

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
BLACK CREEK CULVERT (C4)
REPLACEMENT AT STATION 21+570,
HIGHWAY 522, MUNICIPAL TOWNSHIP OF
NIISSING, ONTARIO, SITE NO. 44-274
GEOCRES NO. 31E-285, G.W.P. 484-98-00**

D. M. Wills Associates Limited

Project: SPT1221D
May, 2009

FINAL REPORT

**FOUNDATION INVESTIGATION
BLACK CREEK CULVERT (C4)
REPLACEMENT AT STATION 21+570,
HIGHWAY 522,
MUNICIPAL TOWNSHIP OF
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FINAL REPORT

May 11, 2009

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

Attention: Mr. Michael Lang, P.Eng.

Dear Sirs:

**RE: Foundation Investigation and Design Reports, Black Creek Culvert (C4) Replacement at
Station 21+570 Highway 522, Municipal Township of Nipissing, Ontario, Site No. 44+274,
GEOCRETS No. 31E-285, G.W.P. 484-98-00**

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

For and on behalf of Coffey Geotechnics Inc.



Ramon Miranda, P.Eng.
Manager, Transportation Division

Attachment A: Attachments

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**FOUNDATION INVESTIGATION
BLACK CREEK CULVERT (C4) REPLACEMENT AT STATION 21+570
HIGHWAY 522, MUNICIPAL TOWNSHIP OF NIPISSING, ONTARIO
SITE NO. 44-274
G.W.P. 484-98-00**

1 INTRODUCTION

Rehabilitation and resurfacing of Highway 522 from 0.6 km west of Highway 522B in Trout Creek, westerly for 19.7 km, will entail the rehabilitation of several existing culverts, one of which is the Black Creek culvert. The Black Creek structural culvert is located on Highway 522, approximately 8 km west of Highway 11, in the Geographical Township of Gurd, Municipal Township of Nipissing in the MTO Sudbury Area.

Coffey Geotechnics Inc. (Coffey), formerly Shaheen & Peaker Limited, was retained by D. M. Wills Associates Limited to carry out a foundation investigation at the site of the proposed rehabilitation of the existing Black Creek culvert (C4) under Highway 522 at Station 21+570. The structure provides for water flow in the Black Creek beneath Highway 522.

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 Black Creek Culvert is located on Highway 522 about 8 km west of the Trout Creek in the Township of Gurd.

Trout Creek is located about 45 km south of North Bay on Highway 11 at the junction with Highway 522. 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, this project site is located within the Physiographic Regions known as the Algonquin Highlands and the Number 11 Strip (i.e. along Highway 11).

The Quaternary deposits found in this area are quite complex, having resulted from a variety of geological process associated with glacial, glaciofluvial and glaciolacustrine conditions. Highway 11 from Gravenhurst to North Bay follows a narrow strip in which sand, silt and clay deposits occupy the hollows. A large proportion of the area consists of bare bedrock with thin drift but thicker accumulations of sediment are also expected locally. Much of the area is underlain by the Kiosk domain which is characterized by alternating zones of amphibolites facies gneiss and hypersthene-bearing granulite developed from highly flattened plutonic rocks.

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

3 FIELD AND LABORATORY WORK

The fieldwork for this project was performed on August 1, 5, 6, 7, September 9 and 16, 2008 and consisted of drilling and sampling five boreholes (FC4-1, FC4-2, FC4-3, FC4-RP1 and FC4-RP2) to depths ranging from 3.0 to 16.3 m below the ground surface. Three of the boreholes (FC4-1, FC4-2 and FC4-3) were advanced adjacent to the existing culvert to depths of 3.0 to 16.3 m below existing grades. Two boreholes (FC4-RP1 and FC4-RP2) were advanced some 10 m west and 17 m east of the existing culvert, to 11.0 and 10.1 m below the existing ground surface. The locations of the boreholes are shown on the Borehole Location Plan Drawing No. 1.

Four of the boreholes were advanced using a track-mounted drilling rig, owned and operated by Landcore Drilling of Chelmsford, Ontario, to depths of 10.1 to 16.3 m below the ground surface, while one borehole (Borehole FC4-3) was put down by manual (i.e. hand drilling) methods, due to the fact that it was not possible to access the toe of the embankment with a mechanized drill rig mounted on a motorized vehicle. This latter borehole was advanced by hand drilling to 3.0 m below the original ground (o.g.) level, and as well, a modified Dynamic Cone Penetration Test was performed below this depth to refusal at a depth of 8.6 m below o.g. level.

Within the overburden the boreholes were advanced by augering but in some cases, where refusal to further augering was encountered due to the presence of cobbles or boulders, the boreholes were further advanced by wash boring with N-type casing. The bedrock was cored at two locations by NQ rock coring method.

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

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

As mentioned before, Borehole FC4-3 was advanced by manual methods to a depth of 3.0 m. This consisted of driving a standard split-spoon sampler into the ground by means of a 70-lb (31.8 kg) hammer dropping a vertical distance of 0.76 m, similar to a Standard Penetration test (SPT). However, since the hammer weight is one half of the weight of a standard (i.e. 140 lb or 63.6 kg) hammer, the number of blows required to drive the sampler into the ground by 0.30 m was divided by two, giving an approximate N-value. After driving the sampler into the ground by 0.6 m, the sampler was retrieved and the sample inside the sampler was examined and labelled after which the sampler was put back in the hole. The hole was

advanced in this fashion to 3.0 m depth below which a modified Dynamic Cone Penetration test was performed from below the hole to 8.6 m below the o.g. level.

Dynamic Cone Penetration Tests (DCPT) were performed adjacent to Boreholes FC4-1 and FC4-2, to refusal at depths of 8.9 m and 11.9 m, respectively. 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 affect the SPT results.

In Borehole FC4-3 a modified DCPT was performed from below the borehole at 3.0 m to a depth of 8.6 m below the ground surface. As mentioned before in this test a 70-lb (31.86 kg) hammer was used instead of the standard hammer. The recorded number of blows were therefore divided by two to arrive at approximate penetration resistances similar to the standard DCPT.

The borehole locations were established in the field by Coffey engineering staff, in relation to the existing features. The borehole geodetic elevations were determined and provided by the client.

Water level observations in the open boreholes were made during the drilling and at completion of each borehole. A piezometer was installed in Borehole FC4-1 to monitor long-term groundwater level.

Upon their completion, the open boreholes were backfilled with a mixture of auger cuttings and hole plug, as per MTO procedures.

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

4 SUMMARIZED SUBSURFACE CONDITIONS

The existing top of highway embankment at the borehole locations has an elevation 328.7 to 329.2 m at the project location, while the ground surface elevation at the borehole advanced beyond the embankment is 326.9 m.

Boreholes drilled from the top of the embankment contacted an embankment fill which extends to depths ranging from 2.3 to 3.0 m below the ground surface or to Elevations 326.4 to 326.0 m. The embankment fill and the alluvial soils in Borehole FC4-3 (drilled from the toe of the embankment from the o.g. level) is underlain by organic or partly organic soils. The thickness of these soils range from 0.3 to 1.9 m and they extend to El. 325.6-324.3 m. Below the organic soils, surficial sand layers were contacted in three of the boreholes. Underlying these sand layers or the organic soils, the boreholes contacted a 3.4 to 5.3 m thick silty clay to clayey silt/silt deposit which extends to depths 6.7 to 10.6 m below the embankment surface or to El. 322.1 to 318.5 m. This basically cohesive deposit is underlain by granular deposits to the surface of the bedrock which was encountered in two of the boreholes at depths of 9.3 and 13.3 m below the embankment surface or at El. 319.4 and 315.8 m.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. An inferred stratigraphic profile and an inferred stratigraphic section are shown in Drawings No. 1 and 2. The following description of the individual soil strata is to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site. It should be noted that the soil and groundwater conditions may vary in between and beyond borehole locations.

4.1 Fill

4.1.1 Granular Pavement Fill

Boreholes FC4-1, FC4-2, FC4-RP1 and FC4-RP2, which were advanced from the top of the road shoulders, contacted a granular pavement fill (typically sand & gravel) which ranges in thickness from 0.4 to 0.6 m. Standard Penetration tests performed in the granular pavement fill ranged from 4 to 12 blows/0.3 m which indicate a very loose to compact but typically loose relative density.

4.1.2 Embankment Fill

Underlying the pavement fill, all four boreholes drilled from the top of the highway embankment contacted a embankment fill which extends to depths of 2.3 to 3.0 m below the surface of the road shoulder or to El. 326.4-326.0 m. The embankment fill is a granular (i.e. non-cohesive) soil type and its composition ranges from gravelly sand to sand with some silt and traces of gravel. Some organic mixture in the fill was noted near the bottom. As well, a sandy silt layer with some gravel, and traces of organics & slag was also noted near the bottom of the deposit.

The grain-size distribution of four samples from the embankment fill is presented in Figure B-1 in Appendix B. These indicate the following grain-size distribution.

| | |
|--------------|--------|
| Gravel: | 1-38% |
| Sand: | 54-86% |
| Silt & Clay: | 8-13% |

N-values recorded in the embankment fill range from 4 to 15 blows/0.3 m which indicates a very loose to compact but typically a compact condition.

4.1.3 Alluvial Granular Soils

In Borehole FC4-3, which was drilled from the o.g. level near the culvert location (on the north side of the roadway), alluvial deposits (i.e. river fill) were contacted immediately below the ground surface. These granular soils consisted of sand with traces of gravel, silt and organics to 1.2 m (El. 325.7 m) underlain by a sand & gravel layer to 1.8 m (El. 325.1 m).

Standard Penetration tests yielded N-values of 1 and 3 in the upper alluvial sand deposit, indicating a very loose condition while in the underlying sand & gravel an N-value of 12 blows/0.3 m was recorded which indicates a compact relative density.

4.2 Organic Soils

Underlying the embankment fill or the river fill (Borehole FC4-3), the boreholes contacted organic or somewhat organic soils, which range from non-cohesive (granular) to cohesive materials. These deposits were found to be 0.3 m (Borehole FC4-3) to 1.9 m (Borehole FC4-RP1) thick and extended to El. 325.6 m (Borehole FC4-RP2) to 324.3 m (Borehole FC4-RP1).

N-values recorded in these deposits range from 1 to 10 blows/0.3 m (typically 1-2 blows/0.3 m) indicating typically very loose relative density (where the soil is primarily fine-grained granular) to very soft consistency (where the soil is cohesive).

Natural moisture contents recorded in these organic or somewhat organic soils range from about 30 to 112%.

4.3 Upper Sand

Granular soil layers were contacted underlying organic soils in Boreholes FC4-2, FC4-3 and FC4-RP1 at depths of 4.2 m (El. 324.9 m), 2.1 m (El. 324.8 m) and 4.9 m (El. 324.3 m), respectively. In Boreholes FC4-2, FC4-3 and FC4-RP1, the thickness of these surficial granular soils were recorded to be 1.1 m, 0.3 m and 0.4 m, respectively.

This surficial granular deposit contain occasional organic silt interbeds and decayed wood pieces (Borehole FC4-2).

The grain-size distribution of a sample recovered from the sand layer in Borehole FC4-2 is presented in Figure B-2 in Appendix B. This indicates 92% sand and 8% silt & clay size particles.

From the SPT values recorded in these surficial granular soils range from 1 to 14 blows/0.3 m which indicate a very loose to compact relative density.

4.4 Silty Clay

The organic and/or the surficial granular soils in Boreholes FC4-1, FC4-2, FC4-RP1 and FC4-RP2 are underlain at depths 3.2 m (El. 325.6 m) to 5.3 m (El. 323.8-323.9 m) below the ground surface by a silty clay deposit with frequent clayey silt and occasional silt interbeds. A distinct silt zone was also found in Borehole FC4-RP1 near the bottom of this cohesive deposit. Borehole FC4-3 was terminated at a depth of 3.0 m below the o.g. level and as such did not contact this cohesive deposit. The thickness of this cohesive material at the borehole locations was found to range from 3.4 to 5.3 m and the unit extended to depths of 6.7 m to 10.6 m below the ground surface or to El. 322.9 to 318.5 m.

The grain-size distribution of three samples from the deposit is given in Figure B-3 in Appendix B, which indicates the following gradation.

| | |
|---------|--------|
| Gravel: | 0% |
| Sand: | 1-4% |
| Silt: | 43-56% |
| Clay: | 42-54% |

The results of Atterberg Limits tests performed on samples recovered from the deposit (seven samples) are given on the individual Record of Borehole Sheets and also on a plasticity chart in Figure B-4 in Appendix B. The following index values were obtained:

| | |
|-------------------|--------|
| Liquid Limit: | 28-37% |
| Plastic Limit: | 18-21% |
| Plasticity Index: | 9-16% |

These results are characteristic of clayey soils of low plasticity (i.e. typically CL material). The recorded natural moisture contents are in excess of the recorded liquid limits which indicate a normally consolidated or a slightly over-consolidated, weak and compressive material.

Standard Penetration tests performed in this deposit generally yielded N-values which are typically 1 blow/0.3 m with occasional higher values in the upper and lower zones (i.e. more silty zones).

Field vane tests performed in-situ gave undrained shear strengths of between 24 and 48 kPa. Based on these field tests the deposit is described as soft to firm.

4.5 Gravelly Sand

Underlying the silty clay deposit the deeper boreholes contacted at depths of 6.7 to 10.6 m below the ground surface (or at El. 322.1-318.5 m) a coarse grained granular soil, the grain-size distribution of which ranges from sand to gravel & sand, but typically gravelly sand. In Boreholes FC4-1 and FC4-2 refusal to augering was contacted after penetrating this deposit to depth of 8.4 and 11.9 m or at Elevations of 320.3 and 317.2 m, respectively. The boreholes were further advanced by coring through a layer of cobbles and boulders or a very highly fractured rock. Borehole FC4-3 contacted a gravelly sand at a depth of 2.4 m (El. 324.5 m) underlying the upper sand and borehole was terminated within this deposit at a depth of 3.0 m (El. 323.9 m) after penetrating the deposit by 0.9 m. Boreholes FC4-RP1 and FC4-RP2 encountered practical refusal to augering at 11.0 m (El. 318.2 m) and 10.1 m (El. 318.7 m), respectively, where the boreholes were terminated possibly on boulders or highly weathered bedrock.

The grain-size distribution of three samples from this basal coarse grained granular soil (i.e. typically gravelly sand) is given in Figure B-5. The results indicate the following grain-size distribution.

| | |
|--------------|--------|
| Gravel: | 25-50% |
| Sand: | 44-56% |
| Silt & clay: | 6-21% |

The presence of cobbles and boulders can be expected in this deposit.

N-values recorded on this basal granular deposit range from 9 to 46 blows/0.3 m. Based on these and DCPT results the relative density of the deposit is described as loose to dense but typically compact to dense.

4.6 Cobbles and Boulders

As mentioned in the preceding section, Boreholes FC4-1 and FC4-2 encountered practical refusal to further augering at depths of 8.4 m (El. 320.3 m) and 11.9 m (El. 317.2 m), respectively. The obstruction was cored and was found to consist of a series of cobbles and boulders or very highly fractured rock immediately above the bedrock at 9.3 m (El. 319.4 m) and 13.3 m (315.8 m), respectively in Boreholes FC4-1 and FC4-2.

4.7 Bedrock

In Boreholes FC4-1 and FC4-2 bedrock was proven by coring. In these two boreholes bedrock was contacted at depths 9.3 m (El. 319.4 m, Borehole FC4-1) and 13.3 m (El. 315.8 m, Borehole FC4-2) and cored for a vertical distance of 3.0 m.

From the cores recovered, the bedrock is believed to consist of a grey gneiss with some pinkish grey bands. The percentage of Total Core Recovery (TCR) was found to be 95 to 100% and RQD values ranged from 23 to 96%. Based on these, the rock within the upper 1.2 m (i.e. from El. 319.4 m to 318.2 m) in Borehole FC4-1 is described as extremely fractured and sound below. In Borehole FC4-2, a 0.4 m thick highly fractured rock zone was found between about 315.2 and 314.8 m and relatively sound above and below these elevations.

Rock core photographs are given in Appendix D.

4.8 Groundwater Conditions

Groundwater conditions in the open boreholes were observed during the drilling and upon completion of each borehole, as well as upon their completion. As well, a piezometer was installed in Borehole FC4-1 to enable us to monitor the groundwater levels over a prolonged period of time, without interference from surface water. The observations are shown on the individual Record of Borehole Sheets.

Water levels in the open boreholes ranged from 1.0 to 2.0 m below the ground surface upon completion or between El. 327.4 and 325.7 m. The water level in the piezometer installed in Borehole FC4-1 was recorded at 1.0 m below the ground surface or at El. 327.7 m.

Based on the observations made and the moisture contents of the soil samples, it is our opinion that the groundwater level at the time of our investigation was between 0.6 m below the o.g. level at Borehole FC4-3 and about 2.2 m below the top of the road shoulder at Borehole FC4-2, or between Elevations 327.7 m and 326.3 m.

For and on behalf of Coffey Geotechnics Inc.


Ramon Miranda, P.Eng.




Zuhtu Ozden, P.Eng.



Drawings

METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.

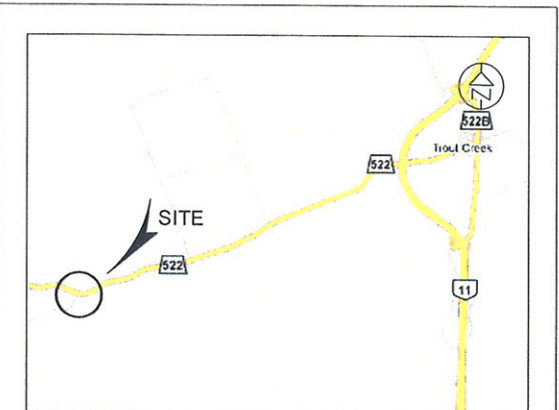
GWP: 484-98-00

HIGHWAY 522, BLACK CREEK
(STATION 21+570)
BH LOCATION PLAN & STRATIGRAPHY



SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

- Borehole & Cone
- 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 |
|---------|-------|-------------|-------------|
| FC4-1 | 328.7 | 21+576 | 5.0m Rt C/L |
| FC4-2 | 329.1 | 21+564 | 4.0m Lt C/L |
| FC4-3 | 326.9 | 21+575 | 9.5m Lt C/L |
| FC4-RP1 | 329.2 | 21+560 | 4.0m Rt C/L |
| FC4-RP2 | 328.8 | 21+587 | 4.5m Rt C/L |

NOTE

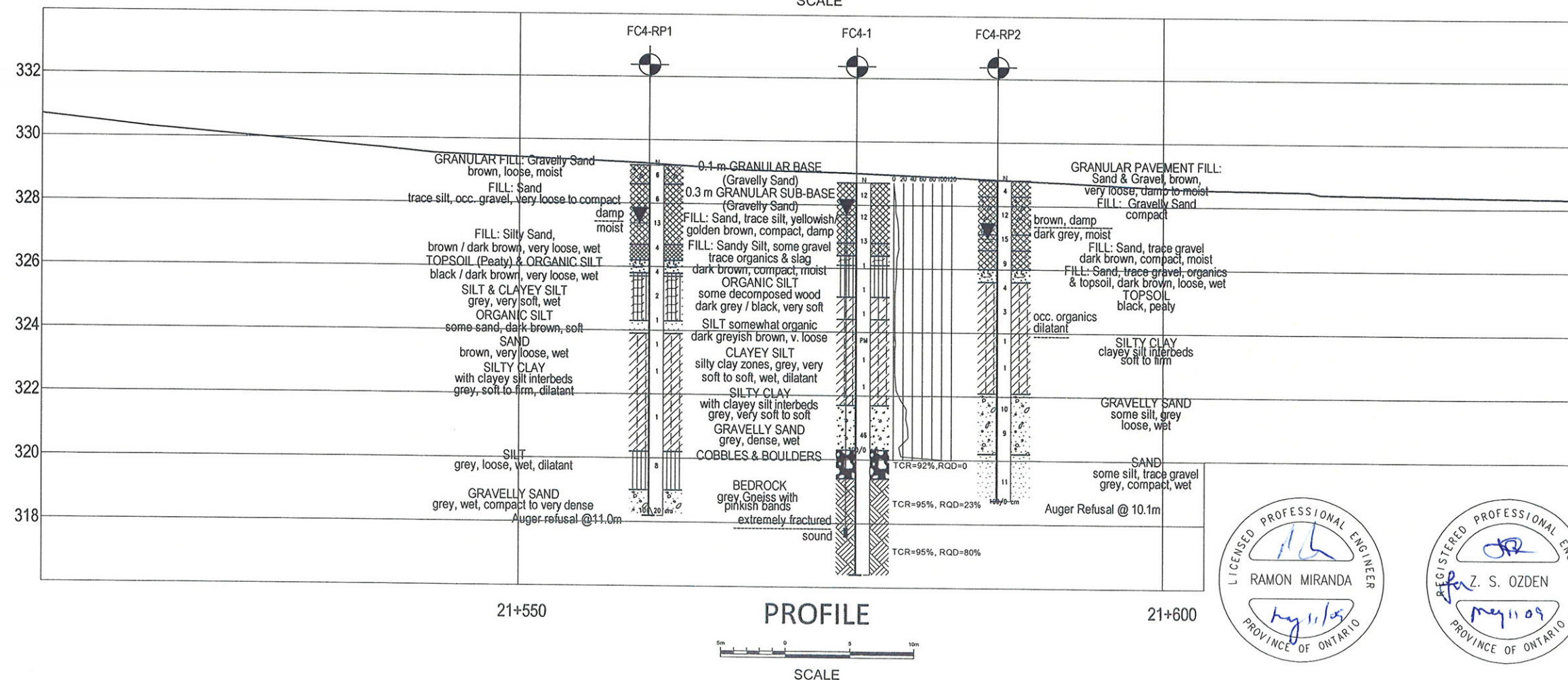
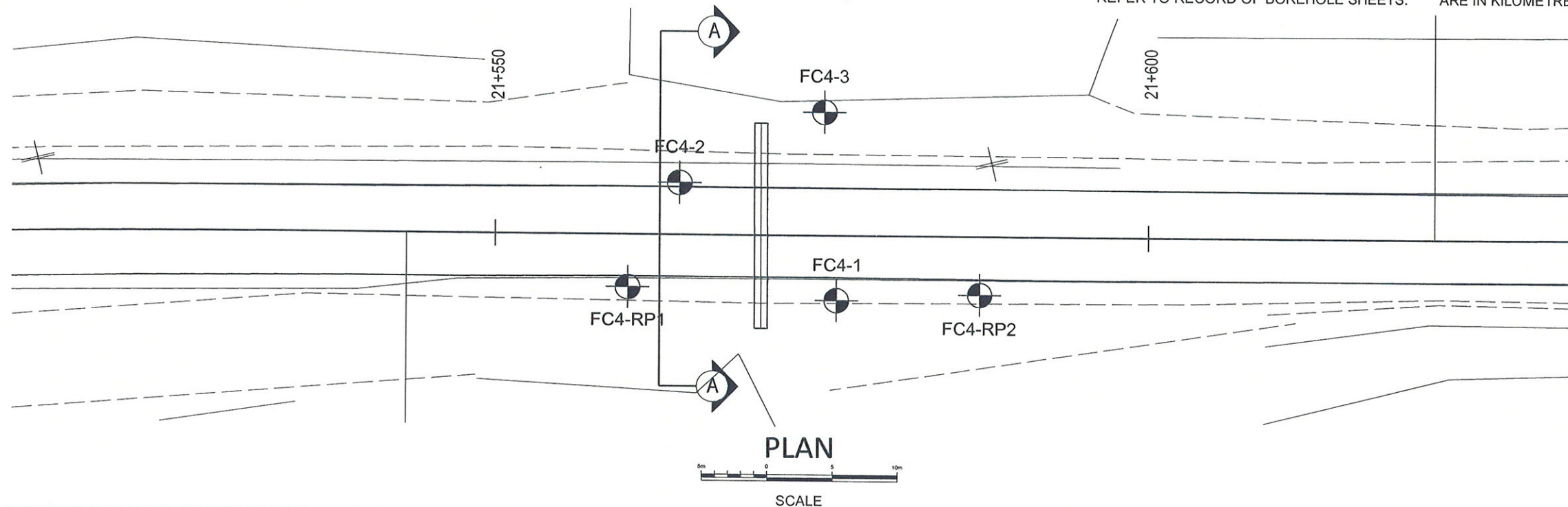
The boundaries between soil strata have been established only at Borehole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

| REV. | DATE | BY | DESCRIPTION |
|------|------|----|-------------|
| | | | |

Geocres No. 31E-285

| SPT 1221D | | | DIST |
|-----------|------------|----------------|-------|
| SUBM'D | CHECKED | DATE Dec. 2008 | SITE |
| DRAWN PHK | CHECKED RM | APPROVED ZO | DWG 1 |



METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

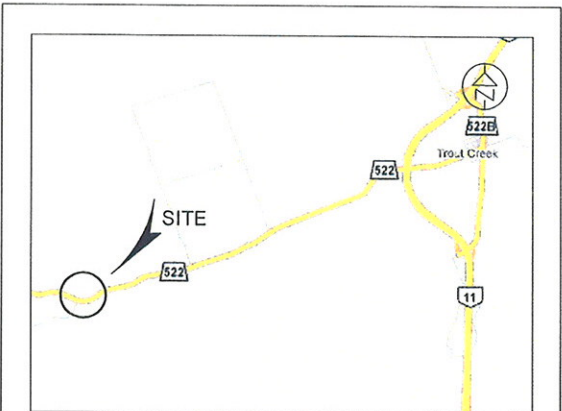
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No.

GWP: 484-98-00

HIGHWAY 522, BLACK CREEK
(STATION 21+570)
CROSS SECTION

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

- Borehole & Cone
- 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 |
|-------|-------|-------------|-------------|
| FC4-1 | 328.7 | 21+576 | 5.0m Rt C/L |
| FC4-2 | 329.1 | 21+564 | 4.0m Lt C/L |
| FC4-3 | 326.9 | 21+575 | 9.5m Lt C/L |

NOTE

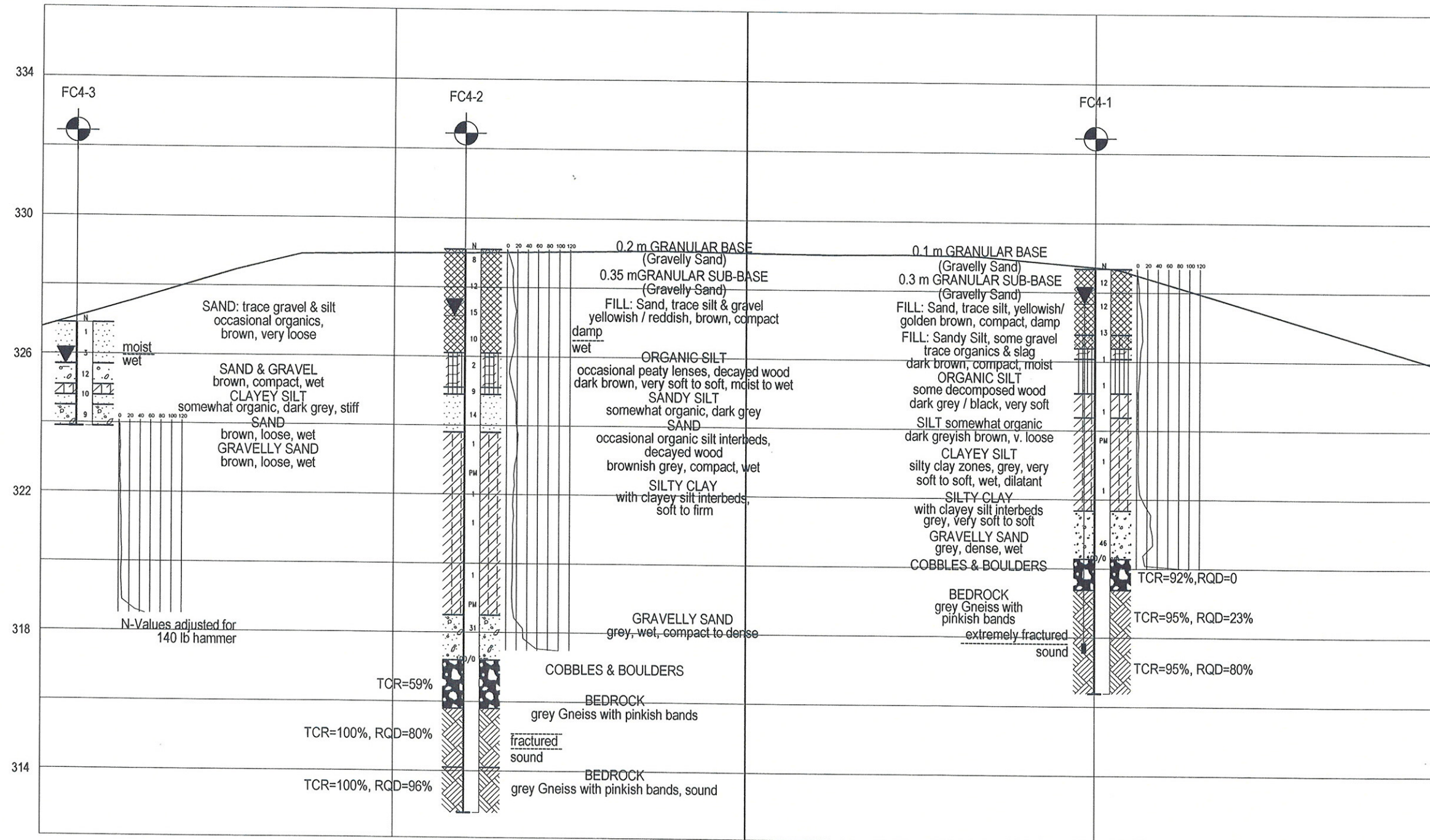
The boundaries between soil strata have been established only at Borehole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

| REV. | DATE | BY | DESCRIPTION |
|------|------|----|-------------|
| | | | |

Geocres No. 31E-285

| SPT 1221D | | | DIST |
|-----------|------------|----------------|-------|
| SUBM'D | CHECKED | DATE Jan. 2009 | SITE |
| DRAWN PHK | CHECKED RM | APPROVED ZO | DWG 2 |



SECTION A-A



Appendix A

Record of Borehole Sheets

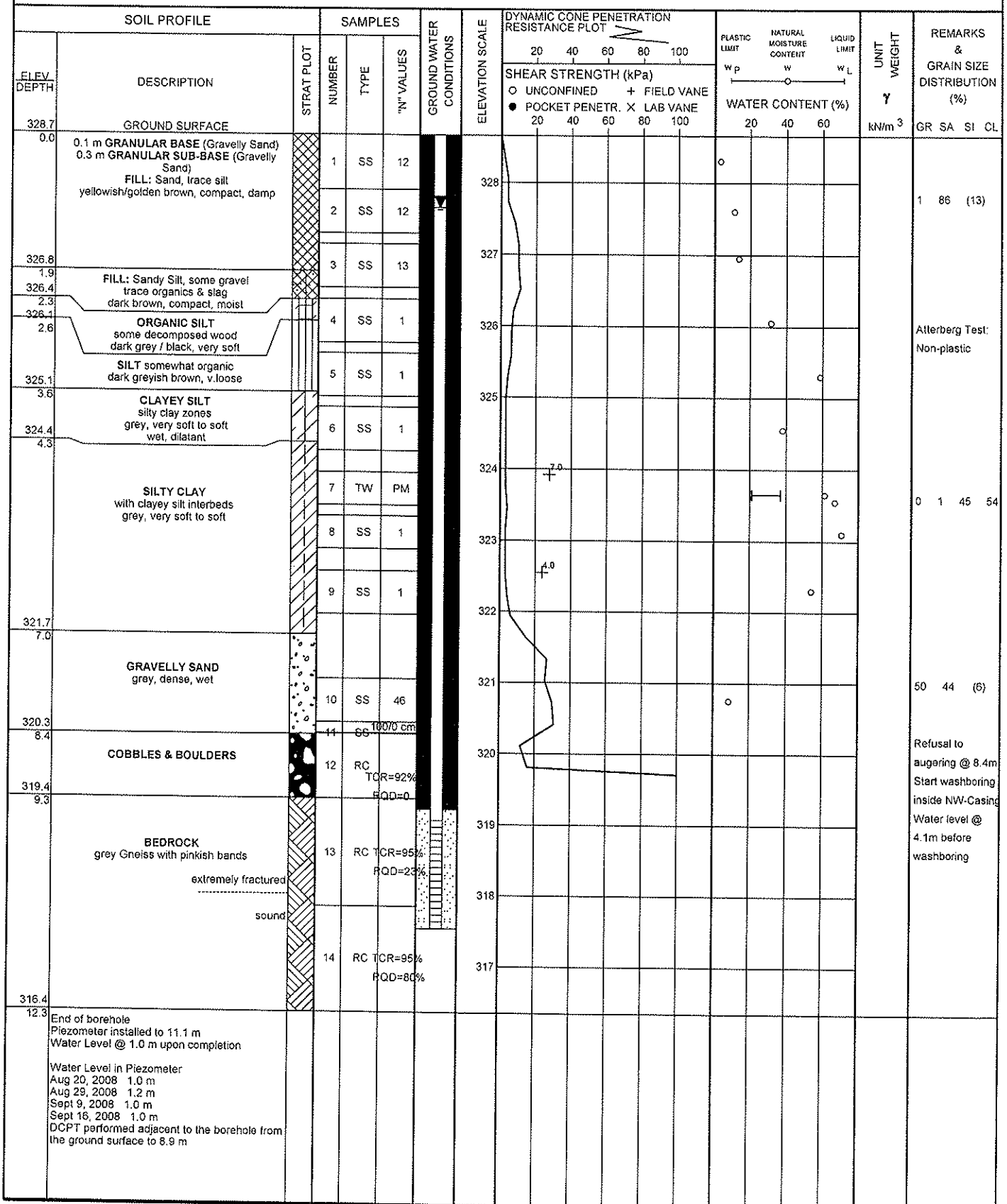
SPT 1221D

RECORD OF BOREHOLE No FC4-1

1 OF 1

METRIC

GWP 484-98-00 LOCATION Sta : 21+576, 5.0 m Rt C/L of Hwy 522 ORIGINATED BY SK
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger, NW Casing & DCPT COMPILED BY SS
DATUM Geodetic DATE 8/5/2008 8/6/2008 CHECKED BY ZO



+ 3, X 3: Numbers refer to
Sensitivity

20
15-10
10
(%) STRAIN AT FAILURE

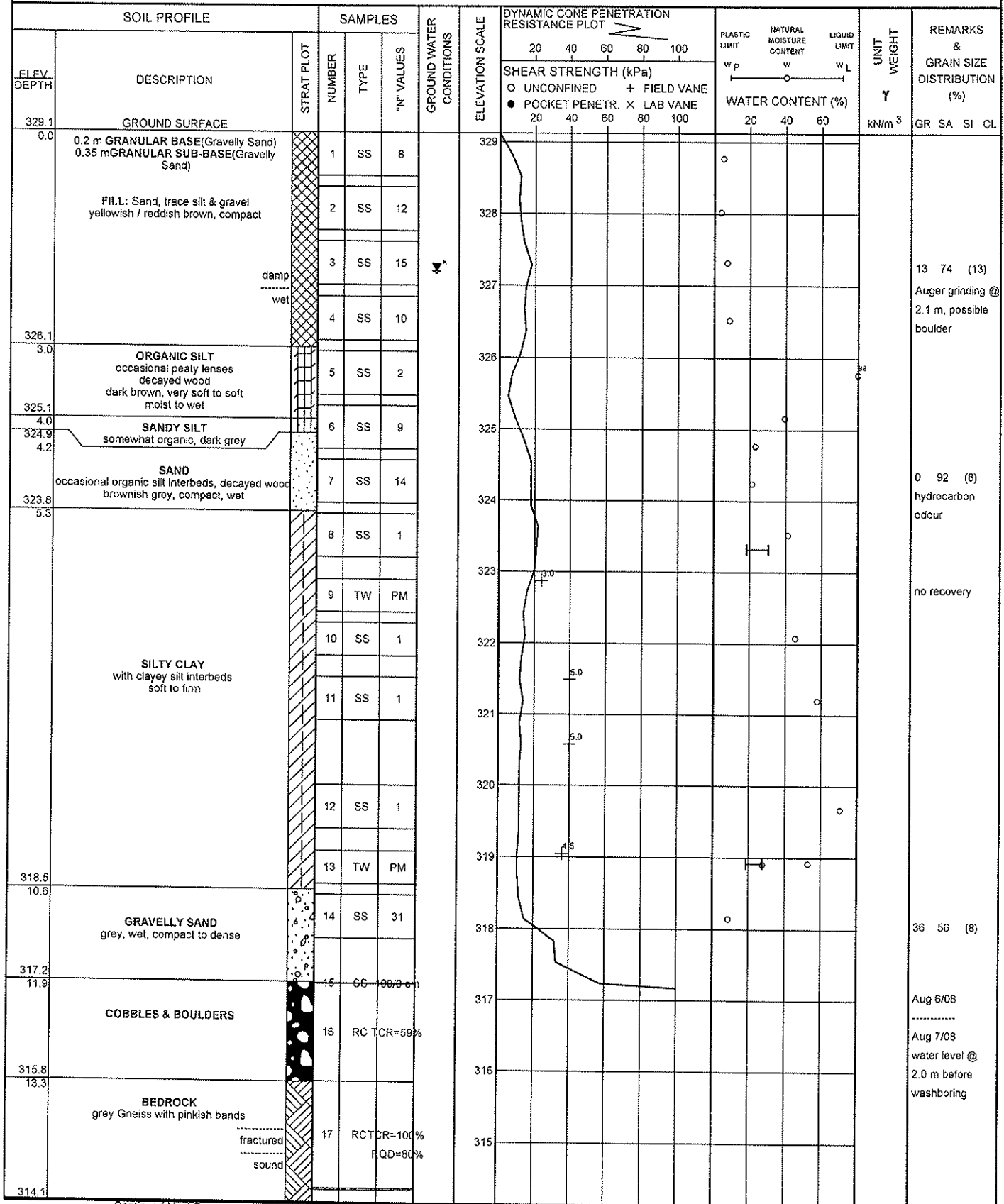
SPT 1221D

RECORD OF BOREHOLE No FC4-2

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 21+564, 4.0 m Lt C/L of Hwy 522 ORIGINATED BY SK
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger, NW Casing & DCPT COMPILED BY SS
DATUM Geodetic DATE 8/6/2008 8/7/2008 CHECKED BY ZO



Continued Next Page

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1221D

RECORD OF BOREHOLE No FC4-2

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 21+564, 4.0 m Lt C/L of Hwy 522 ORIGINATED BY SK
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger, NW Casing & DCPT COMPILED BY SS
 DATUM Geodetic DATE 8/6/2008 8/7/2008 CHECKED BY ZO

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT | NATURAL MOISTURE CONTENT | LIQUID LIMIT | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|----------------|-------------|------------|---------|------|------------|----------------------------|-----------------|---|--|--|------------------|--------------------------------|-----------------|---|--|
| ELEV. DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH (kPa) | | | | | | | |
| | | | | | | | | 20 40 60 80 100 | | | | | | | |
| | | | | | | | | ○ UNCONFINED + FIELD VANE | | | | | | | |
| | | | | | | | | ● POCKET PENETR. X LAB VANE | | | | | | | |
| | | | | | | | | 20 40 60 80 100 | | | | | | | |
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+³, x³: Numbers refer to
Sensitivity

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15 10 5
(%) STRAIN AT FAILURE

SPT 1221D

RECORD OF BOREHOLE No FC4-3

1 OF 1

METRIC

GWP 484-98-00 LOCATION Sta : 21+575, 9.5 m Lt C/L of Hwy 522 ORIGINATED BY SK
DIST HWY 522 BOREHOLE TYPE Manual & DCPT COMPILED BY SS
DATUM Geodetic DATE 9/16/2008 CHECKED BY ZO

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|---------------|--|--------------|---------|------|------------|----------------------------|-----------------|---|-----------------|-----------------|---|---|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 40 60 80 100 | 20 40 60 80 100 | 20 40 60 80 100 | | |
| 326.9 0.0 | GROUND SURFACE | | | | | | | | | | | |
| | SAND: trace gravel & silt, occasional organics, brown, very loose | moist wet | 1 | SS | 1 | | | | | | | |
| 325.7 1.2 | SAND & GRAVEL brown, compact, wet | | 2 | SS | 3 | | | | | | | |
| 325.1 1.8 | | | 3 | SS | 12 | | | | | | | |
| 324.8 2.1 | CLAYEY SILT somewhat organic, dark grey, stiff | | 4 | SS | 10 | | | | | | | |
| 324.5 2.4 | SAND brown, loose, wet | | 5 | SS | 9 | | | | | | | |
| 323.9 3.0 | GRAVELLY SAND brown, loose, wet | | | | | | | | | | | |
| | End of Hand Drilled Borehole. Dynamic Cone Penetration Test (DCPT) performed from 3.0 to 8.6 m | | | | | | | | | | | |
| 318.3 8.6 | N-Values adjusted for 140 lb hammer Water Level @ 1.2 m upon completion (not stabilized)* DCPT performed from 3.1 m & ended @ 8.6 m | | | | | | | | | | | |

+³ . X³ : Numbers refer to
Sensitivity

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15 10 5
(%) STRAIN AT FAILURE

SPT 1221D

RECORD OF BOREHOLE No FC4-RP1

1 OF 1

METRIC

GWP 484-98-00 LOCATION Sta : 21+560, 4.0 m Rt C/L of Hwy 522 ORIGINATED BY SK
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
 DATUM Geodetic DATE 9/9/2008 CHECKED BY ZO

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | | | |
|---------------|---|---------------|---------|------|------------|----------------------------|-----------------|--|----|----|---|---|---|--|--|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. x LAB VANE | | | | | WATER CONTENT (%) W _P W W _L | | | | |
| 329.2 0.0 | GROUND SURFACE | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | | |
| 328.6 0.6 | GRANULAR FILL: Gravelly Sand brown, loose, moist | | 1 | SS | 6 | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | FILL: Sand trace silt, occasional gravel very loose to compact | | 2 | SS | 6 | | | | | | | | | | | | |
| | | | 3 | SS | 13 | | | | | | | | | | | | |
| 326.7 2.5 | | damp moist | | | | | | | | | | | | | | | |
| 326.2 | FILL: Silty Sand, brown / dark brown very loose, wet | | 4 | SS | 4 | | | | | | | | | | | | |
| 3.0 | | | | | | | | | | | | | | | | | |
| 325.8 | TOPSOIL (Peaty) & ORGANIC SILT black / dark brown, very loose, wet | | 5 | SS | 4 | | | | | | | | | | | | |
| 3.4 | | | | | | | | | | | | | | | | | |
| 325.7 3.5 | SILT & CLAYEY SILT grey, very soft, wet | | | | | | | | | | | | | | | | |
| | | | 6 | SS | 2 | | | | | | | | | | | | |
| | ORGANIC SILT some sand, dark brown soft | | | | | | | | | | | | | | | | |
| 324.3 | | | 7 | SS | 1 | | | | | | | | | | | | |
| 4.9 | | | | | | | | | | | | | | | | | |
| 323.9 | SAND brown, very loose, wet | | | | | | | | | | | | | | | | |
| 5.3 | | | 8 | SS | 1 | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | SILTY CLAY with clayey silt interbeds grey, soft to firm dilatant | | 9 | SS | 1 | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| | | | 10 | SS | 1 | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | |
| 320.2 9.0 | | | | | | | | | | | | | | | | | |
| | SILT grey, loose wet, dilatant | | 11 | SS | 8 | | | | | | | | | | | | |
| 319.0 | | | | | | | | | | | | | | | | | |
| 10.2 | | | | | | | | | | | | | | | | | |
| | GRAVELLY SAND grey, wet, compact to very dense | | | | | | | | | | | | | | | | |
| 318.2 11.0 | | | 12 | SS | 100/20 cm | | | | | | | | | | | | |
| | End of borehole Auger refusal @ 11.0 m Water Level @ 1.8 m upon completion (not stabilized)* | | | | | | | | | | | | | | | | |

+³, ×³: Numbers refer to
Sensitivity

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(%) STRAIN AT FAILURE

SPT 1221D

RECORD OF BOREHOLE No FC4-RP2

1 OF 1

METRIC

GWP 484-98-00 LOCATION Sta : 21+587, 4.5 m Rt C/L of Hwy 522 ORIGINATED BY SK
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 9/9/2008 CHECKED BY ZO

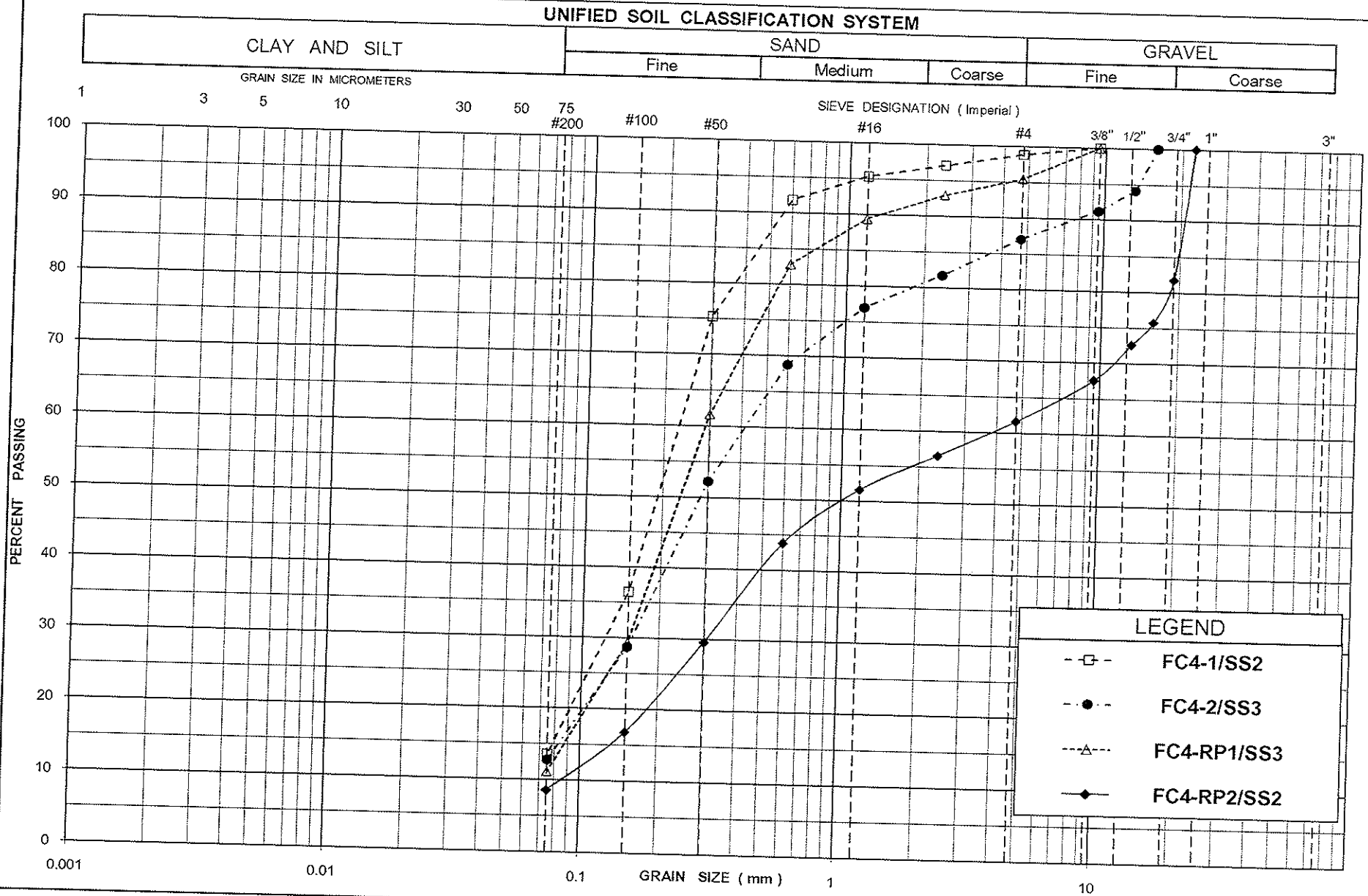
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|----------------|---|------------|---------|------|------------|----------------------------|-----------------|--|--|---|--|--|---|---|
| ELEV. DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE | | WATER CONTENT (%) W _P W W _L | | | | |
| 328.8 | GROUND SURFACE | | | | | | | | | | | | | |
| 0.0 | GRANULAR PAVEMENT FILL: Sand & Gravel, brown, very loose, damp to moist | | 1 | SS | 4 | | | | | | | | | |
| 328.3 | | | | | | | | | | | | | | |
| 0.5 | FILL: Gravelly Sand compact | | 2 | SS | 12 | | | | | | | | | |
| | brown, damp | | | | | | | | | | | | | |
| 327.1 | FILL: Sand, trace gravel dark brown, compact, moist | | 3 | SS | 15 | | | | | | | | | |
| 1.7 | | | | | | | | | | | | | | |
| 326.6 | FILL: Sand, trace gravel, organics & topsoil, dark brown, loose, wet | | 4 | SS | 9 | | | | | | | | | |
| 2.2 | | | | | | | | | | | | | | |
| 326.0 | TOPSOIL black, peaty | | | | | | | | | | | | | |
| 2.8 | | | | | | | | | | | | | | |
| 325.6 | occ. organics | | 5 | SS | 4 | | | | | | | | | |
| 3.2 | dilatant | | | | | | | | | | | | | |
| | SILTY CLAY clayey silt interbeds soft to firm | | 6 | SS | 3 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 7 | SS | 1 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 8 | SS | 1 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| 322.1 | | | | | | | | | | | | | | |
| 6.7 | | | | | | | | | | | | | | |
| | GRAVELLY SAND some silt, grey loose, wet | | 9 | SS | 10 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 10 | SS | 9 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| 320.2 | | | | | | | | | | | | | | |
| 8.6 | | | | | | | | | | | | | | |
| | SAND some silt, trace gravel grey, compact, wet | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 11 | SS | 11 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| 318.7 | | | | | | | | | | | | | | |
| 10.1 | End of borehole Auger Refusal @ 10.1 m Water Level @ 1.8 m (not stabilized)* upon completion | | | | | | | | | | | | | |

+ 3, X 3: Numbers refer to
Sensitivity

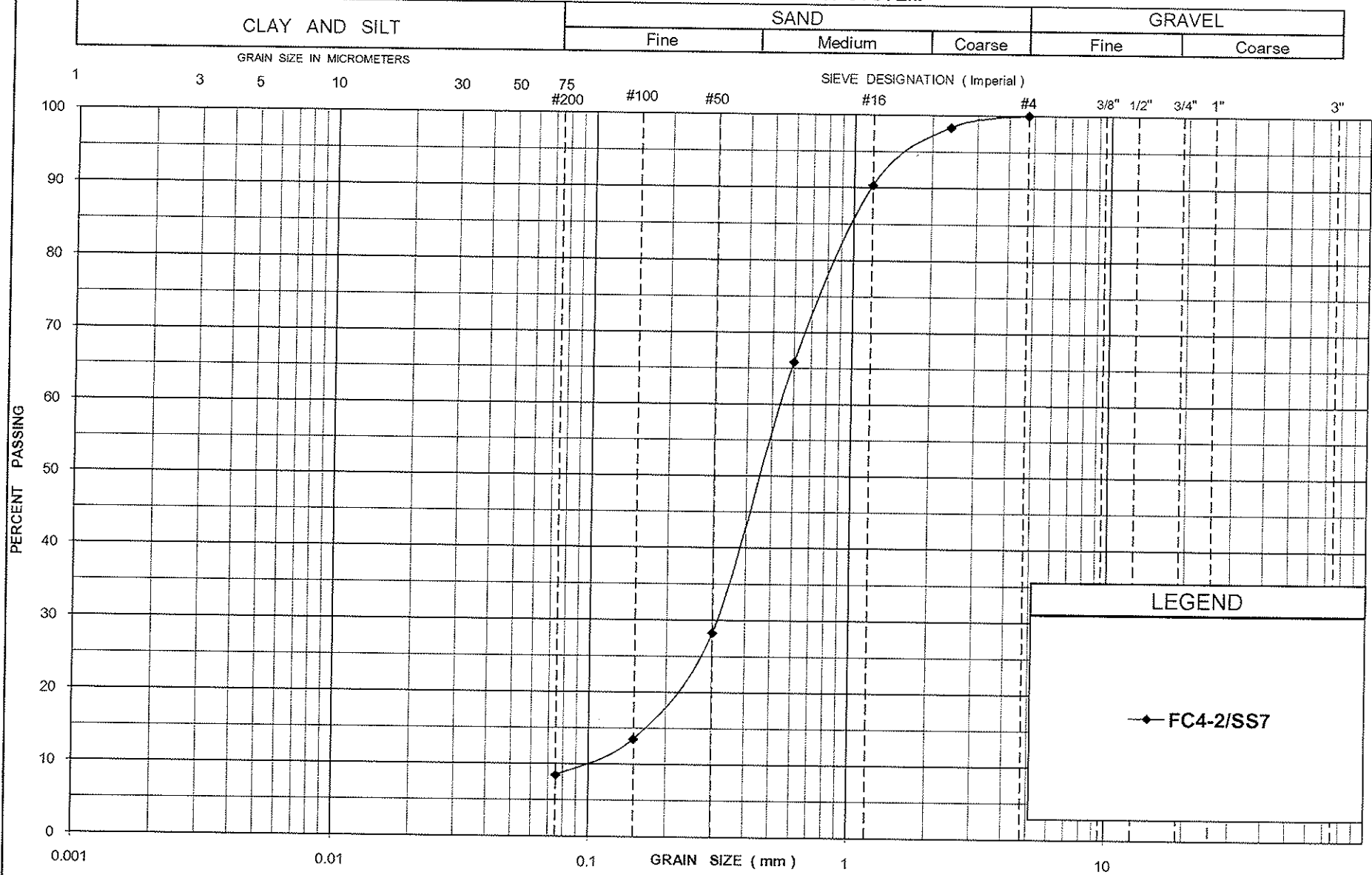
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(%) STRAIN AT FAILURE

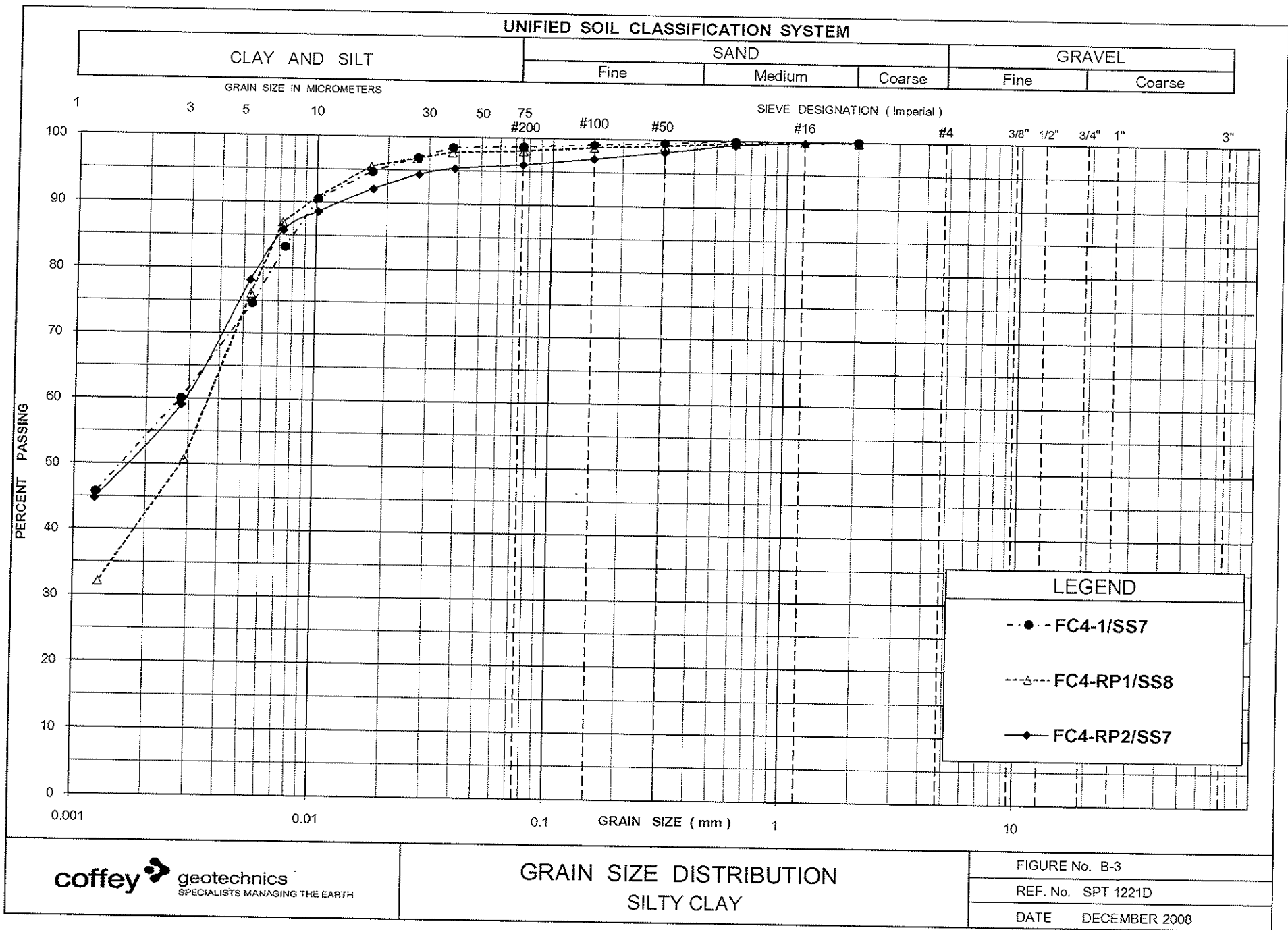
Appendix B

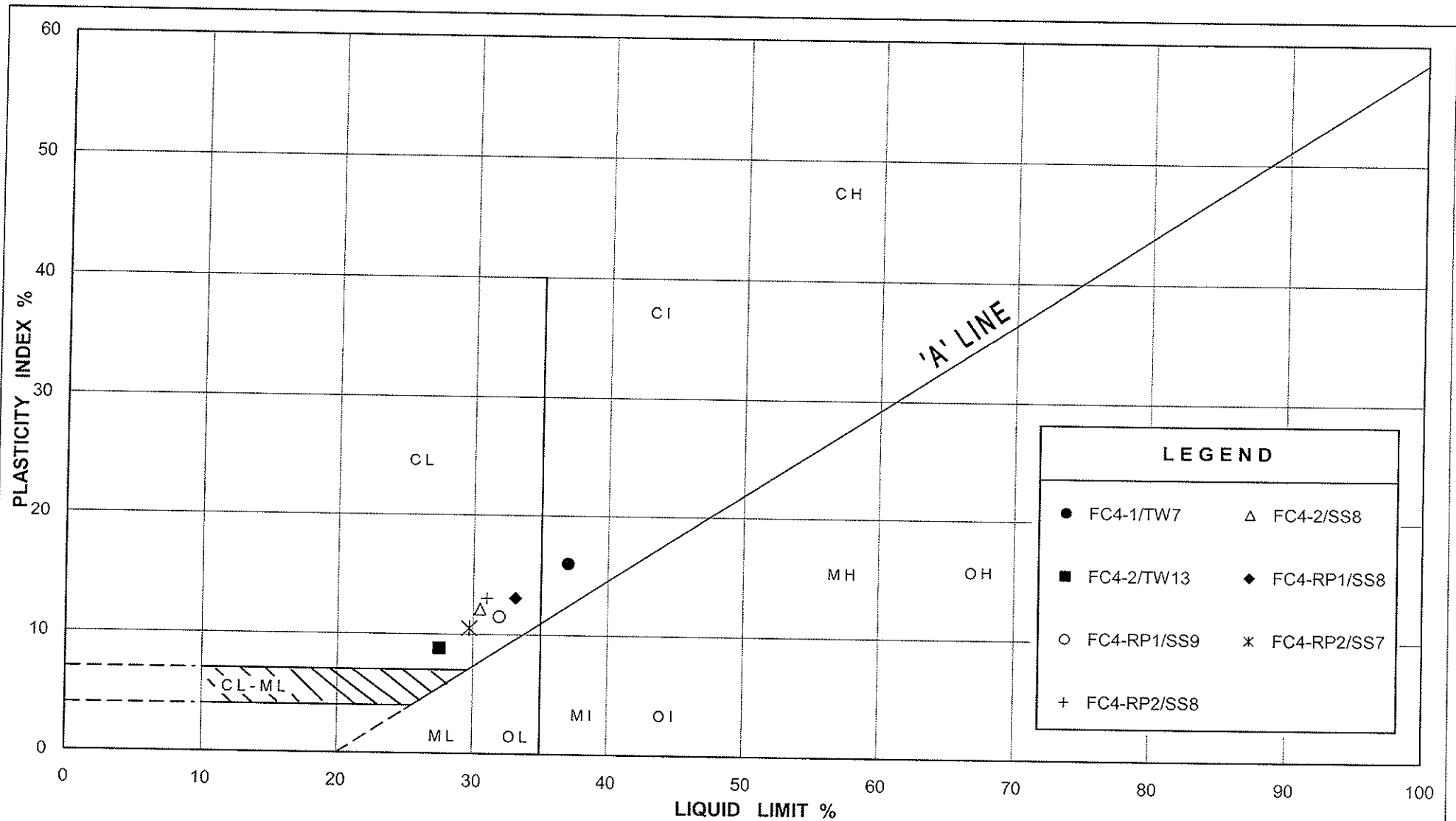
Laboratory Test Results

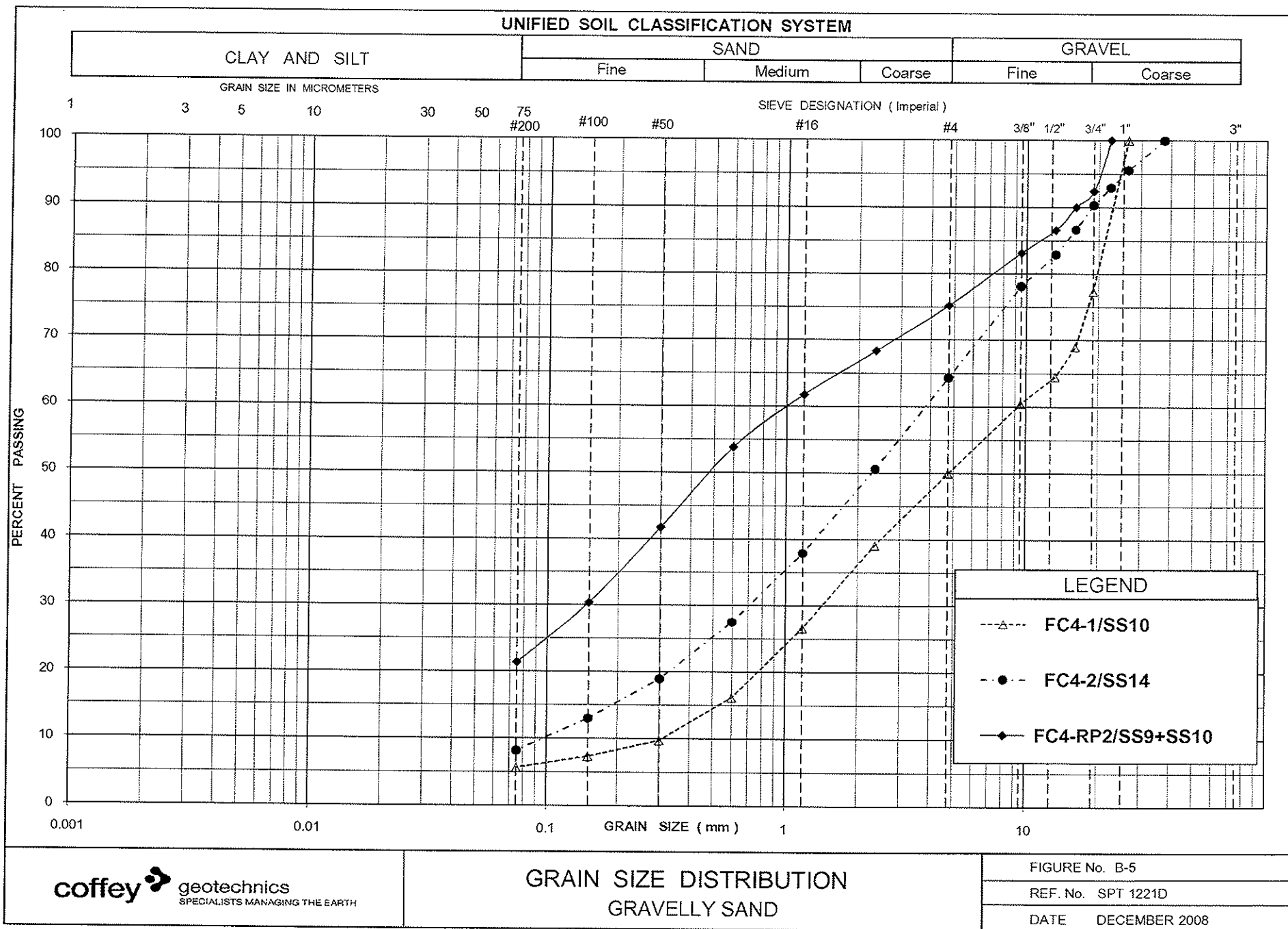


UNIFIED SOIL CLASSIFICATION SYSTEM









Appendix C

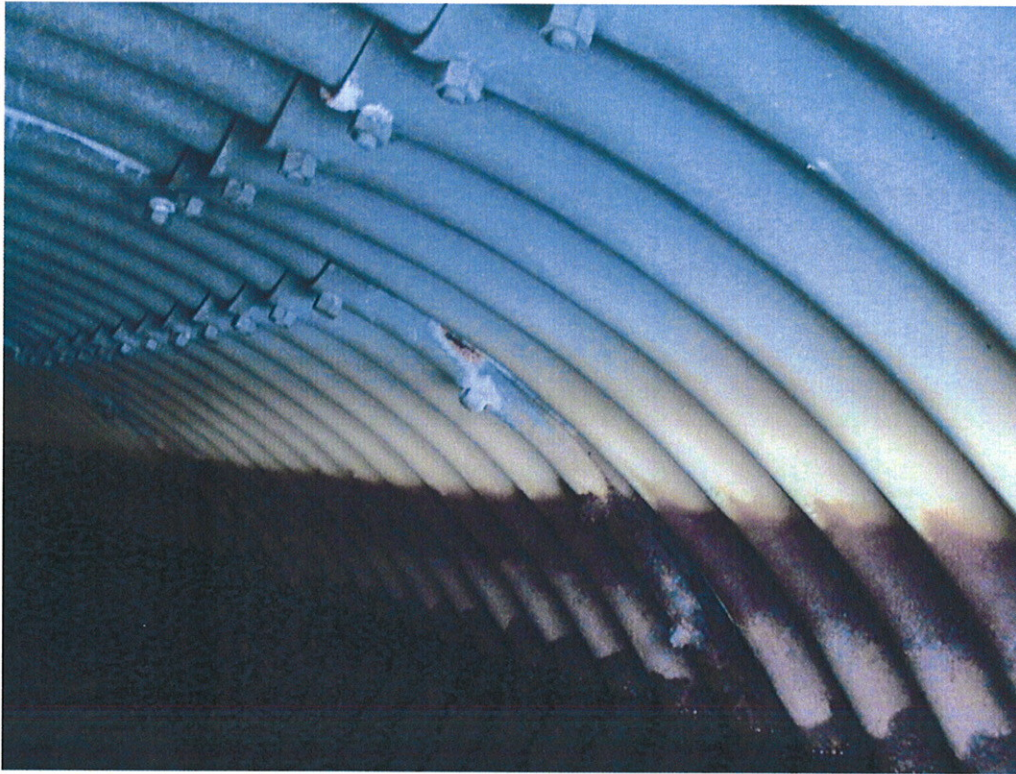
Site Photographs



Photograph 1. View West From South Side of Highway 522



Photograph 2. View East From South Side of Highway 522



Photograph 3. Extent of Corrosion on Interior of Pipe Wall



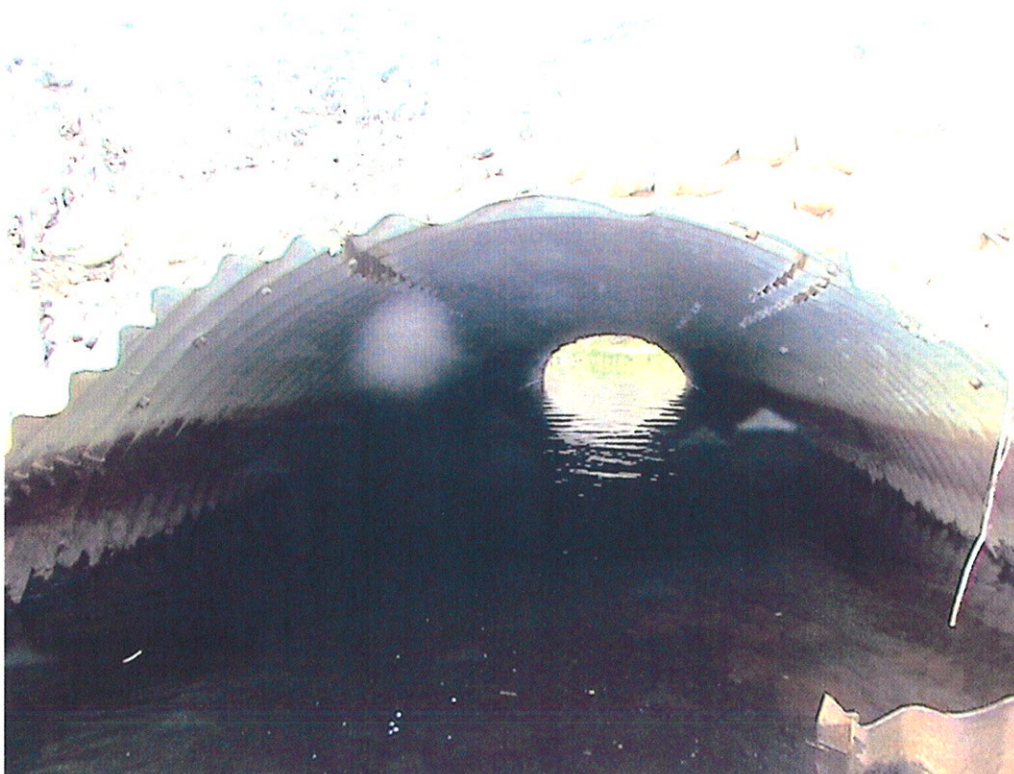
Photograph 4. Detail of Corrosion at Water Line



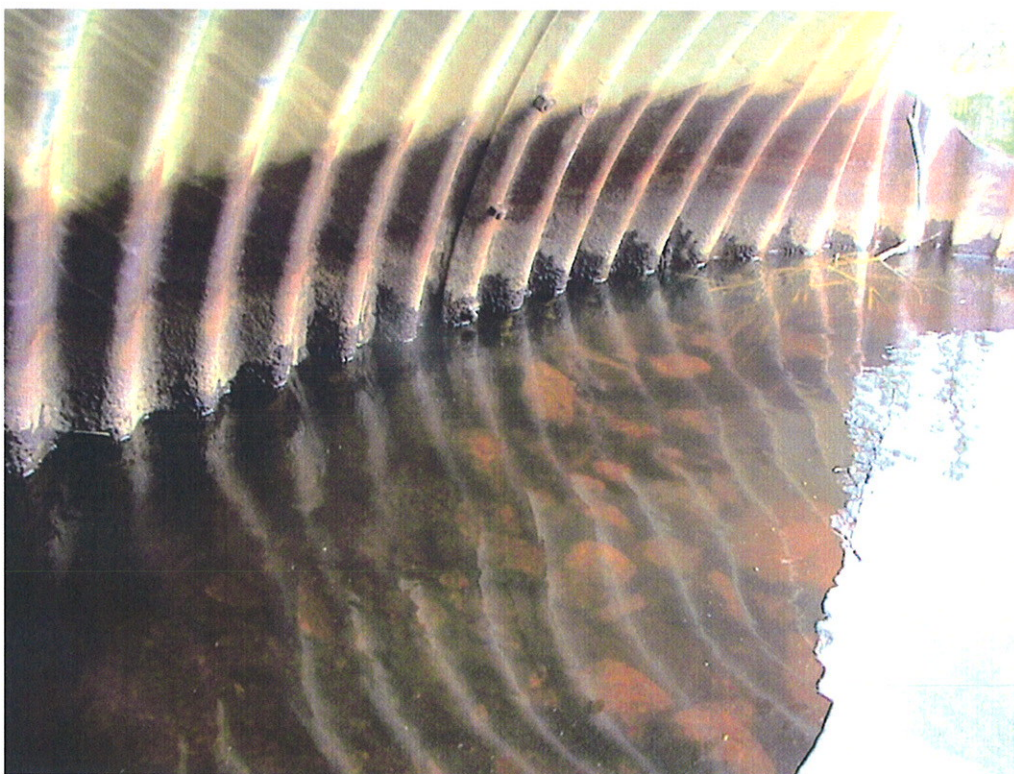
Photograph 5. View Out From Culvert Outlet



Photograph 6. Toe of Slope at Culvert Inlet



Photograph 7. Inlet of Culvert



Photograph 8. Corrosion at Waterline at Inlet



Photograph 9. View South From Shoulder



Photograph 10. View North From Shoulder

Appendix D

Rock Core Photographs



Rock Core - BH FC4-1



Rock Core – BH FC4-2

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 \bar{N} .

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

**FOUNDATION DESIGN REPORT
BLACK CREEK CULVERT (C4)
REPLACEMENT AT STATION 21+570,
HIGHWAY 522,
MUNICIPAL TOWNSHIP OF
NIPISSING, ONTARIO, SITE NO. 44-274
GEOCRES NO. 31E-285, G.W.P. 484-98-00**

D. M. Wills Associates Limited

Project: SPT1221D
May, 2009

FINAL REPORT

CONTENTS

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Appendices

Appendix F: OPSD

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**FOUNDATION DESIGN REPORT
BLACK CREEK CULVERT (C4) REPLACEMENT AT STATION 21+570
HIGHWAY 522, MUNICIPAL TOWNSHIP OF NIPISSING, ONTARIO
SITE NO. 44-274
G.W.P. 484-98-00**

5. DISCUSSION AND RECOMMENDATIONS

The existing Black Creek Structure is a structural plate corrugated steel pipe arch (SPCSPA) culvert. The walls of the pipe arch are cold-formed, corrugated galvanized structural steel with a corrugation profile of 150 mm x 50 mm x 4 mm wall thickness. The SPCSPA has a span (maximum width) of 3.0 metres and a rise (maximum clear height) of 2.2 metres. The length of the culvert is approximately 21 metres, as measured along the invert of the structure. Stone headwalls are present at both the inlet and outlet of the culvert.

The culvert is installed perpendicular to Highway 522 with no skew angle. There is approximately 1.5 m cover over the culvert including the road that provides for a single lane of traffic in each direction. Three-cable guiderail extends along the north shoulder of Highway 522 over the culvert.

A visual inspection of the culvert was carried out during site visits in the Winter and Spring of 2008 by D. M. Wills Associates Limited. The following is a summary of the significant findings.

- Heavy corrosion with pitting is present at the normal water line and below. Some rusting is evident in the upper portions of the culvert at the seams.
- There is a small pond located upstream of the culvert inlet.
- Utility poles on the south side of the road are in close proximity to the culvert.
- There are signs of flattening at the top of the arch.

The depth of the water at the time of the inspections by D. M. Wills ranged from 1.0 m to 1.2 m.

The invert of the existing culvert is at El. 325.11 m on the south (inlet) side and 324.97 m of the north (outlet) side. The invert of the new culvert is expected to match that of the existing. No widening or grade raise of the existing embankment is planned.

This present investigation has shown, in general, below an approximately 2.3 to 3.0 m of embankment fill, the presence of 0.3 to 1.9 m thick organic or partially organic soil to El. 325.6 to 324.3 m. The organic soils are underlain in some of the boreholes by surficial sand layers. Beneath the organic soils and/or the underlying sand deposits, the site is underlain by a cohesive soil deposit ranging from silt to silty clay. This deposit extends to El. 322.1 to 318.5 m and is considered to have a generally soft to firm consistency. The silty clay is underlain by compact to dense coarse grained granular overburden to the surface of the bedrock. The gneissic bedrock was cored at two locations where the surface of the bedrock was contacted at El. 319.4 and 315.8 m. The groundwater level at the time of our investigation was found between El. 327.7 and 326.3 m (i.e. close to the original ground surface). Water level in the existing culvert was measured at about El. 326.3 m in January 2008.

We understand that it is preferred to replace the existing SPCSPA culvert with a 2.7 m diameter corrugated aluminium alloy pipe culvert. As was mentioned before, the invert levels will be practically the same as the existing (i.e. El. 325.0 m \pm). Assuming a bedding thickness of 0.3 m, the bottom of the bedding can be expected at about El. 324.7 m. At this elevation the boreholes show the presence of sand and gravelly sand at Borehole FC4-2 and FC4-3 locations and very soft to soft clayey silt at Borehole FC4-1. Borehole FC4-3 was terminated in the gravelly sand deposit at El. 323.9 m, while in Borehole FC4-2 the sand was found to extend to El. 323.8 m where it is underlain by soft to firm silty clay.

From foundation engineering point of view, with the prevailing soil and groundwater conditions, the use of flexible structure such as a CSP is the preferred option for the replacement of the existing culvert, but if necessary, a pre-cast concrete box structure can also be considered as discussed below.

The use of an open bottom culvert is not recommended as the silty clay underlying the site is very weak and does not offer a suitable resistance for the support of normal spread footing foundations. A concrete box (i.e. closed bottom) structure can be considered but in view of the weak foundation soils and the high water table, as well as the presence of a large body of water on both sides of the highway, the construction of a cast-in-place structure is considered less practical, than a pre-cast concrete structure. As was mentioned before, a flexible CSP type culvert would be more suitable but a pre-cast concrete structure with flexible joints can also be considered, if necessary.

5.1 Corrugated Steel Pipe (CSP) Type Culvert

The native loose to compact sand or soft to firm silty clay/clayey silt soils, in their undisturbed state are suitable to support a flexible structure, provided a suitable bedding is placed between the undisturbed soil and the structure. Due to high water table, extensive dewatering effort will be required, to preserve the load carrying capacity of the soils and to facilitate the construction, as discussed later in this report.

A minimum bedding thickness of 300 mm is recommended for a CSP type culvert. After excavating, the site to the underside of the bedding (i.e. to 0.3 m below the invert level), the exposed subgrade should be carefully inspected and approved. If organic or other unsuitable soils are found they should be removed to the surface of the inorganic, suitable soil and replaced with suitable soils.

Provided that all the unsuitable soils are removed and, where necessary, replaced with suitable granular soils (i.e. where the grade needs to be raised after sub-excavation) and the subgrade is not unduly disturbed, there should be no problems with bearing resistance and settlements, since there will be virtually no load increases over and above the existing conditions (i.e. no widening or grade raise of the embankment). However, for the sake of completeness, the following resistances can be assumed for undisturbed subgrade soils.

| | | |
|---------------------------------------|---|---------|
| Factored Bearing Resistance at U.L.S. | = | 120 kPa |
| Geotechnical Resistance at S.L.S. | = | 50 kPa |

Under the embankment, the recommended value at S.L.S. 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 consolidated and settled under the stresses generated by the existing embankment. Therefore,

since there will be no additional loading, theoretically there should be negligible additional settlements. However, a settlement of about 25 mm should be allowed for, due to rebound during the brief construction period (i.e. the embankment will be excavated during which time there will be a rebound due to stress relief, subsequently backfilling and thus settlement will take place) as well as due to exchange of the lighter, unsuitable soils with granular backfill which is relatively heavier.

Based on the fact that if the founding natural subgrade is not unduly disturbed during the construction, the settlements should not exceed 25 mm, it is our opinion that cambering is not necessary at this site.

5.2 Precast Concrete Box Culvert

We understand that the use of a precast concrete box culvert was also considered but at present a 2.7 m diameter corrugated aluminium alloy pipe is preferred. If a concrete box culvert is to be installed it is expected to be 3.5 m wide and 2.6 m high (outside dimensions).

A minimum granular bedding of 400 mm is recommended for a precast concrete box type culvert to provide a relatively uniform support as the boreholes show varying soils at the proposed invert elevation. After excavating the site to the underside of the bedding layer (i.e. to 0.4 m below the proposed invert of the box), the exposed subgrade should be carefully inspected and approved. If organic or otherwise unsuitable soils are encountered, these should be removed to the surface of inorganic, suitable soil and replaced with suitable granular soils.

As mentioned before, provided that all the unsuitable soils are removed and replaced with suitable soils and that the subgrade soils are not unduly disturbed, there should be minimum settlements and bearing resistance problems since neither a grade raise nor an embankment widening is to take place. In this case (i.e. as opposed to CSP type culvert) the concrete box culvert may be slightly heavier than a CSP (if the new culvert is to be placed at the same location as the existing), as well, the construction time period may be somewhat longer. For these reasons, a settlement of 50 mm should be allowed for. This normally does not present a problem for a pre-cast concrete box culverts with short sections (i.e. typically 2.4 m lengths) but this aspect should be verified with the supplier that a 50 mm total and 40 mm differential settlement (i.e. in between two adjacent precast sections) will not present problems.

Cambering is not considered necessary, especially since it will be difficult to implement cambering with the adverse groundwater conditions, prevailing at the site. This is because with the high water table, coupled with weak subgrade soils it will be difficult to implement a camber of about 30 mm at the centre, reducing to zero at the ends (i.e. at the inlet and the outlet).

5.3 Deep Foundations

A cast-in-place concrete box type culvert can be supported on deep foundations. Due to the presence of cohesionless soils, exhibiting upward seepage forces, underlying the silty clay deposit, the use of drilled and cast-in-place concrete foundations (i.e. caisson foundations) is considered impractical. The caissons can be socketed into the bedrock which will mitigate this problem. This approach is however considered to be very cost ineffective for a culvert structure, especially due to the presence of cobbles and boulders, overlying the bedrock.

Steel tube H-piles could be considered, driven to refusal at about El. 320 to 316 m. Since the invert is at about El. 325 m, the length of the piles is expected to be between 5 and 9 m. A tentative resistance of about 600 kN/pile at SLS and 900 kN/pile at ULS is suggested for 250 mm diameter steel tube or size 250 steel H-piles (e.g. HP 250 x 62). It is however believed that the use of driven piles at this site will be impractical as well as being cost ineffective. Additionally, vibrations may also be objectionable and the time required to effect the construction will likely be objectionable.

The deep foundation option is considered to be the most reliable but highly impractical and costly for this project. Further details will therefore not be given herewith; however, we will be pleased to further discuss it should you wish to consider this option in more detail.

5.4 Bedding

The bedding material should consist of an approved granular material, such as Granular B Type II. The bedding material should be placed as soon as practicable after the preparation of the subgrade, its inspection and approval, as discussed in Sections 5.1 and 5.2. The bedding material should be in accordance with the appropriate standards (e.g. OPSD-802.010 and 802.014) and should consist of not less than the following thicknesses:

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

The bedding material should be compacted to MTO standards (SP 105S10).

5.5 Backfilling

The bedding and embedment material should be extended along the sides and the top to cover the pipe. The selection and placing of the backfill should be in accordance with OPSD-802.010 and OPSD-802.014. The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular 'A' or 'B' (OPSS-1010). All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted to at least 96% of the material's SPMDD. The Granular 'A' and Granular 'B' sub-base courses should be compacted to 100% of the material's SPMDD. The backfill should be inspected by QVE in accordance with SP 902S01

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

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

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

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

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

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

Unit weight = 22 kN/m³

Coefficient of Lateral Earth Pressure:

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

Compacted Granular 'B' Type I

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

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

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

Note: K_a is the coefficient of active earth pressure

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

K_o is the coefficient of earth pressure at rest

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

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

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

5.6 Construction Comments

We understand that presently a staged construction is proposed, requiring a full roadway protection, with single lane closures, to be staged on the existing roadway platform. With this approach, shoring will be required while no change in the existing side slopes is anticipated for staging.

Dewatering the site to a sufficient degree to facilitate the construction and to effect proper subgrade stripping will require considerable effort, as ponded water generally prevails on both sides of the highway embankment, particularly on the inlet side, as shown on the site photographs in Appendix C of this report.

If the flow in the existing watercourse is 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. In the later case, after the construction, the flow would be diverted to the completed new culvert. As was mentioned before, the CSP type culverts, especially the arch types, require good side support for proper performance. For this reason when removing the existing or the temporary pipe after the completion of construction, careful construction techniques will need to be applied to avoid the loss of side support for the new culvert. Grouting the pipe is always a good practice. If, however, this is not cost effective, proper removal and backfilling procedures should be applied.

For dewatering, it is envisaged that first the surface water which ponds on the inlet side (and possibly on the outlet side) of the culvert will need to be considered. This will likely require dewatering and pumping the water into a confined area retained by cofferdams. After containing the surface water, the subgrade will need to be dewatered. This will require, as a minimum, pumping from strategically placed, filtered sumps. When designing the dewatering system, it should be remembered that while the silty clay is of low permeability, the surficial sand deposits, which were encountered in Boreholes FC4-2, FC4-3 and FC4-RP1, are relatively more pervious soils, which can yield significant water into excavations below the groundwater level. The silt deposits can be classified as soils with intermediate permeability.

It should also be pointed out that the site will need to be sufficiently dewatered to prevent the disturbance of the subgrade soils, as well as preventing the dilation of the silt and clayey silt soils.

We recommend that the Contractor be asked to submit their method of diversion and containment of surface water and that of the dewatering to the CA for information purposes.

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

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

SP 105 S19 – Protection Systems

SP 902 S01 – Excavation and Backfilling to Structures

In accordance with the Province's Safety Regulations, the following soil classification would be applicable.

Fill : Type 3 soil above groundwater level and Type 4 soil below groundwater level.

Sand: Type 3 soil above groundwater level (or if the soil is dewatered) and Type 4 soil below groundwater level

Organic Silt/Peat/Topsoil: Type 4 soil

Silt/Clayey Silt/Silty Clay: Type 4 soil

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

As mentioned before, it is expected that temporary shoring will be required to support the excavations. In Ontario shoring typically consists of soldier pile and timber lagging. But tight interlocking steel sheet piling system may also be considered with internal bracing, deadman anchors or tiebacks whichever ones are appropriate. Sheet piling is considered to be less suitable for this site due to the presence of cobbles and boulders, especially immediately above the bedrock surface. The shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this case, the required performance level is considered 2. The shoring system should be designed by a Professional Engineer, experienced in this type of work.

The coefficient of lateral earth pressures given in Table 5.6.1 can be used for the design of the temporary shoring system, based on the borehole results.

Table 5.6.1: Recommended Unfactored Parameters for Temporary Shoring Design

| Soil Type | K_a | K_o | K_p | γ (kN/m ³) |
|-----------------------------------|-------|-------|-------|----------------------------------|
| Granular Fill and Embankment Fill | 0.33 | 0.50 | 3.0 | 21.5 |
| Organic silt/peat/topsoil | 0.49 | 0.70 | 1.4 | 13.0 |
| Surficial Silt/Sandy Silt | 0.39 | 0.56 | 2.5 | 17.0 |
| Upper sand/Gravelly sand | 0.36 | 0.53 | 2.8 | 18.5 |
| Silty Clay/Clayey Silt | 0.48 | 0.65 | 2.0 | 16.0 |
| Lower silt | 0.36 | 0.53 | 2.8 | 18.0 |
| Basal Sand/Gravelly Sand | 0.32 | 0.48 | 3.1 | 20.5 |
| Bedrock upper 0.6 m | 0.24 | 0.32 | 3.6 | 23.0 |
| Below 0.6 m | 0.12 | 0.15 | 5.0 | 24.0 |

It should be pointed out that the presence of cobbles and boulders can be expected within the overburden (e.g. within the gravelly sand deposits overlying the bedrock and especially immediately above the bedrock – see Record of Borehole FC4-1 and FC4-2), as well possibly within the embankment fill. These can be expected to cause problems during the installation of the caisson holes and especially during the driving of steel sheet piling.

As well, there may be some difficulty in advancing the caisson holes into the rather hard bedrock, if this is required to effect shoring.

5.7 Erosion Protection

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

We recommend that concrete cut-off (apron) be constructed both at the inlet and outlet to prevent seepage beneath and around the culvert, especially through the granular bedding and granular backfill around the culvert, as well as the surficial sand layers. 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, after removing any peaty soils. The purpose of the clay seal is to ensure that water flow is channeled through the culvert and does not seep through the backfill around the structure and from beneath the structure. The clay seal should therefore be continuous and typically 0.6 m thick. It should comply with the material specifications given in OPSS 1205. It should be extended around the culvert from at least 0.5 m above the high water level in the watercourse down to the channel bed and up the other side in a continuous manner. It should be ensured that it extends to cover all the granular backfill materials to prevent any seepage through them. Typically, the clay

seal is protected by laying a 0.6 m thick rock protection over it. The clay seal would generally be extended at about 8 m beyond the inlet.

At the outlet as well as at the inlet (if clay seal is not used), in addition to the concrete cut-off and/or impervious seal or in conjunction with these, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered. As the subgrade can be expected to consist of fine-grained soils, a layer of granular or man-made filter material should be placed beneath the rock protection. This would generally be extended about 8 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.

5.8 Frost Protection

Design frost protection for the general area is 1.8 m. A permanent soil cover of at least 1.8 m or its thermal equivalent is therefore required for frost protection. In case of riprap (rock fill), only one half of the rock fill thickness should be assumed to be effective in providing frost protection.

6. CLOSURE

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

For and on behalf of Coffey Geotechnics Inc.


Ramon Miranda, P.Eng.

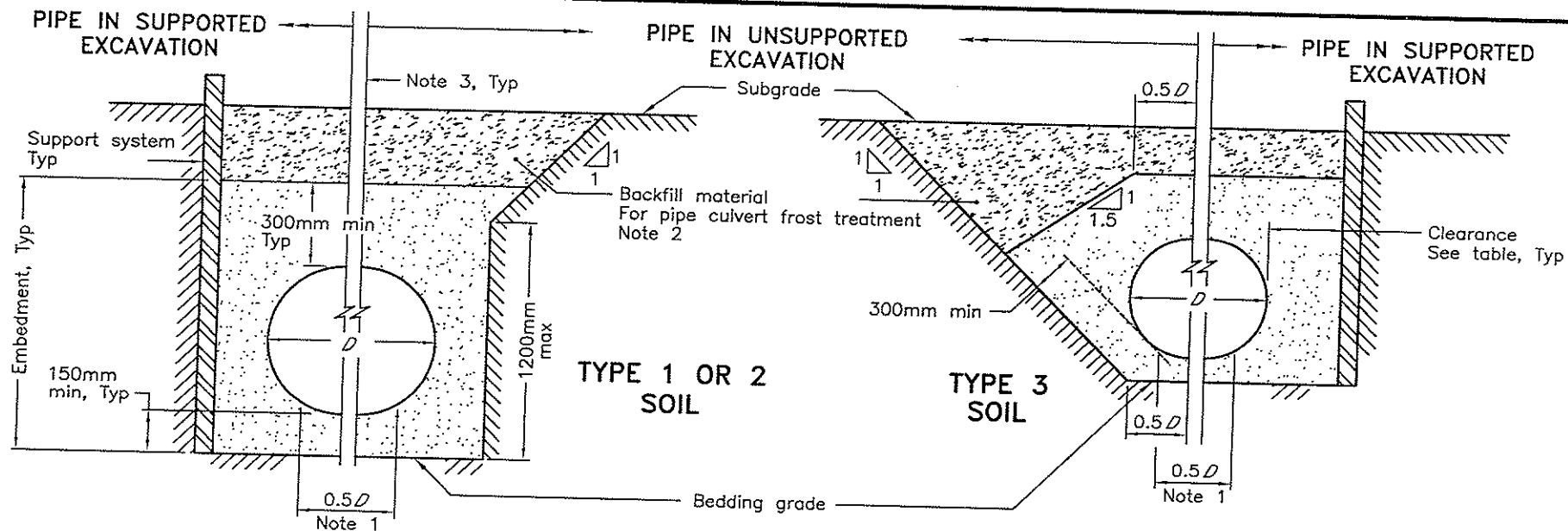



Zuhtu Ozden, P.Eng.



Appendix F

OPSD

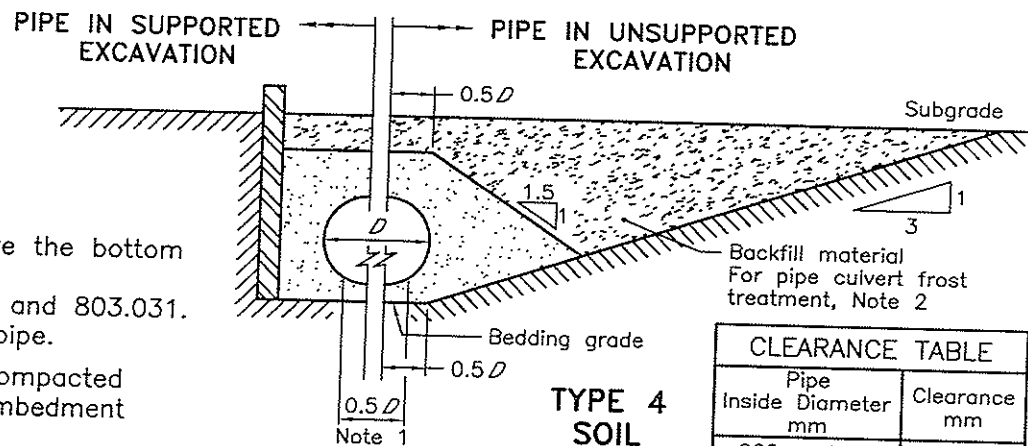


LEGEND:

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.

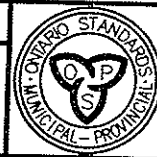


| CLEARANCE TABLE | |
|-------------------------|--------------|
| Pipe Inside Diameter mm | Clearance mm |
| 900 or less | 300 |
| Over 900 | 500 |

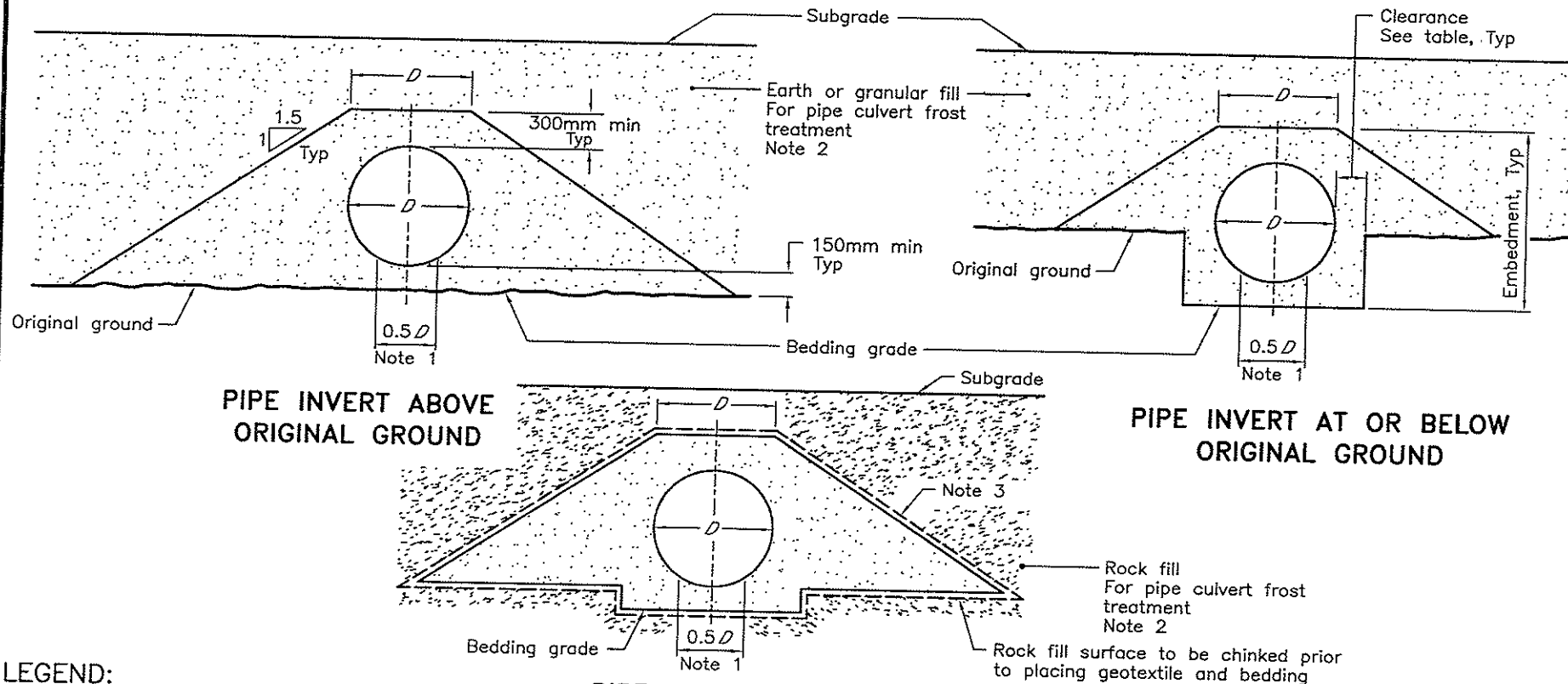
ONTARIO PROVINCIAL STANDARD DRAWING

FLEXIBLE PIPE
EMBEDMENT AND BACKFILL
EARTH EXCAVATION

Nov 2005 Rev 1



OPSD - 802.010




LEGEND:

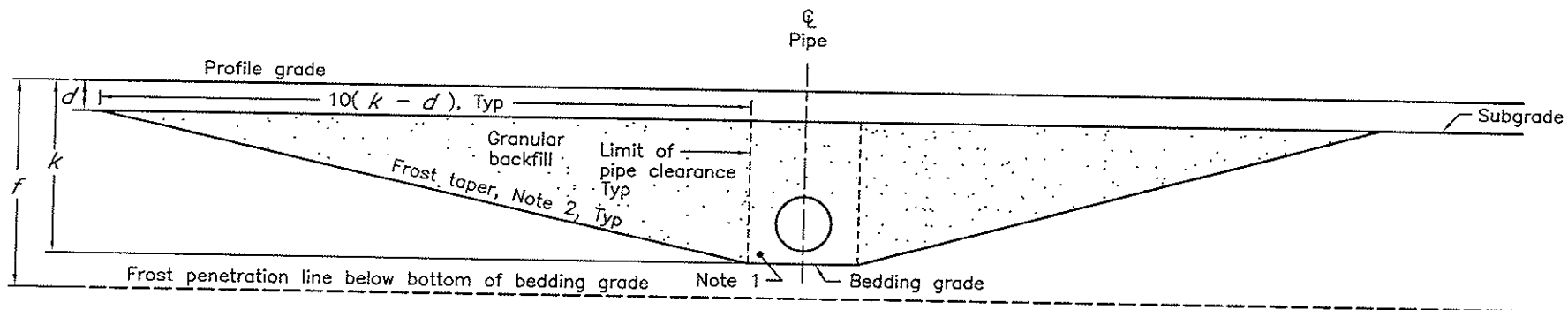
D - Inside diameter

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 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 | | |



FROST TREATMENT – RIGID AND FLEXIBLE PIPE

NOTES:

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

A Frost tapers are not required in rock embankment.

LEGEND:

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

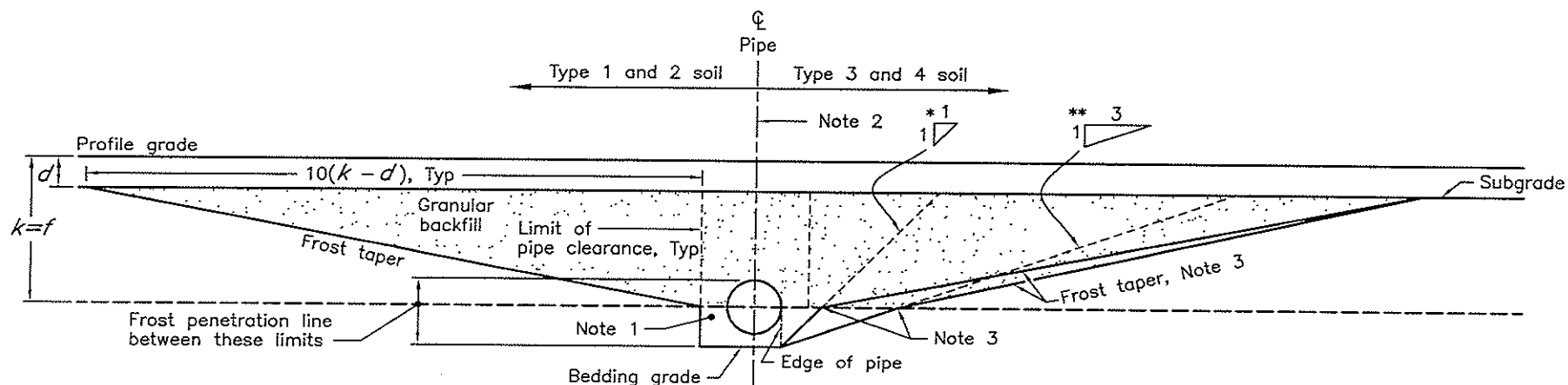
ONTARIO PROVINCIAL STANDARD DRAWING

FROST TREATMENT – PIPE CULVERTS
 FROST PENETRATION LINE BELOW
 BEDDING GRADE

Nov 2005 Rev 1



OPSD – 803.030



FROST TREATMENT – RIGID AND FLEXIBLE PIPE

NOTES:

- 1 Pipe embedment or bedding, cover, and backfill according to:
 - a) Flexible – OPSD-802.010, 802.013, 802.014, 802.020, 802.023 and 802.024
 - b) Rigid – OPSD-802.030, 802.031, 802.032, 802.033, 802.034, 802.050, 802.051, 802.052, 802.053, and 802.054
- 2 Condition of frost treatment symmetrical about centreline of pipe.
- 3 Frost tapers start at the intersection of the 1H:1V or 3H:1V slope and the frost penetration line.
- A Frost tapers are not required in rock embankment.
- B Frost tapers not required when frost line is above the top of pipe.
- C Soil types as defined in the Occupational Health and Safety Act and Regulations for Construction Projects.

LEGEND:

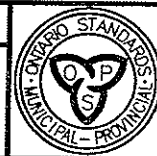
- d – depth of roadbed granular
 k – depth of frost treatment
 f – depth of frost penetration
 $*$ – Type 3 soil
 $**$ – Type 4 soil

ONTARIO PROVINCIAL STANDARD DRAWING

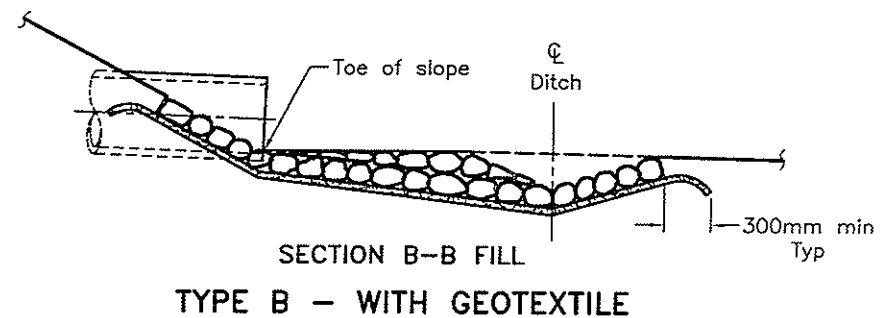
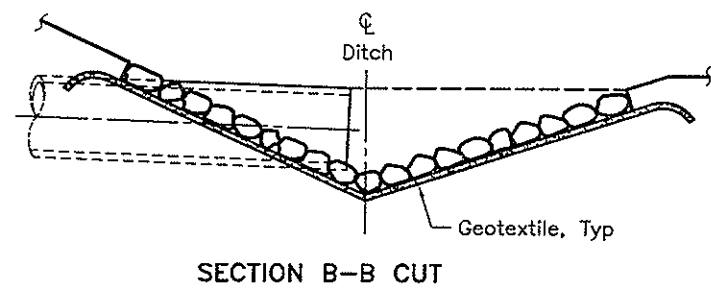
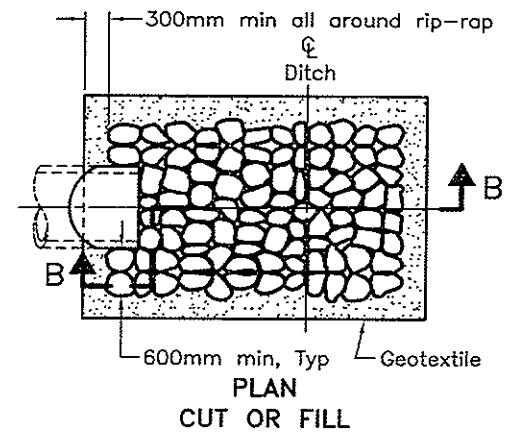
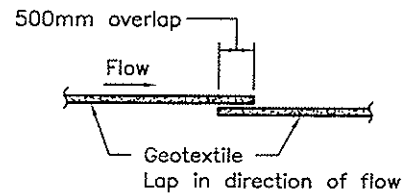
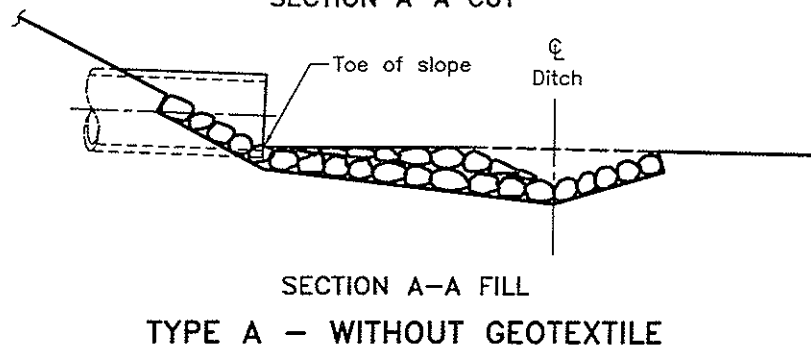
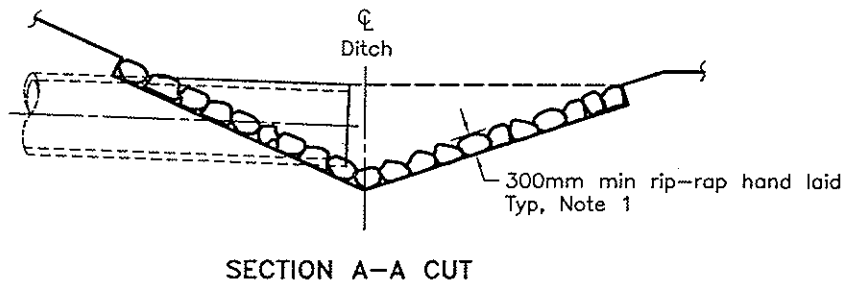
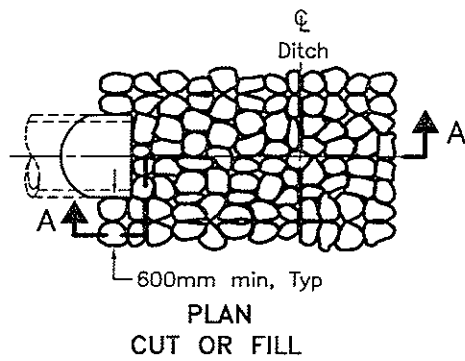
FROST TREATMENT – PIPE CULVERTS
FROST PENETRATION LINE BETWEEN
TOP OF PIPE AND BEDDING GRADE

Nov 2005

Rev 2



OPSD – 803.031



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

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

Nov 2007

Rev 1



OPSD 810.010

Appendix G

Limitations of Report

LIMITATIONS OF REPORT

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

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

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

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

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