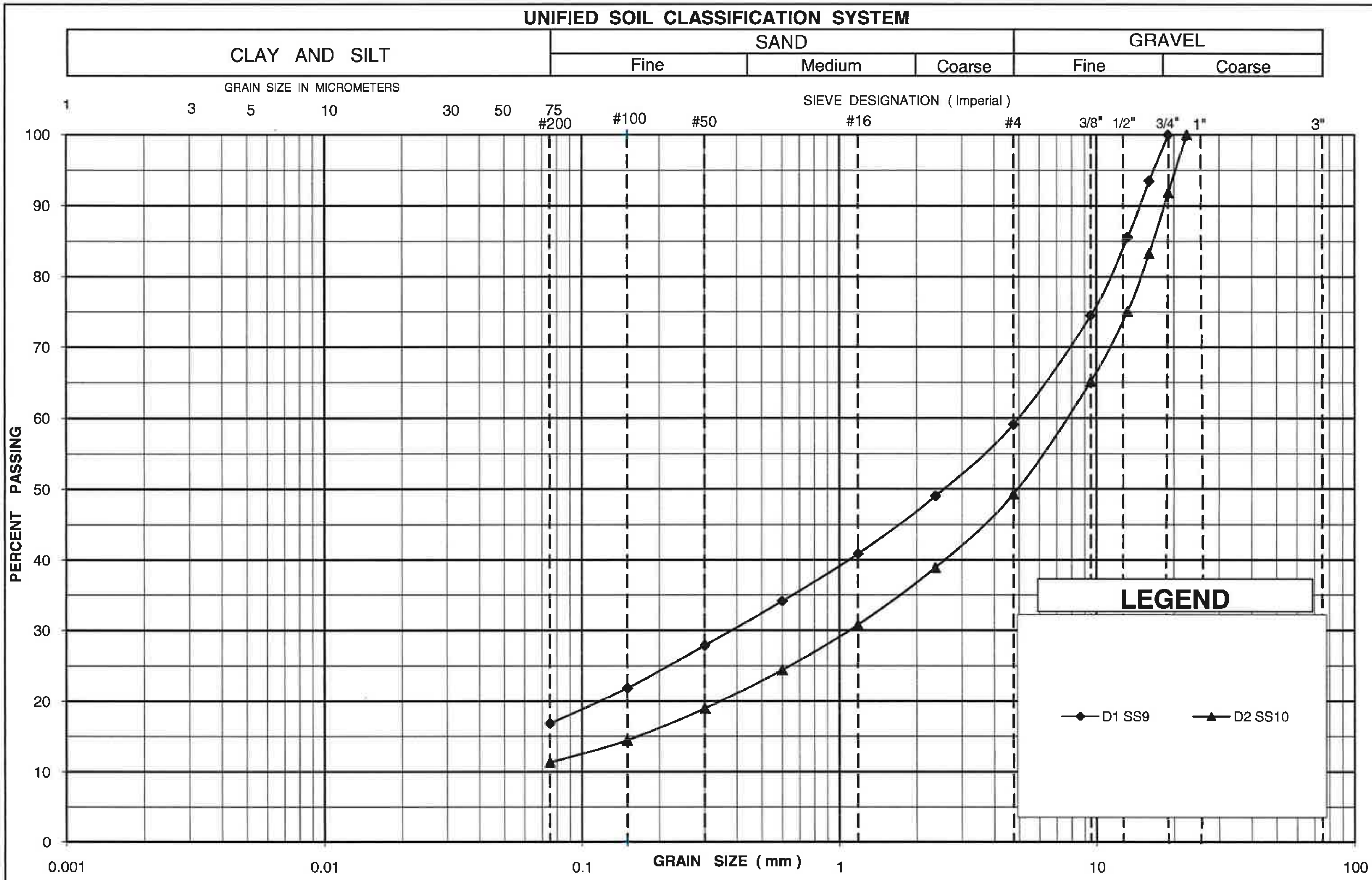
GRAIN SIZE DISTRIBUTION
SILTY SAND / SANDY SILT

FIGURE NO.:	B4
PROJECT NO:	TRANETOB01241AA
DATE:	NOV. 2009



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GRAIN SIZE DISTRIBUTION SANDY GRAVEL TILL	FIGURE NO.: B5 PROJECT NO:TRANETOB01241AA DATE: NOV. 2009

Appendix C

Site Photographs

Photograph 1. Existing Highway 66 (looking west)

Photograph 2. Existing Highway 66 (looking east)

Photograph 3. Northern slope of the road embankment (looking east)

Photograph 4. Southern slope of the road embankment (looking west)

Photograph 5. Rock cutting on the southern side of the highway

Appendix D

Rock Core Photographs

BOREHOLE C1

BOREHOLE C3

Appendix E

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

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FOUNDATION DESIGN REPORT
CULVERT REPLACEMENT AT STATION 17+625 ON HIGHWAY 66, TOWNSHIP OF MCGARRY, ONTARIO
G.W.P. NO. 5117-03-00
GEOCRES NO. 32D-10

Stantec Consulting Ltd

TRANETOB01241AA-AA
April 9, 2010

Coffey Geotechnics Inc.
20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

CONTENTS

FOUNDATION DESIGN REPORT

CULVERT REPLACEMENT AT STATION 17+625

HIGHWAY 66, TOWNSHIP OF MCGARRY, ONTARIO

G.W.P. 5117-03-00

5 DISCUSSIONS AND RECOMMENDATIONS

At Station 17+625, water flows through an existing culvert under Highway 66, about 1.2 km west of the Quebec-Ontario boundary, within the Township of McGarry, Ontario. We understand that the existing 914-mm diameter, 39 m long CSP culvert will be replaced with a similar diameter pipe with an upstream invert elevation of 324.6 m and a downstream invert elevation of 324.4 m. The proposed elevations are similar to the existing culvert elevations. At this stage, the type of culvert replacement is yet not finalized and consideration is being given to both flexible and rigid pipe culverts.

A total of six boreholes were drilled within the site. Boreholes C1 to C3 were drilled along the culvert alignment at Station 17+625 and Boreholes D1 to D3 were drilled along the alignment of Highway 66, on the north side and at the bottom of the road embankment, from about Station 17+580 to 17+705. Based on the borehole information, the site is underlain by embankment fill over basically granular (i.e. non-cohesive) natural soils which in turn are underlain by bedrock. At the culvert location, Borehole C2 showed that the embankment fill is comprised 300 mm thick sand and gravel fill over gravelly sand fill up to 5.9 m thick (i.e. total embankment fill thickness of 6.2 m). The embankment fill was described as compact within the upper 3 m zone and very loose to loose within the lower 3 m zone. As discussed in Section 4.2 of this report, rock fill may also have been used to construct the embankment. Topsoil about 0.1 m to 0.2 m thick was encountered in Boreholes C1, C3, D1, D2 and D3. A 0.6 m thick topsoil was contacted in Borehole C2 underlying the embankment fill. Below the embankment fill and/or topsoil, natural soil was encountered that consisted of silt, sandy silt/silty sand and sandy gravel till deposits. Silt about 1.1 m to 5.2 m thick was encountered in all six boreholes at Elevations 322.8 m to 327.3 m. The silt was found to be in a loose to compact condition. Below the silt, a sandy silt/silty sand deposit, about 0.9 m to 2.0 m thick, was encountered at Elevations 320.5 m to 325.9 m. This deposit was also encountered in all six boreholes in typically a loose to compact condition. Boreholes D1, D2 and D3 encountered a sandy gravel till deposit at Elevations 319.1 m to 325.0 m with thicknesses ranging from 0.3 m to 2.0 m. This deposit was not encountered in Boreholes C1, C2 and C3. The glacial till extended to auger refusal depths and was found to be in a very dense condition, based on the SPT results. Refusal to further augering (probably representing the surface of the bedrock or close to it) was encountered at Elevations 317.8 m to 324.7 m (or to depths of about 2.6 m / Borehole C1 to 8.0 m / Borehole D2). In Boreholes C1 and C3, the presence of the bedrock was proven by coring. From the cores, the bedrock is described as argillite, slightly weathered to fresh rock. Rock cuts and rock outcrops were observed on the sides of the road.

At the time of our field program, at Boreholes C1, C2 and C3 (i.e. at the culvert location), the groundwater was found at close to the o.g. levels while in Boreholes D1 and D2, the groundwater was at about Elevation 325.0 m. In Borehole D3 it was found at Elevation 325.6 m. Note that groundwater levels are subject to variations due to the influence of rainfall, temperature, local drainage, seasons and other factors. There may also be potential for development of perched groundwater tables following periods of rainfall and groundwater may rise to the ground surface. In addition the water level in the watercourse would influence the groundwater level at the site.

5.1 Culvert Foundations

We understand that the locations, invert elevations and the length of the new culvert will be approximately the same as the existing. Therefore the invert of the proposed 900 mm diameter culvert can be expected to be at approximately Elevation 324.6 m upstream and 324.4 m downstream. Assuming an approximate 0.5 m thick granular bedding beneath the culvert, the founding level can be expected to be at Elevation 324.1 m to 323.9 m. Based on the borehole data at these elevations, the founding natural soils can be expected to consist of very loose to compact silts but topsoil may also be encountered under the embankment fill (see Record of Borehole C2 in Appendix A). The groundwater table at the time of our investigation was inferred at the o.g. (original ground) level. The silt underlying the site is a dilatant material, especially in the presence of water. It is also prone to disturbance during the construction. If the pipe is placed on the disturbed, dilated soils, excessive settlements could ensue after backfilling. The construction will therefore have to be carried out carefully with proper dewatering to avoid excessive settlements.

We also understand that there will be no grade raise from the present highway grade, thus, if the founding soils are not unduly disturbed during construction, theoretically there should be little or no settlements since there are no anticipated additional stresses imposed by the new culvert construction. In fact, if the new culvert is placed adjacent to the existing one, since the culvert itself will weigh less than the soil removed, there will be a stress decrease, but this will be somewhat compensated by the soil exchange beneath the culvert (i.e. the loose silt or organic topsoil below the culvert invert will be removed and replaced with heavier granular soils) for the construction of the bedding.

With the prevailing conditions, the use of a flexible pipe such as a CSP (corrugated steel pipe) or a sufficiently strong flexible plastic pipe (HDPE) culvert would be best suited for the project. If a CSP or HDPE type culvert is unacceptable, then consideration can be given to a precast concrete. It is our opinion however the latter should be installed in short sections (e.g. 2.4 m) for flexibility purposes.

An open bottom concrete culvert, which requires the use of spread footings and therefore deeper excavations below the groundwater table, is not a suitable option for this site, with the prevailing unfavourable subsurface conditions, including the highly scour prone characteristics of the underlying silt. Consideration can be given to the use of deep foundations in conjunction with an open bottom culvert. But this will be prohibitively expensive as well as highly time consuming (i.e. the construction under a major highway will take a very long time).

A closed bottom concrete culvert suffers from similar disadvantages as the open bottom concrete culvert, but to a lesser degree. While it is not recommended, consideration may also be given to a precast concrete box culvert with relatively short section (i.e. 2.4 m long individual sections).

5.1.1 Corrugated Steel Pipe (CSP) Type Culvert

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

We recommend that a 0.5 m thick granular bedding be provided beneath the CSP pipe in view of the typically very loose to loose condition of the underlying dilatant silt, especially near the ends, along with the high water table noted during the investigation.

To carry out this operation (i.e. excavating to the required elevation to 0.5 m below the pipe invert) without excessively disturbing the dilatant silt, it must be ensured that the site is dewatered at least 0.4m below the excavation level. This means that dewatering needs to be carried out so that the groundwater level is lowered to at least 0.4 m below the anticipated excavation level (e.g. for a pipe invert elevation of 324.6 m the excavation would be carried to Elevation 324.1 m less 0.4 m, bringing the effective dewatering level to Elevation 323.7 m). Depending on the site conditions at the time of construction, it may be possible to achieve this by means of perimeter ditches, gravity drainage and rigorous pumping from properly filtered and strategically placed sumps. If this proves to be ineffective then a well pointing system may also be required. As will also be discussed later on in this report, since well points can only be installed near the ends of the embankment (i.e. installation of well points from the top of the existing embankment is impractical and ineffective), the use of deep wells may also be required.

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

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

Assuming a 900 mm diameter pipe, the excavation and soil replacement will need to extend at least 0.9 m beyond the perimeter of the foot print of the pipe.

After the excavation to 0.5 m below the invert of the pipe and its replacement with Granular ‘B’ Type II material to 0.3 m (i.e. to 0.2 m below the invert) is completed, the upper 0.2 m of the bedding can be placed. This should consist of Granular ‘A’ Type material. It is recommended that if feasible the Granular ‘A’ material be compacted from the surface to not less than 95% of its Standard Proctor Maximum Dry Density (SPMDD), immediately before placing the pipe, using a suitably small compactor in order not to disturb the underlying dilatant silt soil. For the same reason, depending on the site conditions, vibration should be applied very sparingly or not at all.

The dewatering of the site should be continued (i.e. dewatering to at least 0.4 m below the bottom of the granular soil and the surface of the underlying silt) until the pipe is placed and sufficiently backfilled. To avoid unbalanced loading on the culvert, the height of the backfill around the culvert should be maintained equal on both sides throughout the construction, as much as practically possible. The backfill should be placed in suitably thin layers and each layer should be compacted to MTO standards.

The entire construction, including excavation, backfilling of the bedding underneath the culvert, as well as the placement and compaction of the backfill around the culvert structure, should be carried out under geotechnical supervision.

Based on the above, the following geotechnical resistances can be assigned for design purposes.

- Factored Bearing Resistance at U.L.S. = 200 kPa under the footprint of the full height portion of the embankment, gradually reducing from the shoulder rounding to 80 kPa at the toe of the embankment.
- Bearing Resistance at S.L.S. = 60 kPa

Under the embankment portion, the bearing resistance at S.L.S. is less than the embankment stresses but as mentioned before, imposed loads due to the culvert will be less than the existing. Therefore, in theory, there should be no problem associated with settlements, provided that the bearing soil is undisturbed during the construction. However, an allowance of 30 mm of possible total settlement should be made due to possible rebound during construction owing to stress relief. With this amount of settlement, cambering is not believed to be necessary. We would like to point out that the quoted settlement is based on undisturbed founding subgrade. If the subgrade is unduly disturbed during construction, greater settlements could occur. To prevent this, adequate dewatering needs to be applied, as discussed before.

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

These recommendations would be applicable to precast concrete pipes in short sections (i.e. 1.8 m to 2.4 m individual lengths), except that the depth of excavation (i.e. soil replacement) should be increased from 0.5 m to 0.6 m.

In this instance, for a precast concrete pipe culvert, the maximum anticipated magnitude of settlement would be 35 mm (since concrete weights more than a CSP). This may translate into differential settlements between individual pipe lengths. While in our experience such settlement should not present a problem. This aspect should be discussed with the supplier of the pipe to ensure that this will not cause a snapping or leaking of the gaskets in between individual precast sections.

5.1.2 Precast Concrete Box Culvert

As mentioned before, with the prevailing subsurface conditions the use of a CSP type culvert is the preferred option at the site. However, the use of a relatively flexible precast concrete box culvert with relatively short individual sections (panels) of about 1.8 m to 2.4 m length is also feasible, as discussed below.

The procedure is basically similar to the construction method discussed in the previous section for the CSP type culvert. In this case, however, the minimum thickness of the bedding beneath the concrete pipe culvert would be increased to 0.7 m from 0.5 m and the excavation and soil exchange would extend at least 1.0 m beyond the footprint of the culvert.

The site would be dewatered to ensure the groundwater level is depressed to not less than 0.4 m below the bottom of excavation (i.e. to at least Elevation 323.5 m at the inlet and 323.5 m at the outlet), as discussed before.

The excavation would be carried out in narrow widths and immediately backfilled with Granular ‘B’ Type II material to within 0.2 m below the proposed bottom elevation of the box culvert. In this event, since the unsupported height of the uncompacted Granular ‘B’ Type II material would be even higher (i.e. 0.5 m), good workmanship and careful construction techniques would be required to prevent a cave-in condition.

Following the completion of soil replacement to 0.2 m below the bottom of the concrete culvert invert elevations (i.e. to the top of the Granular ‘B’ Type II), some compaction of the granular material can be effected (e.g. operating track-mounted construction equipment), being cognizant of not disturbing the silt subgrade. The grade can then be raised to the underside of the concrete precast panels by placing a 0.2 m thick Granular ‘A’ material. This Granular ‘A’ layer would also be lightly compacted by the operation of track-mounted equipment followed by, if site conditions permit, operating a light compactor with little or no vibration. It is recommended that if feasible, the Granular ‘A’ be compacted to not less than 95% SPMDD.

Assuming that the above recommendations are followed, similar to CSP culverts the following geotechnical resistances can be assigned:

- Factored Bearing Resistance at U.L.S. = 200 kPa under the footprint of the full height portion of the embankment, gradually reducing from the shoulder rounding to 80 kPa at the toe of the embankment.
- Bearing Resistance at S.L.S. = 60 kPa

While the use of a precast concrete box culvert is not well suited for this site, it should be confirmed with the supplier that a differential settlement of up to 50 mm between individual precast segments will not be detrimental to the integrity of the culvert. In our experience this should be acceptable. The transportation and placement of the precast concrete box culvert segments will need to proceed with caution such that the weight of the adjacent embankment and that of the construction equipment including the loaded crane will not cause disturbance and/or failure of the newly constructed bedding and/or the underlying weak soil. The concrete box culvert segments and the crane should not be brought to the site, until the site is fully backfilled by Granular ‘B’ Type II and Granular ‘A’ bedding material. The suitability of the existing embankment to carry the loaded crane will need to be determined. This is the Contractor's responsibility. This applies to a certain extent to concrete pipe culverts, as well.

As it requires the most aggressive dewatering, precast box culvert is the least suitable of the three types of culverts discussed, for the prevailing site subsurface conditions.

5.2 Embankment for Possible Detour

The construction of a detour embankment is not expected with the proposed method of construction (as shown in Appendix F) but the following are some brief comments for the construction of detour road (if necessary) based on Boreholes D1, D2, D3.

If a detour is to be constructed it will probably be lower than the existing Highway 66 embankment as the existing embankment is about 5 m high. The detour embankment can be expected to be 2 to 3 m high.

Based on Boreholes D1, D2 and D3, the detour is underlain by a 100 to 150 mm thick topsoil layer over silt, silty sand/sandy silt and till. The silt was described as loose near the surface and compact with depth. The silty sand/sandy silt deposit is in a loose to compact condition and the till deposit was found to be in a very dense condition. Based on the borehole data, no foundation failures are anticipated for up to about 3 m high embankments. Normal 2H:1V side slopes can be used, provided that the embankment is built to MTO standards.

All organic and 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 existing embankments, as per MTO procedures and in accordance with OPSD 208.010. Where the existing embankment slope is comprised of possible rock fill (as discussed in Section 4.2), we recommend that unsuitable materials on the surface of the rock fill be removed prior to abutting the detour embankment. We recommend that if possible the materials used for the new embankment should match that of the existing embankment.

Normally, the materials used for the construction of the embankment fills consist of approved, acceptable earth fill. The embankment earth fill should be placed on the previously stripped, approved and properly rolled (where feasible) 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 SPMDD. Embankment construction should be carried out in conformance with SP206S03. Where the existing embankment consists of rock fill and the detour embankment is to consist of earth fill, a properly graded transition zone should be placed in between the new and the existing embankment to prevent infiltration of the soil into the cavities in the rock fill. For the granular profile, a minimum of 0.4 m thick Granular 'A' compacted to 100% SPMDD should be provided, for the proposed temporary detour road.

Embankment loadings would likely result in a settlement of the order of 30 mm (for an embankment height of 2.5 m) 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 six months. In addition, the settlement of the new embankment fills under their own weight can be expected to occur. If the embankment is constructed to MTO standards, this should not exceed 15 mm (for up to 2.5 m high embankment). The time rate 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 three months. As these settlements are not excessive, neither surcharging nor preloading is considered necessary for a detour embankment.

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).

5.3 Embankment Reconstruction

Proper benching of the existing embankment slope should be implemented when abutting into the existing embankment, as per MTO procedures and in accordance with OPSD 208.010.

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). In as much as possible, the fill used should match the existing embankment fill, within the frost zone. 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 SPMDD.

For the granular profile, a minimum of 0.4 m thick compacted Granular 'A' should be provided. For permanent fills immediately below any roadway, it is recommended that Granular 'A' or 'B' aggregates be used and compacted to 100% SPMDD. Where necessary, proper tapering as per standards should be provided. Fill within 1.5 m of the roadway subgrade should be compacted to at least 98% of the SPMDD. The Granular 'A' base and Granular 'B' sub-base courses should be compacted to 100% of the material's SPMDD.

We anticipate that no further settlement of the natural foundation soils as only embankment replacement will be carried out (i.e. no raising or widening of the existing embankment). However, the settlement of the replaced embankment fills under their own weight can be expected to occur. If the embankment is constructed to MTO standards, this should not exceed 35 mm (for 6 m high embankment). The time rate will depend on the material used for construction. However, if select subgrade materials (SSM) or granular soils are used, the settlement should be completed within three months.

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

5.4 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 and 802.014 for flexible pipes, OPSD 802.031, 802.032 and 802.034 for rigid pipes. The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular 'A' or 'B' (OPSS-1010). All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted to at least 95% of the material's SPMDD. Uplift of the pipe can be minimized by means of dewatering and/or placing sufficient fill above it.

We would like to point out that the performance of pipe culverts and especially the flexible pipes (e.g. CSP) is largely dependent on the side support provided by the backfill and the adjacent soils. The use of proper backfill material and especially good compaction are, therefore, necessary for proper side support and successful performance of the pipe. 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 culvert, as per MTO practice. During backfill placement, the height of the backfill should be maintained at approximately the same level on both sides of the structure, to avoid lateral displacement and/or damage of the structure.

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

We understand that the wing walls and other type retaining walls are not required for this project. In any event, the subsurface conditions are not favourable for retaining walls supported on normal spread footing foundations. The following are however presented for the sake of completeness.

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, (CHBDC) 2006. For design purposes, the following properties can be assumed for backfill.

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

Angle of Internal Friction ϕ=35° (unfactored)

Unit weight = 22 kN/m^3

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 ϕ=30° (unfactored)

Unit Weight = 21 kN/m^3

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. This case occurs when some movement (yield) of the retaining structure occurs but not in a sufficient magnitude to fully mobilize an active condition (as such an intermediate condition between K_o and K_a occurs).

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 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 as per SP 599S22 and SP 599S23, in which case the designer will have to include the geometric, performance and appearance requirements (i.e: medium performance and medium appearance). If such walls are to be utilized, a thorough analysis should be implemented.

5.5 Construction

The proposed culvert replacement is to be located at the same location and elevation as the existing culvert.

Borehole C2 and, to a certain extent C3, indicate the presence of granular earth fill in the immediate vicinity of the culvert. The presence of rock fill was however observed on the side slopes of the highway embankment. In addition, boreholes advanced by Stantec for pavement investigation from the top of the embankment encountered refusal at shallow depths. These lead us to surmise that rock fill may also be encountered during the construction and we believe that it may be prudent to point this probability to the Contractor.

Based on the drawings provided by the client, the culvert replacement will be carried out using staged construction. In the staging process, half of the highway embankment is to be excavated at a time while removing and replacing the culvert. Lowering of the highway to provide for a wider platform will also be carried out during the staging. Traffic will be reduced to a single lane and continuous, 24 hour flagging will be implemented. Upon removal and replacement of the culvert, the highway will then be reinstated. The finished embankment is to be similar to the height and width of the existing embankment. Appendix F presents the proposed staged construction for the culvert, as provided by the client.

Temporary inner slopes of 1H:1V are proposed during the staged construction. 1H:1V slopes especially within the very loose to loose zones of the existing embankment combined with traffic loads at the crest of the embankment may cause instability of the embankment. We recommend that the temporary side slopes be no steeper than 1.5H:1V for cuts within the existing embankment fill. Where existing fill is rock fill, a steeper slope of between 1H:1V and 1¼ H:1V can be adopted. It may be prudent to further determine the presence of rock fill within the embankment fill and get a better handle on the details of staging (i.e. temporary side slope configuration). A minimum 1.5 m clearance should be maintained between the moving traffic and the edge of the slopes. We also recommend that these slopes be visually monitored for any movement especially if workers are present at the toe of the slopes. Temporary slopes of 1.5H:1V within the constructed granular fill is recommended. These temporary slopes should be utilized for a short duration. The proposed outer side slopes of 2H:1V are considered stable on a short and long term basis.

It is anticipated that the construction comprise the excavations of the existing embankments and any unsuitable materials below the proposed culvert invert levels. Excavations should be possible using heavy equipment such as a hydraulic excavator. As mentioned before, there may also be rock fill in the make-up of the embankment and their removal may create difficulties.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the soils can be classified as follows:

Topsoil	Type 4 soil	
Embankment Fill	Type 3 soil above water level;	Type 4 soil below water level
Silt	Type 3 soil above water level;	Type 4 soil below water level
Sandy Silt and Sandy Silt	Type 3 soil above water level;	Type 4 soil below water level
Sandy Gravel Till	Type 2 soil above water level;	Type 4 soil below water level

Stockpiles should be placed well away from the edge of the excavation and their height should be controlled so they do not surcharge the sides of the excavation. Surface drainage should be controlled to prevent flow of surface water into the excavations. Excavation safety and stability of temporary construction slopes and lateral support systems are the Contractor's responsibility.

The excavated soils, free from topsoil and organics, can be used as general construction backfill where they can be compacted with smooth drum type rollers. Loose lifts of soil, which are to be compacted, should not exceed 300 mm. On site verification of the excavated fill for re-use as backfill by suitably qualified personnel during construction would be required. During wet periods, the finer grained soil will likely be unsuitable for reuse. Selective stockpiling and double handling may be required for reuse of these materials.

The excavated soils are not considered to be free draining. Where free draining backfill is required, imported granular fill such as OPSS Granular B Type I should be used.

Note that the excavated soils are subject to moisture content increase during wet weather which would make these materials too wet for adequate compaction. Stockpiles should therefore be compacted at the surface or be covered with tarpaulins to help minimize moisture uptake.

Foundation bearing soils (silts) near the water table and in wet weather are susceptible to disturbance from construction activity. Care should be taken during excavation and construction to minimize disturbance of the bearing soil. Stabilization of wet silt subgrade should be anticipated. Disturbance of the underlying soils during construction of the structures, in proximity to the groundwater table, could influence the future settlements of the proposed structures. Dewatering will therefore be required to stabilize the dilatant soil and to facilitate the construction. In addition, flow of water in the existing watercourse will need to be maintained during the construction.

To maintain the flow of water in the existing water course, consideration can be given to divert the water and contain the collected water in a pool then pump it across the highway downstream. Alternatively, the flow can be provided in a temporary pipe in the excavated area but in this case it must be ensured that the extra pipe will not interfere with the construction activities.

Depending on the site conditions at the time of construction, it may be possible to effect the dewatering at the site by means of pumping from strategically placed filtered sumps/wells near the toe of the existing embankment prior to start of the excavation. In this case, the sumps/wells may need to extend into the silty fine sand/sandy silt deposit underlying the dilatant silt. This will be effective in reversing the flow of water in the silt deposit downwards thus stabilizing it. This scheme will however not be fully effective in the middle portion of the embankment where the excavation will take place. Thus, additional dewatering will be required within the excavation area, including pumping from strategically placed filtered deep sumps. If these measures do not appear to be effective, additional measures such as vacuum well points may be required. Since well points can only lift the water by only about 5.5 m, they will not be effective if they are installed from the top of the embankment. In addition this may not be practical. In the design of the dewatering system, the presence of bedrock underlying the silty sand/sandy silt deposit should be taken into consideration. In any event, the dewatering system must be effective in lowering the groundwater level to at least 0.4 m below the bottom of the excavation, including trenching to place granular bedding materials, as was discussed before.

The degree of difficulty with dewatering will largely depend on the site conditions at the time of construction. It is recommended that both water diversion and site dewatering schemes be left to the Contractor. We recommend however that as NSSP be included in the Contract Documents alerting the Contractor of the difficult subsurface condition and the high groundwater level encountered at the site, including the presence of dilatant silt and that the groundwater control should be planned and executed accordingly. We also recommend that the Contractor be asked to submit their water diversion and dewatering plan to the CA, for information purposes, prior to the start of the constructions.

5.6 Erosion Protection

Erosion and scour protection should be provided at the culvert inlet and outlet (including the slopes and sides). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. It should be pointed out that the silt is a highly erodible and scour prone material. The following are some general suggestions for erosion and scour protection.

We recommend that erosion and scour protection measures be taken at the inlet to prevent seepage beneath and around the culvert, especially through the granular bedding and granular backfill around the culvert. Beneath the culvert, the erosion and scour measures should extend to a suitable depth (e.g. below any possible scour depth). Consideration may also be given to a low permeability seal at the inlet and outlet.

At the inlet, consideration may also be given to the use of a clay seal. The purpose of the clay seal is to allow water flow to be channelled 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 is typically 0.6 m thick. It should comply with the material specifications given in OPSS 1205. It should be extended around the culvert from at least 0.3 m above the high water level in the watercourse down to the channel bed and up the other side in a continuous manner. It should be ensured that it extends to cover all the granular backfill materials to prevent any seepage through them. Typically, the clay seal is protected by laying a 0.6 m thick rock protection over it. The clay seal would generally be extended at about 6 m beyond the inlet.

At the outlet as well as at the inlet (if clay seal is not used), in addition a or low permeability seal or in conjunction with this, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered. As the subgrade can be expected to consist of silt, a layer of granular or man-made filter material should be used. This would generally be extended about 6 m along the channel and the sides (to at least 0.3 m above the high water). The granular filter material underlying the rock protection can consist of a suitable granular material such as Granular ‘A’. Alternatively, a suitable geotextile can be used underneath the rock fill, in lieu of the granular filter material. Another reference for consideration is OPSD 810.010 Rip-Rap Treatment for Culvert Outlets.

At the culvert outlet a filter diaphragm could also be considered to minimize the risk of migration of fines.

5.7 Frost Protection

Design frost protection depth for the site is about 2.4 m. A minimum 2.4 m thick permanent soil cover or equivalent thermal insulation is required for frost protection of foundations.

In case of rip-rap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

6 CLOSURE

We recommend that once the details of the culverts are finalized, our recommendations be reviewed for their specific availability. The "Limitations of Report" presented in Appendix H, are an integral part of this report.

For and on behalf of Coffey Geotechnics Inc.

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LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

Ramon Miranda. P.Eng.
Manager, Transportation

Zuhtu Ozden, P.Eng.
Senior Principal

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Appendix F

Proposed Staged Construction provided by the Client

Staging Typicals

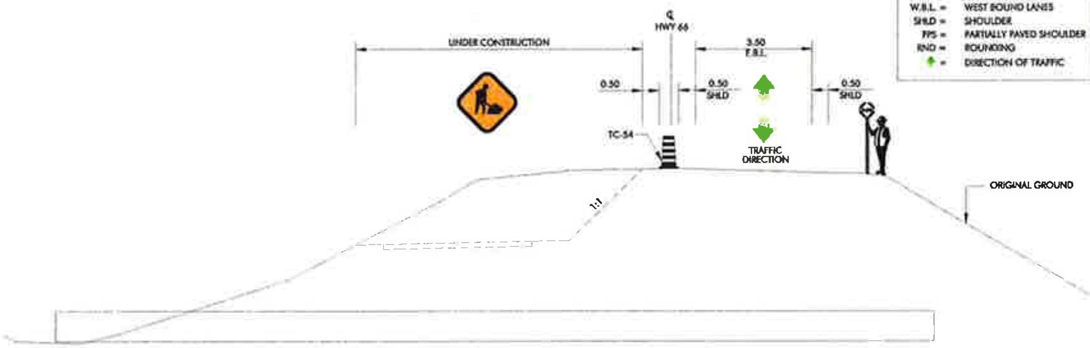
HIGHWAY 66
from Quebec-Ontario Boundary Westerly 19.2 kmto 2.3 km east of Highway 624
GWP 5117-03-00

CONSTRUCTION STAGING CULVERT REPLACEMENT 17+625

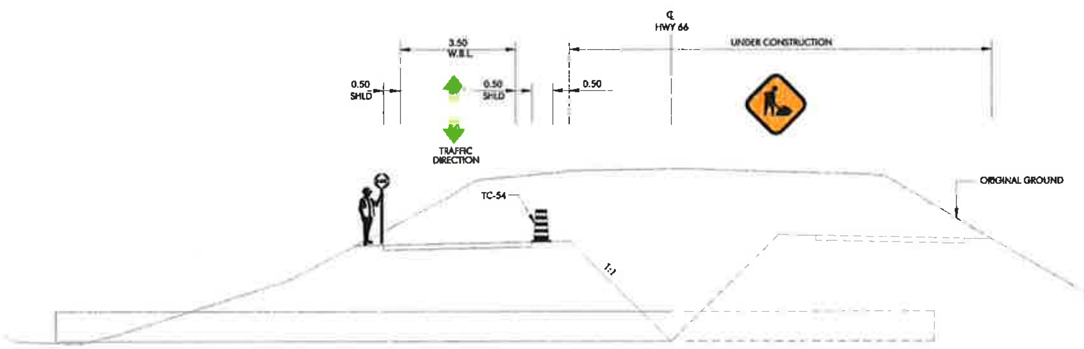
- Traffic on Highway 66 will be reduced to a single-lane using flagging

LEGEND

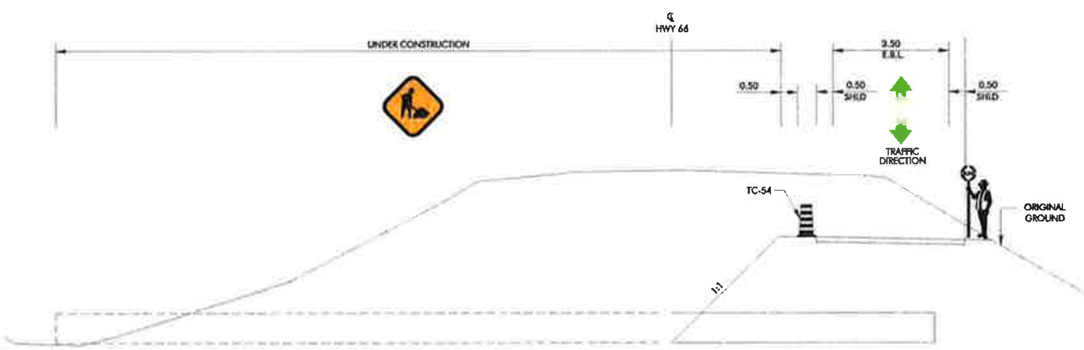
CL =	CENTRELINE
E.B.L. =	EAST BOUND LANES
W.B.L. =	WEST BOUND LANES
SHLD =	SHOULDER
PPS =	PARTIALLY PAVED SHOULDER
RND =	ROUNDING
=	DIRECTION OF TRAFFIC



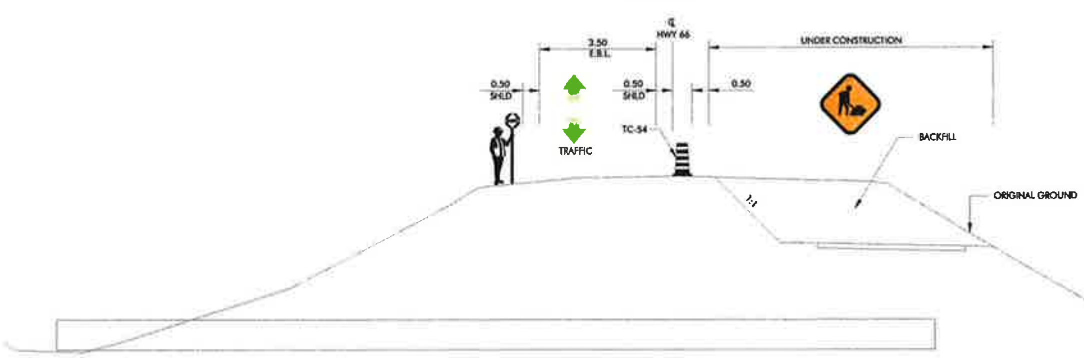
STAGE 1



STAGE 2



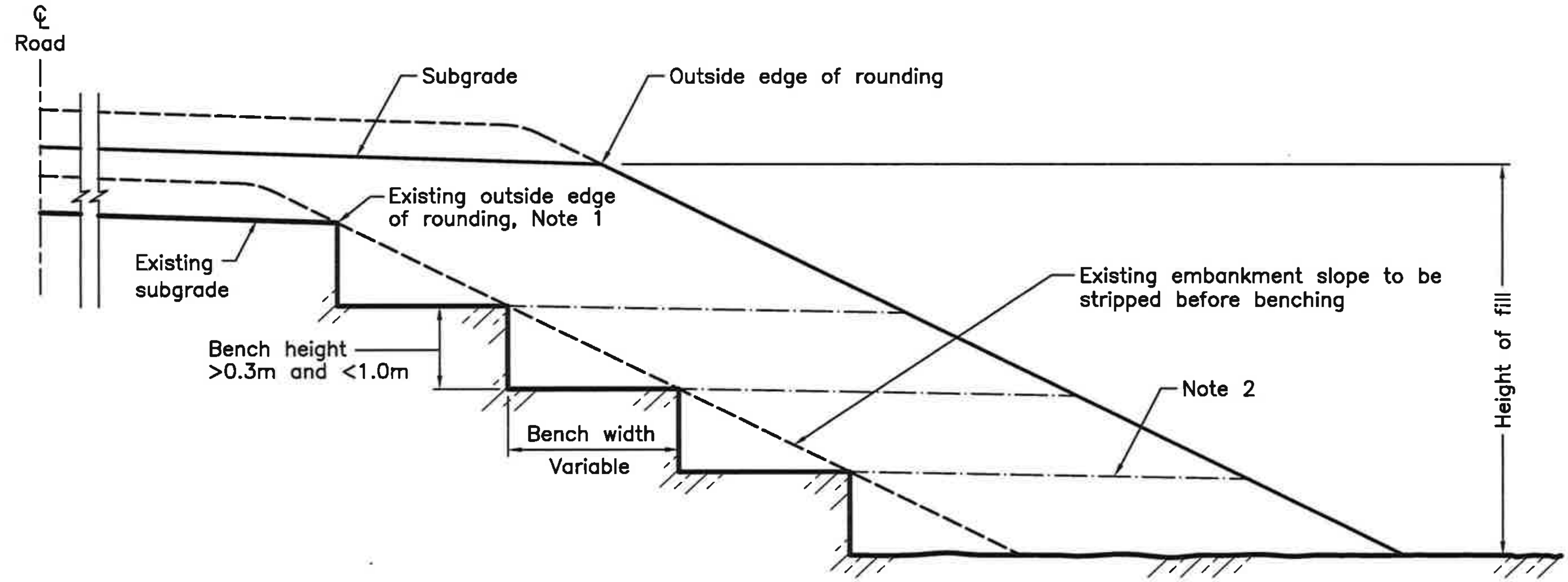
STAGE 3



STAGE 4

Appendix G

OPSD



NOTES:

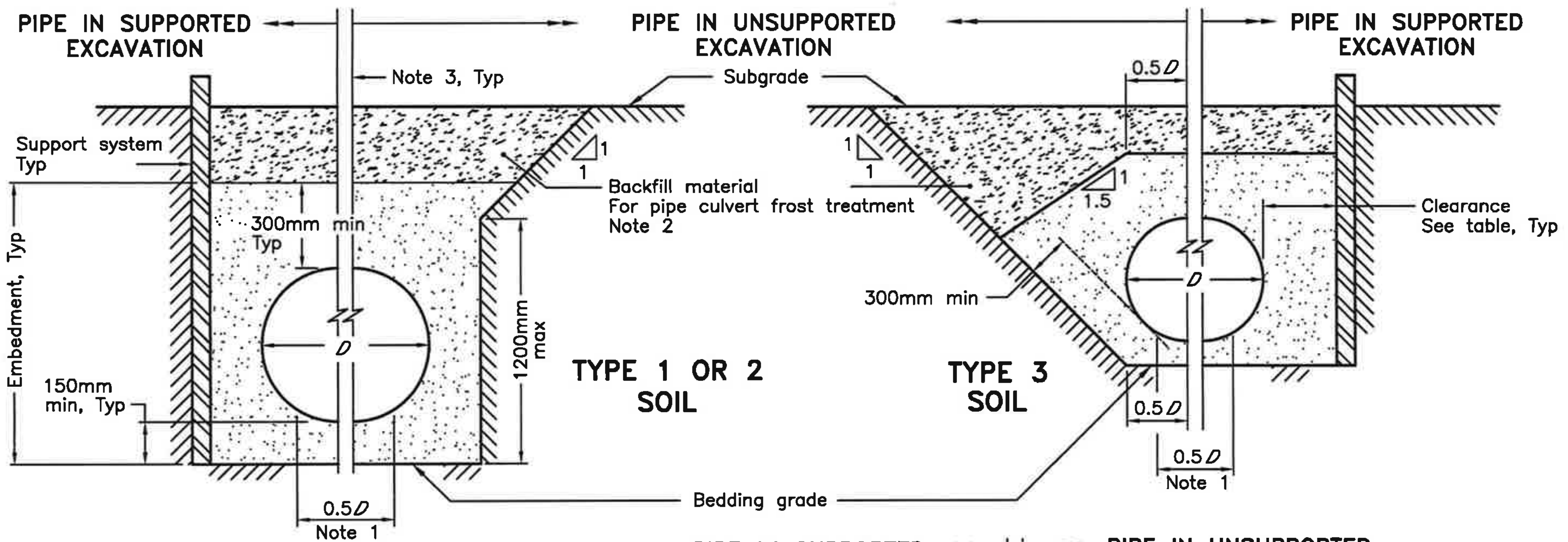
1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.

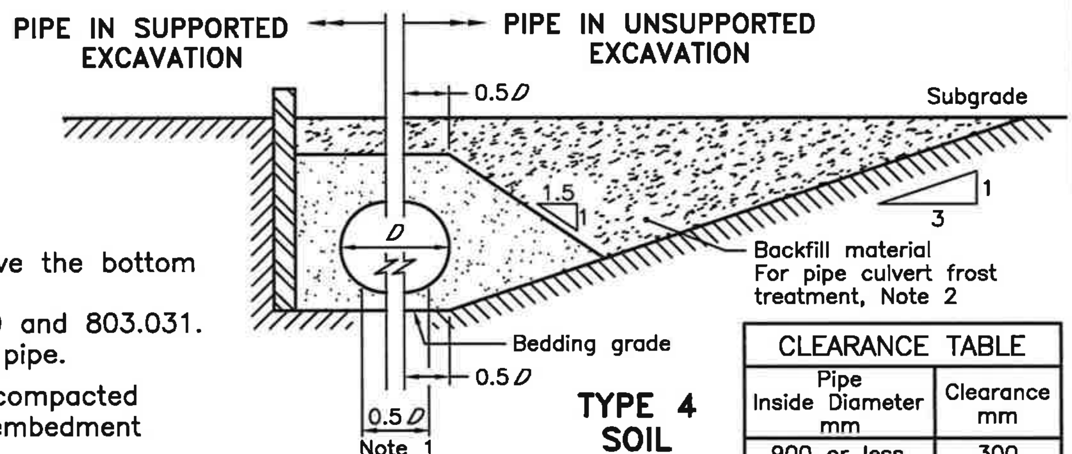
2 Benches are to be excavated one level at a time and the fill placed and compacted before the next bench is excavated.

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	





LEGEND:

D – Inside diameter

NOTES:

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

A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.
B Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
C All dimensions are in metres unless otherwise shown.

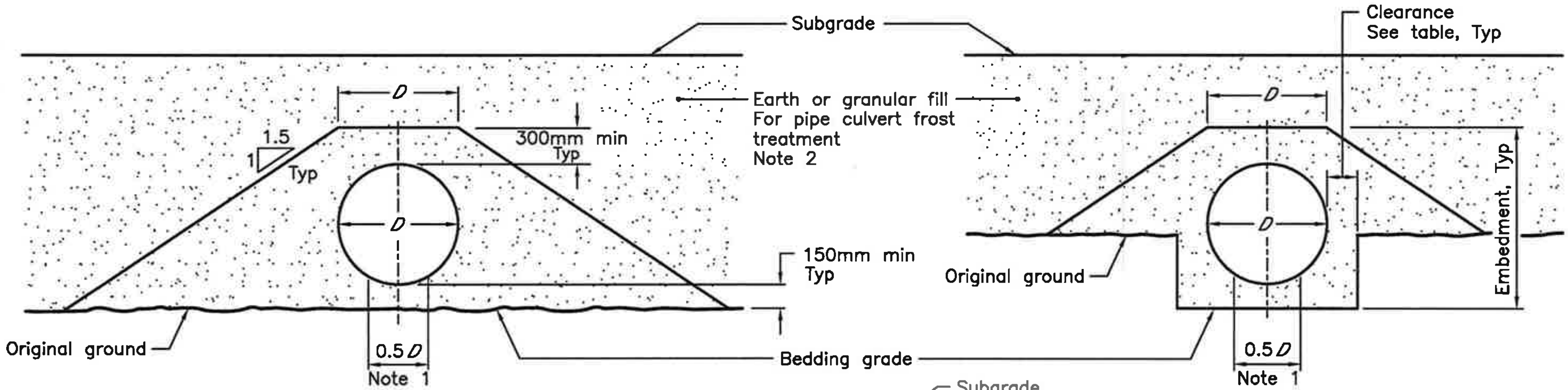
CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 Rev 1

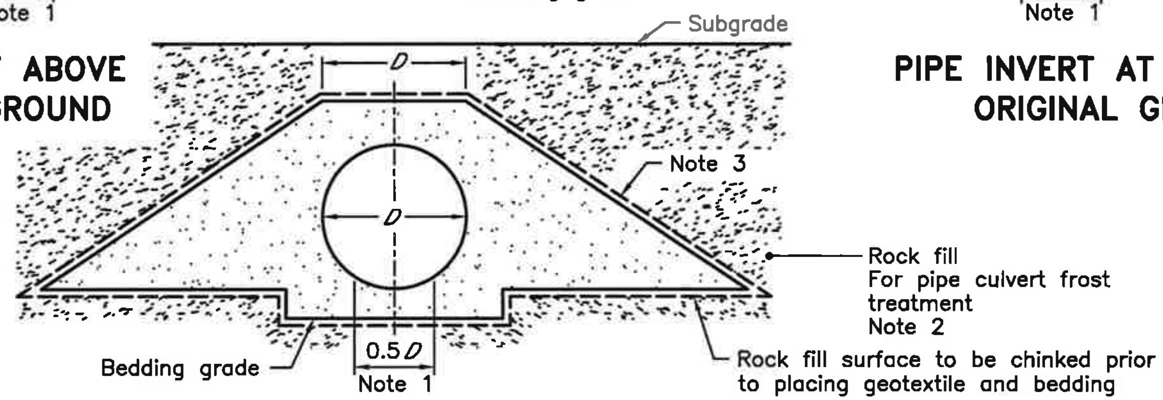
FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION

OPSD – 802.010



PIPE INVERT ABOVE ORIGINAL GROUND

PIPE INVERT AT OR BELOW ORIGINAL GROUND



PIPE EMBEDMENT WITH ROCK FILL UNDER AND OVER THE PIPE

LEGEND:

D – Inside diameter

NOTES:

1 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
2 Pipe culvert frost treatment according to OPSD–803.030 and 803.031.
3 Embedment material to be wrapped in non–woven geotextile when specified.

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

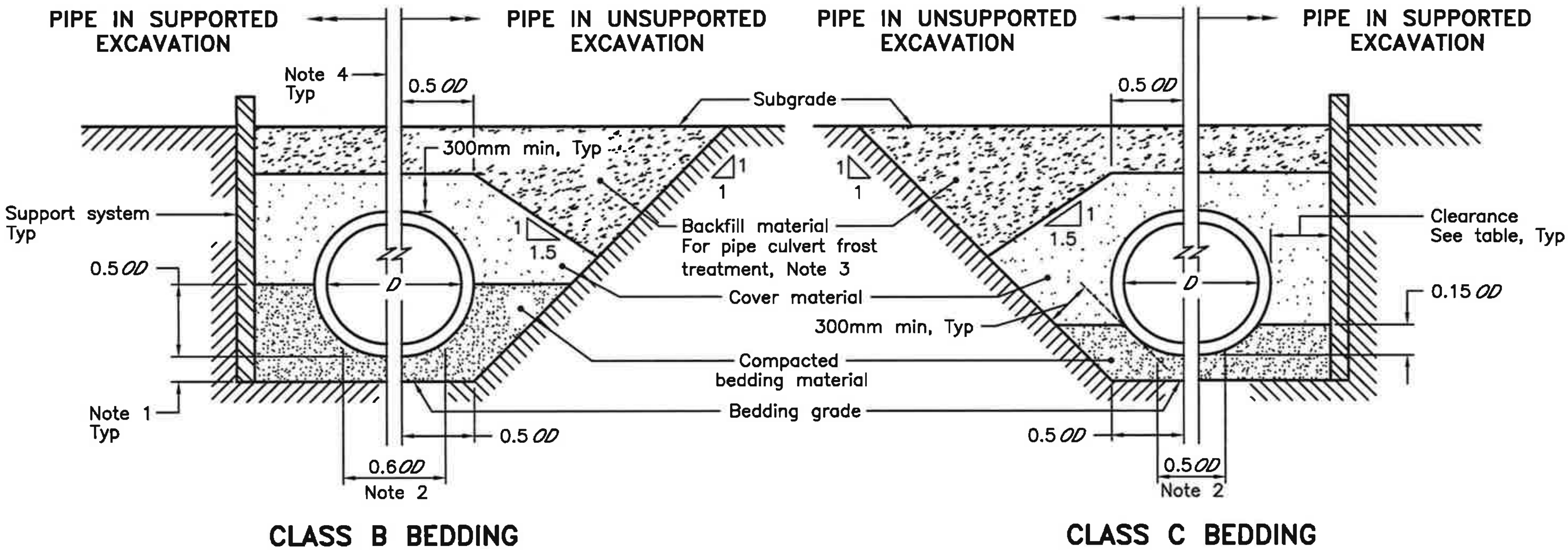
B All dimensions are in metres unless otherwise shown.

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

ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2005 | Rev 1

FLEXIBLE PIPE EMBEDMENT IN EMBANKMENT ORIGINAL GROUND: EARTH OR ROCK

OPSD – 802.014



NOTES:

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

LEGEND:

D – Inside diameter
OD – Outside diameter

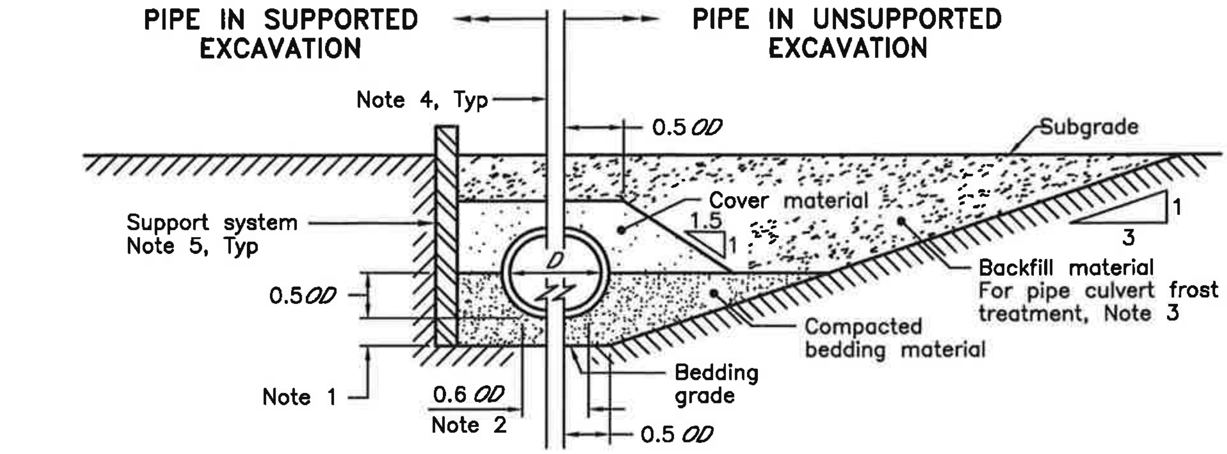
CLEARANCE TABLE

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

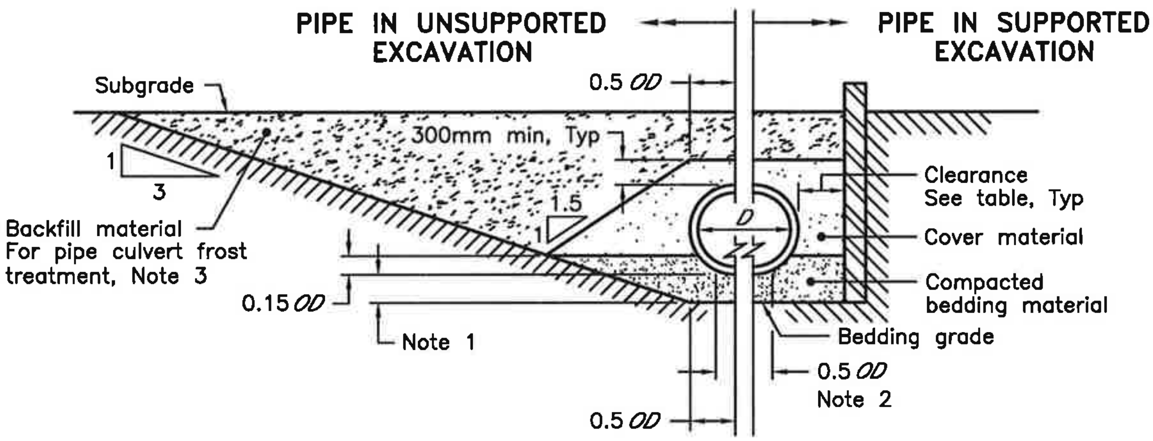
ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2005 | Rev 1

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

OPSD – 802.031



CLASS B BEDDING



CLASS C BEDDING

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

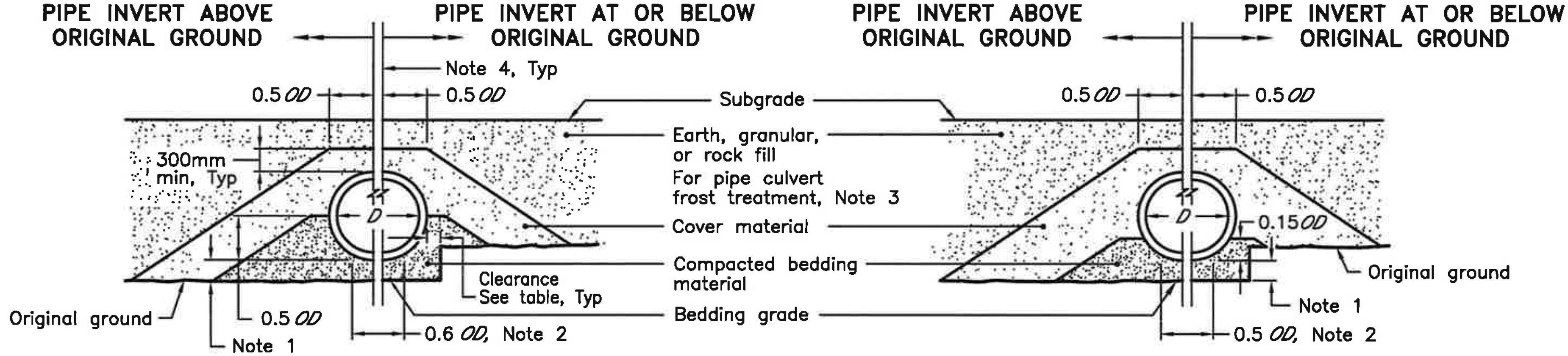
LEGEND:

D – Inside diameter
OD – Outside diameter

NOTES:

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

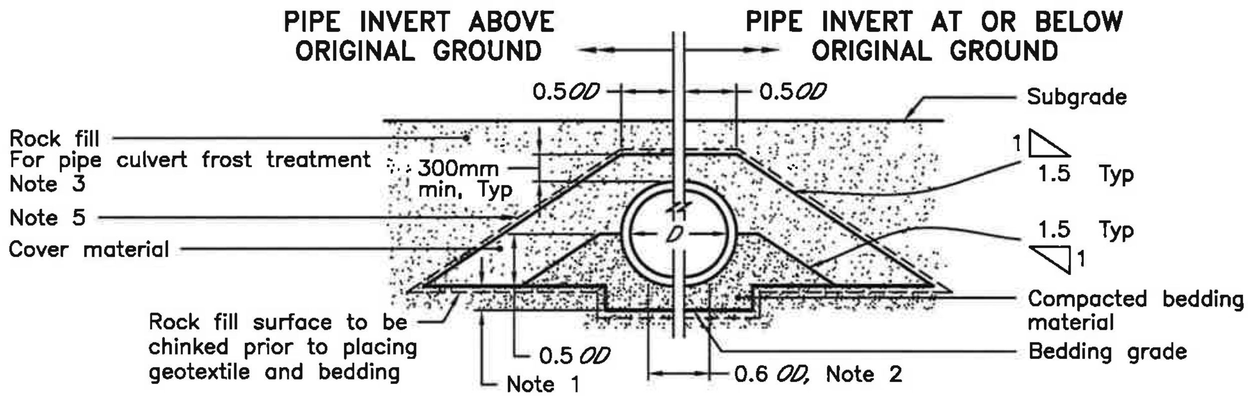
ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2005	Rev 1
RIGID PIPE BEDDING, COVER, AND BACKFILL TYPE 4 SOIL – EARTH EXCAVATION	**OPSD – 802.032**	



CLASS B BEDDING

CLASS C BEDDING

EARTH AND ROCK EXCAVATION



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

NOTES:

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

A All dimensions are in metres unless otherwise shown.

LEGEND:

D – Inside diameter
OD – Outside diameter

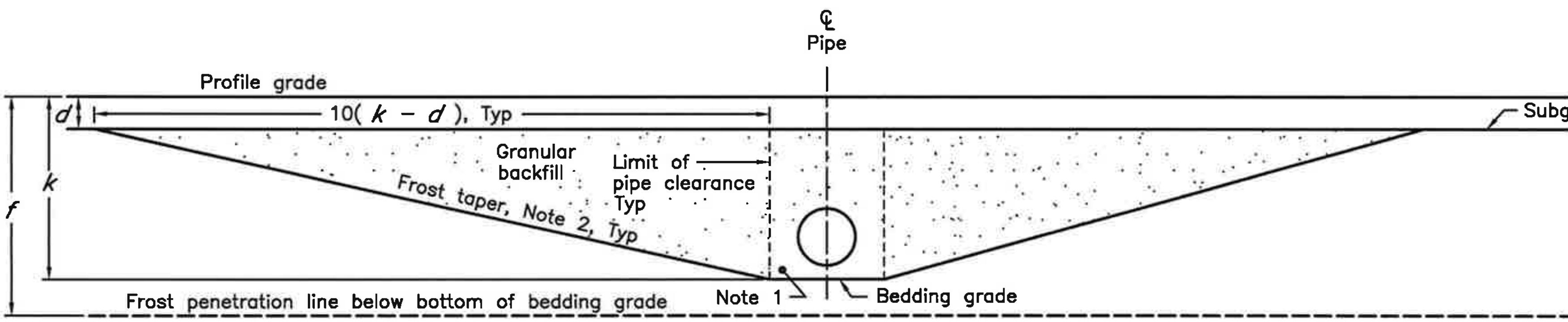
CLEARANCE TABLE	
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 | Rev 1

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

OPSD – 802.034



FROST TREATMENT – RIGID AND FLEXIBLE PIPE

NOTES:

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

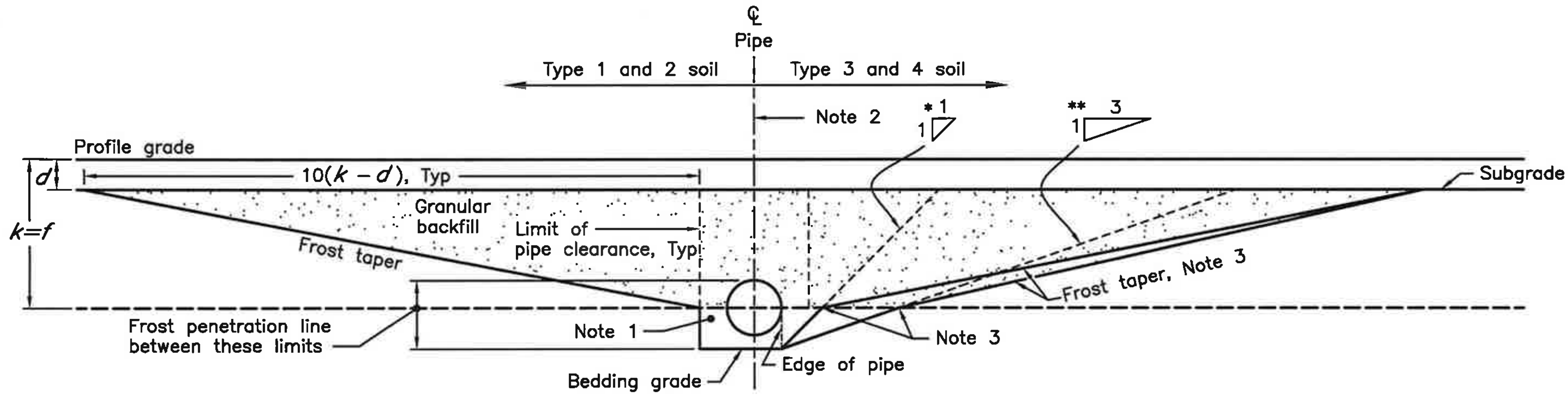
2 Frost tapers start at bedding grade.

A Frost tapers are not required in rock embankment.

LEGEND:

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

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



FROST TREATMENT – RIGID AND FLEXIBLE PIPE

NOTES:

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

2 Condition of frost treatment symmetrical about centreline of pipe.

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

A Frost tapers are not required in rock embankment.

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

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

LEGEND:

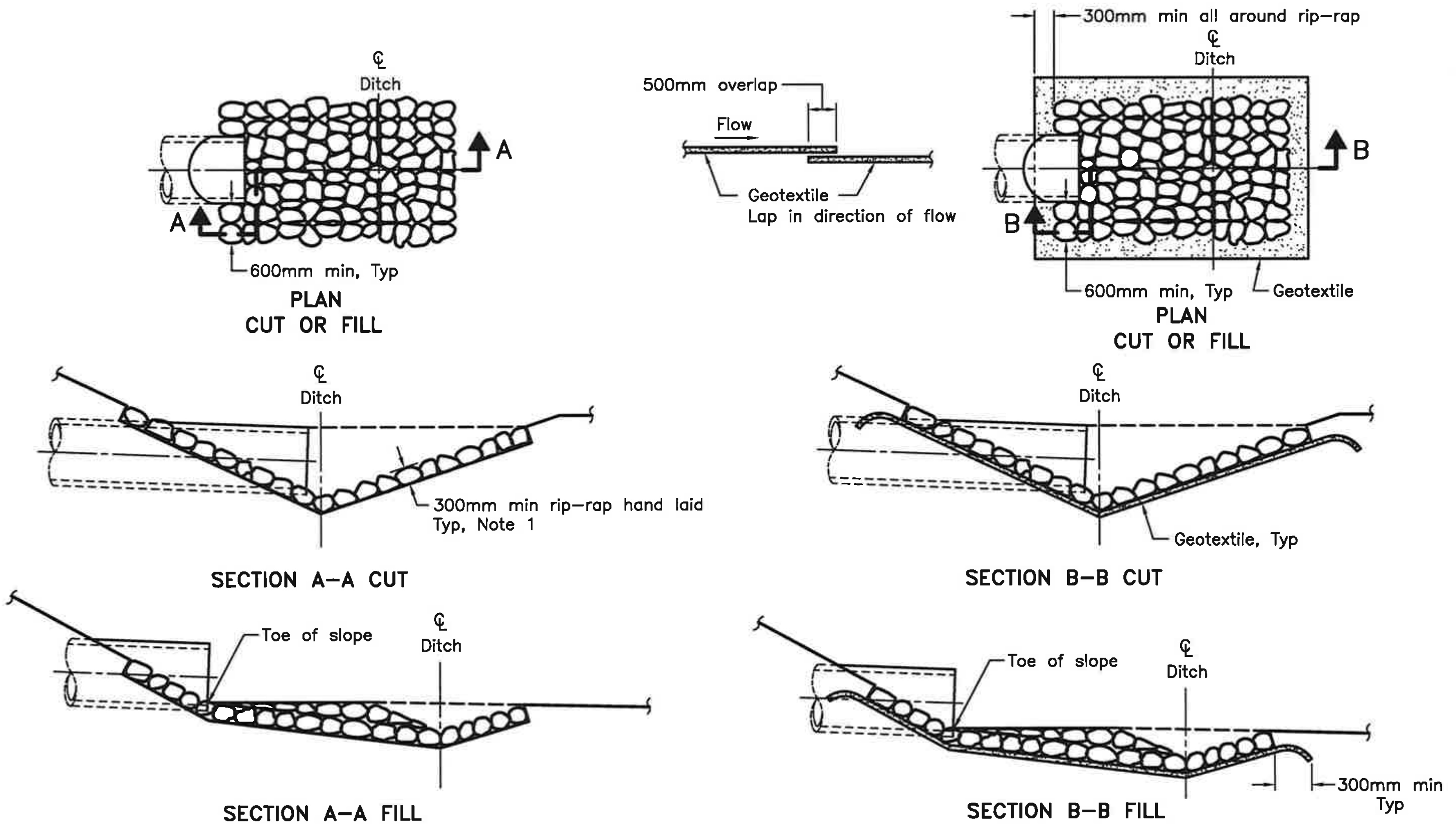
d – depth of roadbed granular
k – depth of frost treatment
f – depth of frost penetration
* – Type 3 soil
** – Type 4 soil

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 | Rev 2

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

OPSD – 803.031



TYPE A – WITHOUT GEOTEXTILE

TYPE B – WITH GEOTEXTILE

NOTES:

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

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2007	Rev	1
RIP–RAP TREATMENT FOR SEWER AND CULVERT OUTLETS	OPSD 810.010		

ONTARIO STANDARDS · MUNICIPAL–PROVINCIAL · OPS

Appendix H

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.