

**FOUNDATION INVESTIGATION AND  
DESIGN REPORTS - PROPOSED  
CULVERT REPLACEMENT/EXTENSION  
ON HIGHWAY 11, TOWNSHIP OF  
ARMSTRONG, ONTARIO  
G.W.P. 161-98-00, GEOCRES NO. 31M-82**

D.M. Wills Associates Limited

TRANETOB01237AA-AA  
June 04, 2010

June 04, 2010

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

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

Dear Mr. Lang,

**RE: Foundation Investigation and Design Reports, Proposed Culvert Replacement/Extension on Highway 11, Township of Armstrong, Ontario, G.W.P. 161-98-00, Geocres No. 31M-82**

Coffey Geotechnics Inc (Coffey) is pleased to present the Foundation Investigation and Design Reports for the proposed culvert replacement/extension on Highway 11, within the Township of Armstrong, Ontario.

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

For and on behalf of Coffey Geotechnics Inc.



*for* **Ramon Miranda, P. Eng.**  
Manager, Transportation

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PROPOSED CULVERT  
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ARMSTRONG, ONTARIO  
G.W.P. 161-98-00, GEOCRES NO. 31M-82**

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# CONTENTS

<b>1</b>	<b>INTRODUCTION</b>	<b>1</b>
<b>2</b>	<b>SITE DESCRIPTION AND PHYSIOGRAPHY</b>	<b>1</b>
<b>3</b>	<b>FIELD AND LABORATORY WORK</b>	<b>2</b>
<b>4</b>	<b>SUBSURFACE CONDITIONS</b>	<b>3</b>
<b>4.1</b>	<b>Culvert C16 at Station 13+694</b>	<b>4</b>
4.1.1	Embankment Fill	4
4.1.2	Silty Clay	5
4.1.3	Silty Sand Till	6
4.1.4	Bedrock	7
4.1.5	Groundwater Conditions	7
<b>4.2</b>	<b>Culvert C19 at Station 15+328</b>	<b>8</b>
4.2.1	Topsoil	8
4.2.2	Embankment Fill	8
4.2.3	Silty Sand Till	9
4.2.4	Bedrock	10
4.2.5	Groundwater Conditions	11
<b>4.3</b>	<b>Culvert C20 at Station 16+268</b>	<b>12</b>
4.3.1	Topsoil	12
4.3.2	Embankment Fill	12
4.3.3	Silty Clay	13
4.3.4	Silty Sand Till	14
4.3.5	Bedrock	15
4.3.6	Groundwater Conditions	15

## Tables

Table 1: Culvert Locations

Table 2: Borehole and DCPT Details

Table 3: Summary of Bedrock Levels at Culvert C19

Table 4: Summary of Groundwater Levels at C19



# CONTENTS

Table 5: Summary of Bedrock Levels at Culvert C20

Table 6: Summary of Groundwater Levels at C20

## Drawings

Drawing 1: Site Location Plan

Drawing 2A: Borehole Locations Plan at Culvert C16

Drawing 2B: Section along the culvert alignment – C16

Drawing 3A: Borehole Locations Plan at Culvert C19

Drawing 3B: Section along the culvert alignment – C19

Drawing 3C: Section along the highway alignment – C19

Drawing 4A: Borehole Locations Plan at Culvert C20

Drawing 4B: Section along the culvert alignment – C20

Drawing 4C: Section along the highway alignment – C20

## Appendices

Appendix A1: Record of Borehole Sheets – C16

Appendix A2: Record of Borehole Sheets – C19

Appendix A3: Record of Borehole Sheets – C20

Appendix B1: Laboratory Test Results – C16

Figure B1-1: Plasticity Chart – Earth Fill: Clayey Silt

Figure B1-2: Grain Size Distribution – Silty Clay

Figure B1-3: Plasticity Chart – Silty Clay

Figure B1-4: Grain Size Distribution – Silty Sand Till

Appendix B2: Laboratory Test Results – C19

Figure B2-1: Grain Size Distribution – Granular Pavement Fill

Figure B2-2: Grain Size Distribution – Embankment Fill

Figure B2-3: Grain Size Distribution – Silty Sand Till

Figure B2-4: Plasticity Chart – Till Exhibiting Some Cohesion

Figure B2-5: Grain Size Distribution – Clayey Silt Interbed in the Till

# CONTENTS

Figure B2-6: Plasticity Chart – Clayey Silt Interbed in the Till

## Appendix B3: Laboratory Test Results – C20

Figure B3-1: Grain Size Distribution – Granular Pavement Fill

Figure B3-2: Grain Size Distribution – Silty Clay Seam in Silty Sand Embankment Fill

Figure B3-3: Grain Size Distribution – Silty Clay

Figure B3-4: Plasticity Chart – Silty Clay

Figure B3-5: Grain Size Distribution – Transition Zone from Silty Clay to underlying Silty Sand Till

Figure B3-6: Plasticity Chart – Transition Zone from Silty Clay to underlying Silty Sand Till

Figure B3-7: Grain Size Distribution – Silty Sand Till

Figure B3-8: Plasticity Chart – Slightly Cohesive Zones in the Till Deposit

## Appendix C: Rock Core Photographs

## Appendix D: Site Photographs

## Appendix E: Explanation of Terms Used in Report

**FOUNDATION INVESTIGATION REPORT**  
**CULVERT REPLACEMENT/EXTENSION ON HIGHWAY 11**  
**TOWNSHIP OF ARMSTRONG, ONTARIO**  
**G.W.P. 161-98-00, GEOCREC NO. 31M-82**

## **1 INTRODUCTION**

As part of the rehabilitation and resurfacing of Highway 11, from the south junction of Highway 569 northerly for 27.6 km, including construction of new North Bound (NB) and South Bound (SB) passing lanes, it is proposed to extend existing Culvert C16 and replace existing Culverts C19 and C20. The culverts are located in the Armstrong Township, south of Englehart, Ontario.

Although other "pavement culverts" will be replaced as part of the project, the culverts noted in this report were identified as culverts requiring foundation investigation due to their location and/or depth of placement.

Coffey Geotechnics Inc. (Coffey) was retained by D.M. Wills Associates Limited (Wills) to carry out a foundation investigation at the proposed culvert sites. Table 1 presents the details of the proposed culvert replacement/extension.

**Table 1: Culvert Locations**

<b>Culvert Number</b>	<b>Station</b>	<b>Proposed Work</b>
C16	13+694	Culvert extension on the new NB passing lane
C19	15+328	Culvert replacement and new SB passing lane
C20	16+268	Culvert replacement and new SB passing lane

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

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

## **2 SITE DESCRIPTION AND PHYSIOGRAPHY**

The sites are located approximately 2.3 km, 3.3 km and 4.9 km south of the junction of Highway 571 with Highway 11 in the Township of Armstrong as shown in Drawing No. 1. Photographs of the sites are presented in Appendix D.

The physiography of the area is strongly controlled by the structure of the underlying Precambrian Canadian Shield, comprising the Timiskaming fault system and the fault of the Larder Lake block. The boundary between the two fault systems is followed, approximately, by the course of the Blanche River. Thick quaternary sediments occupy the lowlands and several of the intermediate terraces, while the uplands and tablelands tend to be more sporadically veneered with till, outwash, moraine and lacustrine

sediments. Many of the sediments in the area are lacustro-deltaic in character and comprise layered silts and sands, while finer lacustrine varved clays appear more typical of the tableland. Several major delta structures have been identified. Many of the rock hills have been washed clean by wave action during high water stages of late-glacial Lake Barlow-Ojibway and the plane of highest water level has been established.

Based on "Map 2543 Bedrock Geology of Ontario – East-Central Sheet" published by Ministry of Northern Development and Mines, and "Map P.2700 Geological Series – Preliminary Map Paleozoic Geology – Lake Timiskaming Area" published by Ontario Geological Survey, the bedrock on site is Middle Lower Silurian Thornloe Formation described as medium to thick bedded, fine to medium crystalline dolostone and cherty dolostone with some thin bedded limestone.

### 3 FIELD AND LABORATORY WORK

The fieldwork for this project was performed during the period from June 04, 2009 to August 12, 2009, and consisted of drilling and sampling a total of twenty two boreholes. At Culvert C16 Boreholes C8, C9, F8, F9, F10 and F11 were advanced to depths ranging from 5.0 to 7.8 m below the existing grades. At Culvert C19 Boreholes C5, C6, C7, D2, and D3 were advanced to depths ranging from 9.9 to 13.1 m below existing grades. At Culvert C20 Boreholes C1, C3, C4, H10, and H11 were advanced to depths ranging from 2.5 to 7.1 m below existing grades. Dynamic cone penetration tests (DCPT) were carried out adjacent or in proximity to the boreholes at nine locations. The location of the boreholes and DCPT are shown on the Borehole Location Plan, Drawings 2A, 3A, and 4A. Table 2 presents the borehole and DCPT details.

**Table 2: Borehole and DCPT Details**

<b>Culvert Number</b>	<b>Borehole / DCPT Number</b>	<b>Station</b>	<b>Offset</b>	<b>Elevation (m)</b>	<b>Drilled/Tested Depth (m)</b>
C16	C8	13+695	7.8 m Rt C/L	258.7	6.4/6.4 <sup>a</sup>
C16	A4	13+675	13.9 m Rt C/L	258.1	5.3 <sup>b</sup>
C16	C9	13+700	24.6 m Rt C/L	256.6	7.8/4.9 <sup>a</sup>
C19	C5	15+326	15.0 m Rt C/L	260.9	13.1
C19	C5A	15+329.8	15.2 m Rt C/L	260.9	9.3 <sup>b</sup>
C19	C6	15+331	19.5 m Lt C/L	258.4	12.3
C19	C6A	15+325.5	18.8 m Lt C/L	258.9	11.2 <sup>b</sup>
C19	C7	15+327	6.5 m Rt C/L	262.3	12.2
C19	C7A	15+330	6.7 m Rt C/L	262.3	11.9 <sup>b</sup>
C19	D2	15+275	9.0 m Lt C/L	261.7	11.0
C19	D3	15+375	15.3 m Lt C/L	259.4	9.9
C20	C1	16+269	16.6 m Lt C/L	252.7	6.3/3.3 <sup>a</sup>
C20	C3	16+265	17.8 m Rt C/L	253.1	7.1
C20	C3A	16+269	9.3 m Rt C/L	254.9	5.8 <sup>b</sup>
C20	C4	16+272	8.0 m Lt C/L	254.5	5.1
C20	C4A	16+265	8.8 m Lt C/L	254.9	5.2 <sup>b</sup>
C20	H10	16+230	6.7 m Lt C/L	255.0	6.3
C20	H11	16+305	18.2 m Lt C/L	252.6	2.5

Notes: a. Borehole and DCPT location, b. DCPT location

The boreholes were advanced using a track-mounted drilling rig owned and operated by Landcore Drilling of Chelmsford, Ontario, with full-time monitoring conducted by technical personnel (Mr. Gem Jiang, EIT) from Coffey. The boreholes were advanced using continuous flight, hollow-stem augers. To advance the boreholes through cobbles and boulders, rotary core drilling was carried out utilizing N-size casing and NQ core barrel. Bedrock was cored in Boreholes C1, C3, C5, C6 and C9.

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

In addition to the SPT, where the consistency permitted, in-situ shear vane tests were carried out within the cohesive soils to assess the shear strength of the soil. The field vane shear tests were performed using MTO 'N' vane.

Dynamic cone penetration tests (DCPT) were performed at the test hole location, adjacent or in proximity to the boreholes, at C8, A4 and C9 for Culvert C16, C5A, C6A, and C7A for Culvert C19 and C1, C3A, and C4A for Culvert C20. The DCPT consists of driving an uncased 50 mm diameter cone, attached to A-size drill rods, with a driving energy of 475 kJ (63.5 kg hammer free falling for a distance of 0.76 m) per blow, continuously adjacent to the borehole. The number of blows for each 0.3 m of penetration is recorded, providing an indication of the relative changes in the soil density with respect to depth.

The borehole and DCPT locations were established in the field by Coffey engineering staff, in relation to the existing features. The ground elevations at the borehole locations were measured by the client's surveyors and were provided to Coffey.

Groundwater conditions in the boreholes were observed during and on completion of drilling in the open boreholes. A standpipe piezometer was installed in each of Boreholes C1, C6, and C9 upon their completion. The remaining boreholes were grouted using a cement/bentonite mixture as per MTO procedures. Note the piezometers installed have not been decommissioned as they may be useful in monitoring water level prior or during the construction. As part of the construction, the piezometers need to be decommissioned in accordance with Ontario Regulation 903 (amended to Ontario Regulation 372/07).

A laboratory testing program, consisting of natural water content tests, Atterberg Limits tests and grain size analyses including hydrometer testing, was performed on selected samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets in Appendices A1, A2 and A3 and also in Appendices B1, B2, and B3 for culverts C16, C19 and C20, respectively.

## **4 SUBSURFACE CONDITIONS**

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendices A1, A2 and A3 for Culverts C16, C19 and C20, respectively. Appendix C presents

rock core photographs and Appendix E presents the Explanation of Terms Used in Report. Soil strata along the culvert alignment and the highway alignment are shown in Drawings 2B, 3B and 4B, and 2C, 3C and 4C, respectively.

The subsurface conditions encountered at each culvert location are described in the following sections. The following descriptions of the individual strata are to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site. It should be noted that the subsurface conditions may vary in between and beyond the borehole locations.

#### **4.1 Culvert C16 at Station 13+694**

Boreholes C8 and C9 were drilled along Culvert C16 alignment to depths of 6.4 m to 7.8 m below the existing ground surface. The borehole locations are shown in Drawing 2.

Borehole C8 was drilled from the crest of the existing Highway 11 embankment at Elevation 258.7 m and at about 7.8 m right (north east) of the highway centreline. Borehole C9 was drilled to the north east of the existing culvert, along the toe of the existing slope, at Elevation 256.6 m and at about 24.6 m right (north east) of the highway centreline.

A DCPT was carried out adjacent to each of Boreholes C8 and C9. As well, a third DCPT (No. A4) was advanced to the southeast of Borehole C8 and at about 13.9 m right (north east) of the highway centreline from Elevation 258.1 m. The test results are plotted on the Record of Boreholes C8, A4 and C9. DCPT C8, A4 and C9 were terminated at depths of approximately 6.4 m, 5.3 m and 4.9 m below existing grades, respectively. Borehole C9 was advanced beyond auger refusal (Elevation 251.7 m) in an adjacent borehole by bedrock coring techniques.

In general, the encountered subsurface conditions at the Culvert C16 location consisted of approximately 0.9 to 2.1 m of embankment fill, over natural overburden soils which are in turn underlain by bedrock. The embankment fill at Borehole C8 included a 0.5 m thick granular fill layer that is part of the road pavement section. In Borehole C8, underlying the embankment fill at Elevation 256.6 m the natural soils consist of a 1.3 m thick silty clay layer, underlain by silty sand till to the surface of the inferred bedrock at Elevation 252.3 m or at 6.4 m below the ground surface. In Borehole C9, the silty clay layer was not encountered. Instead, the silty sand till deposit was contacted immediately below the embankment fill at Elevation 255.7 m and extended to the surface of the proven bedrock at Elevation 251.7 m, or at a depth of 4.9 m below the ground surface.

Drawing 2B presents the inferred stratigraphic section at Culvert C16 based on the borehole data.

The following paragraphs are presented only to amplify and complement these data.

##### **4.1.1 Embankment Fill**

###### **4.1.1.1 Granular Fill**

Borehole C8 was drilled on top of the highway embankment shoulder and encountered a 0.5 m pavement shoulder granular fill from surface. The granular fill consists of sand with gravel with trace silt and clay.

An SPT value of 5 blows/0.3 m was measured within the granular fill, indicating a loose relative density. This value indicates that the granular fill had not received a systematic compaction during the fill placement.

A natural water content measured from a collected sample of the granular fill was approximately 10%.

#### 4.1.1.2 Embankment Fill

Underlying the pavement granular fill, at Borehole C8 an embankment fill was encountered. The fill was found to extend to a depth of 2.1 below existing grade, or to Elevation 256.6 m. The fill material consisted of a mixture of silt with sand and silty clay, trace gravel, trace to some organic matter. Organic rich matter was also found intermixed with the fill material. This fill is classified as granular (i.e. non cohesive) to cohesive material, depending on the clay content.

Standard Penetration Tests performed in the embankment fill yielded N-values of 6 and 10 blow/0.3 m which indicate a loose or firm to stiff condition,

#### 4.1.1.3 Earth Fill

In Borehole C9, which was advanced beyond the toe of the embankment, a 0.9m thick earth fill was encountered at the ground level. This fill which extends to Elevation 255.7 m, was found to consist of silty clay and sand with traces of gravel size particles. Some organic matter was also found to be intermixed with the fill. Similar to the embankment fill, the fill found in Borehole C9 is classified as a granular to cohesive soil, depending on the clay content.

An Atterberg Limits test was carried out on a selected sample of the cohesive portion of the fill material and yielded the following.

Liquid Limit:	30.2%
Plastic Limit:	17.5%
Plasticity Index:	12.7%

As shown on the Plasticity Chart in Figure B1-1 in Appendix B1, these results indicate a clayey soil of low plasticity.

An SPT 'N'-value of 8 blows/0.3 m was recorded within the fill indicating a loose or firm to stiff condition.

### 4.1.2 Silty Clay

A grey brown silty clay deposit was encountered in Borehole C8 at a depth of 2.1 m below existing grade to Elevation 256.6m. This deposit was found to be 1.3 m thick and extended to a depth of 3.4 m (Elevation 255.4 m).

A grain size analyses was carried out on a representative sample of this cohesive deposit. The results are presented on the Record of Borehole sheet in Appendix A1 and the grain size curve is presented in Figure B1-2, Appendix B1. The curve shows the following grain-size distribution:

Gravel:	0%
Sand:	2%
Silt:	32%
Clay:	66%

Based on these results, the silty clay is considered to be a low permeability material.

An Atterberg Limits test was performed on a sample from the deposit. As shown in Figure B1-3 in Appendix B1, the test indicated the following index values:

Liquid Limit:	43.4%
Plastic Limit:	25.7%
Plasticity Index:	17.7%
Natural Water Content:	42%

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

DCPT values of 12 to 17 and an 'N'-value of 8 blows/0.3 m were recorded within the silty clay deposit. A field vane test was also carried out within this cohesive soil and the undrained, in-situ shear strength was measured as 80 kPa. Based on these test results and a tactile evaluation, the silty clay deposit is considered to have a consistency of firm to stiff.

Based on the laboratory test results together with a visual and tactile examination of the soil samples, this deposit is considered to be slightly over-consolidated.

#### **4.1.3 Silty Sand Till**

Underlying the silty clay in Borehole C8 and the fill in Borehole C9, a silty sand till deposit was encountered in both boreholes at depths of 0.9 m and 3.4 m below the existing ground surface or at Elevations of 255.4 m and 255.7 m. Borehole C8 was terminated within this deposit at a depth of 6.4 m below the ground surface or at Elevation 252.3 m, upon encountering auger refusal, possibly on the surface of the bedrock or near it, while Borehole C9 the deposit was found to extend to the surface of the bedrock at a depth of 4.9 m or at Elevation 251.7 m.

The deposit consists of moist to wet, light brown to grey, heterogeneous mixture of silty sand with traces of gravel size particles. In the boreholes, auger grinding was observed indicating the presence of frequent cobbles and boulders within the till deposit. Rock coring methods were used to advance Borehole C9 through the cobbles and boulders at the base of the till deposit.

The following are results of grain size distribution tests conducted on samples from the till deposit, as shown on Figure B1-4 in Appendix B1.

Gravel:	7-10%
Sand:	53-55%



Silt: 25-26%

Clay: 12%

The deposit is classified as a basically granular (non-cohesive) soil. Due to the clay content some cementation can be expected.

Natural water contents measured on samples recovered from this stratum are between 8% and 14%.

SPT 'N'-values ranging from 3 to in excess of 100 blows/0.3m were recorded within the till indicating a very loose to very dense condition. Recorded DCPT values ranged from approximately 3 to 100 also indicating a very loose to very dense conditions. These field test results also indicate that the upper 2.0 m to 2.5 m of the deposit is typically in a very loose to loose condition, becoming compact to very dense below.

As mentioned before, cobbles and boulders were inferred in the deposit while drilling, and from the high "N" and DCPT values. The presence of cobbles and boulders should always be anticipated in glacial till deposits, due to their mode of deposition.

#### **4.1.4 Bedrock**

In Borehole C9, bedrock was contacted at a depth of 4.9 m or at Elevation 251.7 m.

Borehole C8 encountered auger refusal on possible bedrock or close to it at a depth of about 6.4 m below the road surface or at Elevation 252.3 m. DCPT A4 encountered refusal on possible bedrock (or close to it) at a depth of about 5.3 m below the existing ground surface or at Elevation 252.8m.

The bedrock was cored in Borehole C9 for a vertical distance of 2.9 m. The bedrock in this borehole is described as dolomitic limestone, brown and grey, fresh to slightly weathered. The total core recovery (TCR) in the bedrock was measured 88% to 89%, and Rock Quality Designation (RQD) values of 45% to 64% were recorded in the borehole. The RQD results indicate a poor to fair rock quality.

#### **4.1.5 Groundwater Conditions**

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. In addition, a piezometer was installed in Borehole C9 to enable us to monitor the groundwater level over a prolonged period of time, without interference from surface water. The tip of piezometer was installed in the bedrock to investigate a possible excess hydrostatic pressure emanating from the bedrock.

Borehole C8 was found to be dry on completion but is believed to be unstabilized. In Borehole C9 in the piezometer installed, the groundwater level was measured 41 days after completion at 3.1 m below the ground surface of at Elevation 253.5 m. Based on these observations and the change of the soil from brown to grey, the groundwater table at the time of our investigation was at generally between 255 m and 253 m. At the time of our investigation, the surface water at the invert of the existing culvert was at about Elevation 256 m.

It should be pointed out that the groundwater would be subject to seasonal fluctuations and fluctuations in response to major weather events. The groundwater would also be controlled by a water level in the watercourse. A perched water table may also occur due to the accumulation of the surface water in the surficial fills overlying the practically impervious silty clay.

## **4.2 Culvert C19 at Station 15+328**

Boreholes C5, C6 and C7 were drilled along the Culvert C19 alignment to depths of 12.2 m to 13.1 m below the existing ground surface. In addition, two other boreholes (D2 and D3) were put down in the vicinity of Culvert C19 along the south western side of the existing highway on the proposed detour embankment/new SB passing lane to depths of 11.0 m and 9.9 m below the existing ground. The ground elevations at the borehole locations ranged from 259.4 m to 262.5 m.

DCPT's were carried out adjacent or at a short distance from Boreholes C5, C6 and C7 and the tests were named as C5A, C6A, C7A, respectively.

The borehole and DCPT locations are shown in Drawing 3A.

Borehole C7 was drilled from top of the existing Highway 11 at Elevation 262.3 m, at about 6.5 m right (north east) of the centerline of the highway. Boreholes C5 and C6 were drilled to the north east and south west side of the highway at Elevations 260.9 m and 258.4 m, respectively, and at about 15.0 m right (north east) and 19.5 m left (south west) of the highway centreline, respectively. Rock was cored in Boreholes C5 and C6. Boreholes D2 and D3 were drilled at Stations 15+275 and 15+375, at Elevations 261.7 m and 259.4 m, and at a distance of 9.0 m and 15.3 m left (south west) of the highway centreline, respectively.

In general, the subsurface conditions at Culvert C19 location consisted of about 3.5 m thick embankment fill, as encountered in Borehole C7 that was drilled on top of the existing highway, over an 8.6 m to 10.2 m thick silty sand till layer which is in turn underlain by bedrock. The embankment fill included a 0.8 m thick granular fill layer that is part of the road pavement over embankment fill. Topsoil was encountered in Boreholes C5, C6 and D3 which were drilled on the sides of the highway and at the toe of the road embankment. Below the topsoil and embankment fill, the silty sand till was encountered at Elevations 258.2 m to 260.9 m. Based on the boreholes and DCPT results, the top of bedrock was encountered/inferred at Elevations 247.7 m to 251.63 m.

Drawings 3B and 3C present the inferred stratigraphic sections along Culvert C19 alignment and highway alignment based on the borehole data.

### **4.2.1 Topsoil**

Boreholes C5, C6 and D3 encountered topsoil with thicknesses ranging from 0.2 m to 0.3 m.

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

### **4.2.2 Embankment Fill**

#### **4.2.2.1 Granular Fill**

Boreholes C7 and D2 were drilled on top of the highway embankment shoulder and encountered a 0.8 m pavement shoulder granular fill. The granular fill consisted of gravelly sand with traces of silt and clay. The grain size distribution of two samples from the granular fill from Boreholes C7 and D2 is given in Figure B2-1 in Appendix B2. The following are the grain size distribution of the granular fill:

Gravel:	20-30%
Sand:	61-66%
Silt and Clay:	9-14%

An SPT 'N' value of 5 blows/0.3 m was measured within the granular fill, indicating a relative density of very loose to loose. This indicates that the granular fill has not received a systematic compaction during the fill placement.

The natural water content obtained from samples of the fill was 5% to 9% indicating a damp condition.

#### 4.2.2.2 Embankment Fill

Underlying the pavement granular fill or topsoil, Boreholes C7 and C5 encountered a 2.7 m and 1.2 m thick layer of earth fill, extending to depths of 3.5 m and 1.5 m or to Elevations 258.8 and 259.4 m, respectively. The embankment fill consisted of silty sand with traces to some gravel and trace clay. It also contained trace to some organics and sand layers in Borehole C5 and amorphous organics in Borehole C7.

The grain size distribution of the one sample from the embankment fill in Boreholes C5 is given in Figure B2-2 in Appendix B2. The following is a summary of the grain size distribution of the material.

Gravel:	4%
Sand:	57%
Silt:	28%
Clay:	11%

These fills are described as granular (i.e. non-cohesive) materials.

SPT 'N'-values ranging from 2 to 8 were recorded within the silty sand fill, as well as DCPT values of 1 to 14 blows/0.3 m. The results indicate that the fill layer is in a very loose to loose condition and is unlikely to have received as systematic compaction when it was first placed.

#### 4.2.3 Silty Sand Till

Below the fill and topsoil, a silty sand glacial till deposit was encountered in the boreholes at depths ranging from 0.2 m to 3.5 m below the existing ground surface or at Elevations 258.2 m to 260.9 m. At the borehole locations, the deposit was found to be about 8.6 m to 10.2 m thick. The till deposit consists of a heterogeneous mixture of silty sand with traces to some gravel and clay size particles. In the boreholes, auger grinding was observed indicating the presence of frequent cobbles and boulders within the till deposit. In some cases, rock coring methods were used to advance the boreholes through cobbles and boulders.

The following are the results of grain size testing on nine samples from the till deposit in Boreholes C5, C6 C7, D2 and D3. Figure B2-3 in Appendix B2 presents the grain size distribution of the samples from the till deposit, in an envelope format.

Gravel:	3-11%
Sand:	40-57%
Silt:	25-35%
Clay:	12-21%

The till is classified as a basically granular (i.e. non-cohesive) soil. But it also exhibits some apparent cohesion, due to its clay content, especially where the clay content is relatively high. An Atterberg Limits test was performed on a such sample which exhibited some cohesion. This test gave the following index values as given in Figure B2-4 in Appendix B2.

Liquid Limit:	17.4%
Plastic Limit:	10.1%
Plasticity Index:	7.3%

The till was also found to contain occasional thin clayey silt interbeds. The grain size distribution of a sample from a clayey silt interbed from Borehole C6 is given in Figure B2-5 in Appendix B2. The results of an Atterberg Limits test performed on the same sample are presented in Figure B2-6 in Appendix B2. As mentioned before, cobbles and boulders was inferred in the deposit while drilling. In any event the presence of cobbles and boulders should always be anticipated in the glacial till deposits, due to their mode of deposition.

SPT 'N'-values ranging from 1 to in excess of 100 blows/0.3m were recorded within the till indicating a very loose to very dense condition. The high 'N'-values were recorded due to the presence of cobbles and boulders. From the recorded test results, the relative density of the deposit is described as loose to compact with occasional very loose and/or dense to very dense zones.

#### 4.2.4 Bedrock

Bedrock was contacted and proven by coring in Boreholes C5 and C6 at the following depths and elevations:

**Table 3: Summary of Bedrock Levels at Culvert C19**

Borehole Number	Ground Elevation (m)	Depth to Top of the Bedrock (m)	Elevation to Top of the Bedrock (m)
C5	260.9	10.1	250.8
C6	258.4	9.3	249.1

Borehole C7 encountered auger refusal and an N-value of 100 blows/0m on possible bedrock at a depth of 12.2 m below the road surface or at Elevation 250.1 m. Boreholes D2 and D3 encountered auger refusal at depths of 11.0 m and 9.9 m or Elevations 250.7 m and 249.5 m that are assessed to represent boulders or the surface of the bedrock. DCPT C6A and C7A encountered refusal on possible bedrock at depths of 11.2 m and 11.9 m below the ground surface or Elevations 247.7 m and 250.4m, respectively, while DCPT C5A encountered refusal on possible boulder at a depth of 9.3 m or Elevation 251.6 m.

The bedrock is described as dolomitic limestone, brown and grey, fresh to slightly weathered. The total core recovery (TCR) in the cores from the bedrock was measured 87% to 100%, and Rock Quality Designation (RQD) values of 57% to 97% were recorded. The RQD results indicate a fair to excellent rock quality.

From the table presented above, the surface of the bedrock appears to be sloping down mildly in the south west direction.

#### 4.2.5 Groundwater Conditions

Groundwater levels were observed in the open boreholes while drilling and upon completion of each borehole. The groundwater levels observed during the investigation are summarised below in Table 4 and are also presented on the Record of Borehole Sheets in Appendix A2.

**Table 4: Summary of Groundwater Levels at C19**

Borehole Number	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Measured Groundwater Depth/Elevation (m)	Date Measured	Piezometer Installed?
C5	260.9	-	None Observed* (dry on completion)	18 Jun 2009	No
C6	258.4	9.2/249.2	1.9/256.5	28 Aug 2009 (69 days after completion)	Yes
C7	262.3	-	11.4/250.9* (completion, cave @ 11.7 m)	19 Jun 2009	No
D2	261.7	-	7.9/253.8* (completion, cave @ 10.4 m)	5 Jun 2009	No
D3	259.4	-	None Observed* (dry on completion, cave @ 9.8 m)	19 Jun 2009	No

Note: \* Groundwater level measured not stabilized.

Groundwater levels measured on completion are not considered to have stabilized and therefore do not represent the groundwater table at the site.

Based on the results of the piezometer readings in Borehole C6, at the time of our field program, the groundwater table at the site was at about 1.5 m to 2.5 m below the existing original ground surface (o.g.) level or approximately at Elevations 256.5 m to 257.0 m.

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

### **4.3 Culvert C20 at Station 16+268**

Five boreholes were put down at the site of Culvert C20. Boreholes C1, C3 and C4 were drilled at the close vicinity of the proposed culvert. Borehole C1 was drilled at 16.6 m left (south west) of the highway centerline, near the toe of embankment from the original ground level (Elevation 252.7 m) while Borehole C3 was drilled at 17.8 m right (north east) of the highway centreline, near the ditch from the original ground level (Elevation 253.1 m). Borehole C4 was drilled from the top of the roadway, 8.0 m left (south west) of the highway centreline (Elevation 254.5 m). In addition, Boreholes H10 and H11 were drilled on the south west (left) side of the existing Highway 11 embankment at Stations 16+230 and 16+305, at Elevations 255.0 m and 252.6 m and at about 6.7 m and 18.2 m left (south west) of the highway centreline, respectively. The boreholes were drilled 2.5 m to 7.1 m below the existing ground surface.

DCPT's were carried out at the borehole location, adjacent or near Boreholes C1, C3 and C4 and the tests were numbered C1, C3A, C4A.

The borehole and DCPT locations are shown in Drawing 4A.

In general, at Culvert C20, Boreholes C4 and H10 drilled from the top of road embankment encountered embankment fill to a depth of 1.7 m and 2.3 m (Elevation 252.8 m and 252.7 m), underlain by 2.1 m and 3.0 m thick silty clay layer, respectively. In Boreholes C1, C3, H11, similar silty clay layer was encountered below the topsoil at Elevations 252.2 m to 252.5 m. The silty clay layer was found to extend to Elevations 249.7 m to 251.1 m (i.e. 1.1 m to 3.0 m thick) and is underlain by silty sand glacial till deposit. The glacial till deposit encountered in the boreholes is 1.0 m to 1.6 m thick. The surface of bedrock was encountered below the silty sand till deposit at Elevations 249.4 m (3.3 m depth) and 249.2 m (3.9 m depth) in Boreholes C1 and C3, respectively. Boreholes C4, H10 and H11 were terminated in the till deposit upon encountering auger refusal at depths of 5.1 m, 2.5 m and 6.3 m or Elevations 249.4 m, 250.1 m and 248.7 m, respectively, which may represent boulders or the surface of the bedrock.

Drawings 4B and 4C present the inferred stratigraphic sections along Culvert C19 alignment and highway alignment based on the borehole data.

#### **4.3.1 Topsoil**

Boreholes C1, C3 and H11 encountered a veneer of topsoil at surface which ranged in thickness from 0.3 to 0.6 m.

We would like to point out that in our experience, the thickness of organic rich soils frequently varies in between and beyond borehole locations.

#### **4.3.2 Embankment Fill**

##### **4.3.2.1 Granular Fill**

Boreholes C4 and H10 were advanced from the top of the highway embankment shoulder and encountered 0.6 m granular pavement fill. The granular fill consisted of sand with gravel, trace silt. The grain-size distribution of a sample from the granular fill in Borehole C4 is shown in Figure B3-1 in Appendix B3. The curve shows the following grain-size distribution:

Gravel:	25%
Sand:	67%
Silt and Clay:	8%

The natural water content from samples collected were 4% to 5% indicating a damp condition.

#### 4.3.2.2 Embankment Fill

In Boreholes C4 and H10, underlying the granular pavement fill at a depth of 0.6 m (Elevation 253.9 m and 254.4 m) embankment fill was encountered. The embankment fill was 1.1 m to 1.7 m thick (i.e. to Elevation 252.8 m and 252.7 m) and consisted of silty sand, some gravel, trace clay and occasional organic clay pockets. In Borehole H10, a silty clay layer/pocket was encountered within the fill and the results of a grain size distribution test on this silty clay layer/pocket is given in Figure B3-2 in Appendix B3.

The embankment fill encountered in Boreholes C4 and H10 is considered to be a granular (non-cohesive) material, except for the silty clay layer/pocket contacted within the deposit in Borehole H10, which is considered to be a cohesive soil type.

SPTs performed in the fill material yielded 'N'-values of 5 to 15 blows/0.3 m, indicating a relative density of loose to compact. DCPT values of 5 to 31 blows/0.3 m were measured within the silty sand fill layer. These results indicated that the silty sand fill material received some degree of compaction during placement however the degree of compaction appears to be variable.

#### 4.3.3 Silty Clay

A greyish brown to grey silty clay deposit was encountered in all of the boreholes, at depths of 0.3 m to 2.3 m below existing grades or at Elevations 252.2 m to 252.8 m. This deposit was found to be 1.1 m to 3.0 m thick and extended to depths 2.2 m to 5.3 m below existing grades (Elevation 249.7 m to 251.1 m).

Four grain size analyses were carried out on representative samples from the deposit collected from the boreholes. The results are presented on the Record of Borehole sheets in Appendix A2, and the grain size curves are presented in Figure B3-3 in Appendix B3. The curves show the following grain-size distribution:

Gravel:	0 %
Sand:	1 – 3%
Silt:	28 – 38%
Clay:	61 – 69%

Atterberg Limits tests were performed on four samples from the deposit recovered from the boreholes. As shown in Figure B3-4 in Appendix B3, these tests indicated the following index values:

Liquid Limit:	43 – 47%
Plastic Limit:	27 – 28%
Plasticity Index:	15 – 19%

Natural Water Content: 31- 49%

The above values are characteristic of a clayey soil of intermediate plasticity. The measured natural water contents are very close to the measured liquid limit values.

Based on the grain size and Atterberg Limits test results together with a visual and tactile examination of the soil samples this deposit is considered to have a low permeability in comparison with the embankment fill and also the underlying silty sand till. Based on the measured natural water content and Liquid Limit results, the silty clay is considered to be only slightly over-consolidated. The deposit is considered to be a cohesive material.

In some boreholes a transition zone was noted from the silty clay deposit to the underlying silty sand till. The results of the grain size distribution test and Atterberg Limits test from this transition zone are given in Figures B3-5 and B3-6 in Appendix B3.

Measured 'N'-values in the silty clay deposit range from 4 to 11 blows/0.3 m. Field vane tests were carried out within this cohesive soil and the measured shear strengths ranged from 40 to in excess of 100 kPa, with a sensitivity of approximately 3 to 6. Based on these test results, the silty clay deposit is considered to be moderately sensitive and has a consistency of firm to very stiff.

#### **4.3.4 Silty Sand Till**

The silty clay deposit is underlain by a glacial till deposit which consists of a heterogeneous mixture of silty sand with some clay, traces to some gravel. This basically granular deposit was encountered at depths of 1.5 m to 5.3 m below the existing ground levels or at Elevations 249.7 m to 251.1 m. This till deposit extended to the surface of the bedrock at Elevation 249.4 m in Borehole C1 and Elevation 249.2 m in Borehole C3. Boreholes C4, H10 and H11 were terminated within the deposit at Elevations 248.7 m to 250.1 m upon encountering refusal to augering possibly on the surface of the bedrock or close to it.

The results of grain size distribution tests conducted on five selected samples from the till deposit are shown in an envelope form in Figure B3-7 in Appendix B3 and are summarized below:

Gravel:	5 - 21%
Sand:	41 - 56%
Silt:	24 - 33%
Clay:	5 - 17%

The till is basically granular (non-cohesive) material but, especially where the clay content is high, it exhibits some apparent cohesion. In addition to grain size analyses, Atterberg Limits tests were performed on two selected samples from the deposit, where some cohesion was evident. As shown in Figure B3-8 in Appendix B3, these tests indicated the following index values:

Liquid Limit:	15 - 16%
Plastic Limit:	10 - 11%
Plasticity Index:	5%



Natural Water Content: 6 - 19%

These results indicate some slight cohesion may be present in zones where the clay content is relatively high.

SPTs performed in the deposit yielded 'N'-values which ranged widely from 4 to in excess of 100 blows/0.3 m. These results indicate a very loose to very dense relative density.

#### 4.3.5 Bedrock

Bedrock was proven by coring at Boreholes C1 and C3 and the results are as follows:

**Table 5: Summary of Bedrock Levels at Culvert C20**

Borehole Number	Ground Elevation (m)	Depth to Top of the Bedrock (m)	Elevation to Top of the Bedrock (m)
C1	252.7	3.3	249.4
C3	253.1	3.9	249.2

Boreholes C4, H10 and H11 encountered auger refusal and high blow counts (100 blows per 0.1 m to 0.24 m penetration) at depths of 2.5 m to 6.3 m below the existing grades, or Elevations 248.7 m to 250.1 m, possibly on the surface of the bedrock or near to it.

From the table presented above, the surface of the bedrock appears to be fairly flat with elevation difference of 0.2 m over a horizontal distance of 34.4 m.

The bedrock from the cores recovered is described as dolomitic limestone, brown and grey, fresh to slightly weathered. The total core recovery (TCR) in the bedrock was measured 96% to 100%, while Rock Quality Designation (RQD) values of 60% in Borehole C1 and 29% in Borehole C3 were recorded. The RQD results indicate a fair rock quality in Borehole C1 and poor rock quality in Borehole C3.

#### 4.3.6 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 in Borehole C1. The observations are shown on the individual Record of Borehole sheets in Appendix A2 and summarized below in Table 6.

**Table 6: Summary of Groundwater Levels at C20**

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date Measured	Piezometer Installed?
C1	252.7	3.5/249.2	1.3/251.4	26 Aug 2009 (71 days after completion)	Yes
C3	253.1		None Observed* (dry on completion)	16 Jun 2009	No

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date Measured	Piezometer Installed?
C4	254.5		None Observed* (dry on completion, cave @ 4.4 m)	25 Jun 2009	No
H10	255.0		None Observed* (dry on completion, cave @ 5.8 m)	20 Jun 2009	No
H11	252.6		None Observed* (dry on completion)	20 Jun 2009	No

Note: \* Groundwater level measured not stabilized.

Based on the wetness condition of the soil samples, colour of the soil and observations made in the piezometer, the groundwater level at the time of the field program was generally between Elevations 251 m to 252 m.

It should be noted that groundwater levels are subject to variation due to the influence of rainfall, seasons and other factors. There may also be potential for development of perched groundwater tables following periods of rainfall and groundwater may rise to the ground surface. As well, the water level in the watercourse may influence the water table at the site.

For and on behalf of Coffey Geotechnics Inc.



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Senior Principal



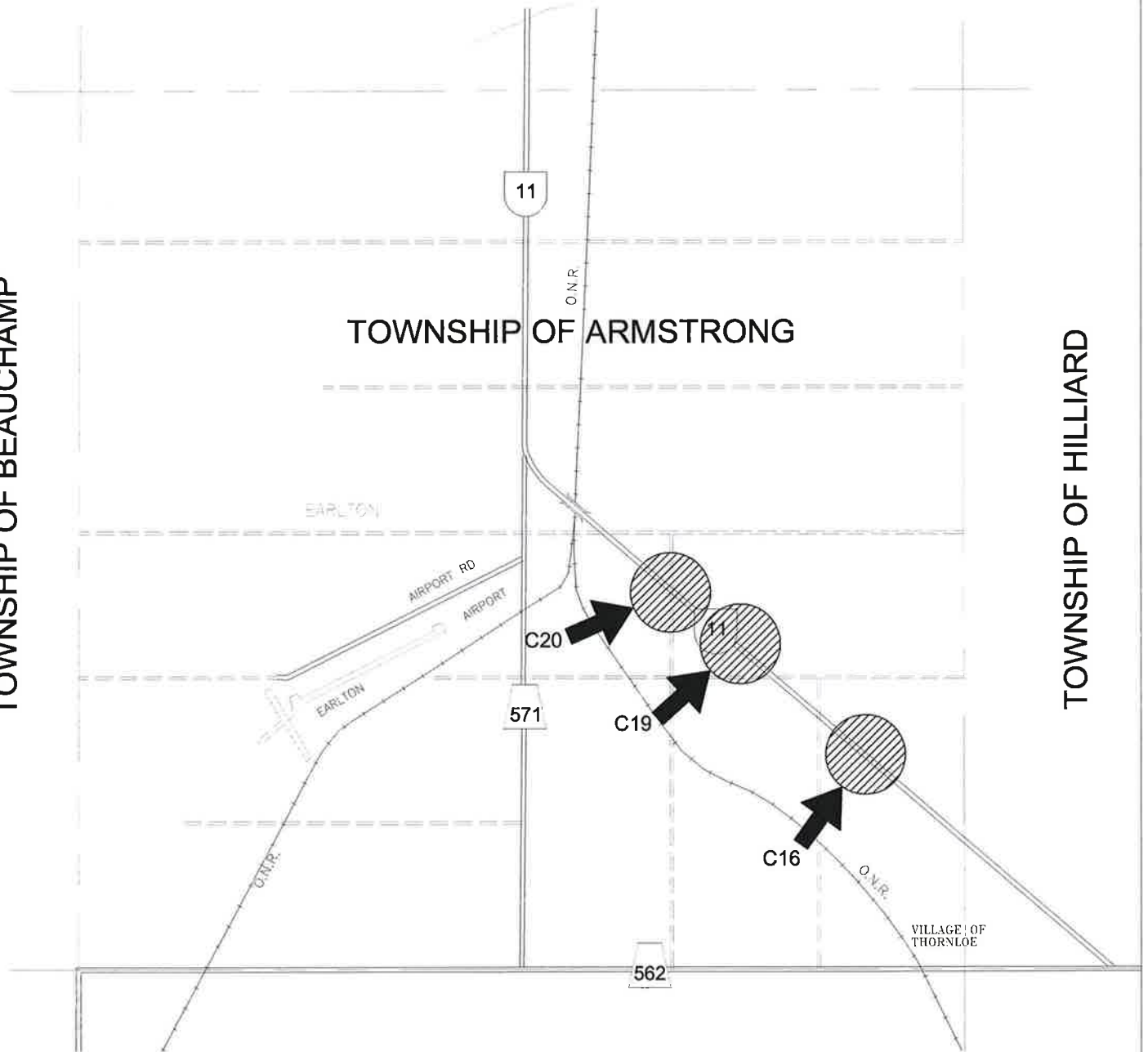
## Drawings

# TOWNSHIP OF EVANTUREL

TOWNSHIP OF BEAUCHAMP


## TOWNSHIP OF ARMSTRONG

TOWNSHIP OF HILLIARD



## TOWNSHIP OF KERNS

TOWNSHIP OF HARLEY

drawn:	PHK	 <b>coffey</b> <b>geotechnics</b> SPECIALISTS MANAGING THE EARTH	G.W.P. 161-98-00
checked:	RM		project:
approved:	ZO		HIGHWAY 11, ARMSTRONG TWP PROPOSED CULVERTS
date:	June 3, 2010		title:
scale:	AS SHOWN		SITE LOCATION PLAN
		project no:	TRANETOB01237AA
		drawing no:	1

**METRIC**

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.

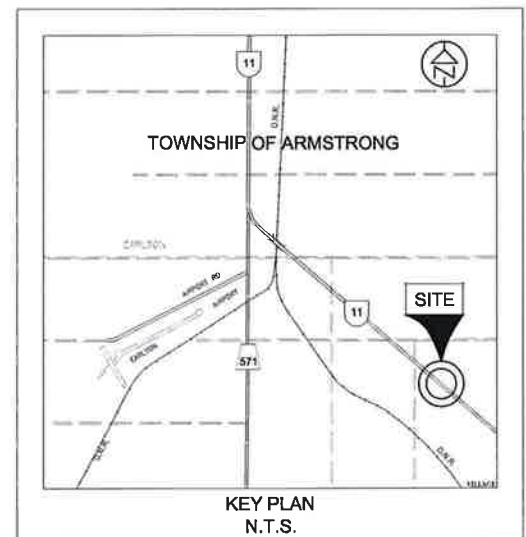
GWP: 161-98-00

HIGHWAY 11, ARMSTRONG TWP  
CULVERT C16 @ STA 13+694  
BOREHOLE LOCATION PLAN



SHEET

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SPECIALISTS MANAGING THE EARTH



LEGEND

- Borehole
- DCPT (Dynamic Cone Penetration Test)
- Borehole and DCPT

No.	ELEVATION	STATION	OFFSET
C8	258.7	13+695	7.8m Rt C/L
C9	256.6	13+700	24.6m Rt C/L
A4	258.1	13+675	13.9m Rt C/L

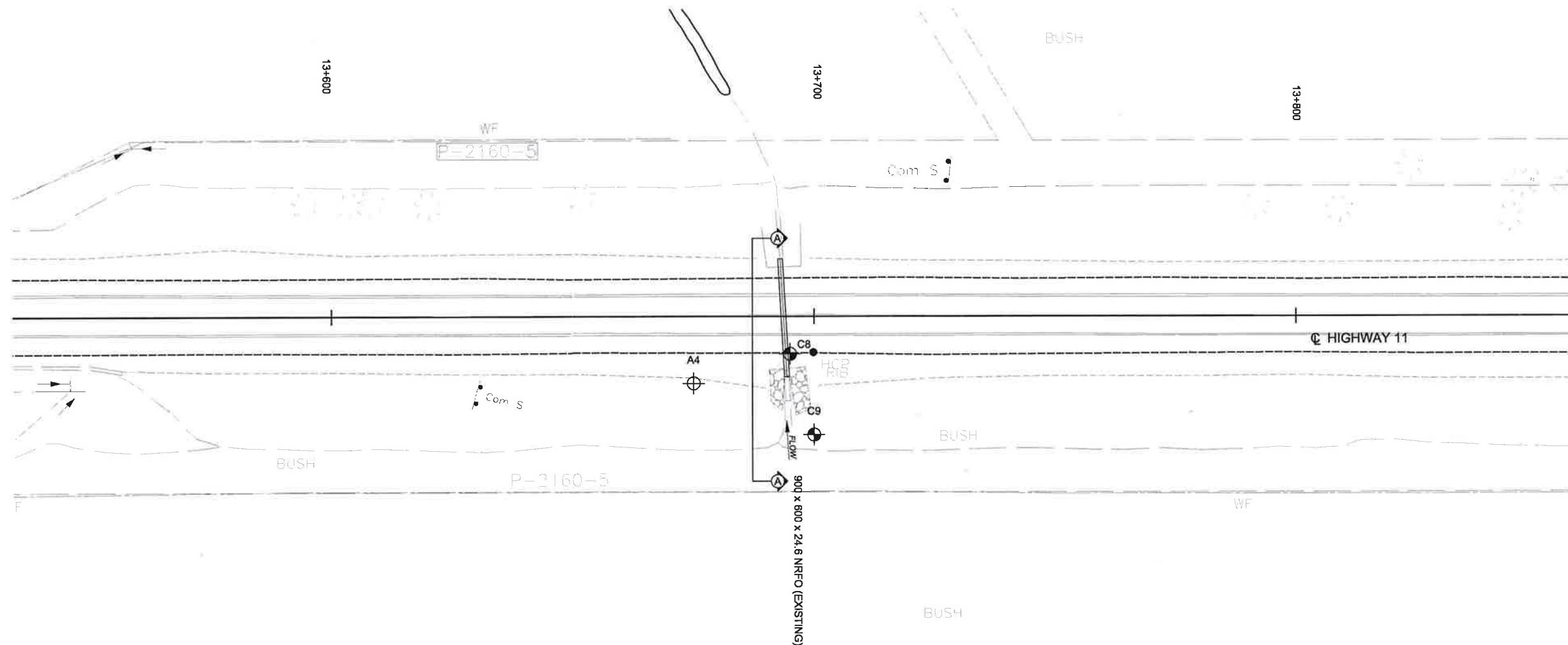
**-NOTE-**

The boundaries between 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 31M-82			
TRANETOB01237AA		DATE	DIST
SUBMD	CHECKED	June 3, 2010	SITE
DRAWN	PHK	APPROVED	DWG
CHECKED	RDP	ZO	2A



PLAN  
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: 161-98-00

HIGHWAY 11, ARMSTRONG TWP  
CULVERT C16 @ STA 13+694  
SOIL STRATA (SECTION A-A)

SHEET



KEY PLAN  
N.T.S.

LEGEND

- Borehole
- Dynamic Cone Penetration Test (DCPT)
- Borehole & DCPT
- 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
C8	258.7	13+695	7.8m Rt C/L
C9	256.6	13+700	24.6m Rt C/L
A4	258.1	13+675	13.9m Rt C/L

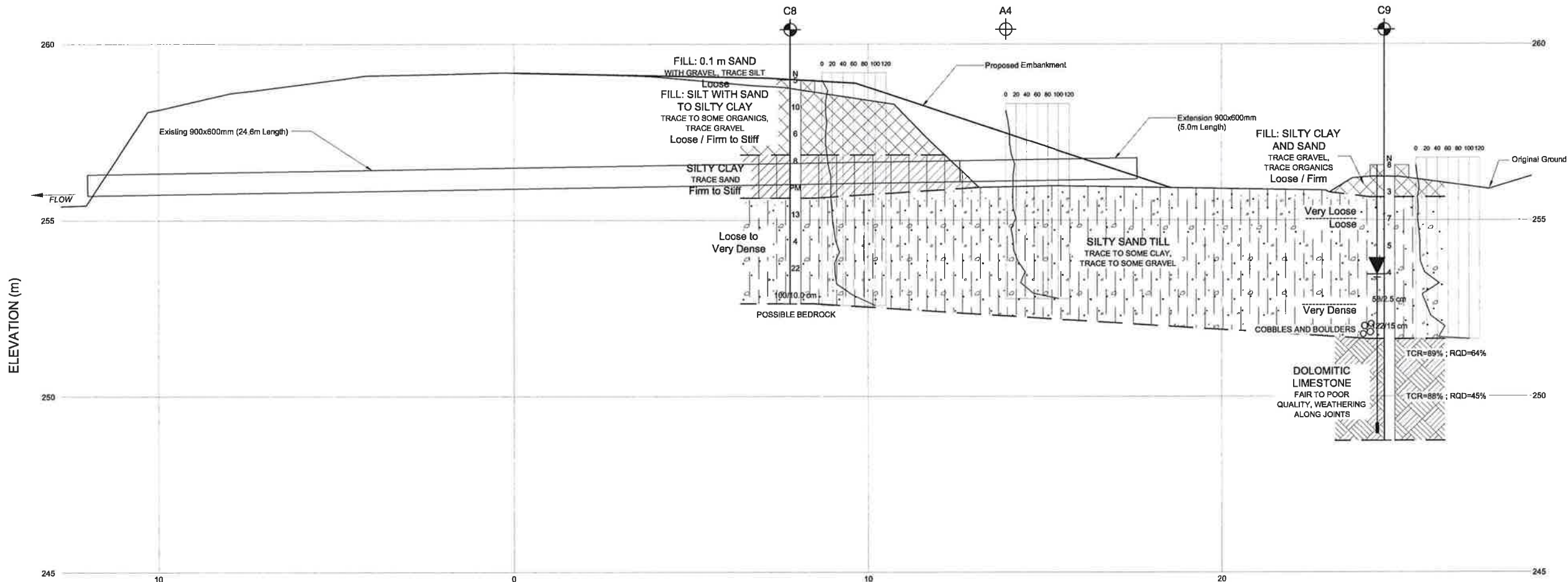
-NOTE-

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

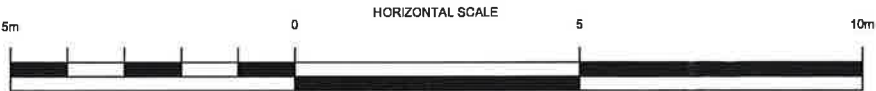
NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31M-82	TRANET0801237AA	DIST
SUBMD	CHECKED	DATE June 3, 2010
DRAWN	PHK	CHECKED RDP
APPROVED	ZO	DWG 28



SECTION A-A





**METRIC**

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.

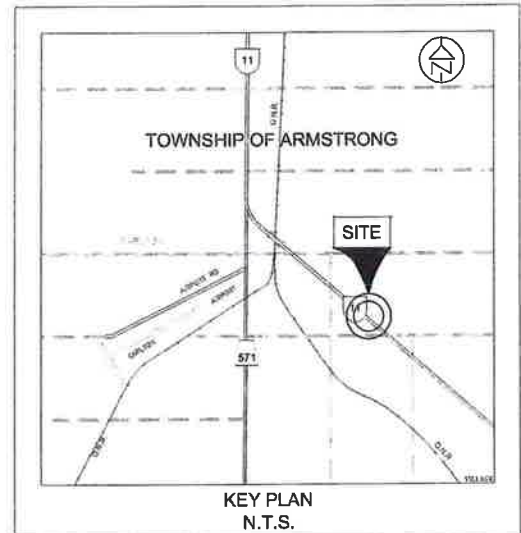
GWP: 161-98-00

HIGHWAY 11, ARMSTRONG TWP  
CULVERT C19 @ STA 15+328  
BOREHOLE LOCATION PLAN



SHEET

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LEGEND

- Borehole
- DCPT (Dynamic Cone Penetration Test)

No.	ELEVATION	STATION	OFFSET
C5	260.9	15+326	15.0m Rt C/L
C6	258.4	15+331	19.5m Lt C/L
C7	262.3	15+327	6.5m Rt C/L
C5*	260.9	15+329.8	15.2m Rt C/L
C6*	258.9	15+325.5	18.8m Lt C/L
C7*	262.3	15+330	6.7m Rt C/L
D2	261.7	15+275	9.0m Rt C/L
D3	259.4	15+375	15.3m Rt C/L

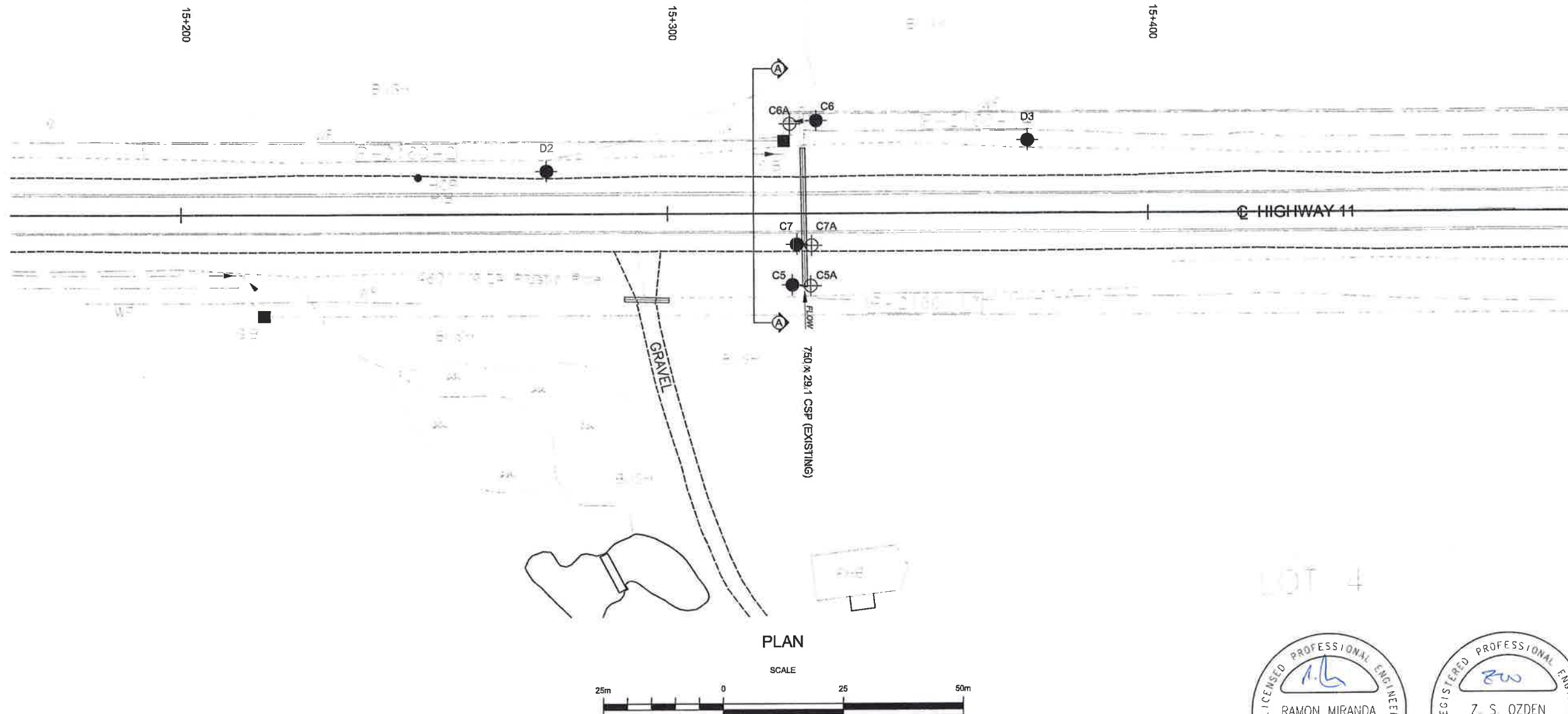
-NOTE-

The boundaries between 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 31M-82			
TRANET0801237AA			
SUBMD	CHECKED	DATE	DIST
DRAWN	PHK	CHECKED	RM
APPROVED	ZO	DWG	3A



**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.

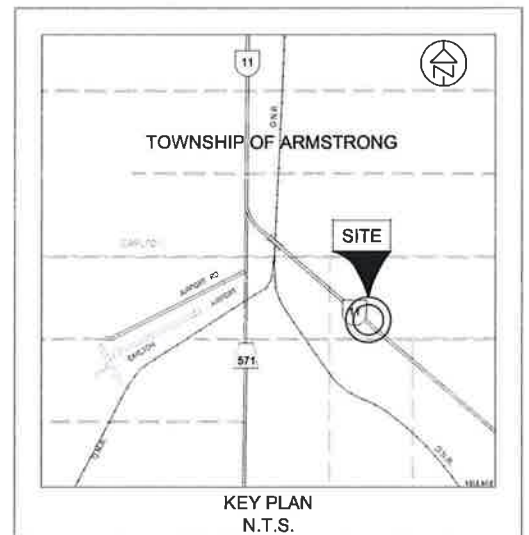
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GWP: 161-98-00

HIGHWAY 11, ARMSTRONG TWP  
CULVERT C19 @ STA 15+328  
SOIL STRATA (SECTION A-A)

SHEET

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SPECIALISTS MANAGING THE EARTH



LEGEND

- Borehole
- Dynamic Cone Penetration Test (DCPT)
- 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
C5	260.9	15+326	15.0m Rt C/L
C6	258.4	15+331	19.5m Lt C/L
C7	262.3	15+327	6.5m Rt C/L
C5A	260.9	15+329.8	15.2m Rt C/L
C6A	258.9	15+325.5	18.8m Lt C/L
C7A	262.3	15+330	6.7m Rt C/L

-NOTE-

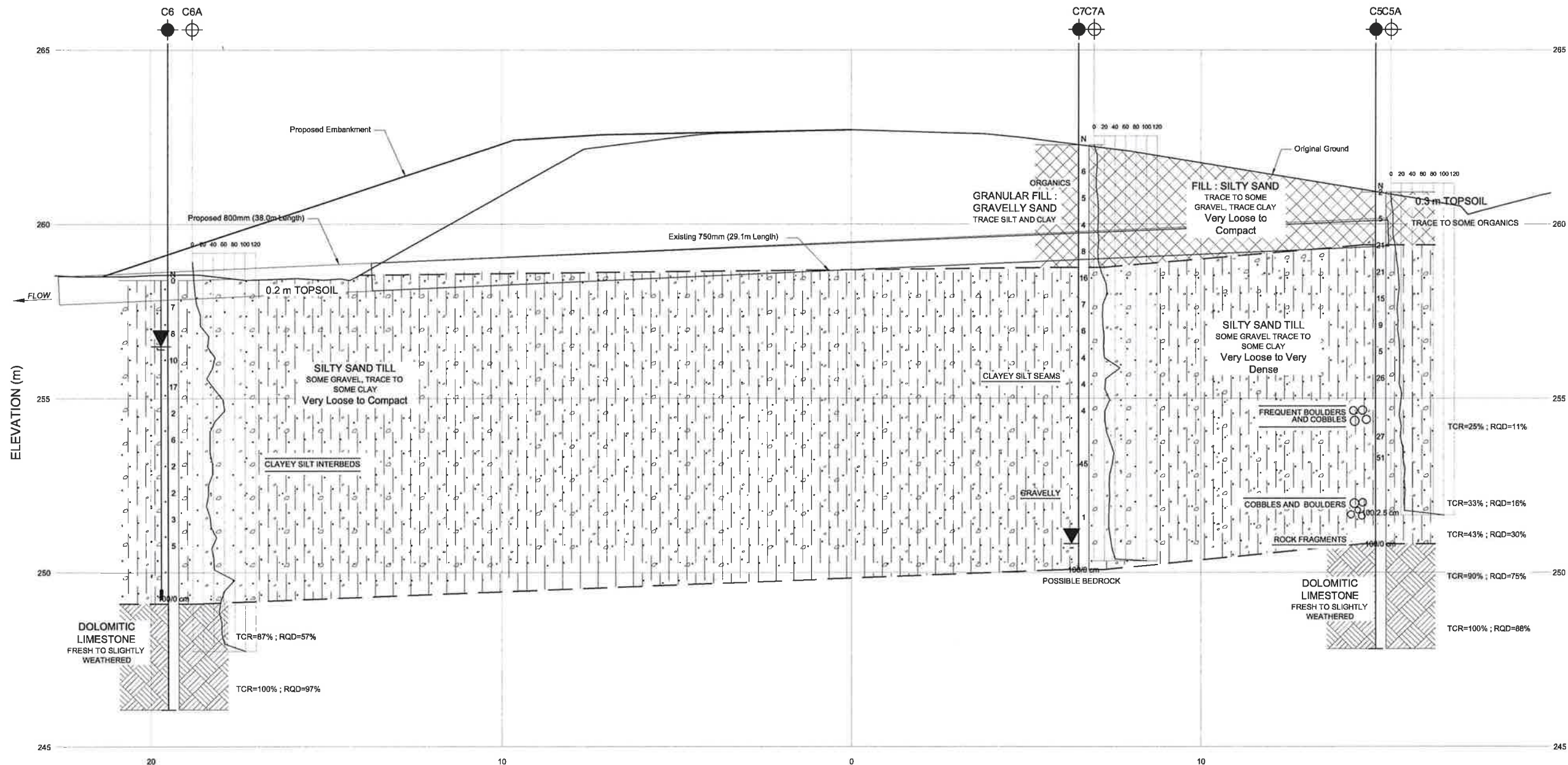
The boundaries between strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31M-82

TRANETO01237AA				DIST	
SUBM'D		CHECKED		DATE June 3, 2010	
DRAWN PHK		CHECKED RM		APPROVED ZO	
				DWG 3B	



SECTION A-A





METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

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

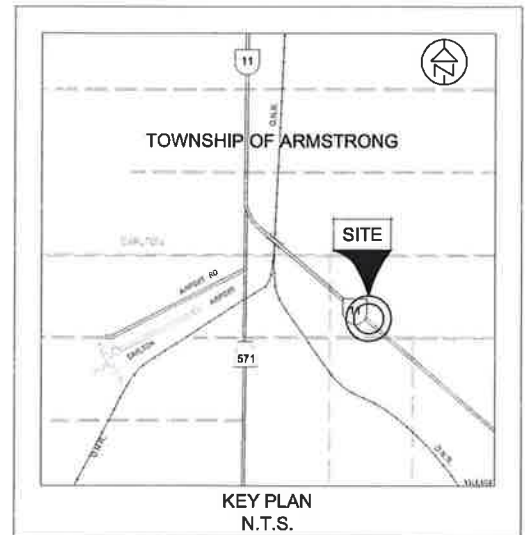
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GWP: 161-98-00

HIGHWAY 11, ARMSTRONG TWP  
CULVERT C19 @ STA 15+328  
SOIL STRATA (Q PROFILE)

SHEET

coffey geotechnics  
SPECIALISTS MANAGING THE EARTH



KEY PLAN  
N.T.S.

LEGEND

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

No.	ELEVATION	STATION	OFFSET
C6	258.4	15+331	19.5m Lt C/L
D2	261.7	15+275	9.0m Rt C/L
D3	259.4	15+375	15.3m Rt C/L

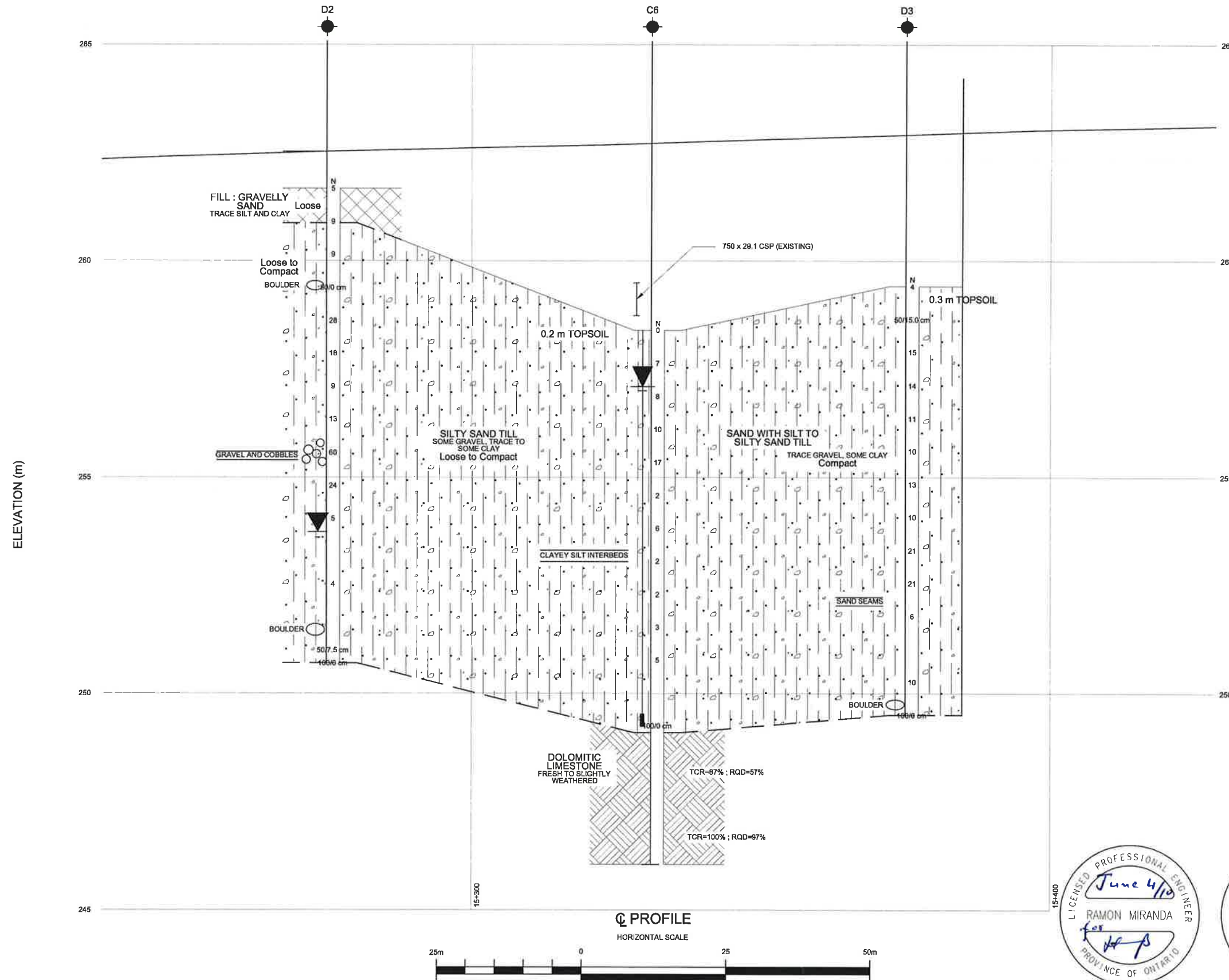
-NOTE-

The boundaries between 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 31M-82	TRANETOB01237AA	DIST
SUBMD	CHECKED	DATE June 3, 2010
DRAWN	PHK	CHECKED RM APPROVED ZO
DWG	3C	SITE



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.

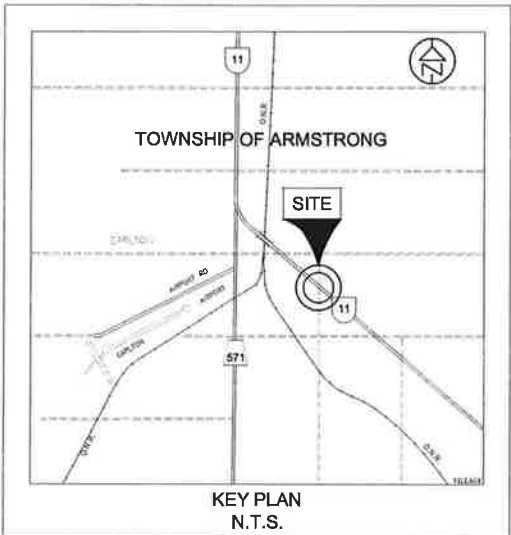
CONT No.  
GWP: 161-98-00

HIGHWAY 11, ARMSTRONG TWP  
CULVERT C20 @ STA 16+268  
BOREHOLE LOCATION PLAN



SHEET

coffey geotechnics  
SPECIALISTS MANAGING THE EARTH



LEGEND

- Borehole
- DCPT (Dynamic Cone Penetration Test)
- Borehole and DCPT

No.	ELEVATION	STATION	OFFSET
C1	252.7	16+265	16.6m Lt C/L
C3	253.1	16+265	17.8m Rt C/L
C4	254.5	16+272	8.0m Lt C/L
C3A	254.9	16+269	9.3m Rt C/L
C4A	254.9	16+265	8.8m Lt C/L
H10	255.0	16+230	6.7m Lt C/L
H11	252.6	16+305	18.2m Lt C/L

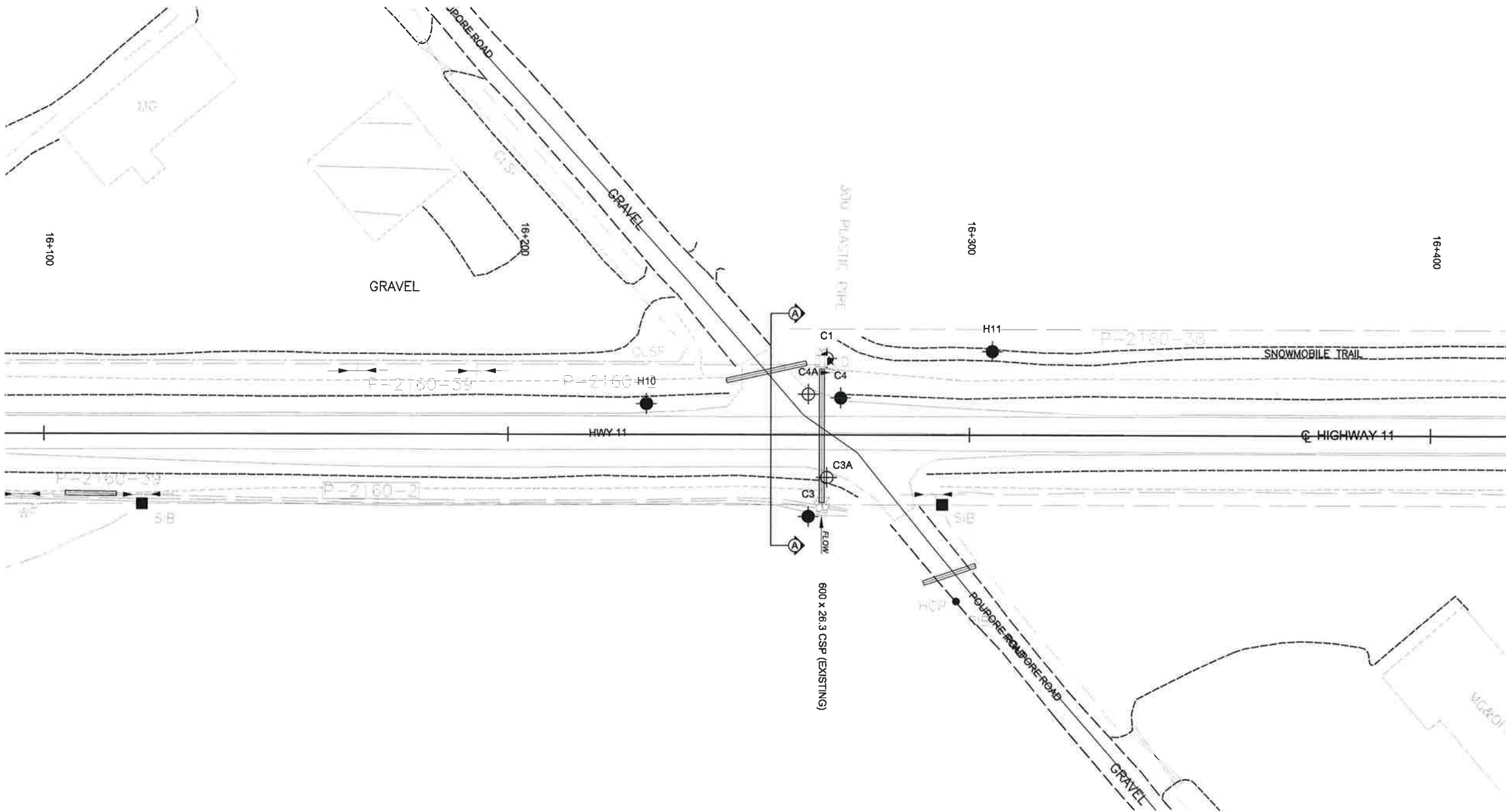
-NOTE-

The boundaries between 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 31M-82			
TRANETO801237AA			
SUBMD	CHECKED	DATE	JUNE 3, 2010
DRAWN	PHK	CHECKED	RM
APPROVED	ZO	DWG	4A



PLAN  
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.

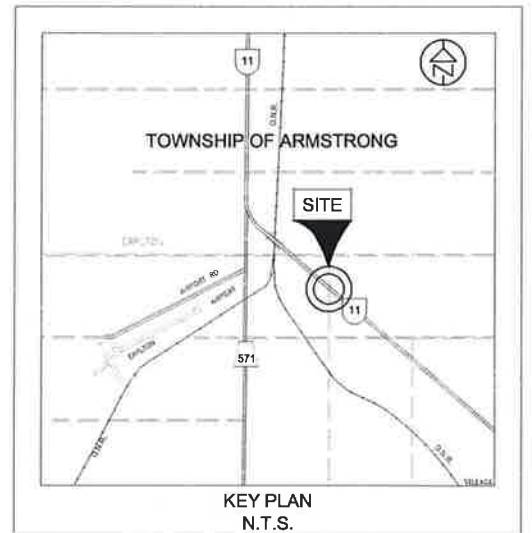
CONT No.

GWP: 161-98-00

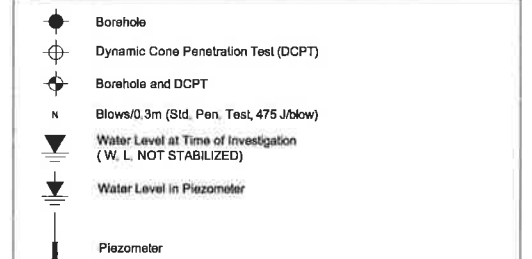
HIGHWAY 11, ARMSTRONG TWP  
CULVERT C20 @ STA 16+268  
SOIL STRATA (SECTION A-A)

SHEET

coffey geotechnics  
SPECIALISTS MANAGING THE EARTH



LEGEND



No.	ELEVATION	STATION	OFFSET
C1	252.7	16+265	16.6m Lt C/L
C3	253.1	16+265	17.8m Rt C/L
C4	254.5	16+272	8.0m Lt C/L
C3A	254.9	16+269	9.3m Rt C/L
C4A	254.9	16+265	8.8m Lt C/L

\* Existing culvert length is based on the information provided to us by D. M. Wills.

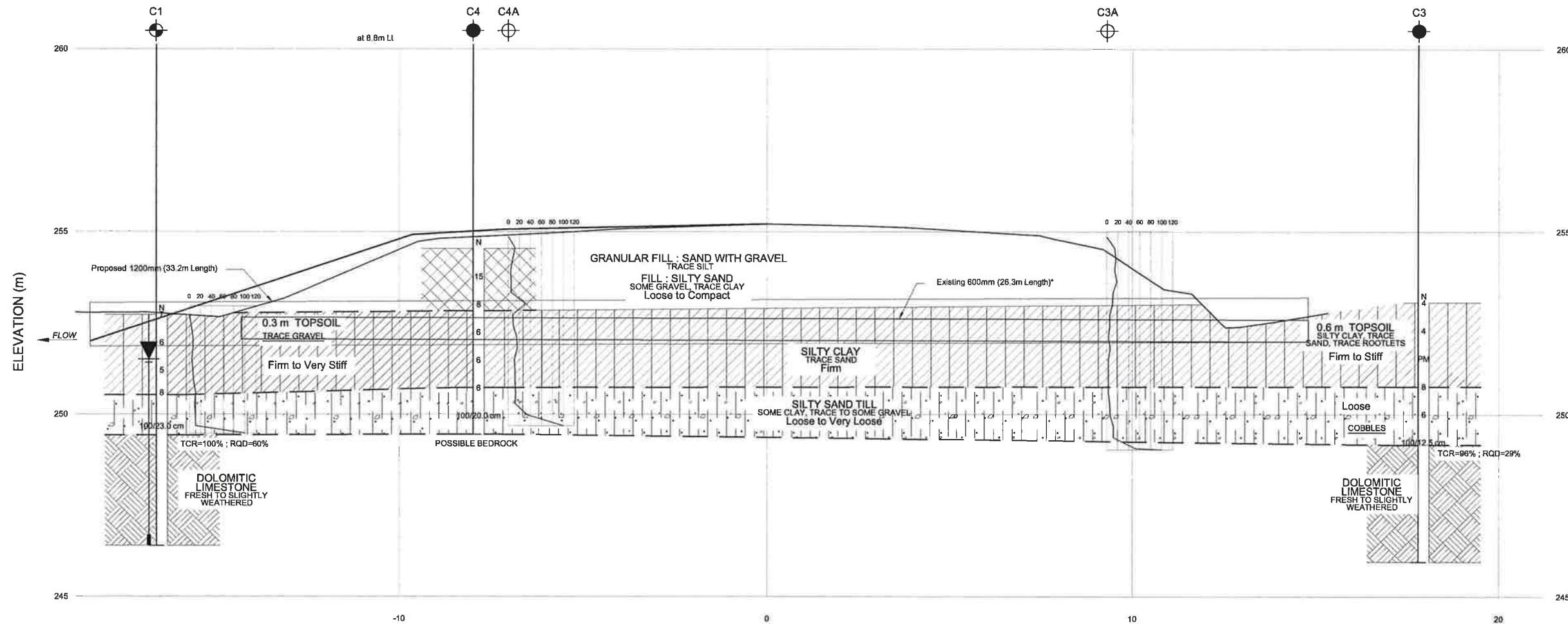
-NOTE-

The boundaries between 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 31M-82			
TRANETOB01237AA			
SUBMD	CHECKED	DATE	JUNE 3, 2010
DRAWN	PHK	CHECKED	RM
APPROVED	ZO	DWG	4B



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.

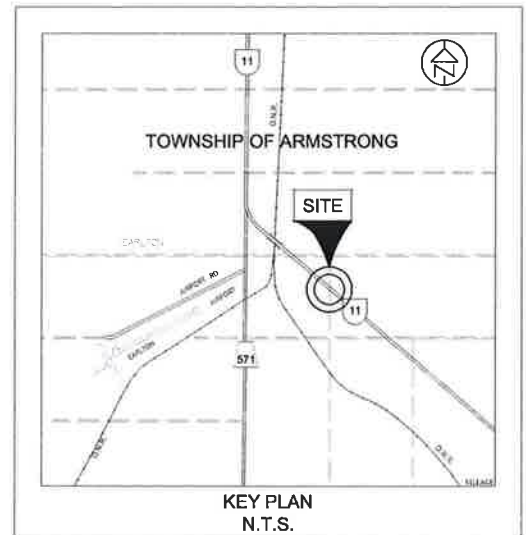
CONT No.

GWP: 161-98-00

HIGHWAY 11, ARMSTRONG TWP  
CULVERT C20 @ STA 16+268  
SOIL STRATA (Q PROFILE)

SHEET

coffey geotechnics  
SPECIALISTS MANAGING THE EARTH



LEGEND			
	Borehole		
	Dynamic Cone Penetration Test (DCPT)		
	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
C4	254.5	16+272	8.0m Lt C/L
C4A	254.9	16+265	8.8m Lt C/L
H10	255.0	16+230	6.7m Lt C/L
H11	252.6	16+305	18.2m Lt C/L

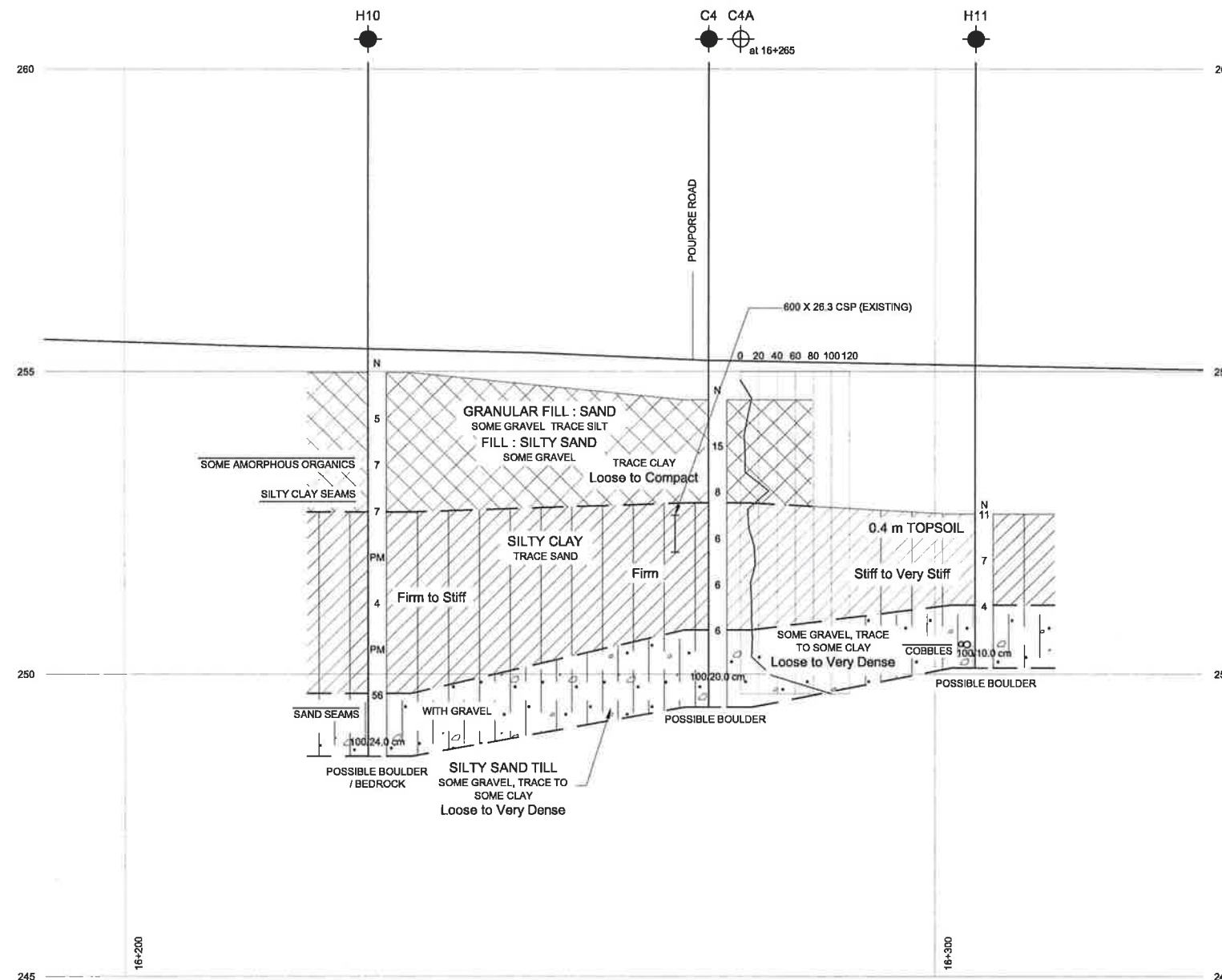
**-NOTE-**  
The boundaries between strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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

REVISIONS			
DATE	BY		DESCRIPTION

Geocres No 31M-82			
TRANETO01237AA		DIST	
SUBMD	CHECKED	DATE	SITE
DRAWN	PHK	CHECKED	DWG
	RM	APPROVED	4C

ELEVATION (m)



Q PROFILE



# Appendix A1

**Record of Borehole Sheets – C16**

TRANETOB01237AA: HWY 11

# RECORD OF BOREHOLE No A4

1 OF 1

METRIC

GWP 161-98-00 LOCATION Culvert # 16 / Passing Lane, Sta:13+675, 13.9 m Rt C/L of Hwy 11 ORIGINATED BY G.J.  
 DIST HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY S.K.  
 DATUM Geodetic DATE 7/15/2009 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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>		
258.1 0.0	GROUND SURFACE						258	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR × LAB VANE						
252.8 5.3	End of DCPT @ 5.3 m on poss. bedrock						253	WATER CONTENT (%) 10 20 30						

TRANETOB01237AA: HWY 11

## RECORD OF BOREHOLE No C8

1 OF 1

**METRIC**

GWP	161-98-00	LOCATION	Culvert # 16 / Passing Lane, Sta:13+695, 7.8 m Rt C/L of Hwy 11	ORIGINATED BY	Z.I.
DIST		HWY	11	BOREHOLE TYPE	Hollow Stem Auger, DCPT
DATUM	Geodetic	DATE	8/10/2009	COMPILED BY	S.K.
				CHECKED BY	Z.O.

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

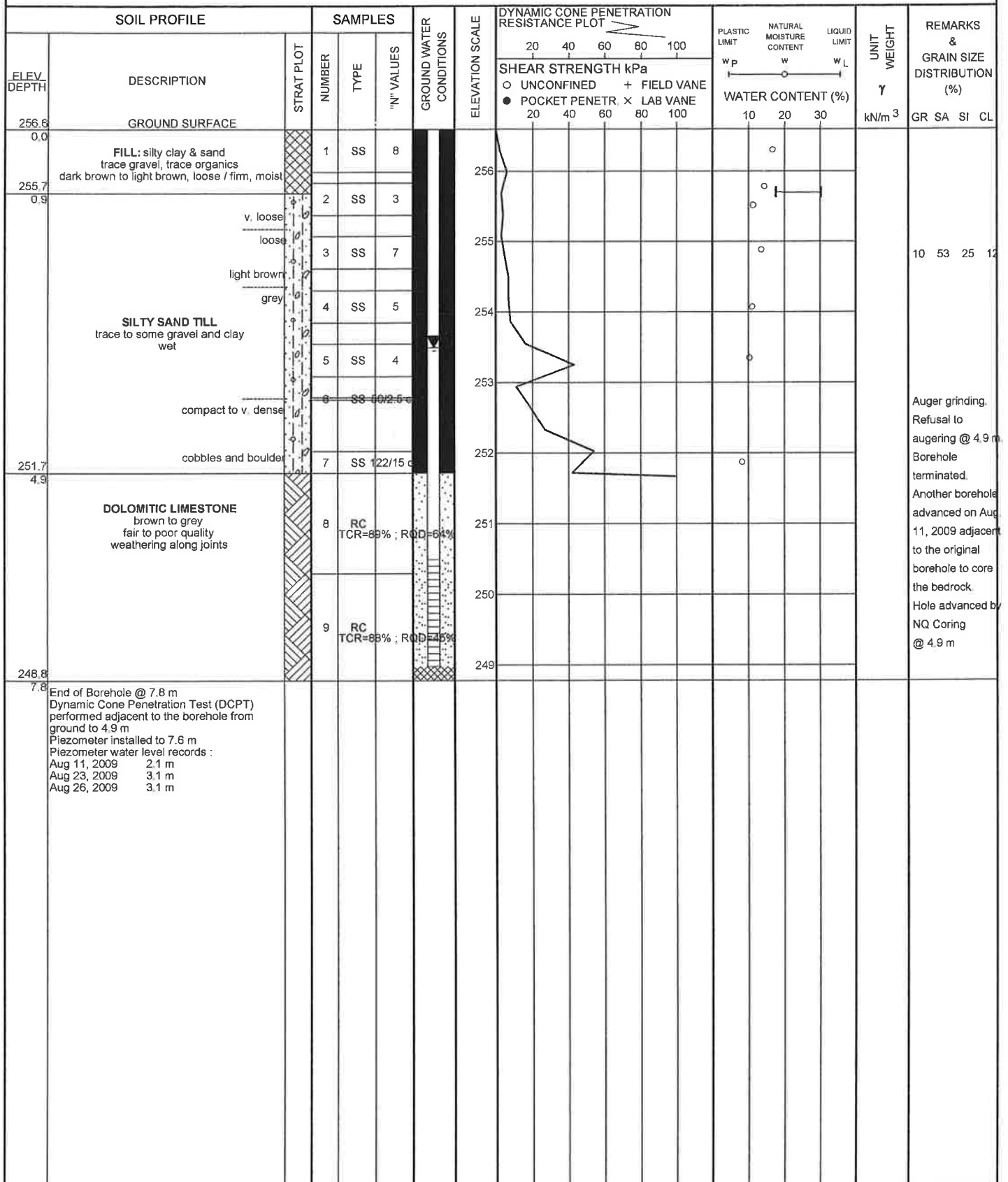
TRANETOB01237AA: HWY 11

# RECORD OF BOREHOLE No C9

1 OF 1

METRIC

GWP 161-98-00 LOCATION Culvert # 16 / Passing Lane, Sta: 13+700, 24.6 m Rt C/L of Hwy 11 ORIGINATED BY G.J.Z.I.  
DIST HWY 11 BOREHOLE TYPE Hollow Stem Auger, NQ Coring, Wash Boring, DCPT COMPILED BY S.K.  
DATUM Geodetic DATE 7/16/2009 8/11/2009 CHECKED BY Z.O.





# Appendix A2

**Record of Borehole Sheets – C19**

TRANETOBO1237AA: HWY 11

# RECORD OF BOREHOLE No C5

1 OF 1

METRIC

GWP 161-98-00 LOCATION Culvert # 19, Sta: 15+326, 15.0 m Rt C/L of Hwy 11 ORIGINATED BY G.J.  
DIST HWY 11 BOREHOLE TYPE Hollow Stem Auger, NQ Coring, Wash Boring COMPILED BY S.K.  
DATUM Geodetic DATE 6/17/2009 6/18/2009 CHECKED BY Z.O.

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						
260.9 0.0	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30				
	0.3 m TOPSOIL		1	SS	2									
	FILL : SILTY SAND trace gravel, trace clay trace to some organics brown, moist to wet v. loose to compact		2	SS	5									4 57 28 11
259.4 1.5			3	SS	21									
	SILTY SAND TILL some gravel trace to some clay grey, moist loose to v.dense		4	SS	21									
			5	SS	15									9 40 33 18 auger grinding
			6	SS	9									11 49 25 15
			7	SS	5									
			8	SS	26									
			9	RC	TCR=25%									spoon refusal on boulder & get through by coring
		frequent boulders and cobbles	10	SS	27									no recovery
			11	SS	51									
			12	RC	TCR=33%									spoon refusal on boulder & get through by coring
		frequent cobbles and boulders	13	SS	100/2.5 cm									spoon refusal on boulder & get through by coring
			14	RC	TCR=43%									spoon refusal on bedrock & hole advanced by NQ Coring
250.8 10.1		highly fragmented	15	SS	100/0 cm									
	DOLOMITIC LIMESTONE fresh to slightly weathered		16	RC	TCR=90% ; RQD=75%									
		grey brown	17	RC	TCR=100% ; RQD=88%									
247.8 13.1	End of Borehole @ 13.1 m Hole dry (not stabilized) and open upon completion.													

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

TRANETOB01237AA: HWY 11

# RECORD OF BOREHOLE No C5A

1 OF 1

METRIC

GWP 161-98-00 LOCATION Culvert # 19, Sta: 15+330, 15.2 m Rt C/L of Hwy 11 ORIGINATED BY Z.L.  
 DIST HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY S.K.  
 DATUM Geodetic DATE 8/12/2009 CHECKED BY Z.O.

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 (%)					
260.9 0.0	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE	10 20 30					
260														
259														
258														
257														
256														
255														
254														
253														
252														
251.6 9.3	End of DCPT @ 9.3 m on possible boulder													

TRANETOB01237AA: HWY 11

## 1 OF 1

## METRIC

GWP	161-98-00	LOCATION	Culvert # 19 / Embankment Fill # 3 , Sta: 15+331, 19.5 m Lt C/L of Hwy 11	ORIGINATED BY	G.J.
DIST	HWY 11	BOREHOLE TYPE	Hollow Stem Auger, NQ Coring , Wash Boring	COMPILED BY	S.K.
DATUM	Geodetic	DATE	6/18/2009	CHECKED BY	Z.O.

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

TRANETOB01237AA: HWY 11

# RECORD OF BOREHOLE No C6A

1 OF 1

METRIC

GWP 161-98-00 LOCATION Culvert # 19 / Embankment Fill # 3, Sta. 15+325.5, 18.8 m Lt C/L of Hwy 11 ORIGINATED BY Z.I.  
 DIST HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY S.K.  
 DATUM Geodetic DATE 8/11/2009 CHECKED BY Z.O.

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								
258.9 0.0	GROUND SURFACE											GR SA SI CL
258												
257												
256												
255												
254												
253												
252												
251												
250												
249												
248												
247.7 11.2	End of DCPT @ 11.2 m on possible bedrock											

TRANETO01237AA: HWY 11

# RECORD OF BOREHOLE No C7

1 OF 1

METRIC

GWP 161-98-00 LOCATION Culvert # 19, Sta: 15+327, 6.5 m Rt C/L of Hwy 11 ORIGINATED BY G.J.  
 DIST HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.K.  
 DATUM Geodetic DATE 6/19/2009 CHECKED BY Z.O.

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				
							20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT		
							20 40 60 80 100	W <sub>P</sub>	W	W <sub>L</sub>		
							20 40 60 80 100	WATER CONTENT (%)				
							20 40 60 80 100					
262.3	GROUND SURFACE						262					GR SA SI CL
0.0	GRANULAR FILL GRAVELLY SAND trace silt and clay brown, moist		1	AS								20 66 (14)
261.5			2	SS	6		261					less recovery
0.8	black amorphous organics		3	SS	5		260					
	FILL : SILTY SAND some gravel, trace clay moist to wet v. loose to loose		4	SS	4		259					
258.8			5	SS	8		258					8 54 26 12
3.5			6	SS	16		257					
	SILTY SAND TILL some gravel, trace clay grey, moist, v. loose to loose with v. dense layer		7	SS	7		256					
			8	SS	6		255					8 47 32 13
			9	SS	4		254					
			10	SS	4		253					auger grinding
			11	SS	4		252					
			12	SS	45		251					
			13	SS	1							
250.1			14	SS	100/0 cm							
12.2	End of Borehole. Auger refusal @ 12.2 m on possible bedrock Water level @ 11.4 m (not stabilized)* upon completion Hole caved-in @ 11.7 m upon completion											

+ 3, x 3, 20  
Sensitivity 15 5 10 (%) STRAIN AT FAILURE

TRANETOB01237AA: HWY 11

## 1 OF 1

**METRIC**

DATUM Geodetic DATE 8/11/2009 CHECKED BY Z.O.

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

TRANETO01237AA: HWY 11

# RECORD OF BOREHOLE No D2

1 OF 1

METRIC

GWP 161-98-00 LOCATION Embankment Fill Location # 3, Sta: 15+275, 9.0 m Lt C/L of Hwy 11 ORIGINATED BY G.J.  
DIST HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.K.  
DATUM Geodetic DATE 6/5/2009 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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					
								20 40 60 80 100					
261.7	GROUND SURFACE												
0.0	FILL : GRAVELLY SAND tr. silt and clay brown, loose, moist		1	SS	5								30 61 (9)
260.9													
0.8	SILTY SAND TILL some gravel, tr. to some clay brown, loose to compact		2	SS	9								
			3	SS	9								
	boulder		4	SS	50/0 cm								refusal on boulder
	brown, moist		5	SS	28								
	grey, wet		6	SS	18								7 43 35 15
			7	SS	9								
			8	SS	13								
	gravel and cobbles		9	SS	60								less recovery auger grinding
			10	SS	24								
			11	SS	5								
			12	SS	4								

+ 3 × 3

Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



TRANETO01237AA: HWY 11

# RECORD OF BOREHOLE No D3

1 OF 1

METRIC

GWP 161-98-00 LOCATION Embankment Fill Location # 3, Sta: 15+375, 15.3 m Lt C/L of Hwy 11 ORIGINATED BY G.J.  
 DIST HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.K.  
 DATUM Geodetic DATE 6/19/2009 CHECKED BY Z.O.

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				
							20 40 60 80 100	○ UNCONFINED	+ FIELD VANE			
							20 40 60 80 100	● POCKET PENETR	× LAB VANE			
								WATER CONTENT (%)				
								PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT		
								W <sub>P</sub>	W	W <sub>L</sub>		
259.4 0.0	GROUND SURFACE 0.3 m TOPSOIL		1	SS	4		259				14.4	
			2	SS	50/15.0 cm							
			3	SS	15		258					
			4	SS	14		257					
			5	SS	11		256					
			6	SS	10		255					3 43 33 21
			7	SS	13		254					
			8	SS	10		253					
			9	SS	21		252					9 65 (26)
			10	SS	21		251					
			11	SS	6		250					
			12	SS	10							
249.5 9.9	End of Borehole. Auger refusal @ 9.91 m on possible boulder. Hole dry (not stabilized) and caved-in @ 9.8 m upon completion.		13	SS	100.0 cm							no recovery

+ 3, x 3: Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

# Appendix A3

**Record of Borehole Sheets – C20**

TRANETOB01237AA: HWY 11

## 1 OF 1

METRIC

GWP	161-98-00	LOCATION	Culvert # 20, Sta: 16+269, 16.6 m Lt C/L of Hwy 11	ORIGINATED BY	G.J.
DIST		HWY	11	BOREHOLE TYPE	Hollow Stem Auger, NQ Coring, Wash Boring, DCPT
DATUM	Geodetic	DATE	6/16/2009	COMPILED BY	S.K.
				CHECKED BY	Z.O.

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    × LAB VANE				
252.7 0.0	GROUND SURFACE 0.3 m TOPSOIL		1	SS	7							
	SILTY CLAY trace sand brown, wet, firm to v. stiff		2	SS	6							
			3	SS	5							
250.5 2.2	SILTY SAND TILL some clay, some gravel grey, loose to v. dense		4	SS	8							
249.4 3.3	DOLOMITIC LIMESTONE fresh to slightly weathered		5	SS100/23.0								
246.4 6.3			6	RC								
End of Borehole @ 6.3 m Borehole dry (not stabilized) and open upon completion Dynamic Cone Penetration Test (DCPT) performed adjacent to the borehole from ground up to 3.3 m Piezometer installed to 3.5 m Piezometer water level records : June 25, 2009    1.2 m July 21, 2009    1.1 m July 24, 2009    1.2 m Aug 23, 2009    1.3 m Aug 26, 2009    1.3 m												

 $+^3, \times^3$ 

Numbers refer to  
Sensitivity

(%) STRAIN AT FAILURE

METRIC

(%) STRAIN AT FAILURE

TRANETOB01237AA: HWY 11

# RECORD OF BOREHOLE No C3A

1 OF 1

METRIC

GWP 161-98-00 LOCATION Culvert # 20, Sta: 16+269, 9.3 m Rt C/L of Hwy 11 ORIGINATED BY Z.I.  
 DIST HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY S.K.  
 DATUM Geodetic DATE 8/12/2009 CHECKED BY Z.O.

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					
254.9 0.0	GROUND SURFACE						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE	20 40 60 80 100	10 20 30				
254													
253													
252													
251													
250													
249.1 5.8	End of DCPT @ 5.8 m on poss. bedrock												

TRANETO01237AA: HWY 11

# RECORD OF BOREHOLE No C4

1 OF 1

METRIC

GWP 161-98-00 LOCATION Culvert # 20, Sta 16+272, 8.0 m Lt C/L of Hwy 11 ORIGINATED BY G.J.  
 DIST HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.K.  
 DATUM Geodetic DATE 6/25/2009 CHECKED BY Z.O.

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 (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE							
254.5	GROUND SURFACE						20	40	60	80	100				
0.0	GRANULAR FILL SAND WITH GRAVEL trace silt, brown, moist		1	AS											25 67 (8)
253.9															
0.6	FILL : SILTY SAND some gravel, trace clay brown, moist loose to compact		2	SS	15										
252.8			3	SS	8										
1.7	SILTY CLAY trace sand wet, firm														
			4	SS	6										
			5	SS	6										
250.7			6	SS	6										0 2 35 63
3.8	SILTY SAND TILL some clay, trace gravel grey, moist, loose to v. dense														9 41 33 17
			7	SS	100/20.0 cm										5 56 25 14
249.4															
5.1	End of Borehole Auger refusal @ 5.1 m on probable bedrock Borehole dry (not stabilized) upon completion Hole caved-in @ 4.4 m upon completion														

TRANETOB01237AA: HWY 11

# RECORD OF BOREHOLE No C4A

1 OF 1

METRIC

GWP 161-98-00 LOCATION Culvert # 20, Sta: 16+265, 8.8 m Lt C/L of Hwy 11 ORIGINATED BY Z.I.  
 DIST HWY 11 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY S.K.  
 DATUM Geodetic DATE 8/12/2009 CHECKED BY Z.O.

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					
254.9 0.0	GROUND SURFACE						○ UNCONFINED + FIELD VANE ● POCKET PENETR × LAB VANE						
254													
253													
252													
251													
250													
249.7 5.2	End of DCPT @ 5.2 m on poss. bedrock												

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

20  
15  
10  
(%) STRAIN AT FAILURE

TRANETO01237AA: HWY 11

# RECORD OF BOREHOLE No H10

1 OF 1

METRIC

GWP 161-98-00 LOCATION Culvert # 20, (For Detour) & Embankment Fill Location # 5, Sta: 16+230, 6.7 m Lt ORIGINATED BY G.J.  
DIST HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.K.  
DATUM Geodetic DATE 6/20/2009 CHECKED BY Z.O.

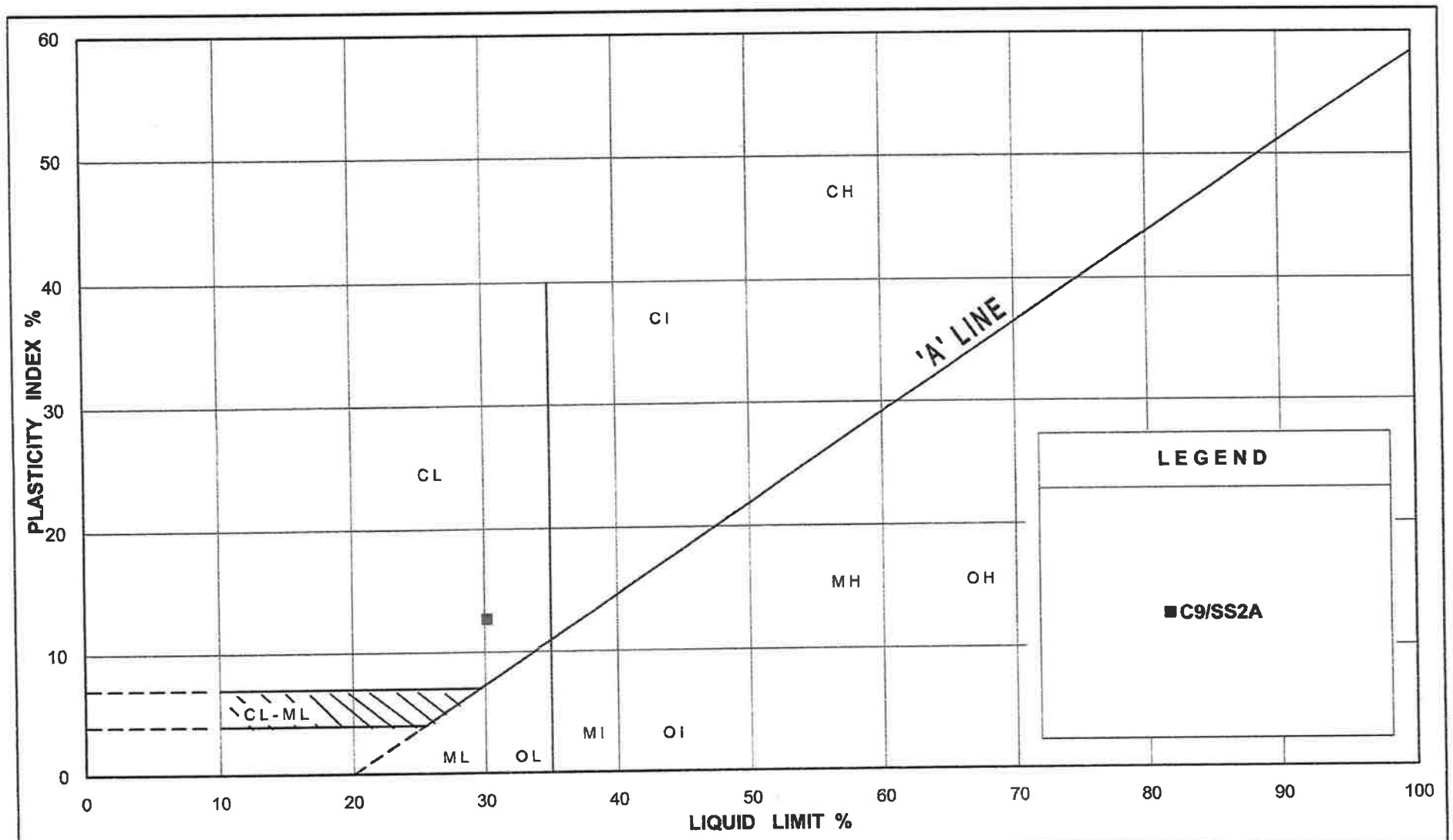
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
							20 40 60 80 100	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>		
							20 40 60 80 100	WATER CONTENT (%)				
								10 20 30				
255.0	GROUND SURFACE						255					
0.0	GRANULAR FILL SAND, SOME GRAVEL trace silt brown, moist		1	AS								
254.4												
0.6	FILL : SILTY SAND some gravel		2	SS	5		254					
	some amorphous org. dark grey silty clay seams, firm		3	SS	7		253					0 23 35 42
252.7												
2.3	SILTY CLAY trace sand greyish brown, wet, firm to stiff		4	SS	7		252	2.9			47.3	0 1 35 64
			5	TW	PM		251	3.8				
			6	SS	4		250	3.3				13 25 40 22
	some sand and grave (transition to till)		7	TW	PM		249					21 50 24 5
249.7												
5.3	SILTY SAND TILL with gravel, trace clay grey, moist, v. dense		8	SS	56		249					
248.7			9	SS	100/24.0 cm							
6.3	End of Borehole Auger refusal @ 6.3 m on possible boulder Borehole dry (not stabilized) upon completion Hole caved-in @ 5.8 m upon completion											





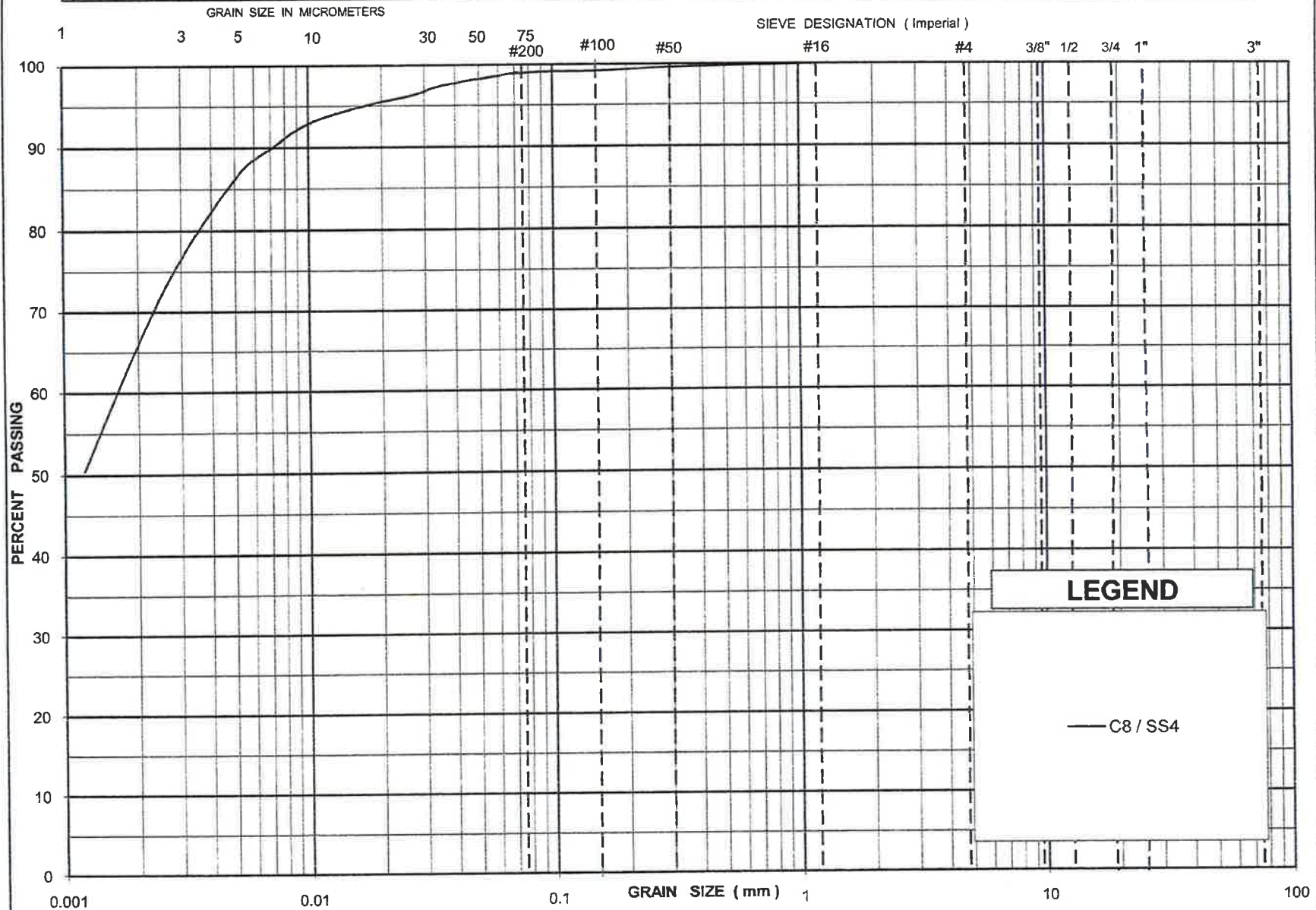
# Appendix B1

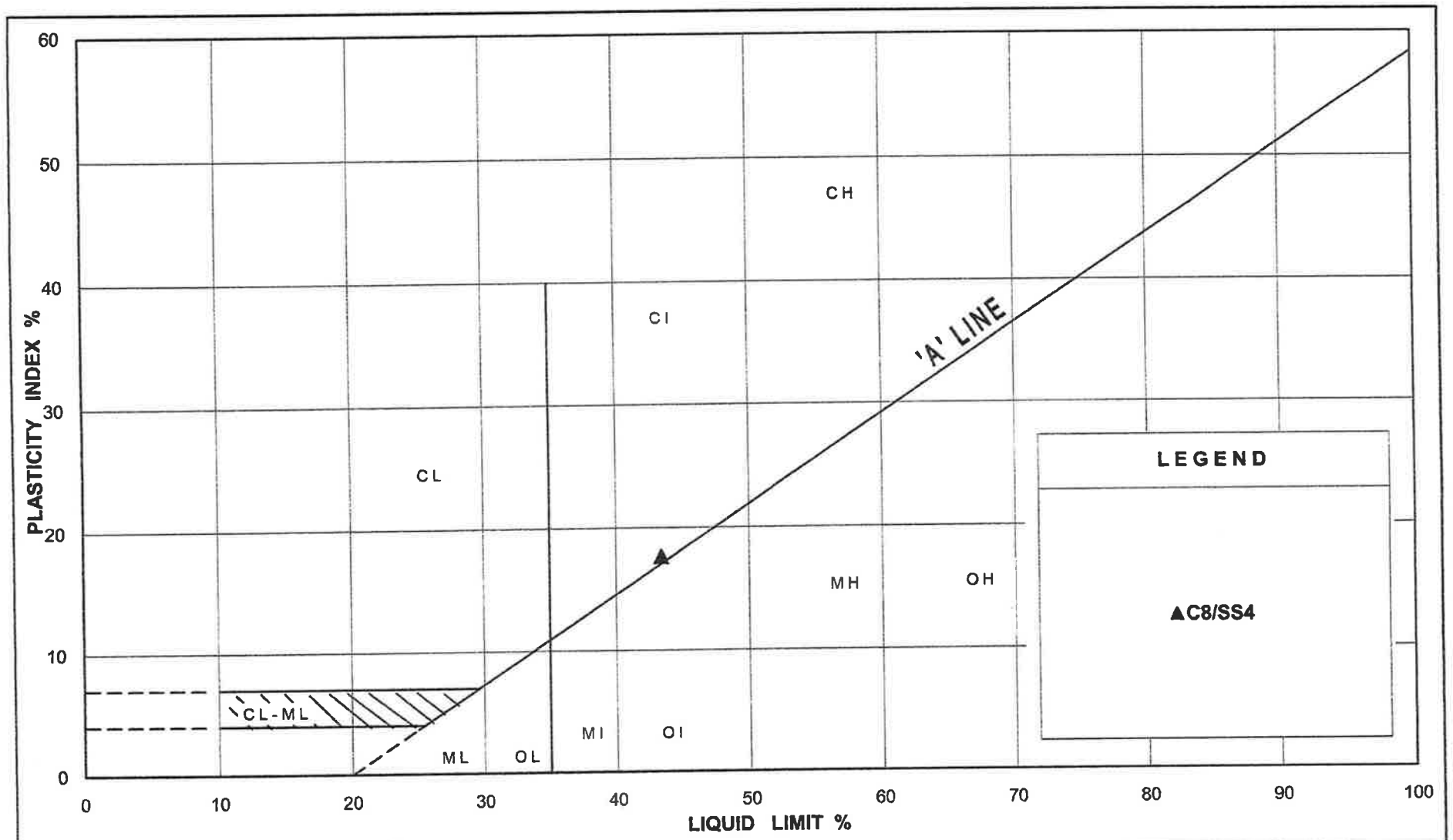
**Laboratory Test Results– C16**



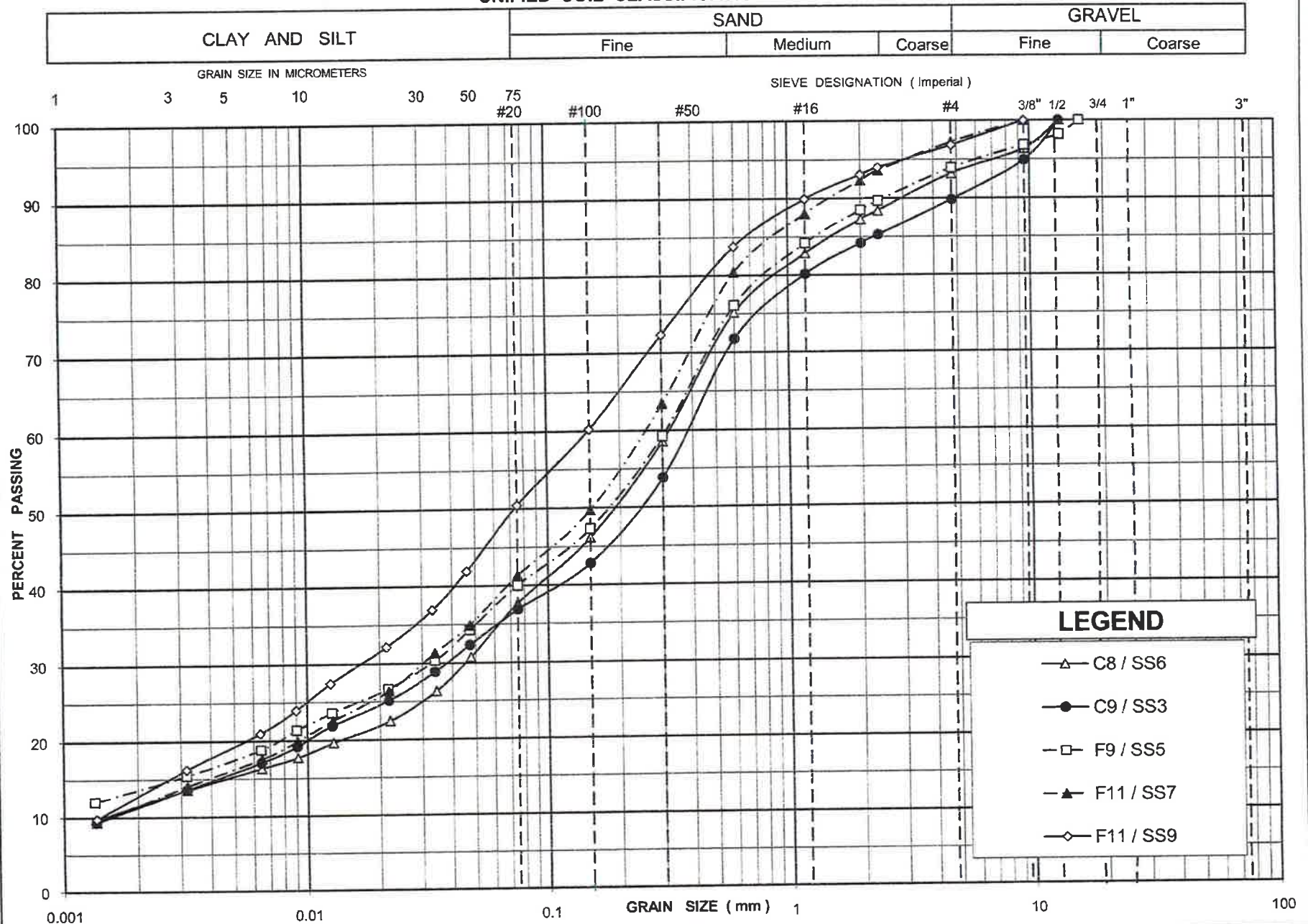
# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse





# UNIFIED SOIL CLASSIFICATION SYSTEM

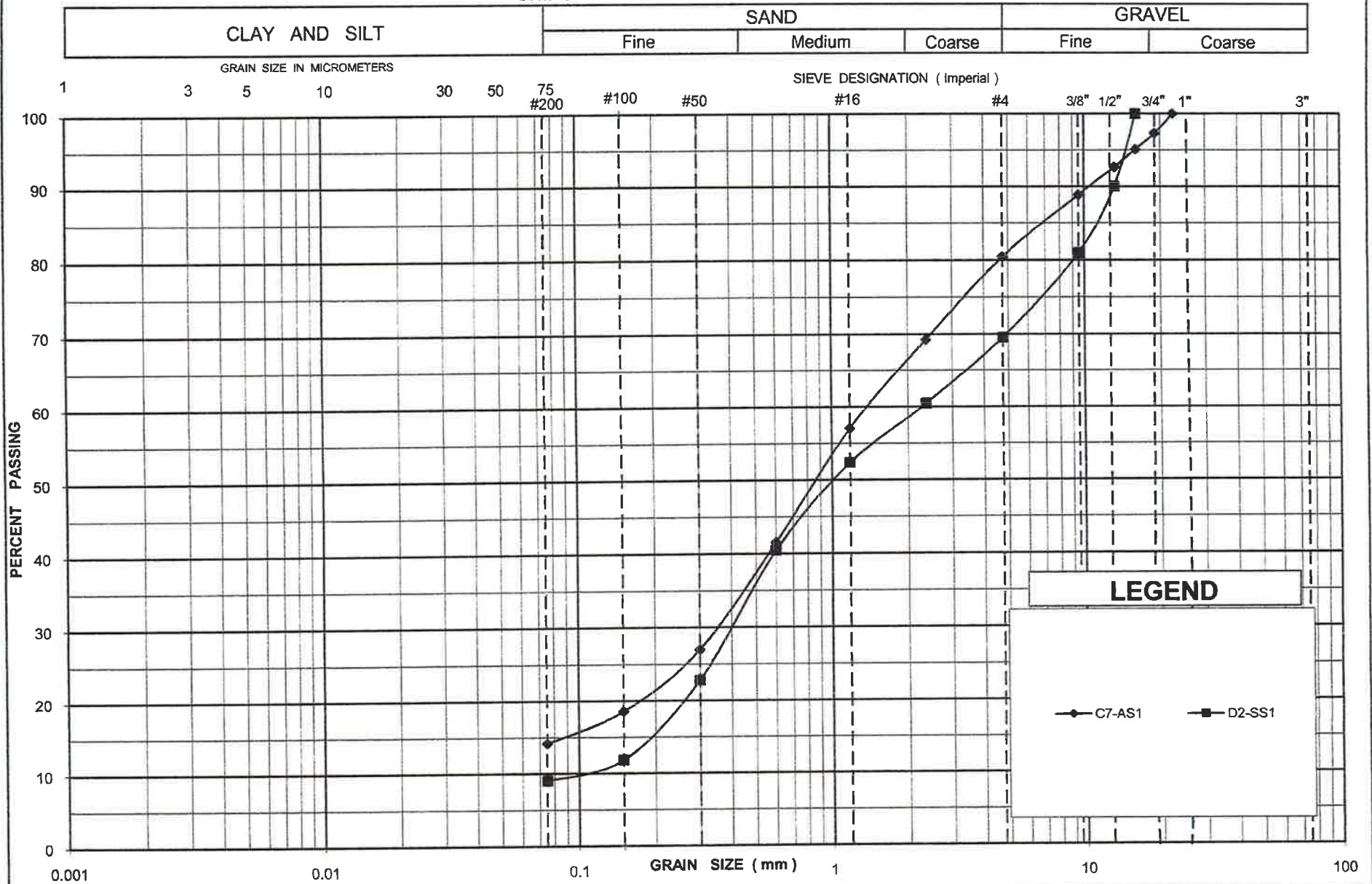


## Appendix B2

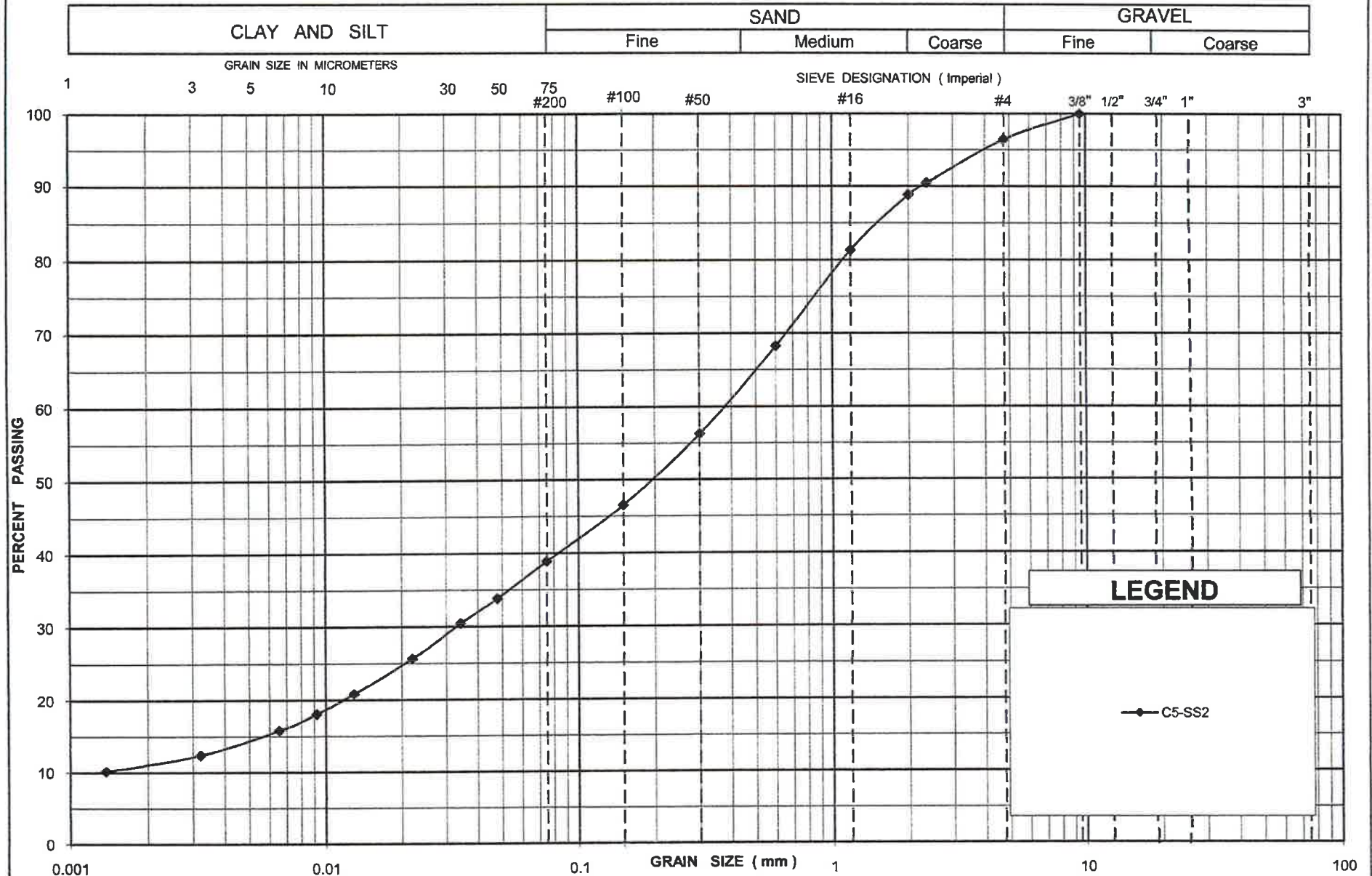
**Laboratory Test Results – C19**

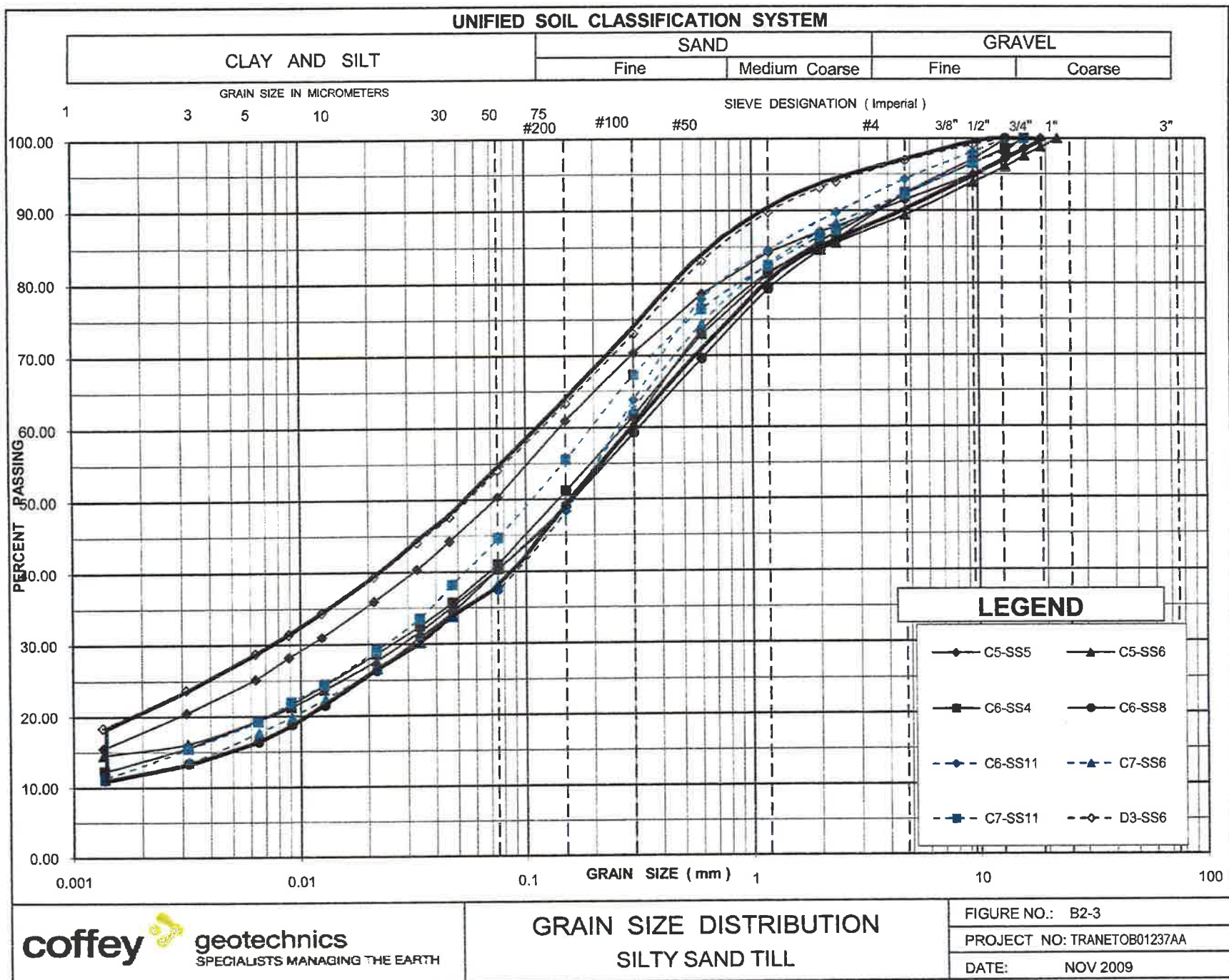


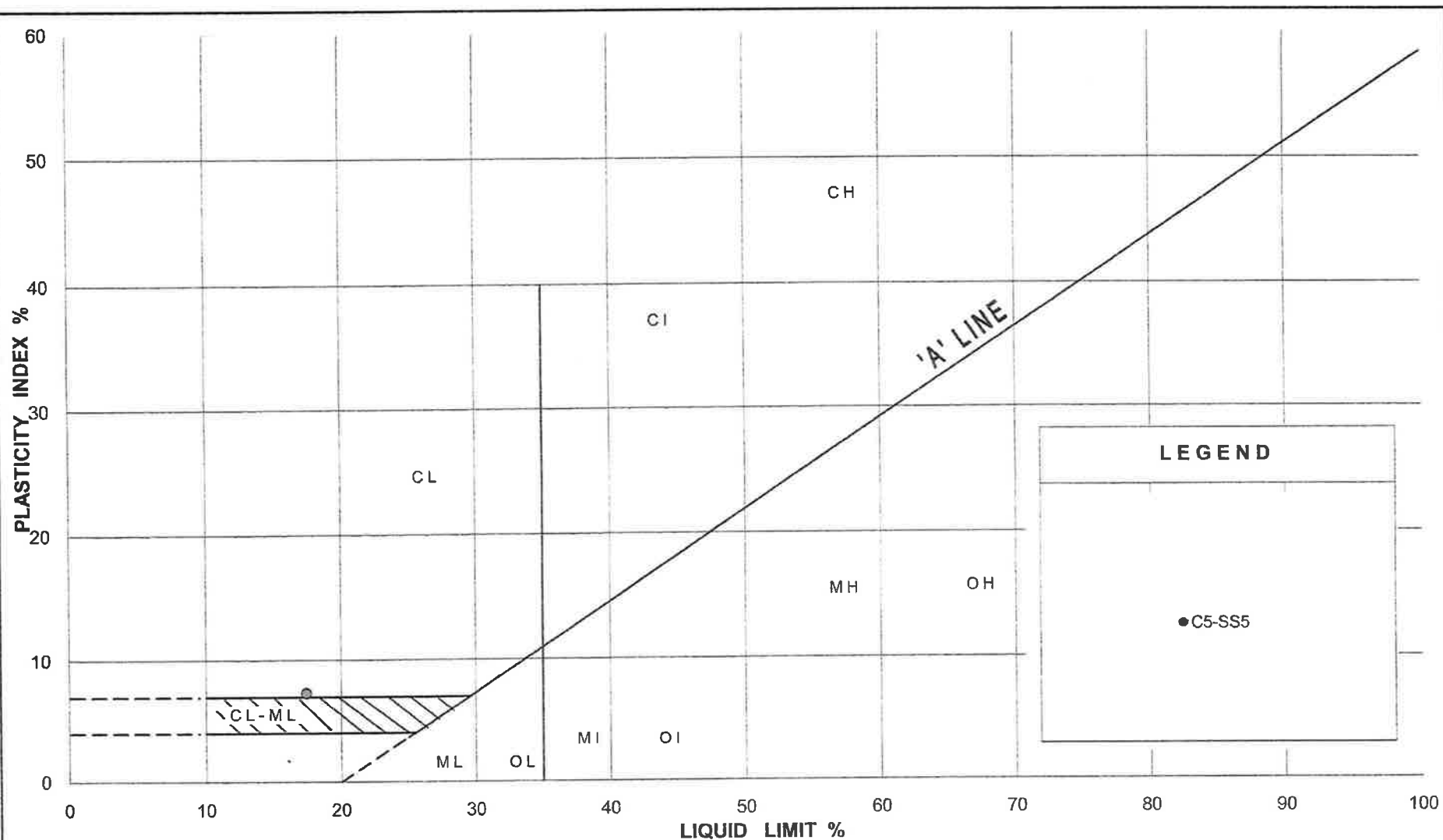
# UNIFIED SOIL CLASSIFICATION SYSTEM



# UNIFIED SOIL CLASSIFICATION SYSTEM

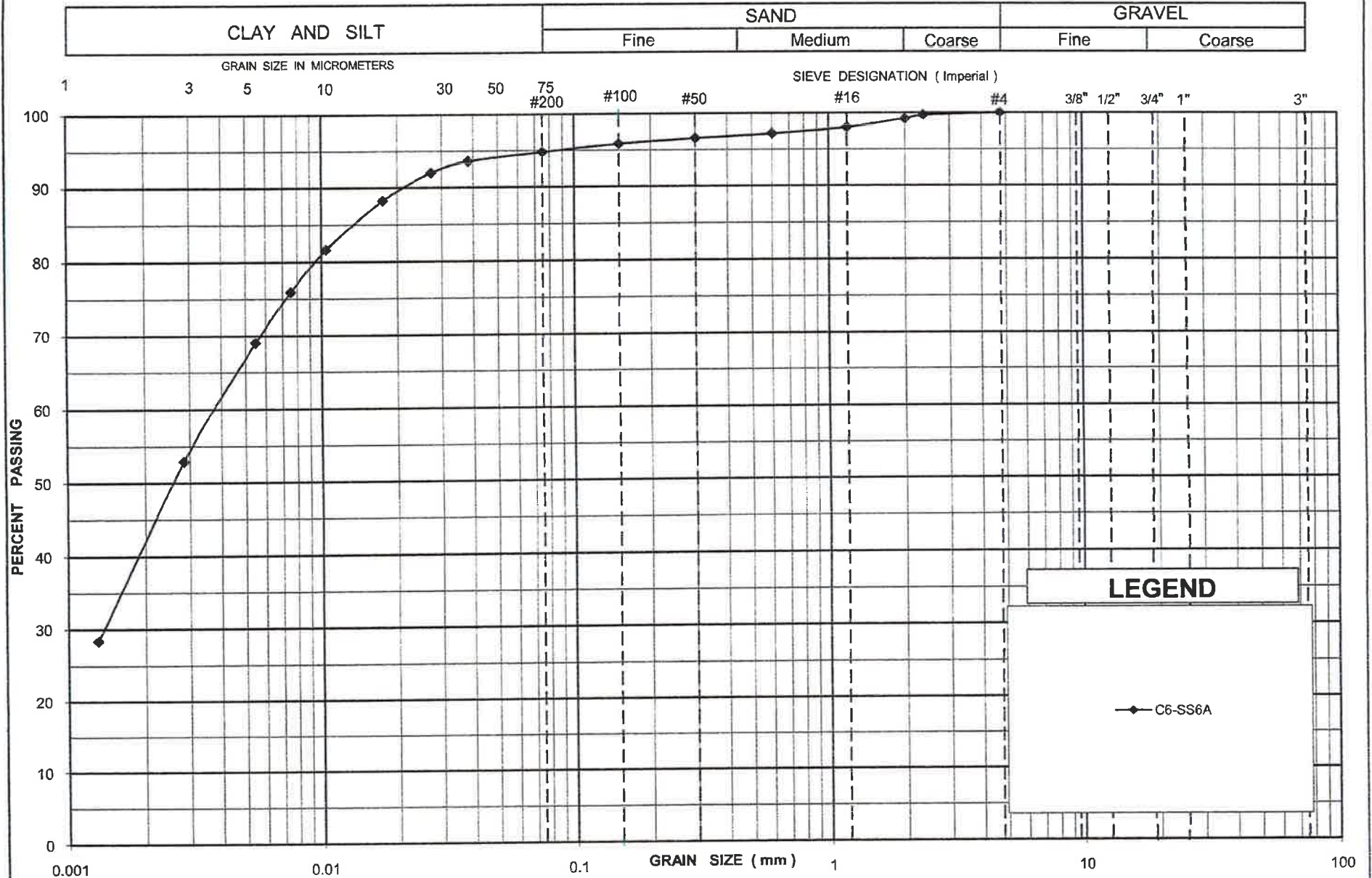


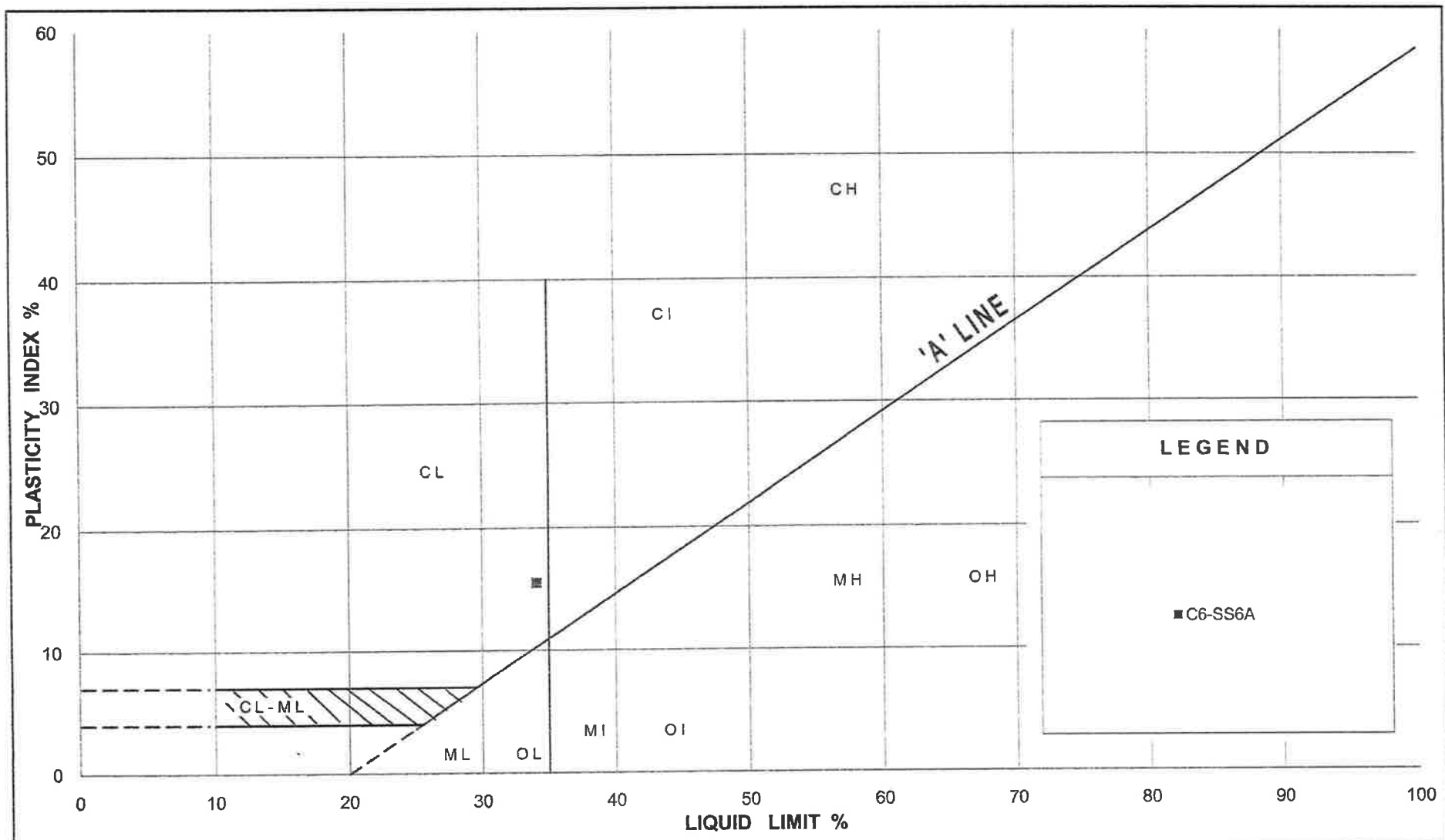






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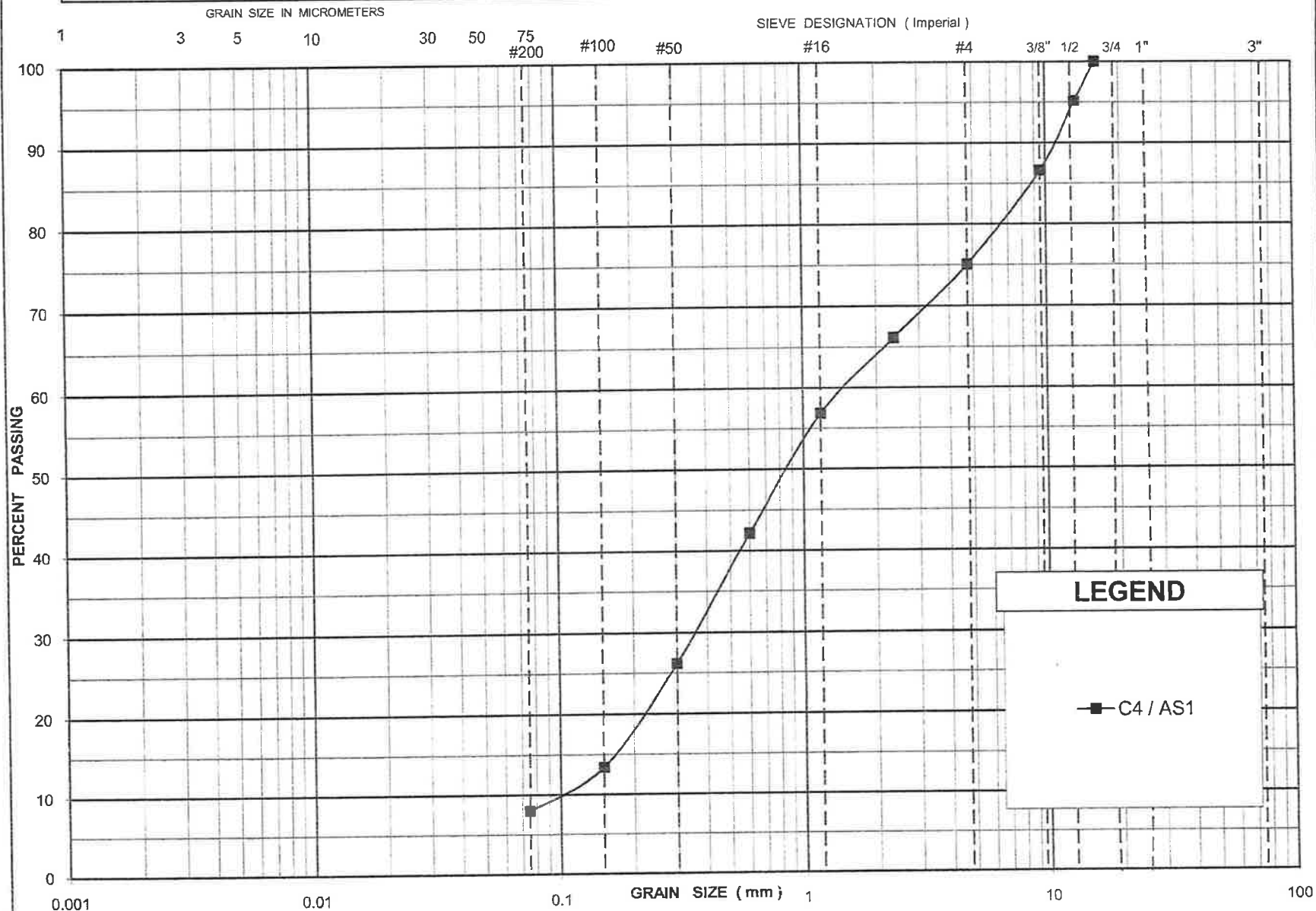


# Appendix B3

**Laboratory Test Results – C20**

# UNIFIED SOIL CLASSIFICATION SYSTEM

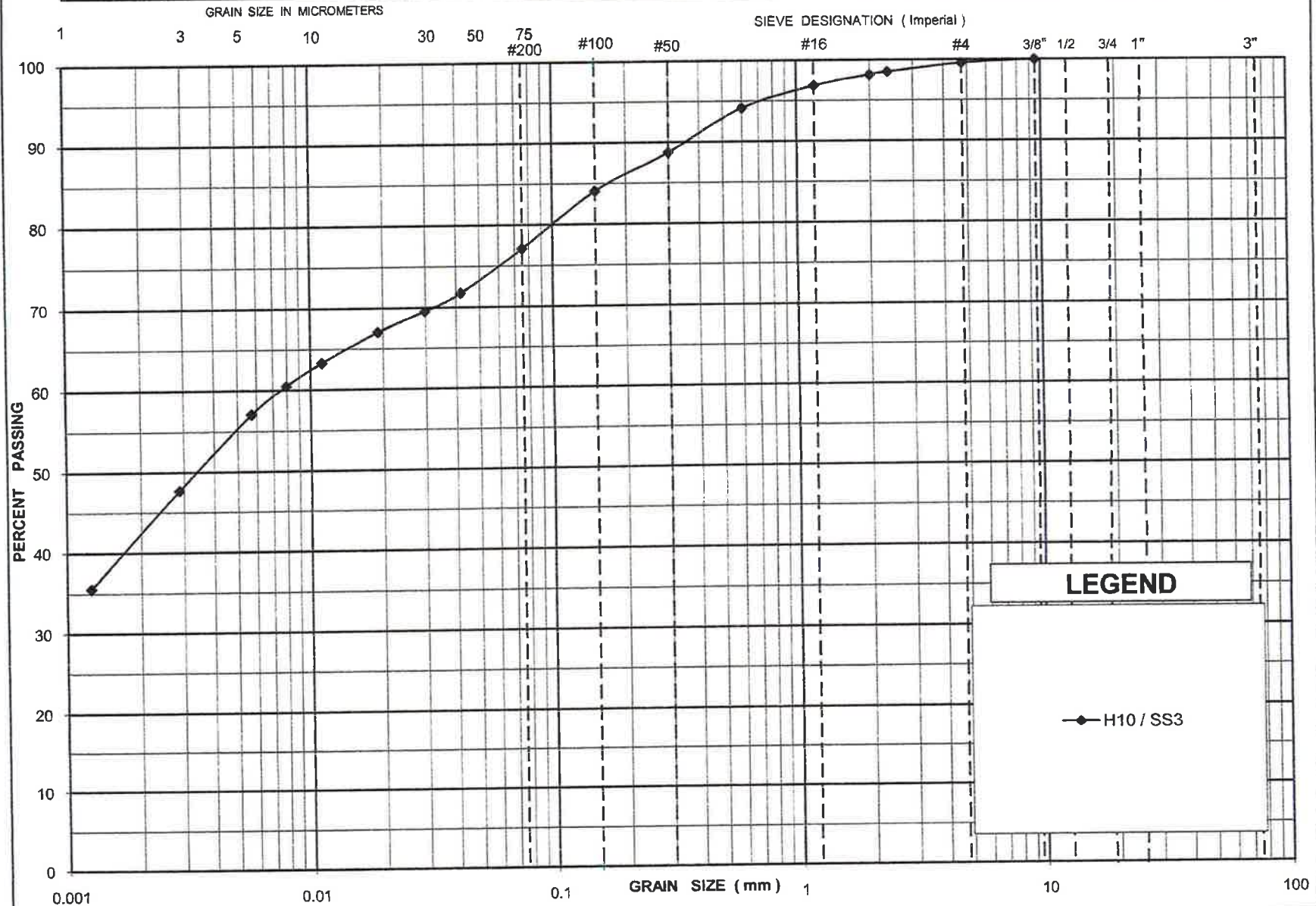
CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

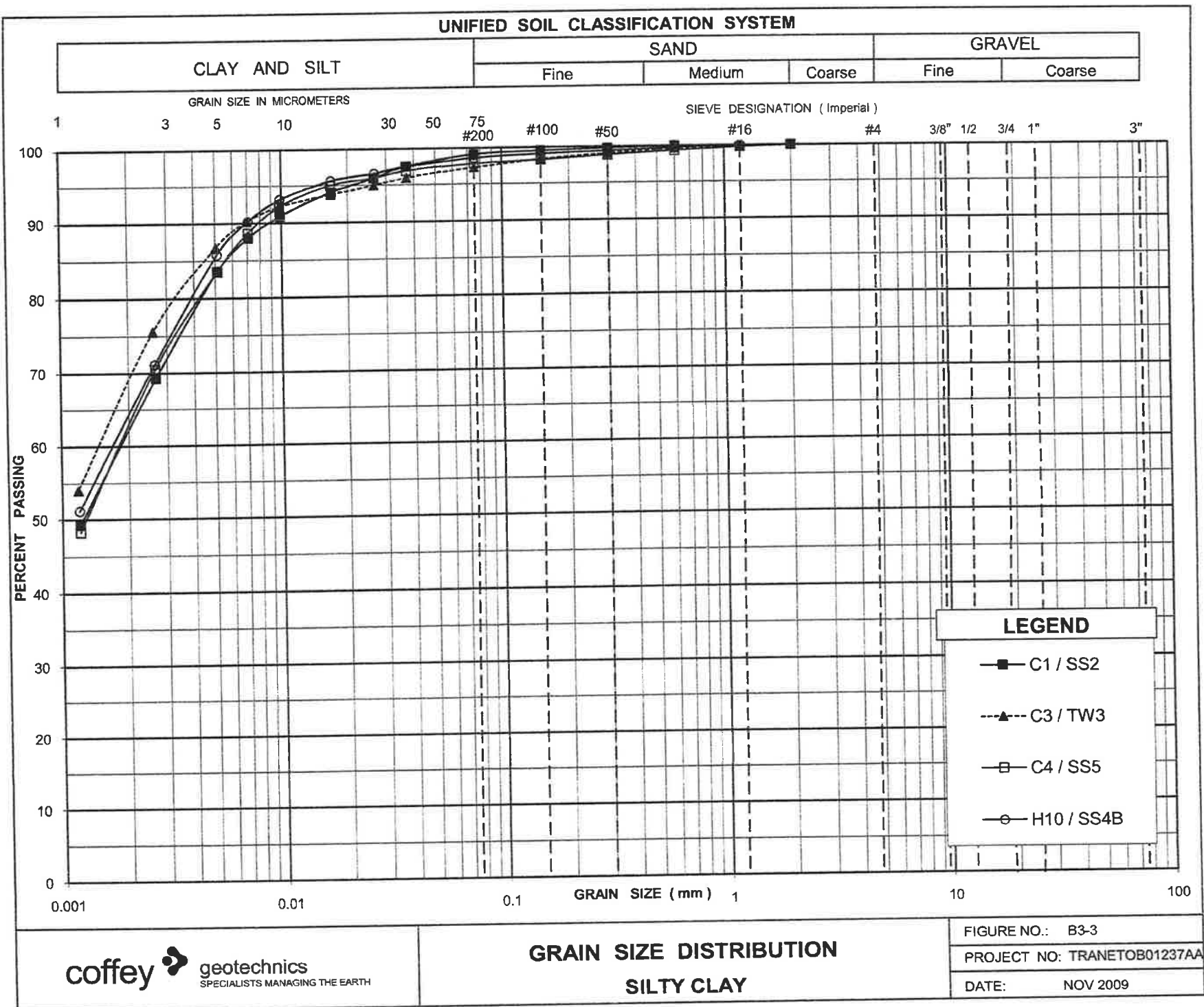


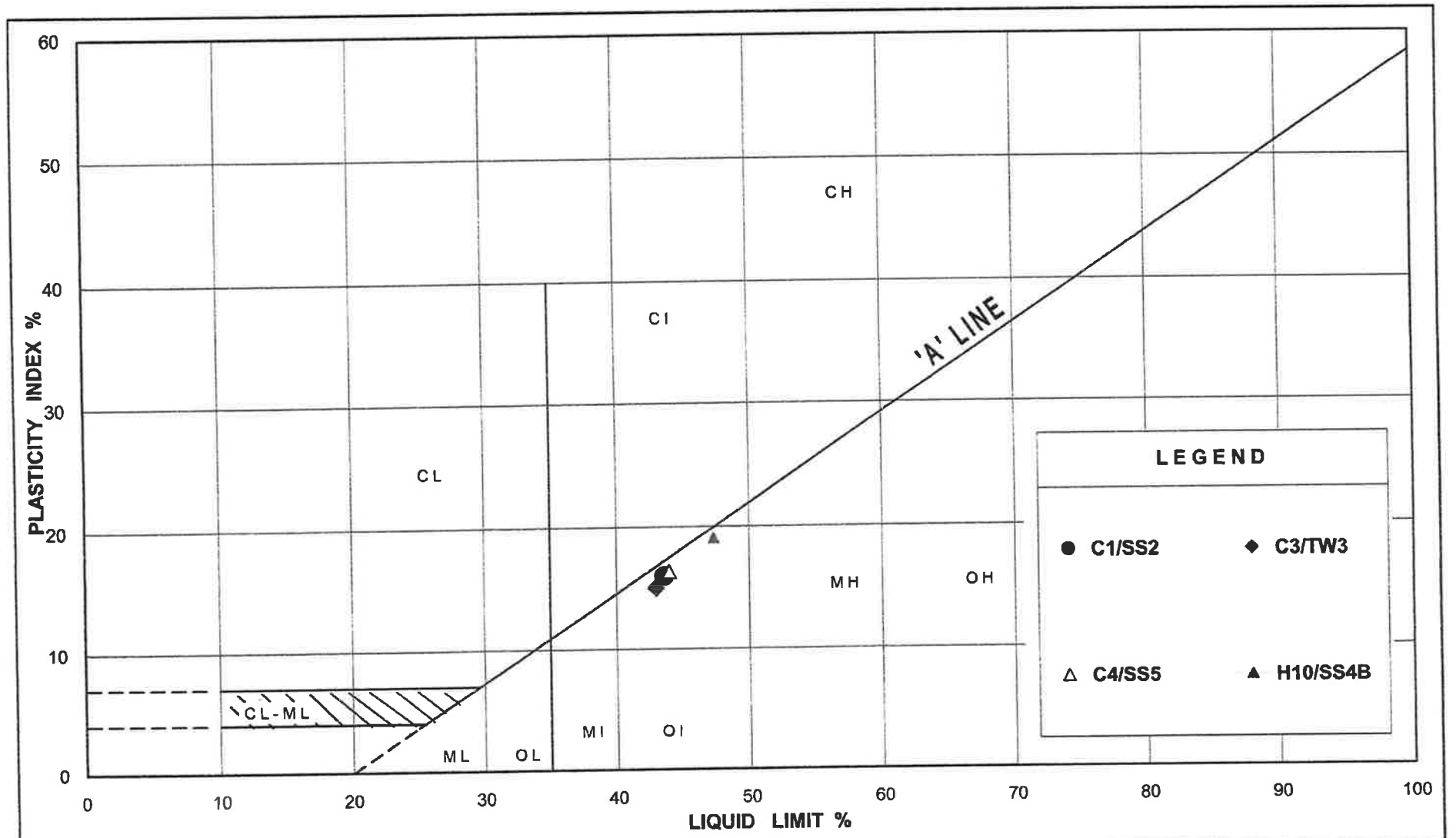


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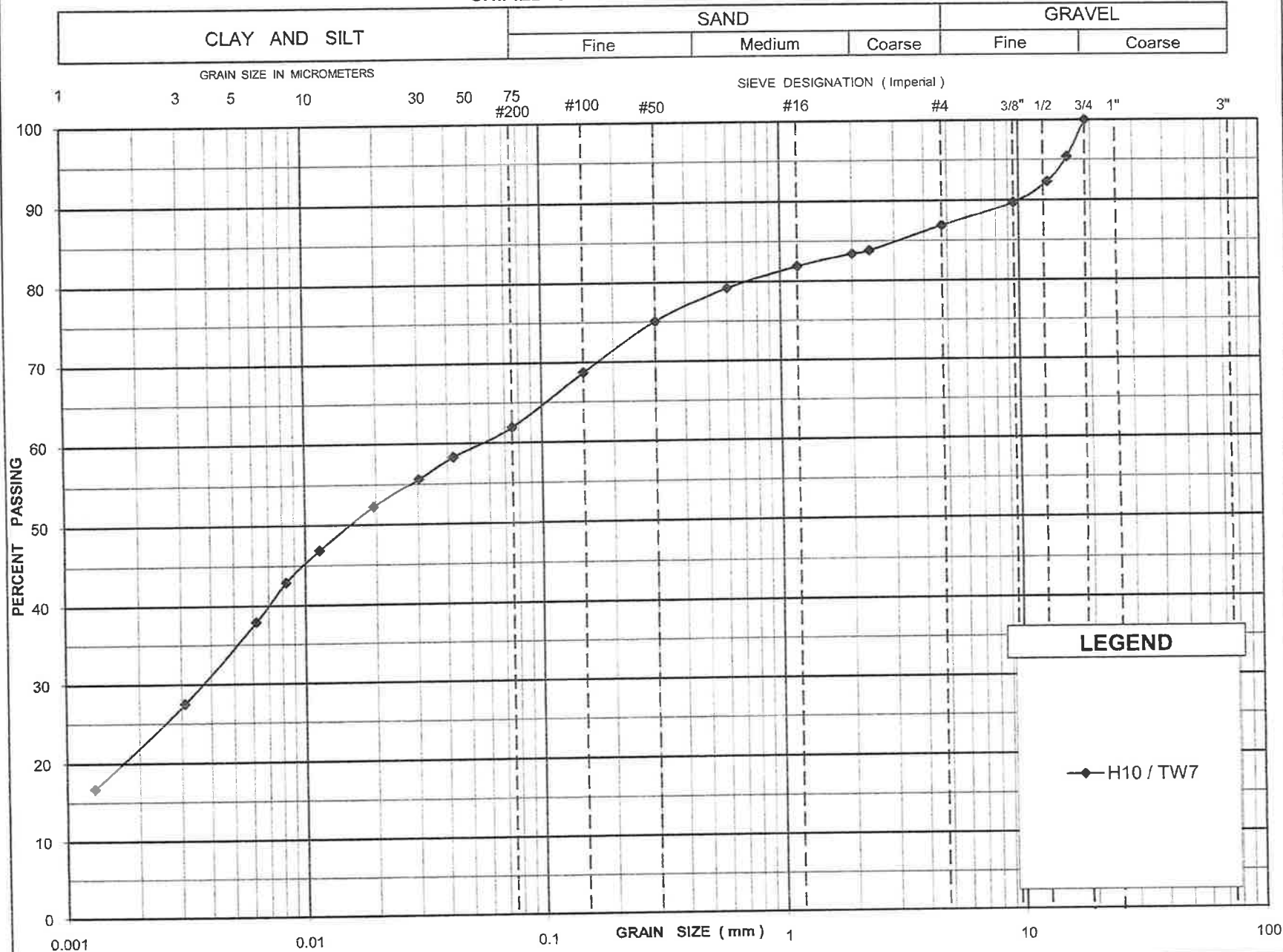
CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

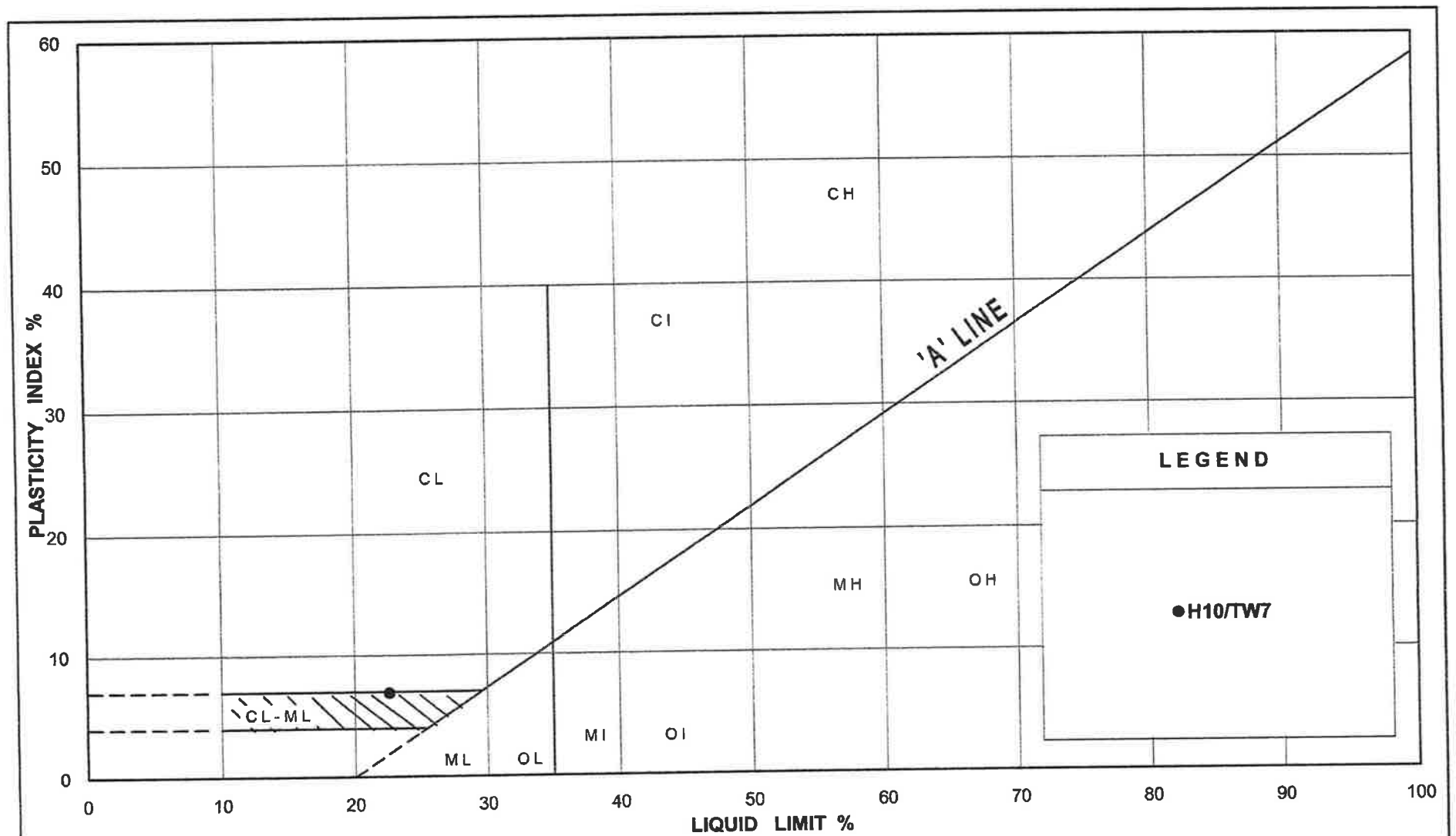






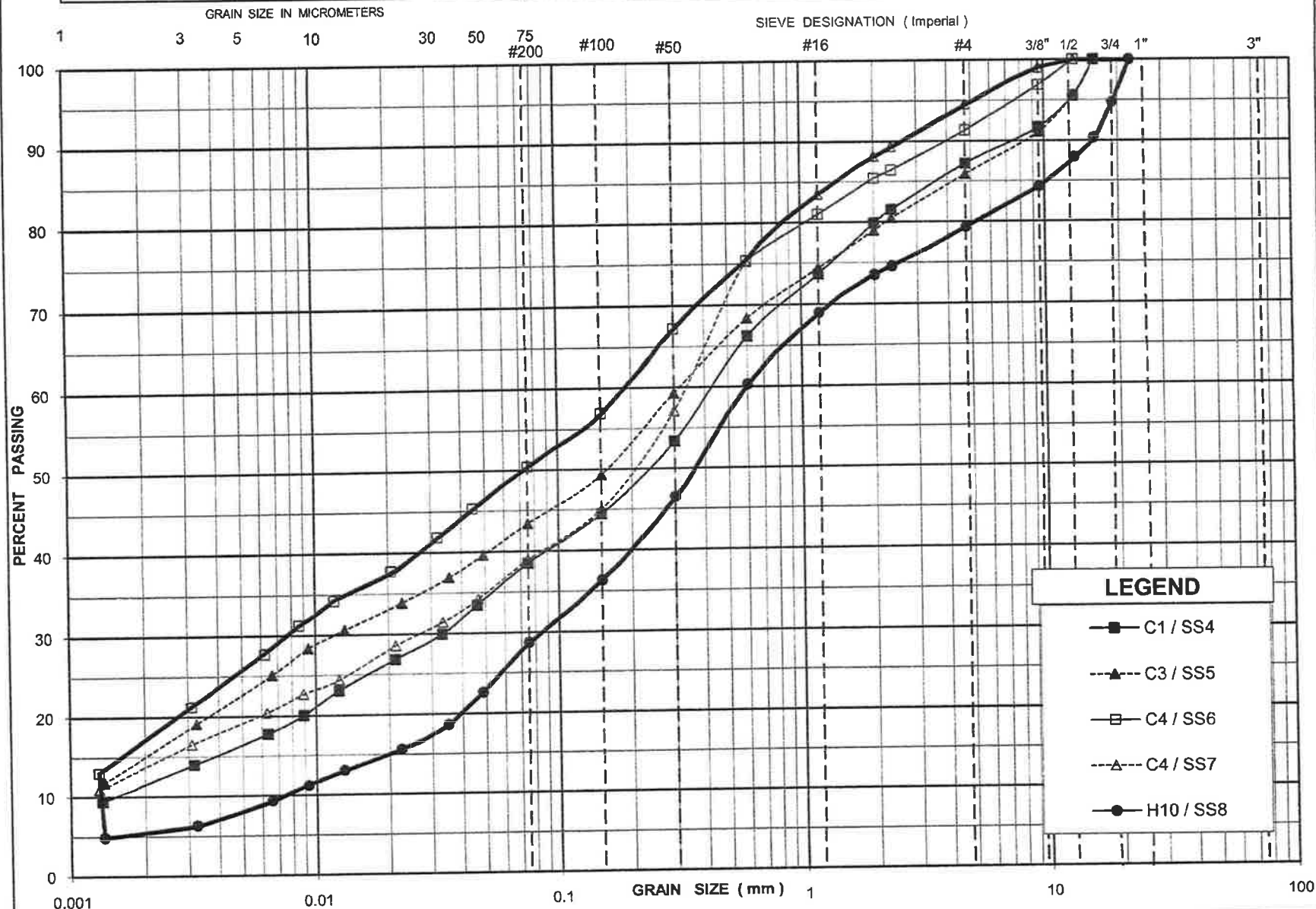
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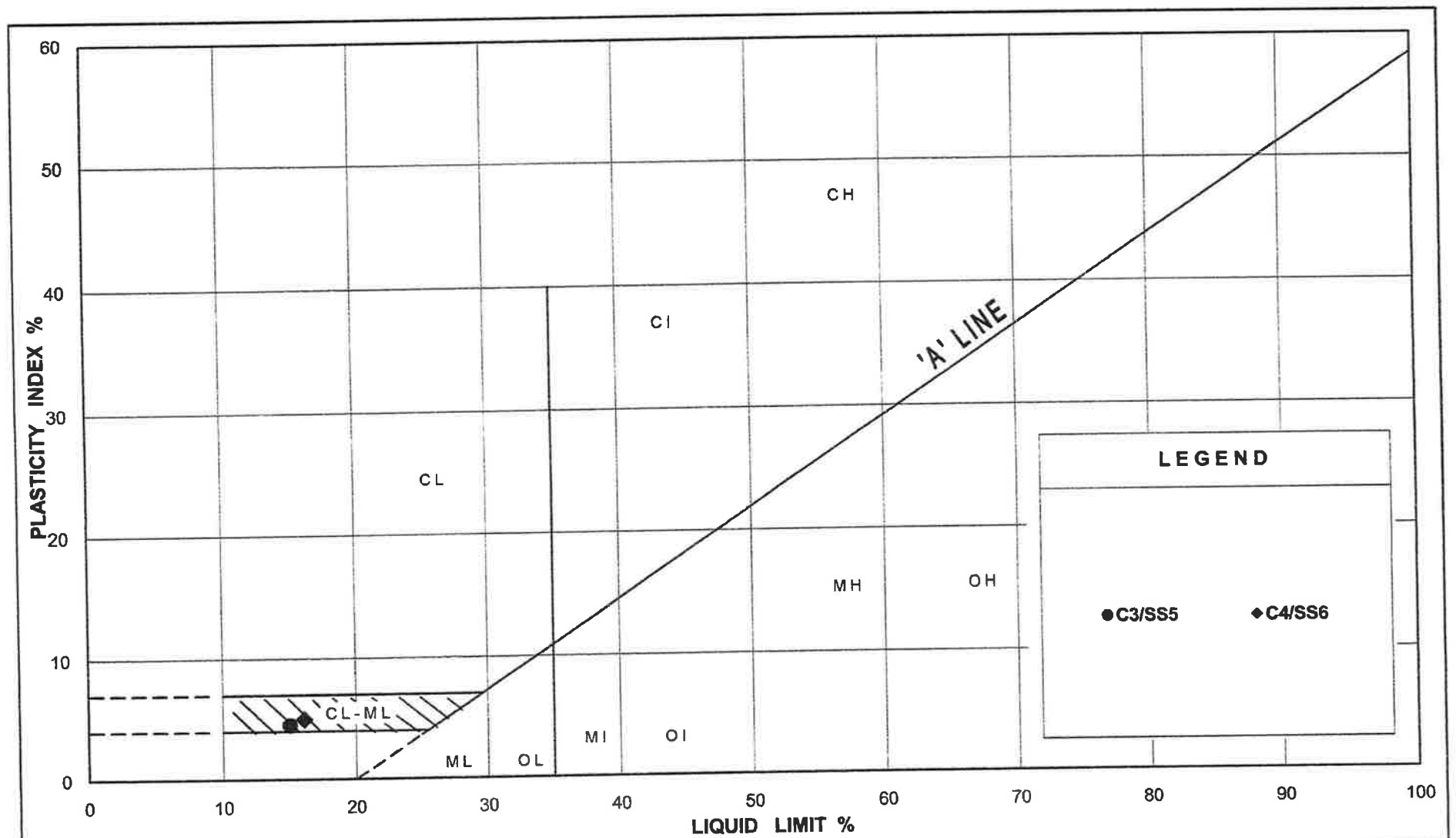




# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse





# Appendix C

## Rock Core Photographs





BH # C1 / Rock Core RC6, Depth (3.27 – 6.32) m



BH # C3 / Rock Core RC7, Depth (3.93 – 7.11) m





BH # C5 / Rock Core RC9 (6.1-7.0) m, RC12 (8.38-9.14) m, RC14 (9.32-9.91) m and Partial RC16 (10.06-11.08) m Depth



BH # C5 / Rock Core Partial RC16 (11.08-11.58) m, RC17 (11.58-13.11) m Depth





BH # C6 / Rock Core RC13, Depth (9.30 – 10.82) m



BH # C6 / Rock Core RC14, Depth (10.82 – 12.34) m





BH # C9 / Rock Core RC8 and RC9, Depth (4.9-7.8) m

# Appendix D

## Site Photographs





Culvert C16: Looking at East End of Culvert



Culvert C16: Looking at East End of Culvert





Culvert C19: at Station 15+328



Culvert C19: Looking at West End of Culvert





Culvert C19: Looking at East End of Culvert



Culvert C19: Looking at East End of Culvert





Culvert C20: Picture along pavement surface at 16+267 Station



Culvert C20: Northwest corner of Intersection of Highway 11 with Poupore Road





Culvert C20: Looking at West End of Culvert



Culvert C20: Looking at East End of Culvert

# 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 STRUCTURAL 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
TV	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

### MECHANICAL PROPERTIES OF SOIL

$m_v$	$\text{kPa}^{-1}$	COEFFICIENT OF VOLUME CHANGE
$c_c$	1	COMPRESSION INDEX
$c_s$	1	SWELLING INDEX
$c_a$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	$\text{m}^2/\text{s}$	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
$S_r$	1	SENSITIVITY = $c_u / \tau_r$

## PHYSICAL PROPERTIES OF SOIL

$P_s$	$\text{kg}/\text{m}^3$	DENSITY OF SOLID PARTICLES	$e$	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOLID PARTICLES	$n$	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$P_w$	$\text{kg}/\text{m}^3$	DENSITY OF WATER	W	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF WATER	$s_r$	%	DEGREE OF SATURATION	$D_n$	mm	N PERCENT – DIAMETER
$P$	$\text{kg}/\text{m}^3$	DENSITY OF SOIL	$W_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma'$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOIL	$W_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$P_d$	$\text{kg}/\text{m}^3$	DENSITY OF DRY SOIL	$W_s$	%	SHRINKAGE LIMIT	q	$\text{m}^3/\text{s}$	RATE OF DISCHARGE
$\gamma_d$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $(W_L - W_p)$	v	$\text{m}/\text{s}$	DISCHARGE VELOCITY
$P_{sat}$	$\text{kg}/\text{m}^3$	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $(W_L - W) / I_p$	k	$\text{m}/\text{s}$	HYDRAULIC CONDUCTIVITY
$P'$	$\text{kg}/\text{m}^3$	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	$\text{kN}/\text{m}^2$	SEEPAGE FORCE
$\gamma'$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT  
PROPOSED CULVERT  
REPLACEMENT/EXTENSION ON  
HIGHWAY 11, TOWNSHIP OF  
ARMSTRONG, ONTARIO  
G.W.P. 161-98-00, GEOCRES NO. 31M-82**

D.M. Willis Associates Limited

TRANETOB01237AA-AA  
June 04, 2010

# CONTENTS

<b>5</b>	<b>DISCUSSIONS AND RECOMMENDATIONS</b>	<b>17</b>
<b>5.1</b>	<b>Culvert C16 Extension</b>	<b>17</b>
<b>5.1.1</b>	<b>Culvert Foundations</b>	<b>18</b>
<b>5.1.2</b>	<b>Embankment Widening</b>	<b>20</b>
<b>5.2</b>	<b>Culvert C19 Replacement</b>	<b>21</b>
<b>5.2.1</b>	<b>Culvert Foundations</b>	<b>22</b>
<b>5.2.2</b>	<b>Embankment Widening</b>	<b>23</b>
<b>5.3</b>	<b>Culvert C20 Replacement</b>	<b>24</b>
<b>5.3.1</b>	<b>Culvert Foundations</b>	<b>25</b>
<b>5.3.2</b>	<b>Embankment Widening</b>	<b>26</b>
<b>5.4</b>	<b>Bedding</b>	<b>27</b>
<b>5.5</b>	<b>Backfilling</b>	<b>28</b>
<b>5.6</b>	<b>Construction</b>	<b>30</b>
<b>5.7</b>	<b>Erosion Protection</b>	<b>31</b>
<b>5.8</b>	<b>Frost Protection</b>	<b>32</b>
<b>6</b>	<b>CLOSURE</b>	<b>32</b>

## Tables

Table 8: Proposed Culvert Replacement/Extension

## Appendices

Appendix F: OPSD

Appendix G: Proposed Staged Construction

Appendix H: Culvert Options

Appendix I: Limitations of Report



**FOUNDATION DESIGN REPORT**  
**PROPOSED CULVERT REPLACEMENT/EXTENSION**  
**HIGHWAY 11, TOWNSHIP OF ARMSTRONG, ONTARIO**  
**G.W.P. 161-98-00, GEOCREs No. 31M-82**

## 5 DISCUSSIONS AND RECOMMENDATIONS

As part of the rehabilitation and resurfacing of Highway 11, two culverts will be replaced and one culvert will be extended. During the replacement and extension of the culverts, additional new lanes (either south bound (SB) or north bound (NB) lane) will be constructed at the culvert locations. The following is a summary of the proposed works.

**Table 8: Proposed Culvert Replacement/Extension**

<b>Culvert Number</b>	<b>Station</b>	<b>Proposed Work</b>	<b>Existing Culvert</b>	<b>Proposed Culvert</b>
C16	13+694	Culvert extension on the NB lane	NRFO 900 mm x 600 mm, 24.6 m length, with culvert invert elevation at 255.7 m to 256.1 m	Extension 900 mm x 600 mm, 5.0 m length, with culvert invert elevation at 256.1 m to 256.2 m
C19	15+328	Culvert replacement	Corrugated steel pipe (CSP) 750 mm diameter, 29.1 m length, with culvert invert elevation at 258.1 m to 259.5 m	Replacement 800 mm diameter, 38 m length, with culvert invert elevation at 257.9 m to 259.5 m
C20	16+268	Culvert replacement	Corrugated steel pipe (CSP) 600 mm diameter, 26.3 m length, with culvert invert elevation at 252.0 m to 252.1 m	Replacement 1200 mm diameter, 33 m length, with culvert invert elevation at 251.9 m to 252.0 m

The proposed replacement and extension of the culverts will be carried out using staged construction. Based on information provided, no highway embankment raises will be carried out as part of this work. However, embankment widening will be carried out to accommodate the new NB/SB lanes. The following sections provide discussions and recommendations for each culvert and general design recommendations.

### 5.1 Culvert C16 Extension

The existing concrete non-rigid frame open (NRFO) culvert at Station 13+694 is a 0.9 m wide x 0.6 m high x 24.6 m long structure. We understand that the existing NRFO culvert is to be extended by 5 m (total length 29.6 m) to facilitate the widening of the NB lane of the highway and the construction of the north bound passing lane. We understand that the extension of the culvert structure may be an open bottom concrete structure or a box culvert with the invert matching that of the existing culvert and rising slightly to

the new inlet location (Elevations 256.1 m to 256.2 m). An extension using corrugated steel pipe (CSP) is also being considered. CSP extension shall be in accordance with OPSD 800.011.

We further understand that the construction of the new NB lane will likely coincide with the construction of the culvert extension and that a detour lane will not be necessary. However, placement of fill for the proposed embankment widening will not be carried out until completion of the culvert extension. The proposed finished embankment side slope has been planned as 3 Horizontal:1 Vertical (3H:1V).

Boreholes C8 and C9 and DCPT A4 were advanced in proximity to the existing and the proposed alignment of Culvert C16. In general, the encountered subsurface conditions at Culvert C16 location consists of approximately 0.9 to 2.1 m of embankment fill, over natural soils which are in turn underlain by proven or inferred bedrock. The embankment fill at Borehole C8 included a 0.5 m thick granular fill layer that was part of the road pavement section. The top of the natural soils was encountered at Elevations 255.7 m to 256.6 m. The natural soils consisted of a 1.3 m thick silty clay layer (Borehole C8 only – not encountered in Borehole C9), underlain by silty sand till. In the boreholes, auger and/or DCPT refusal was encountered at Elevations 251.7 m to 252.8 m. As previously mentioned, Borehole C9 was advanced beyond auger refusal (Elevation 251.7 m), in an adjacent borehole, by bedrock coring techniques.

Based on observations made in the open boreholes, the results of the piezometer readings in Borehole C9, and the change of the colour of the soil from brown to grey, at the time of our field program, the groundwater table at the site was at about Elevation 255 m to 253 m. The surface water was at the invert of the culvert at the time our subsurface exploration program was recorded at approximately Elevation 256 m.

Note that groundwater and surface water levels are subject to variation due to the influence of rainfall, seasons and other factors. There may also be potential for development of a perched groundwater table following periods of rainfall and groundwater may rise to the ground surface.

### **5.1.1 Culvert Foundations**

We understand that there will be no grade raise of the existing embankment and the extension of the existing NRFO will consist of a matching NRFO or a concrete box culvert or a CSP. The undisturbed till deposit, encountered at the site, is considered suitable to support the foundations for NRFO, box culvert, or CSP type extensions. It is our opinion that with the prevailing subsurface conditions, a closed bottom (box) culvert would be more suitable than an open bottom culvert. The following geotechnical resistances can be used for design purposes, for the undisturbed till deposit at or below Elevation 255.3 m.

Factored Geotechnical Resistance at ULS = 120 kPa

Geotechnical Resistance at SLS = 80 kPa

When recommending the ULS value, a minimum footing width of 0.8 m was assumed.

Higher resistances would be available at or below Elevation 253.0 m but are considered to be unnecessary for this project. It should be noted that foundation bearing soils near the water table are susceptible to disturbance from construction activity. Care should be taken during the excavation and construction of the footings to minimize disturbance of the bearing soils. Stabilization of wet subgrades should be anticipated. Disturbance of the underlying soils during construction of the structure, in proximity to the groundwater



table, could influence the settlement of the structure. In this regard, consideration should be given to placing a 100 mm thick layer of lean concrete (mud mat) on foundation bearing surface supporting an open bottom concrete culvert structure, as soon as possible after the excavation and the approval of the subgrade.

For NRFO or box culvert extension, footings should be founded at the same level or stepped so that the new footings do not influence the adjacent (existing) footings. For CSP extension, we recommend that the bedding should extend to the base of the existing footing but not below so as not to undermine the existing structure. Bedding thickness of the CSP extension can be reduced/tapered to the minimum bedding thickness specified in Section 5.4 at a distance of at least 3 m from the existing culvert and the CSP extension interface (i.e. bedding thickness at the interface = depth between the base of CSP extension and existing culvert footing and bedding thickness at 3 m from the interface = 200 mm). Care should be exercised during construction of foundations adjacent to the existing culvert to avoid possible undermining and/or influence on the existing structure.

Provided that the bearing subgrade is not unduly disturbed during the construction, with the recommended serviceability resistance value, the total and differential settlements should not exceed 25 mm and 20 mm, respectively. As will be discussed later in this report, good construction techniques, including dewatering, will be required to achieve this. As the total settlements will translate into differential settlements between the existing and the extension, allowance should be made to accommodate a differential settlement of about 25 mm between the existing and the new culvert.

A Geotechnical Engineer who is familiar with the findings of this investigation should evaluate all bearing surfaces prior to placement of reinforcement and concrete to confirm that the founding conditions are consistent with the recommendations given in this report. All organic, very loose/soft/firm or otherwise unsuitable soils should be removed prior to pouring the concrete or placing the bedding materials. Where unsuitable bearing conditions are observed, remedial procedures can be established in the field to avoid construction delays.

Cobbles and boulders within the till may be encountered during construction. Where oversized materials are encountered to protrude from the exposed subgrade, these materials should be removed and replaced with compacted structural fill or mass concrete.

Frost and scour depths need to be considered when choosing the footing elevations.

The unfactored horizontal resistance against sliding between the poured concrete and the approved undisturbed till deposit surfaces can be calculated using a friction angle of  $28^\circ$ . Sliding is however unlikely to present a problem for this project.

To provide sufficiently dry conditions during the extension of the culvert, it will be necessary to divert the water flowing in the watercourse. This could consist of the construction of a temporary cofferdam, such as pre-cast concrete barrier (e.g. jersey barrier), low permeability soil cofferdam barrier, sand bags, etc. to divert the water away from the culvert extension, or the construction of a temporary culvert, although the latter would be impractical and not cost-effective. In the case of diversion, consideration should be given to temporary diversion and storage of the surface water and promptly pumping the water downstream of the new construction area, into the existing watercourse channel near the outlet. In this regard, measures

would need to be taken within the existing culvert to prevent back flow of water in to the construction area and installation of suitable sedimentation control.

In addition to diversion of surface water, and the depending on the site conditions at the time of construction, dewatering will be necessary. This is likely to consist of gravity drainage in shallow perimeter ditches and pumping from strategically placed deep filtered sumps. Depending on the conditions at the site at the time of the construction, and the depth of excavation required, dewatering by means of vacuum well pointing may also be necessary. When designing the well pointing (or deep wells), the presence of boulders and the position of the bedrock below about Elevation 252 m should be taken into consideration. For this reason, we recommend that, in order to reduce the severity of dewatering, the construction be carried out during a dry period, if possible. It is, however, normally up to the Contractor to come up with a plan to achieve a suitable diversion and dewatering (if necessary). We recommend that the requirement for dewatering be 'red-flagged' to the Contractor and that the Contractor be asked to submit their diversion and dewatering method to the CA for information purposes prior to construction.

We also recommend that NSSP be included alerting the Contractor of the subsurface and groundwater conditions which may cause the disturbance of the new and the existing foundations during the construction as well as the fact that existing footing shall not be undermined.

### **5.1.2 Embankment Widening**

As shown on Drawing 2B, the existing embankment will be widened to accommodate a passing lane and the slopes of the new embankment will be flattened to 3H:1V configuration. The crest of the widened embankment will match that of the existing embankment.

Based on the borehole results, no foundation failures are anticipated due to the proposed widening, assuming that all organic, weak or otherwise unsuitable materials will be removed as per MTO standards prior to placing the embankment fills.

All organic and other unsuitable soils should be removed within an envelope and given by an imaginary slope not steeper than 1:1 from the toe of the proposed embankment. After stripping, the exposed subgrade should be inspected, approved and properly rolled from the surface, using a suitably heavy compactor. The existing site conditions (e.g. high water table) could influence the choice of compaction equipment. Where wet conditions prevail, dewatering will be needed in order to achieve proper compaction and the first lift of the fill may need to consist of free-draining granular materials. The dewatering can consist of gravity drainage and pumping from perimeter ditches and from strategically placed sumps.

Proper benching of the embankment slope should be implemented during widening of the embankment, as per MTO procedures and in accordance with OPSD 208.010.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill. Fill used for the construction of the embankment should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of OPSS 501 and OPSS 206. In general, the fills should be placed in lifts not exceeding 300 mm before compaction and each fill should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. In as much as possible, the fill used should match the existing embankment fill, within the frost zone.

It is anticipated that the proposed widening will impose a maximum grade raise of about 2 m (see Drawing 2B). With the stresses imposed by this configuration, settlements should not exceed 25mm. A settlement of this magnitude is considered acceptable and should necessitate neither surcharging nor preloading.

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

## 5.2 Culvert C19 Replacement

The proposed Culvert C19 replacement is to consist of an 800 mm diameter rigid or flexible pipe culvert, to replace the existing 750mm diameter CSP, and will be extended to about 9 m (total length of 38 m) on the SB side of the highway. The proposed culvert replacement invert levels will be similar to the existing culvert invert levels which are currently at Elevations 257.9 m to 259.5 m. The extended length of the proposed 800 mm diameter culvert will facilitate the construction of the new SB passing lane. The following are the options considered by the client for culvert types:

- 825 mm Reinforced concrete, 50-D, Class B bedding, no restriction to wall type
- 900 mm Reinforced concrete, 50-D, Class B bedding, no restriction to wall type
- 900 mm HDPE, Class 210, open profile wall – smooth inside and corrugated outside
- 850 mm HDPE, RSC 160, closed profile wall – smooth inside and outside
- 900 mm PVC, Class 210, solid wall – DR smooth inside.

There will be no grade raise over and above the existing pavement elevation of Highway 11. We also understand that the replacement of the culvert and the construction of the new SB passing lane will be carried out using a staged construction where half of the highway embankment will be excavated followed by the removal and replacement of the culvert on the same side, while the traffic is maintained on the other side. Upon removal and replacement of the culvert on one side, the highway embankment will be constructed to about 1.5 m below the original grade and the traffic will be diverted to the newly constructed, lower embankment on the completed side. At Stage 2, the embankment and the existing pipe will be removed, and the remaining half of the culvert will be installed, followed by the construction of the road embankment to the original grade. Following this, traffic will be diverted to the newly completed side and the height of the embankment on the other side will be raised to the original grade. In this way, the finished highway embankment will not be raised but widened to accommodate the new SB lane. A slope of 3H:1V will be used for the finished embankment side slopes. Appendix G presents the proposed staged construction prepared by the client.

Boreholes C5, C6 and C7 were drilled in the vicinity of Culvert C19. In addition, two other boreholes (D2 and D3) were advanced along the south western side of the existing highway on the proposed detour embankment/new SB passing lane. The boreholes indicate that at the culvert location (i.e. Borehole C5, C6 and C7) the site is underlain by embankment fill over silty sand till, encountered at Elevations 258.2 m to 259.4 m and the silty sand till is in turn underlain by bedrock (Elevations 249.1 m to 250.8 m).

The embankment fill included a 0.8 m thick granular fill over earth fill, up to 2.7 m thick. It is described as very loose to loose. The underlying silty sand till contains some cobbles and boulders. Based on drilling resistance and grinding of augers in the boreholes, SPT 'N'-values recorded within the till varied from 1 to in excess of 100 blows/0.3 m. The high 'N'-values were likely due to the presence of cobbles and boulders. Based on these results, the relative density of the till deposit is generally loose to compact with occasional very loose and very dense zones. The groundwater level at the time of our investigation is believed to be at Elevation 256.5 – 257.0 m.

### 5.2.1 Culvert Foundations

Based on the borehole data, below the proposed culvert invert elevations (i.e. Elevation 259.5 m at the inlet and Elevation 257.9 m at the outlet), silty sand till will be encountered at subgrade level at most locations. At these levels, the till deposit was encountered to be in a loose to compact condition with some very loose zones.

In its undisturbed state, the following geotechnical resistances can be assigned to the silty sand till for design purposes.

Factored Geotechnical Resistance at ULS = 140 kPa

Geotechnical Resistance at SLS = 70 kPa

A minimum foundation width of 0.9 m was assumed when calculating the ULS value. The depth of the strip footing foundations should be chosen with scour consideration in mind. Since there will be no grade raise, theoretically there should be no settlements under the existing abutment (i.e. there will be no additional loading). In practice however, there will be some settlements due to the rebound after excavating the soil for construction and subsequent settlement when loads are re-applied. An allowance of 15 mm should be made for this. Good construction techniques, including dewatering and water diversion, as discussed below, will be also required to achieve this. Otherwise excessive settlements could occur.

Beyond the existing embankment portion, under the widened sections, settlements due to stresses imposed by the widening can be expected to take place. Typical widening loading is expected to be due to about 2 m fill and this can be expected to cause a settlement of the order of 40 mm. Depending on the staging scheme, some of this settlement can be expected to take place at an earlier stage. In this event, there will be no need for cambering. If the staging is unfavourable (i.e. if the widened section will not be loaded at a stage earlier than the rest of the embankment), then this will translate into differential settlements. This can be alleviated by providing about 25 mm cambering under the widened section.

Due to the weak nature of the till underlying the site, the use of a rigid frame culvert is not recommended at the site, for the reasons discussed above. For the prevailing subsurface conditions, a flexible type structure such as a CSP would be better suited. If the environmental conditions are aggressive for a CSP (i.e. corrosion) then the use of a precast concrete culvert can be considered. This type of culvert can withstand differential settlements and has the advantage of more rapid construction than a cast-in-place concrete structure and is usually favoured. An allowance of up to 40 mm differential settlement should be made between individual precast segments, which in our experience, does not present problems. But this aspect should be checked with the supplier.

A Geotechnical Engineer who is familiar with the findings of this investigation should evaluate all bearing surfaces prior to placement of bedding for the proposed culvert. Where unsuitable bearing conditions are observed, remedial procedures can be established in the field to avoid construction delays. Bedding recommendations are discussed in Section 5.4.

Cobbles and boulders may be encountered during the construction. Where oversized materials are encountered to protrude from the exposed subgrade, these materials should be removed and replaced with compacted structural fill or lean concrete.

The unfactored horizontal resistance against sliding between approved subgrade (granular till) and the bedding can be calculated using a friction angle of  $27^\circ$ . This value can also be used if a geotextile is utilized in conjunction with the bedding. Sliding is however unlikely to present a problem for this project.

To provide sufficiently dry conditions during construction of the culvert, it will be necessary to divert the water flowing in the watercourse. Surface water diversion measures are discussed in Section 5.1.1.

In addition to diversion of surface water, and the depending on the site conditions at the time of construction, some local dewatering will likely be necessary. Dewatering measures were also discussed in Section 5.1.1.

### **5.2.2 Embankment Widening**

Based on the information provided by the client, the existing Highway 11 embankment will be widened (about 9 m wide from the toe of the existing embankment) to accommodate a new SB passing lane at Culvert C19, as shown in Drawing 3B. The crest of the widened embankment will match that of the existing highway embankment elevation (i.e. about Elevation 262.4 m to 262.6 m) and can therefore be expected to be up to 4 m high over and above the original grades (o.g.), but typically 2 m high above the toe of the existing embankment. We understand that 3H:1V side slopes are proposed for the final configuration.

For site preparation, the following are recommended:

- Strip surface vegetation, topsoil, other organics and other unsuitable materials within an envelope given by an imaginary slope no steeper than 1H:1V from the toe of the proposed embankment as per MTO procedures.
- Proof-roll the exposed surface with a suitable compactor.
- If localized soft spots or excessive heave occurs during proof-rolling, further excavate and replace with suitable fill.

Topsoil thickness of up to 0.3 m can be used for preliminary estimating purposes of stripping depths. However, the boreholes also indicated that organic rich materials were intermixed with the fill material and as such the thickness and extent of organic rich material will likely vary in between and beyond borehole locations. As such stripping depths will likely vary and should be adjusted to remove all unsuitable materials. After stripping, the exposed surface should be inspected and approved.

Groundwater is unlikely to be encountered during stripping of unsuitable material. However, perched water conditions may occur, water seeping into the excavations can likely be controlled by gravity drainage, perimeter ditches and pumping from sumps.

Based on the drawings provided, temporary inner slopes of 1H:1V are proposed during the staged construction. 1H:1V slopes combined with traffic loads at the crest of the embankment may cause instability of the embankment. We recommend a flatter slope of 1.5H:1V be constructed and a minimum 1.5 m clearance should be maintained between the moving traffic and the edge of the slopes. These temporary slopes should be utilized for a short duration only, say maximum three days. We also recommend that these slopes be monitored continuously for any movement or impending instability during the construction. As well, a suitable speed limit should be imposed for the traffic. The proposed outer side slopes of 3H:1V are considered stable on short and long term basis.

Proper benching of the existing embankment slopes should be implemented during widening of the embankments, as per MTO procedures and in accordance with OPSS 208.010.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. select subgrade materials (SSM) or Granular 'B' Type I – OPSS 1010). In as much as possible, the fill used should match the existing embankment fill, within the frost zone. The embankment fill should be placed on the approved and properly rolled subgrade in lifts not exceeding 300 mm when loosely placed and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density.

The widening of the embankment will induce loadings of up to about 40 kPa to the natural foundation soils and to the existing embankment fills. Embankment loadings would likely result in a settlement of less than 40 mm. due to the settlement of natural foundation soils and of the existing embankment fills. In addition, the settlement of the new embankment fills under their own weight can be expected to occur. If the embankment is constructed to MTO standards, this should not exceed 20 mm. The foundation settlements should be substantially completed within a period of about one month while the settlement due to the own weight of the embankment will depend on the type of soil used to build the embankment (e.g. the settlement of granular soils will be relatively rapid while clayey soils will settle more slowly). Assuming an average SSM type soil, the settlement of the embankment under its own weight should also be substantially completed within about three months. As these settlements are not excessive, neither surcharging nor preloading is considered necessary for the embankment. We recommend however that the paving of the road be delayed by as much as practicable (i.e. by up to four weeks, if possible) to effect some of the anticipated settlements, especially on the widened side.

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

### **5.3 Culvert C20 Replacement**

Based on the drawings provided by the client, the existing 600 mm diameter CSP culvert will be replaced by a 1200 mm diameter culvert. The proposed 1200 mm diameter culvert is to be about 30.4 m in length which means that the existing culvert will be extended by about 4 m on the SB side of the highway. The proposed culvert replacement invert levels are to be similar to the existing culvert invert levels which are currently at Elevations 251.9 m to 252.0 m. The extended length of the proposed 1200 mm culvert will facilitate the construction of the new SB passing lane. The following are the options considered by Wills for culvert types:

- 1200 mm Reinforced concrete, 50-D, Class B bedding, no restriction to wall type
- 1220 mm HDPE, RSC 160, closed profile wall – smooth inside and outside
- 1200 mm PVC, Class 210, solid wall – DR smooth inside.

The replacement of the culvert and the construction of the new SB passing lane are likely to be carried out using staged construction similar to Culvert C19. Half of the highway embankment is to be excavated at a time while removing and replacing the culvert. Upon removal and replacement of the culvert, the highway embankment will be replaced including the construction of the new SB passing lane (i.e. the highway will be widened but not raised). Diversion of the traffic, depending on which lane is available, will be carried out during the course of the staged construction. Finished side slopes of 3H:1V are proposed for the construction.

The boreholes (C1, C3, C4, H10 and H11) drilled in the vicinity of Culvert C20, indicated that at the culvert location the site is underlain by embankment fill over an approximately 2 m thick silty clay layer which is in turn underlain by silty sand till which was encountered in the immediate vicinity of the existing culvert at Boreholes C1, C3 and C4 at Elevations 250.8 m to 250.5 m. Below the till, bedrock was encountered at Elevations 249.2 m to 249.4 m. Underlying a 0.6 m thick granular pavement fill, the embankment fill in Borehole C4 consisted of silty sand which extended to 1.7 m below the road shoulder surface. From the recorded N-values it is described as loose to compact. The silty clay deposit was described as greyish brown to grey and firm to stiff. The underlying silty sand till is a basically granular soil but may exhibit some apparent cohesion due to its clay content. SPT 'N'-values recorded within the till material varied from 6 to in excess of 100 blows/0.3 m, indicating a loose to very dense condition. The groundwater level at the time of the field program was measured at Elevations 251 to 252 m.

### 5.3.1 Culvert Foundations

Below the proposed culvert invert elevations (i.e. below about elevation 252.0 m), the boreholes show the presence of a firm to stiff silty clay deposit which at its undisturbed state is considered suitable to support the proposed culvert.

The following geotechnical resistances can be used for design purposes, for the undisturbed subgrade soils.

Factored Geotechnical Resistance at ULS = 120 kPa

Geotechnical Resistance at SLS = 70 kPa

Provided that the bearing subgrade is not unduly disturbed during the construction, the SLS value will result total and differential settlements not exceeding 25 mm and 20 mm, respectively. Good construction techniques, including dewatering and water diversion as discussed below, will be also required to achieve this.

A Geotechnical Engineer who is familiar with the findings of this investigation should evaluate all bearing surfaces prior to placement of bedding for the proposed culvert. Where unsuitable bearing conditions are observed, remedial procedures can be established in the field to avoid construction delays. Bedding requirements are discussed in Section 5.4.

The unfactored horizontal resistance against sliding between the approved subgrade (silty clay) and the bedding can be calculated using a friction angle of 25°. This value can also be used if a geotextile is utilized in conjunction with the bedding. Sliding is however unlikely to present a problem for this project.

In our opinion, with the prevailing subsurface conditions, the use of a flexible structure such as a CSP would be better suited, due to its flexibility and ease and time frame of construction (i.e. short construction period). This however would be contingent upon whether the use of a CSP is feasible with the prevailing environmental conditions (e.g. corrosion) and on local MTO Section preference. If a concrete structure is to be used, then a precast concrete culvert would be better suited, in comparison with a cast-in-place rigid concrete structure. This is because a flexible structure would be better suited with the subsoil conditions encountered in the boreholes, as well as expediency in construction (i.e. a precast concrete culvert would take less time to construct at site in comparison with a cast-in-place concrete culvert).

To provide sufficiently dry conditions during construction of the culvert, it will be necessary to divert the water flowing in the watercourse. Surface water diversion measures are discussed above in Section 5.1.1.

The groundwater table at the time of the field program was at about the anticipated excavation level, although a perched water level could occur. As such, depending on the site conditions at the time of construction, dewatering may be necessary. Since the clay deposit, which is likely to be encountered at or immediately below the anticipated excavation depths, is a relatively impervious soil type, the use of perimeter ditches, gravity drainage and pumping from open sumps would be sufficient. If, however, the excavation and/or sumps extend into the underlying silty sand till then proper filtering would be necessary.

### **5.3.2 Embankment Widening**

Based on the information provided by the client, the existing Highway 11 embankment will be marginally widened, as shown on Drawing 4B, to accommodate a new SB lane at Culvert C20. The height of the existing highway embankment will be maintained and that is approximately 2 m high.

We recommend that site preparation for the embankment widening similar to that of Culvert C19, as discussed in Section 5.2.2, can be adopted for Culvert C20.

A topsoil thickness of about 0.4 m can be used for preliminary estimating purposes of stripping depths. However, in our experience the thickness of organic soils can be variable at or adjacent to watercourses and in depressed areas. As well, organic materials were encountered within the fill and stripping depths maybe increased to remove the unsuitable organics within the fill. After stripping, the exposed surface should be inspected and approved.

Groundwater is unlikely to be encountered during stripping of unsuitable materials. However, if perched water conditions are encountered and water seeping into the excavations, this can likely be controlled by gravity drainage and pumping from sumps.

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

Similar to Culvert C19, temporary inner slopes of 1.5H:1V is recommended and a minimum 1.5 m clearance should be maintained between the moving traffic and the edge of the slopes. Monitoring of the slope movement during operations is also recommended. These temporary slopes should be utilized for a



short duration only, say maximum three days. We also recommend that these temporary side slopes be monitored continuously for any movement or any impending signs of instability during the construction. The proposed outer side slopes of 3H:1V are considered stable on short and long term basis.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. select subgrade materials (SSM) or Granular 'B' Type I – OPSS 1010). . In as much as possible, the fill used should match the existing embankment fill, within the frost zone. The embankment fill should be placed on the approved and properly rolled subgrade in lifts not exceeding 300 mm when loosely placed and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. The first lift may need to be somewhat thicker (e.g. 0.5 m thick), depending on the site conditions at the time of construction.

Embankment loadings would likely result in a settlement of less than 10 mm (for an embankment widening of up to 2 m, 4.0 m extension from the toe of the existing embankment and 3H:1V side slopes, see Drawing 4B) due to the settlement of natural foundation soils and the existing embankment. As this settlement is only minor, neither surcharging nor preloading is considered necessary for the embankment.

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

## **5.4 Bedding**

For flexible pipes, bedding and backfilling should be in accordance with OPSD 802.10 and 802.014. We recommend that a minimum 200 mm thick bedding material be placed beneath the flexible pipe to provide a uniform support underneath the culvert structure. For rigid pipes, the minimum bedding thickness should be of 0.15D, where D is the inside diameter of the rigid pipe as per OPSD 802.031, 802.032 and 802.034. The recommended thickness of bedding for rigid pipes is between 200 mm and 300 mm. For precast box culverts, backfilling and bedding should be in accordance with OPSD803.010 but the bedding should have a minimum thickness of 400mm. Thicker bedding may be required depending on the prevailing subgrade conditions at the time of construction.

As discussed in Section 5.1.1, the bedding thickness of the possible CSP extension for C16 is equal to depth between the base of CSP extension and existing culvert footing at the existing and extension culvert interface and can be reduced to 200 mm thick about 3 m away from the existing and extension culvert interface. It should be noted that at the existing and extension culvert interface, excavation for the placement of bedding should not extend below the base existing footings so as not to undermine the existing structure.

The bedding should consist of a well-graded granular material such as a Granular 'A' or a Granular 'B' Type II (OPSS 1010). For ease of construction, consideration may also be given to the use of 20 mm clear stone or preferably an HL4 type material. In this case (i.e. if a well-graded bedding material is not used) however, the bedding should be protected to minimize the risk of migration of the fine particles from subgrade by placing a suitable geotextile against the subgrade soil. The geotextile (OPSS 1860) should be a Class II non woven type of filter cloth with Filtering Opening Size (F.O.S.) not larger than 115 micron

(such as Terraxfix 400R, or approved equivalent). We also recommend that the compatibility of the geotextile with the exposed subgrade be reviewed and approved during the construction.

The bedding material should be placed as soon as practicable after the preparation of the subgrade, its inspection and approval, as was discussed in the previous sections of this report. The bedding material 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, 802.032 and 802.034 for rigid pipes whichever is applicable).

The bedding material should be compacted to MTO standards (OPSS 501 or SP 105S10) whichever is applicable).

## **5.5 Backfilling**

The bedding and embedment material should be extended along the sides and the top to cover the pipe. The selection and placing of the backfill should be in accordance with OPSD-802.010 and 802.014 for flexible pipes, OPSD 802.030, 802.031, 802.032 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 each lift should be compacted to at least 95% of the material's SPMDD (Standard Proctor Maximum Dry Density). Although this is not an MTO requirement, we recommend that, if feasible, the degree of compaction of the fill materials within 1.5 m of pavement subgrade be increased to not less than 98% SPMDD. The Granular 'A' base and Granular 'B' sub-base courses (OPSS 1010) should be compacted to not less than 100% of the material's SPMDD.

The use of proper backfill material and especially good compaction are necessary for proper side support and successful performance of the pipe/culvert. For the same reason, the organic soils should be removed within a suitable distance from the footprint of the culvert. The use of heavy compaction equipment should be avoided immediately adjacent and above the culvert, as per MTO practice. During backfill placement, the height of the backfill should be maintained at approximately same level on both sides of the structure, to avoid lateral displacement (dislodging) and/or damage of the structure.

For fills immediately below the roadway, we recommend that Granular 'A' or 'B' aggregates (OPSS 1010) be used. Where necessary, proper tapering as per standards should be provided. Below a depth of about 1.5 m to 2.0 m from the finished road grade, an approved compactable fill, such as select subgrade materials (SSM) can be used.

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

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

Computation of earth pressures acting against rigid culvert walls and any wing walls should be in accordance with the Canadian Highway Bridge Design Code, (CHBDC) 2006. For design purposes, the following properties can be assumed for backfill.

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

Angle of Internal Friction  $\phi=35^\circ$  (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  
 $K_o$  is the coefficient of earth pressure at rest  
 $K^*$  is the earth pressure coefficient for a soil loading a fully restrained  
 structure and includes compaction effects

Where  $K_b$  is the 'intermediate' earth pressure coefficient for a partially restrained structure. This case occurs when some movement (yield) of the retaining structure occurs but not in a sufficient magnitude to fully mobilize an active condition (as such an intermediate condition between  $K_o$  and  $K_a$  occurs).

$K^*$  is the earth pressure coefficient for a soil loading a fully-restrained structure, including compaction surcharge effects.

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

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. The use of vibratory

compaction equipment behind the culvert and the retaining walls should be restricted in size as per current MTO practice.

## 5.6 Construction

It is anticipated that the excavations for the construction comprise the excavations of the existing embankments and stripping of unsuitable soils from beneath the embankment widening areas. Relatively deep excavations can be anticipated to extend the excavation to invert/founding levels, the latter especially in the case of an open bottom concrete culvert structure for Culvert C16. Excavations should be possible using heavy equipment such as a hydraulic excavator. Boulders and cobbles can be expected within the fill and till layers.

Discussions regarding groundwater issues during excavation were provided in the previous sections.

All excavations must be carried out in accordance with the Safety Regulation of the Province (i.e. Occupational Health and Safety Act (OHSA) O. Reg. 213/91), as well as the following specifications:

SP 105 S19 – Protection Systems

SP 902 501 – Excavation and Backfilling to Structures

In accordance with OHSA, the soils can be classified as follows

Embankment Fill	Type 3 soil above water level
Silty Clay	Type 3 soil
Sandy Silt Till	Type 3 soil above water level; Type 4 soil below water level

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

The excavated soils free from topsoil and organics by can be used as general construction backfill where it can be compacted with smooth drum type rollers. Loose lifts of soil, which are to be compacted, should not exceed 300 mm. On site verification of the excavated fill for re-use as backfill by suitably qualified personnel during construction would be required. The measured material moisture contents of the silty clay deposits are close to its measured Liquid Limit and therefore proper compaction of this material should not be anticipated. In addition, during wet periods, the sand fill and the silty sand till will likely be unsuitable for reuse. Selective stockpiling and double handling may be required for reuse of these materials.

The on-site excavated soils are not considered to be free draining. Where free draining backfill is required, imported granular fill such as Granular 'B' (OPSS 1010) should be used.

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

Foundation bearing soils near the water table and in wet weather are susceptible to disturbance from construction activity. Care should be taken during excavation and construction to minimize disturbance of the bearing soil. Stabilization of wet subgrades should be anticipated. Disturbance of the underlying soils during construction of the structures, in proximity to the groundwater table, could influence the future settlements of the proposed structures.

## **5.7 Erosion Protection**

Erosion and scour protection should be provided at the culvert inlet and outlet (including the slopes and sides). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report.

The use of cut-off wall/apron can be guided with by the existing conditions, soil erodability and watercourse dynamics. Based on our experience, a cut-off is recommended for box culverts. For CSP, a cut-off is used sometimes but not always. The existing conditions can be used to check if a cut-off is required.

The following are some general suggestions for erosion and scour protection.

We recommend that concrete cut-off (apron) be constructed both at the inlet to prevent seepage beneath and around the culvert, especially through the granular bedding and granular backfill around the culvert. Consideration can be given to providing a concrete apron at the outlet, if there is none. 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 a low permeability seal at the inlet and outlet.

At the inlet, consideration may also be given to the use of a clay seal. The purpose of the clay seal is to allow water flow to be channelled through the culvert and does not seep through the backfill around the structure and from beneath the structure. The clay seal should therefore be continuous and is typically 0.6 m thick. It should comply with the material specifications given in OPSS 1205. At Culvert C20 site, the existing silty clay may be suitable for this purpose. In any event, the clay seal should be extended around the culvert from at least 0.3 m above the high water level in the watercourse down to the channel bed and up the other side in a continuous manner. It should be ensured that it extends to cover all the granular backfill materials to prevent any seepage through them. Typically, the clay seal is protected by laying a 0.6 m thick rock protection over it. The clay seal would generally be extended at about 6 m beyond the inlet.

At the outlet as well as at the inlet (if clay seal is not used), in addition to the concrete cut-off and/or low permeability seal or in conjunction with these, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered. As the subgrade can be expected to consist of silt, a layer of granular or man-made filter material should be used. This would generally be extended about 6 m along the channel and the sides (to at least 0.3 m above the high water). The granular filter material underlying the rock protection can consist of a suitable granular material such as Granular 'A' (OPSS 1010). Alternatively, a suitable geotextile can be used underneath the rock fill, in lieu of the granular filter material. Another reference for consideration is OPSD 810.010 Rip-Rap Treatment for Culvert Outlets.

At the culvert outlet a filter diaphragm could also be considered to minimize the risk of migration of fines. The existing conditions at the culvert sites should be examined for the selection of appropriate scour and erosion protection schemes.

## 5.8 Frost Protection

Design frost protection depth for the site is about 2.3 m. A minimum 2.3 m thick permanent soil cover or equivalent thermal insulation is required for frost protection of foundations.

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

## 6 CLOSURE

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

For and on behalf of Coffey Geotechnics Inc.



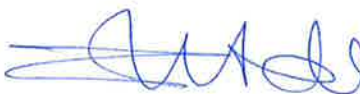
**Delfa Sarabia, M.Eng.**

Senior Geotechnical Engineer



**Ramon Miranda, P.Eng.**

Manager, Transportation



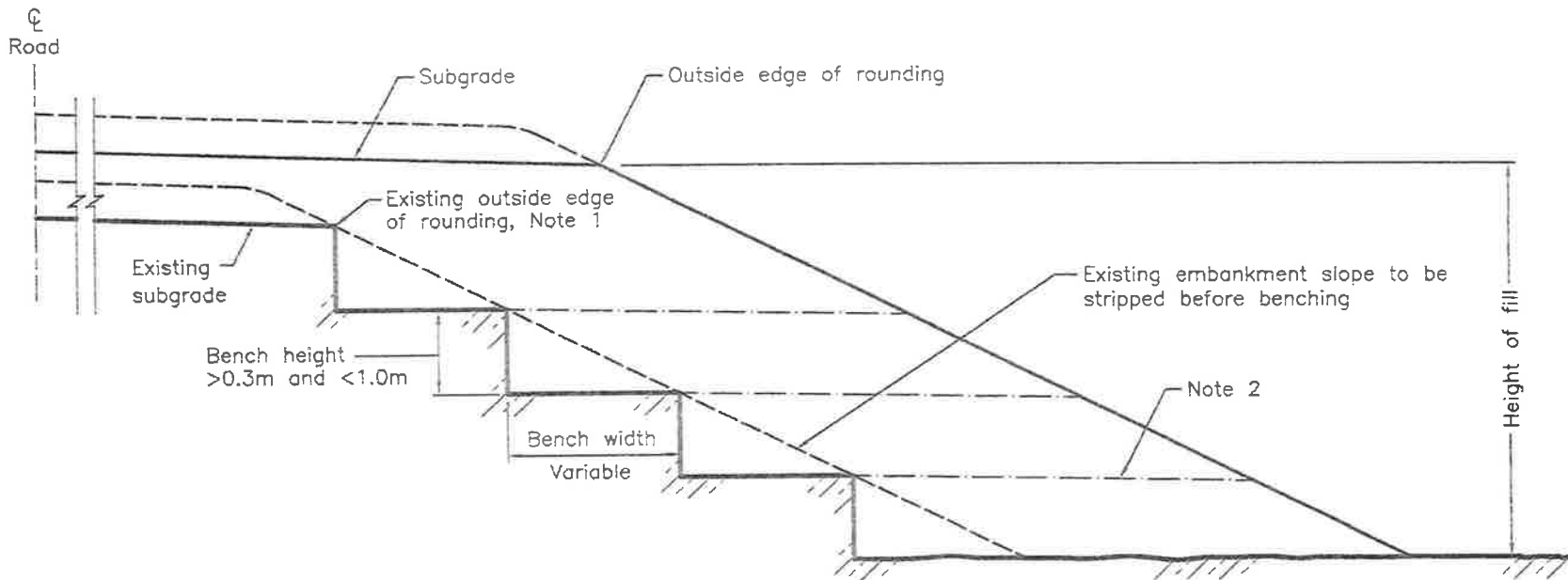
**Zuhtu Ozden, P.Eng.**

Senior Principal



# Appendix F

OPSD



**NOTES:**

- 1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.
  - 2 Benches are to be excavated one level at a time and the fill placed and compacted before the next bench is excavated.
- A Benching is not required on existing slopes flatter than 3H:1V.
- B All dimensions are in metres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2008

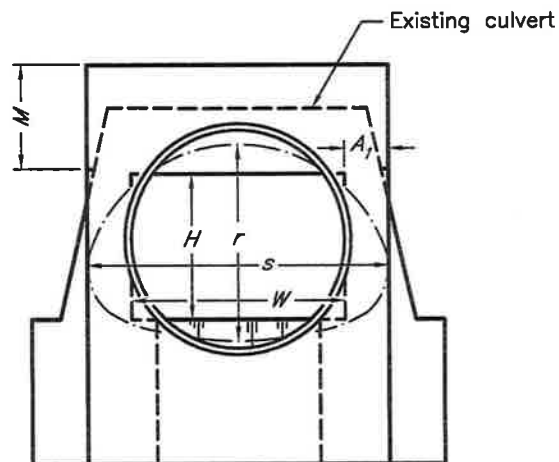
Rev 2

**BENCHING OF EARTH SLOPES**

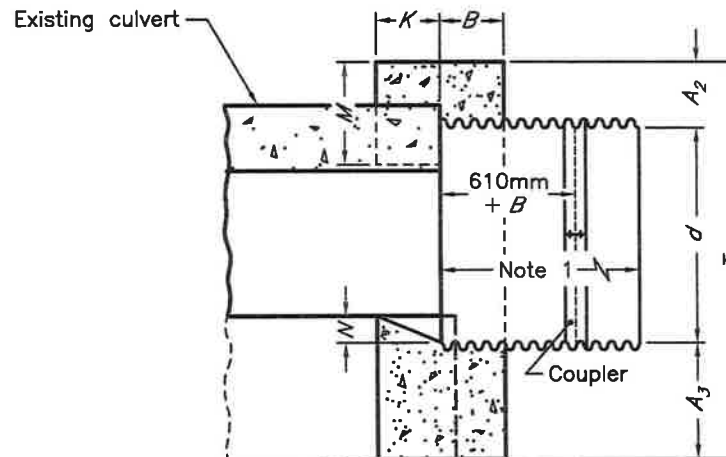


**OPSD 208.010**





**FRONT ELEVATION**  
**NOTE 2**



**LONGITUDINAL SECTION**  
**NOTE 2**

**TABLE SHOWING DIMENSIONS**

Culvert $W \times H$	CSP $d$	CSP Arch $s \times r$	$A_1$	$A_2$	$A_3$	$B, K$	$M$	$N$
610x 610	800	—	300	300	840	300	380	95
900x 610	900	—	300	300	760	300	450	145
900x 900	1000	—	300	300	840	300	380	50
1220x 900	1200	—	300	300	760	300	450	150
1220x1220	1400	—	300	300	840	300	380	90
1520x 900	—	1390x 970	330	330	840	380	400	35
1520x1220	1500	—	330	330	760	380	480	140
1520x1520	1600	—	330	330	840	380	400	40
1830x 900	—	1880x1260	355	355	815	450	450	180
1830x1220	—	* 2060x1520	355	355	760	450	480	150
1830x1520	1800	—	355	355	760	450	500	140

\* Structural plate construction

**NOTES:**

- 1 Corrugated steel pipe to be cut 610mm +  $B$  from end and installed as in longitudinal section.
  - 2 Open frame culvert shown. For box frame culverts,  $K=0$ .  $T$  is the same for open and box frame culverts.
- A Culverts with headwalls:  $K=0$  and  $M=0$ .  
 B Class of concrete to be 30MPa.  
 C Bedding and backfill as specified.  
 D All dimensions are in millimetres unless otherwise shown.

**ONTARIO PROVINCIAL STANDARD DRAWING**

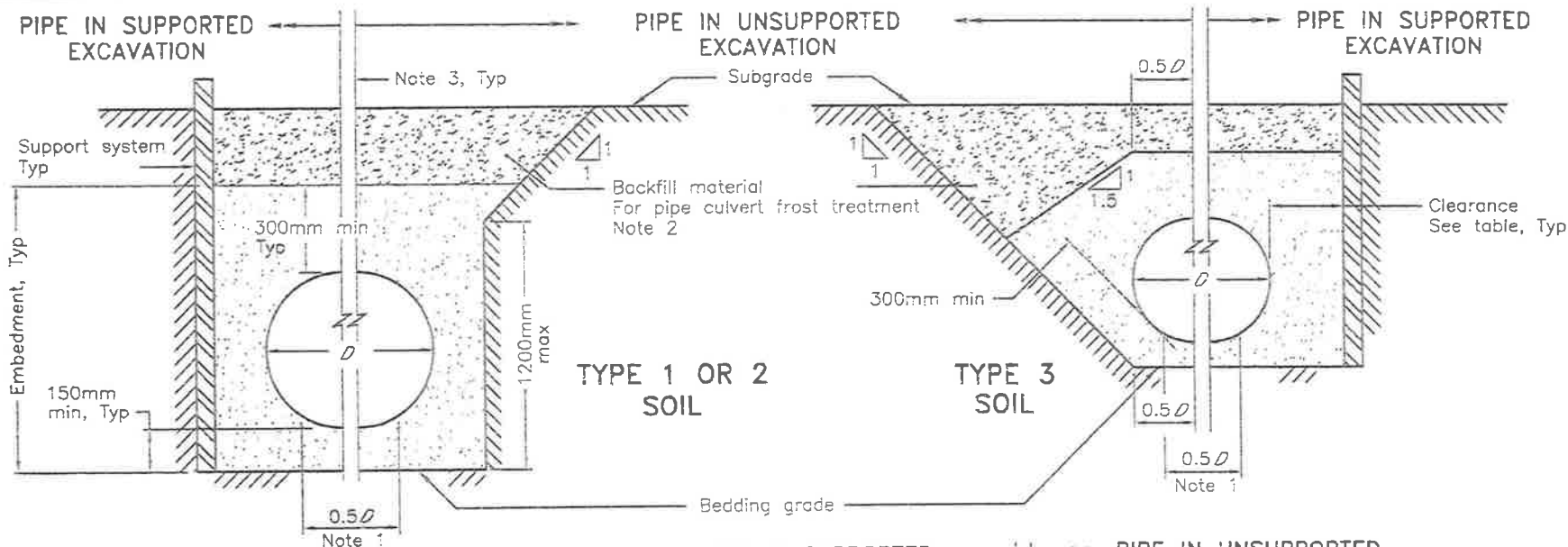
**CONCRETE RIGID FRAME BOX  
AND OPEN CULVERT EXTENSIONS USING  
CORRUGATED STEEL PIPE**

Nov 2007

Rev 1



**OPSD 800.011**



#### LEGEND:

$D$  - Inside diameter

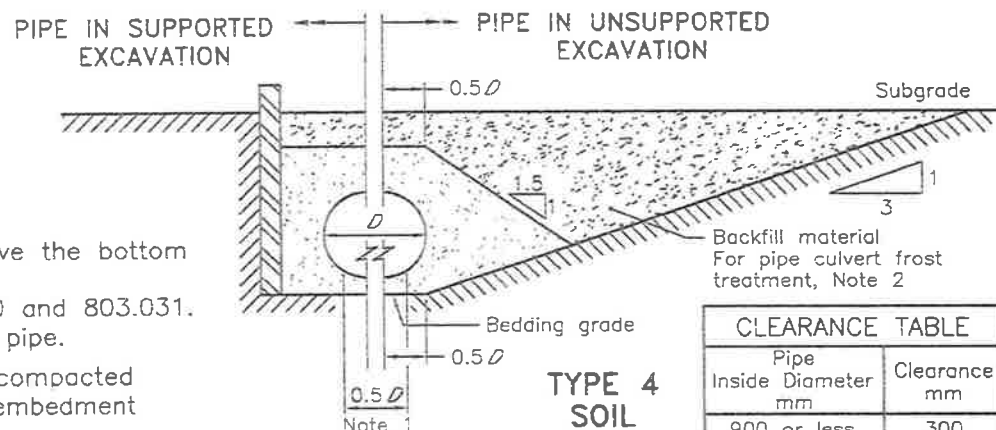
#### NOTES:

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

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

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

C All dimensions are in metres unless otherwise shown.



#### CLEARANCE TABLE

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

ONTARIO PROVINCIAL STANDARD DRAWING

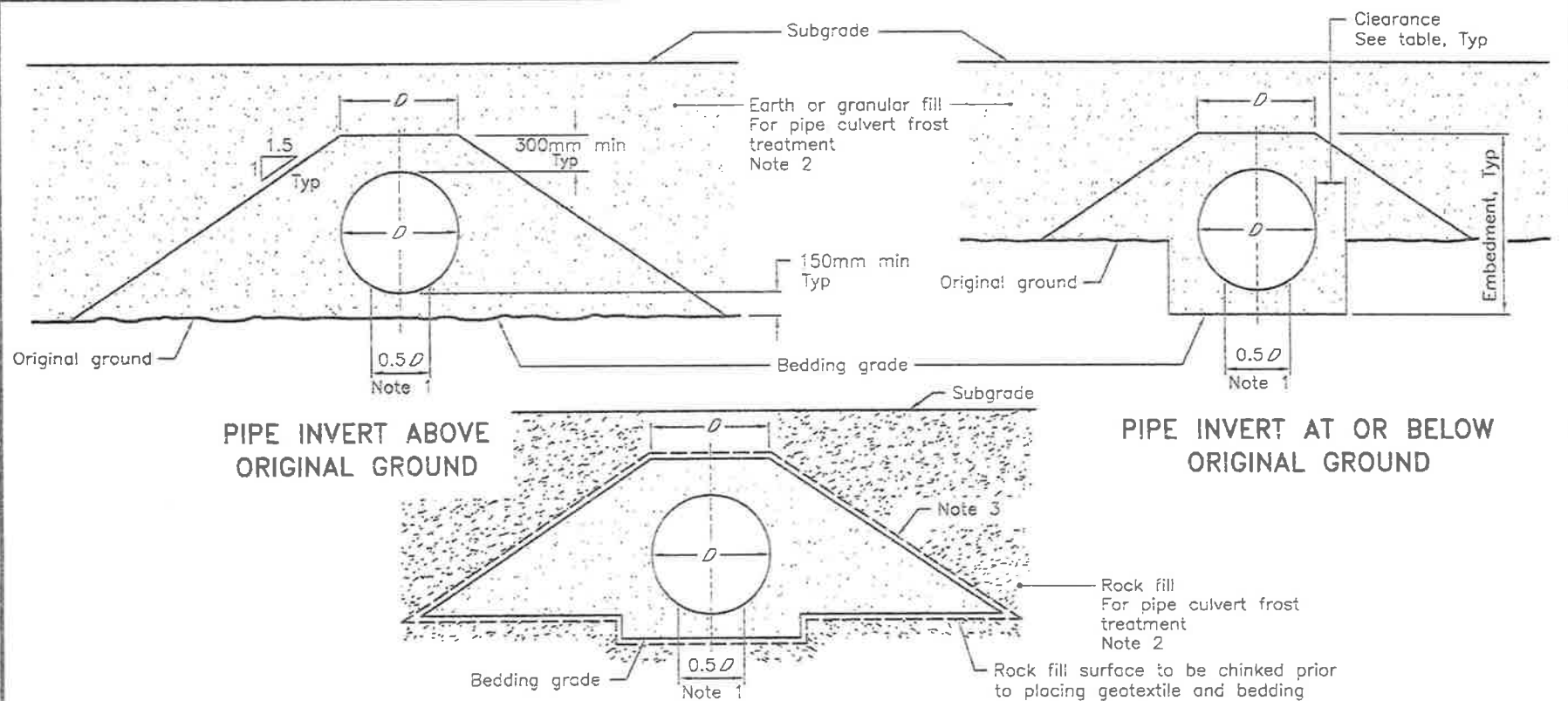
FLEXIBLE PIPE  
EMBEDMENT AND BACKFILL  
EARTH EXCAVATION

Nov 2005

Rev 1



OPSD - 802.010



#### LEGEND:

$D$  - Inside diameter

#### NOTES:

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

#### 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

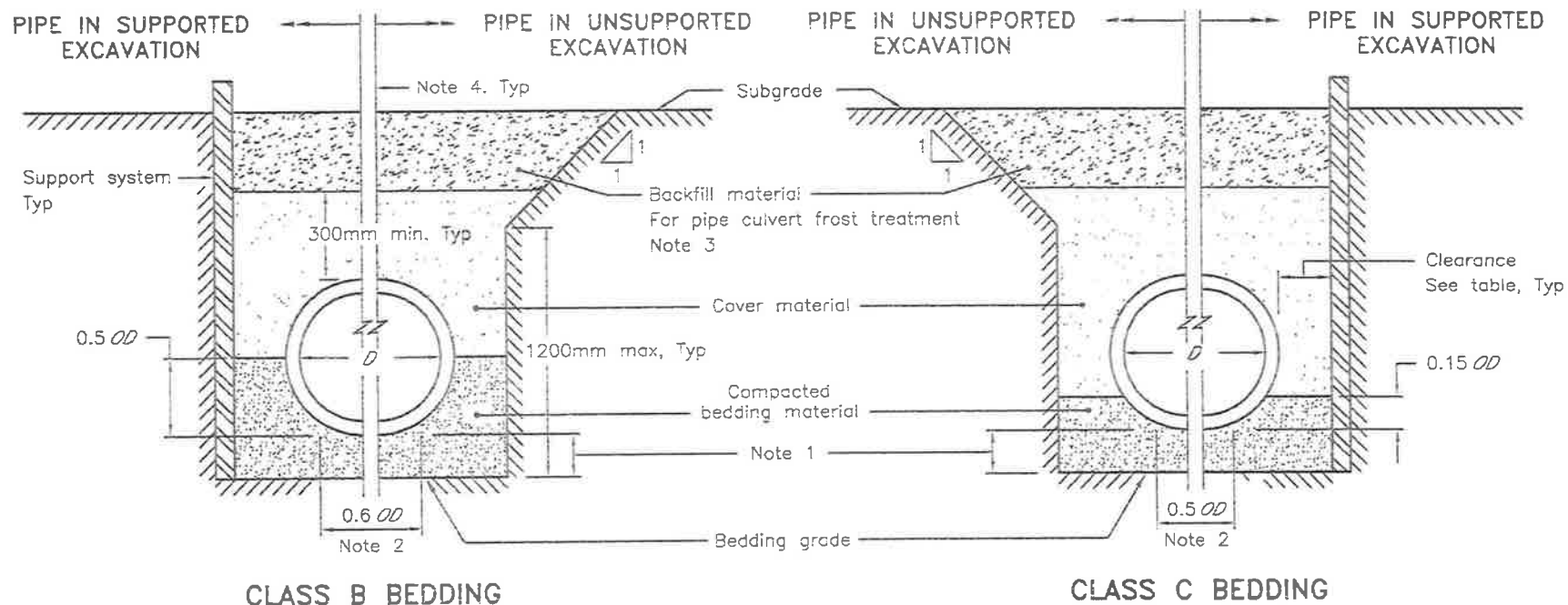
ONTARIO PROVINCIAL STANDARD DRAWING

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

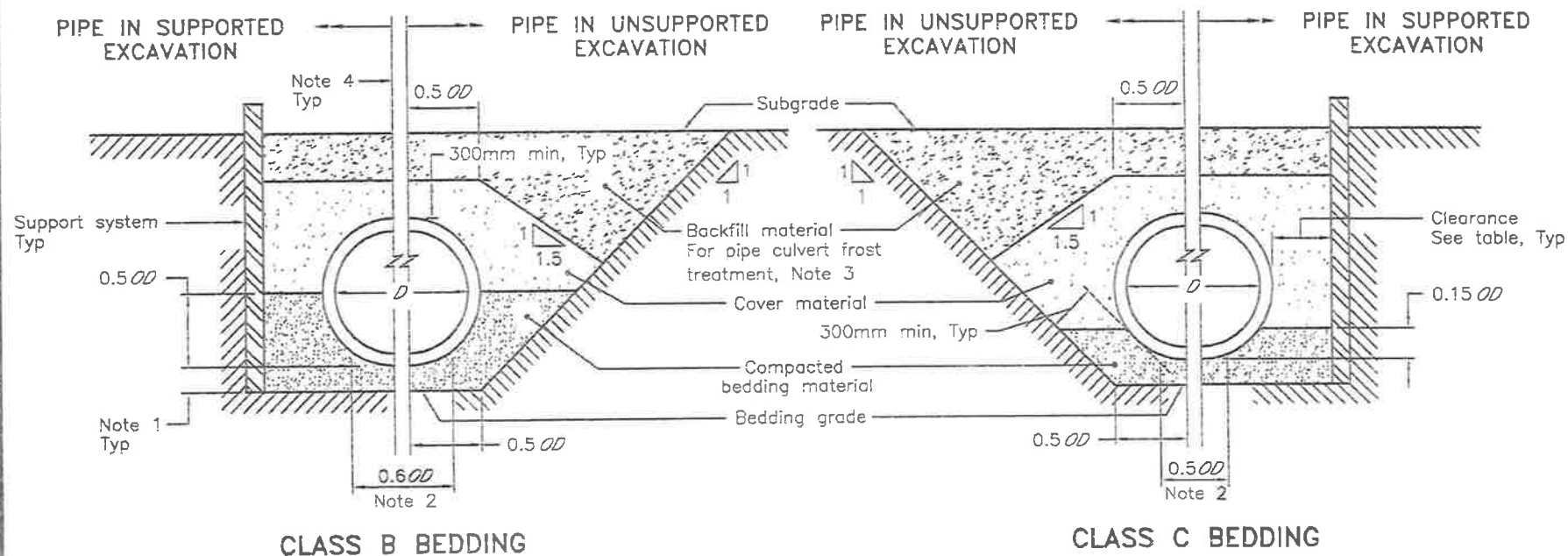
Nov 2005

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

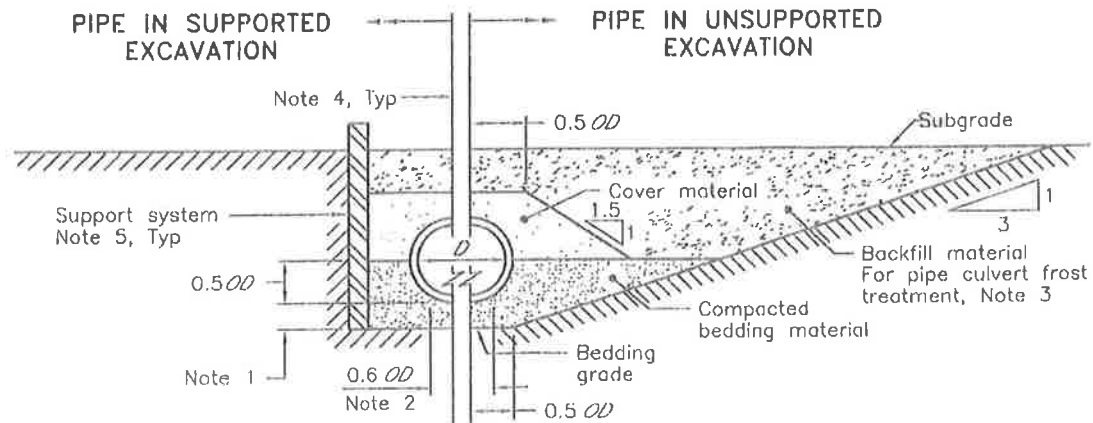
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 Rev 1

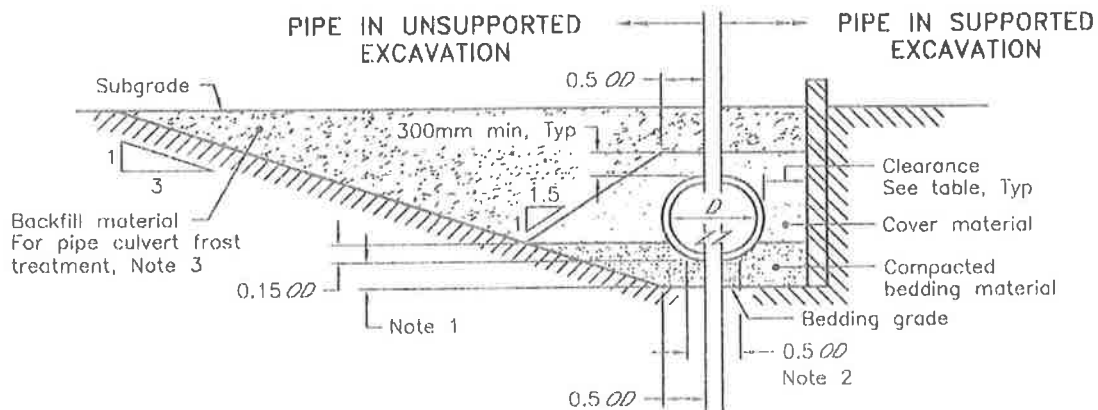
RIGID PIPE BEDDING,  
 COVER, AND BACKFILL  
 TYPE 3 SOIL - EARTH EXCAVATION

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

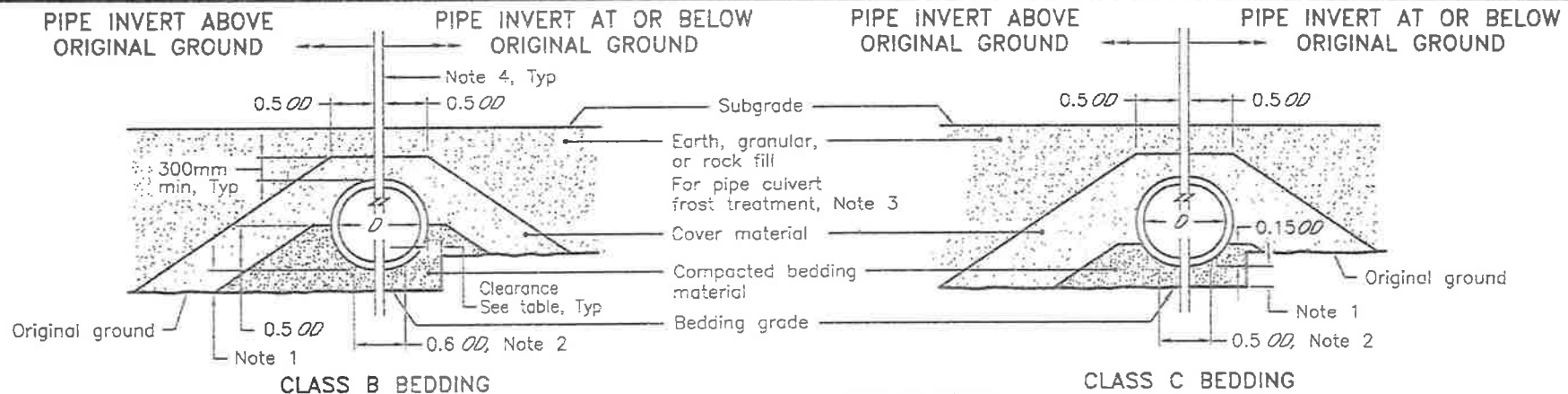
Nov 2005 Rev 1

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

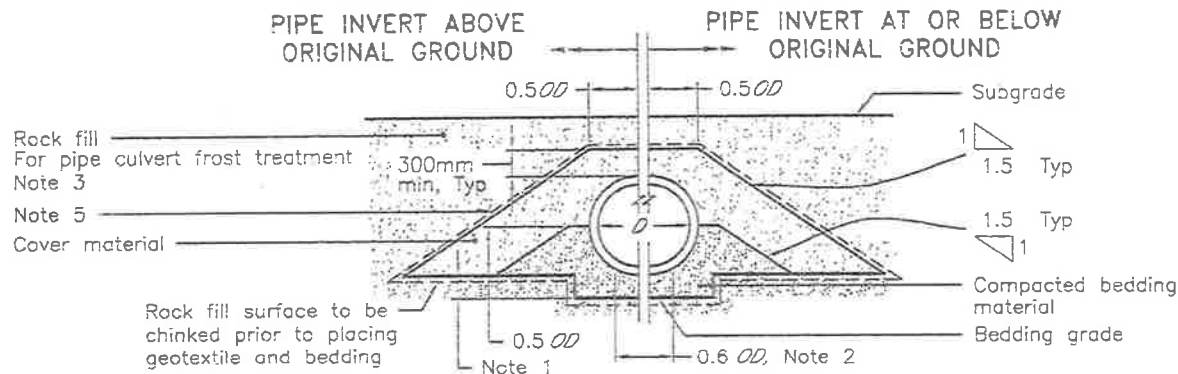
**OPSD - 802.032**







### 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

ONTARIO PROVINCIAL STANDARD DRAWING

RIGID PIPE BEDDING AND COVER

IN EMBANKMENT

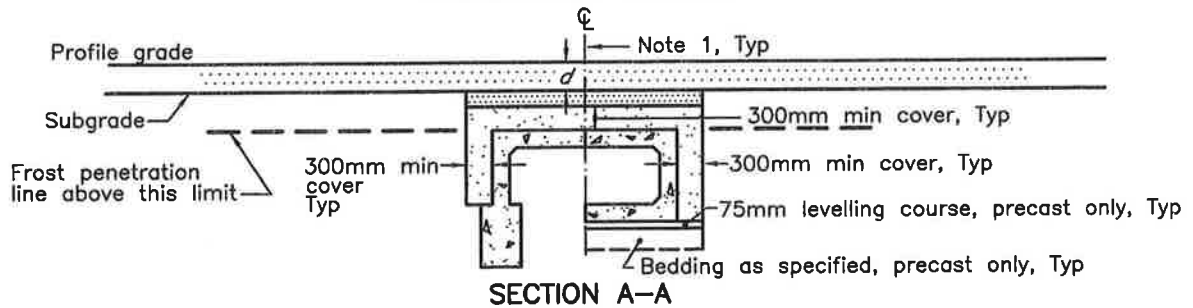
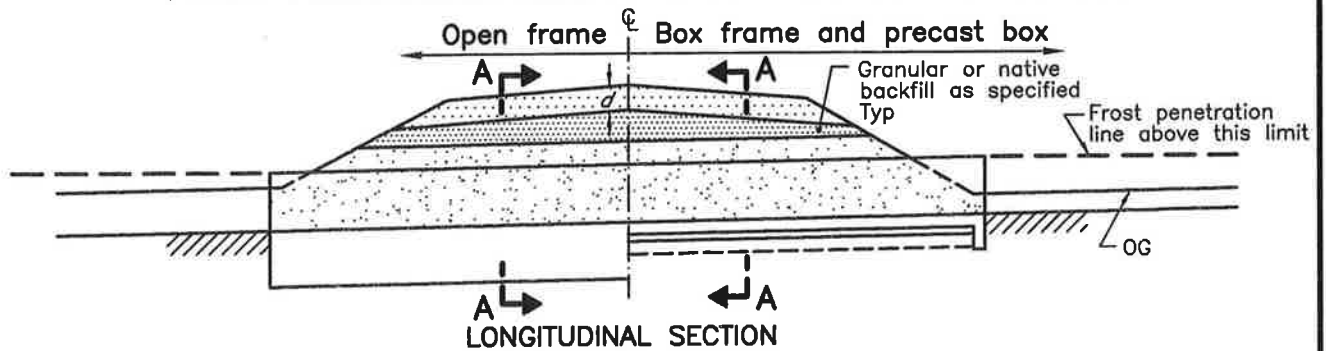
ORIGINAL GROUND: EARTH OR ROCK

Nov 2005 Rev 1

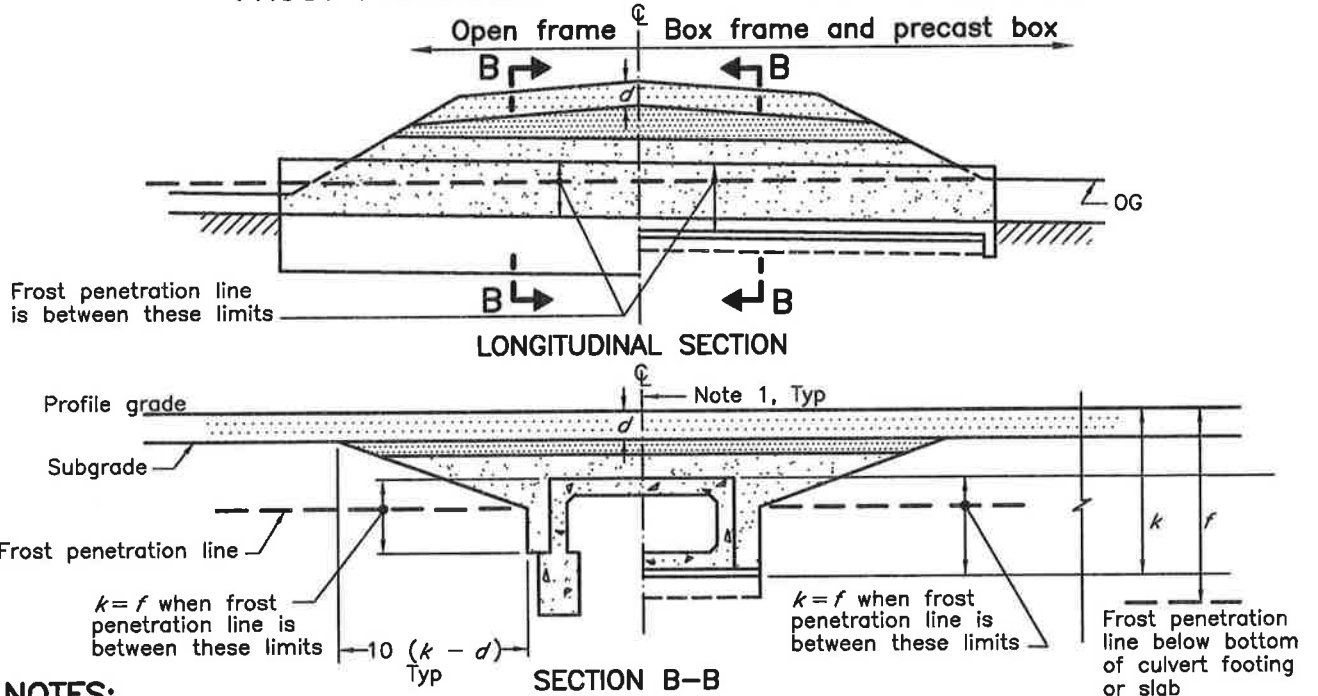


OPSD - 802.034

## FROST PENETRATION LINE AT OR ABOVE TOP OF CULVERT



## FROST PENETRATION LINE BELOW TOP OF CULVERT



### 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 rigid and non-rigid cast-in-place and precast concrete culverts.

- C All dimensions are in millimetres unless otherwise shown.

### LEGEND:

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

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 1999 | Rev

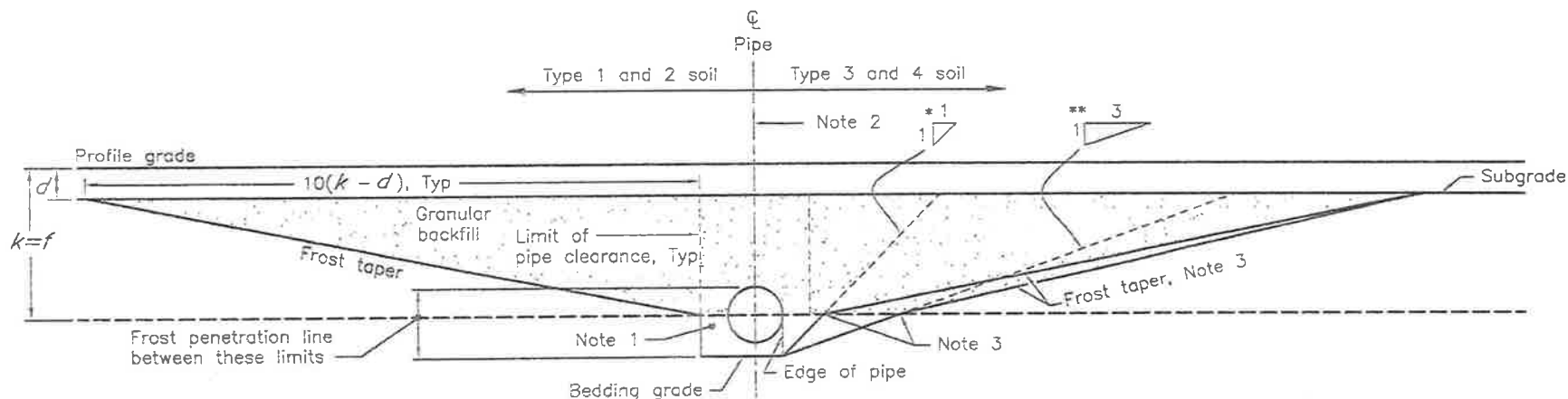
# BACKFILL AND COVER FOR CONCRETE CULVERTS

OPSD – 803.010





OPSD - 803.030



## FROST TREATMENT – RIGID AND FLEXIBLE PIPE

### NOTES:

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

### LEGEND:

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

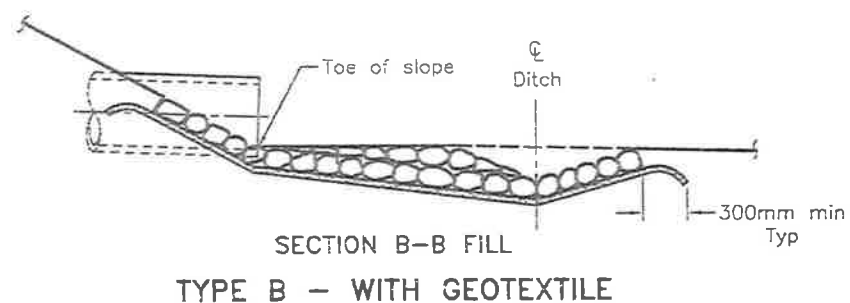
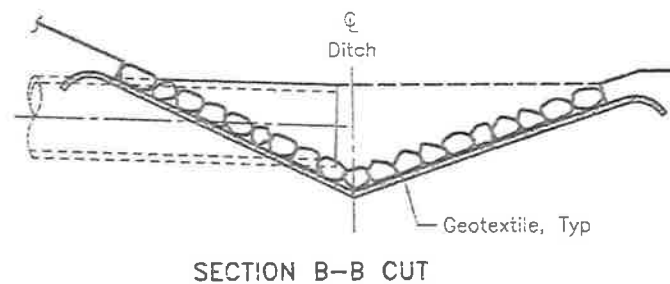
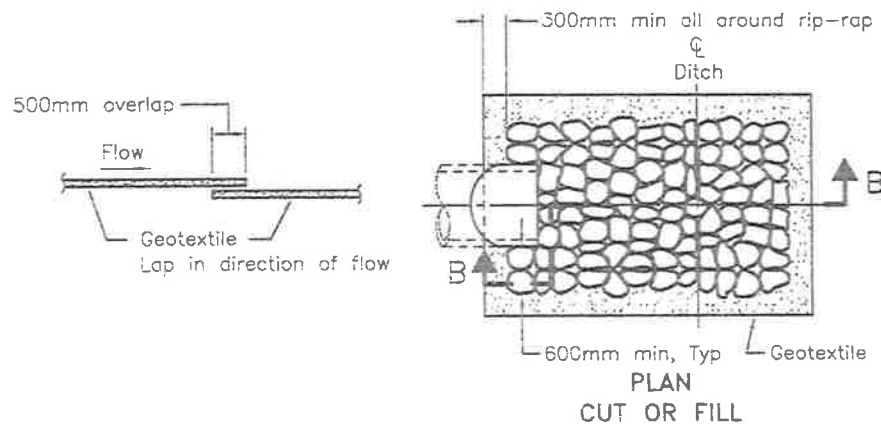
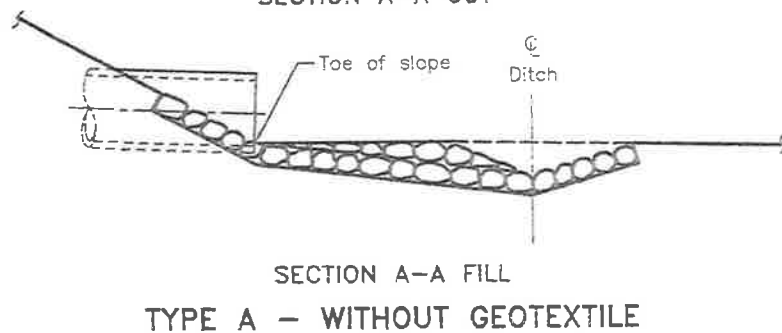
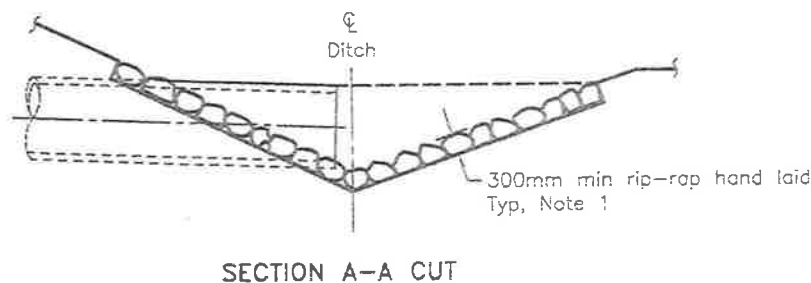
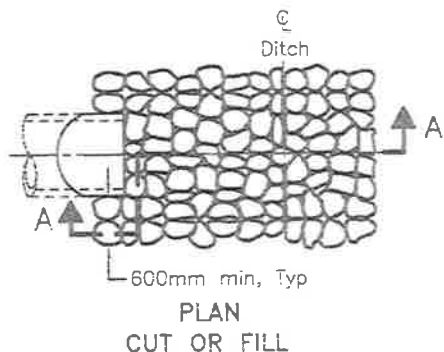
ONTARIO PROVINCIAL STANDARD DRAWING

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

Nov 2005 Rev 2



OPSD – 803.031



#### NOTES:

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

A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2007 Rev 1

## RIP-RAP TREATMENT FOR SEWER AND CULVERT OUTLETS

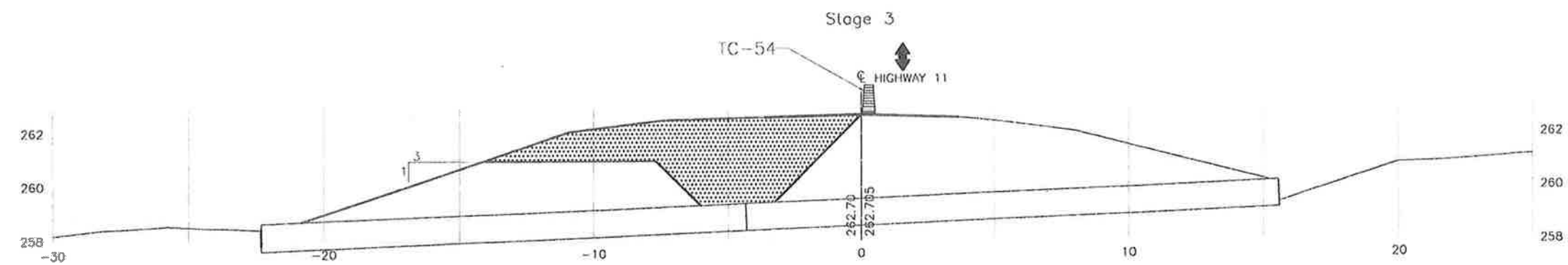
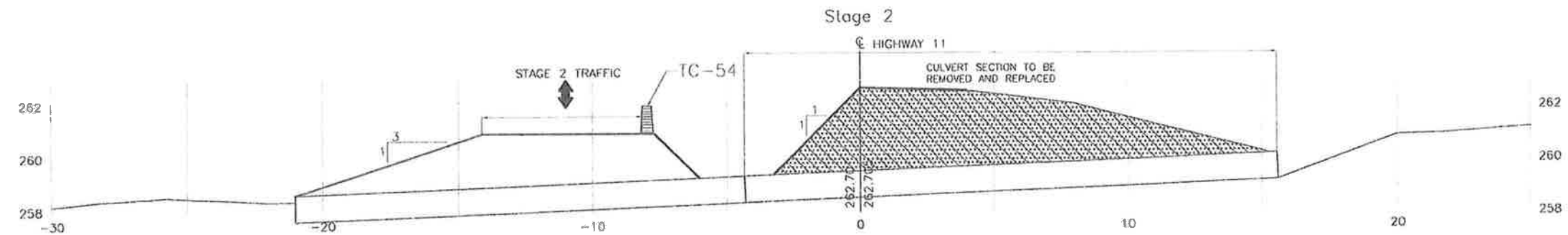
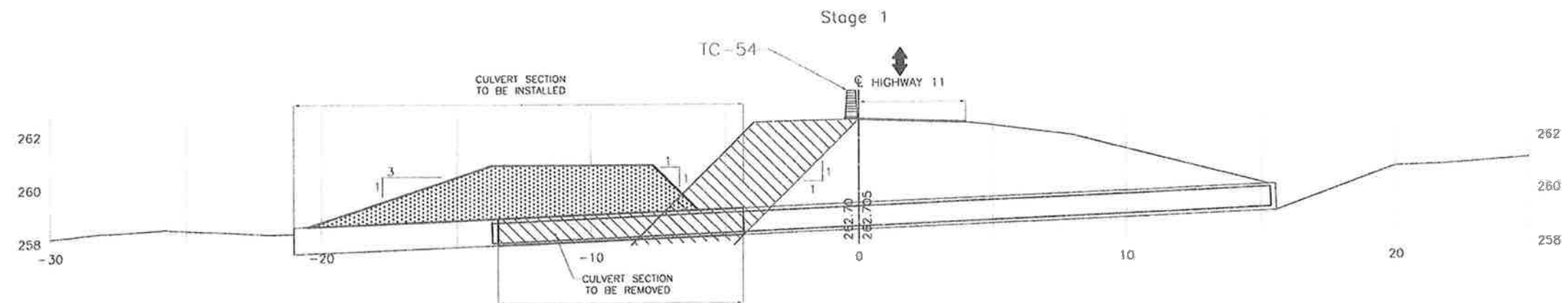
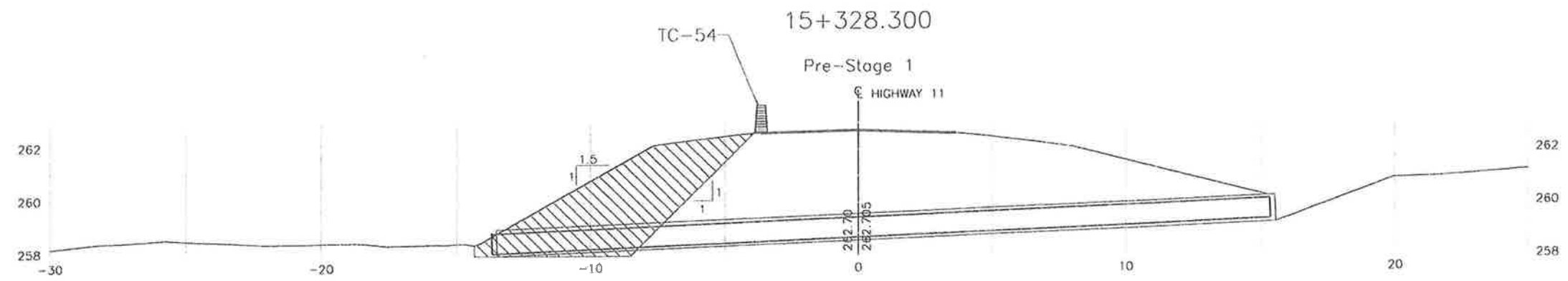
OPSD 810.010





# Appendix G

## **Proposed Staged Construction**





-  EXCAVATION
-  NEW CONSTRUCTION (FILL)

# Appendix H

## **Culvert Options**

### Culvert Extension-Options (C16)

Culvert Type	Advantages/Disadvantages	Risks/Consequences/Relative Cost	Preferred Alternative
NRFO (existing culvert)	<ul style="list-style-type: none"> <li>Will require footings extending below the possible scour depths and as such will require relatively deep excavations below the groundwater table.</li> <li>Requires extended construction period. In this case however this may not be an issue if the traffic is maintained on the existing embankment.</li> </ul>	<ul style="list-style-type: none"> <li>Requires rigorous dewatering, due to relatively deep foundation excavation required.</li> <li>May cause adverse effects on the existing culvert.</li> <li>Relatively more sensitive to differential settlements. Relatively higher cost.</li> </ul>	Not recommended.
Rigid Frame Open Bottom Concrete Culvert (RFOC)	Similar to NRFO	<ul style="list-style-type: none"> <li>Similar to NRFO but even more sensitive to differential settlements.</li> <li>Relatively higher cost.</li> </ul>	Not recommended.
Rigid Frame Concrete Box Culvert (RFCB)	Less excavation extending into the watertable in comparison with NRFO or RFOC.	<ul style="list-style-type: none"> <li>Sensitive to differential settlements.</li> <li>Relatively higher cost.</li> </ul>	More suitable than NRFO and RFOC but not the recommended option with the prevailing soil and groundwater conditions.
Concrete Box Culvert	Similar to RFCB	<ul style="list-style-type: none"> <li>Somewhat less sensitive to differential settlements than NRFO, RFOC and RFCB culvert types.</li> <li>Relatively higher cost.</li> </ul>	May be considered but not the preferred alternative.
Corrugated Steel Pipe (CSP)	<ul style="list-style-type: none"> <li>Flexible and can withstand relatively high settlements without significant damage, except at the connections with the existing culvert.</li> <li>Can be placed very rapidly.</li> <li>Not very resistant to corrosions especially in case of an aggressively corrosive environment (i.e. soft water) on the water course.</li> </ul>	<ul style="list-style-type: none"> <li>Least risky and most suitable for the prevailing soil and groundwater conditions.</li> <li>Adequate connection which can withstand some differential settlements will be required at the connections with the existing culverts. Least expensive.</li> </ul>	Recommended type of culvert with the prevailing subsurface conditions.

### Culvert Replacement –Options (C19 and C20)

<b>Culvert Type</b>	<b>Advantages/Disadvantages</b>	<b>Risks/Consequences/Relative Cost</b>	<b>Preferred Alternative</b>
Corrugated Steel Pipe (CSP) type culvert	<ul style="list-style-type: none"> <li>• Flexible and as such can withstand relatively high settlement without significant damage.</li> <li>• Can be placed very rapidly.</li> <li>• Not very resistant to corrosion, especially in case of aggressive environment (i.e. soft water) in the watercourse.</li> <li>• Needs adequate cover.</li> </ul>	Least expensive but may not be preferred by MTO due to shorter life span especially if it is exposed to the aggressive environment. The use of a flexible structure such as CSP is considerably better suited at the site, due to prevailing subsurface conditions.	Recommended option
Precast Concrete Box Culvert	<ul style="list-style-type: none"> <li>• More flexible than a rigid concrete box or a rigid concrete open bottom culvert but less flexible than a CSP type culvert. Typically used when subsurface conditions warrant the use of a flexible culvert but a CSP is unsuitable due to environmental reasons.</li> </ul>	Used when a CSP type culvert is not feasible and subsurface condition require the use of a relatively flexible structure. High cost.	Can be considered as an alternative.
Rigid Frame Concrete Box Culvert	<ul style="list-style-type: none"> <li>• Requires relatively competent soil conditions.</li> <li>• Requires considerable construction time and as such is not frequently used under existing highway embankments. In this case, however, this is not the case as the construction will proceed along the new alignment.</li> </ul>	Cannot withstand high differential settlements. Somewhat more expensive than an open bottom rigid box culvert but avoids scour issue, if adequate measures are taken.	Not recommended.
Rigid Frame Open Bottom Concrete Culvert	Similar to rigid frame concrete box culvert.	Similar to rigid frame concrete box culvert but the subgrade soils are susceptible to scour (i.e. less suitable than a closed bottom culvert).	Not recommended.

# Appendix I

## 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.