

**FOUNDATION INVESTIGATION AND DESIGN  
REPORTS**

**CULVERTS C77, C78 AND C79 AT STATIONS  
12+398, 12+896 AND 13+010 HIGHWAY 522,  
TOWNSHIP OF SOUTH HIMSWORTH,  
DISTRICT 54, SUDBURY, ONTARIO,  
G.W.P. 484-98-00, GEOCRES NO. 31E-293**

D. M. Wills Associates Limited

Project: TRANETOB01221AC  
October 01, 2009

**FOUNDATION INVESTIGATION REPORT  
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AT STATIONS 12+398, 12+896 AND 13+010  
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HIMSWORTH, DISTRICT 54, SUDBURY,  
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D. M. Wills Associates Limited  
452 Charlotte Street  
Peterborough, Ontario  
K9J 2W3

**Attention: Mr. Michael Lang, P.Eng.**

Dear Sir:

**RE: Foundation Investigation and Design Reports, Culverts C77, C78 and C79 at Stations 12+398, 12+896 and 13+010, Highway 522, Township of South Himsworth, District 54, Sudbury, Ontario, G.W.P. 484-98-00, GEOCREs No 31E-293**

Please find attached the 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

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**FOUNDATION INVESTIGATION REPORT  
CULVERTS C77, C78 AND C79 AT  
STATIONS 12+398, 12+896 AND 13+010  
HIGHWAY 522, TOWNSHIP OF SOUTH HIMSWORTH  
DISTRICT 54, SUDBURY, ONTARIO  
G.W.P. 484-98-00**

## **1 INTRODUCTION**

As part of the rehabilitation of Highway 522, from 0.6 km west of Highway 522B westerly for 19.7 km, it is proposed to also rehabilitate and replace several existing culverts.

Coffey Geotechnics Limited (Coffey) was retained by D.M. Wills Associates Limited (Wills) to carry out a foundation investigation at the site of the proposed replacement of three culverts (Culvert Numbers C77, C78 and C79) at Stations 12+398, 12+896 and 13+010 under Highway 522 in the Township of South Himsworth, 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 sites are located approximately 0.5 to 1.5 km west of Highway 11 in the Township of South Himsworth, as shown in Drawing Nos. 1, 4 and 7.

According to the Quaternary Geology of Ontario Map M2556 (Ministry of Northern Development and Mines, Ontario), and the Quaternary Geology of South River Area Map P.3160 (Ontario Geological Survey, 1990), the site is in an area of glaciolacustrine deposits, consisting of deltaic, valley fill and nearshore sands and gravels, and prodeltaic or lakebottom silts and clays. The silts and clays are typically laminated, rhythmically bedded to massive. Generally after the last glacial withdrawal, glacial Lake Algonquin deposited glaciolacustrine (silts and clays) over the sandy glacial till. The glaciolacustrine deposits were then overlain by glaciofluvial sediments (sands, gravels and silts). Local creeks and rivers then bisected the glaciofluvial materials depositing sands, gravels, organic deposits and muck.

According to the Bedrock Geology of Ontario Map 2544 (Ministry of Northern Development and Mines, Ontario), the bedrock underlying the site consists of strongly foliated gneissic rocks of the Central Gneiss Belt, which is part of the Grenville Province (a structural subdivision of the Canadian Shield).

## **3 PROCEDURES**

The fieldwork for this project was performed from July 7 to 9, 22 to 31, August 14 to 22, and September 4, 8, 11 and 15, 2008 and consisted of drilling and sampling a total of fifteen boreholes. Five boreholes were advanced at each culvert location. At Culvert C77 Boreholes FC7-1 to FC7-3, FC7-RP1 and FC7-RP2

were advanced to depths ranging from 15.9 to 26.5 m below existing grades. At Culvert C78 Boreholes FC8-1 to FC8-3, FC8-RP1 and FC8-RP2 were advanced to depths ranging from 4.9 to 26.4 m below existing grades. At Culvert C79 Boreholes FC9-1 to FC9-3, FC9-RP1 and FC9-RP2 were advanced to depths of 14.5 to 20.8 m below existing grades. Dynamic cone penetration tests (DCPT) were carried out adjacent to some of the boreholes (FC7-1 to FC7-3, FC8-2, FC9-1 to FC9-3) or below the base of the borehole (FC7-1 to FC7-3). The locations of the boreholes at the sites are given on the Borehole Location Plan Drawing Nos. A1-1, A2-1 and A3-1.

The boreholes, except Boreholes FC8-1 and FC8-3, were advanced using a track-mounted drilling rig owned and operated by Landcore Drilling of Chelmsford, Ontario, under the full-time supervision of technical personnel from Coffey. These boreholes were advanced using continuous flight hollow-stem augers. In order to advance the boreholes through cobbles and boulders, rotary core drilling was carried out, where required, utilizing N-size casings and NQ core barrel. Bedrock was cored in Boreholes FC8-2, FC9-2 and FC9-3.

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

Boreholes FC8-1 and FC8-3 were advanced by manual sampling equipment. Samples were obtained continuously by a modification of the Standard Penetration method. The modified test consists of freely dropping a 31.75 kg hammer a vertical distance of 0.76m 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 was recorded and correlated to the Standard Penetration Resistance or the N-value of the soil by dividing the recorded value in half. This correlated N-value is an indication of the compactness condition of granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

In-situ shear vane tests were carried out within the cohesive soils to obtain an indication of the shear strength of the soil. The field vane shear tests were carried out with an MTO 'N' vane.

Dynamic Cone Penetration Tests (DCPT) were performed in Boreholes FC7-1, 2, 3, FC8-2, FC9-1, 2 and 3, from bottom of the boreholes or from the ground surface level. The DCPT consists of continuously driving a 60° point, 51 mm diameter cone attached to the drill rod, into the undisturbed ground with a driving energy of 475 kJ (63.5 kg hammer free falling for a distance of 0.76 m) per blow. The number of blows for each 0.3 m of penetration is recorded, providing an indication of the relative changes in the soil density with depth.

The borehole locations were established in the field by Coffey engineering staff, in relation to the existing features. The locations were then tied in and the geodetic elevations of the ground at the borehole locations were determined by the client's surveyors. This survey information was provided to us.

Groundwater conditions in the boreholes were observed during and on completion of drilling in the open boreholes. Upon their completion, the boreholes were grouted using a cement/bentonite mixture as per MTO procedures. A standpipe piezometer was installed in each of Boreholes FC7-3 and FC9-3, upon their completion.

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

## **4 SUMMARIZED SUBSURFACE CONDITIONS**

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. An inferred stratigraphic section and a profile at each culvert location are shown in Drawing Nos. 2, 3, 5, 6, 8 and 9. 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.

The summarized subsurface conditions for each culvert location are described in the following paragraphs.

### **4.1 Culvert C77**

The culvert boreholes were advanced from 20.0 m south (Borehole FC7-1) and 16.5 m north (Borehole FC7-3) of the existing road centreline of Highway 522. Borehole FC7-2 was advanced from the existing Highway 522 shoulder. Boreholes FC7-RP1 and FC7-RP2 were advanced from near the top of the highway embankment, west and east of the culvert location, respectively.

The ground surface elevations range from 300.3 to 306.4 m at the borehole locations (FC7-1 to FC7-3, FC7-RP1 and FC7-RP2) for Culvert C77. The existing top of highway embankment at the culvert location has an elevation 305.9 m, while the ground surface elevation at the boreholes advanced beyond the embankment range adjacent to the culvert range from 300.3 to 302.0 m.

Boreholes FC7-2, FC7-RP1 and FC7-RP2 were advanced from the top of the highway embankment and encountered a 5.3 to 6.0 m thick embankment fill. Below the fill and beyond the highway embankment (Boreholes FC7-1 and FC7-3), an interlayered silt, sandy silt to silty sand and sand deposit was encountered, with a discontinuous 2.6 to 6.1 m thick silty clay and an underlying gravelly sand to sand and gravel deposit. These are in turn underlain by a lower sand deposit. The groundwater table is close to the ground surface beyond the highway embankment and at elevation 301 to 302 m within the embankment fill.

#### **4.1.1 Fill**

Boreholes FC7-2, FC7-RP1 and FC7-RP2 were advanced from the top of the highway embankment and encountered fill to a depth of 5.3 to 6.0 m below grade, or to Elevation 300.0 to 300.6 m. The pavement fill consisted of a 0.3m thick layer of granular base over granular sub-base. Below the granular pavement fill, the embankment fill generally consisted of sand with trace to some silt and trace to some gravel. An

inferred boulder was also encountered in Borehole FC7-RP2 at the bottom of the fill. The lower portion of the fill embankment consists of a silty sand to silt to sandy silt below about Elevation 302 m.

The grain size distribution of two samples from within the upper zones of the fill (i.e. above El. 202.0 m) is presented in Figure B1-1 in Appendix B1. The results indicate the following grain-size distribution:

Gravel:	2 - 4%
Sand:	60 - 76%
Silt & Clay:	22 - 36%

Figure B1-2 presents the results of grain size analyses on two samples from the lower portion of the embankment fill. The following grain-size distribution is indicated:

Gravel:	0%
Sand:	54%
Silt:	38 - 39%
Clay:	7 - 8%

This embankment fill is a basically granular (i.e. non-cohesive) soil type.

Measured N-values range from 1 to 29 blows/0.3 m within the fill, while DCPT values of 2 to 37 blows/0.3 m were measured, indicating a relative density of very loose to compact. It is noted that the bottom 1.5 m portion of fill is in a very loose condition.

Measured moisture contents of 3 to 21% were obtained indicating a damp to wet condition.

#### **4.1.2 Upper Silts and Sands**

Boreholes FC7-1 and FC7-3 encountered 0.05 m of topsoil at the surface. Below the topsoil in these boreholes and below the fill in Boreholes FC7-2, FC7-RP1 and FC7-RP2, a fine grained granular soil deposit was contacted which consists of interlayered silt to silty sand & sandy silt, fine sand and sand to depths of 12.2 to 18.3 m below the ground surface or to Elevation 291.6 to 286.6 m. The upper zones of this deposit also contains traces of organics, rootlets, wood pieces. The presence of traces of some clay size particles was noted.

Three grain size analyses were carried out on representative samples of this deposit. The results are presented on the Record of Borehole sheets in Appendix A1, and the grain size curves are presented in Figures B1-3 and B1-4 in Appendix B1. The results indicate 0% gravel, 13 to 72% sand, 14 to 65% silt and 14 to 22% clay size particles.

N-values of 2 to 27 blows/0.3 m and DCPT values of 3 to 30 blows/0.3 m were measured, indicating a compactness condition of very loose to compact, but generally loose to compact. Measured moisture contents of 15 to 32% were obtained indicating a moist to wet condition.

#### **4.1.3 Silty Clay**

A reddish grey to grey silty clay deposit was encountered in Boreholes FC7-1, FC7-3 and FC7-RP1 at depths of 0.8 to 7.6 m or El. 300.0 - 298.8 m. This deposit was found to be 2.6 – 6.1 m thick and extended to depths 4.6 – 13.7 m (El. 297.4 – 292.7 m).

Atterberg Limits tests were performed on three samples from the deposit. As shown in Figure B1-5, these tests indicate the following index values:

Liquid Limit:	38 - 42%
Plastic Limit:	21 - 24%
Plasticity Index:	16 - 20%
Natural moisture content:	27 - 51%

The above values are characteristic of a clayey soil of intermediate plasticity.

Three grain size analyses were carried out on representative samples of the cohesive soil. The results are presented on the Record of Borehole sheets in Appendix A1, and the grain size curves are presented in Figure B1-6, Appendix B1. The results indicate 0% gravel, 2 to 10% sand, 30 to 51% silt and 39 to 67% clay size particles.

Measured N-values range from 3 to 11 blows/0.3 m. Field vane tests were carried out within this cohesive soil and the measured shear strengths range from 48 to greater than 100 kPa, with a sensitivity of 5.1 to 8.0. Based on these test results, the silty clay deposit is considered to have a consistency of firm to very stiff.

Based on the grain size and Atterberg test results together with a visual and tactile examination of the soil samples this deposit is considered to be considerably less pervious than the overlying and underlying soils.

#### **4.1.4 Gravelly Sand to Sand & Gravel**

Underlying the upper stratified silts and sands, Boreholes FC7-2, FC7-3 and FC7-RP2 contacted a 1.5 to 4.6 m thick coarser grained granular soil consisting of gravelly sand to sand & gravel. This deposit was contacted at depths of 13.7 to 18.3 m below the ground surface or at El. 291.6 – 286.6 m and extended to El. 290.0 – 282.0 m.

In this deposit, the measured N-values ranged from 15 to 36 blows/0.3 m which indicate a compact to dense condition.

This deposit is considered more pervious than the other soil types contacted at the site.

#### **4.1.5 Lower Sand**

Boreholes FC7-1, FC7-2, FC7-3 and FC7-RP2 encountered a lower sand deposit at depths of 12.2 to 19.8 m or at El. 290.0 – 282.0 m. The boreholes were terminated in this deposit at depths of between 15.9 and 26.5 m or at El. 289.4 – 278.5 m. Borehole FC7-RP1 was terminated at a depth of 15.9 m (El. 290.5 m) before contacting this deposit.

This is a granular (non-cohesive) soil deposit. N-values recorded in this deposit range from 14 to 73 blow/0.3 m indicating a compact to dense relative density, except in Borehole FC7-RP2 where an N-value of 7 blows was recorded. This latter value may however be due to disturbance while drilling.

#### **4.1.6 Groundwater Conditions**

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of each borehole. A standpipe piezometer was installed at Borehole FC7-3. The observations are shown on the individual Record of Borehole sheets.

The observed water level in the open boreholes on completion ranged from 0.6 to 14.4 m below grade, or elevation 290.9 to 301.4 m. It should be noted that these water levels had not stabilized and unlikely represent the actual groundwater table. A piezometer was installed within the gravelly sand of Borehole FC7-3. In the piezometer the water level was measured at 9.5 m or at El. 290.8 m. In our opinion this represents the lower water level (aquifer).

Based on the above measurements, measured moisture contents and observations of the recovered soil samples, the upper groundwater table is likely close to the ground surface (o.g.) beyond the highway embankment. Below the highway embankment the groundwater table is likely in the order of elevation 301 to 302 m.

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

## **4.2 Culvert C78**

The ground surface elevations range from 304.0 to 311.6 m at the borehole locations (FC8-1 to FC8-3, FC8-RP1 and FC8-RP2). The existing top of highway embankment at the culvert location has an elevation 310.9 m, while the ground surface elevation at the boreholes advanced beyond the embankment range adjacent to the culvert is about 304.0 m.

The culvert boreholes were advanced from 18.0 m south (Borehole FC8-1) and 20.0 m north (Borehole FC8-3) of the existing road centreline of Highway 522. Borehole FC8-2 was advanced from the existing Highway 522 shoulder. Boreholes FC8-RP1 and FC8-RP2 were advanced west and east of the culvert location, respectively.

Boreholes FC8-2, FC8-RP1 and FC8-RP2 were advanced from the top of the highway embankment and encountered embankment fill which extended to 3.4 to 7.8 m below the ground surface or to El. 306.5 – 303.4 m). Below the fill and beyond the highway embankment (Boreholes FC8-1 and FC8-3), a 0.4 to 2.0 m thick discontinuous silt deposit was encountered, followed by a 7.6 to more than 12 m thick silty clay deposit. In Boreholes FC8-2 and FC8-RP2, the silty clay is underlain by a silt deposit. In Borehole FC8-2, which was extended deeper, the silt deposit is 1.5 m thick, and is underlain by a 3.3 m thick basal sand. The basal sand is in turn underlain by bedrock at a depth of 23.1 m (Elevation 287.9 m) below existing grade. The groundwater table is likely close to the ground surface beyond the highway embankment and in the order of elevation 306 to 307 m within the embankment fill.

#### **4.2.1 Fill**

Boreholes FC8-2, FC8-RP1 and FC8-RP2 were advanced from the top of the highway embankment and encountered embankment fill to 3.4 to 7.6 m below grade (Elevation 303.4 to 306.5 m). Asphalt was encountered at the surface of Boreholes FC8-2 and FC8-RP2 with a thickness of 40 to 50 mm. Under the asphalt a 0.1 to 0.15 m thick granular base and 0.23 to 0.4 m thick granular sub-base course was encountered. Below the granular pavement fill, in Boreholes FC8-2 and FC8-RP2, and immediately below grade in Borehole FC8-RP1, the embankment fill generally consisted of a range of sand, silty sand, sandy silt and silt. The embankment fill is considered as a generally granular (non-cohesive) soil with occasional cohesive zones/pockets. The fill in Borehole FC8-RP1 contained trace topsoil pockets and decayed wood pieces.

The measured N-values range from 2 to 24 blows/0.3 m within the fill, while DCPT values of 5 to 29 blows/0.3 m were measured, indicating a compactness condition of very loose to compact. The measured N-values are however typically 2 to 10 blows/0.3 m indicating a generally very loose to loose condition. From these results it can be summarized that the embankment fill has not received a systematic compaction when it was first placed, especially in the lower zones. Measured moisture contents of 3 to 62% were obtained indicating a damp to wet condition. Higher moisture contents were measured where decayed wood pieces or topsoil were present.

Three grain size analyses were carried out on representative samples of the fill from Borehole FC8-2. The results are presented on the Record of Borehole sheets in Appendix A2, and the grain size curves are presented in Figures B2-1 and B2-2 in Appendix B2. The results indicate 0% gravel, 3 to 56% sand, 36 to 63% silt and 8 to 34% clay size particles.

#### **4.2.2 Upper Silts**

Borehole FC8-3 encountered a veneer of 0.05 m thick topsoil at the surface. Below the topsoil in Borehole FC8-3 and below the fill in Boreholes FC8-RP1 and FC8-RP2, a 0.4 to 2.0 m thick silt deposit was encountered to Elevation 303.4 to 306.1 m. This deposit contains some clay, sand and trace organics and rootlets. It is considered to be an essentially fine grained granular (i.e. non cohesive) soil.

Standard Penetration Tests yielded N-values of 3 to 14 blows/0.3 m, while DCPT gave 13 to 14 blows/0.3 m, indicating a very loose to compact, but generally very loose to loose condition. Measured natural moisture contents were 25 to 31%, indicating a wet condition.

#### **4.2.3 Silty Clay**

Below the topsoil, fill and/or the upper silt, the boreholes encountered a reddish grey to grey silty clay deposit. Boreholes FC8-1, FC8-3 and FC8-RP1 were terminated within this deposit at depths of 4.9 to 15.9 m, or Elevations 294.0 to 299.1 m, while in Boreholes FC8-2 and FC8-RP2, the deposit was found to be 7.6 to 10.7 m thick and extended to depths of 18.3 m and 15.2 m or to El. 292.7 m and 296.4 m, respectively.

Four grain size analyses were carried out on representative samples of this cohesive soil. The results are presented on the Record of Borehole sheets in Appendix A2, and the grain size curves are presented in



Figure B2-3 in Appendix B2. The results indicate 0% gravel, 1 to 9% sand, 44 to 60% silt and 39 to 47% clay size particles.

Atterberg Limits tests were performed on five samples from the deposit. These indicate the following index values (see Figure B2-4):

Liquid Limit:	32 - 42%
Plastic Limit:	17 - 23%
Plasticity Index:	14 - 19%
Natural moisture content:	29 - 59%

The above values are characteristic of a low to intermediate plasticity clay soil.

Measured N-values range from 1 to 12 blows/0.3m. Field vane tests were also carried out within this cohesive soil and the measured undrained, in-situ shear strengths range from 32 to in excess of 100 kPa, with a sensitivity of 2.6 to 7.3. Based on these test results the silty clay can be considered to have a consistency of soft to very stiff.

#### **4.2.4 Lower Silts and Sands**

Underlying the silty clay deposit, Boreholes FC8-2 and FC8-RP2, which were extended deeper, encountered a silt deposit at depths of 18.3 m (El. 292.7 m) and 15.2 m (El. 296.4 m), respectively. Borehole FC8-RP2 was terminated within the silt at a depth of 15.9 m (Elevation 295.7 m) after penetrating the deposit 0.7 m, while in Borehole FC8-2 the deposit was found to 1.5 m thick and extended to El. 291.2 m. The silt deposit contains some clay and is considered to be a generally cohesive material.

Measured N-values within this cohesive silt deposit are 6 and 16 blows/0.3 m, indicating a firm to very stiff consistency. Measured moisture contents range from 27 to 29%.

Underlying the lower silt deposit, Borehole FC8-2 encountered a 3.3 m thick basal sand deposit to a depth of 23.1 m (Elevation 287.9 m) or to the surface of bedrock. The sand contains traces of silt and a boulder was cored within this deposit, (immediately above the bedrock surface). This is a granular material.

An N-value of 11 blows/0.3 m was obtained in the deposit indicating a compact relative density. A moisture content of 20% was measured.

#### **4.2.5 Bedrock**

Borehole FC8-2 encountered gneiss bedrock at a depth of 23.1 m, or at Elevation 287.9 m. The bedrock was cored with a length of 3.3 m. The formation belongs to the Pre-Cambrian Era.

The rock core recovery (T.C.R.) in the bedrock was 100% over three rock core runs, and a Rock Quality Designation (R.Q.D.) value between 94 and 100% was measured. These results indicate a sound rock with excellent rock quality.

#### **4.2.6 Groundwater Conditions**

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

The observed water levels in the open boreholes on completion ranged from 0.6 to 3.7 m below grade, or elevation 303.4 to 307.3 m. Boreholes FC8-RP1 and FC8-RP2 were dry on completion. It should be noted that these water levels had not stabilized and unlikely represent the actual groundwater table.

Based on the above measurements, measured moisture contents and observations of the recovered soil samples, the groundwater table at the time of our investigation was likely close to the ground surface beyond the highway embankment (i.e. between El. 303 and 304 m). Below the highway embankment the groundwater table was likely in the order of elevation 304 to 305 m in Borehole FC8-2 and between El. 305 and 307 m in Boreholes FC8-RP1 and FC8-RP2, at the time of our investigation.

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.

### **4.3 Culvert C79**

At Culvert C79 site, the ground surface elevations m at the borehole locations range from 309.9 to 314.4 m. The existing top of highway embankment at the culvert location has an elevation 314.4 m, while the ground surface elevation at the boreholes advanced near or beyond the embankment range adjacent to the culvert range from 309.9 to 310.8 m.

The culvert boreholes were advanced from 11.5 m south (Borehole FC9-1) and 15.0 m north (Borehole FC9-3) of the existing road centreline of Highway 522. Borehole FC9-2 was advanced from the existing Highway 522 shoulder. Boreholes FC9-RP1 and FC9-RP2 were advanced west and east of the culvert location, respectively.

Boreholes FC9-2, FC9-RP1 and FC9-RP2 were advanced from the top of the highway embankment and encountered a 2.1 to 4.7 m thick embankment fill extending to El. 312.1 – 309.6 m. Borehole FC9-1 was advanced near the base of the embankment and encountered 1.1 m of sand fill. Below the fill and beyond the highway embankment, a 0.7 to 1.4 m thick silt to deposit was encountered, followed by a 7.6 to more than 12.8 m thick silty clay deposit. The silty clay is underlain by a lower silt deposit. In the deeper boreholes, a 0.8 to 1.5 m thick basal sand to sand & gravel deposit was contacted. In Boreholes FC9-2 and FC9-3 bedrock was proven by coring, where the surface of the bedrock was contacted at a depth of 12.2 to 17.7 m (Elevation 296.6 to 297.7 m) below existing grade. The groundwater table is likely close to the original ground surface (o.g. level) beyond the highway embankment (i.e. about El. 310 m) and in the order of elevation 310 to 311 m within the embankment fill.

#### **4.3.1 Fill**

Boreholes FC9-2, FC9-RP1 and FC9-RP2 were advanced from the top of the highway embankment and encountered fill to 2.1 to 4.7 m below grade (Elevation 309.6 to 312.1 m). The pavement fill consisted of a 0.2 to 0.3 m thick layer of granular base over a 0.5 to 0.7 m thick granular sub-base. The grain-size distribution of a sample from the granular pavement fill is given in Figure B3-1 in Appendix B3.

Borehole FC9-1 was advanced near the toe of the embankment and encountered 0.1 m of topsoil over a 1.0 m thick uncompacted sand fill. This is a basically granular (i.e. non-cohesive) soil and based on N-values of 2 and 4 blows/0.3m, it is considered to be in a very loose condition.

Below the granular pavement fill, the embankment fill in Boreholes FC9-2, FC9-RP1 and FC9-RP2 generally consisted of sand with trace to some silt and trace to some gravel, with occasional clayey silt interbeds. It is a basically granular soil (i.e. non-cohesive) with occasional cohesive zones/pockets. Measured N-values range from 4 to 9 blows/0.3 m were obtained within the fill and DCPT values of 2 to 7 blows/0.3 m were measured, indicating a very loose to compact relative density. Moisture contents of 3 to 21% were measured in the laboratory indicating a damp to wet condition. Grain size analysis was carried out on three samples of the fill. The results are presented on the Record of Borehole sheets in Appendix A3, and the grain size curves are presented in Figure B3-2 Appendix B3. The results indicate 0-1% gravel, 67 - 82% sand and 17 - 33% silt and clay size particles.

#### **4.3.2 Upper Silts**

Boreholes FC9-1 and FC9-3 encountered 0.1 m of topsoil at the surface. A 0.05 m thick topsoil layer was also contacted in Borehole FC9-2, underlying the embankment fill. Below the topsoil and fill, all five boreholes encountered a 0.7 to 1.4 m thick silt deposit to Elevation 308.2 to 311.3 m. This deposit contains trace to some clay, trace to some organics, trace gravel, sand and rootlets. In Borehole FC9-1, the deposit was found to consist primarily of sandy silt. Depending on the clay content, the deposit changes from a cohesive (non-granular) to a basically a fine grained granular (non-cohesive) material.

Measured N-values of 1 to 10 blows/0.3 m, while DCPT values of 0 to 6 blows/0.3 m were measured, indicating a compactness condition of very loose to loose or a consistency of soft to stiff. Natural moisture contents of 19 to 60% were obtained indicating a moist to wet condition. The higher moisture contents reflect the amount of organics within the soil.

Three grain size analyses were carried out on representative samples of the silt. The results are presented on the Record of Borehole sheets in Appendix A3, and the grain size curves are presented in Figure B3-3 in Appendix B3. The results indicate 0% gravel, 26 to 48% sand, 34 to 54% silt and 18 to 24% clay size particles.

#### **4.3.3 Silty Clay**

Underlying the upper silt, a 7.6 to in excess of 12.8 m thick silty clay deposit was encountered to depths of 9.1 to 13.7 m below grade (or Elevation 300.1 to 300.8 m). Borehole FC9-RP2 was terminated within this deposit at a depth of 15.9 m, or at Elevation 298.5 m.

This is a cohesive material. Six grain size analyses were carried out on samples of the cohesive soil. The results are presented on the Record of Borehole sheets in Appendix A, and the grain size curves are presented in an envelope form in Figure B3-4 in Appendix B3. The results indicate the following distribution.

Gravel:	0%
Sand:	3 to 9%
Silt:	48 to 61%
Clay:	36 to 44%

Atterberg Limits tests were performed on seven samples from the deposit. As shown in Figure B3-5, these indicate the following index values:

Liquid Limit:	30 to 43%
Plastic Limit:	18 to 21%
Plasticity Index:	10 to 22%
Natural moisture content:	28 to 45%

The above values are characteristic of a low to intermediate plasticity clay soil.

Standard Penetration tests performed in the deposit yielded N-values which range from 1 to 13 blows/0.3 m. Field vane tests were carried out within this cohesive soil and the measured undrained shear strengths range from 28 to greater than 100 kPa, with a sensitivity of 2.2 to 6.3. Based on these test results the silty clay can be considered to have a consistency of soft to very stiff.

#### **4.3.4 Lower Silts**

Underlying the silty clay deposit, Boreholes FC9-1, FC9-2, FC9-3 and FC9-RP1 encountered a 1.6 to 3.1 m thick silt deposit to a depth of 10.7 to 16.8 m below grade or to Elevation 297.1 to 299.2 m. Borehole FC9-RP1 was terminated within this deposit at a depth of 15.9 m, or at Elevation 298.0 m.

The silt deposit contains trace to some clay. The deposit is considered to be a basically cohesive soil except in Borehole FC9-2 where the sandy silt layer below El. 298.6 m is described as a fine grained granular soil. Measured N-values within this deposit range from 3 to 18 blows/0.3 m. These results indicate a soft to very stiff consistency. Measured natural moisture contents range from 18 to 30%.

#### **4.3.5 Gravels and Sands**

Boreholes FC9-1, FC9-2 and FC9-3 (which were extended relatively deeper) encountered below the silt deposit, a basal sand to silty sand to sand & gravel deposit at depths 10.7-16.8 m or El. 299.2-297.1 m. In Boreholes FC9-2 and FC9-3, these granular (i.e. non-cohesive), 0.9 to 1.5 m thick, basal soils were found to extend to a depth of 12.2 to 17.7 m, or at Elevations 296.6 to 297.7 m. Borehole FC9-1 was terminated within these deposits at a depth of 14.5 m, or at Elevation 296.3 m. N-values recorded within these deposits range from 16 to 63 blows/0.3 m, indicating a compactness condition of compact to very dense. Measured moisture contents range from 7 to 23%.

#### 4.3.6 Bedrock

Bedrock was encountered and cored in Boreholes FC9-2 and FC9-3, with auger refusal encountered on probable bedrock in Borehole FC9-1, as follows:

Borehole No.	Ground Elevation (m)	Overburden Depth to the Surface of Bedrock (m)	Elevation of the Surface of Bedrock (m)
FC9-1	310.8	14.5*	296.3*
FC9-2	314.3	17.7	296.6
FC9-3	309.9	12.2	297.7

\* Inferred

The bedrock was identified as gneiss. The formation belongs to the Pre-Cambrian Era. The bedrock in Boreholes FC9-2 and FC9-3 was cored a length of 2.9 to 3.1m

A rock core recovery (TCR) in the bedrock of 100% was measured, with Rock Quality Designation (R.Q.D.) values generally between 80 and 100%, except for the upper 0.6 m of the rock cores obtained from Borehole FC9-2 where an RQD of 32% was measured. These results indicate a relatively sound rock with an excellent rock quality except for the upper 0.6 m of the bedrock in Borehole FC9-2 where poor rock quality is inferred.

#### 4.3.7 Groundwater Conditions

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of each borehole. One standpipe piezometer was installed at Borehole FC9-3. The observations are shown on the individual Record of Borehole sheets.

The observed water level in the open boreholes on completion ranged from 1.1 m to 3.7 m below grade, or at elevation 307.1 to 310.6 m. It should be noted that these water levels had not stabilized and unlikely represent the actual groundwater table. A piezometer was installed within the bedrock at Borehole FC9-3. In the piezometer the water level was measured at 12.2 m (El. 297.7 m) or at the interface of overburden soils with bedrock.

Based on the above measurements, measured moisture contents and observations of the recovered soil samples, at the time of our investigation the groundwater table was likely close to the original ground surface (o.g.) beyond the highway embankment, while below the highway embankment it was likely in the order of elevation 310.5 to 311.5 m.

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

For and on behalf of Coffey Geotechnics Inc.



**Gwangha Roh, Ph.D.**



**Ramon Miranda, P.Eng.**



**Zuhtu Ozden, P.Eng.**



Drawings



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

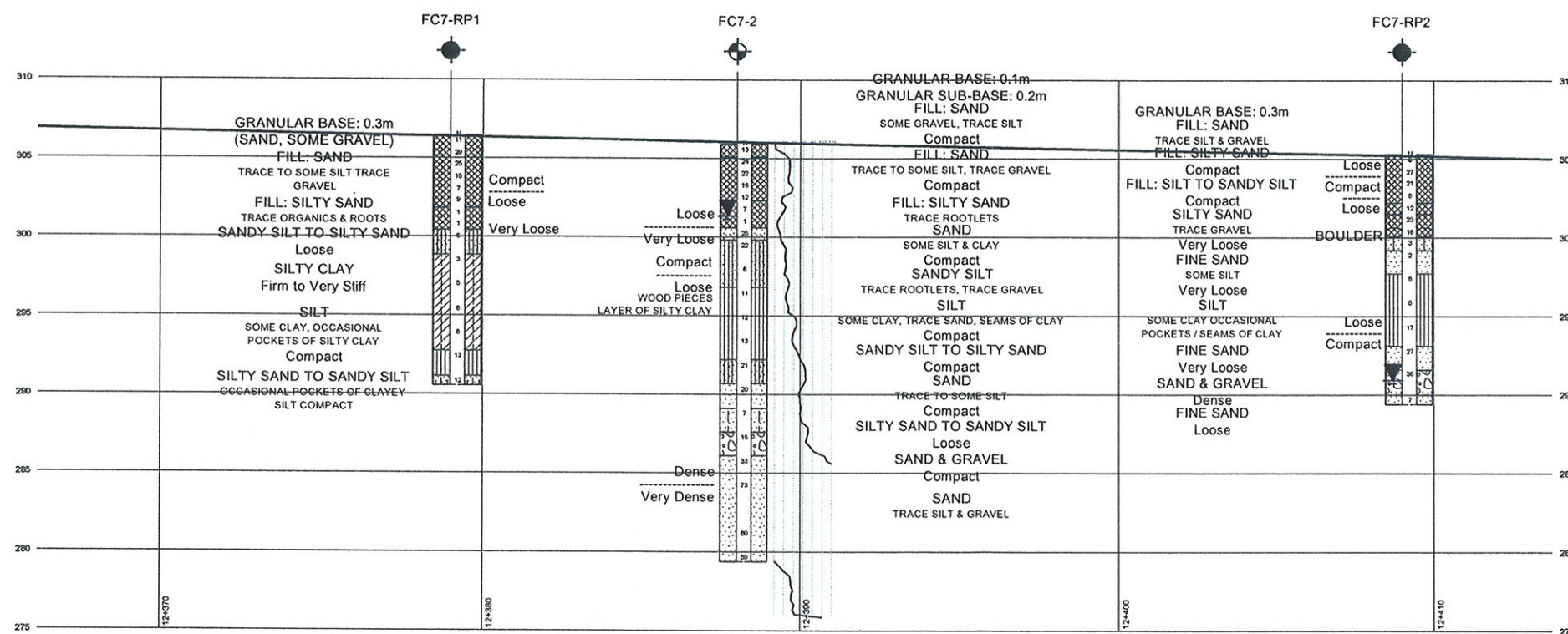
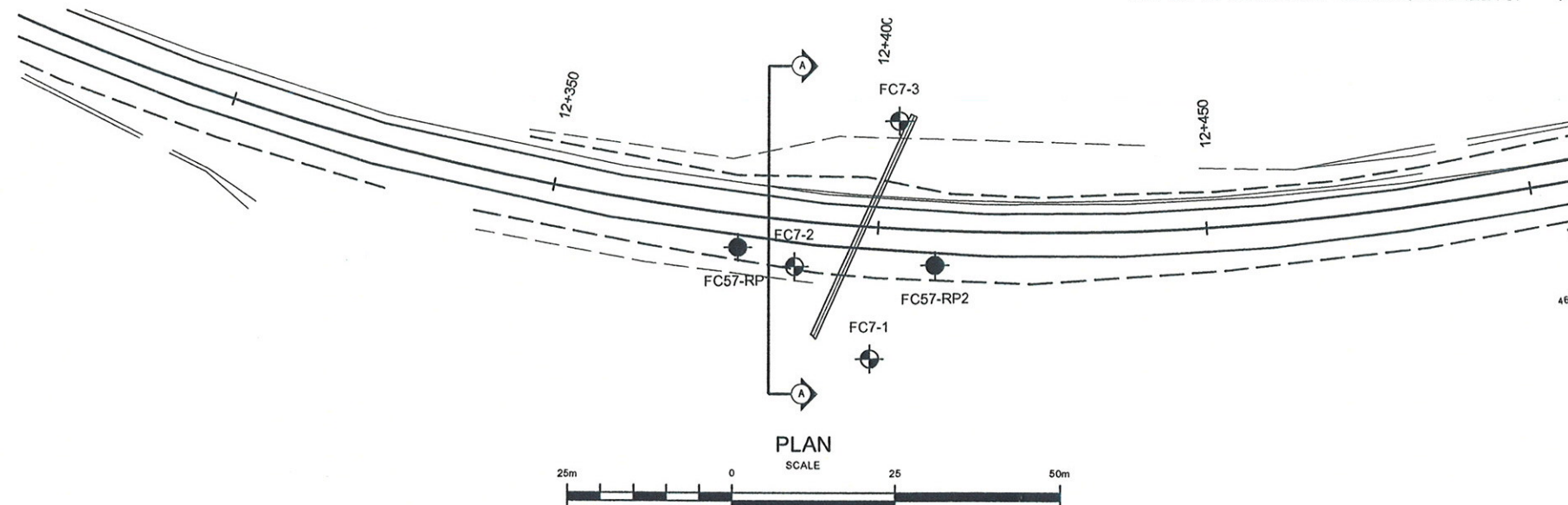
GWP: 484-98-00

HIGHWAY 522, TROUT CREEK  
CULVERT C77 @ 12+398  
BOREHOLE LOCATION PLAN AND PROFILE



SHEET

FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

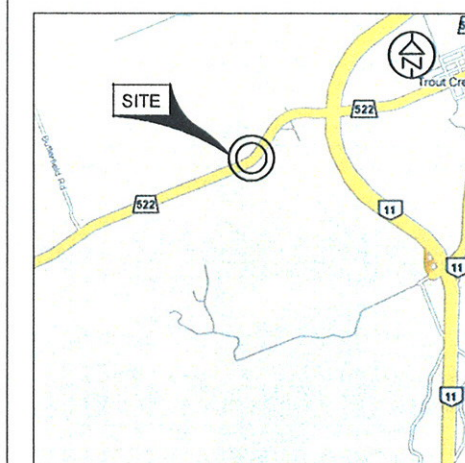


### Q PROFILE

HORIZONTAL SCALE

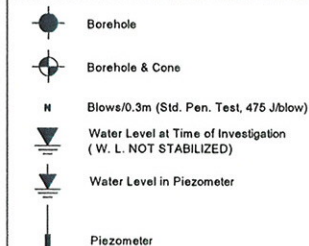


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KEY PLAN  
N.T.S.

## LEGEND



No.	ELEVATION	STATION	OFFSET
FC7-1	302.0	12+400	20.0m Rt C/
FC7-2	305.9	12+388	7.0m Lt C/
FC7-3	300.3	12+402	16.5m Lt C/
FC7-RP1	306.4	12+379	5.0m Rt C/
FC7-RP2	305.3	12+409	5.0m Rt C/

-NOTE-

**-NOTE-**

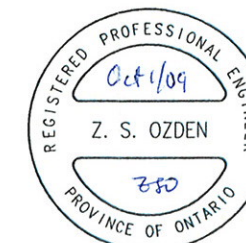
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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

REVISIONS			
	DATE	BY	DESCRIPTION

Geocres No 31E-293

TRANETOBO1221AC				DIST	
SUBM'D		CHECKED	DATE	Sep. 2000	SITE
DRAWN	PHK	CHECKED RM	APPROVED	ZO	DWG A1-1





METRIC

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

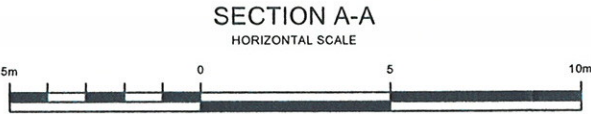
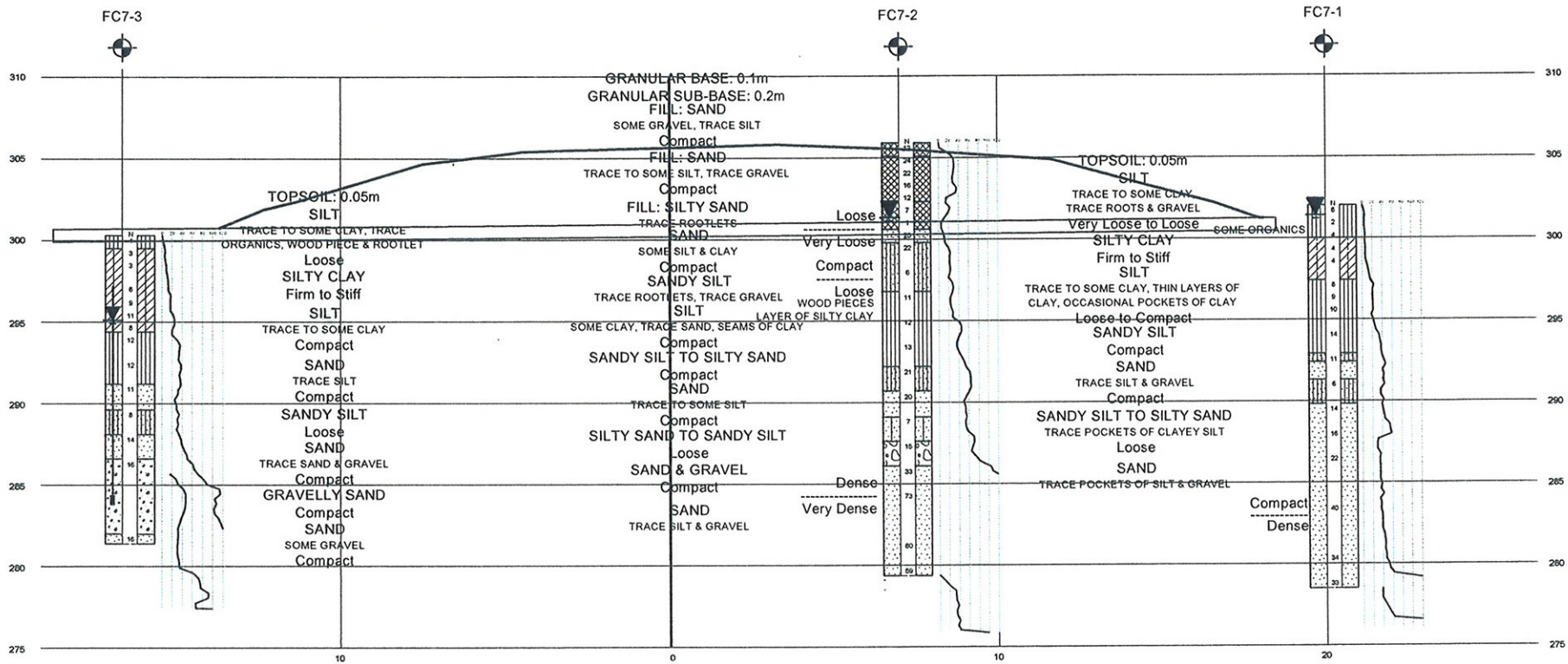
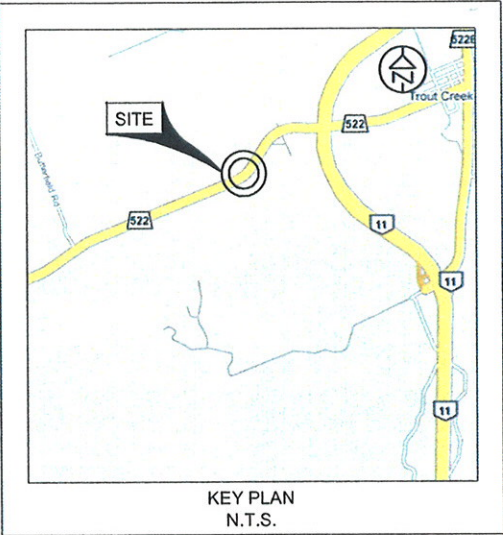
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OTHERWISE SHOWN. STATIONS  
ARE IN KILOMETRES + METRES.

CONT No.  
GWP: 484-98-00

HIGHWAY 522, TROUT CREEK  
CULVERT C77 @ 12+398  
CROSS SECTION

SHEET

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LEGEND			
	Borehole		
	Borehole & Cone		
	Blows/0.3m (Std. Pen. Test, 475 Jblow)		
	Water Level at Time of Investigation (W. L. NOT STABILIZED)		
	Water Level in Piezometer		
	Piezometer		
No.	ELEVATION	STATION	OFFSET
FC7-1	302.0	12+400	20.0m Rt C/L
FC7-2	305.9	12+388	7.0m Lt C/L
FC7-3	300.3	12+402	16.5m Lt C/L

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

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REVISIONS		
DATE	BY	DESCRIPTION

Geocres No 31E-293			
TRANETOB01221AC		DIST	
SUBMD	CHECKED	DATE	Sep. 2009
DRAWN	PHK	CHECKED	RM
APPROVED	ZO	DWG	A1-2



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.

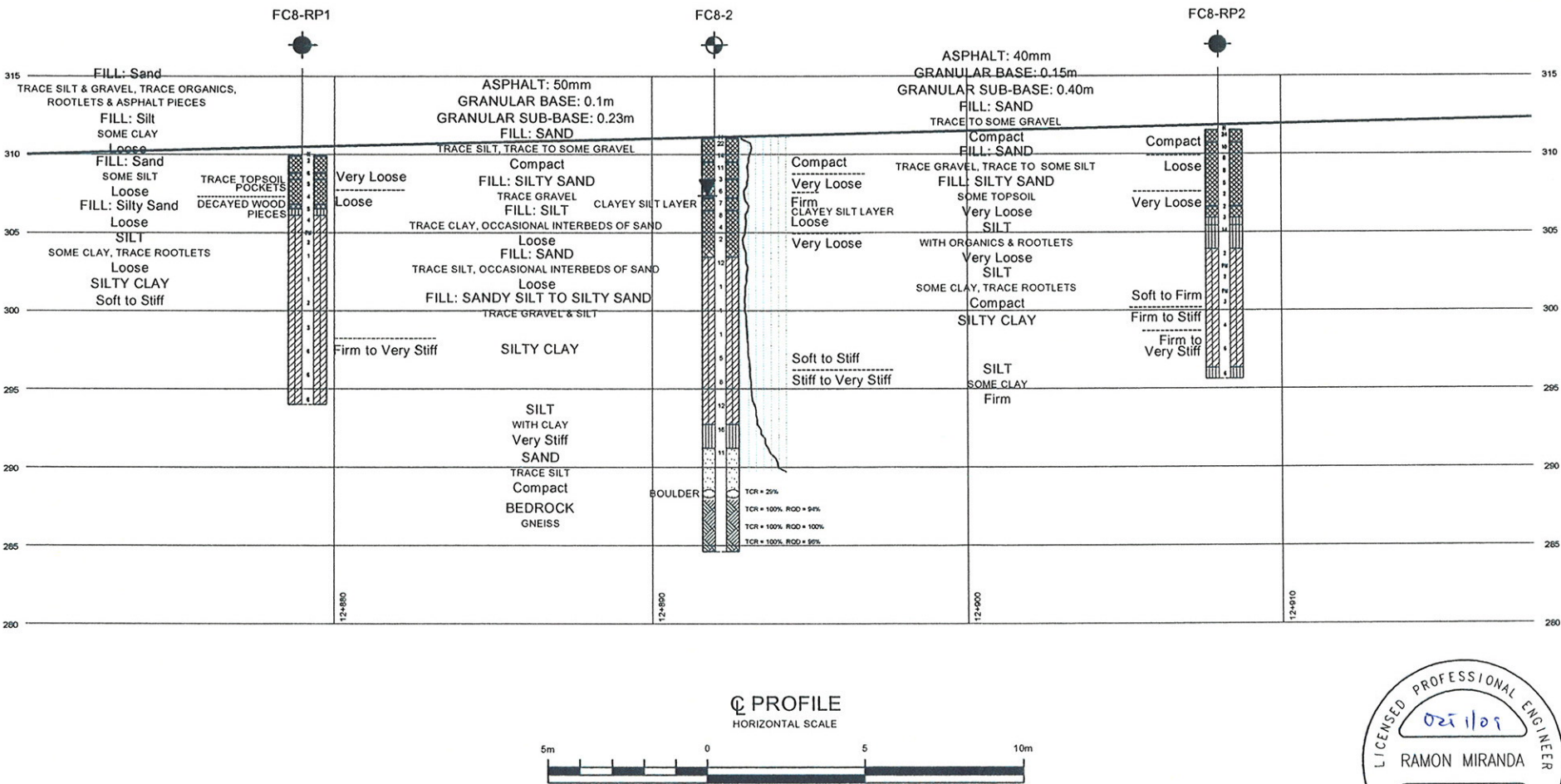
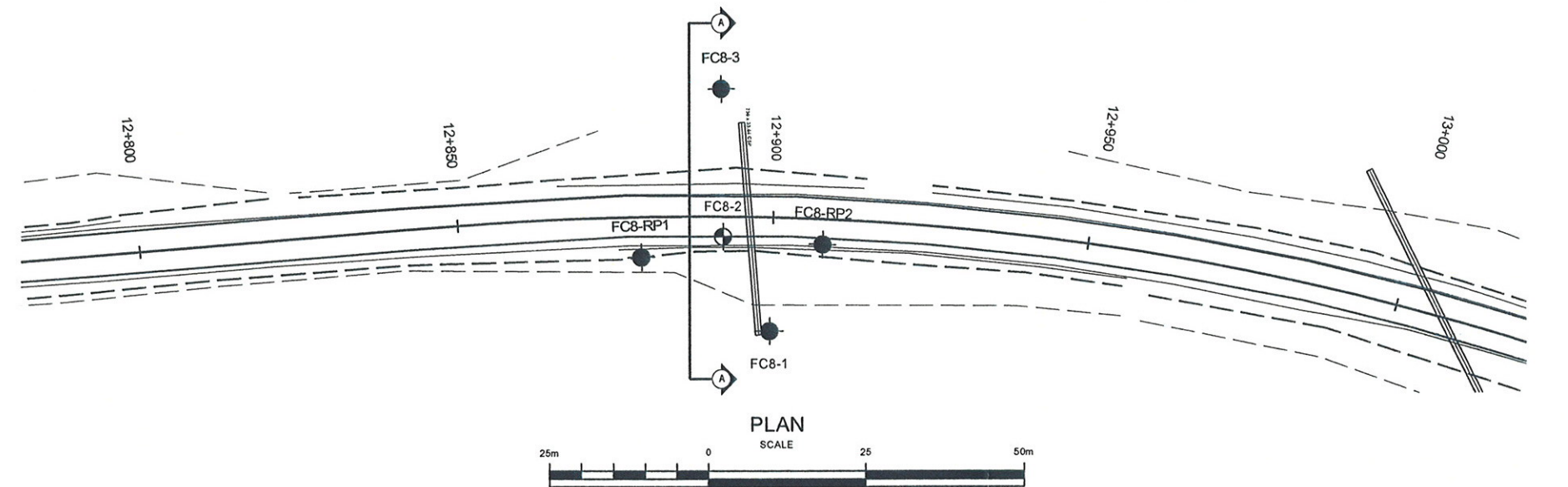
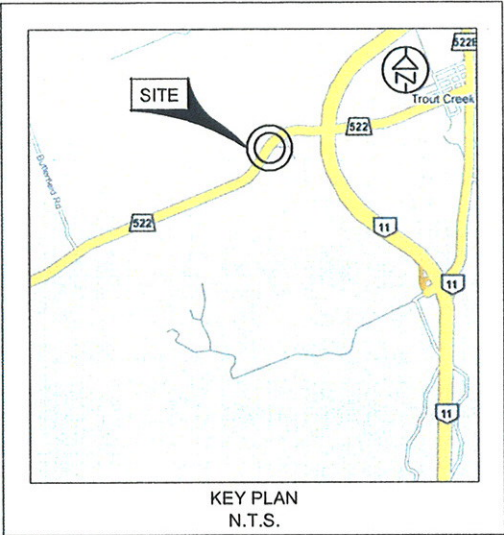
CONT No.  
GWP: 484-98-00

HIGHWAY 522, TROUT CREEK  
CULVERT C78 @ 12+896  
BOREHOLE LOCATION PLAN AND PROFILE



SHEET

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SPECIALISTS MANAGING THE EARTH



LEGEND			
	Borehole		
	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.	ELEVATION	STATION	OFFSET
FC8-1	304.0	12+900	18.0m Rt C/L
FC8-2	311.0	12+892	3.2m Rt C/L
FC8-3	304.0	12+891	20.0m Lt C/L
FC8-RP1	309.9	12+879	6.5m Rt C/L
FC8-RP2	311.6	12+908	4.0m Rt C/L

-NOTE-  
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REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31E-293			
TRANETO01221AC			
SUBMD	CHECKED	DATE	Sep. 2009
DRAWN	PHK	CHECKED	RM
APPROVED	ZO	DWG	A2-1





METRIC

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

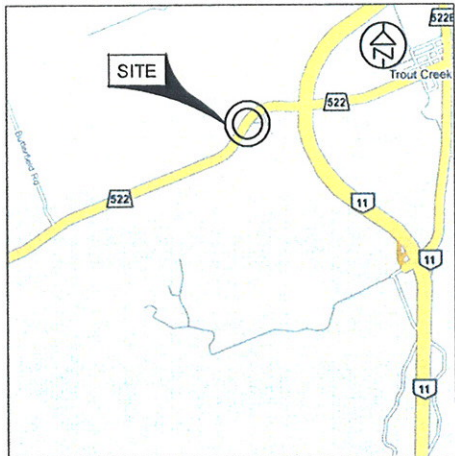
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CONT No.  
GWP: 484-98-00

HIGHWAY 522, TROUT CREEK  
CULVERT C78 @ 12+896  
CROSS SECTION

SHEET

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SPECIALISTS MANAGING THE EARTH



KEY PLAN  
N.T.S.

LEGEND

- Borehole
- 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.	ELEVATION	STATION	OFFSET
FCB-1	304.0	12+900	18.0m Rt C/L
FCB-2	311.0	12+892	3.2m Rt C/L
FCB-3	304.0	12+891	20.0m Lt C/L

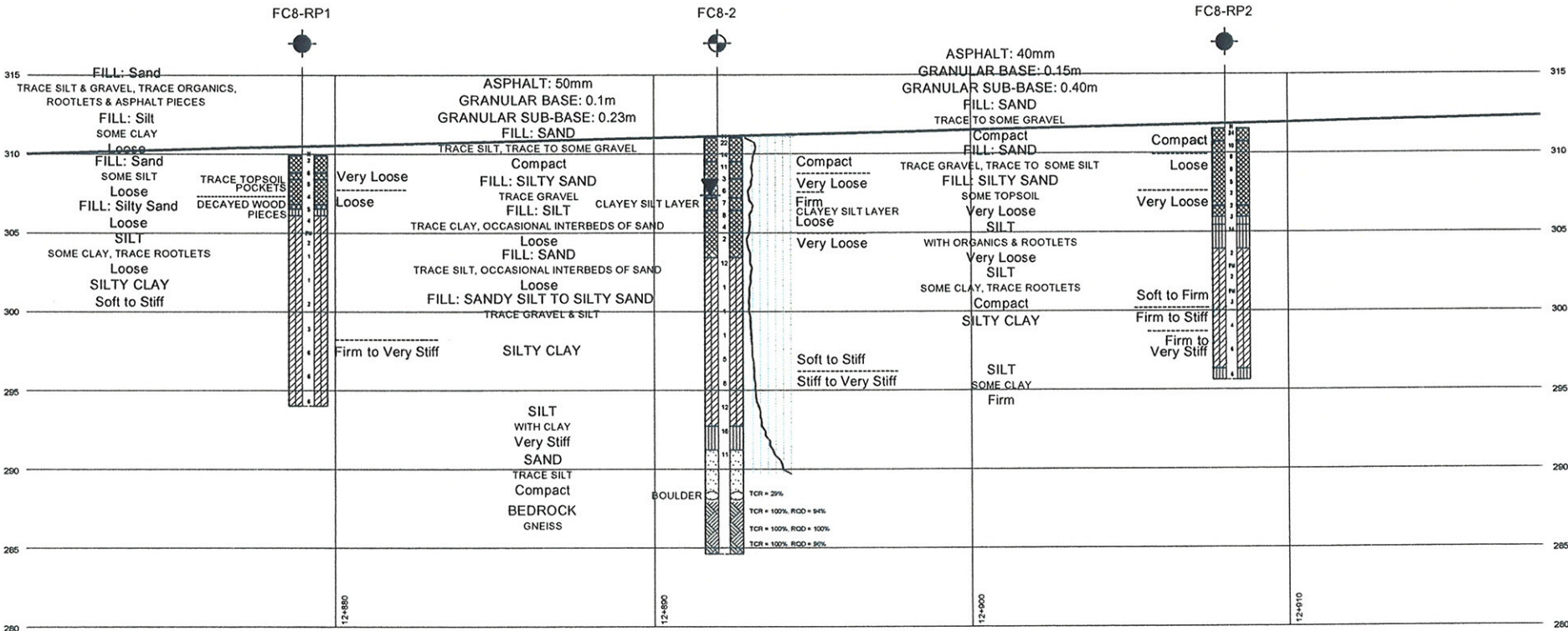
-NOTE-

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REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31E-293						
TRANETO01221AC					DIST	
SUBM'D		CHECKED		DATE Sep. 2009		SITE
DRAWN PHK		CHECKED RM		APPROVED ZO		DWG A2-2



SECTION A-A  
HORIZONTAL SCALE



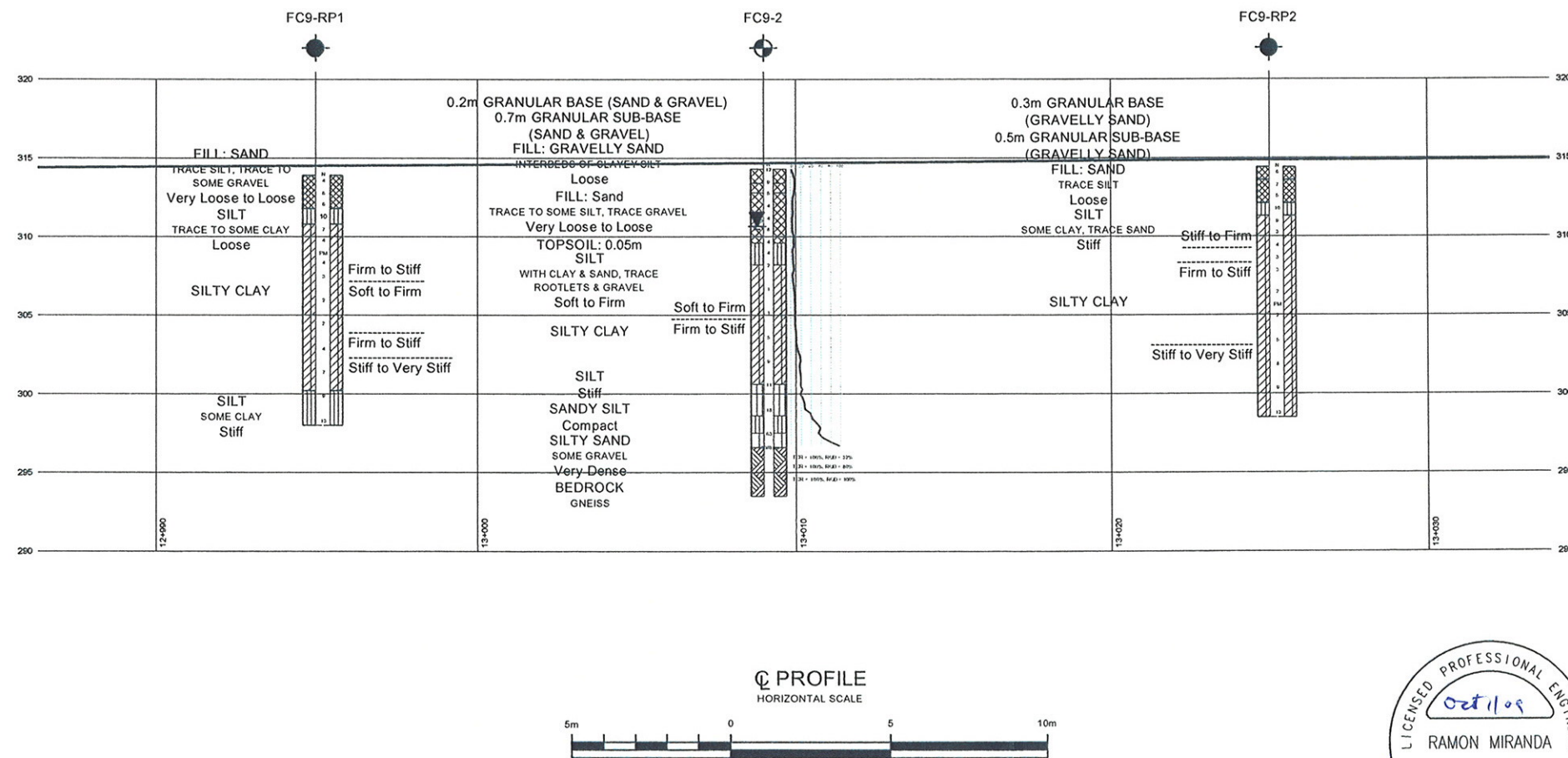
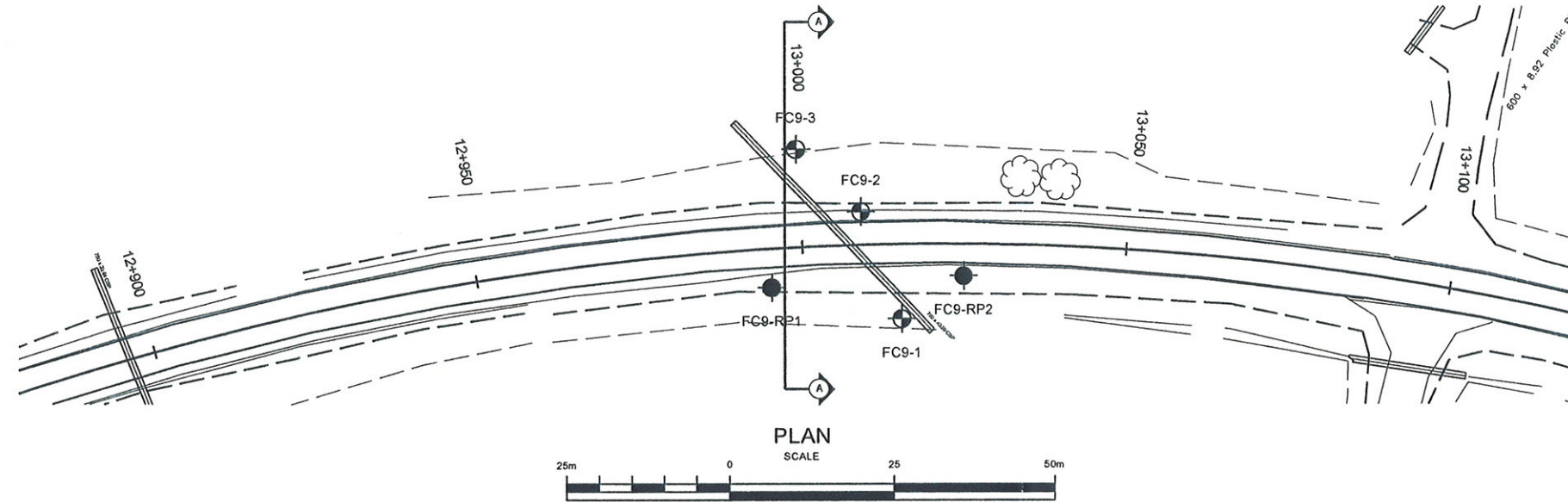


**METRIC**

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.



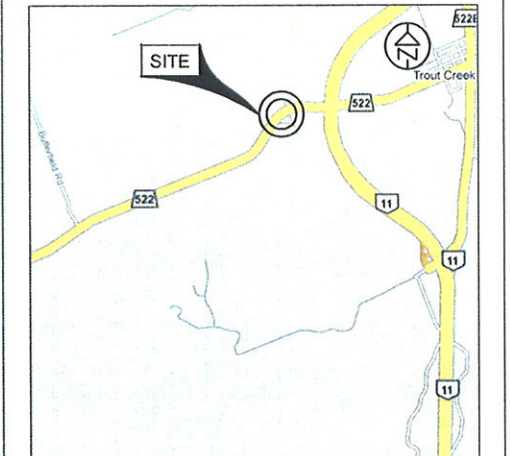
CONT No.  
GWP: 484-98-00

HIGHWAY 522, TROUT CREEK  
CULVERT C79 @ 13+010  
BOREHOLE LOCATION PLAN AND PROFILE



SHEET

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SPECIALISTS MANAGING THE EARTH



KEY PLAN  
N.T.S.

LEGEND

- Borehole
- 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.	ELEVATION	STATION	OFFSET
FC9-1	310.8	13+015	11.5m Rt C/L
FC9-2	314.3	13+009	5.0m Rt C/L
FC9-3	309.9	13+000	15.0m Lt C/L
FC9-RP1	313.9	12+995	6.0m Rt C/L
FC9-RP2	314.4	13+025	5.0m Rt C/L

**-NOTE-**

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REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31E-293			
TRANETOB01221AC			
SUBM'D	CHECKED	DATE	Sep. 2009
DRAWN	PHK	CHECKED	RM
APPROVED		20	DWG
A3-1		DIST	





METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

DIMENSIONS ARE IN METRES  
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OTHERWISE SHOWN. STATIONS  
ARE IN KILOMETRES + METRES.

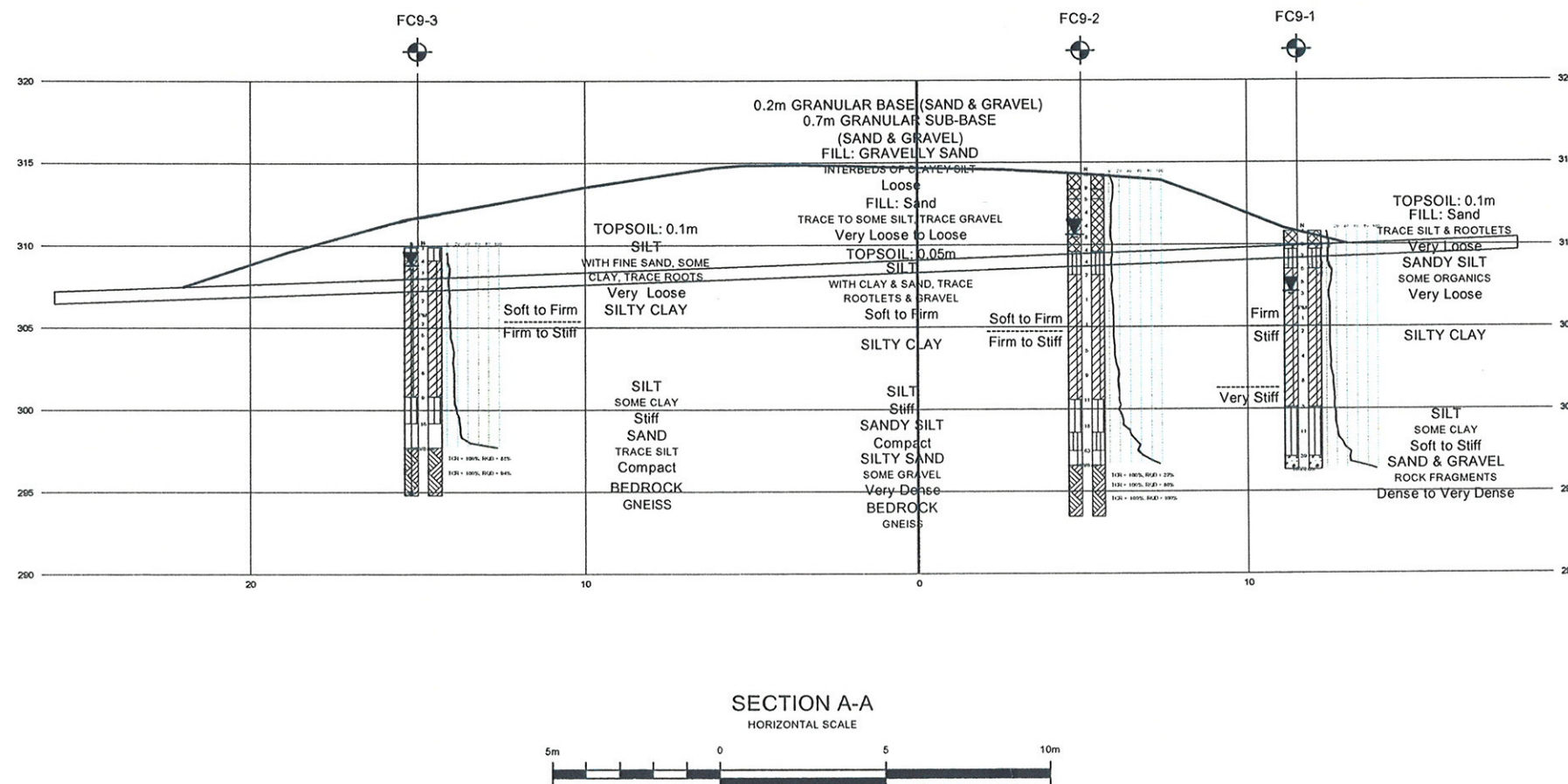
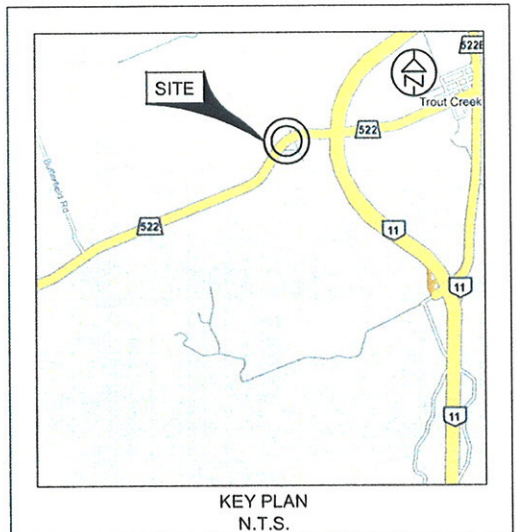
CONT No.

GWP: 484-98-00

HIGHWAY 522, TROUT CREEK  
CULVERT C79 @ 13+010  
CROSS SECTION

SHEET

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SPECIALISTS MANAGING THE EARTH



LEGEND

- Borehole
- 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.	ELEVATION	STATION	OFFSET
FC9-1	310.8	13+015	11.5m Rt C/L
FC9-2	314.3	13+009	5.0m Rt C/L
FC9-3	309.9	13+000	15.0m Lt C/L

-NOTE-

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31E-293			
TRANETO801221AC			
SUBM'D	CHECKED	DATE	SITE
DRAWN	PHK	APPROVED	DWG



# Appendix A1

**Record of Borehole Sheets for Culvert C77**

TRANETO01221AC

# RECORD OF BOREHOLE No FC7-1

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+400, 20.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger, NW Casing & DCPT COMPILED BY SD  
 DATUM Geodetic DATE 7/24/2008 7/29/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
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 W <sub>P</sub> W W <sub>L</sub> PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT WATER CONTENT (%)				
302.0	GROUND SURFACE										
0.0	0.05 m TOPSOIL		1	SS	6						0 16 64 20
	SILT trace to some clay trace roots & gravel greyish brown, moist v. loose to loose  some organics at 1.2 m		2	SS	2						0 13 65 22
			3	SS	4						
300.0											
2.0	SILTY CLAY reddish grey, firm to stiff		4	SS	4						0 2 41 57
			5	SS	4						
			6	TW	PM						
297.4											
4.6	SILT trace to some clay thin layers of clay, occ. pockets of clay grey, wet, loose to compact		7	SS	8						
			8	SS	9						
			9	SS	10						
			10	SS	14						
292.9											
9.1	SANDY SILT		11	SS	11						
292.4	greyish brown, wet, compact										
9.6	SAND trace silt & gravel greyish brown, wet, compact										
291.3											
10.7	SANDY SILT TO SILTY SAND trace pockets of clayey silt pinkish grey, wet, loose		12	SS	6						spoon wet
289.8			13	SS	14						
12.2											
	SAND trace pockets of silt & gravel pinkish brown, wet		14	SS	16						

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15 10 5  
 (%) STRAIN AT FAILURE

TRANETOB01221AC

# RECORD OF BOREHOLE No FC7-1

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+400, 20.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger, NW Casing & DCPT COMPILED BY SD  
 DATUM Goodotic DATE 7/24/2008 7/29/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		WATER CONTENT (%)			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>		
287.0	<p><b>SAND</b> trace pockets of silt &amp; gravel pinkish brown, wet</p> <p>compact dense</p>		15	SS	22								<p>wash boring used for further drilling</p>	
278.5			16	SS	40								<p>casing stuck abandon borehole</p>	
278.5														
278.5														
278.5														
278.5														
278.5														
278.5	<p>End of borehole Water Level @ 0.6 m (not stabilized)* upon completion Hole caved-in @ 2.1 m upon completion End of DCPT DCPT performed adjacent to borehole from ground to 24.0 m</p>													
276.4	<p>End of DCPT DCPT performed from bottom of borehole to 25.6 m</p>													

+<sup>3</sup> ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



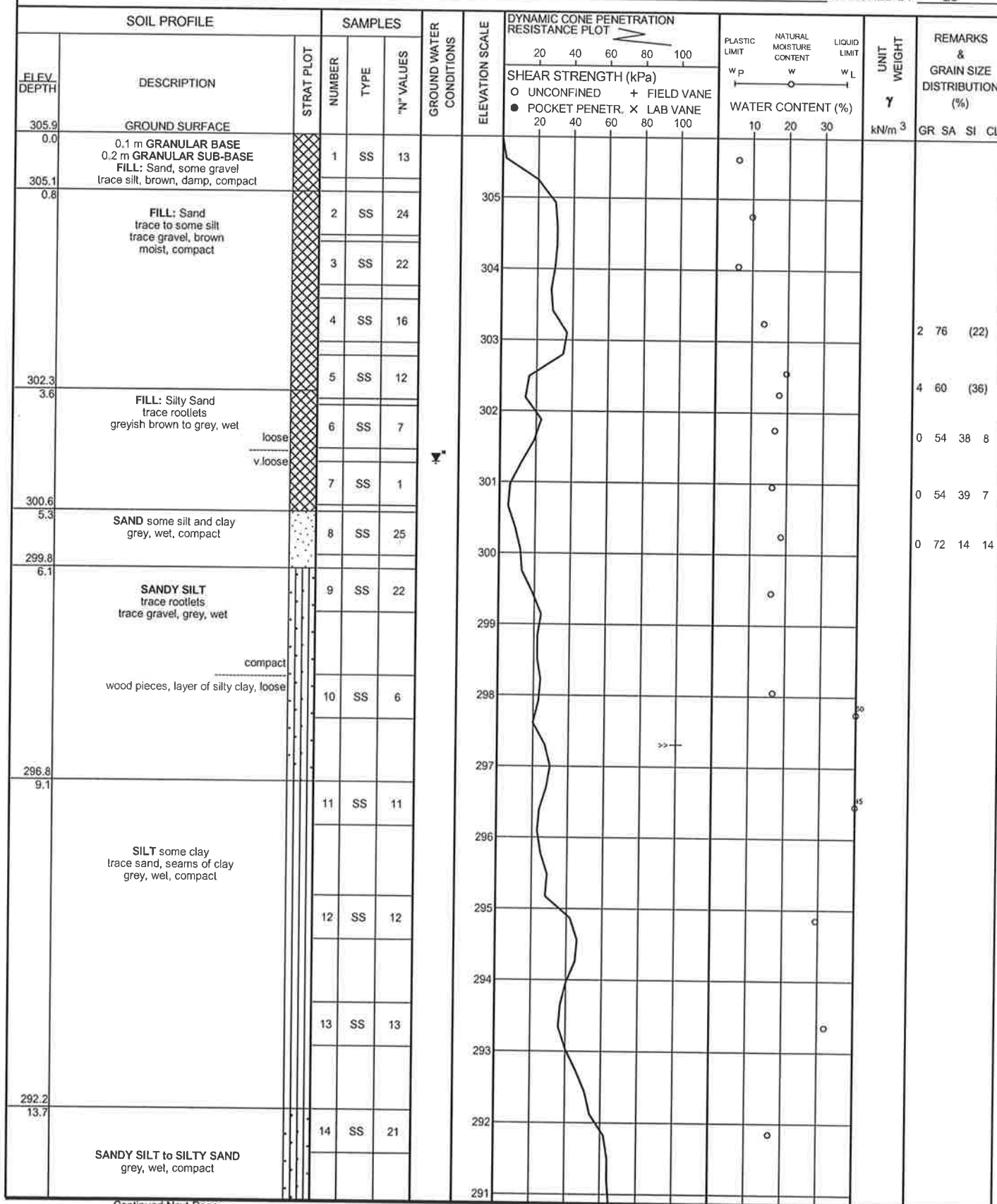
TRANETOBO1221AC

# RECORD OF BOREHOLE No FC7-2

1 OF 3

METRIC

GWP 484-98-00 LOCATION Sta : 12+388, 7.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST            HWY 522 BOREHOLE TYPE Hollow Stem Auger, NW Casing & DCPT COMPILED BY SD  
 DATUM Geodetic DATE 8/14/2008 8/19/2008 CHECKED BY ZO



Continued Next Page

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

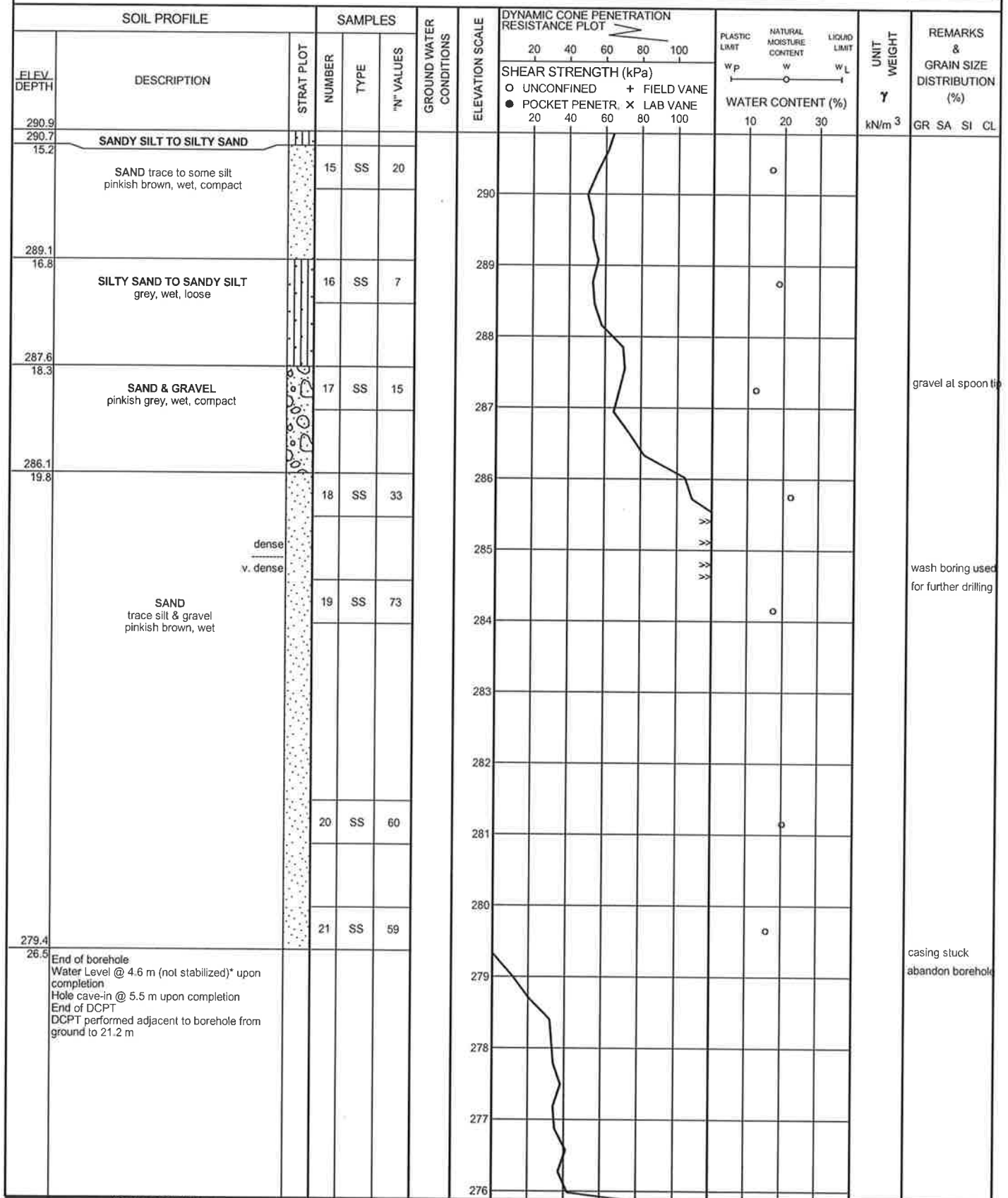
TRANETOB01221AC

# RECORD OF BOREHOLE No FC7-2

2 OF 3

METRIC

GWP 484-98-00 LOCATION Sta : 12+388, 7.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
DIST            HWY 522 BOREHOLE TYPE Hollow Stem Auger, NW Casing & DCPT COMPILED BY SD  
DATUM Geodetic DATE 8/14/2008 8/19/2008 CHECKED BY ZO



Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity 20 15 10 (%) STRAIN AT FAILURE

TRANETOB01221AC

## 3 OF 3

METRIC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
FLYV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES			
275.9							SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE	W.P. — W — W.L.		
275.8							20 40 60 80 100	WATER CONTENT (%) 10 20 30		

[illegible]

+ 3, X 3: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

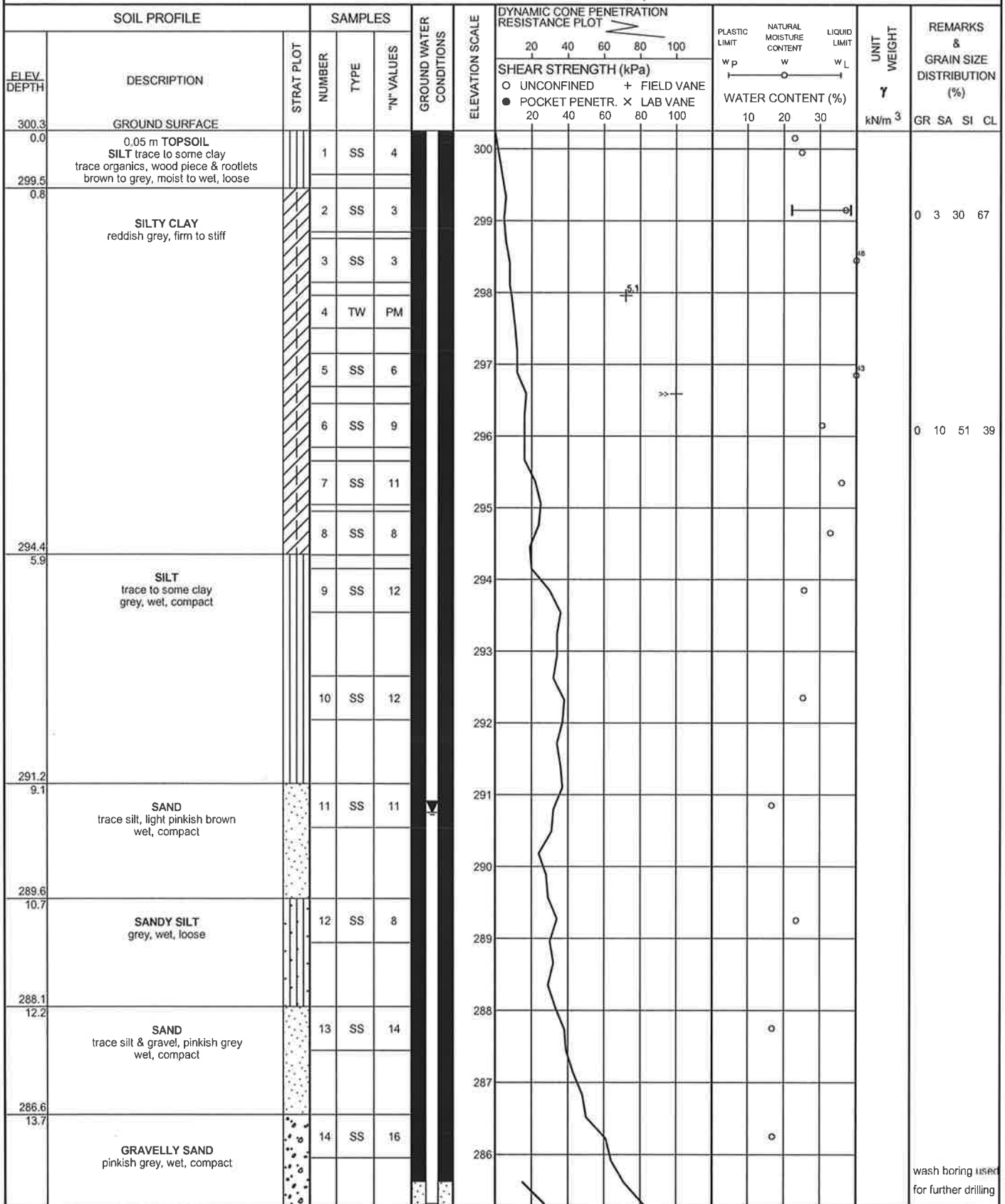
TRANETO01221AC

# RECORD OF BOREHOLE No FC7-3

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+402, 16.5 m Lt C/L of Hwy 522 ORIGINATED BY SK  
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
 DATUM Geodetic DATE 8/21/2008 8/22/2008 CHECKED BY ZO



Continued Next Page

+ 3, × 3 Numbers refer to  
Sensitivity

20  
15  
10  
5  
0

(%) STRAIN AT FAILURE

wash boring used  
for further drilling

TRANETOB01221AC

RECORD OF BOREHOLE No FC7-3

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+402, 16.5 m Lt C/L of Hwy 522 ORIGINATED BY SK  
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 8/21/2008 8/22/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE	WATER CONTENT (%)					
285.3	GRAVELLY SAND pinkish grey, wet, compact						285							
282.0														
18.3	SAND some gravel pinkish brown, wet, compact		15	SS	16		282							blow counts not reliable
281.4	End of borehole Water level @ 3.7 m (not stabilized)* upon completion in borehole. Piezometer installed to 16.5 m Piezometer water level records: Aug. 29, 2008 12.2 m Sept 04, 2008 9.5 m Sept 16, 2008 9.5 m DCPT performed from ground surface to 18.3 m adjacent to borehole.						282							
18.9														
277.4	End of DCPT DCPT performed from 14.6 m to 18.3 m and from 19.2 m to 22.9 m in the borehole.						277.4							
22.9														

TRANETOB01221AC

# RECORD OF BOREHOLE No FC7-RP1

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+379, 5.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 9/4/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	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				
								20 40 60 80 100				
						$\circ$ UNCONFINED    + FIELD VANE $\bullet$ POCKET PENETR.    x LAB VANE	WATER CONTENT (%) 20 40 60 80 100					
								PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>		
306.4	GROUND SURFACE											
0.0	0.3 m GRANULAR BASE (sand, some gravel)  FILL: Sand trace to some silt trace gravel, brown to dark brown		1	SS	11		306					
	damp		2	SS	29		305					
	moist		3	SS	25		304					
	compact		4	SS	15		303					
	wet, loose		5	SS	7		302					
			6	SS	9		301					
301.8	FILL: Silty Sand trace organics & roots brownish grey, wet		7	SS	1		300					
4.6			8	SS	1		299					
300.4	very loose		9	SS	6		298					
6.0	SANDY SILT to SILTY SAND brownish grey, wet, loose		10	SS	3		297					
298.8			11	SS	5		296					
7.6	SILTY CLAY reddish grey to grey wet, firm to very stiff		12	SS	8		295					
			13	SS	8		294					
292.7	SILT some clay (occ. pockets of silty clay) grey, wet, compact		14	SS	13		293					
13.7							292					

Continued Next Page

+ <sup>3</sup>, x <sup>3</sup>; Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

TRANETOB01221AC

# RECORD OF BOREHOLE No FC7-RP1

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+379, 5.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
DIST            HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 9/4/2008 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  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100						20	40	60
291.4																				
291.1	SILT some clay																			
15.3	SILTY SAND TO SANDY SILT occ. pockets of clayey silt greyish brown, wet, compact		15	SS	12															
290.5																				
15.9	End of borehole Borehole dry (not stabilized) upon completion Borehole cave-in @ 4.6 m upon completion																			

+<sup>3</sup>, x<sup>3</sup>: Numbers refer to Sensitivity  
20  
15  
10  
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No FC7-RP2

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+409, 5.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 9/4/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
FLEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
305.3 0.0	GROUND SURFACE											
	0.3 m GRANULAR BASE		1	SS	8		305					
	loose											
	compact		2	SS	27		304					
	FILL: Sand trace silt & gravel brown to dark brown, damp		3	SS	21							
	moist, loose		4	SS	8		303					
302.2 3.1	FILL: Silty Sand brown, wet, compact		5	SS	12		302					
301.5 3.8	FILL: Silt to sandy silt brownish grey to grey, moist, compact		6	SS	23		301					
	boulder		7	SS	16		300					
300.0 5.3	SILTY SAND trace gravel, brownish grey wet, v. loose		8	SS	3		299					
299.2 6.1	FINE SAND some silt, greyish brown wet, v. loose		9	SS	2		298					
297.7 7.6	SILT some clay occ. pockets/ seams of clay grey, wet		10	SS	8		297					
	loose		11	SS	8		296					
	compact		12	SS	17		295					
293.1 12.2	FINE SAND pinkish brown, wet, v. loose		13	SS	27		294					
291.6 13.7	SAND & GRAVEL pinkish brown, wet, dense		14	SS	36		293					
							292					
							291					

Continued Next Page

+ <sup>3</sup> . x <sup>3</sup> : Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



TRANETOB01221AC

## 2 OF 2

METRIC

GWP	484-98-00	LOCATION	Sta : 12+409, 5.0 m RI C/L of Hwy 522	ORIGINATED BY	SK
DIST	HWY 522	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	SD
DATUM	Geodetic	DATE	9/4/2008	CHECKED BY	ZO

[illegible]

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity

# Appendix A2

**Record of Borehole Sheets for Culvert C78**

TRANETOB01221AC

# RECORD OF BOREHOLE No FC8-1

1 OF 1

METRIC

GWP 484-98-00 LOCATION Sta : 12+900, 18.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST            HWY 522 BOREHOLE TYPE Hand Drilling COMPILED BY SD  
 DATUM Geodetic DATE 9/15/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
304.0	GROUND SURFACE													
0.0	0.15 m TOPSOIL		1	SS	2									
	SILTY CLAY grey, soft to firm		2	SS	2									
			3	SS	3									
			4	SS	5									
			5	SS	6									
		stiff	6	SS	8									
			7	SS	9									
			8	SS	10									
299.1	End of borehole * N-values approximate (obtained using a 31.8 kg hammer and by dividing the number of blows by two) Water level @ 0.6 m (not stabilized)* upon completion													

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE

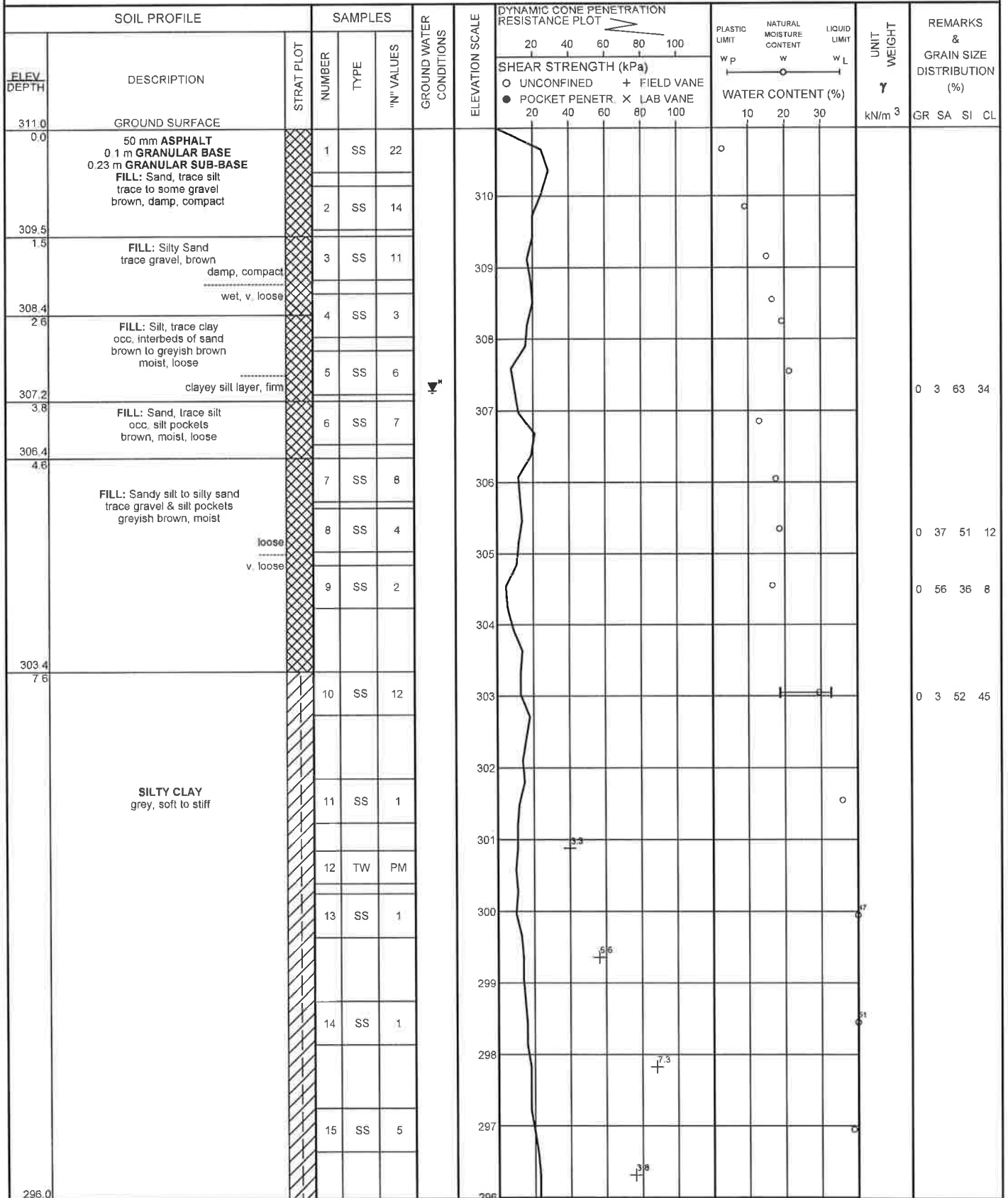
TRANETO01221AC

# RECORD OF BOREHOLE No FC8-2

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+892, 3.2 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger & NW Casing COMPILED BY SD  
 DATUM Geodetic DATE 7/30/2008 7/31/2008 CHECKED BY ZO



Continued Next Page

+ 3, X 3: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



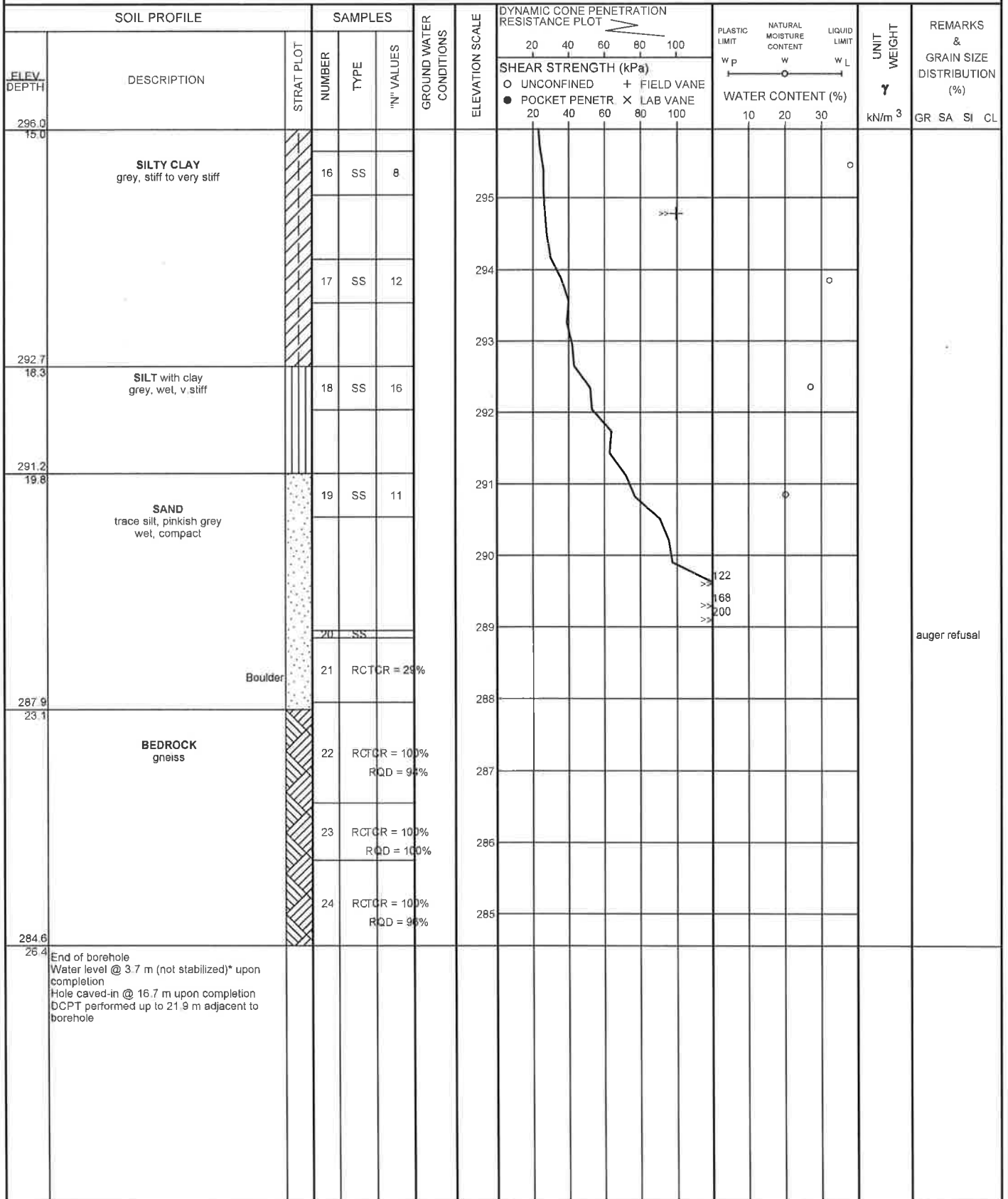
TRANETO01221AC

# RECORD OF BOREHOLE No FC8-2

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta 12+892, 3.2 m Rt C/L of Hwy 522 ORIGINATED BY SK  
DIST            HWY 522 BOREHOLE TYPE Hollow Stem Auger & NW Casing COMPILED BY SD  
DATUM Geodetic DATE 7/30/2008 7/31/2008 CHECKED BY ZO



TRANETOB01221AC

# RECORD OF BOREHOLE No FC8-3

1 OF 1

METRIC

GWP 484-98-00 LOCATION Sta : 12+891, 20.0 m Lt C/L of Hwy 522 ORIGINATED BY SK  
 DIST            HWY 522 BOREHOLE TYPE Hand Drilling COMPILED BY SD  
 DATUM Geodetic DATE 9/15/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60					
304.0 0.0	GROUND SURFACE														
303.4 0.6	0.05 m TOPSOIL SILT some sand trace organics & roots greyish brown, wet, v. loose		1	SS	3										
	SILTY CLAY grey  soft firm		2	SS	3										0 2 59 39
			3	SS	4										
			4	SS	8										0 1 60 39
			5	SS	10										
			6	SS	11										
			7	SS	12										
			8	SS	12										
299.1 4.9	End of borehole * N-values approximate (obtained using a 31.8 kg hammer and by dividing the number of blows by two) Water level @ 0.6 m in BH at completion (not stabilized)*														

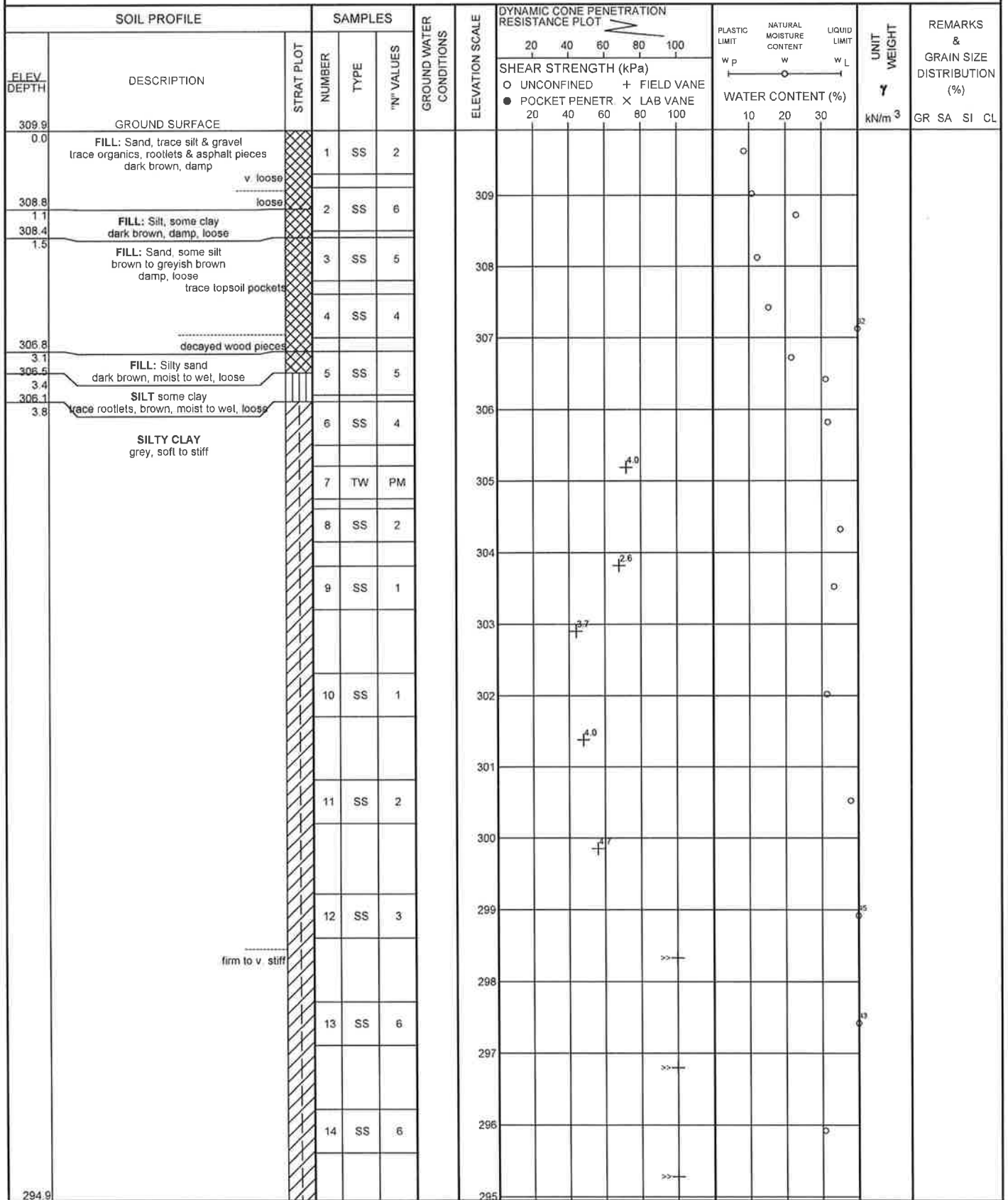
TRANETO01221AC

RECORD OF BOREHOLE No FC8-RP1

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+879, 6.5 m Rt C/L of Hwy 522 ORIGINATED BY SK  
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 9/8/2008 CHECKED BY ZO



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


TRANETOB01221AC

RECORD OF BOREHOLE No FC8-RP1

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+879, 6.5 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST            HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
 DATUM Geodetic DATE 9/8/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100	20	40
294.9 15.0	SILTY CLAY grey, firm to very stiff		15	SS	6		294												
294.0 15.9																			
End of borehole Borehole dry upon completion (not stabilized)																			

+<sup>3</sup>, ×<sup>3</sup> = Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE



TRANETO01221AC

# RECORD OF BOREHOLE No FC8-RP2

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+908, 4.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
 DATUM Geodetic DATE 9/11/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
311.6	GROUND SURFACE													
0.0	40 mm ASPHALT		1	SS	24		311							
310.8	0.15 m GRANULAR BASE													
0.8	0.40 m GRANULAR SUB-BASE		2	SS	10									
	FILL: Sand, trace to some gravel													
	brown, damp, compact													
	compact		3	SS	8		310							
	loose		4	SS	8		309							
	FILL: Sand, trace gravel		5	SS	5		308							
	trace to some silt		6	SS	2									
	brown, damp													
	v. loose		7	SS	2		307							
306.7														
4.9	FILL: Silty sand		8	SS	3		306							
306.0	some topsoil, dark brown													
	moist, very loose													
5.6	SILT with organics & rootlets		9	SS	14		305							
305.5	dark brown to blackish, moist, very loose													
6.1	SILT some clay													
	trace rootlets, greyish brown													
	wet, compact													
304.0			10	SS	2		304							
7.6														
	SILTY CLAY		11	TW	PM		303							
	grey													
			12	SS	2		302							
			13	TW	PM		301							
	soft to firm		14	SS	3		300							
	firm to stiff													
			15	SS	4		299							
	firm to v. stiff													
			16	SS	6		298							
							297							

Continued Next Page

+<sup>3</sup>, x<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15 10 5 0 (%) STRAIN AT FAILURE

TRANETOB01221AC

# RECORD OF BOREHOLE No FC8-RP2

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+908, 4.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST            HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
 DATUM Geodetic DATE 9/11/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT  W <sub>P</sub>	NATURAL MOISTURE CONTENT  W	LIQUID LIMIT  W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100										SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. x LAB VANE 20 40 60 80 100		
296.6																				
296.4																				
15.2	SILT some clay grey, wet, firm																			
295.7			17	SS	6															
15.9																				
	End of borehole Borehole dry upon completion (not stabilized)																			

# Appendix A3

**Record of Borehole Sheets for Culvert C79**

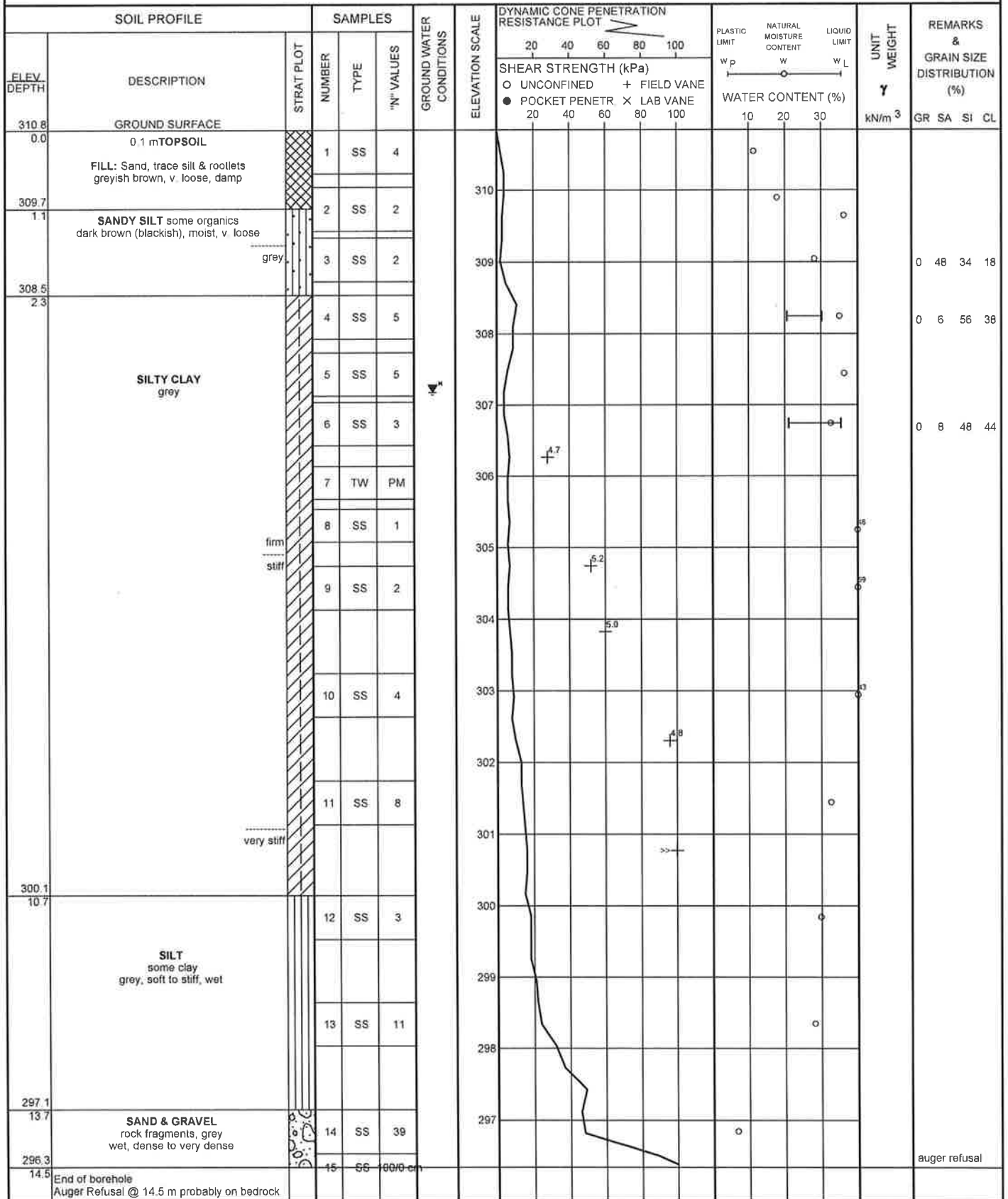
TRANETOB01221AC

# RECORD OF BOREHOLE No FC9-1

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 13+015, 11.5 m Rt C/L of Hwy 522 ORIGINATED BY SK  
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 7/22/2008 7/23/2008 CHECKED BY ZO



Continued Next Page

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

TRANETOB01221AC

RECORD OF BOREHOLE No FC9-1

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 13+015, 11.5 m Rt C/L of Hwy 522 ORIGINATED BY SK  
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 7/22/2008 7/23/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W P NATURAL MOISTURE CONTENT W LIQUID LIMIT W L	WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES							
295.8	Water level @ 3.7 m upon completion (not stabilized)* DCPT performed adjacent to borehole											

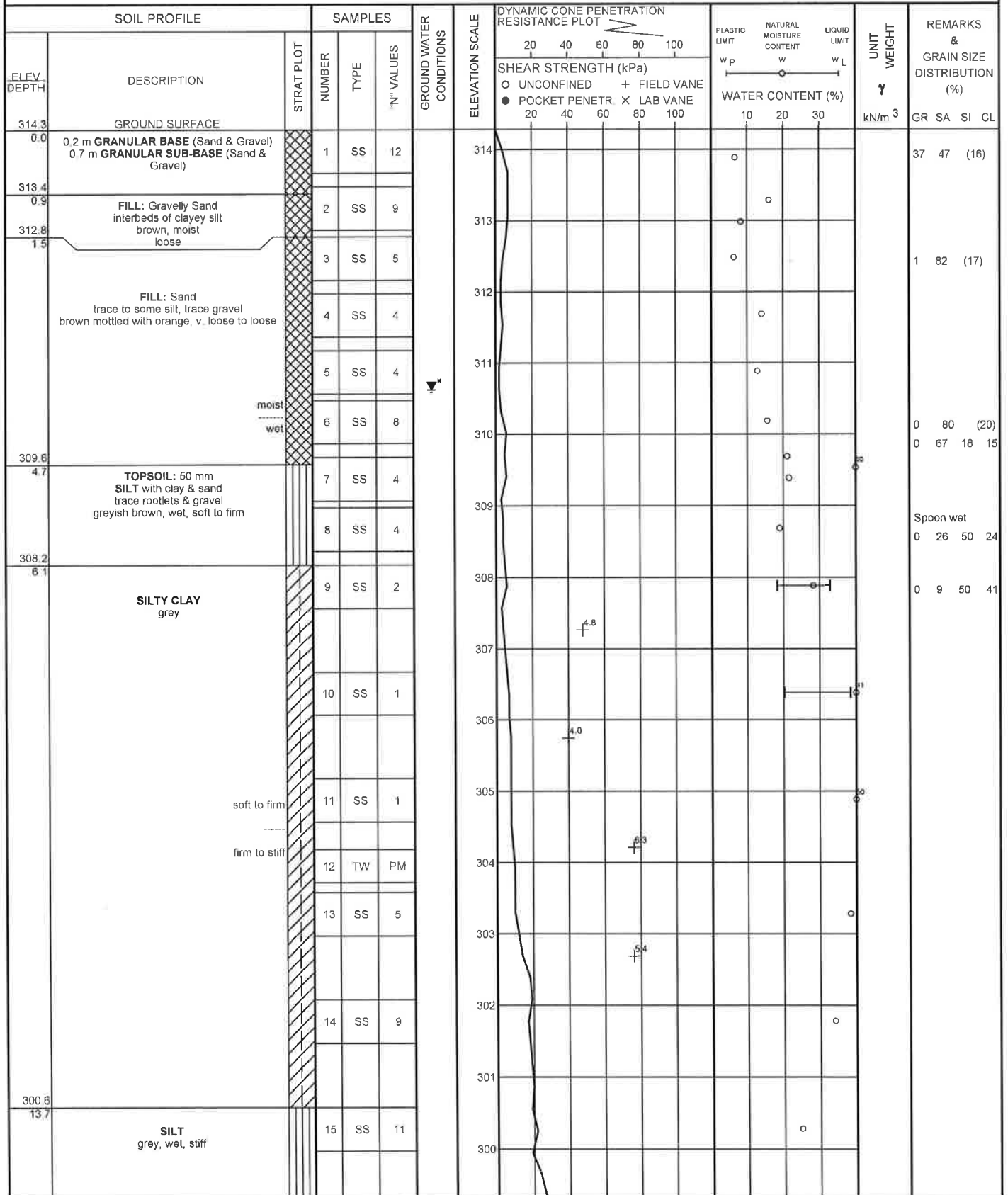
TRANETO01221AC

# RECORD OF BOREHOLE No FC9-2

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta: 13+009, 5.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST            HWY 522 BOREHOLE TYPE Hollow Stem Auger & NW Casing COMPILED BY SD  
 DATUM Geodetic DATE 7/7/2008 7/9/2008 CHECKED BY ZO



Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity  
 20  
 15  
 10  
 (%) STRAIN AT FAILURE

TRANETO01221AC

# RECORD OF BOREHOLE No FC9-2

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 13+009, 5.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST            HWY 522 BOREHOLE TYPE Hollow Stem Auger & NW Casing COMPILED BY SD  
 DATUM Geodetic DATE 7/7/2008 7/9/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)				
299.3								20 40 60 80 100				
298.6	<b>SILT</b> grey, wet, stiff		16	SS	18		299	20 40 60 80 100				
15.7	<b>SANDY SILT</b> brownish grey, wet, compact						298	20 40 60 80 100				
297.5	<b>SILTY SAND</b> some gravel, grey wet, very dense		17	SS	63		297	20 40 60 80 100				
16.8			18	SS	100% RQD		296	20 40 60 80 100				
296.6			19	RCGR = 100% RQD = 32%			295	20 40 60 80 100				
17.7	<b>BEDROCK</b> gneiss		20	RCGR = 100% RQD = 80%			294	20 40 60 80 100				
293.5			21	RCGR = 100% RQD = 100%				20 40 60 80 100				
20.8	End of borehole Water level @ 3.7 m and Cave-in @ 7.3 m upon completion DCPT performed adjacent to borehole											

+<sup>3</sup> . X<sup>3</sup> : Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

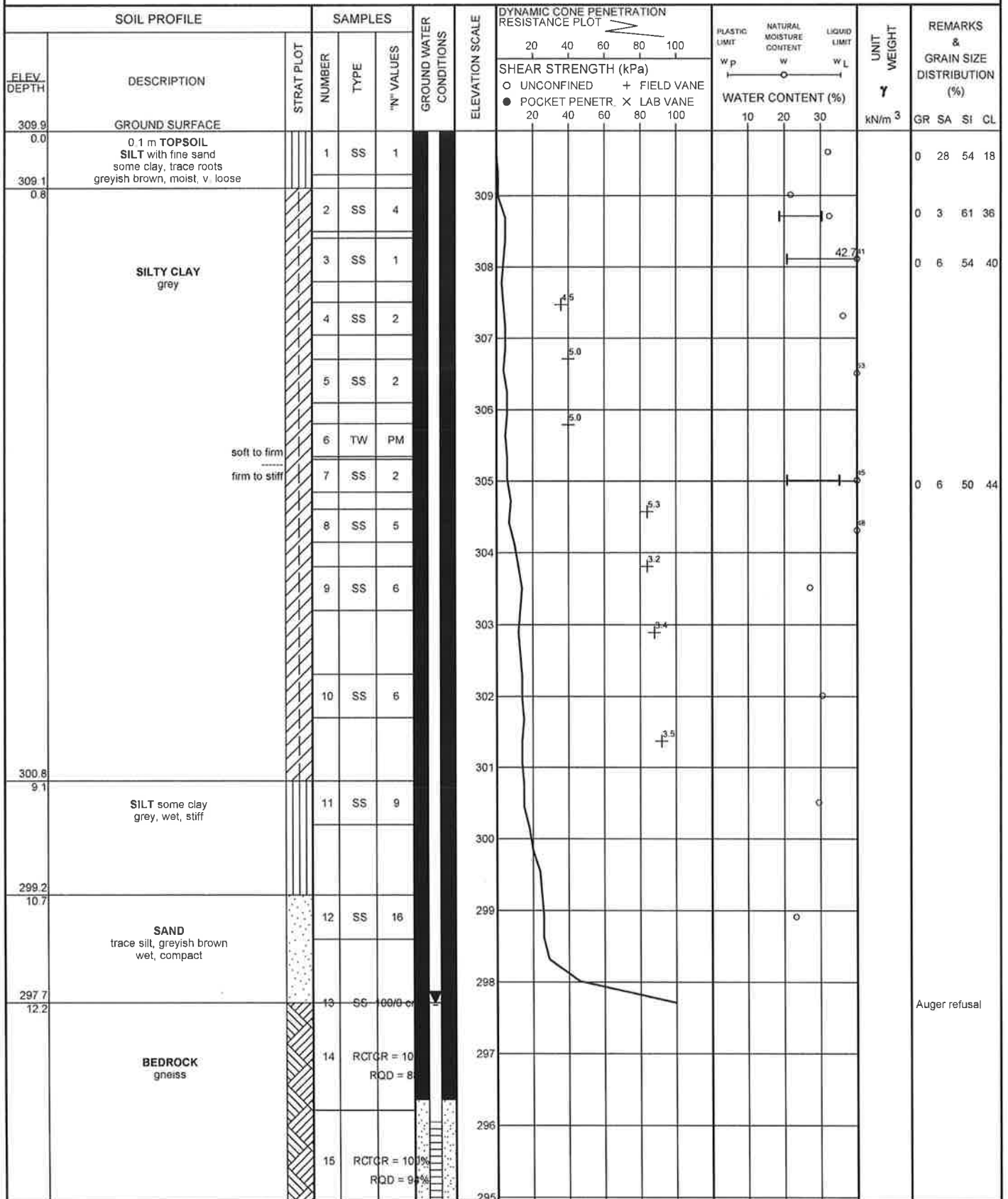
TRANETO01221AC

# RECORD OF BOREHOLE No FC9-3

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 13+000, 15.0 m Lt C/L of Hwy 522 ORIGINATED BY SK  
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger & NW Casing COMPILED BY SD  
 DATUM Geodetic DATE 7/23/2008 7/24/2008 CHECKED BY ZO



Continued Next Page

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



TRANETOB01221AC

# RECORD OF BOREHOLE No FC9-3

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 13+000, 15.0 m Lt C/L of Hwy 522 ORIGINATED BY SK  
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger & NW Casing COMPILED BY SD  
 DATUM Geodetic DATE 7/23/2008 7/24/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
294.9													
294.8													
15.1	End of borehole Water level @ 1.1 m upon completion Piezometer installed to 15.1 m Piezometer water level records: Aug 20, 2008 12.2 m Sept 05, 2008 12.2 m Sept 16, 2008 12.3 m DCPT performed adjacent to borehole up to 12.1 m												

TRANETOB01221AC

# RECORD OF BOREHOLE No FC9-RP1

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+995, 6.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST            HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
 DATUM Geodetic DATE 9/4/2008 9/5/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED      + FIELD VANE ● POCKET PENETR      x LAB VANE				
313.9 0.0	GROUND SURFACE						20 40 60 80 100					
311.8 2.1	FILL: Sand, trace silt trace to some gravel brown, damp, v. loose to loose		1	SS	4							
			2	SS	6							
			3	SS	6							
310.8 3.1	SILT trace to some clay brown, moist to wet, loose		4	SS	10							
300.2 13.7	SILTY CLAY grey  firm to stiff soft to firm  firm to stiff  stiff to v. stiff		5	SS	7							
			6	SS	4							
			7	TW	PM			3.6				
			8	SS	4			3.1				
			9	SS	3							
			10	SS	2							
								2.8				
			11	SS	2							
								4.3				
			12	SS	4							
			13	SS	7							
	SILT some clay, grey, stiff, wet		14	SS	9							

Continued Next Page

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity  
 20 15 10 5 0  
 (%) STRAIN AT FAILURE

TRANETOB01221AC

# RECORD OF BOREHOLE No FC9-RP1

2 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 12+995, 6.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
 DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
 DATUM Geodetic DATE 9/4/2008 9/5/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)	WATER CONTENT (%)					
298.9														
	SILT some clay, grey, stiff, wet		15	SS	13									
298.0														
15.9	End of borehole Borehole dry upon completion (not stabilized) Cave-in @ 4.9 m* upon completion													

+ <sup>3</sup> . × <sup>3</sup> : Numbers refer to  
Sensitivity

20  
15  
10  
5  
(%) STRAIN AT FAILURE

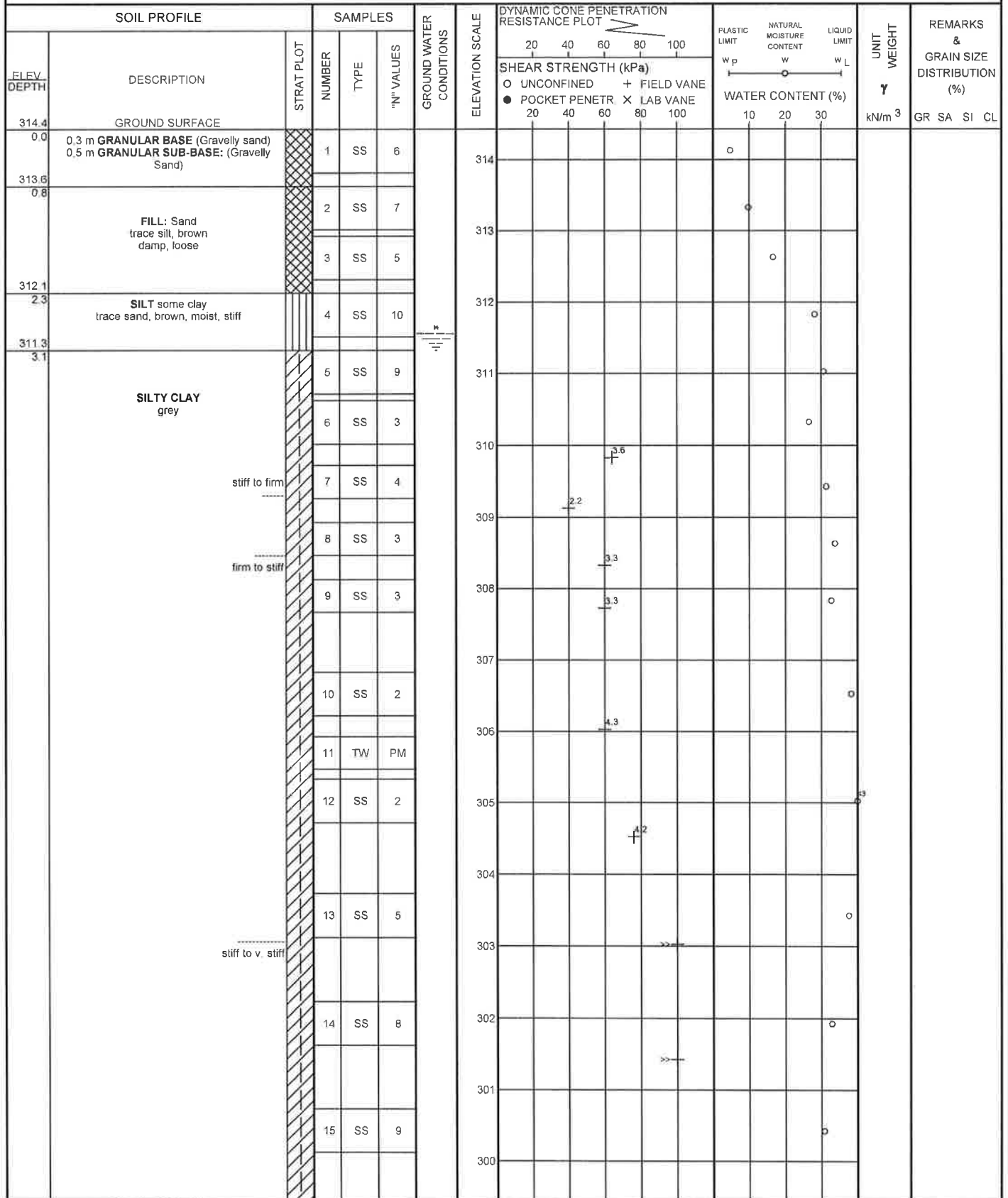
TRANETOB01221AC

# RECORD OF BOREHOLE No FC9-RP2

1 OF 2

METRIC

GWP 484-98-00 LOCATION Sta : 13+025, 5.0 m Rt C/L of Hwy 522 ORIGINATED BY SK  
DIST HWY 522 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 9/5/2008 CHECKED BY ZO



Continued Next Page

+ 3, X 3 Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

TRANETOB01221AC

## METRIC

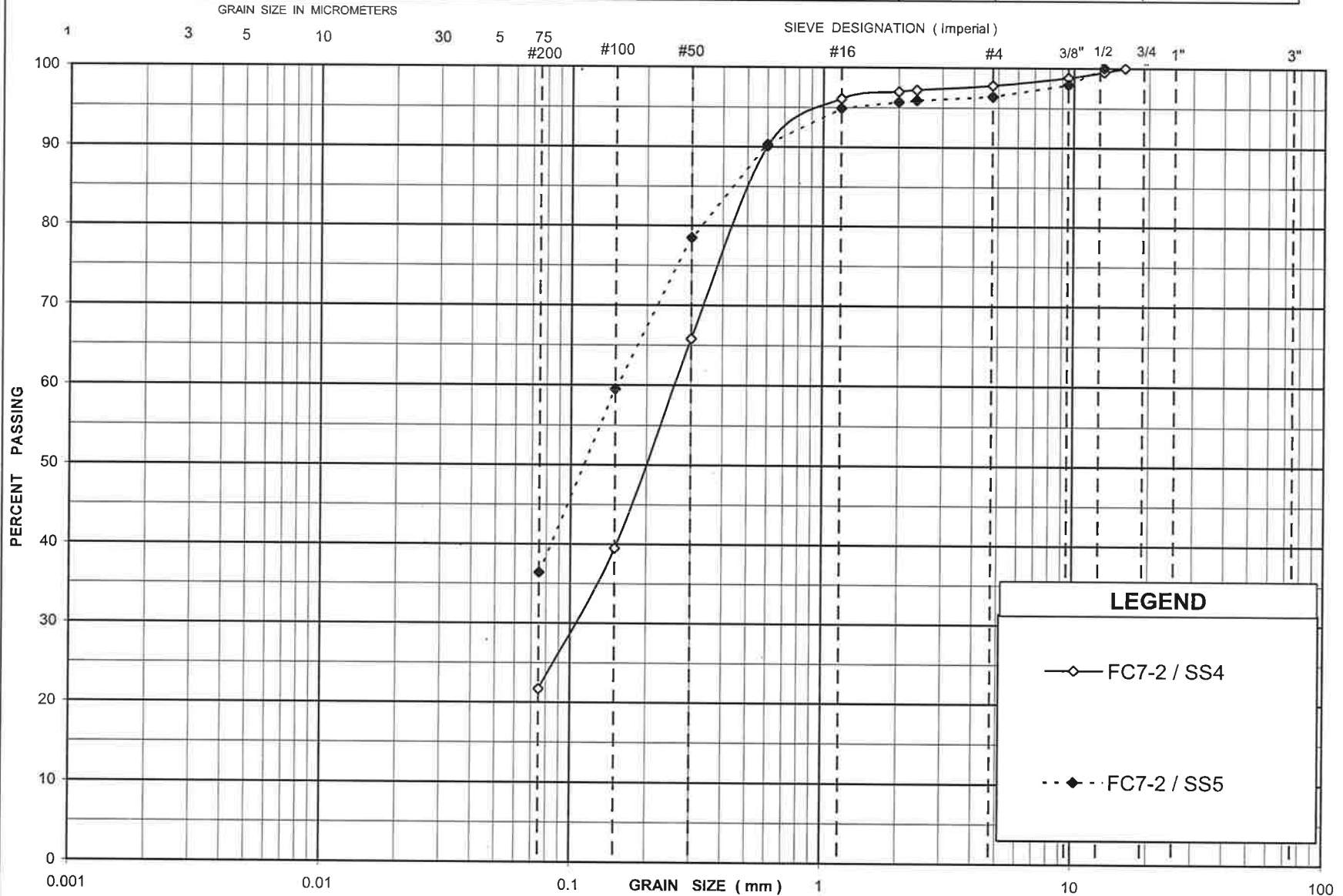
+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

# Appendix B1

**Laboratory Test Results for Culvert C77**

# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

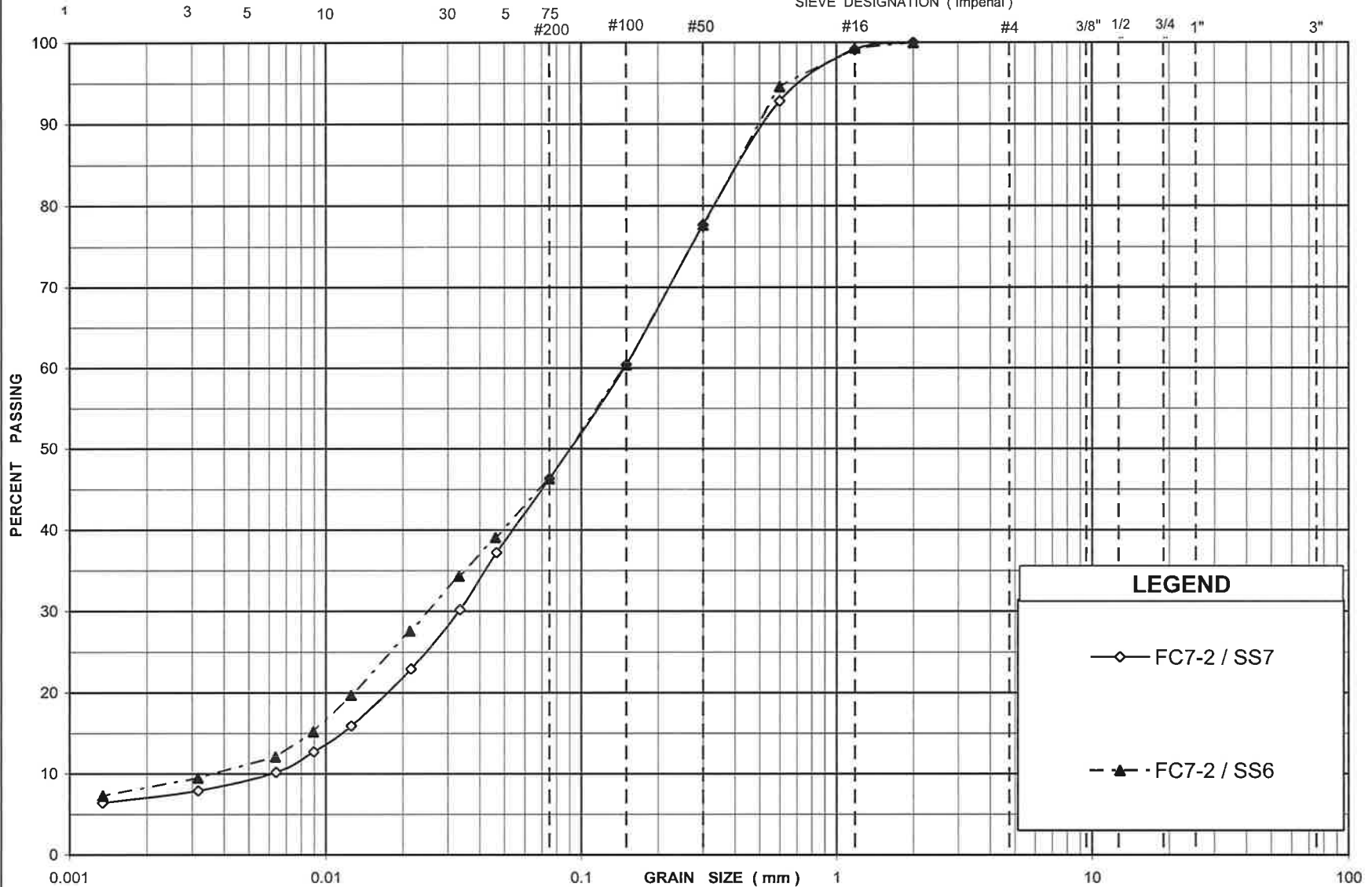


# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

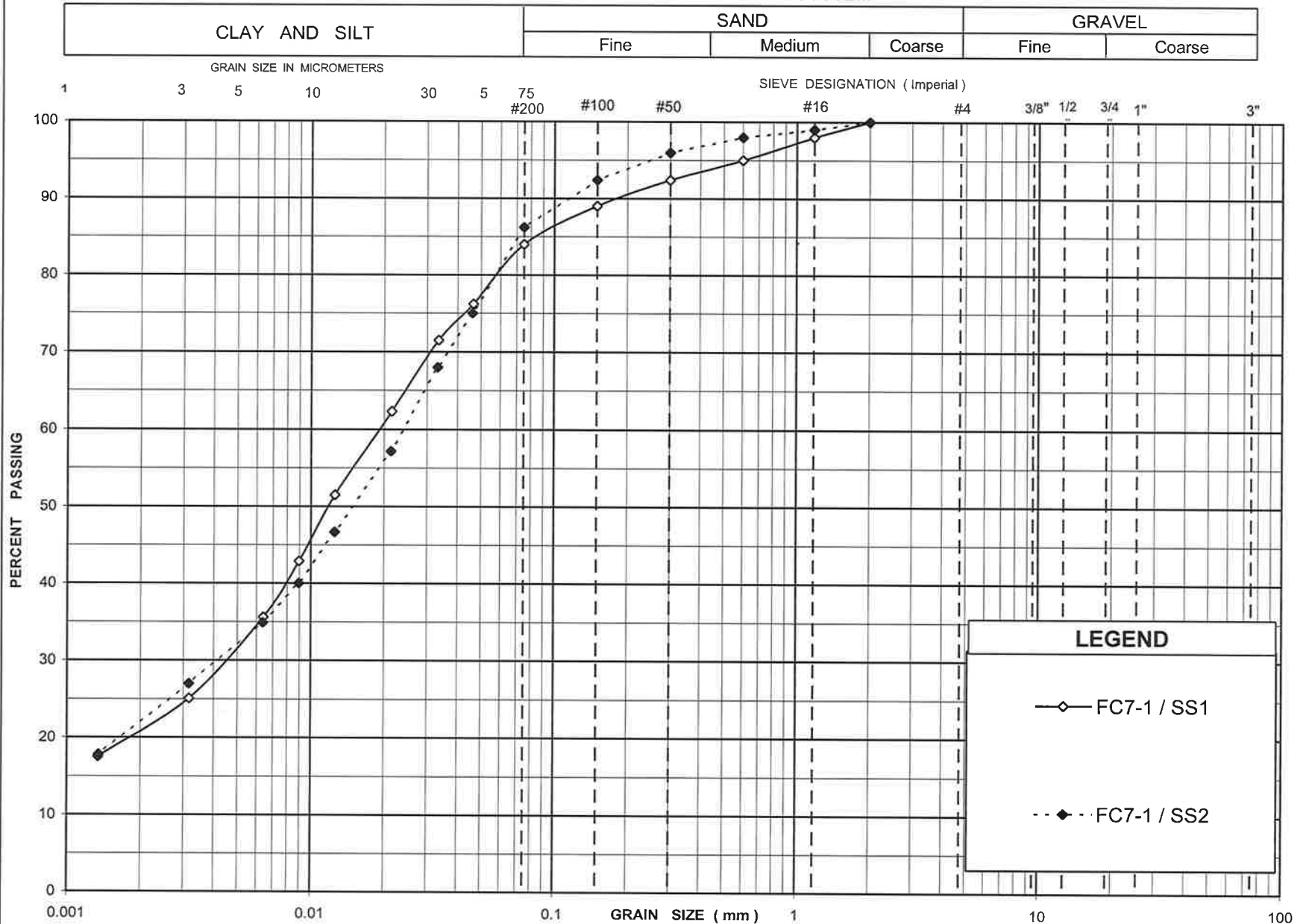
GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)





# UNIFIED SOIL CLASSIFICATION SYSTEM



# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT

SAND

GRAVEL

Fine

Medium

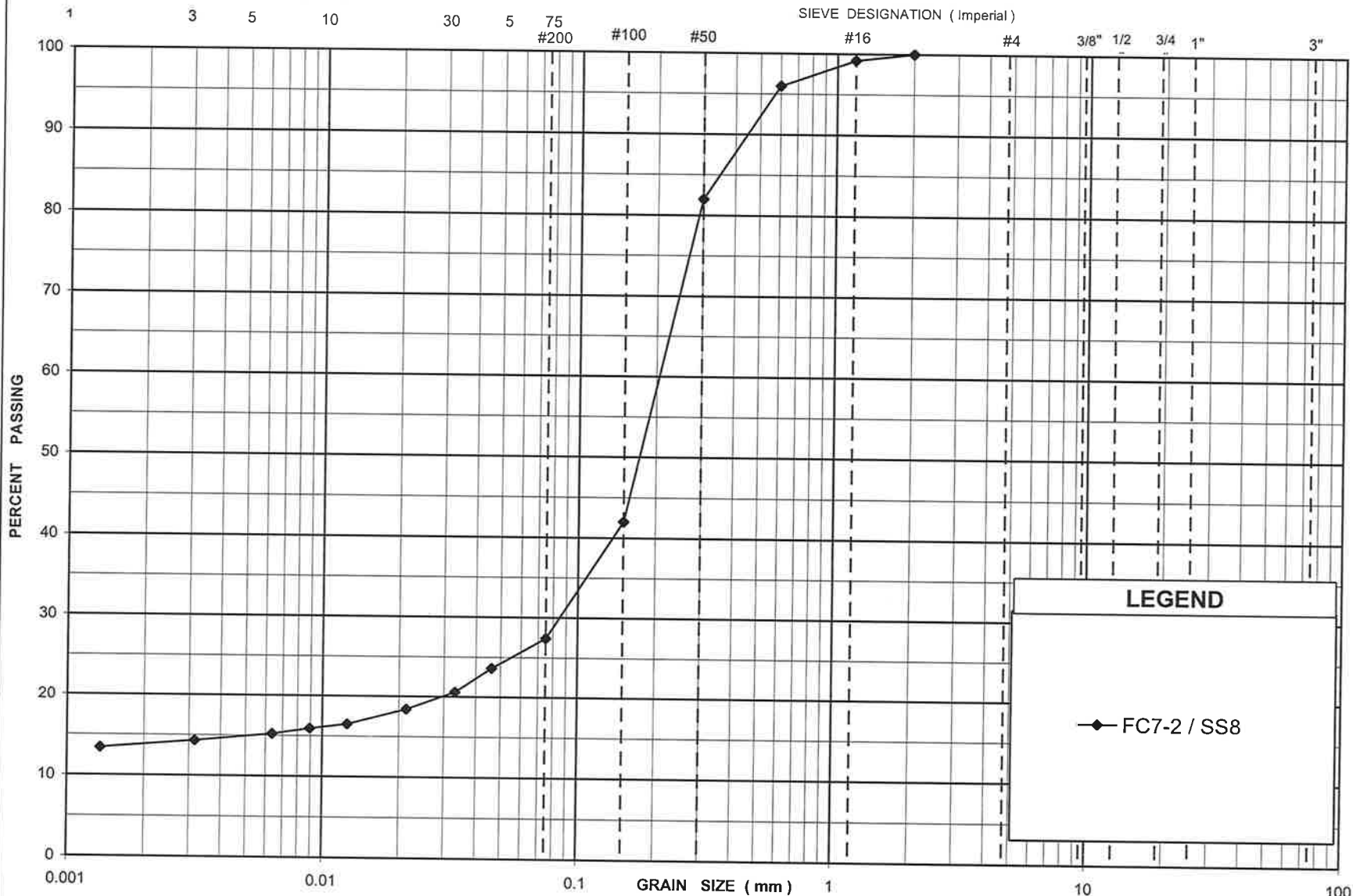
Coarse

Fine

Coarse

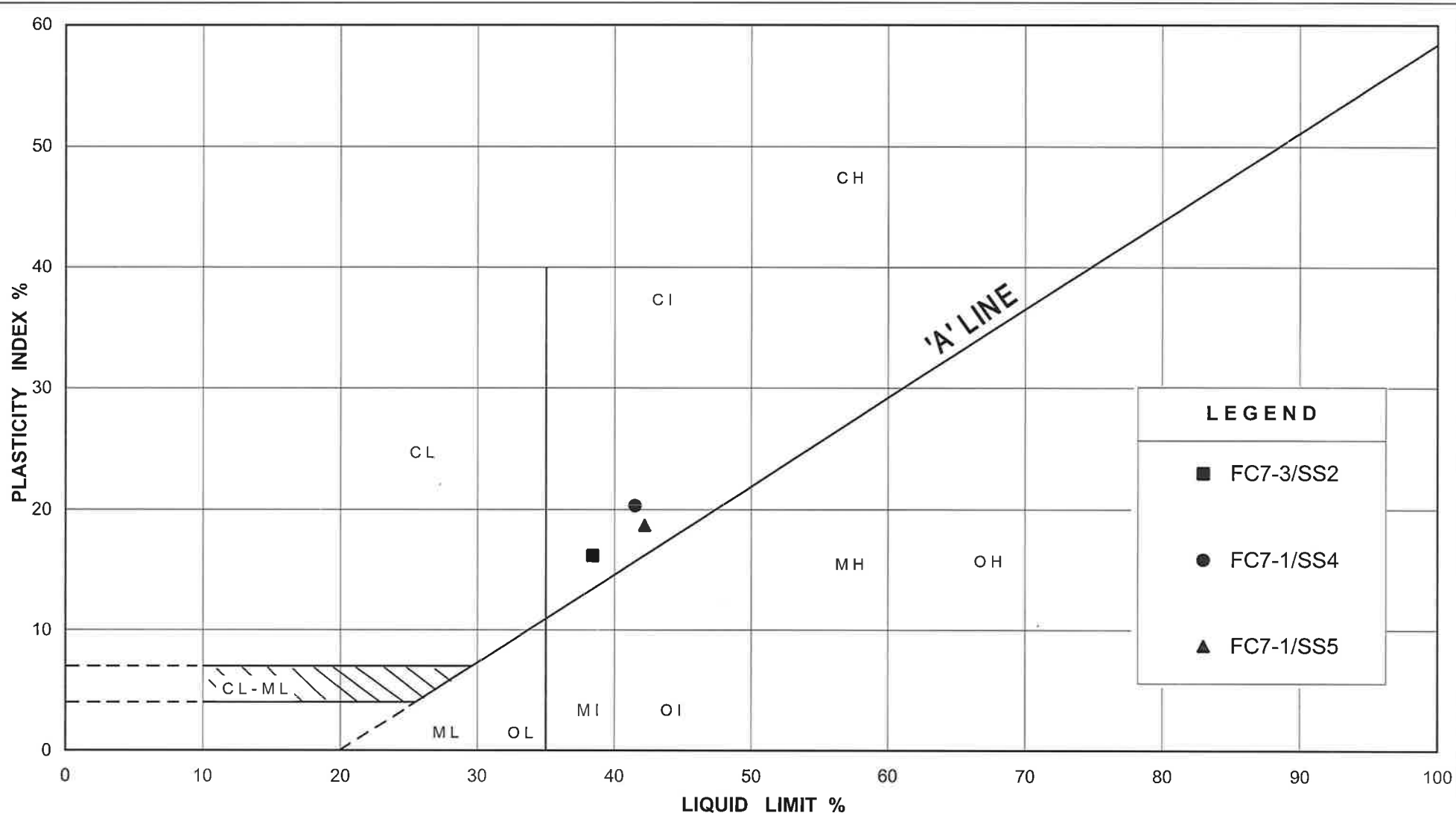
GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (Imperial)



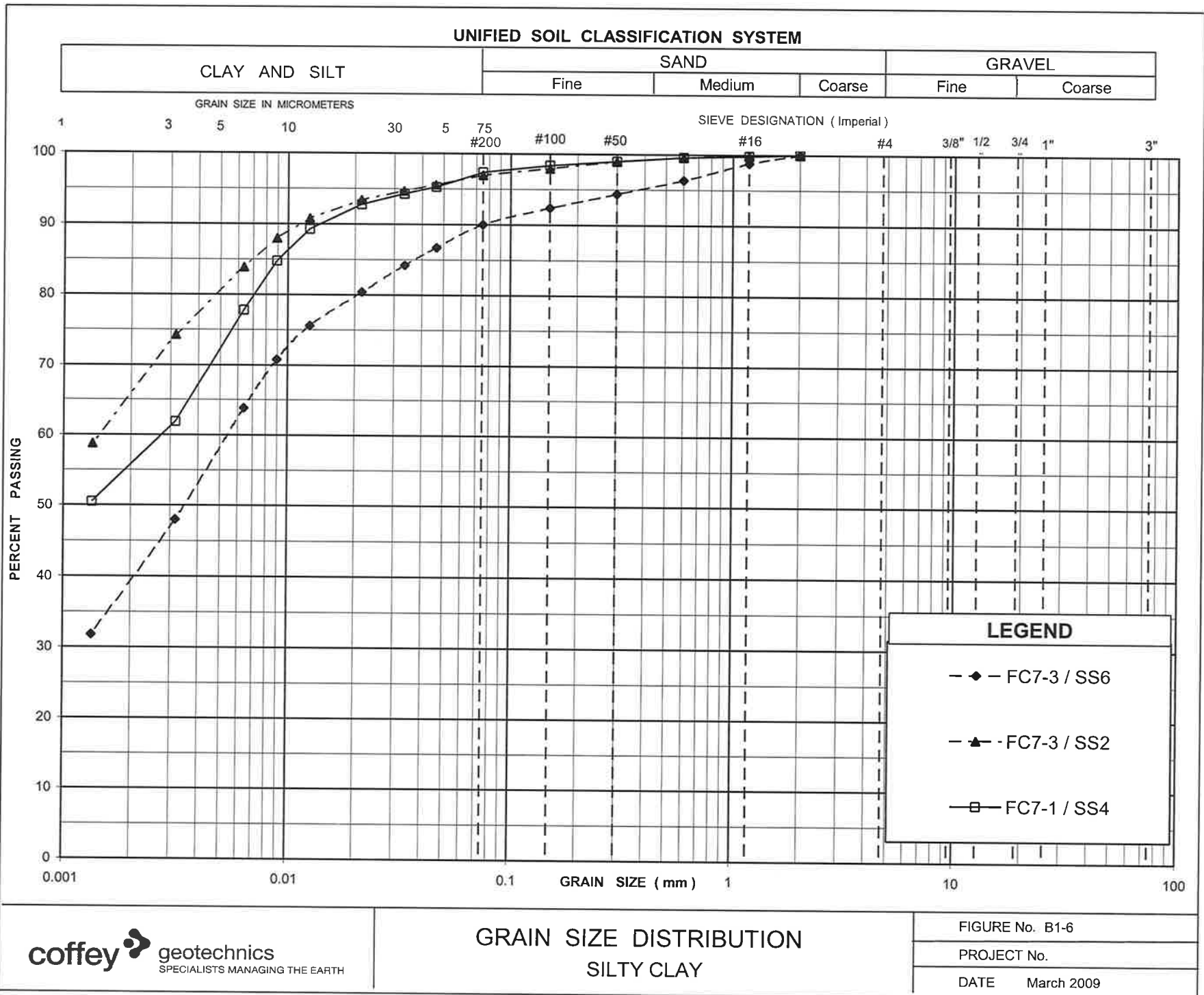
## LEGEND

FC7-2 / SS8



## PLASTICITY CHART

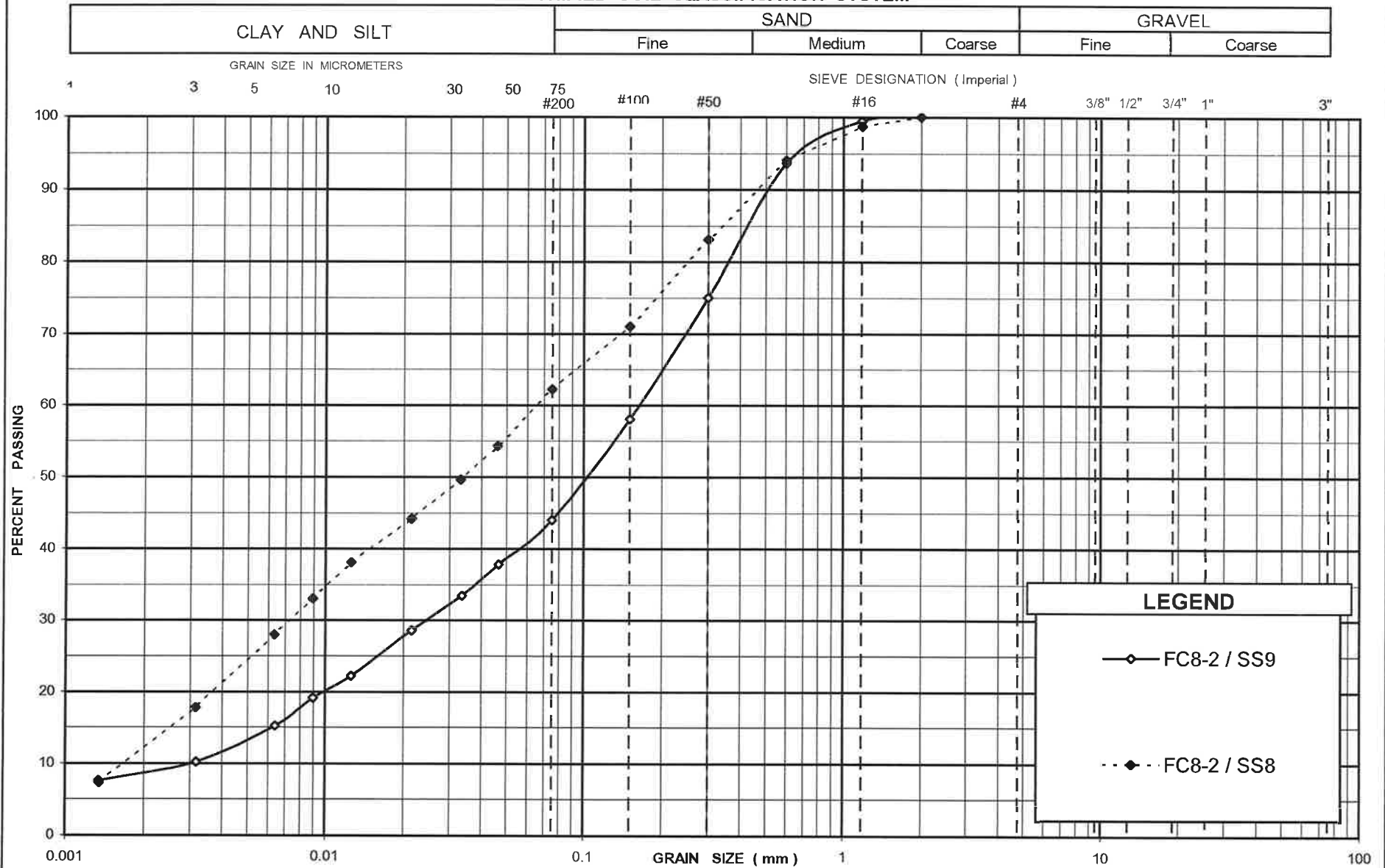
SILTY CLAY

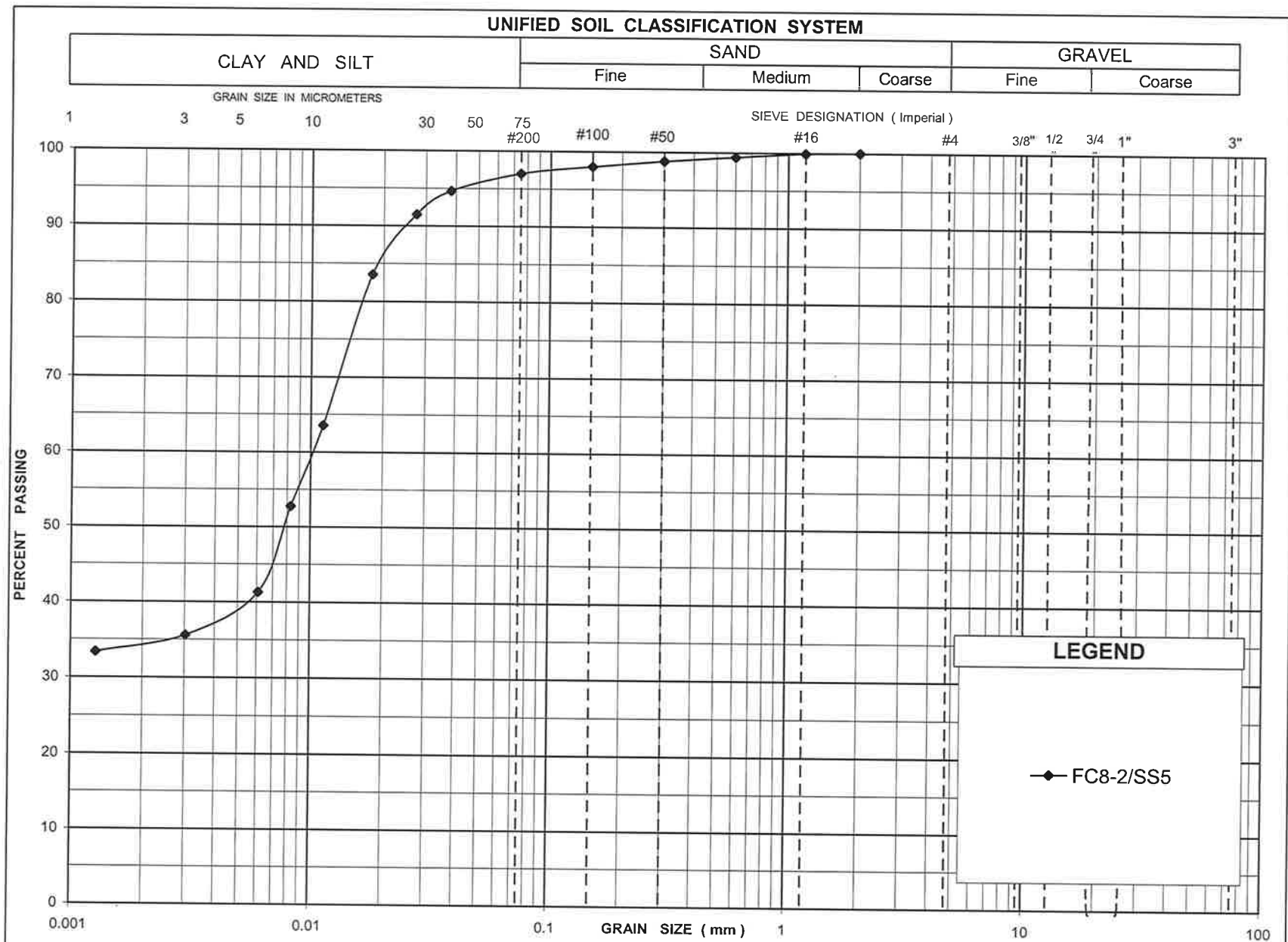


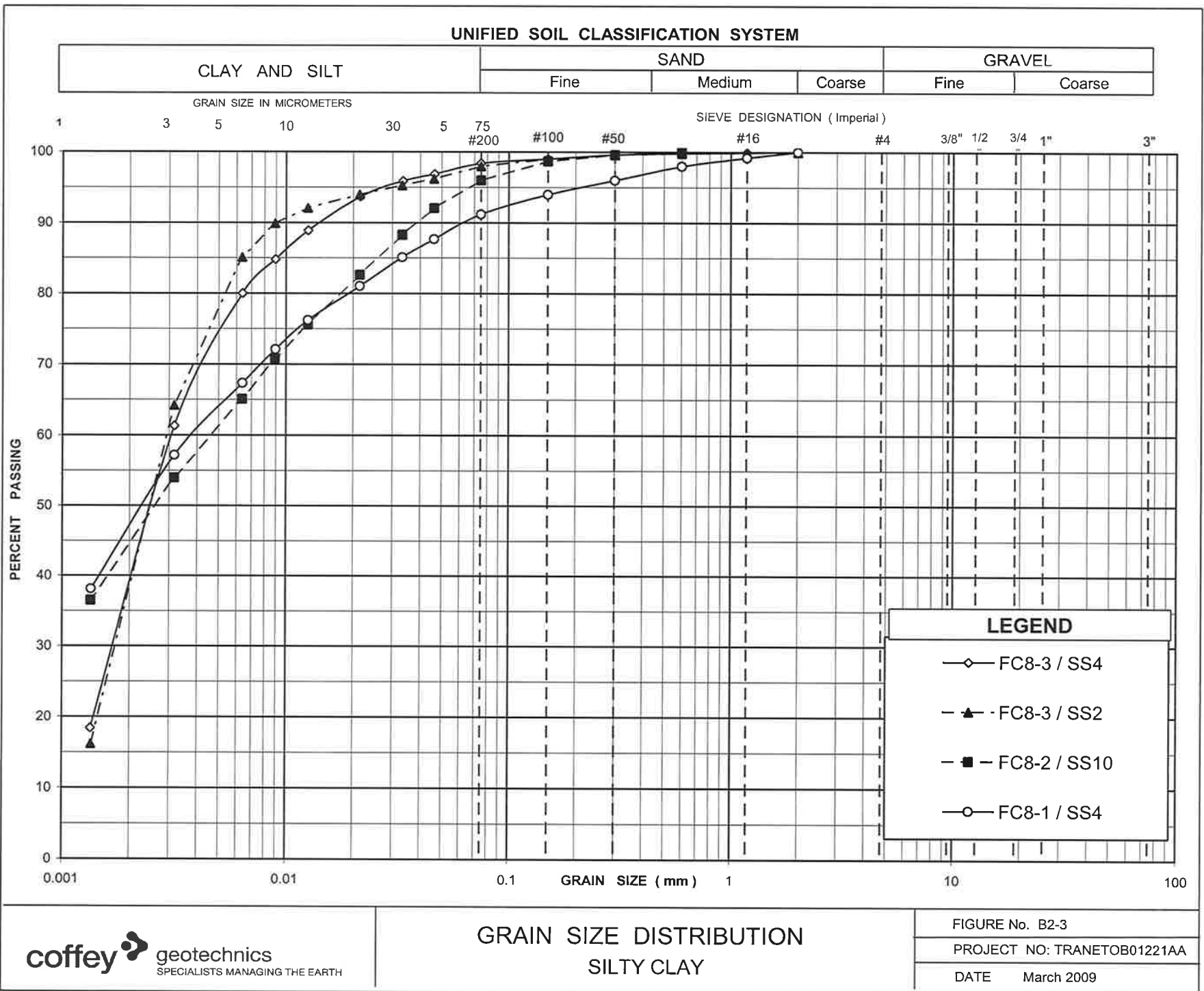
# Appendix B2

**Laboratory Test Results for Culvert C78**

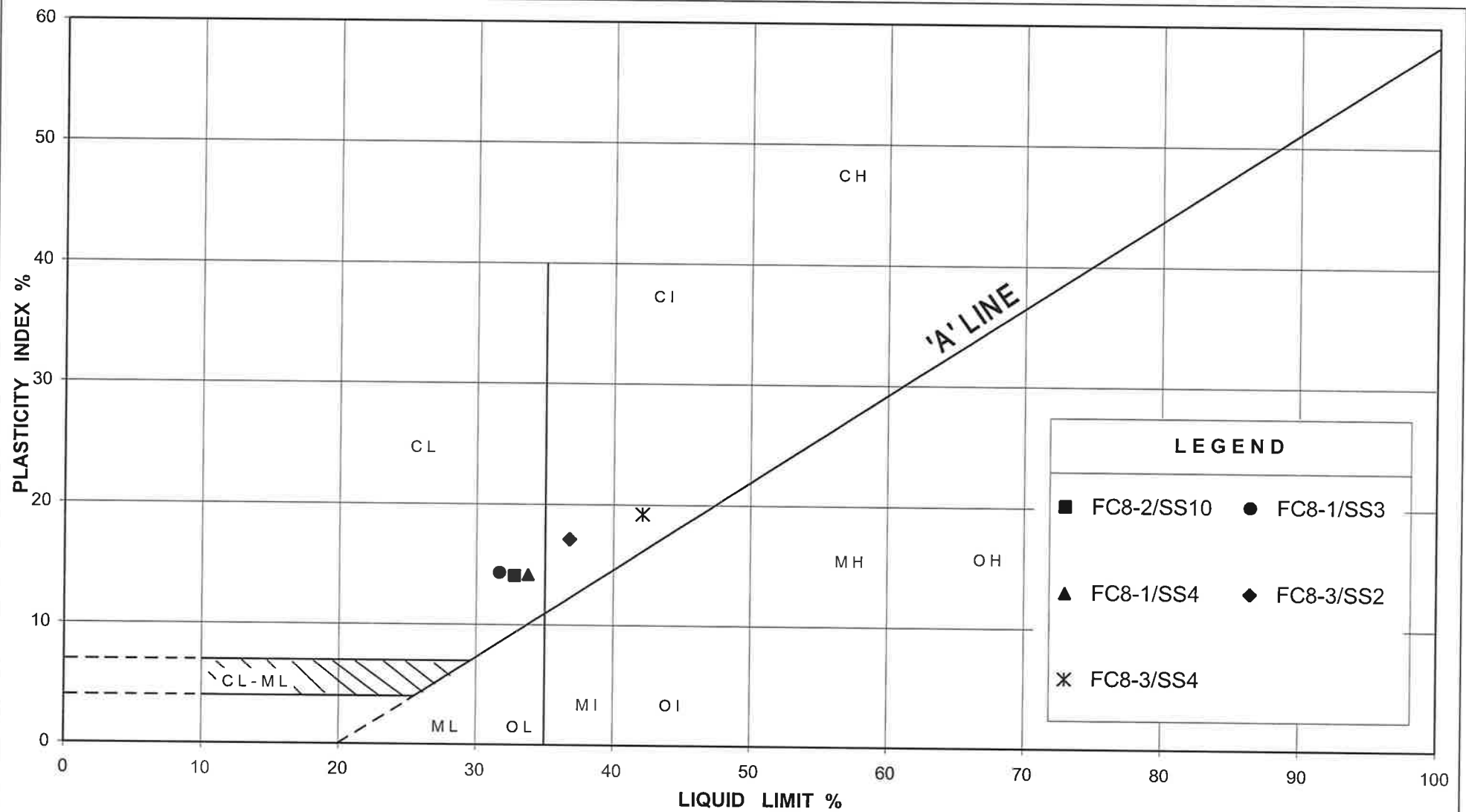
# UNIFIED SOIL CLASSIFICATION SYSTEM







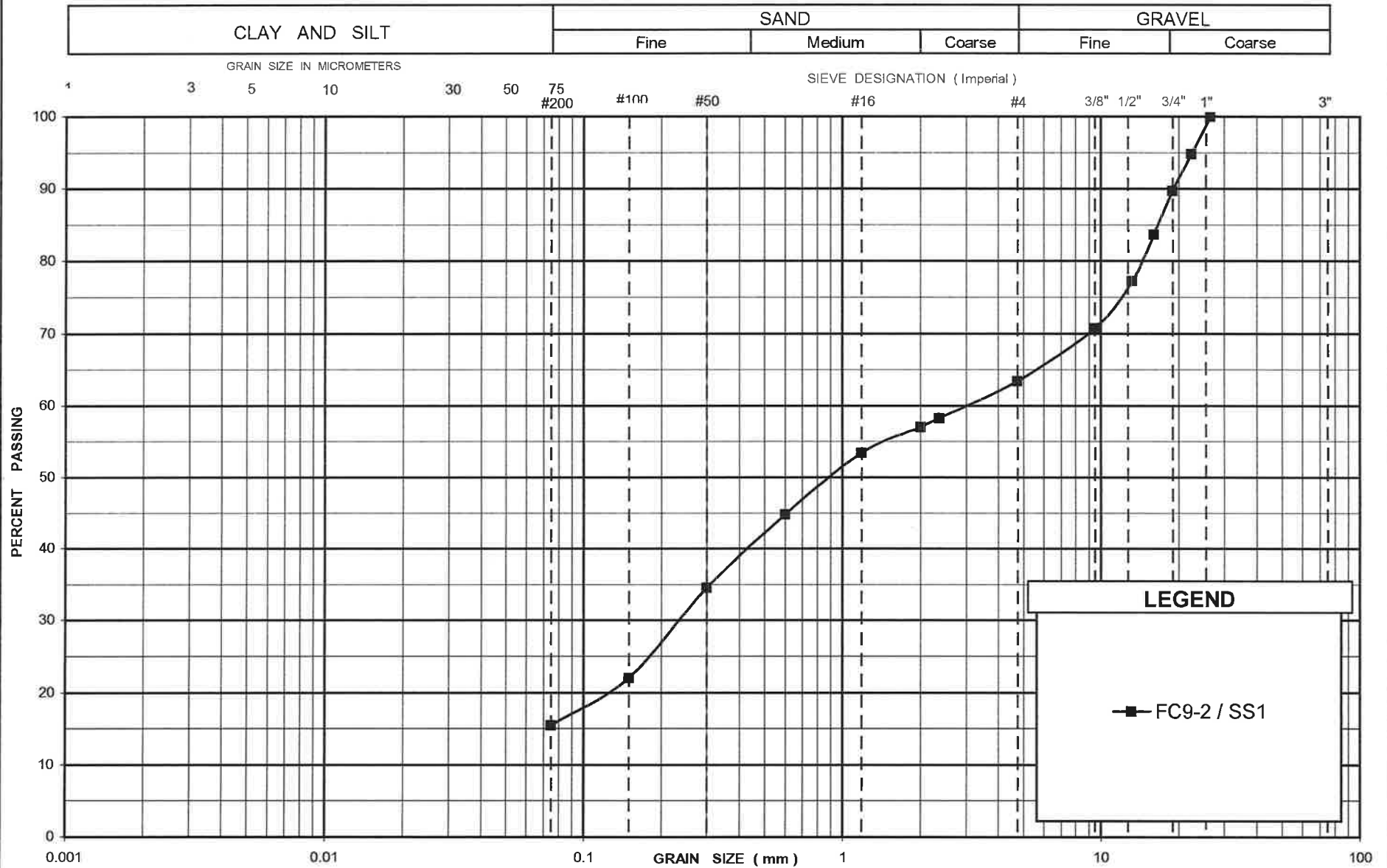




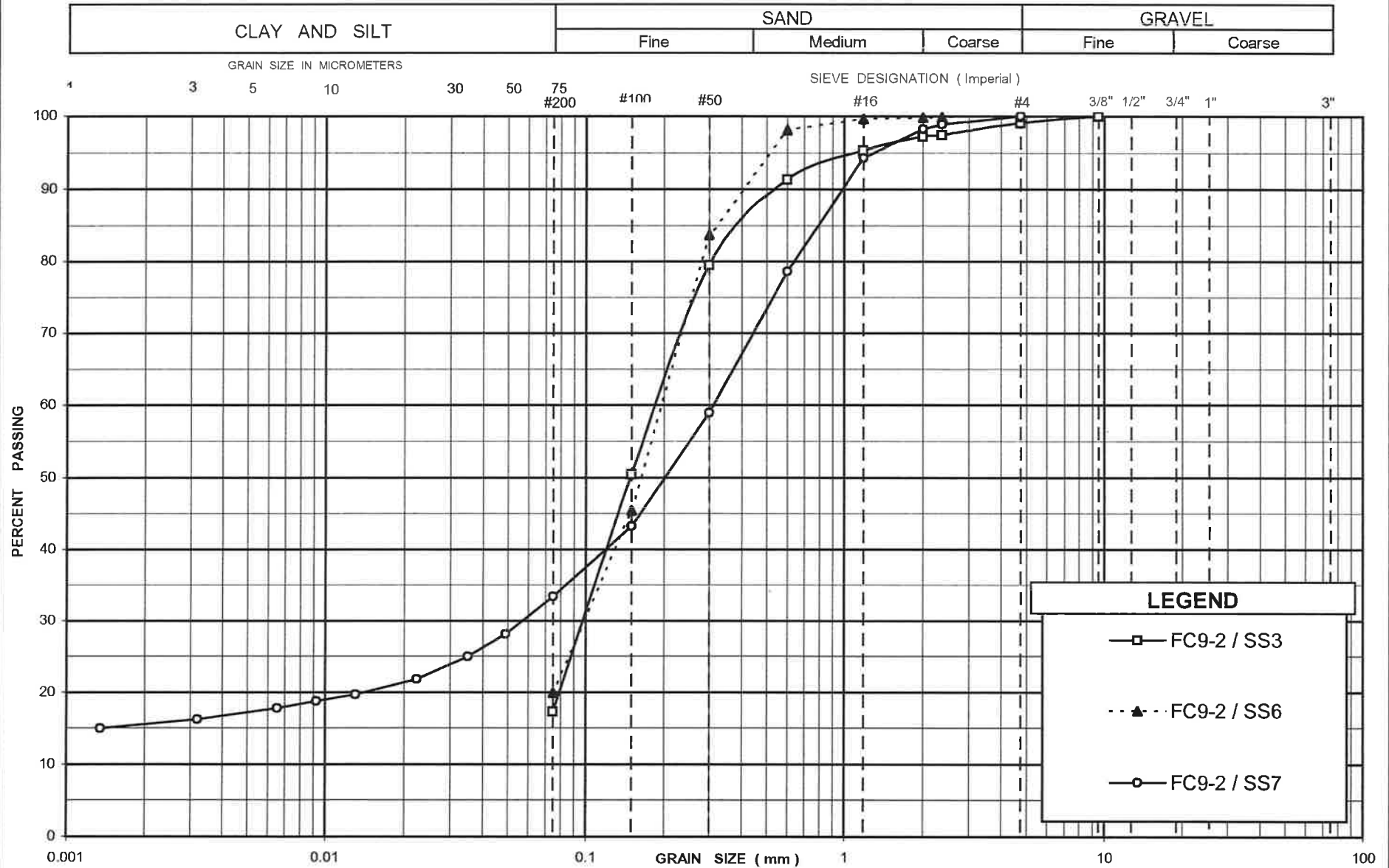
# Appendix B3

**Laboratory Test Results for Culvert C79**

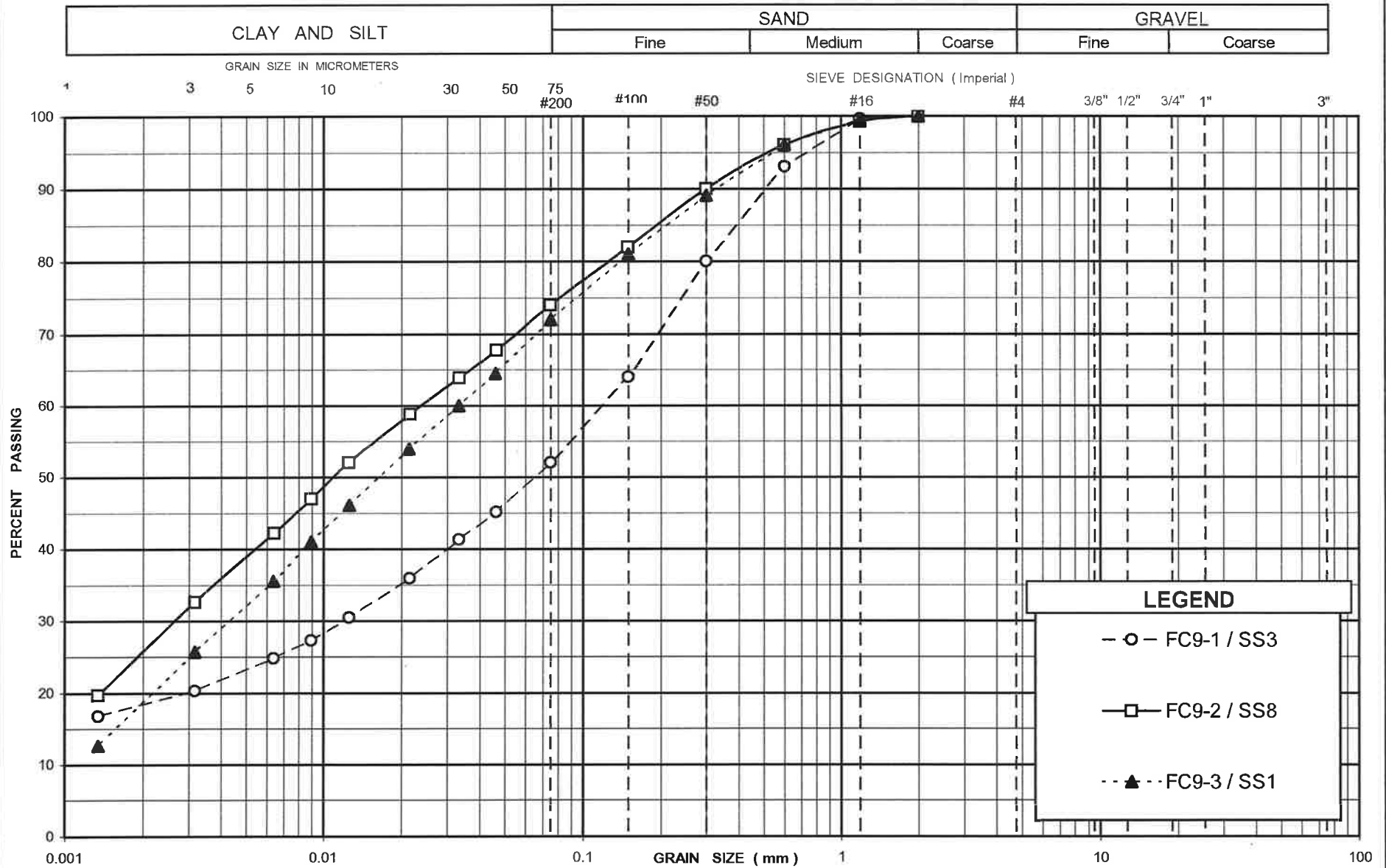
# UNIFIED SOIL CLASSIFICATION SYSTEM



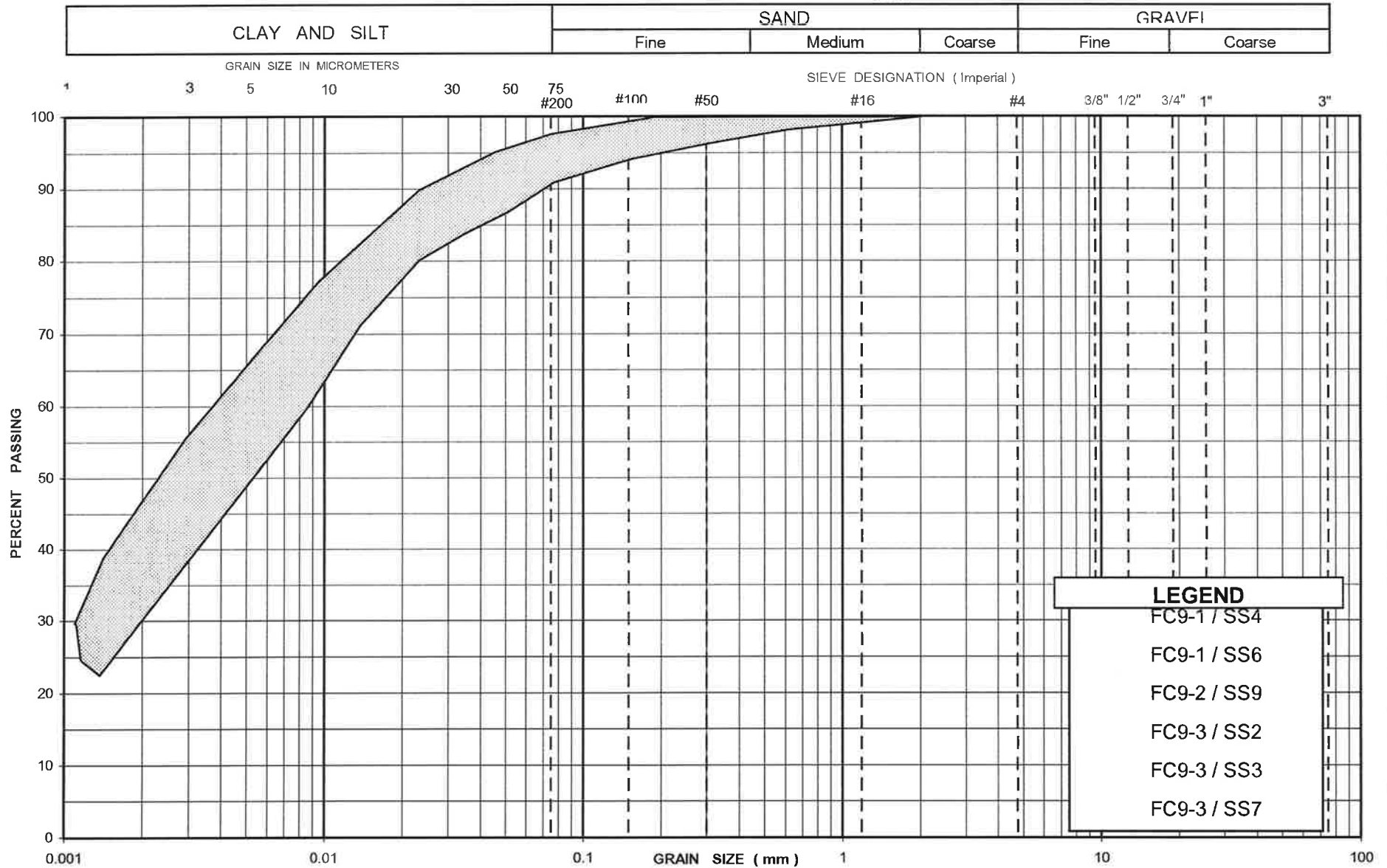
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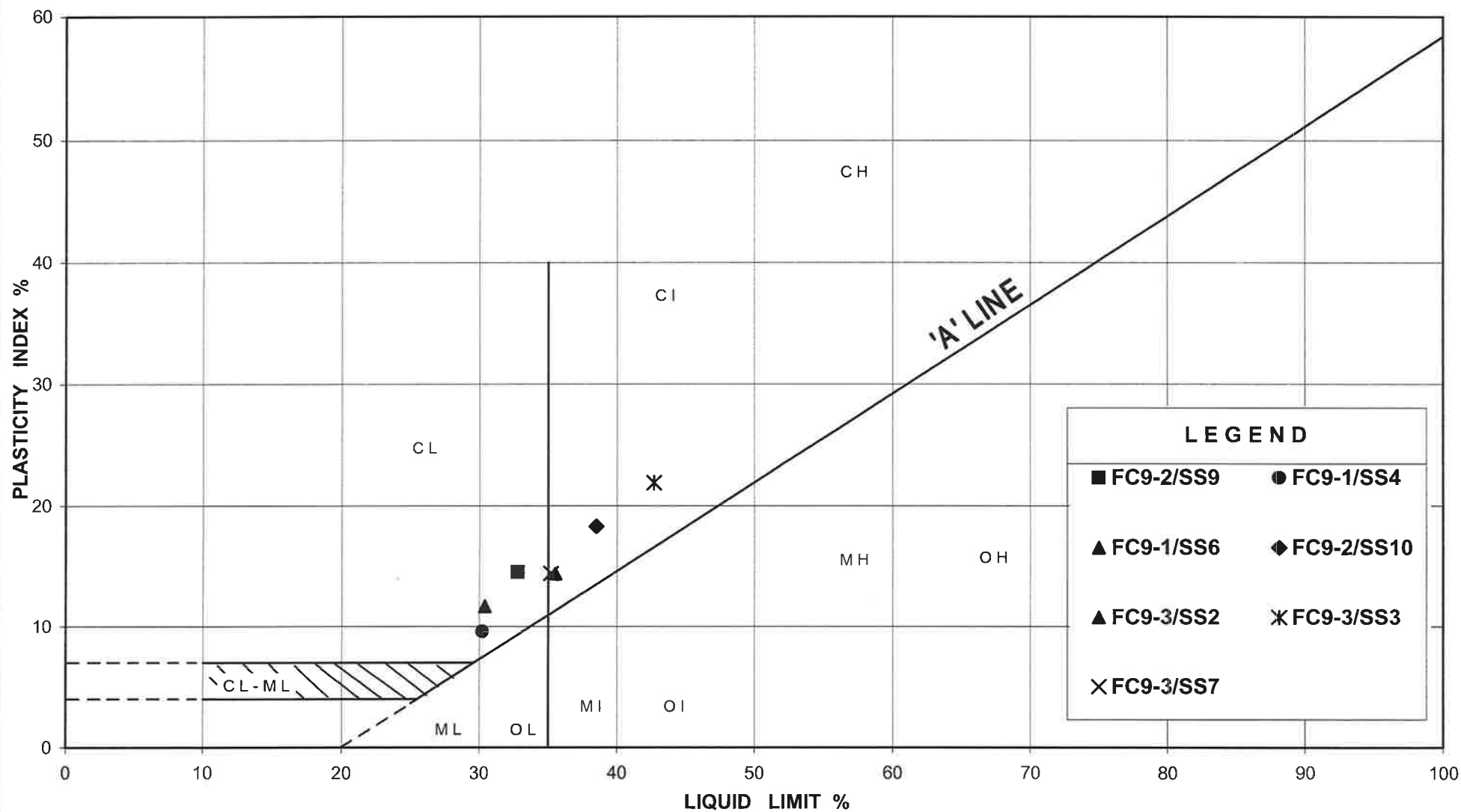


# UNIFIED SOIL CLASSIFICATION SYSTEM



# UNIFIED SOIL CLASSIFICATION SYSTEM







# Appendix C

## **Site Photographs**



Culvert C77: Looking Towards West (South Embankment)



Culvert C77: Looking Towards East (North Embankment)



Culvert C77: Looking Towards East





Culvert C78: Looking Towards West



Culvert C78: Looking Towards East





Culvert C78: Looking Towards East





Culvert C79: Looking Towards West (South Embankment)



Culvert C79: Looking Towards East (South Embankment)





Culvert C79: Looking Towards West (North Embankment)

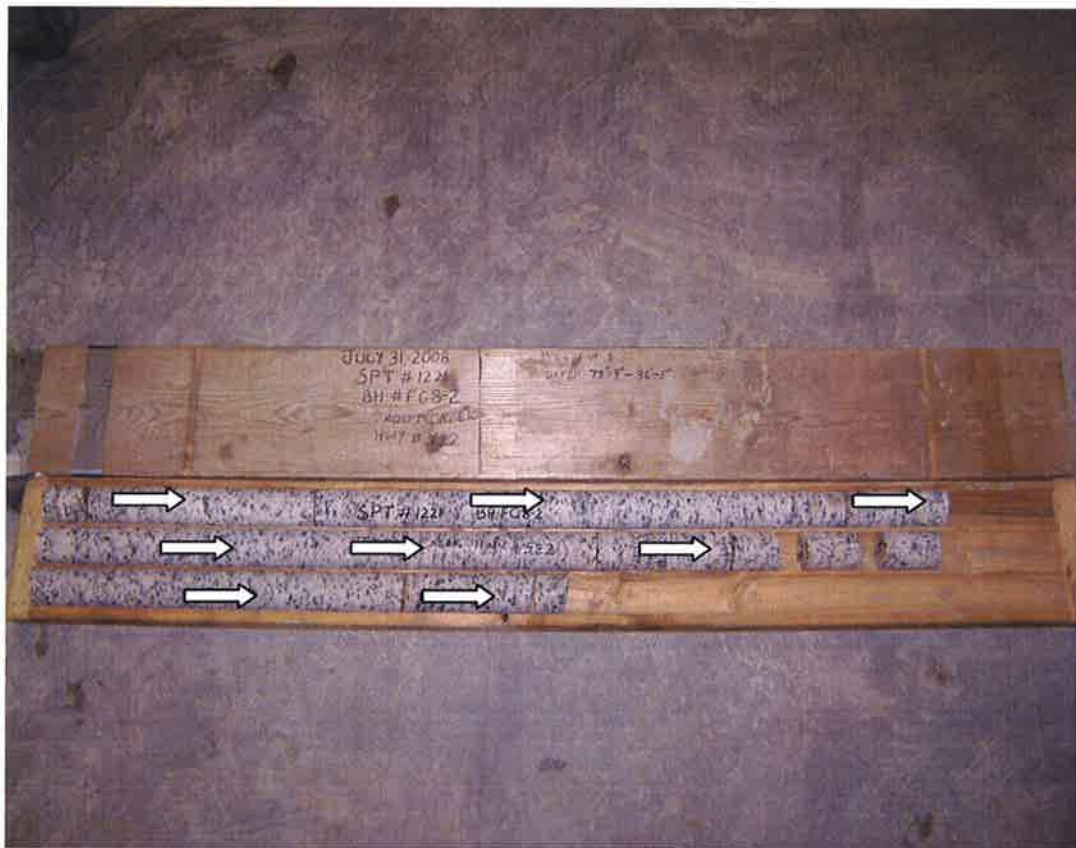


Culvert C79: Looking Towards West

# Appendix D

## Rock Core Photographs





BH#FC8-2/Rock Core RC22, RC23 & RC24, Depth (23.4 – 26.8) m



BH#FC9-2/Rock Core RC19, RC20 & RC21, Depth (18.0 -21.15 ) m



BH#FC9-3/Rock Core RC14 & RC15, Depth (12.4 – 15.4 ) m

# 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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICALL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$c_c$	1	COMPRESSION INDEX
$c_s$	1	SWELLING INDEX
$c_a$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $c_u / \tau_r$

## PHYSICAL PROPERTIES OF SOIL

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

**FOUNDATION DESIGN REPORT  
CULVERTS C77, C78 AND C79  
AT STATIONS 12+398, 12+896 AND 13+010  
HIGHWAY 522, TOWNSHIP OF SOUTH  
HIMSWORTH, DISTRICT 54, SUDBURY,  
ONTARIO, G.W.P. 484-98-00  
GEOCRES NO. 31E-293**

D. M. Wills Associates Limited

Project: TRANETOB01221AC  
October 01, 2009

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## Appendices

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Appendix H Limitations of Report

**FOUNDATION DESIGN REPORT  
CULVERTS C77, C78 AND C79 AT  
STATIONS 12+398, 12+896 AND 13+010  
HIGHWAY 522, TOWNSHIP OF SOUTH HIMSWORTH  
DISTRICT 54, SUDBURY, ONTARIO  
G.W.P. 484-98-00**

## **5 DISCUSSION AND RECOMMENDATIONS**

The culvert replacement options and recommendations for Culverts C77, C78 and C79 at Stations 12+398, 12+896 and 13+010, respectively, are discussed in the following sections.

### **5.1 Culvert Replacement**

We understand that the existing culverts are 750 mm diameter corrugated steel pipes (CSPs) and have the following lengths and inlet/outlet elevations:

**Table 5.1.1: Existing CSPs**

<b>Culvert</b>	<b>Station</b>	<b>Length</b>	<b>Outlet Invert Elevation</b>	<b>Inlet Invert Elevation</b>
C77	12+398	35.9 m	299.9 m	300.5 m
C78	12+896	43.4 m	302.3 m	304.1 m
C79	13+010	43.6 m	307.5 m	309.7 m

Based on the information provided to us by D.M. Wills Associates Limited (Wills), the existing culverts will be replaced with 800 mm diameter CSP culverts. The lengths of new culverts will be same as the existing culverts. The new inverts will essentially be the same as the existing inverts. There will be no grade raise or widening of the embankments at the culvert locations.

Five boreholes were advanced at each culvert location. Boreholes FC7-1, FC7-2, FC7-3, FC7-RP1 and FC7-RP2 were advanced in the vicinity of Culvert C77. Boreholes FC8-1, FC8-2, FC8-3, FC8-RP1 and FC8-RP2 were advanced in the vicinity of Culvert C78. Boreholes FC9-1, FC9-2, FC9-3, FC9-RP1 and FC9-RP2 were advanced in the vicinity of Culvert C79.

Borehole FC7-2 shows that the highway embankment at the Culvert C77 location consists of a 5.3 m high very loose to compact sand fill. Below the embankment are interbedded fine grained granular soils, attaining generally a coarser nature with increasing depth. These deposits are interlayered by a discontinuous 2.6 to 6.1 m thick firm to very stiff silty clay. The groundwater table is likely close to the o.g. surface beyond the toe of the highway embankment and at about 4 to 5 m below road grade within the embankment fill.

Borehole FC8-2 shows that the highway embankment at Culvert C78 consists of a 7.6 m high very loose to compact sand and silt fill. Below the embankment lies an up to 2.0 m thick, discontinuous, very loose to compact silt, followed by a 7.6 to more than 12.1 m thick soft to very stiff silty clay. The silty clay is underlain by a lower silt and a basal sand to the surface of the bedrock at a depth of 23.1 m



(Borehole FC8-2). The groundwater table is likely close to the o.g. surface beyond the toe of the highway embankment and about 3 to 5 m below road grade within the embankment fill.

The highway embankment at the Culvert C79 location consists of a 4.7 m high, very loose to compact sandy fill material (Borehole FC9-2). The subsurface conditions below the embankment consist of a 0.7 to 1.4 m thick very loose to compact silt, underlain by a 7.6 to more than 12.8 m thick soft to very stiff silty clay, which is in turn underlain by a 1.6 to 3.1 m thick silt deposit at depths ranging between 10.7 and 16.8 m below grade. The silt is followed by a 0.8 to 1.5 m thick compact to very dense sand to sand & gravel, underlain by bedrock at a depth of 12.2 to 17.7 m. The groundwater table is likely close to the ground surface beyond the toe of the highway embankment and about 3 to 4 m below road level within the embankment fill.

Due to the presence of soil conditions sensitive to disturbance and a high water table at each culvert site, as well as the weak nature of the subsoil, the use of a CSP type culvert is preferred at all three sites. If a concrete culvert must be used, a pre-cast closed bottom (box) culvert would be better suited in comparison with a rigid open bottom concrete type culvert, especially since the soils at the culvert locations appear to be highly erodible. As the proposed design by Wills is a CSP type culvert (which is the preferred option from geotechnical point of view), this type of culvert is discussed in the following paragraphs. If, however, you need more information on other type of culverts, we will be pleased to discuss this.

The proposed 800 mm diameter CSP culverts can be installed using an open cut construction or alternatively by tunnelling methods, as summarized in the following table.

**Table 5.1.2: Comparison of Installation Methods**

Construction Method	Comments	Recommendations
Open Cut Construction	It is our understanding that major/substantial detour would be required to facilitate open cut construction at these three culvert locations. The culvert sites are underlain by soils sensitive to disturbance, a high groundwater table and are poor locations for construction staging due to culvert at mid-curve in the highway (i.e. sight line issue, worker safety concern) and the height of the highway embankment.	Not very suitable due to height of fill, sensitive soil and high groundwater conditions. It will also cause traffic disruptions during construction, which may not be acceptable to MTO.
Tunnelling	The culvert alignments will intersect variable soil conditions, along with a high water table which render tunneling somewhat risky with regards to embankment settlement and side slope instability issues unless dewatering is implemented during construction. The risk can be reduced by dewatering during construction. However, tunnelling will not cause traffic disruptions during the construction and will also improve safety of travelling motorist. It will also reduce	Although risky (i.e. settlement, side slope stability issues due to vibration and liquefaction potential if not adequately dewatered) this method may be more suitable if properly implemented, as it will not disrupt traffic during construction.

Construction Method	Comments	Recommendations
	the need of property acquisition for detour and minimize 'throw away' costs such as those that would be put into a detour. Tunneling also allows for the provision for future relining activities due to the use of bigger diameter liner.	

### 5.1.1 Open Cut Construction

For an open cut construction, the existing embankment fill will be excavated to the proposed invert level. We understand that staged construction and detours are undesirable at these culvert locations from highway design point of view due to the curvature of the road. In this case, vertical excavations will likely be required and therefore temporary shoring (roadway protection) will probably be necessary for ground support due to limited space on the roadway.

Locally, temporary shoring typically consists of soldier piles and lagging, while sometimes driven interlocking steel sheet piling is also used. Open cut construction will thus require lane closures which may be undesirable (i.e. a disadvantage of open cut construction), as well since the embankment height is in excess of 5 m, temporary shoring will be costly.

#### 5.1.1.1 Culvert C77

The existing culvert at this location is a 750 mm diameter, 35.9 m long CSP. The invert of the culvert is at El. 300.5 m at the inlet and El. 299.9 m at the outlet. It will be replaced with an 800 mm diameter CSP of same length.

Borehole FC7-2 advanced from the top of the embankment shows the presence of sand to silty sand fill to a depth of 5.3 m or to El. 300.6 m. The bottom 1.5± m of the fill appears to be uncompacted. In this borehole the fill is underlain by loose to compact sand, silt and silty sand to sandy silt deposits to about El. 287.6 m followed by compact to very dense sand & gravel and sand deposits. In Boreholes FC7-1 and FC7-3 a 2.6 to 5.1 m thick, firm to stiff silty clay deposit was also contacted below about El. 300.0 and 299.5 m, interlayered with very loose to compact silt and sand.

The groundwater table at the time of our investigation was found near the o.g. levels.

#### 5.1.1.1.1 Culvert C77 Foundation Support

The boreholes show, at the proposed invert elevations (El. 300.0±m), the presence of very loose silt to firm silty clay (Borehole FC7-1), compact sand to sandy silt (Borehole FC7-2) and very loose silt to firm silty clay (Borehole FC7-3). With the prevailing subsurface conditions, the use of an open bottom concrete culvert is not recommended, since the silt is considered to be a highly erodible material. As well, the soils encountered in Boreholes FC7-1 and FC7-3 are not considered competent enough to support normal spread footings. A concrete box (i.e. closed bottom) structure can be considered, but in view of the weak foundation soils, sensitive to disturbance and the high water table, the use of a cast-in-place concrete culvert is considered less practical in comparison with a pre-cast concrete structure.

Consideration can be given to the use of an open or closed bottom reinforced concrete structure supported on deep foundations. This would however be costly, as well as being time consuming (i.e. impractical) to construct. As well, the subsurface conditions are not well suited for the utilization of deep foundations (i.e. not a well defined bearing stratum to support driven piles). For these reasons, the use of a cast-in-place concrete structure is not recommended.

From a 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 in the following paragraphs.

#### 5.1.1.1.1 Corrugated Steel Pipe (CSP)

The native compact sand to sandy silt (Borehole FC7-2) and the stiff (i.e. undrained shear strength in excess of 40 kPa) silty clay (Boreholes FC7-1 and FC7-3) are suitable, in their undisturbed state, to support a flexible structure, provided a suitable bedding is placed between the undisturbed soil and the culvert. Due to the high water table, some dewatering effort will be required to preserve the load carrying capability of the soils and to facilitate the construction, as discussed later in the report.

A minimum bedding thickness of 400 mm is recommended for a CSP type culvert. After excavating, the site to the underside of the bedding (i.e. to 0.4 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 granular soils. The silty clay should be excavated until a stiff clay (i.e. undrained shear strength in excess of 40 kPa) is encountered. If this is impractical then the granular bedding thickness should be increased to 600 mm from 400 mm.

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 increase over and above the existing conditions, except near the edges where temporary widening may be applied during the construction period.

The following geotechnical resistances can be assumed for the undisturbed subgrade soils.

Factored Bearing Resistance at U.L.S	=	100 kPa
Geotechnical Resistance	=	60 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 be 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 some settlement will take place) as well as due to exchange of the lighter, unsuitable soils with granular backfill which is relatively heavier. Cambering is not required.

If however widening is required (temporary or especially permanent) then considerable settlements would take place. Comments and recommendations can be provided if and where widening is required and details are available.

#### 5.1.1.1.2 Precast Concrete Box Culvert

A minimum granular bedding of 500 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. 0.5 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. The silty clay should be excavated until a stiff silty clay (i.e. undrained shear strength in excess of 50 kPa) is encountered. If this is impractical then the thickness of the granular bedding should be increased by 200 mm to 700 mm from 500 mm.

The geotechnical resistances given in section 5.1.1.1.1 can be used for the design of the pre-cast concrete culvert.

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 a grade raise is not proposed. 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 total settlement of 60 mm and a differential settlement (i.e. between adjacent precast sections) of 40 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 60 mm total and 40 mm differential settlement (i.e. in between two adjacent precast sections) will not present problems.

Cambering is not considered necessary under the existing embankment; however, consideration will need to be given to temporary and/or permanent widening and its effects on the structure.

As mentioned before, a CSP type culvert is the preferred selection at the site. However, if a precast concrete box culvert is to be utilized, cambering details should be further looked into.

#### 5.1.1.2 Culvert C78

Borehole FC8-2, advanced from the highway shoulder at the Culvert C78 site, showed the presence of 7.6 m of embankment fill to El. 303.4 m, underlain by a major silty clay deposit to El. 292.7 m. Boreholes FC8-1 and FC8-3 were put down from the o.g. levels beyond the toe of the embankment and these boreholes show the presence of a veneer of topsoil to about El. 303.9 m underlain by a 0.5 m thick surficial very loose silt layer to El. 303.4 m in Borehole FC8-3. Underlying this silt layer and topsoil (Borehole FC8-1), the boreholes show the presence of silty clay to the termination depth of 4.9 m or El 299.1 m. The silty clay has a soft to firm consistency, becoming stiff to very stiff below about El. 296 m. In Borehole FC8-2 which was extended deeper, the silty clay is underlain by a layer of very stiff silt which is in turn underlain by a compact basal sand deposit to the surface of bedrock at El. 287.9 m or at about 23.1 m below the

surface of the road. The groundwater table at the time of our investigation was contacted at about o.g. level beyond the embankment and about 2 m higher in the embankment fill itself.

As the invert of the proposed 800 mm diameter CSP culvert is at about El.304.1 m at the inlet and 302.3 m at the outlet, the founding level is expected to be within the soft to firm silty clay deposit.

#### *5.1.1.2.1 Culvert C78 Foundation Support*

The silty clay is considered unsuitable (i.e. too weak) to support normal spread footing foundations and therefore, the use of an open bottom concrete structure is not considered feasible from foundation engineering point of view. A closed bottom concrete box culvert can be considered, but in view of the subsurface conditions, if a concrete box culvert is to be used, then the use of a pre-cast reinforced concrete structure is preferable. An open bottom or a closed bottom reinforced concrete structure can be considered in conjunction with deep foundations supported on bedrock at about El. 288 m (i.e. about 15 m long) but this will be extremely costly as well as being time consuming (i.e. impractical for this project).

From a foundations engineering point of view, the use of a flexible structure such as a CSP is the preferred choice for this project but, if necessary, the pre-cast concrete box culvert can also be considered, as discussed in the following sections of this report.

##### *5.1.1.2.1.1 Corrugated Steel Pipe (CSP)*

The native firm to stiff silty clay with a minimum 30 kPa undrained shear strength is suitable to support the proposed CSP type culvert, provided that a suitable granular bedding is placed. A minimum 500 mm thick bedding is recommended to be placed beneath the bedding. After excavating the subgrade to the underside of the bedding (i.e. to 0.5 m below the bottom of the proposed pipe), the exposed subgrade subsurface should be carefully inspected, evaluated and approved. If organic or otherwise unsuitable soils are found, they should be removed to the surface of the inorganic suitable soil from beneath the pipe and extending at least 0.5 m beyond the foot-print of the pipe on each side. If the exposed subgrade (i.e. silty clay) appears to have an undrained shear strength of less than 30 kPa, then a 100 to 150 mm size rock fill (i.e. clear rock fill) should be pushed into the subgrade to strengthen it. After pushing the rock fill (if required), the 500 mm thick bedding can be placed.

For the subgrade prepared in the manner described above the Factored Bearing Resistance at U.L.S of 80 kPa and a Geotechnical Resistance at S.L.S equal to 40 kPa can be assigned.

Under the embankment, the recommended values are less than the existing embankment loading. This, however is not considered to be a problem, since the silty clay stratum under the existing embankment would have been consolidated under the stresses imposed by the embankment. Since there will be no grade raise, theoretically there should be negligible additional settlements under the existing embankment. However, a settlement of about 30 mm should be allowed for due to rebound during the construction. Based on this, if the founding subgrade is undisturbed during the construction, the settlements should be tolerable for the proposed pipe culvert and cambering is considered unnecessary.

These comments, however, do not apply to any widening that may be required. If widening is required, considerable settlements can be expected. We will be pleased to discuss this aspect further if needed.

#### 5.1.1.2.1.2 Precast Concrete Box Culvert

The recommended minimum granular bedding thickness under a precast concrete box type culvert is 600 mm. The subgrade should be prepared as discussed in the previous section for CSP type culvert, including rock pushing, if necessary.

As mentioned before, for subgrade prepared as detailed and since there will be neither any grade raise nor any widening, the settlements should be minimal. However, in this case, since the concrete culvert itself will be heavier than a CSP culvert and since the construction period will be somewhat longer in comparison with a CSP culvert, a total settlement of up to 70 mm should be allowed for, including a differential settlement of 50 mm in between pre-cast segments. Settlements of these magnitudes should be normally not present a problem but this aspect will need to be verified by the supplier.

As mentioned before, a CSP culvert is the preferred type at this particular site, but if it is necessary to use a concrete box culvert, details should be looked into.

#### 5.1.1.3 Culvert C79

The existing 750 mm diameter CSP culvert will be replaced with a new 800 mm diameter CSP. The length, inlet elevation and outlet elevation will match the existing and will be 43.6 m, El. 309.7 m and El. 307.5 m, respectively.

Borehole FC9-2 was advanced from the shoulder of the highway and this borehole shows the presence of 1.5 m thick loose to compact granular pavement fill, underlain by very loose to loose embankment sand fill to 4.7 m (El. 309.6 m). Underlying the embankment fill, the borehole contacted a 1.4 m thick very loose silt layer to El. 308.2 m. Borehole FC9-1, drilled near the inlet area, contacted, below a 1.1 m thick fill layer, a sandy silt to silty sand deposit to 2.3 m or to El. 308.5 m. Borehole FC9-3, which was put down towards the outlet area contacted a 0.8 m thick, very loose surficial silt layer to El. 309.1 m. These surficial deposits are underlain by a massive silty clay deposit in all three boreholes. Field vane tests show that the silty clay is typically soft to firm near the surface becoming firm to stiff with increasing depth below about El. 304 - 305 m. This deposit extends to El. 300-301 m and is underlain by a cohesive silt layer which has a soft to very stiff consistency. Below this deposit the boreholes contacted basal sandy silt, silty sand, sand and sand & gravel deposits, underlain by bedrock at about El. 297-298 m. The groundwater table at the time of our investigation was found near the o.g. levels beyond the highway embankment and at about El. 311 m within the embankment fill.

##### 5.1.1.3.1 Culvert C79 Foundation Support

With the proposed invert levels, after making allowance for bedding, the pipe is likely to be supported on the very loose sandy silt (Borehole FC9-1) at the inlet and on the silty clay throughout much of the embankment and at the outlet (Boreholes FC9-2 and FC9-3).

The foundation materials are considered too weak to support normal spread footing foundations and therefore, the use of an open bottom concrete structure is not considered feasible from a foundation engineering point of view.

As was mentioned for Culvert C78, a closed bottom concrete box culvert can be considered, but in view of the prevailing unfavourable subsurface conditions, if a concrete structure must be used, then a pre-cast concrete structure is preferable. An open bottom or closed bottom reinforced concrete structure, supported on deep foundations can also be considered. Deep foundations (e.g. steel H or tube piles) would be driven to the surface of the bedrock at about El. 296-297 m (i.e. about 12 m deep) but this would be very costly as well as requiring prolonged construction period and measures against vibrations, etc. As such it is not recommended for this project.

From a foundation engineering point of view, the use of a flexible structure, such as a CSP is the preferred choice for this project, but if necessary, the use of a pre-cast concrete box culvert can also be considered, as discussed in the following paragraphs.

#### 5.1.1.3.1.1 Corrugated Steel Pipe (CSP)

The native firm to stiff silty clay with a minimum of 30 kPa undrained shear strength is suitable to support the proposed CSP type culvert, provided that a suitable granular bedding is placed. The minimum thickness of the bedding should be 500 mm. As was described for Culvert C78, the following procedures can be followed. After excavating the site to the underside of the bedding (i.e. to 0.5 m below the bottom of the proposed pipe), the exposed subgrade should be carefully Inspected, evaluated and approved. If any organic, or otherwise unsuitable soils are detected, they should also be removed to the surface of the inorganic, suitable soil from beneath the pipe and extending at least 0.6 m beyond the foot-print of the pipe, on each side. From the borehole data, it appears that on the outlet side and throughout much of the embankment footprint, after stripping, the subgrade will consist of silty clay but towards the inlet side under the embankment and at the inlet area a very loose sandy silt deposit will be encountered. To provide a uniform bearing support, it is recommended that the very loose silt be removed to the surface of the silty clay (i.e. to El. 308.5 m at Borehole FC9-1 location) and replaced with suitable granular bedding materials.

If the evaluation of the exposed subgrade (throughout the entire length of the proposed culvert) reveals the silty clay to have an undrained shear strength of less than 30 kPa, then a 100 to 150 mm size rock fill (i.e. clear rock fill) should be pushed into the subgrade to strengthen the silty clay. After pushing the rock fill, the grade would be raised using a suitable, compacted, granular bedding material which would be at least 0.5 m thick.

For subgrade prepared in this manner a Factored Bearing Resistance at U.L.S of 80 kPa and a Geotechnical Resistance at SLS of 50 kPa can be assigned.

Under the embankment, the recommended values are less than the existing embankment loading. This, however is not considered to be a problem, since the silty clay stratum under the existing embankment would have consolidated under the stresses imposed by the embankment. Since there will be no grade raise, theoretically there should be negligible additional settlements under the existing embankment. However, a settlement of about 30 mm should be allowed for due to rebound during the construction. Based on this, if the founding subgrade is undisturbed during the construction the settlements should be tolerable for the proposed pipe culvert and cambering is considered unnecessary.

These comments, however, do not apply to any widening that may be required. If widening is required, considerable settlements can be expected. We will be pleased to discuss this aspect further if needed.

#### 5.1.1.3.1.2 Precast Concrete Box Culvert

The recommended minimum granular bedding thickness under a precast concrete box type culvert is 600 mm. The subgrade should be prepared as discussed in the previous section for CSP type culvert, including rock pushing, if necessary (i.e. if the shear strength of the clay is less than about 30 kPa). As mentioned before, for subgrade prepared as detailed and since there will be neither any grade raise nor any widening the settlements should be minimal. However, in this case, since the concrete culvert itself will be heavier than a CSP culvert and since the construction period will be somewhat longer in comparison with a CSP culvert, a total settlement of up to 70 mm should be allowed for, including a differential settlement of 50 mm in between pre-cast segments. Settlements of these magnitudes should normally not present a problem but this aspect will need to be verified by the supplier.

As mentioned before, a CSP culvert is the preferred type at this particular site, but if it is necessary to construct a concrete box culvert, details should be looked into.

#### 5.1.1.4 Bedding

After the completion of excavation to the subgrade level (i.e. to the bottom of bedding material elevation beneath the pipe), the founding subgrade should be inspected, evaluated and approved by qualified personnel. After the removal of all organic, weak or otherwise unsuitable soils to the acceptable approved subgrade level, where feasible, the clayey soil should be strengthened where required (i.e. the clay is weak), as detailed in the previous sections of this report. The grade should be raised, using Granular 'A' or Granular 'B' – Type II materials or their equivalent. Depending on the site conditions at the time of construction, the thickness of the first lift immediately above the approved subgrade level may need to be up to 500 mm, if the base is not sufficiently dry and stable to effect proper compaction.

The excavation, approval of the subgrade and the placement of the fill should be carried out as quickly as possible to avoid the loosening and disturbance of the subgrade soils.

The bedding should be in accordance with the appropriate standards (e.g. OPSD-802.010 and 802.014 for flexible pipes and OPSD-802.030, 802.031, 803.032 and 802.034 for rigid pipes) and should consist of an approved granular material, such as Granular 'A' or Granular 'B'- Type II. The recommended minimum bedding thicknesses were given in the previous sections for each individual culvert.

#### 5.1.1.5 Backfilling

The bedding and embedment material should be extended along the sides and to cover the top of the pipe. The selection and placing of the backfill should be in accordance with OPSD-802.010 and 802.014 for flexible pipes, OPSD-802.030, 802.031, 803.032 and 802.034 for rigid pipes and OPSD-803.010 for concrete culverts. 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 Granular 'B' sub-base courses should be compacted to at least 100% of the SPMDD.

We would like to point out that the performance of flexible pipe culverts is largely dependent on the side support provided by the backfill and the adjacent soils. The use of proper backfill material and especially



good compaction are, therefore, necessary for proper side support. The use of heavy compaction equipment should, however, be avoided immediately adjacent and above the pipes, 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.

Reference should be made to SP105S10 for compaction behind the culvert and any retaining walls.

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 any rigid culvert walls and any wing walls should be in accordance with the Canadian Highway Bridge Design Code, (CHBDC) S6-06. For design purposes, the following properties can be assumed for backfill.

#### **Compacted Granular 'A' and Granular 'B' Type II**

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

Unit weight = 22 kN/m<sup>3</sup>

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/m<sup>3</sup>

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

$K_0$  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.

#### 5.1.1.6 Construction

The construction of the culvert should be in accordance with SP 421S01 for pipe culvert installation and SP 422S01 for precast concrete box culvert installation in open cut method.

To maintain the flow of water across the highway during the construction, either a new temporary culvert can be constructed, which would be removed after the construction or the area can be enclosed with a temporary dyke and the collected water can be pumped across the highway from a pipe buried in the upper granular portion of the pavement fill.

Depending on the groundwater level encountered at the time of the construction, dewatering will be required to facilitate the construction and to preserve the load carrying capability of the founding soils. The groundwater can possibly be depressed by means of gravity drainage and pumping from strategically placed filtered sumps. However, in this manner the groundwater will probably be depressed by not more than about 0.5 m. To depress the groundwater level further deeper, other methods such as deep wells and/or well points would be required. It should, however, be pointed out it may not be feasible to place well points across the highway unless traffic is totally diverted.

We recommend that the contractor be made aware of possible 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.

We recommend that an NSSP be included in the Contract Documents to warn the contractor of possible dewatering requirements and the possible presence of cobbles and boulders in the embankment fill.

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 Fill	Type 3 soil above water level Type 4 soil below water level
Silt, Sandy Silt, Silty Sand	Type 4 soil
Silty Clay	Type 3 soil above water level Type 4 soil below water level

It is expected that temporary shoring will be required to support the excavations. 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 coefficient of lateral earth pressures given in Table 5.1.1.6.1 can be used for the design of the temporary shoring system.

**Table 5.1.1.6.1: Recommended Unfactored Parameters for Temporary Shoring Design**

Soil Type	$K_a$	$K_o$	$K_p$	Unit Weight (kN/m <sup>3</sup> )
<b>Culvert C77</b>				
Pavement Fill	0.27	0.42	3.7	21.5
Embankment Fill	0.33	0.50	3.0	20.0
Upper Silts and Sands	0.42	0.58	2.4	18.0
Silty Clay	0.45	0.60	2.2	17.0
Gravelly Sand to Sand & Gravel	0.27	0.42	3.7	21.0
<b>Culvert C78</b>				
Pavement Fill	0.27	0.42	3.7	21.5
Embankment Fill	0.33	0.50	3.0	20.0
Upper Silts	0.42	0.58	2.4	18.0
Silty Clay	0.43	0.60	2.3	17.0
Lower Silts & Sands	0.35	0.52	2.9	18.5
<b>Culvert C79</b>				
Pavement Fill	0.27	0.42	3.7	21.5
Embankment Fill	0.34	0.51	2.9	20.0
Upper Silts	0.44	0.60	2.3	18.0
Silty Clay	0.43	0.60	2.3	17.0
Lower Silts	0.41	0.59	2.5	17.0
Gravels & Sands	0.28	0.43	3.6	20.5

The shoring system will need to be designed by a Professional Engineer experienced in this type of work. The temporary shoring system (i.e. tiebacks or dead-man and anchor system) may be required due to the height of fill (i.e. in excess of 4 m). We will be pleased to provide information on this aspect, if necessary.

#### 5.1.1.7 Detour Construction

In order to replace the culverts in open cut, if necessary, the highway embankment will need to be temporarily lowered and widened to retain one lane of traffic. In this case, the existing road grade at Culvert C77 will need to be lowered by 1m with a 2m widening on the north and south sides of the highway embankment. The existing road grade at Culvert 78 will need to be lowered by 2 to 2.5 m with a 7 m widening on one side of the highway. The existing road grade at Culvert C79 will need to be lowered by 1.5 m with a 5 m widening on one side of the highway. These temporary embankment widenings with 2 m offset of traffic load from the edge of embankment (at widening side) were modelled using the computer program Slope/W, assuming temporary outside side slopes of 1.5H:1V for the proposed widening. We understand that inside slope will be 1V:1H within the existing embankment. The factor of safety determined was greater than 1.3 for the proposed widening. Steeper side slopes are not recommended. We understand that fill materials used for the widening will be removed after the construction and the original ditch line maintained.

Based on the results of the analyses the proposed widening should not have an adverse effect on the stability of the existing embankment for the duration of construction (less than a week). All unsuitable soils (if encountered during the construction) should be removed beyond the toe area of the embankment within the footprint of the proposed widening. However, based on the boreholes drilled at the toe of embankment, no significant unsuitable soils were encountered and therefore stripping is not considered necessary for temporary widening. We understand that widening at the curve/grade near the South River Bridge will be difficult to construct.

The temporary widening will cause some settlements in the foundation soils. Based on the borehole data and assuming that the widening will be removed promptly after the construction, the anticipated settlements should not exceed 50 mm, over a construction period of about 7days. We will be pleased to elaborate on this subject once the details are known (e.g. cambering).

For the widening of the embankment, proper benching of the existing slopes should be implemented as per OPSD-208.010. The existing material from the embankment grade lowering can be used for the proposed embankment widening. Imported material should consist of an easily compactible material. The upper 300 – 400 mm should consist of Granular 'A' or 'B' Type II material to accommodate the traffic loading.

#### 5.1.2 Tunnelling

For the tunnelling options, we understand that the culverts will possibly be replaced by a 1.0 to 1.2 m diameter smooth steel casing with an inner 800 mm CSP. The inverts of the existing CSPs are presented in Table 5.1.1 in Section 5.1 of this report. A classification of soils for tunnelling purposes, commonly used in Ontario, is given in Appendix F. The following table summarizes the anticipated soil conditions along the alignment of each tunnelled culvert, as well as the categories each of the soils fall into.

**Table 5.1.2.1: Subsurface Conditions Along Tunnel Alignment**

Culvert	Anticipated Soil Condition	Tunnelman's Ground Classification
C77	Loose Silt, above groundwater table	"Slow Ravelling"
	Loose Silt, below groundwater table	"Flowing"
	Very loose Silty Sand to Sandy Silt below groundwater table	"Fast Ravelling" or Flowing"
	Firm to stiff Silty Clay	"Squeezing to Firm"
C78	Very loose Silt, above the groundwater table	"Slow to Fast Ravelling"
	Very loose Silt, below the groundwater table	"Flowing"
	Soft to firm Silty Clay	"Squeezing"
	Very loose to loose Sandy Silt to Silty Sand FILL, below groundwater table	"Flowing" / "Slow Ravelling"
C79	Very loose Sandy Silt to Silt, below/above groundwater table	"Flowing" / "Slow to fast Ravelling"
	Soft to firm Clayey Silt to Silty Clay	"Squeezing"
	Very loose to loose Sand embankment FILL, below groundwater table	"Flowing"
	Very loose to loose Sand embankment FILL, above groundwater table	"Cohesive Running" / "Slow Ravelling"

From this information, it can be surmised that the tunnel can be expected to possibly proceed on mixed face conditions.

According to this, the above described soils fall into the following categories.

- "Flowing to Slow Ravelling to Squeezing/Firm" conditions for Culvert 77
- "Flowing to Squeezing to Slow to Fast Ravelling" conditions for Culvert 78
- "Flowing to Squeezing to Cohesive Running/Slow Ravelling" conditions for Culvert 79

These are not favourable soil conditions for most tunnelling methods unless groundwater is lowered to stabilize the water-bearing strata. At the time of our investigation, the groundwater level is inferred near the existing ground surface beyond the toe of the highway embankment and perched within the highway embankment, as noted in Section 5.1. From this, it can be concluded that the groundwater level will likely need to be lowered, although to some extent this will depend on the site and weather conditions at the time of construction.

Another potential problem that may arise with regards to tunneling operations is shaft construction at the entrance and possibly at the exit points. The site and especially the silt deposits will need to be dewatered for this purpose. As well the very loose to loose silts and soft/firm silty clays will not provide sufficient thrust block support. The underlying firm to stiff cohesive soils may provide some support but after some yield (i.e. lateral movement). For comments on excavation and dewatering for shaft construction refer to Section 5.1.1.6.

### 5.1.2.1 Tunnelling Options

The following are possible typical options for tunneling which are commonly used in Ontario.

Jack and Bore

Tunnelling with Hand Mining methods

Pipe Jacking with TBM

Micro-Tunneling

Pipe Ramming

Horizontal Directional Drilling (HDD)

The selection of a preferred option will depend, among other factors, the construction cost, practicability of construction, risk of ground subsidence, scheduling, etc.

The following table represents an overview of the six aforementioned methods and is intended only to assist the designers in their choice of most suitable method. However, contractors may come up with more suitable options or variations of such methods.

**Table 5.1.2.1.1: Summary of Tunnelling Options**

Construction option	Advantage	Disadvantage
Jack and Bore	<ul style="list-style-type: none"> <li>• Technique commonly used locally</li> <li>• Skill labour, equipment and contractor available locally</li> <li>• Relatively lower cost</li> <li>• Technique suitable for large variety of soils conditions</li> <li>• Minimum ground subsidence if operated properly</li> </ul>	<ul style="list-style-type: none"> <li>• Not suitable in running and/or flowing conditions</li> <li>• Poor alignment control in mixed face condition</li> <li>• Requires thrust block construction that may be difficult in poor soils</li> <li>• Requires groundwater control</li> <li>• Large boulder may create problems</li> </ul>
Hand Mining	<ul style="list-style-type: none"> <li>• Skill labour, equipment and contractor available locally</li> <li>• Relatively lower cost</li> </ul>	<ul style="list-style-type: none"> <li>• Good alignment control depends on workmanship</li> <li>• Requires temporary support and permanent lining</li> <li>• Higher risk of ground subsidence</li> <li>• Requires groundwater control</li> <li>• Grouting required to control flowing conditions</li> <li>• Poor alignment control in mixed face condition</li> </ul>
Pipe Jacking with TBM using earth pressure balance	Considered uneconomical	
Micro-Tunnelling	Considered uneconomical	
Pipe Ramming	<ul style="list-style-type: none"> <li>• Method can be executed with any ground condition (except large boulders) with adequate precautions</li> <li>• Pipe installed during the operations, avoid double handling</li> <li>• Tolerable ground subsidence if</li> </ul>	<ul style="list-style-type: none"> <li>• Specialized operation requiring good operator skill and experience</li> <li>• Very tight alignment and grade tolerance and expensive corrective actions if mis-aligned</li> <li>• Liquefaction of very loose to loose soils</li> </ul>

Construction option	Advantage	Disadvantage
	operated properly • Good alignment control	due to vibrations if not dewatered.
Horizontal Directional Drilling (HDD)	<ul style="list-style-type: none"> <li>• Method can be executed with any ground condition (except large boulders) with adequate precautions</li> <li>• Minimum ground subsidence if operated properly</li> <li>• Good alignment control</li> </ul>	May be suitable if entrance and exit points can be established some distance beyond the embankment (i.e. property availability).

Details of each option are briefly discussed below.

#### 5.1.2.1.1 Jack and Bore Method

Jack and bore method forms a borehole from a drive shaft to a reception shaft by means of rotating cutting head. Spoil is transported back to the drive shaft by helical auger flights rotating inside a steel casing. The casing is jacked in place simultaneously with the augering operation. After the installation of the steel casing, the utility pipe is installed inside the casing and the gap between the casing and the pipe is grouted. The maximum casing diameter used in this operation is typically limited to about 1.5 m for most contractors in Ontario.

From the findings of the boreholes, this tunnelling method is not suitable in the flowing conditions anticipated along the culvert alignments without extensive dewatering or grouting. Dewatering was discussed in Section 5.1.1.6 of this report and will be further discussed later on in Section 5.1.2.2.

Another aspect of tunnelling with this method is that the construction of shafts in the water bearing upper silts and sands can be expected to be difficult. In addition, the surficial soils encountered at the culvert locations will provide little or no passive resistance for a thrust block to facilitate jacking operations (i.e. deep foundations may be required) and these aspects can be expected to increase the cost of tunnelling by this method.

We recommend that a specialist contractor(s) be consulted for tunnelling by jack and bore method, with the anticipated soil conditions, although extensive grouting/dewatering would be required.

#### 5.1.2.1.2 Tunnelling by Hand Mining

In a hand mining operation, the excavation of the tunnel is accomplished manually and a temporary ground support system is required during the operation. The temporary ground support system can be steel or concrete segmental liner or steel ribs with wood lagging. Groundwater control may be required to minimize water leakage into the tunnel. Workers are required inside the tunnel to perform the excavation and/or spoil removal. The excavation is accomplished by hand mining with the assistance of small excavation tools.

With this method, control of alignment and grade is accomplished by overmining in the direction of the change and the pipe will move into the overmined area as it is pushed forward.

The main advantage of this technique is that it can be economical, and that large boulders can be removed.

This technique is limited by the difficulty of controlling the grouting quality and its impact on the environment. Higher risk of ground subsidence may also be encountered with anticipated settlements

could be in the order of 20 to 40 mm. In addition, with this method, the tunnel project will probably take a longer time to complete, in comparison with many of the other methods mentioned.

Similar to jack and bore method, the presence of flowing conditions at the advancing face can create problems, as well dewatering will be required to lower the groundwater level to below the invert level or extensive grouting. This method is unlikely be suitable for this project, although it too could be discussed with a specialist contractor(s).

#### *5.1.2.1.3 Pipe Jacking with TBM*

The method of pipe jacking with TBM installs a prefabricated pipe through the ground from a drive shaft to a reception shaft. The pipe is pushed by jacks located in the drive shaft and the jacking force is transmitted through the pipe to the face of the excavation. The excavation with this method is accomplished by a TBM (Tunnel Boring Machine) and the spoil is transported out of the jacking pipe and shaft manually or mechanically. Typically, pipe jacking with TBM is applicable to tunnels with relatively larger diameter (e.g. 1 to 3 m).

This tunneling method is so versatile that it can be executed with virtually any ground conditions (except large boulders) with adequate precautions. In unstable soil conditions (i.e. silty sand to sandy silt below the groundwater level), an Earth Pressure Balance Machine (EPBM) or slurry shield TBM is required to counterbalance the ground and hydrostatic pressures and to minimize ground subsidence. With EPBM, dewatering of the wet silty sand to sandy silt and/or the glacial till would be less stringent.

The main disadvantage of this method is its high capital and set-up costs. This technique also requires good operator skill and experience. In addition, this method has a very tight alignment and grade tolerance since the permanent lining (the pipe segments) is being installed during the tunnel operation. If large boulders are encountered, hand-mining may have to be employed which could lead to project delay and extra costs.

With pipe jacking and the use of EPBM, if operated properly, the maximum ground settlement is expected to be minimal in the order of 10 to 20 mm, which is considered acceptable under the highway.

In our opinion this method will not be cost effective in the present case, and is therefore not recommended based on cost factor.

#### *5.1.2.1.4 Micro-Tunnelling*

This technique is the improvement of the pipe jacking technique with TBM. It is a remotely controlled, guided pipe-jacking process that provides continuous support to the excavation face. The guidance system usually consists of a laser mount in the drive shaft, communicating a reference line to a target mounted inside the tunnelling machine. This technique provides ability to control excavation face stability by applying mechanical or fluid pressure to counterbalance the earth and hydrostatic pressures.

The main advantage of this technique is that it is sophisticated and will most likely complete the project in shorter time. It will also complete the tunneling operation with even less ground subsidence, if operated properly, which is estimated to be less than 12 mm.

The main disadvantage of this method is its very high cost and it is therefore not recommended based on economics.



#### 5.1.2.1.5 *Pipe Ramming*

This tunnelling method using smooth steel casing is considered feasible for this site/soil condition with proper dewatering.

In a pipe ramming operation, a pneumatic ramming tool attached to the rear of a steel casing drives the casing into the ground with repeated percussive blows. The installed pipe usually has an open end that allows the soil to enter the casing during the installation. The spoils inside the casing can be removed either during or after the installation, by auger, compressed air or water jetting. After completing the installation, the CSP (800 mm diameter) could be installed inside the casing and the gap between the casing and the pipe can be grouted or the casing itself could remain as the conduit.

This method of installation is mostly used with pipes less than 1.5 m but up to 1.8 m in diameter may be feasible and for pipe installation over relatively short distances (i.e. less than 45 m). Although longer distance (up to 100 m) has been achieved in favorable ground conditions (e.g. firm to stiff clayey soil), the length of the tunnel installed by pipe ramming is limited by the ground conditions. Coffey contacted Marathon Drilling of Ottawa, Ontario (one of pipe ramming contractors operating in Ontario) and they indicated that pipe ramming is a feasible option for this site. Dewatering will however need to be implemented to minimize the liquefaction potential of the very loose to loose sands and silts as well as to facilitate construction. We recommend that the applicability of this method to this particular project as well required dewatering be discussed with a specialist contractor(s).

Another disadvantage of pipe-ramming is the vibrations created and noise generated, which may be objectionable while the traffic is maintained on the highway. We understand that frequency of the pipe ramming could be adjusted to a more tolerable level, if required. Vibrations may also cause settlements due to pipe ramming but those settlements expected to be tolerable. One other potential problem with pipe ramming is the presence of large boulders. Although no boulders were encountered in the boreholes during this investigation, this is always a possibility and this aspect should be relayed to the contractor via an NSSP. As mentioned before, dewatering will be required to minimize the liquefaction potential, although this will be difficult below the highway embankment. Another alternative to minimize liquefaction is ground improvement of the founding sensitive soils by grouting. We recommend that this too should be discussed with a specialist contractor(s), as well as the need for possible additional borehole(s). Close monitoring of embankment slope during the pipe ramming is also recommended.

#### 5.1.2.1.6 *Horizontal Directional Drilling (HDD)*

This trenchless technology with additional berms at the entrance and exit locations is feasible for this site/soil condition provided that adequate space is available well beyond the embankment.

This method consists of pilot boring, back reaming and pipe pulling. Drilling begins with a small diameter pilot hole along a designated alignment, using flexible drill rods with remote controlled steering system. After the pilot boring, a back reamer is installed and drilled back through the pilot hole to achieve the required diameter for the pipe to be installed. Typical ratio of diameter of reamer to pipe is 1.3 to 1.5. Special drilling fluid is used to prevent the collapse of borehole as well as providing a lubricant for the drilling and flushing spoils. Selection of reamer and drilling fluid highly depends on the ground conditions.

The feasible diameter of pipe or tunnel in this method is 0.5 to 1.5 m and drive length can be up to 2000 m for smaller diameter pipes. The subsurface conditions encountered in the boreholes may be feasible for a horizontal directional drilling operation if the drilling mud would support the soil and prevent a 'flowing' or 'ravelling' ground condition into the hole during the drilling.

Reference should be made to OPSS 450 for this method, as well for related specifications.

Often the HDD installation could be impeded by obstructions in the ground, such as boulders. In the present case, no boulders were encountered in the zone where tunnelling would be carried out. If the embankment fill was placed under proper engineering supervision, normally boulders should not exist. However, the contractor would need to be prepared to deal with such a condition, should it occur. In addition, boulders may be present in the alluvial soils depending on the water velocity in the watercourse. These and other details such as the need for dewatering and property requirements should be discussed with a contractor specializing in this method of trenchless construction. Proper distance for entry and exit points and property requirements during the construction will need to be sorted out to determine the feasibility of this method.

#### *5.1.2.1.7 Recommended Tunnelling Option*

Based on the above mentioned tunnelling methods and from geotechnical point of view, it is our opinion that, Pipe Ramming and Horizontal Directional Drilling (HDD) are two of the more suitable tunnelling options for this project. Pipe ramming may cause an adverse effect on the stability of the embankment side slope as well as presenting a potential for liquefaction of the wet, loose silt and sand soils during the construction due to vibrations generated by pipe ramming. Dewatering of the loose and wet sands and silts is required. In addition, close monitoring of the embankment side slopes and embankment settlements is recommended during the pipe ramming. Horizontal Directional Drilling will, on the other hand, create less vibration in comparison with pipe ramming but it requires more work space at entry and exit points than pipe ramming. Also, pipe ramming can be performed over the existing pipe with a larger diameter pipe while Horizontal Directional Drilling can not be done at the exact same location of existing culvert. The existing culvert can, however, be used for the purpose of water diversion during the construction using Horizontal Directional Drilling.

Dewatering at the pit or shaft locations and disposal of the spoils or spoils with drilling mud may present challenges.

An NSSP should be included in the contract to direct the contractor on the requirements for the tunnelling method and associated installation performance requirements.

#### *5.1.2.2 Dewatering for Tunnelling*

The design of the dewatering system for tunneling will depend on the required draw-down for the selected construction (tunnelling) method. For Pipe Ramming, draw-down will be required to stabilize the soil, especially at the pit shaft locations to dewater the entry and exit pits, and the face of the entry shaft, in order to dewater the spoils and to minimize liquefaction at the entry location. Draw-down needs to be effected by pumping from the toe of the embankment (i.e. dewatering from the surface of the travelled portion of the road will unlikely be possible). Therefore, the dewatering system will need to be carefully designed and executed to effect the desired draw-down effect. Due to the low permeability of silt deposits

present at all the culvert locations, consideration should be given to well pointing or deep wells for dewatering purposes where feasible (e.g. clay may interfere with operation of well points). As was mentioned before, we recommend that this responsibility be assigned to the contractor via an NSSP. In addition, although this is unlikely, any effects of dewatering on the nearby structures, services and wells may need to be studied.

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. The flow would then be diverted to the completed new culvert and the existing culvert would be removed or plugged. Alternatively, if the new culvert will be at the existing culvert location (i.e. a larger diameter pipe will be rammed in place around the existing culvert), the water course will need to be dammed and the water flow pumped over to the existing outlet.

#### 5.1.2.3 Design Parameters

The bulk units for the soil types were provided in Section 5.1.1 of the report. For soils surrounding the tunnel, the ratio of the horizontal to vertical earth pressure  $K$ , can be taken as 0.5 (i.e.  $K=0.5$ ), for design purposes.

#### 5.1.2.4 Settlement Due to Tunnelling

Settlements caused by tunneling in the overburden soils are the aggregate of settlement caused by ground loss due to over-excavation, localized liquefaction of the very loose to loose soils from the pipe ramming effect, or ground relaxation in response to excavation, and settlement due to the deformation of flexible primary tunnel support.

The first type of settlement is the direct result of the movement of ground into the tunnel heading. The factors which influence the magnitude of this settlement include soil strength and stiffness, the method of tunnelling and the quality of tunnel operations (including the method of handling localized conditions such as removing boulders). Even when tunnelling is carried out apparently through homogenous soils with the same equipment and crews, ground settlements typically vary by a factor of 2 or 3. This variation can be ascribed to items such as ploughing of the shield or use of overcutters, quality and speed of ring grouting and localized variations in soil type, strength or stiffness.

In summary settlement (or heave) of the road surface due to tunnel construction is difficult to estimate but generally with good workmanship settlements over the tunnel centerline should not exceed 25 mm. Beyond the centerline the settlements would gradually decrease to zero, typically at a distance of about three diameters.

In addition to these settlements, settlements due to dewatering (if any) will need to be taken into consideration. The magnitude of settlement due to dewatering will depend on the depth of the lowering the water table. The amount of settlement due to dewatering should not exceed 10 mm.

The term 'ground loss' as discussed above does not include unexpected, uncontrolled loss of ground resulting from tunnel face instability. Much greater settlements than the estimates given above will occur where there is instability at the face.

To ensure that ground settlements are limited to acceptable values, it is recommended that ground movements along the tunnel in critical areas be monitored during and following the tunnelling operation. We also recommend that during tunnelling operations the installation be inspected fulltime by a geotechnical engineer appointed by the QVE.

Settlement monitoring could consist of paint mark points on the pavement along the centerline of the culvert and beyond the culvert. Surface settlement points should also be installed beyond the paved portion (i.e. in the shoulder). In addition, we recommend that consideration be given to deep settlement points (e.g. placed about 0.8 m above the tunnel's crown) in the shoulders. This is because deep settlement points will react to any ground loss settlement during tunnelling much faster than surface settlement points, since time-dependent arching effect is less pronounced immediately above the crown. The settlements will need to be monitored with reference to reliable, frost free benchmark(s).

We recommend that a minimum of three sets of repeatable baseline readings be taken on all of the settlement points well in advance of the start of tunnelling. Settlement monitoring should be conducted at least three times daily during the installation of the tunnel under the road embankment. The frequency of readings can then be further reduced to once daily for ten days, weekly for a period of one month and then once monthly for four months following grouting of the tunnel annular space.

Should settlement monitoring indicate excessive ground movement prior to the tunnel reaching the travelled lanes, immediate changes to the tunnelling and ground support procedures must be adopted. A contingency plan should be provided to the CA prior to tunnelling. Table 5.1.2.4.1 details the recommended monitoring 'Alert Levels.'

**Table 5.1.2.4.1: Recommended Settlement 'Alert Levels'**

Ground Movement	Notes
<6 mm	Proceed. No action required.
6-15 mm	Immediately notify the MTO and the geotechnical engineer for further assessment. Proceed with caution.
>15 mm	Halt tunnelling and bulkhead the tunnel face until further assessment is carried out by the MTO and the geotechnical engineer; Carry out immediate remedial work to the settlement zone as approved by the MTO and geotechnical engineer.

In addition to settlement monitoring, during tunnelling, the quantity of the excavated soils should also be monitored and compared with the theoretical volume of excavation in order to assess the risk of excessive over-excavation.

## **5.2 Erosion Protection**

Erosion and scour protection should be provided at the culvert inlet and outlet (including the side slopes). 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. The boreholes indicate that the native soils can be expected to consist of silts, silty sands or silty clays. The silts and silt sands or clayey silts. These soils are considered to be highly erodible and frost-susceptible soils.

For a concrete culvert, we recommend that a concrete cut-off (apron)/ head wall be constructed both at the inlet and outlet to prevent seepage beneath and around the culvert, especially through the granular bedding, granular backfill around the culvert and the underlying relatively pervious silts. 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, outlet and at an intermediate location.

For a CSP culvert and a concrete culvert, at the inlet, consideration may also be given to the use of a clay seal or equivalent. The purpose of the clay seal is to ensure that the 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 to 8 m beyond the inlet, unless a fish bearing water course could be impacted. In this case, an alternative option could be considered.

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. Where the subgrade is found to consist of silt or silty sand, a layer of granular 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 level). The granular filter material underlying the rock protection (where necessary) can consist of a suitable granular material such as Granular 'A'. Alternatively, a suitable geotextile can be used beneath the rock fill, in lieu of the granular filter material.

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

### **5.3 Bearing Surfaces**

We recommend that all bearing surfaces should be inspected and approved by a qualified Geotechnical Engineer (QVE).

### **5.4 Frost Protection**

Design frost protection for the general 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 culverts are finalized, our recommendations be reviewed for their specific availability. The Limitation of Report, as quoted in Appendix H, are an integral part of this report.

For and on behalf Coffey Geotechnics Inc.



Gwangha Roh, Ph.D.



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# Appendix F

## **Tunnelman's Ground Classification and Probable Working Conditions**

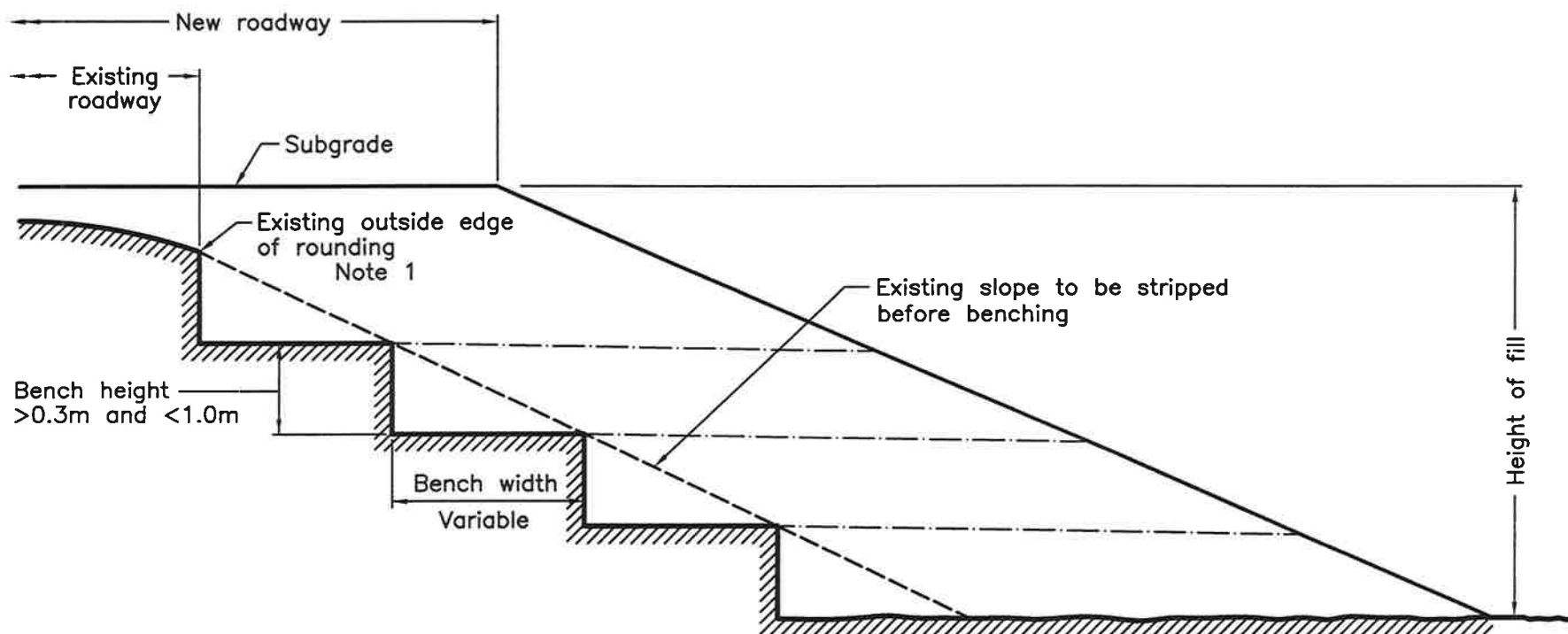
### Tunnelman's Ground Classification and Probable Working Conditions

Soil Classification	Representative Soil Types	Tunnel Working Conditions
Hard	Very hard calcareous clay; cemented sand and gravel	Tunnel heading may be advanced without roof support
Firm	Loess above GWT; Various calcareous clay with low plasticity	Tunnel heading may be advanced without roof support and the permanent support can be constructed before the ground will start to move
Slow Ravelling and Fast Ravelling	<i>Fast Ravelling</i> occurs in residual soils or in sand with clay binder below the GWT. Above the GWT, the same soils may be <i>Slowly Ravelling</i> or even Firm	Chunks or flakes of material begin to drop out of roof or the sides sometime after the ground has been exposed. In <i>Fast Ravelling</i> ground, the process starts within a few minutes; otherwise it is classed as Slow Ravelling
Squeezing	Soft or medium-soft clay	Ground slowly advances into tunnel without fracturing and without perceptible increase of water content in ground surrounding the tunnel (may not be noticed in tunnel but cause surface subsidence)
Swelling	Heavily pre-compressed clays with a plasticity index in excess of about 30; Sedimentary formations containing layers of anhydrite.	Like squeezing ground, moves slowly into tunnel, but the movements is associated with a very considerable volume increase in the ground surrounding the tunnel.
Cohesive Running and Running	<i>Cohesive running</i> occurs in clean, fine moist sand  <i>Running</i> occurs in clean, coarse or medium sand above the GWT	The removal of the lateral support of any surface rising at an angle of more than about 34° to the horizontal is followed by a 'run,' whereby the material flows like granulated sugar until the slope angle becomes equal to about 34°. If the 'run' is preceded by a brief period of raveling, the ground is called <i>Cohesive Running</i>
Very Soft Squeezing	Clays and silts with high plasticity index	Ground advances rapidly into the tunnel in a plastic flow
Flowing	Any ground below the GWT that has an effective grain size in excess of about 0.005 mm	Flowing ground moves like a viscous liquid. It can invade the tunnel not only through the roof and the sides but also through the bottom. If the flow is not stopped, it continues until the tunnel is completely filled.
Bouldery	Boulder glacial till; rip-rap fill; some land slide deposits, some residual soils. The matrix between boulders may be gravel, sand, silt, clay or combinations of thereof.	Problems occurred in advancing shield or in forepoling; blasting or handmining ahead of machine may become necessary.



# Appendix G

OPSD



**NOTES:**

- 1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.
- A Benching is not required on existing slopes flatter than 3H:1V.

- B Benches are to be excavated one level at a time and the compacted fill brought up before the next benching level is excavated.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2003

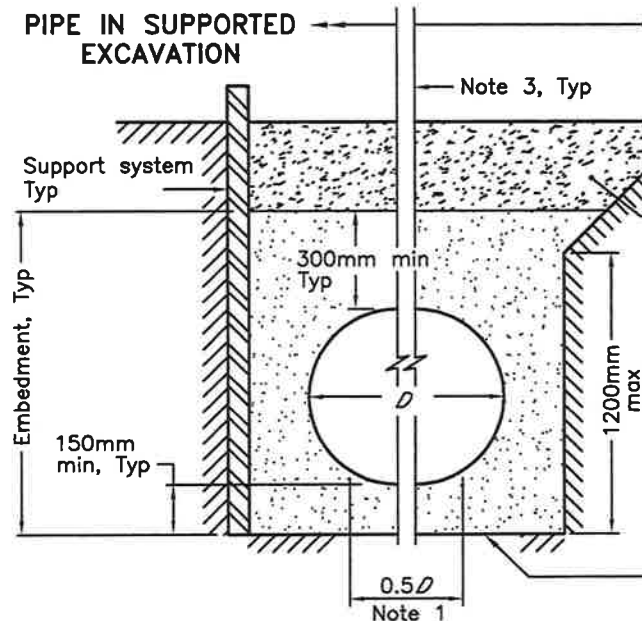
Rev 1

**BENCHING OF EARTH SLOPES**



**OPSD – 208.010**

# PIPE IN SUPPORTED EXCAVATION



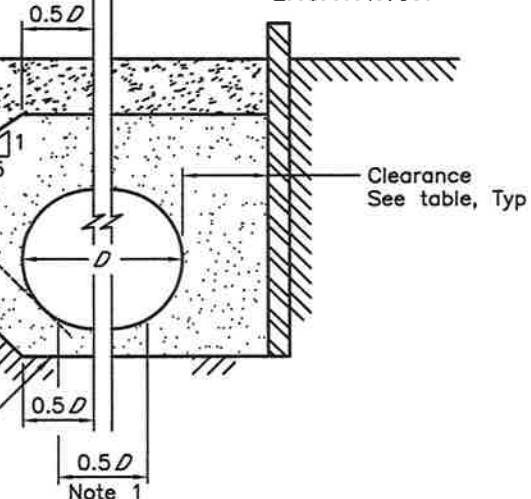
# PIPE IN UNSUPPORTED EXCAVATION

Subgrade  
Backfill material  
For pipe culvert frost treatment  
Note 2

TYPE 1 OR 2  
SOIL

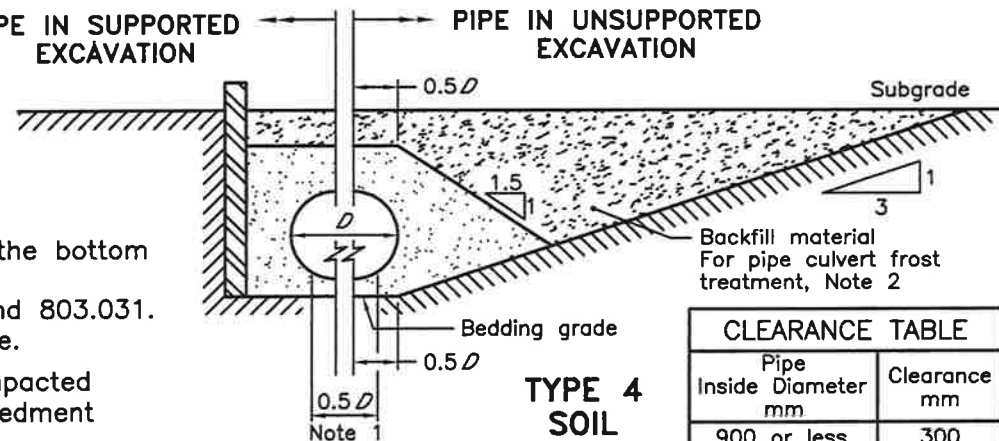
Bedding grade

# PIPE IN SUPPORTED EXCAVATION



TYPE 3  
SOIL

# PIPE IN SUPPORTED EXCAVATION



TYPE 4  
SOIL

## 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

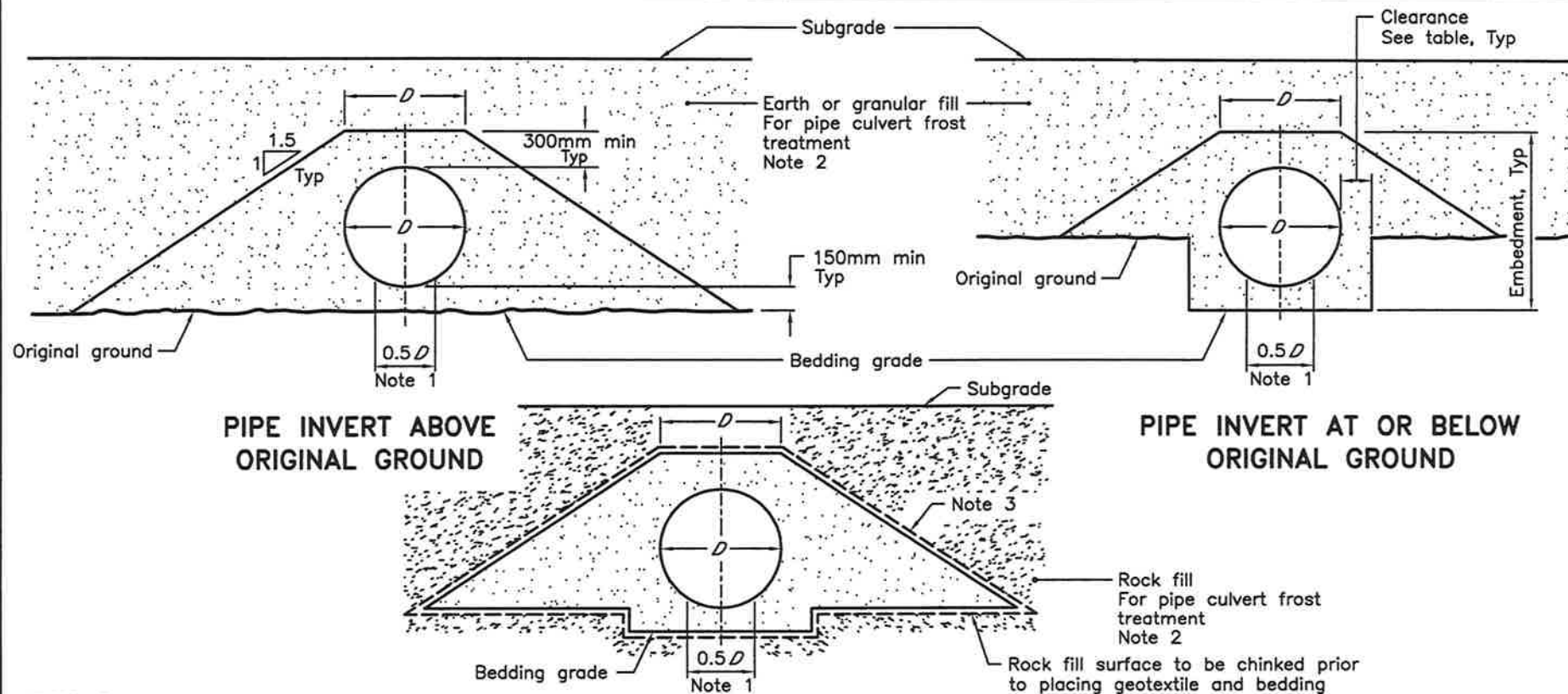
Nov 2005

Rev 1

FLEXIBLE PIPE  
EMBEDMENT AND BACKFILL  
EARTH EXCAVATION

OPSD - 802.010





# **LEGEND:**

$D$  - Inside diameter

## **NOTES:**

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

## **PIPE EMBEDMENT WITH ROCK FILL UNDER AND OVER THE PIPE**

### **CLEARANCE TABLE**

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

ONTARIO PROVINCIAL STANDARD DRAWING

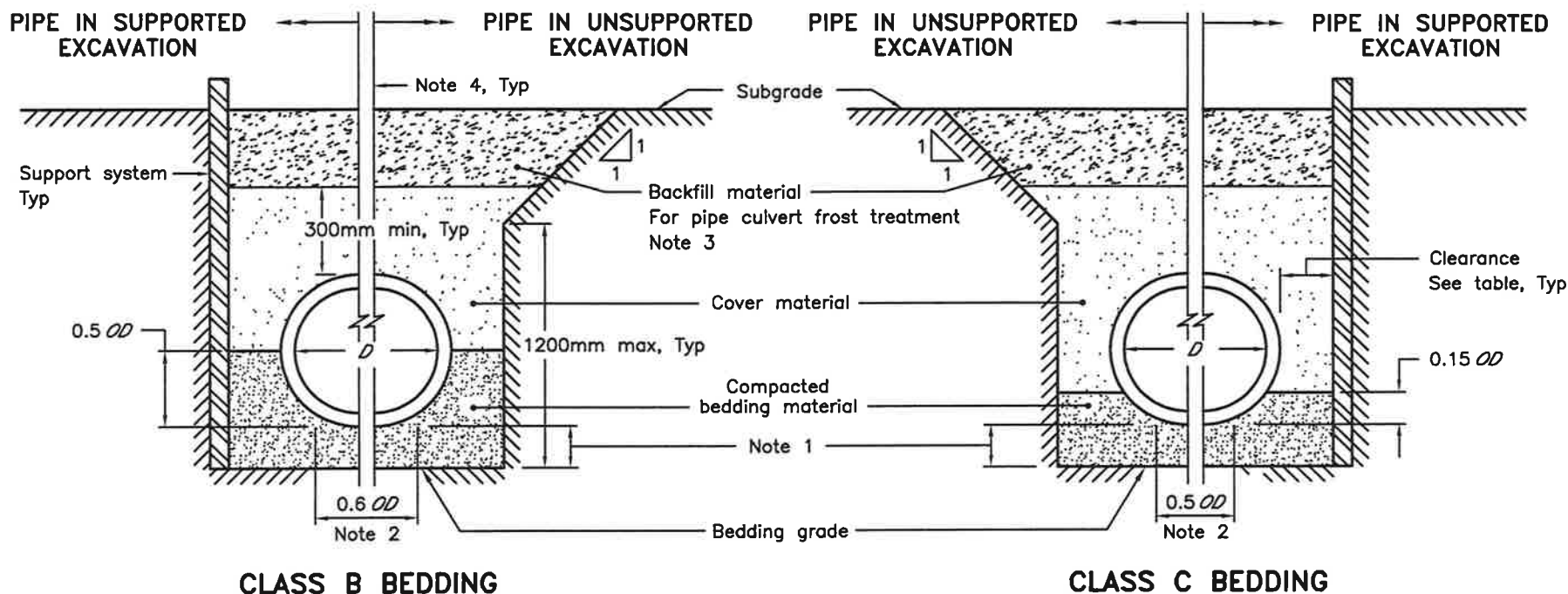
Nov 2005

Rev 1

**FLEXIBLE PIPE EMBEDMENT  
IN EMBANKMENT  
ORIGINAL GROUND: EARTH OR ROCK**

**OPSD - 802.014**





#### NOTES:

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

#### LEGEND:

$D$  - Inside diameter  
 $OD$  - Outside diameter

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

ONTARIO PROVINCIAL STANDARD DRAWING

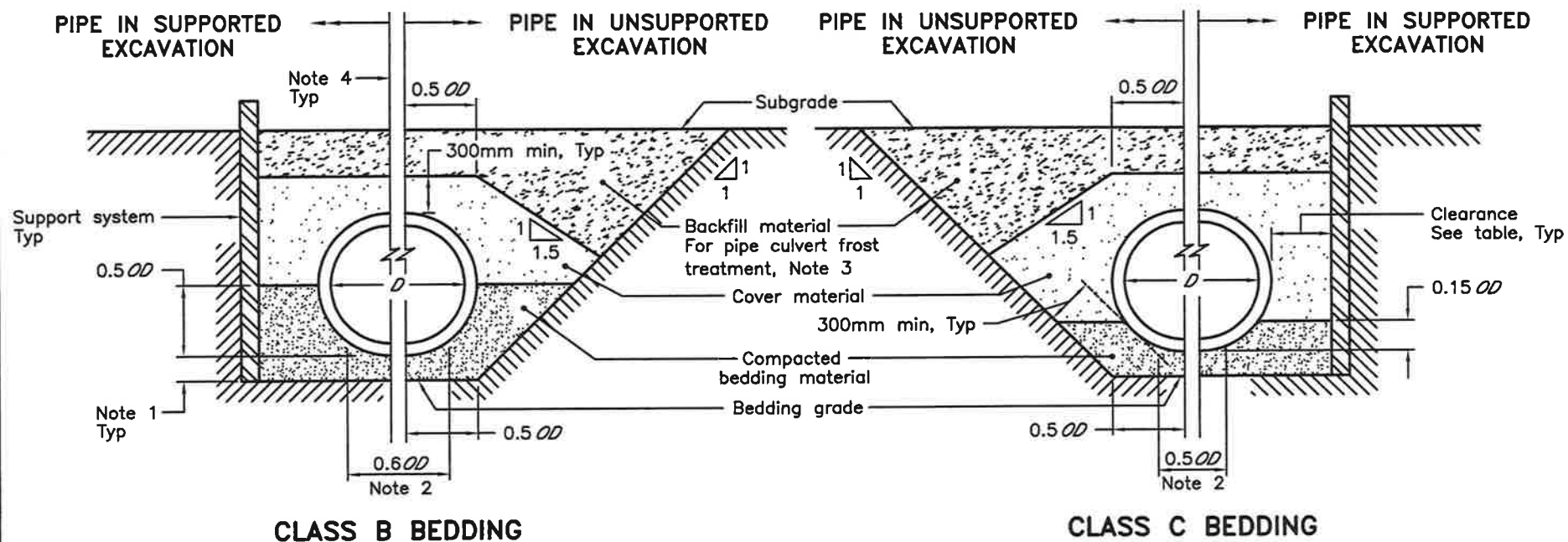
Nov 2005

Rev 1

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

**OPSD - 802.030**





#### NOTES:

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

#### LEGEND:

$D$  – Inside diameter  
 $OD$  – Outside diameter

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

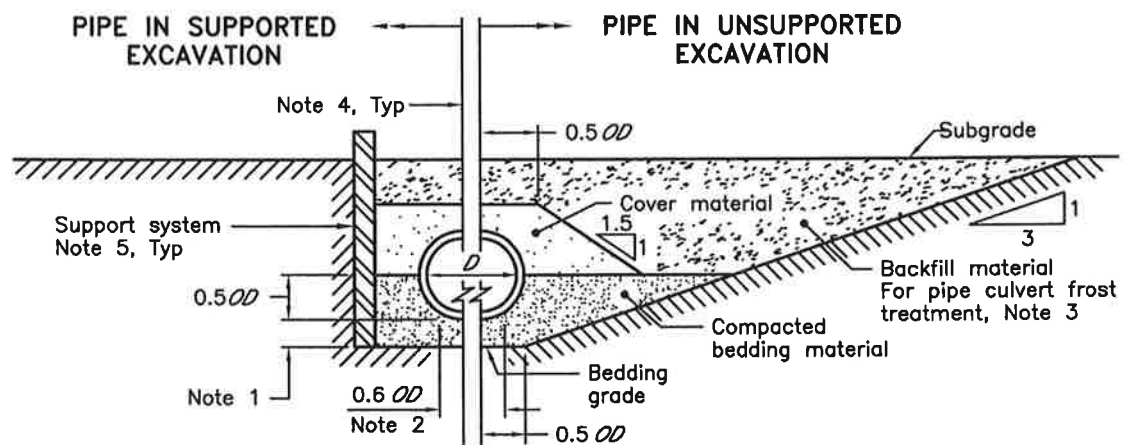
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 Rev 1

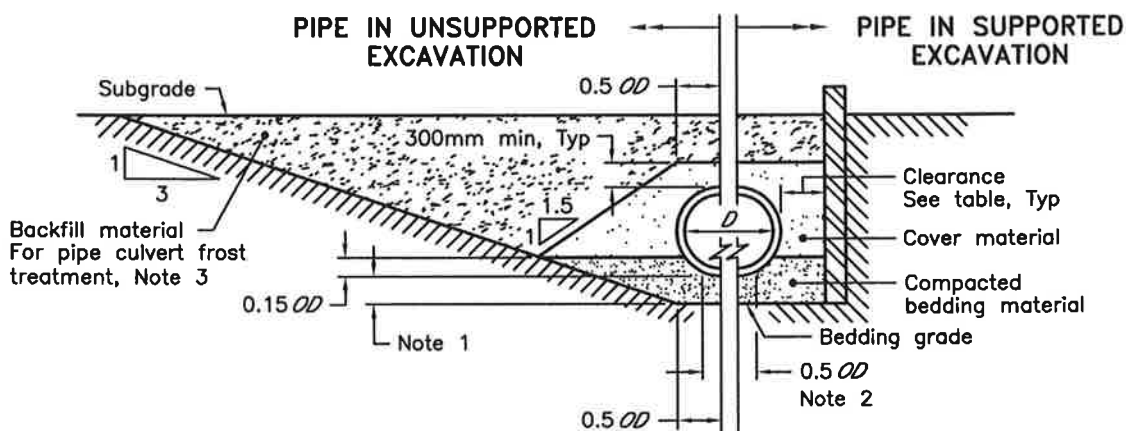
RIGID PIPE BEDDING,  
 COVER, AND BACKFILL  
 TYPE 3 SOIL – EARTH EXCAVATION

OPSD – 802.031





**CLASS B BEDDING**



**CLASS C BEDDING**

**LEGEND:**

$D$  - Inside diameter  
 $OD$  - Outside diameter

**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.

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

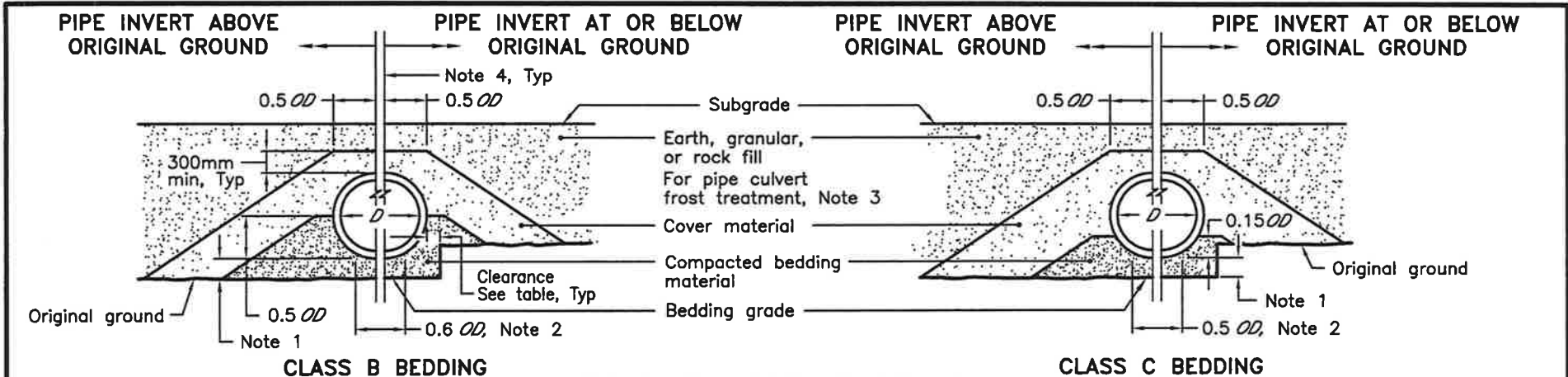
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 | Rev | 1

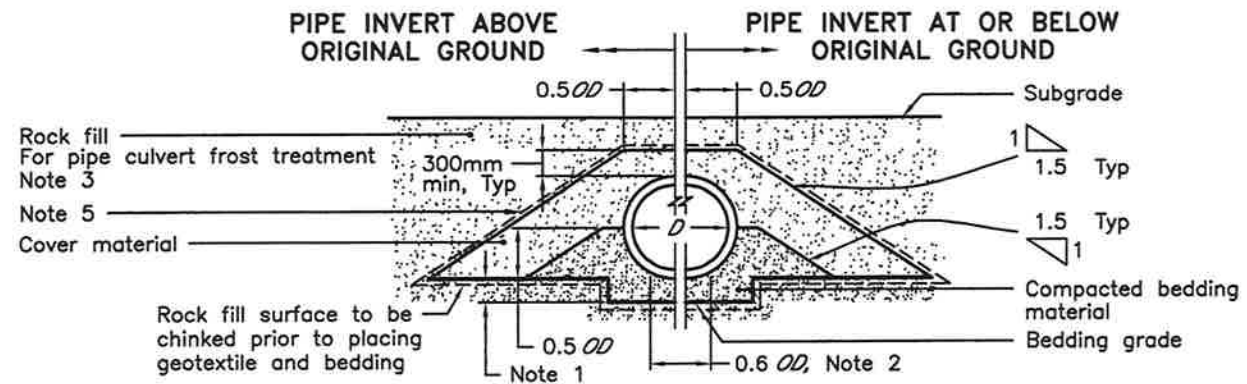
**RIGID PIPE BEDDING,  
 COVER, AND BACKFILL  
 TYPE 4 SOIL - EARTH EXCAVATION**

**OPSD - 802.032**





**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  $0.15D$ , except on a rock foundation where the minimum bedding depth shall be  $0.25D$ . 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:**

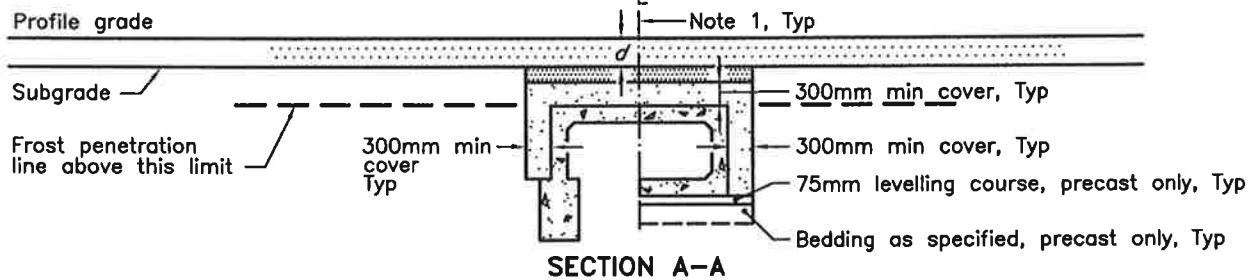
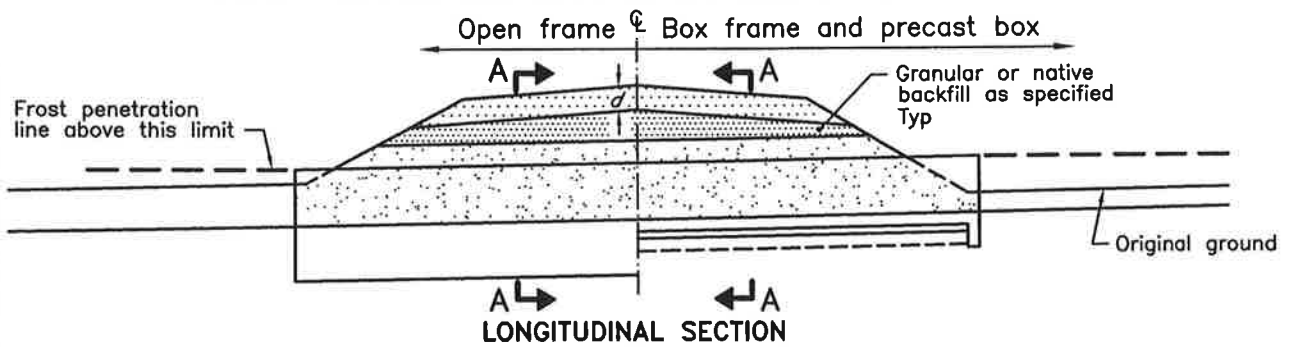
$D$  - Inside diameter  
 $OD$  - Outside diameter

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

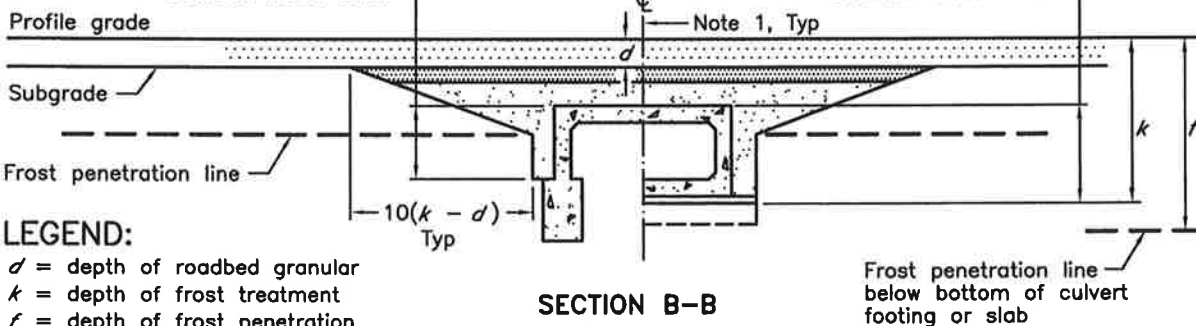
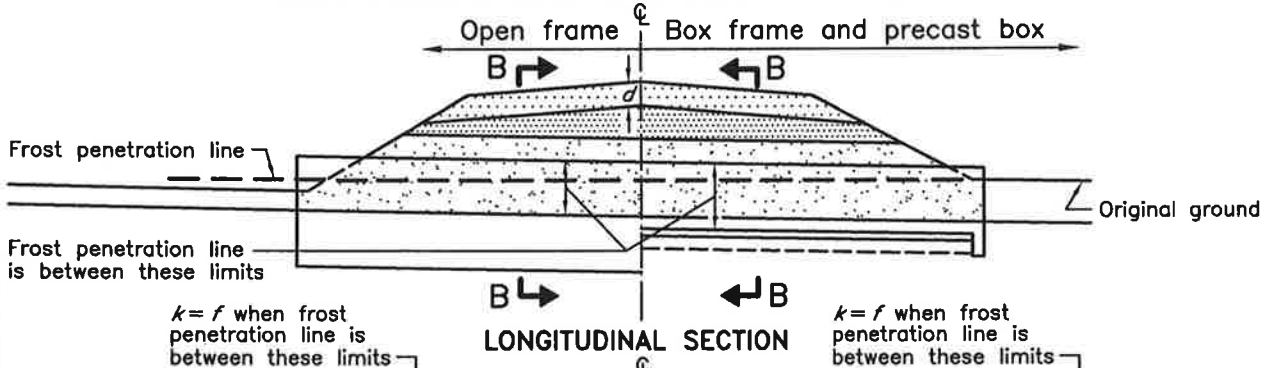
<b>ONTARIO PROVINCIAL STANDARD DRAWING</b>		Nov 2005    Rev 1	
<b>RIGID PIPE BEDDING AND COVER</b>		-----	
<b>IN EMBANKMENT</b>		-----	
<b>ORIGINAL GROUND: EARTH OR ROCK</b>		<b>OPSD - 802.034</b>	



## FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



## FROST PENETRATION LINE BELOW TOP OF CULVERT



### LEGEND:

$d$  = depth of roadbed granular  
 $k$  = depth of frost treatment  
 $f$  = depth of frost penetration

### NOTES:

- 1 Condition of frost treatment symmetrical about centreline of culvert.
- A Bedding, levelling, and cover material to be granular as specified.
- B This standard applies to cast-in-place and precast concrete culverts with spans less than or equal to 3.0m.
- C The depth of roadbed granular to be 600mm minimum.
- D The maximum depth of frost treatment to be bottom of box frame or top of footing.
- E All dimensions are in millimetres unless otherwise shown.

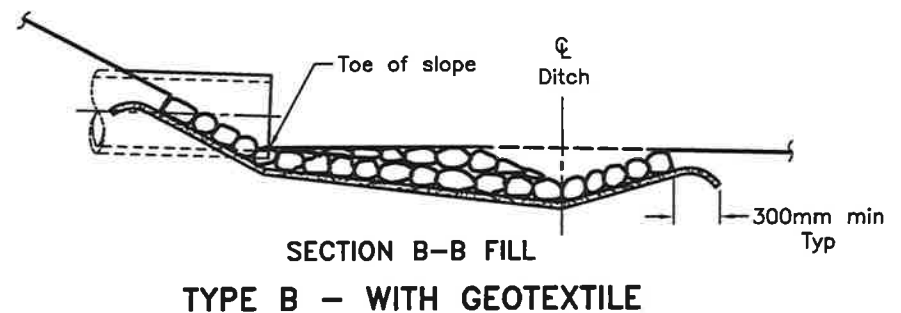
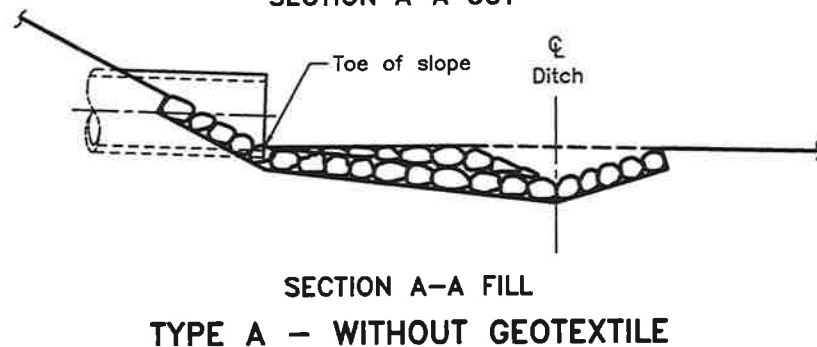
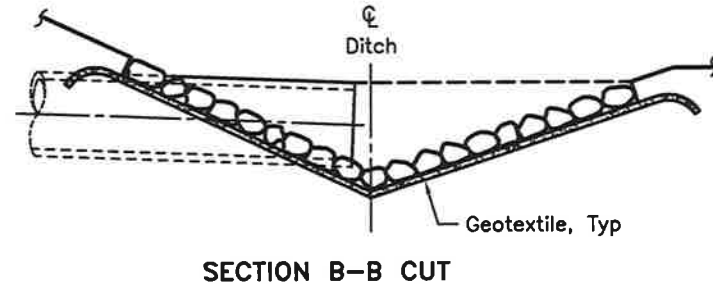
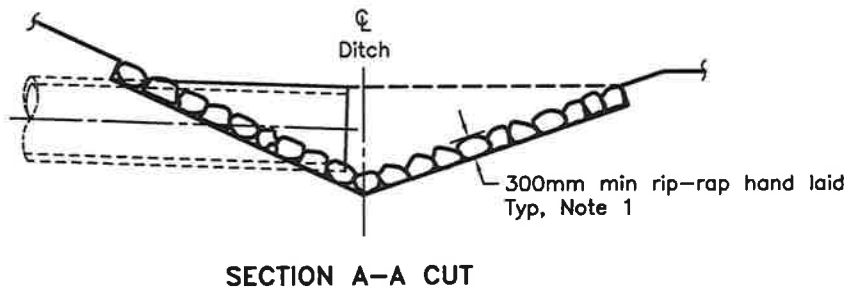
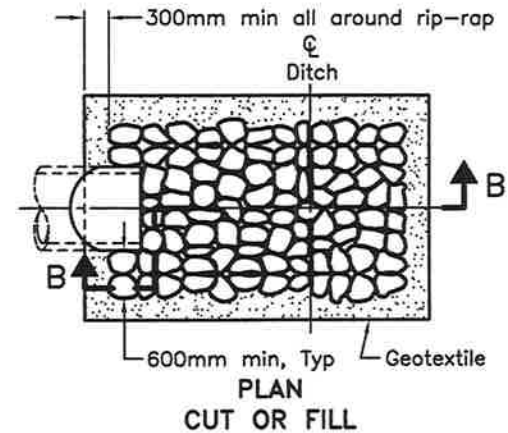
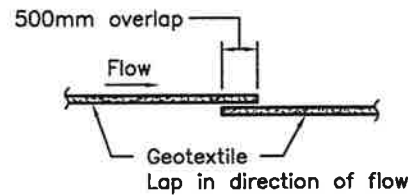
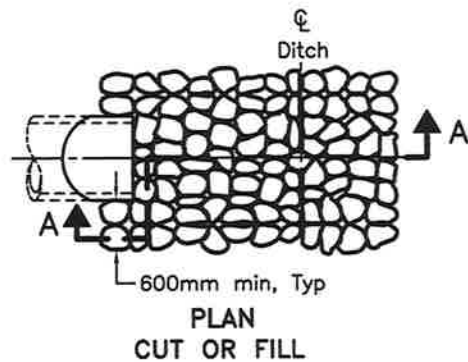
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2006 | Rev | 1

**BACKFILL AND COVER  
FOR CONCRETE CULVERTS**

**OPSD 803.010**





**NOTES:**

- 1 The thickness of the rip-rap layer shall be at least 1.5 times the rip-rap mean diameter.
- A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2007 Rev 1

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

**OPSD 810.010**



# Appendix H

## **Limitations of Report**

## **LIMITATIONS OF REPORT**

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

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

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

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

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