FOUNDATION INVESTIGATION AND DESIGN REPORTS PROPOSED CULVERT REHABILITATION (C17) AT STATION 14+750, HIGHWAY 522 REHABILITATION, FROM 32.2 KM WEST OF HIGHWAY 524 EASTERLY 6 KM G.W.P. 480-98-00, DISTRICT 54, SUDBURY GEOCRES NO. 31E-281

Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.

Project: SPT1218C
November 27, 2008

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1. INTRODUCTION

Shaheen & Peaker, A Division of Coffey Geotechnics Inc., was retained by D.M. Wills Associates to conduct a foundation investigation at the site of proposed rehabilitation of the existing culvert (Culvert C17) at Station 14+750 under Highway 522, near Port Loring in the Township of East Mills, Ontario.

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

The findings of the investigation are presented in this report.

2. SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located on Highway 522 about 2.6 km east of Port Loring.

Port Loring is situated about 60 km west of Trout Creek which is located at the junction of Highway 522 and Highway 11 (some 40 km south of North Bay). The topography near the site is of a rolling nature, with occasional knobs, resulting from bedrock outcrops.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the site is located within the Physiographic Region known as the Algonquin Highlands. The Quaternary deposits found in this area are quite complex, having resulted from a variety of geological processes associated with glacial, glaciofluvial, and glaciolacustrine conditions. A large proportion of the area consists of bare bedrock with thin drift. Much of this region is underlain by Precambrian rocks of the Grenville structural Province. These rocks have been strongly metamorphosed, folded, and then intruded by igneous rock.

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

The highway embankment at the culvert site is about 4 to 5 m above the level of the watercourse (see photographs in Appendix C).

3. PROCEDURES

The fieldwork for this project was performed between June 16 and 17, 2008 and consisted of drilling and sampling three boreholes to depths ranging from 2.3 to 10.2 m below the ground surface. The locations of the boreholes at the site are shown on the Borehole Location Plan Drawing No. 1.

The boreholes were advanced using a track-mounted drilling rig owned and operated by Landcore Drilling of Chelmsford, Ontario under the full-time direction and supervision of a geotechnical Engineer from S&P. The boreholes were advanced using continuous flight hollow-stem augers. The boreholes were extended by augering to depths ranging from 0.8 to 2.3 m below the ground surface, to refusal depths on the augers. Within these depths, the sampling was effected at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. The test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS – split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of granular (cohesionless) soils (gravels, sands and coarse silts) or the consistency of cohesive soils (clays and clayey silts).

In Borehole C3-3, after encountering practical refusal on the augers, the bedrock was proven by diamond drilling whereby NQ size rock cores were obtained from a depth of 1.5 to 4.7 m below the ground surface.

In Borehole C3-2, which was drilled from the top of the highway embankment, refusal to further augering was encountered at about 0.8 m below the road surface on rock fill. This borehole was advanced for 3.0 m (i.e. to 4.5 m depth) by coring method, below which depth the borehole was further advanced by washboring and Standard Penetration testing was resumed. The borehole was advanced by washboring to 7.1 m when coring of the bedrock was commenced. Coring of the bedrock was extended to 10.2 m (i.e. coring of the rock was carried out by 3.1 m of vertical length).

Dynamic Cone Penetration Tests (DCPT) were performed from the ground surface adjacent to Boreholes C3-1 and C3-3. In this test, a 51 mm diameter, 60-degree apex cone, screw attached to the tip of an A-size rod, is driven into the ground, using the same driving energy as the SPT method. By recording the number of blows of the hammer to drive the cone/rod assembly, into the soil every 0.3 m, a qualitative record of soil compactness condition is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic force effects which in some cases (such as the present case) affect the SPT results.

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

Water level observations in the open boreholes were made during drilling and at completion. In addition, a piezometer was installed at Borehole C3-3 to enable us to monitor the ground water level over a prolonged period of time, without interference for surface water. Water level observations in the piezometer were made by subsequent site visits.

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

The soil samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations and grain size analyses, was performed on selected representative samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

4. SUMMARIZED SUBSURFACE CONDITIONS

Borehole C3-1 which was drilled at the toe of the embankment from the o.g. (original ground) level contacted topsoil to a depth of 0.2 m below the ground surface.

Borehole C3-2 was drilled from the top of the highway embankment and therefore contacted embankment fill materials. The depth of the fill was found to be 4.6 m at the borehole location.

Underlying the embankment fill in Borehole C3-2, the veneer of topsoil in Borehole C3-1 and the o.g. (original ground) level in Borehole C3-3, the depth of natural overburden soils ranged from 1.5 m (at Borehole C3-1) to 2.5 m (at Borehole C3-2). The overburden was found to consist of typically granular soils ranging from silty sand with traces of gravel to silty sand till to fine-grained granular silt layers. A cohesive 0.9 m thick clayey silt to silty clay layer was also encountered in Borehole C3-1.

In Boreholes C3-2 and C3-3 the surface of the bedrock was contacted at a depth of 7.1 m (or 2.5 m below o.g. level) and 1.5 m or at El. 232.3 m and 234.2 m, respectively. In Borehole C3-1, refusal to augering and to DCPT was contacted at a depth of 2.3 m or at El. 232.8 m, probably at or close to surface of the bedrock.

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

4.1 TOPSOIL

In Borehole C3-1, which was drilled near the toe of the highway embankment, a 0.2 m thick sandy topsoil layer was contacted.

It should, however, be pointed out that in our experience the thickness of topsoil and other organic rich soils frequently varies in between and beyond borehole locations. In particular, thicker organic soils frequently occur in depressed areas and within watercourse valleys.

4.2 FILL

Fill materials were contacted in Borehole C3-2 (drilled from the top of the road embankment) to a depth/elevation of 4.6 m/234.8 m.

Beneath a 50 mm thick asphaltic concrete layer, a granular pavement fill consisting of sand and gravel was contacted to a depth of 0.8 m below the ground surface. Based on a recorded N-value of 21 blows/0.3 m, the relative density of this granular pavement fill is described as compact.

Between 0.8 m (El. 238.6 m) and 3.8 m (235.6 m), the pavement fill at the location Borehole C3-2 consists of rock fill.

Underlying the rock fill, the borehole contacted another type of fill consisting of sandy gravel. The sandy gravel fill was contacted at 3.8 m/El. 235.6 m and extended to 4.6 m/El. 234.8 m. From a recorded N-value of 21 blows/0.3 m, this granular soil is described as compact.

4.3 SURFICIAL SILTY SAND

Underlying the topsoil, Borehole C3-1 contacted a 0.4 m thick layer of silty sand with traces of gravel, to a depth of 0.6 m below the o.g. level or to El. 234.5 m. Based on an N-value of 5 blows/0.3 m, the relative density of this surficial granular soil is considered loose.

In Borehole C3-3, a silty fine sand layer with some silt and clayey silt seams was encountered from the o.g. level (El. 235.7 m) to 0.7 m (El. 235.0 m). This deposit is considered to be a basically granular type soil. A Standard Penetration test performed in this deposit yielded an N-value of 11 blows/0.3 m, indicating a compact condition.

4.4 SILT

Silt with traces of gravel and sand size particles was contacted in Borehole C3-1 at a depth of 1.5 m (El. 233.6 m) below the o.g. level and also in Borehole C3-2 immediately underlying the embankment fill at El. 234.8 m. This fine-grained granular unit was found to be 0.8 m and 0.7 m thick and extended to 2.3 m/El. 232.8 m (refusal depth) and 5.3 m/ El. 234.1 m in Boreholes C3-1 and C3-2, respectively.

From the recorded N-values of 9 and 15 blows/0.3 m, the relative density of this deposit is described as loose to compact.

4.5 CLAYEY SILT

In Borehole C3-1, underlying the surficial silty sand, a 0.9 m thick cohesive soil consisting of clayey silt to silty clay was contacted at 0.6 m below the o.g. level. This deposit was found to extend to a depth of 1.5 m or to El. 233.6 m.

The grain-size distribution of a sample from the deposit was determined in the laboratory and, as shown in the grain-size distribution curve, Figure B-1 in Appendix B, the following grain-size distribution is indicated:

Gravel:	0%
Sand:	14%
Silt:	56%
Clay:	30%

From these results, the deposit is considered to be less pervious than the other overburden deposits encountered at the site.

A Standard Penetration test performed in the deposit yielded an N-value of 4 blows/0.3 m which indicates a soft consistency.

4.6 BASAL SILTY SAND TILL

In Boreholes C3-2 and C3-3, a basal glacial till deposit was contacted immediately overlying the bedrock. This granular soil deposit was contacted at a depth of 0.7 m below the approximate o.g. levels and extended to depths of 2.5 m and 1.5 m below o.g. or to El. 232.3 m and 234.2 m, respectively, to the surface of the bedrock.

The grain-size distribution of two samples from the deposit was determined in the laboratory as follows (as given in Figure B-2 in Appendix B).

Gravel:	11-12%
Sand:	45-46%
Silt:	29-30%
Clay:	12-15%

Standard Penetration tests performed in this basal, basically granular deposit yielded SPT resistance values of 11 to 15 blows/0.3 m which indicate a compact condition.

4.7 BEDROCK

After encountering refusal on the augers in Borehole C3-3 and to casing advance in Borehole C3-2, coring of the bedrock was effected starting at depths/elevations 1.5 m/234.2 m and 7.1 m/232.3 m, respectively. In Borehole C3-1, refusal to augering as well as to Dynamic Cone Penetration Testing (DCPT) was encountered at 2.3 m or El. 232.8 m. The following table summarizes the proven or inferred surface of the bedrock at the borehole locations.

Table 4.7.1
Inferred Bedrock Surface

Borehole No.	Existing Ground Surface Elevation (m)	Inferred or Proven Bedrock Surface Below Existing Ground Surface (m)	Inferred or Proven Bedrock Surface Elevation (m)	Coring Carried Out
C3-1	235.1	2.3±	232.8±	No
C3-2	239.4	7.1	232.3	Yes
C3-3	235.7	1.5	234.2	Yes

From the above results, it appears that the surface of the bedrock dips down from El. 234.2 m at Borehole C3-3 location to El. 232.3 m at Borehole C3-2 located on the paved portion of the road. This represents a drop of 1.9 m over a horizontal distance of 19 m or about 10%. The inferred drop in elevation from Borehole C3-1 to C3-2 is relatively milder (i.e. about 0.5 m over a horizontal distance of 9 m or about 5.5%).

From a visual examination of the core samples, the bedrock appears to be a light to dark grey gneiss with some reddish to pinkish grey bands typically at about 30 degree inclination to the horizontal.

High Total Core Recovery (TCR) and Rock Quality Designation (RQD) values were recorded on the rock cores, as detailed below.

Table 4.7.2
Rock Core Information

Borehole No.	Rock Core No.	Percentage of Recovery (%)	Measured R.Q.D.* Value (%)
C3-2	10	96	80
	11	98	91
C3-3	3	92	86
	4	98	84
	5	100	95

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

In summary, the measured TCR values range from 92 to 100% and the measured RQD values range from 80 to 95%. Based on these values, the rock is described as sound, with a rock quality designation of good to excellent.

4.8 GROUNDWATER CONDITIONS

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

A piezometer was installed in Boreholes C3-3 at a depth of 4.7 m to enable us to monitor the groundwater level over a prolonged period of time without interference.

The water level in the piezometer was measured at a depth of 0.2 m below the o.g. level or at El. 235.5 m, about two weeks after the installation of the piezometer.

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

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

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

SHAHEEN & PEAKER LIMITED

Ramon Miranda, P.Eng.

Z. S. Ozden, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

ZO:tr/idrive

Drawings

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
Nov 27/08
PROVINCE OF ONTARIO

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

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

METRIC

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

CONT No.
WP: 480-98-00

Highway 522 Port Loring
BOREHOLE LOCATION PLAN & STRATIGRAPHY (Culvert 17 @ STA. 14+750)

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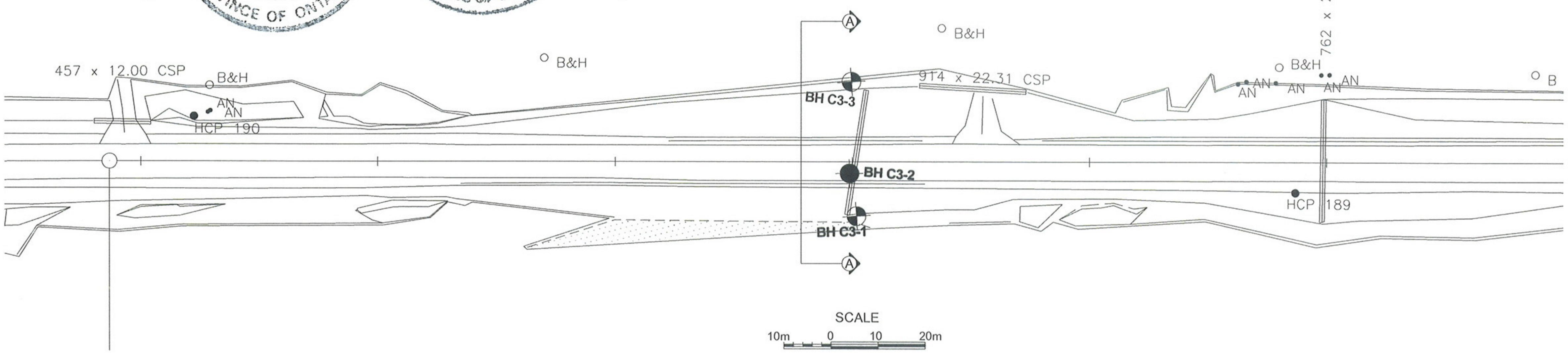
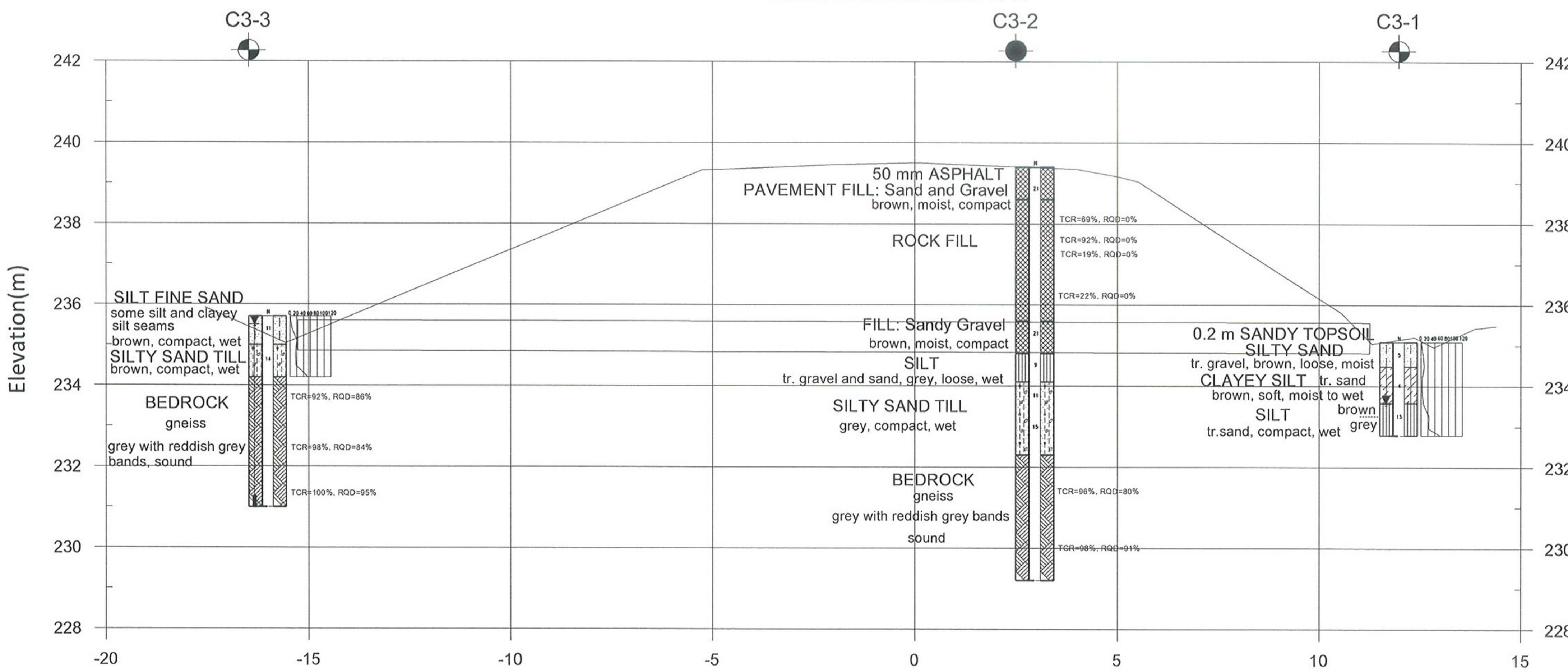
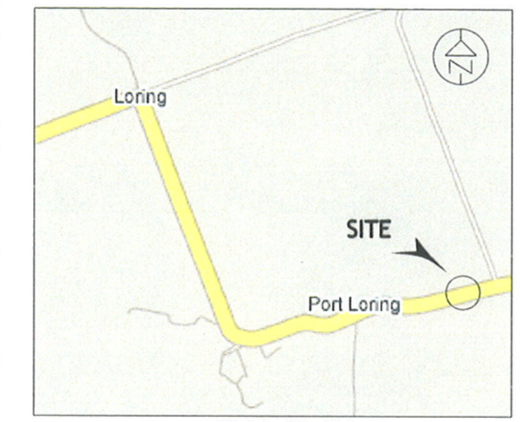
A Division of Coffey Geotechnics Inc.



BOREHOLE LOCATION PLAN



CROSS SECTION (A-A)



KEY PLAN
N.T.S

LEGEND

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

No.	ELEV.	STATION NO.	OFFSET
C3-1	235.1 m	14+752	11.5 m Rt
C3-2	239.4 m	14+748.5	2.5 m Rt
C3-3	235.7 m	14+748	16.5 m Lt

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

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

REV.	DATE	BY	DESCRIPTION

Geocres No. 31E-281

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

Appendix A

Records of Borehole Sheets

SPT1218C: Highway 522 (Port Loring)

RECORD OF BOREHOLE No C3-1

1 OF 1 **METRIC**

GWP 480-98-00 LOCATION Sta : 14+752, 11.5 m Rt C/L of Hwy 522 ORIGINATED BY RK

DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS

DATUM Geodetic DATE 6/17/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
235.1	GROUND SURFACE							
0.0	0.2 m **SANDY TOPSOIL** **SILTY SAND** tr. gravel brown, loose, moist	1	SS	5		235		
234.5 / 0.6	**CLAYEY SILT** tr. sand brown, soft, moist to wet	2	SS	4	▽*	234		0 14 56 30
233.6 / 1.5	**SILT** tr sand, compact, wet (brown / grey)	3	SS	15		233		spoon bouncing and auger refusal @ 2.3 m
232.8 / 2.3	End of borehole Auger refusal @2.3 m Water level in open hole @ 1.5 m upon completion (not stabilized)* Dynamic Cone Penetration Test (DCPT) performed adjacent to the borehole from 0 to 2.3 m.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m^3

SPT1218C: Highway 522 (Port Loring)

RECORD OF BOREHOLE No C3-2

1 OF 1 **METRIC**

GWP 480-98-00 LOCATION Sta : 14+748.5, 2.5 m Rt C/L of Hwy 522 ORIGINATED BY RK

DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS

DATUM Geodetic DATE 6/17/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.4	GROUND SURFACE						
0.0	50 mm **ASPHALT** **PAVEMENT FILL:** Sand and Gravel brown, moist, compact	1	SS	21	239	o	
238.6 / 0.8	**ROCK FILL**	2	RC	TCR=69% RQD=0%	238		Auger refusal @ 0.8 m Coring w/NQ core barrel
		3	RC	TCR=92% RQD=0%			
		4	RC	TCR=19% RQD=0%	237		
		5	RC	TCR=22% RQD=0%	236		
235.6 / 3.8	**FILL:** Sandy Gravel brown, moist, compact	6	SS	21	235	o	
234.8 / 4.6	**SILT** tr. gravel and sand grey, loose, wet	7	SS	9		o	
234.1 / 5.3	**SILTY SAND TILL** grey, compact, wet	8	SS	11	234	o	12 46 30 12
		9	SS	15	233	o	
232.3 / 7.1	**BEDROCK** gneiss grey with some reddish grey bands sound	10	RC	TCR=96% RQD=80%	232, 231		
		11	RC	TCR=98% RQD=91%	230		
229.2 / 10.2	End of borehole						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; 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

20 15 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No C3-3

METRIC

GWP 480-98-00 LOCATION Sta : 14+748, 16.5 m Lt C/L of Hwy 522 ORIGINATED BY RK

DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS

DATUM Geodetic DATE 6/16/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
235.7	GROUND SURFACE					
0.0	**SILT FINE SAND** some silt and clayey silt seams brown, compact, wet	1	SS	11	o	
235.0 / 0.7	**SILTY SAND TILL** brown, compact, wet	2	SS	14	o	11 45 29 15
234.2 / 1.5	**BEDROCK** gneiss grey with reddish grey bands sound	3	RC	TCR=92% RQD=86%	o	Auger refusal @ 1.5 m, core with NQ core barrel
		4	RC	TCR=98% RQD=84%		
		5	RC	TCR=100% RQD=95%		
231.0 / 4.7	End of borehole					

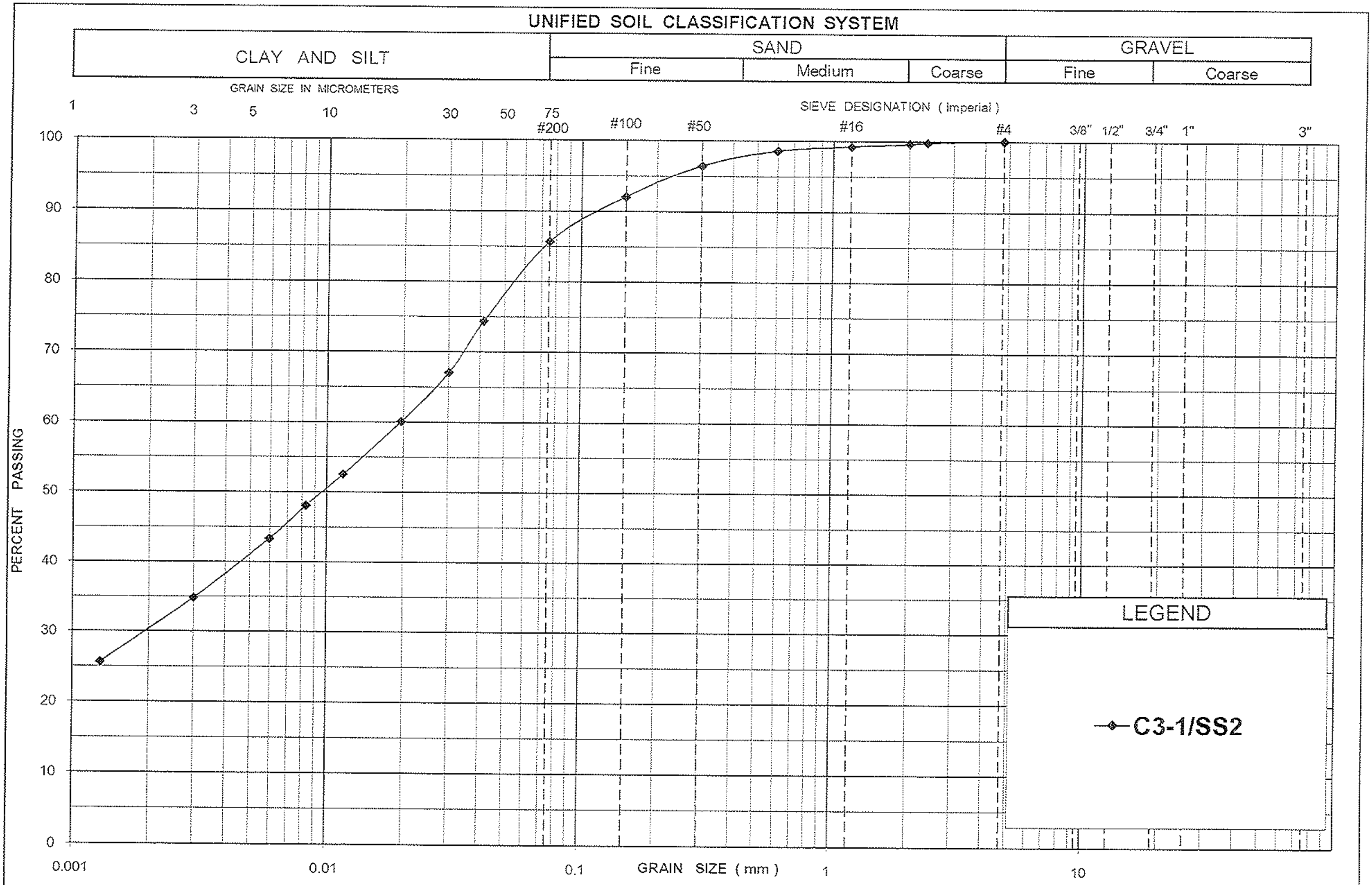
Elevation scale: 235, 234, 233, 232, 231

End of borehole
*Water level in open hole @ 1.2m upon completion, caved @ 2.8 m.
Dynamic Cone Penetration Test (DCPT) performed adjacent to the borehole from 0 to 1.5 m.
Piezometer installed to 4.7 m.
Piezometer reading
-July 02,2008 0.2 m (El. 235.5 m)

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

Appendix B

Laboratory Test Results



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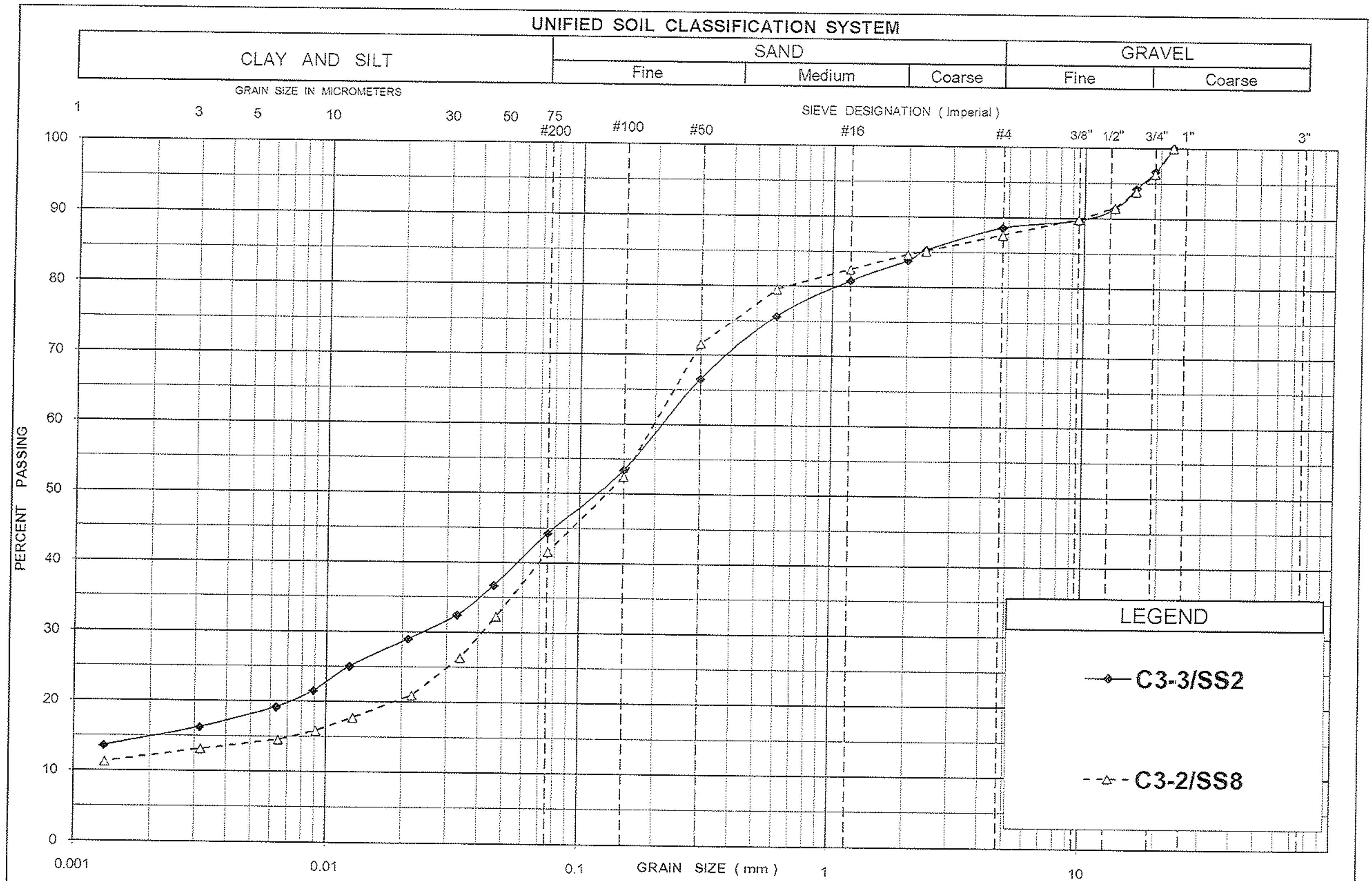
GRAIN SIZE DISTRIBUTION
CLAYEY SILT, trace sand

FIGURE No. B-1
REF. No. SPT 1218 C
DATE SEPTEMBER 2008



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION SILTY SAND TILL	FIGURE No. B-2 REF. No. SPT 1218 C DATE SEPTEMBER 2008

Appendix C

Site Photographs

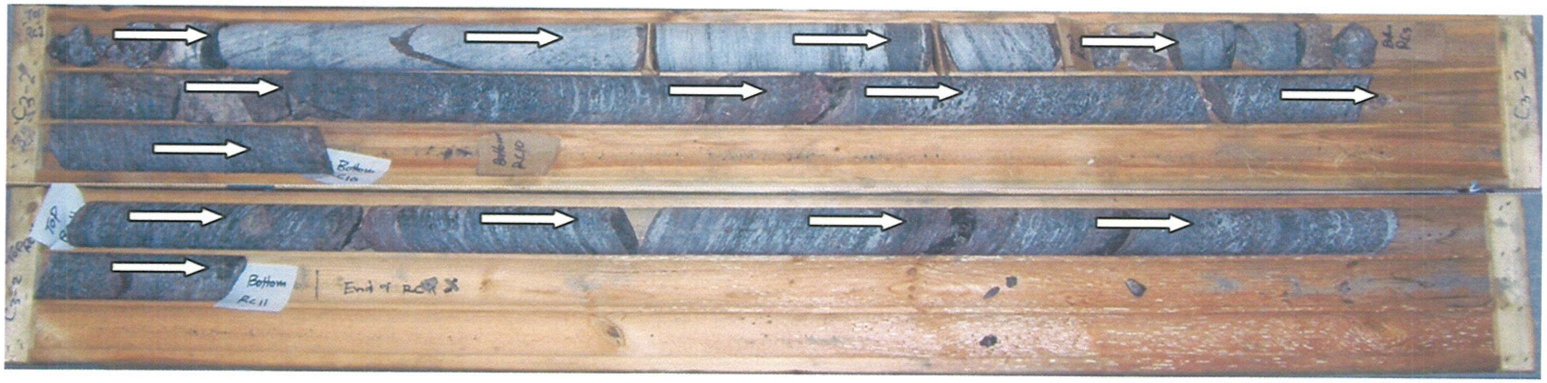
Photograph 1. Culvert C 17 location, north side

Photograph 2. Culvert C 17 location, south side

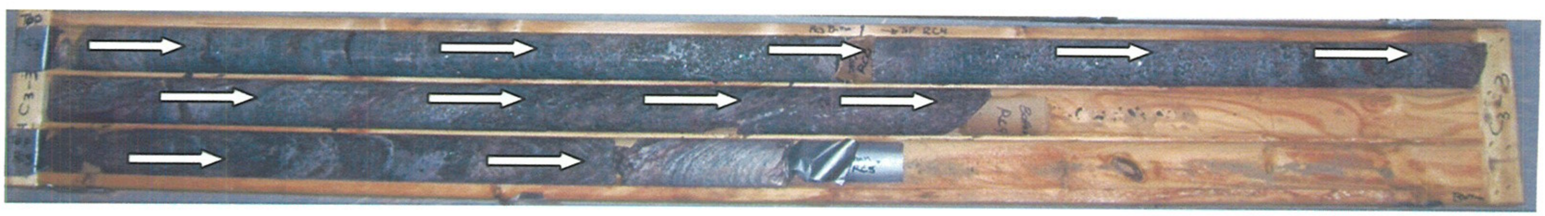
Photograph 3. Culvert C 17 location, south side

Appendix D

Rock Core Photographs



C3-2



C3-3

Appendix E

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON
WS	WASH SAMPLE
ST	SLOTTED TUBE SAMPLE
BS	BLOCK SAMPLE
CS	CHUNK SAMPLE
TW	THINWALL OPEN
TP	THINWALL PISTON
OS	OSTERBERG SAMPLE
RC	ROCK CORE
PH	TW ADVANCED HYDRAULICALLY
PM	TW ADVANCED MANUALLY
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
i_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
P_w	kg/m^3	DENSITY OF WATER
i_w	kN/m^3	UNIT WEIGHT OF WATER
P	kg/m^3	DENSITY OF SOIL
i	kN/m^3	UNIT WEIGHT OF SOIL
P_d	kg/m^3	DENSITY OF DRY SOIL
i_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
i_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
P'	kg/m^3	DENSITY OF SUBMERED SOIL
i'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
s_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $(W_L – W_L)$
I_L	1	LIQUIDITY INDEX = $(W – W_P)/ I_P$
I_C	1	CONSISTENCY INDEX = $(W_L – W) / 1_P$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	N PERCENT – DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

FOUNDATION DESIGN REPORT
PROPOSED CULVERT REHABILITATION (C17) AT
STATION 14+750, HIGHWAY 522 REHABILITATION,
FROM 32.2 KM WEST OF HIGHWAY 524 EASTERLY 6 KM
G.W.P. 480-98-00, DISTRICT 54, SUDBURY
GEOCRES NO. 31E-281

Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.

Project: SPT1218C
November 27, 2008

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FOUNDATION DESIGN REPORT
PROPOSED CULVERT REHABILITATION (C17) AT
STATION 14+750, HIGHWAY 522 REHABILITATION,
FROM 32.2 KM WEST OF HIGHWAY 524 EASTERLY 6 KM
G.W.P. 480-98-00, DISTRICT 54, SUDBURY

5. DISCUSSION AND RECOMMENDATIONS

The existing culvert (C17) at Station 14+750 is an approximately 26 m long, 750 mm diameter CSP. The top of paved road at the culvert location is at about El. 239.4 m, which represents an embankment height of about 4.4 m.

The three boreholes drilled at the site showed that the overburden has a limited thickness of between 1.5 and 2.5 m below the o.g. levels. The natural overburden was found to consist of fine to coarse grained granular soils (typically fine-grained silt to silty sand) of loose to compact relative density, except for a 0.9 m thick layer of clayey silt to silty clay of soft consistency in Borehole C3-1. The thickness of the embankment fill at the location of Borehole C3-2 was found to be 4.6 m. The groundwater table at the time of our investigation was recorded at about the o.g. level, but would be subject to fluctuations, both seasonally and in response to major weather events as well as the water level in the existing water course.

5.1 REHABILITATION OF THE EXISTING CULVERT

We understand that the existing 750 mm diameter CSP will be relined to rehabilitate it. For this purpose a smaller diameter pipe will be placed inside the existing pipe and the space between the two pipes will be grouted. It is anticipated that the thickness of the grout to fill the space in between the pipes will be of the order of 150 mm. Based on this information, the additional stresses on the surface of the subgrade would be less than 15 kPa. Using this figure and the subsurface data obtained from the boreholes, the settlement due to the rehabilitation should not exceed 15 mm (i.e. the settlements due to the additional stresses should be limited to 15 mm).

We also understand that the pavement may be rehabilitated without widening the embankment. The rehabilitation of the pavement may involve up to 100 mm grade raise. This may induce additional settlements but the aggregate settlements (i.e. due to culvert rehabilitation and the minor grade raise) should not exceed 20 mm. This is considered acceptable.

We recommend that the grouting be carried out in a manner so as not to cause an uplift of the liner pipe and also the road surface.

We also recommend that during the construction the amount of grout pumped be checked and compared with calculated volumes and in the event of a discrepancy, the construction will need to be halted and the reason(s) for the discrepancy will need to be investigated.

It is furthermore recommended that the ground surface be observed (especially in areas where the soil cover is limited e.g. side slopes) during grouting for signs of uplift.

5.1.1 EROSION PROTECTION

We recommend that the existing culvert be evaluated for the sufficiency of the existing erosion and scour measures and if observations show that they are deficient or if the realigning is expected to adversely affect erosion and scour potentials, further measures may be necessary. The following is a discussion of possible erosion measures.

Erosion and scour protection should be provided at the culvert inlet and outlet (including the slopes and sides). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. The following are some general suggestions, considering that below some probable organic and alluvial deposits at the watercourse level the boreholes indicate that the native soils can be expected to consist of silt, silty sand, silty sand till and clayey silt. The silt and silty sand are considered to be highly erodible soil types.

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

At the inlet, consideration may also be given to the use of a clay seal. The purpose of the clay seal is to ensure that water flow is channeled through the culvert and does not seep through the backfill around the structure and from beneath the structure. The clay seal should therefore be continuous and 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 0.5 m above the high water level in the watercourse down to the channel bed and up the other side in a continuous manner. It should be ensured that it extends to cover all the granular backfill materials to prevent any seepage through them. Typically, the clay seal is protected by laying a 0.6 m thick rock protection over it. The clay seal would generally be extended at about 6 m beyond the inlet.

At the outlet as well as at the inlet (if clay seal is not used), in addition to the concrete cut-off wall and/or impervious seal or in conjunction with these, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered. As the subgrade can be expected to consist of silty soils, 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.

5.2 CULVERT REPLACEMENT

We understand that culvert replacement is not contemplated for this project but the following are some brief comments.

As we understand, the invert elevation of the existing culvert is about 234.8 m.

The highest suitable subgrade elevation at the borehole locations for receiving the pipe and the bedding material is given in the following paragraphs:

Table 5.2.1
Highest Suitable Culvert Support Elevations

Borehole No.	Existing Ground Elevation (m)	Highest Suitable Subgrade Depth (m)	Highest Suitable Subgrade Elevation (m)
C3-1	235.1	0.3	234.8
C3-2	239.4	4.8	234.6
C3-3	235.7	0.3	235.4

We recommend that all stripping and subgrade preparation be carried out under geotechnical supervision. After stripping, the bearing surface should be inspected, evaluated and approved by the geotechnical engineer appointed by QVE. The boreholes show the presence of dilatant soils and the groundwater table at the time of our investigation was high. For this reason, the site must be properly dewatered and the dilation of the silty soils must be prevented. Otherwise, excessive settlements may ensure after the excavation is backfilled. This will be further elaborated in the next section of the report. As well, for this reason cited above and the fact that the soils are of limited geotechnical resistance (e.g. the clayey silt layer in Borehole C3-1), the use of a flexible pipe such as a CSP is recommended for a culvert replacement option.

Provided that all the unsuitable soils are removed, and where necessary replaced with suitable granular soils (where the grade needs to be raised after sub-excavation, e.g. possibly at Borehole C3-2 location), there should be no problems with bearing resistance and settlements, since there will virtually be no load increases over and above the existing (i.e. no widening and only up to 100 mm grade raise of the road). However, for completeness the following geotechnical resistances can be assumed for the subgrade soils.

Bearing Resistance at U.L.S.	=	120 kPa
Factored Geotechnical Resistance at S.L.S.	=	60 kPa

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

The bedding material should be placed as soon as practicable after the preparation of the subgrade, as discussed, its inspection and approval. The bedding should be in accordance with the appropriate standards (e.g. OPSD-802.010 and 802.014) for flexible pipes or OPSD-802.030, 031, 032 or 034 (for rigid pipes) and should consist of not less than 250 mm thick layer (after compaction) of approved granular material, such as Granular ‘B’ Type II or Granular ‘A’ (Granular ‘B’ Type II is preferred under the pipe.) Under the pipe, the thickness of the bedding material may need to be increased to suit the site conditions at the time of construction. The bedding material should be compacted to at least 95% of the material’s Standard Proctor Maximum Dry Density (SPMDD) using a suitably light compactor to ensure that the underlying subgrade is undisturbed. If the bedding is to consist of a poorly graded material such as clear crushed stone, a suitable geotextile should be placed as a separator at sides of the excavation, as well as the top of the bedding material. However, the use of a poorly graded bedding is not recommended for this project.

The recommended thickness of bedding material beneath the pipe is 250 mm.

The bedding and embedment material should be extended along the sides to cover the pipe. The selection and placing of the backfill should be in accordance with OPSD-802.010 and OPSD-802.014 for flexible pipes or appropriate standards for rigid pipes. The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular ‘A’ or ‘B’ (OPSS-1010). All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and should be compacted to at least 96% of the material’s SPMDD. The Granular ‘A’ base and the Granular ‘B’ sub-base courses should be compacted to 100% of the SPMDD. The fill should be placed simultaneously on each side of the pipe to prevent lateral dislocation of the pipe. Uplift of the pipe must be prevented by means of dewatering and/or placing sufficient fill above it.

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

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

5.3 CONSTRUCTION

As mentioned before, the existing pipe will be rehabilitated by means of relining. During the relining process the water in the existing watercourse will need to be properly diverted and/or pumped. As well, sufficiently dry conditions will need to be maintained by dewatering, if necessary, to provide construction access. We recommend that the Contractor be required to provide their plan of action in this regard to the CA, for information purposes. While culvert replacement is not being planned, the following paragraphs are provided for the sake of completeness.

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

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

Depending on the groundwater level encountered at the time of the construction, some form of dewatering will likely be required to facilitate the construction and to preserve the load carrying capability of the founding soils.

As was mentioned before, the silty soils encountered at the site are dilatent soil types (e.g. silty fine sand in Borehole C3-3, silty sand till in Boreholes C3-2 and C3-3 and particularly the silt in Boreholes C3-1 and C3-2). These soils would be disturbed and dilate in the presence of water, a condition which can be recognized by the jelly-like, liverish appearance of the soil. By means of dewatering such a condition must be prevented. It is believed that after diverting the flow of water to the temporary culvert, the site can be dewatered by means of gravity drainage and pumping from strategically placed filtered sumps. In designing the dewatering system, the presence of the relatively impervious clayey silt to silty clay layer contacted in Borehole C3-1 as well as the presence of the bedrock at shallow depths should be taken into consideration.

We recommend that the contractor be made aware of dewatering requirements to facilitate the construction. In this respect, the contractor may choose to dig some test pits to investigate conditions at the time of construction and the necessity for dewatering, and the methods that may be required for this purpose.

If the pipe is placed on disturbed, dilated soil, excessive settlements can occur after backfilling. For this reason and to minimize dewatering, we recommend that if at all possible, the construction be carried out during a dry period. As well, care should be taken to avoid disturbing subgrade soils by minimizing construction traffic (including foot traffic) and minimizing vibrations. As well, stripping should be carried out under geotechnical supervision to acceptable subgrade level and the bedding material and/or soil to raise the grade should be placed immediately after exposing the suitable subgrade, its inspection and approval. We recommend that the material placed above the approved subgrade to raise the grade and/or as a bedding consist of Granular ‘B’ Type II material. Where the subgrade is relatively weak, we recommend that the Granular ‘B’ Type II material be pushed into the inorganic subgrade, if necessary, in order to improve the subgrade to make it firmer. As well, where subgrade is relatively weak, the first lift of backfill may need to be up to 0.6 m thick.

The contractor should also be made aware of the possible presence of cobbles and boulders in the embankment fill and in the underlying overburden, as well the presence of rock fill in the make-up of the embankment.

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

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

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

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

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

Granular Pavement Fill	Type 3 soil
Embankment Rock Fill	Type 3 soil
Embankment Soil Fill	Type 3 soil above groundwater level
	Type 4 soil below groundwater level
Topsoil/Organic & Alluvial Soils	Type 4 soil
Natural Overburden Soils	Type 4 soil

Regardless of the classification given above, we recommend that side slopes above water level for temporary excavations with unsupported side slopes should be no steeper than

2H:1V. This can be steepened, if approved by the QEV, but no steeper than 1 1/4H:1V. Below water level (e.g. if the site was not properly dewatered), flatter side slopes would be required.

5.4 FROST PROTECTION

Design frost protection for the project area is 1.8 m. Therefore, a permanent soil cover of 1.8 m or its thermal equivalent of artificial insulation is required for frost protection of foundations. In case of riprap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

6. CLOSURE

We recommend that once the details of the culvert are finalized, our recommendations be reviewed for their specific applicability. The Limitations of Report, as quoted in Appendix G, are an integral part of this report.

SHAHEEN & PEAKER

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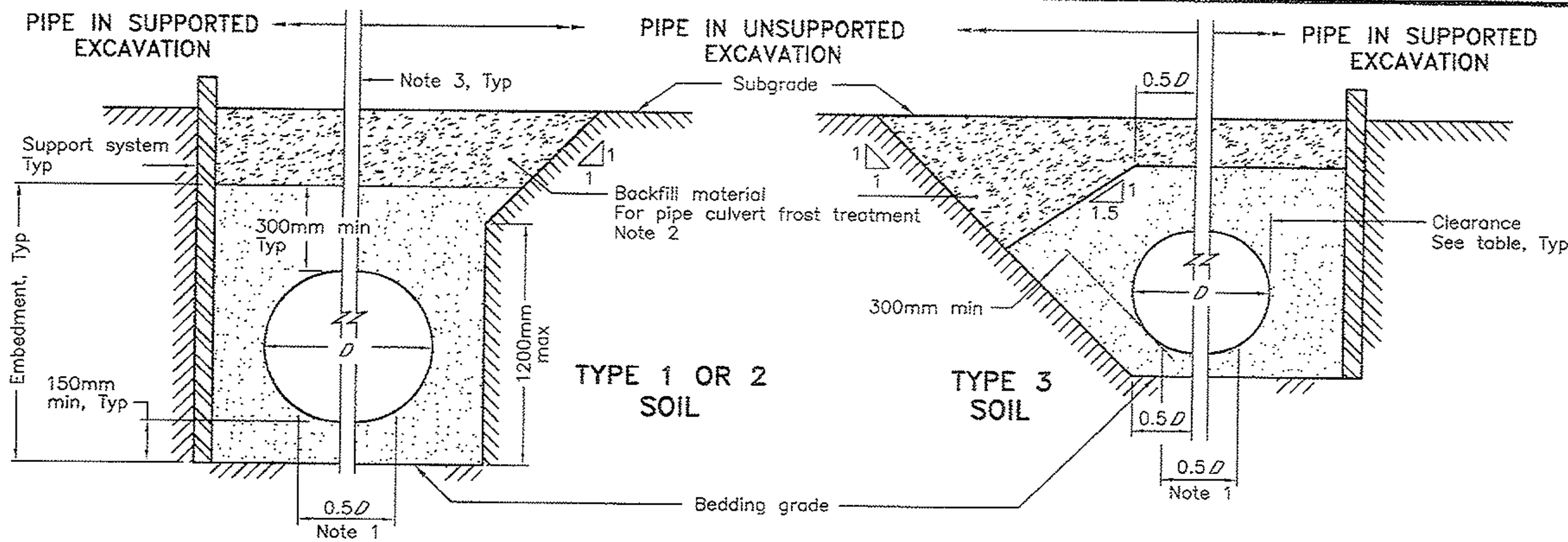
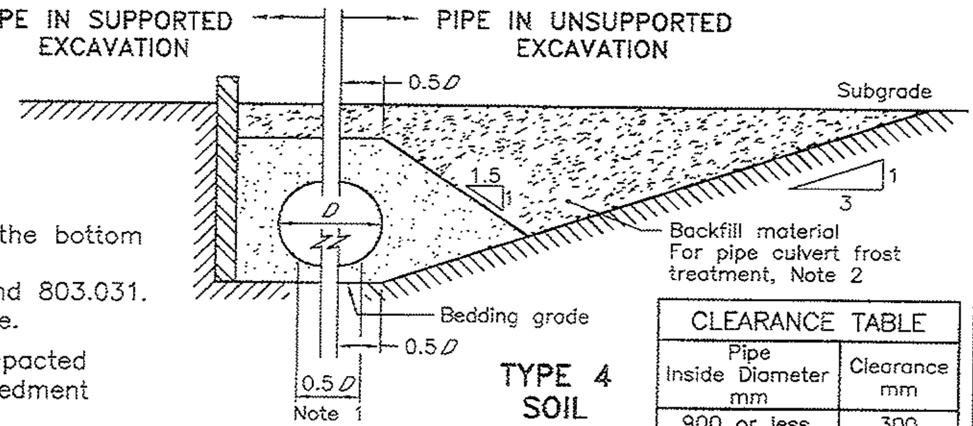
for K. R. Peaker, Ph.D, P. Eng.

Appendix F

OPSD





LEGEND:

D — Inside diameter

NOTES:

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

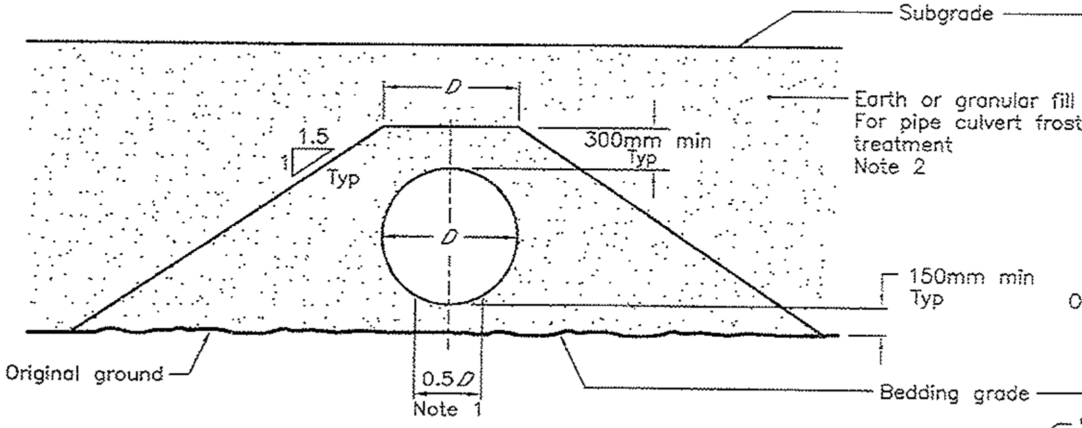
CLEARANCE TABLE

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

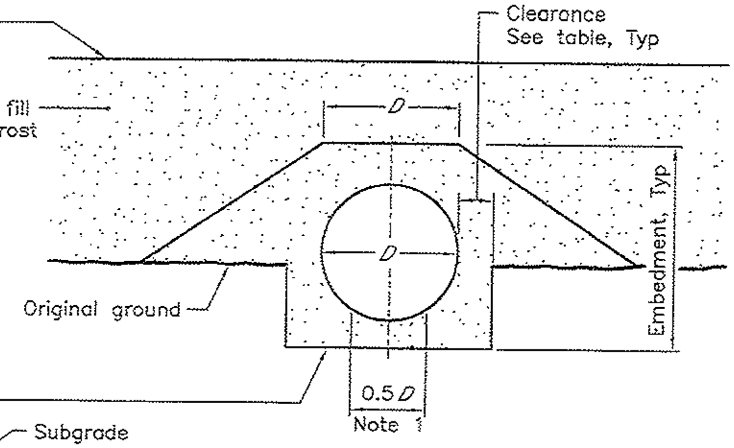
ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2005 | Rev 1

FLEXIBLE PIPE EMBEDMENT AND BACKFILL EARTH EXCAVATION

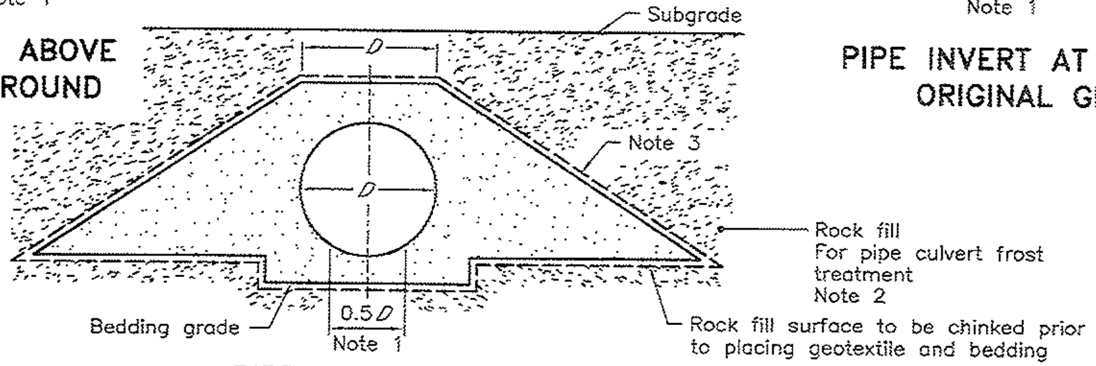
OPSD – 802.010



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.

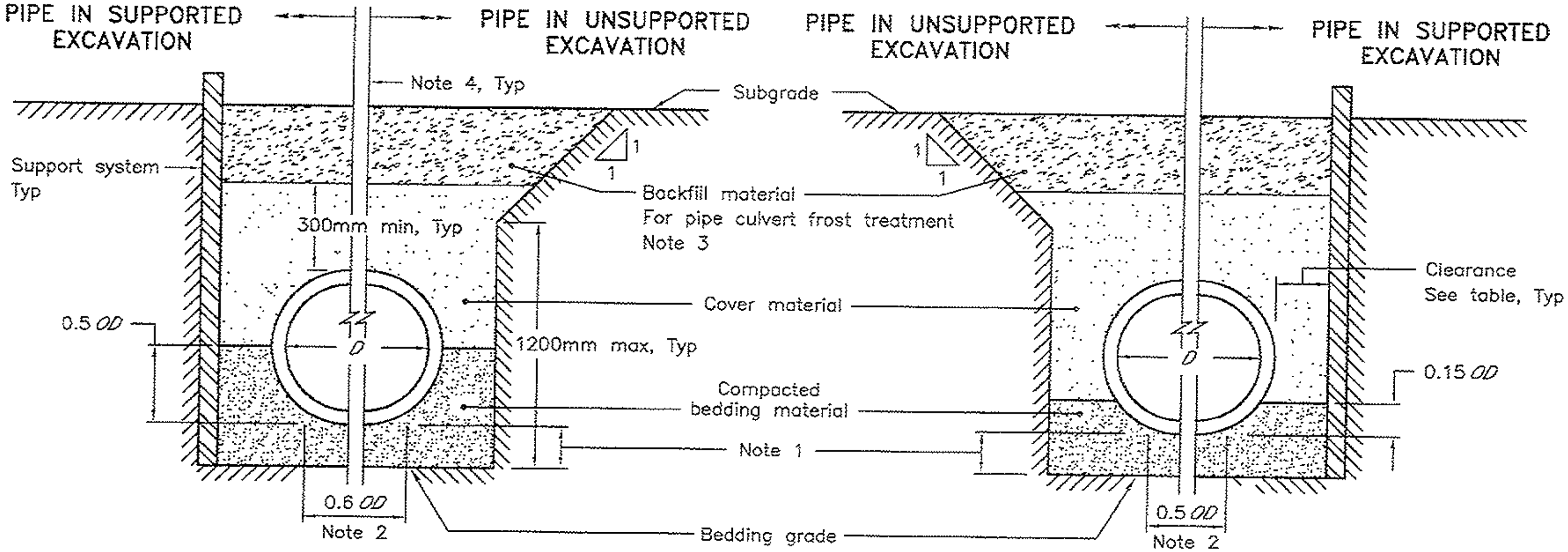
A Granular material placed in the haunch area shall be compacted prior to placing and compacting the remainder of the embedment material.

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	

ONTARIO STANDARDS MUNICIPAL–PROVINCIAL



CLASS B BEDDING

CLASS C BEDDING

NOTES:

1 The minimum bedding depth below the pipe shall be 0.15D. In no case shall this dimension be less than 150mm or greater than 300mm.
2 The pipe bed shall be compacted and shaped to receive the bottom of the pipe.
3 Pipe culvert frost treatment according to OPSD–803.030 and 803.031.
4 Condition of trench is symmetrical about centreline of pipe.

A Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.
B All dimensions are in metres unless otherwise shown.

LEGEND:

D – Inside diameter
OD – Outside diameter

CLEARANCE TABLE

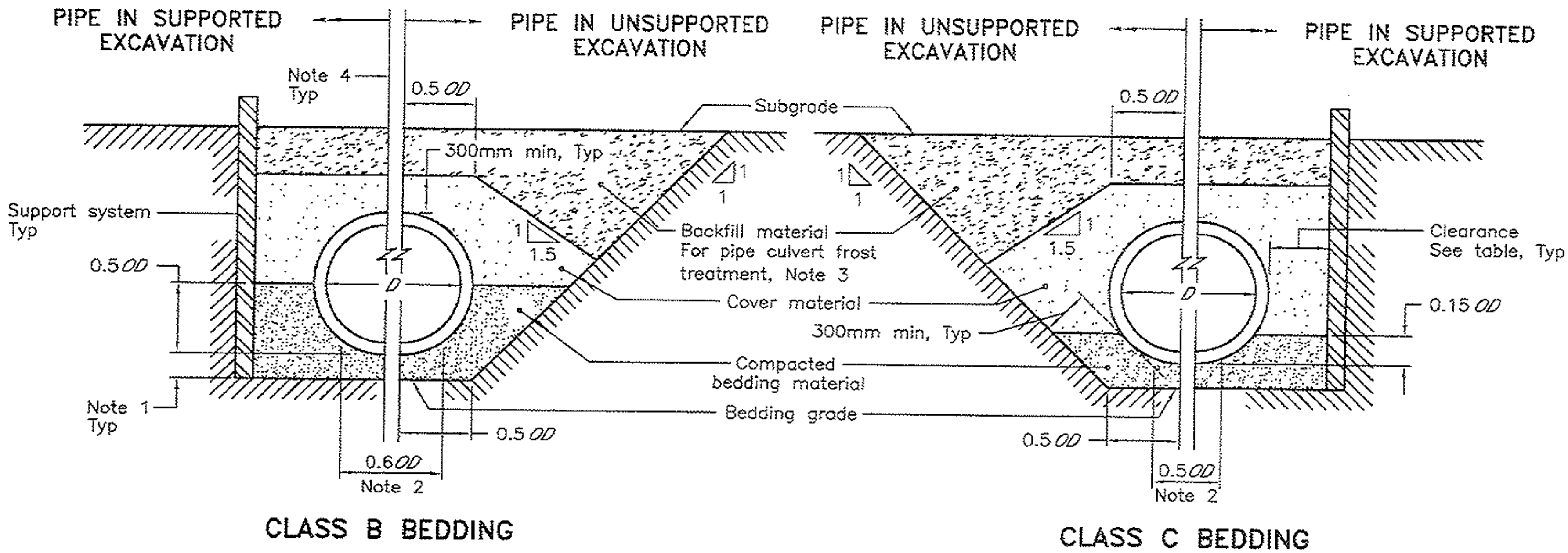
Pipe Inside Diameter mm	Clearance mm
900 or less	300
Over 900	500

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 | Rev 1

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

OPSD – 802.030



NOTES:

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

LEGEND:

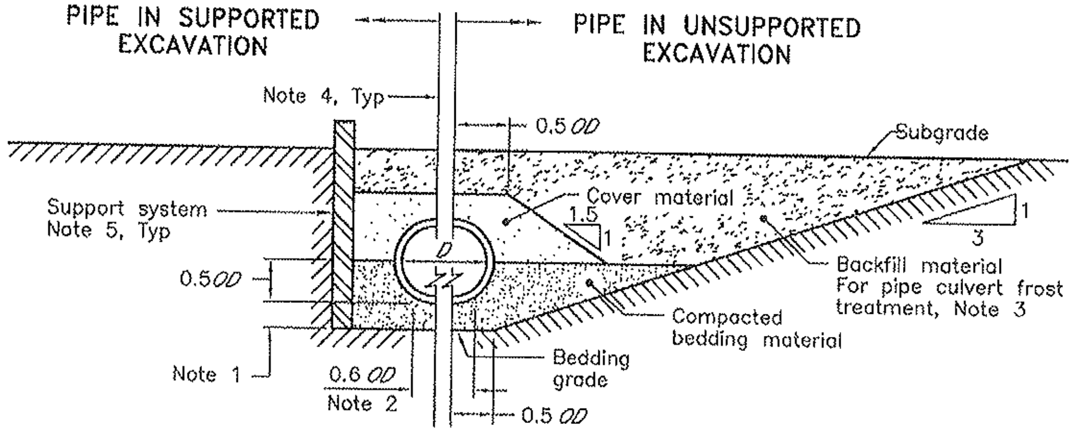
D – Inside diameter
OD – Outside diameter

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

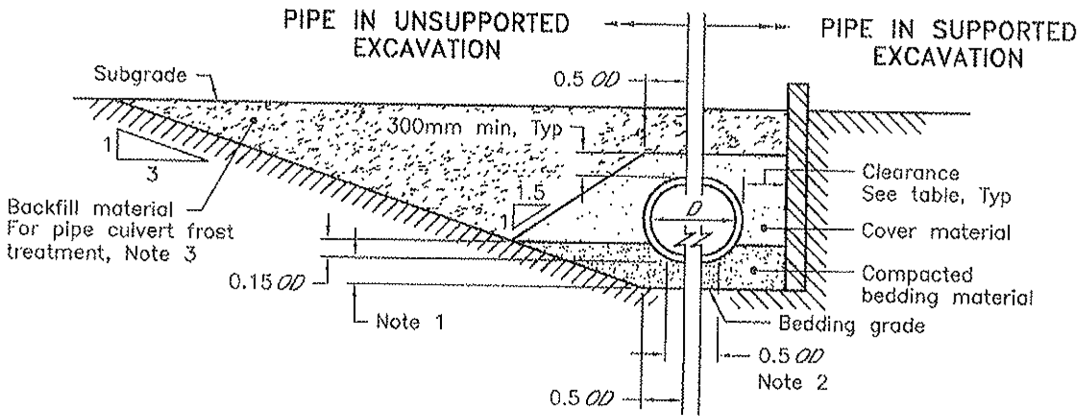
ONTARIO PROVINCIAL STANDARD DRAWING | Nov 2005 | Rev 1

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

OPSD – 802.031



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

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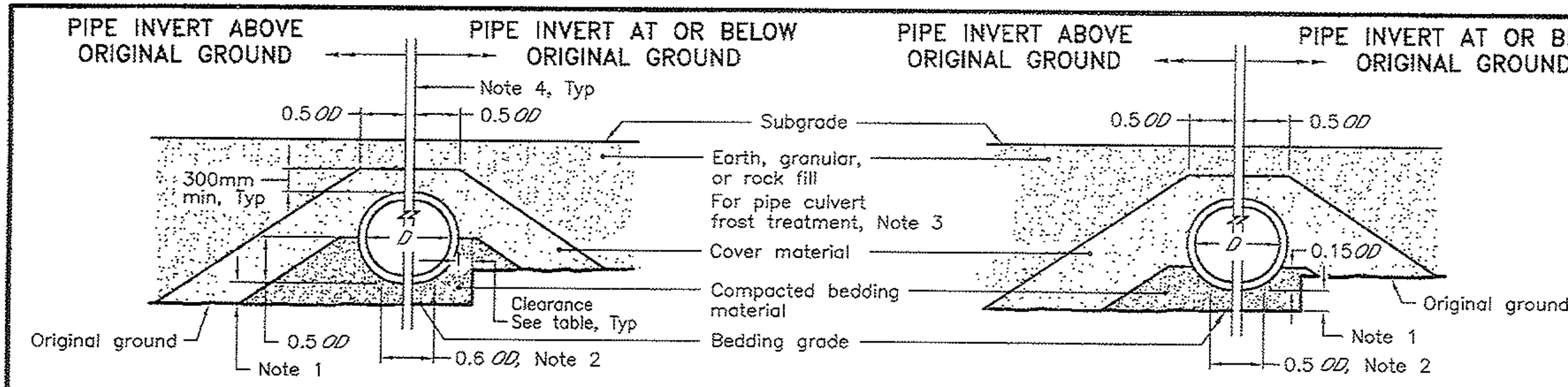
RIGID PIPE BEDDING, COVER, AND BACKFILL TYPE 4 SOIL – EARTH EXCAVATION

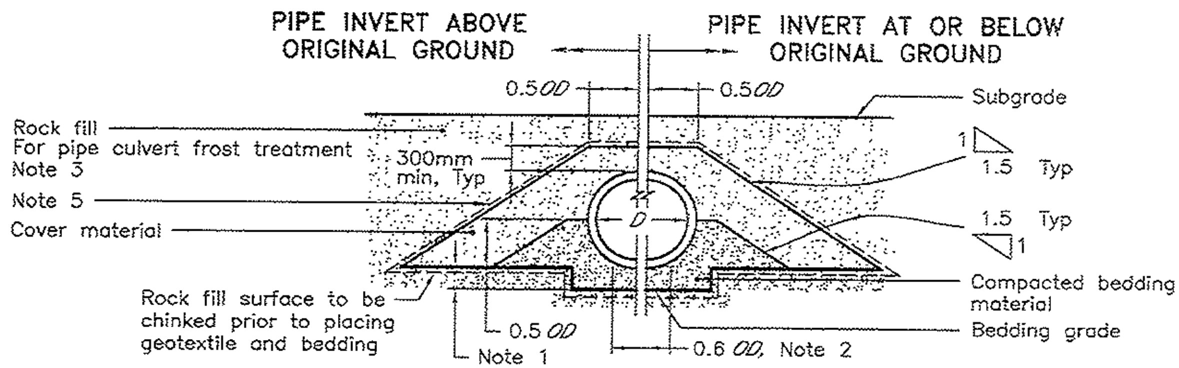
OPSD – 802.032



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.

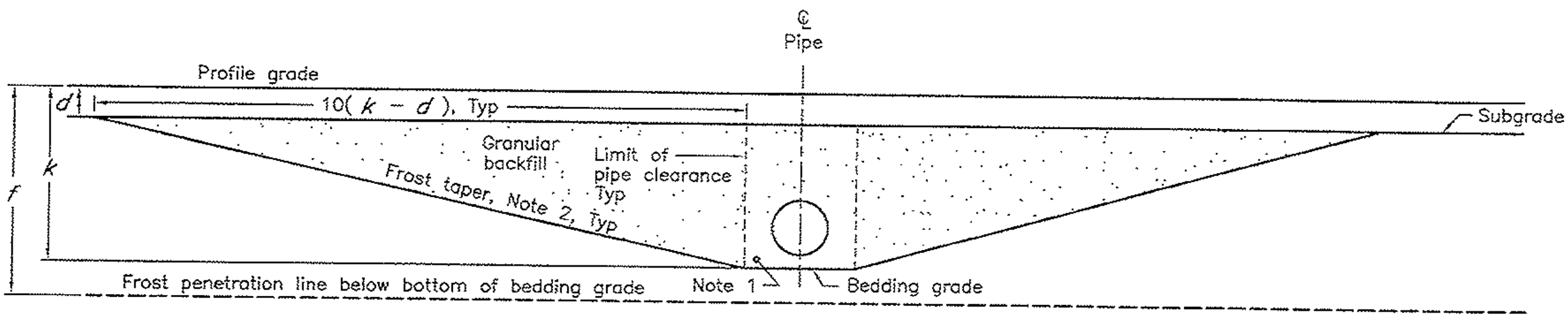
A All dimensions are in metres unless otherwise shown.

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.

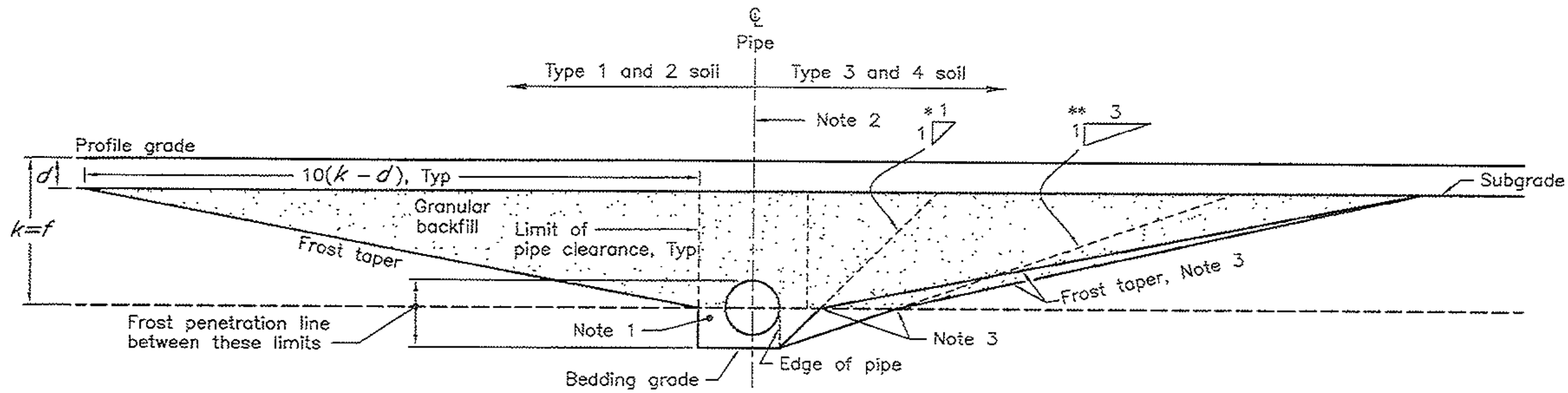
2 Frost tapers start at bedding grade.

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.

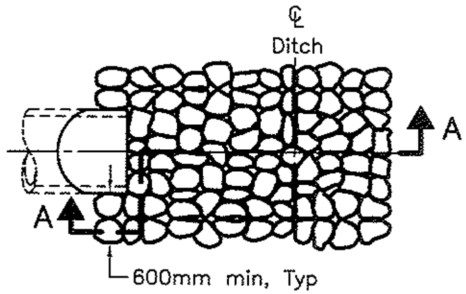
B Frost tapers not required when frost line is above the top of pipe.

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

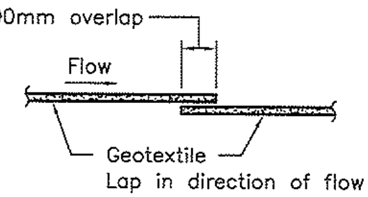
LEGEND:

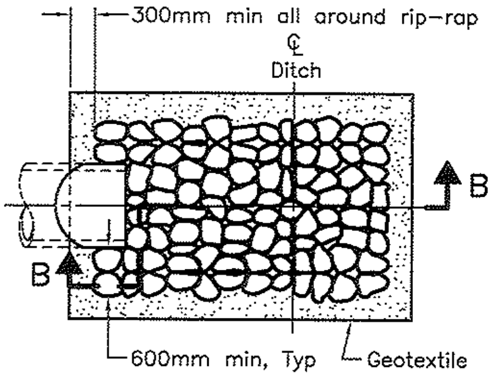
d – depth of roadbed granular
k – depth of frost treatment
f – depth of frost penetration
* – Type 3 soil
** – Type 4 soil

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	

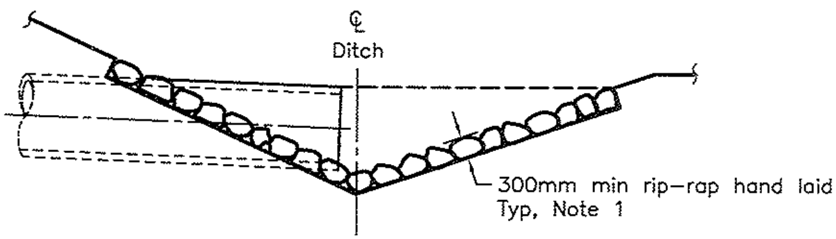


PLAN
CUT OR FILL

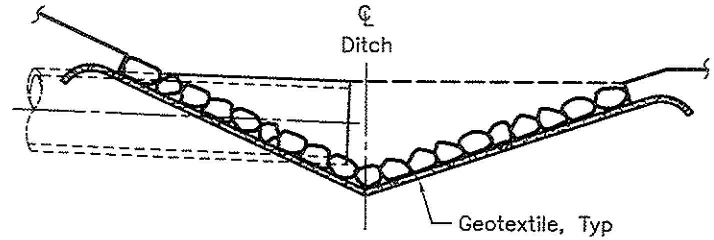




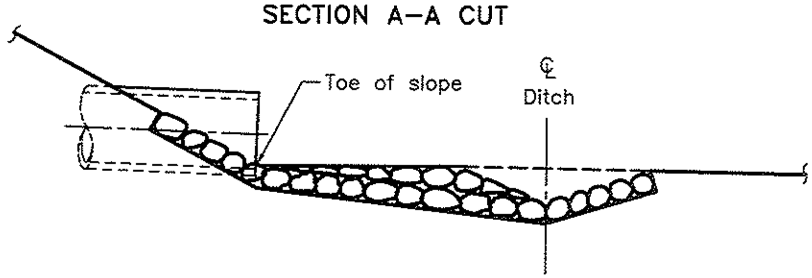
PLAN
CUT OR FILL



SECTION A–A CUT

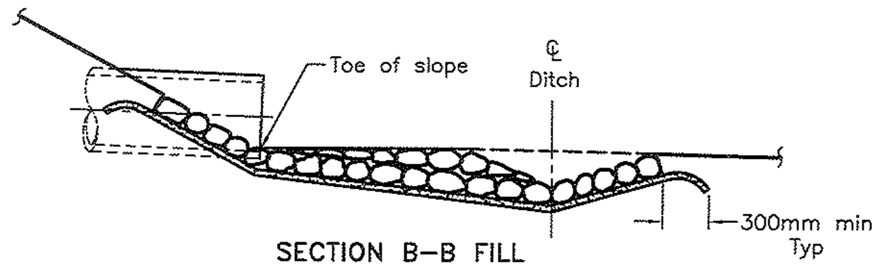


SECTION B–B CUT



SECTION A–A FILL

TYPE A – WITHOUT GEOTEXTILE



SECTION B–B FILL

TYPE B – WITH GEOTEXTILE

NOTES:

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

A All dimensions are in millimetres unless otherwise shown.

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

ONTARIO STANDARDS · MUNICIPAL – PROVINCIAL

Appendix G

Limitations of Report

LIMITATIONS OF REPORT

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

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

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

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

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