

**FOUNDATION INVESTIGATION AND  
DESIGN REPORTS  
HIGHWAY 417 / CYRVILLE ROAD BRIDGE,  
CITY OF OTTAWA, ONTARIO  
G.W.P. 4011-06-00  
GEOCRES NO. 31G5-229**

AECOM

Project: TRANETOB01226AA  
October 27, 2010

October 27, 2010

AECOM  
5080 Commerce Boulevard  
Mississauga, Ontario  
L4W 4P2

**Attention: Mr. Mani Rajendran, P. Eng.**

Dear Sir:

**RE: Foundation Investigation and Design Reports, Highway 417/Cyrville Road Bridge,  
City of Ottawa, Ontario, G.W.P. 4011-06-00, Geocres No. 31G5-229**

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

For and on behalf of Coffey Geotechnics Inc.



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

**FOUNDATION INVESTIGATION REPORT  
HIGHWAY 417 / CYRVILLE ROAD BRIDGE,  
CITY OF OTTAWA, ONTARIO  
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**FOUNDATION INVESTIGATION REPORT  
HIGHWAY 417/CYRVILLE ROAD BRIDGE  
CITY OF OTTAWA, ONTARIO  
G.W.P. 4011-06-00**

## **1 INTRODUCTION**

A new bridge which will replace the existing bridge is planned to be constructed to carry Cyrville Road over Highway 417 in the City of Ottawa. Coffey Geotechnics Inc. (Coffey) was retained by AECOM to carry out a foundation investigation at the site of the proposed bridge.

The existing structure is a two-span bridge with a total length of about 67 m. It was proposed to replace it with a new structure which would be about 14 m longer and 8 m wider than the existing bridge in the previous stage of this project (2008-2009). A foundation investigation was carried out by Coffey for the project in 2008. After the submission of our draft foundation investigation and design reports (issued on September 18, 2009), the bridge design was modified from a two span to a three span structure to accommodate a westbound ramp from Highway 174 to St Laurent Boulevard which will pass beneath Cyrville Road. The more recently proposed version of the bridge will be about 25 m longer than the existing bridge and will have same width as the previously planned bridge (i.e. 8 m wider than the existing bridge). The new bridge will incorporate a Pier No. 2 in the unpaved median of Highway 417 similar to the previous design but will have an additional pier (Pier No.1) in between Highway 417 and westbound ramp from Highway 174 to St Laurent Boulevard. Two additional boreholes were advanced after the design modification, at the newly proposed Pier No. 1 location. As shown on Drawing No. 1, the Pier No.2 and the east abutment will be constructed at the same location as previously planned two span bridge, however, the west abutment will be located some 12.7 m further to the west from the previously proposed west abutment location. Newly proposed Pier No. 1 foundations will be located about 33 m west from the proposed Pier No. 2 foundations. Similar to the existing and the previously proposed bridge structures, the newly proposed three span underpass structure will be skewed at about 43° to the existing Highway 417 centreline.

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

The findings of the investigation are presented in this report.

## **2 SITE DESCRIPTION AND GEOLOGY**

The project site is located at the intersection of Cyrville Road with Highway 417 in the City of Ottawa, Ontario.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the project site is located within the Physiographic Region known as the Russell and Prescott Sand Plains.

The site lies on a glacial till plain characterized by heterogeneous mixture of glacial till and silt/sand deposits. In addition, however, silty clay layers are not uncommon at the site. Topography across the site is generally flat.

According to the Southern Ontario Geological Highway Map (Map 2418), the bedrock underlying this area consists of a dark grey to black shale of the Billings Formation and is found to be considerably weathered and fractured. The geological explanation for rock in this condition is that at the time of the glacials, the frost penetrated to great depths and the softest shale layers were disturbed by frost action. This explanation is also advanced for the presence of shale fragments in the overburden above the parent rock.

The existing approach embankments, which are approximately 6.5 m high close to the bridge abutment, do not exhibit any apparent signs of slope instability or excessive erosion. As well, in the immediate vicinity of the existing bridge, there are no signs of excessive settlements/unusual cracking or deformations in the pavement.

### 3 INVESTIGATION PROCEDURES

The fieldwork for the proposed bridge was performed in two stages. The first field investigation was performed during the period of September 22, 2008 through October 6, 2008 for the previously proposed two span bridge structure (twelve boreholes, Boreholes B1 through B6, B8 through B11, P1 and P2) and the second investigation was carried out March 30 and 31, 2010 for the newly proposed Pier No.1 foundation (two boreholes, Boreholes P3 and P4) after the bridge design modification from two span to three span. The following table summarizes the borehole locations and drilling depths. The borehole locations are shown on Drawing No. 1.

**Table 3.1: Borehole Locations and Drilling Depths**

Borehole No.	Coordinates / Location	Depth of Borehole Below Existing Ground Surface (m)	Piezometer
Previous investigation in 2008			
B1	5031701.5 N, 373328.8 E (east abutment)	12.3	No
B2	5031706.0 N, 373344.1 E (east abutment)	14.3	Yes
B3	5031699.0 N, 373366.2 E (east approach)	10.7	No
B4	5031736.4 N, 373234.6 E (west abutment)	12.9	No
B5	5031741.0 N, 373250.9 E (west abutment)	12.3	Yes
B6	5031748.9 N, 373230.0 E (west approach)	8.1	No
B8	5031713.2 N, 373217.7 E (west embankment)	5.4	No
B9	5031762.6 N, 373264.2 E (west embankment)	6.2	No
B10	5031684.4 N, 373316.7 E (east embankment)	5.5	No
B11	5031726.4 N, 373351.8 E (east embankment)	5.4	No
P1	5031715.1 N, 373274.2 E (Pier No.2)	7.7	No
P2	5031728.0 N, 373301.1 E (Pier No.2)	9.7	No
New investigation in 2010			
P3	5031724.7 N, 373250.8 E (Pier No.1)	5.9	No
P4	5031737.1 N, 373274.4 E (Pier No.1)	5.9	No

\*Borehole B7 which was drilled as part of pavement investigation is not included in this report.

Marathon Drilling near Ottawa, Ontario carried out the drilling, testing and sampling work, under the direction and supervision of technical personnel (Mr. Suresh Kansal, P.Eng. for the first investigation and Mr. Raid Khamis, P.Eng. for the second investigation) from Coffey. The boreholes were advanced using a truck mounted drilling rig, outfitted with tools and equipment for soil sampling and testing. The boreholes were advanced using two different methods (i.e. continuous flight hollow-stem augers and NQ rock coring) depending on the ground conditions.

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

Groundwater conditions in the boreholes were observed during drilling and upon completion in the open boreholes. In addition, a piezometer was installed in each of Boreholes B2 and B5 to enable groundwater level monitoring in the boreholes over a prolonged period of time without interference from surface water. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

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

The soil and rock samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content, grain size analyses, and Atterberg Limits tests, was performed on selected representative soil samples and unconfined compressive strength (UCS) tests were performed (UCS tests at Golder Associates, Mississauga, Ontario) on selected rock core samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

## **4 SUBSURFACE CONDITIONS**

The subsurface conditions were explored at fourteen (14) boreholes (see Table 3.1 in Section 3) for this project. The plan locations of the boreholes and stratigraphic profile are shown on Drawing No. 1, while stratigraphic sections are presented on Drawing Nos 2 and 3. Details of subsurface conditions encountered at each borehole location for the investigation, including the results of in-situ testing, groundwater observations and laboratory test results, are presented on the Record of Borehole Sheets in Appendix A. Detailed laboratory test results are enclosed in Appendix B.

Boreholes B1, B2, B3, B4, B5 and B6 were put down from the Cyrville Road surface level between El. 74.9 and 76.2 m, while the remaining boreholes were drilled from the Highway 417 level or adjacent to it, from elevations ranging from 69.0 to 71.8 m.

Beneath various fill materials (including road pavement and embankment fill), and/or a veneer of topsoil, in general, the boreholes show the presence of some surficial sand and silt, underlain by a silty sand to sandy silt glacial deposit which contains shale fragments. These overburden materials are underlain by a dark grey to black shale bedrock belonging to the Billings Formation at about 0.7 to 4.6 m below the Highway 417 level (i.e. assumed at El. 69.5 m), or at El. 64.9 to 68.8 m.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in appendix A. The following paragraphs are only meant to amplify and complement these data.

## **4.1 Asphalt**

Boreholes B1 through B6, drilled from the existing Cyrville Road contacted a 60 mm (Borehole B3), 160-180 mm (Boreholes B1, B2, B4 and B6) to 250 mm (Borehole B5) thick asphaltic concrete. Boreholes P1, P3 and P4, which were drilled from Highway 417, contacted an about 200 mm thick asphaltic concrete layer.

## **4.2 Topsoil**

A 0.2 to 0.5 m thick topsoil layer was contacted in Boreholes B8 through B11, and in Borehole P2 at the existing grade. As well, traces of topsoil/buried topsoil were found in Boreholes B3 and B6 below the embankment fill material at depths of 4.6 m and 4.0 m, respectively.

## **4.3 Fill**

### **4.3.1 Pavement and Embankment Fill**

Boreholes B1 through B6 drilled from the existing Cyrville Road and Boreholes P1, P3 and P4 drilled from Highway 417 contacted an about 0.1 to 0.6 m sand and gravel (road base) followed by an about 0.2 to 1.8 m thick sand some gravel/gravelly sand materials (road sub-base).

The grain-size distribution of three samples from the pavement fill is given in Figure B-1, in Appendix B. This indicates the following grain-size distribution.

Gravel:	13-41%
Sand:	45-67%
Silt & Clay:	5-20%

The pavement fill is a granular (i.e. non-cohesive) soil. Standard Penetration tests performed in granular pavement fill yielded N-values of 14 to 23 blows/0.3 m. These results indicate that the relative density of the pavement fill can be described as compact.

Below the pavement fill the boreholes drilled from the existing embankment top (Boreholes B1 through B6) contacted an embankment fill consisting primarily of silty sand to sandy silt with varying degrees of clay content. This fill was found to extend to depths of 4.0 to 6.3 m or El. 69.9 to 71.7 m. This embankment fill contains trace to some shale fragments. The presence of organic traces was also noted within the fill. Borehole B3 encountered auger refusal due to a boulder at a depth of 2.1 m and the borehole was moved and re-drilled adjacent to its original location.



The grain-size distribution of seven samples from the embankment fill, underlying the granular pavement fill, is given in Figure B-2, in Appendix B. This indicates the following grain-size distribution.

Gravel:	0-19%
Sand:	48-89%
Silt & Clay:	11-51%

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

Standard Penetration tests performed in the embankment yielded N-values of 3 to 44 blows/0.3 m. These results indicate that the relative density of the fill can be described as very loose to dense but typically compact.

In Borehole B2, an approximately 0.4 m thick clayey silt zone was contacted at a depth of 4.6 m below ground surface.

An Atterberg limits test was performed on a sample from this clayey silt zone within the embankment fill, and the test yielded the following index values (see Figure B-3, Appendix B):

Liquid Limit:	17%
Plastic Limit:	12%
Plasticity Index:	5

From a recorded N-value of 8 blows/ 0.3 m, the consistency of cohesive zone in the embankment fill is described as firm.

The recorded N-values indicate that the fill materials received some compaction when the fill was first placed, except for some local very loose and firm zones.

#### **4.3.2 Other Fill**

Underlying the topsoil in Boreholes B8, B10, B11 and P2 and the pavement fill in Boreholes P3 and P4, a 0.7 to 3.7 m thick fill consisting of silty sand to sandy silt, silty sand and sand was contacted and found to extend to El. 68.0 to 65.7 m. The fill contains traces to some shale fragments in all boreholes and the fill material in Boreholes B8 and B10 exhibits traces of organics. The fill contacted in Borehole B10 and P2 may constitute a sewer trench backfill.

The grain-size distribution of three samples from the deposit is given in Figure B-4. The results of the analysis show the following grain-size distribution:

Gravel:	13-28%
Sand:	45-60%
Silt & Clay:	16-42%

Standard Penetration tests performed in this basically granular fill materials yielded N-values of 4 to 22 blows/0.3 m indicating as a very loose to compact relative density.

#### **4.4 Surficial Sand**

Underlying the fill in Boreholes B3 and P1, and the topsoil in Borehole B9, an about 0.3 to 0.6 m thick sand layer was encountered to El. 69.7 m, 67.8 m and 71.3 m, respectively.

This is a granular deposit.

Standard Penetration tests performed in the deposit yielded N-values which range from 7 to 14 blows/0.3 m indicating a loose to compact relative density.

#### **4.5 Silt**

Underlying the sand in Borehole B9, a silt layer was contacted. This deposit was found at a depth of 0.5 m and the unit extended to 1.1 m below the ground surface or to El. 70.7 m. Trace to some sand was noted in this basically granular soil deposit.

Standard Penetration tests performed in the deposit yielded N-values which range from 7 to 13 blows/0.3 m, indicating a loose to compact relative density.

#### **4.6 Sand (Possible Till)**

Below the sand layer in Borehole B3 and the embankment fill in Borehole B5, a 1.3 to 3.4 m thick sand deposit was encountered, at depths of 5.2 and 6.3 m or El. 69.7 and 69.9 m. This material contains traces of silt, gravel and shale fragments and it was identified as a possible glacial till deposit.

The grain-size distribution of two samples from the deposit is given in Figure B-5 which shows the following grain-size distribution:

Gravel:	2-12%
Sand:	81-86%
Silt & Clay:	7-12%

Standard Penetration tests performed in this granular deposit yielded N-values which range from 9 to 27 blows/0.3 m, indicating a loose to compact relative density.

#### **4.7 Silty Sand to Sandy Silt Till**

An heterogeneous mixture of silty sand to sandy silt deposit was encountered in Boreholes B1, B2, B4, B5, B6, B8, B9, B11, P1, P3 and P4. This is a glacial till deposit and it was contacted at depths of 1.1 to 7.6 m or El. 66.7 to 71.7 m and was found to extend to depths of 2.0 to 10.1 m or to El. 68.8 to 64.9 m. Borehole B6 was terminated in this deposit at a depth of 8.1 m or El. 67.6 m. The material contains trace to some shale fragments, occasional sand, sandy gravel and clayey silt zones/seams.

**Table 4.7.1: Depth/elevation of glacial till deposit**

Borehole No.	Depth Below Ground Surface/Elevation of the Top of the Deposit(m)	Depth of Below Ground Surface/Elevation of the Bottom of the Deposit (m)
B1	5.4/70.4	9.1/66.7
B2	5.3/70.2	10.1/65.4
B4	5.7/70.1	7.9/67.9
B5	7.6/68.6	8.5/67.7
B6	4.0/71.7	8.1/67.6*
B8	1.2/68.0	2.0/67.2
B9	1.1/70.7	3.0/68.8
B11	2.3/66.7	4.1/64.9
P1	1.9/67.8	3.5/66.2
P3	1.7/67.7	2.7/66.7
P4	1.8/67.5	2.8/66.5

\*Auger refusal at that depth/Elevation-bottom of the deposit was not proven.

The presence of cobbles and boulders should be expected to occur throughout in the till deposit, due to its mode of deposition.

The grain-size distribution of six samples from this granular (non-cohesive) deposit is given in Figure B-6, which shows the following grain-size distribution:

Gravel:	14-32%
Sand:	41-47%
Silt:	16-31%
Clay:	5-14%

An Atterberg limits test was performed on a sample from a relatively more clayey zone in this glacial deposit and the test yielded the following index values (see Figure B-7, Appendix B):

Liquid Limit:	26%
Plastic Limit:	20%
Plasticity Index:	6

These results are characteristic of clayey silt deposits (i.e. low plasticity).

Standard Penetration tests performed in the basically granular soil deposit yielded N-values of 4 to in excess of 100 blows/0.3 m. These results indicate that the relative density of the granular soil can be described as very loose to very dense, but generally compact to very dense.

## 4.8 Bedrock

The bedrock underlying this area of Ottawa is known to consist of grey and black shales of the Billings Formation. The formation belongs to the Upper Ordovician Period and is approximately 460 million years old.

A dark grey to black shale bedrock was encountered in all boreholes except for Borehole B6 which was terminated within the glacial till deposit. The boreholes were advanced into the bedrock by augering and NQ coring. Standard Penetration tests performed in the extremely/highly weathered shale bedrock yielded N-values of 8 to in excess of 100 blows/0.3 m but mostly in excess of 100 blows/0.3 m with spoon bouncing. About 2 m top portion of the bedrock in Borehole P2 at a depth of 3.7 m or El. 65.7 m appears to be extremely weathered, as evidenced by N-values of 8 and 24 blows/0.3 m, and visual examination of the split-spoon rock samples. The bedrock was proven by NQ coring in Boreholes B2, B4, B5, P2, P3 and P4 as follows:

**Table 4.8.1: Inferred Bedrock Elevation and Condition**

Borehole No.	Ground Surface Elevation (m)	Bedrock surface depth/ elevation (m)	Penetration into Bedrock by augering Depth (m)	Length of coring (m)	T.C.R. (%)**	R.Q.D. (%)***
B1	75.8	9.1/66.7	9.1-12.3			
B2	75.5	10.1/65.4	10.1-11.0	3.3	45-96	0-76
B3	74.9	8.6/66.3	8.6-10.7			
B4	75.8	7.9/67.9	7.9-9.9	3.0	100	93-94
B5	76.2	8.5/67.7	8.5-9.4	3.0	91-100	80-81
B6	75.7	At or below 8.1/67.6*	-			
B8	69.2	2.0/67.2	2.0-5.4			
B9	71.8	3.0/68.8	3.0-6.2			
B10	70.4	3.9/66.5	3.9-5.5			
B11	69.0	4.1/64.9	4.1-5.4			
P1	69.7	3.5/66.2	3.5-7.7			
P2	69.4	3.7/65.7	3.7-6.4	3.3	59-95	0-72
P3	69.4	2.7/66.7	-	3.2	29-88	18-66
P4	69.3	2.8/66.5	-	3.1	73-79	26-37

\*borehole was terminated within the overburden

\*\* T.C.R.=Total Core Recovery

\*\*\*R.Q.D.=Rock Quality Designation

As shown in the above table and on the individual record of borehole sheets, the percentage of rock core recovery was 29 to 100% while the RQD values vary from 0% to 94%. These results indicate a range rock quality values, from very poor to excellent. In general, these results indicate that bedrock on west side of the bridge (Boreholes B4 and B5) is relatively more sound but on east side of the bridge (Borehole B2), Pier No. 1 location (Boreholes P3 and P4) and Pier No. 2 location (Borehole P2), the bedrock appears to be extremely/highly weathered. In general, the bedrock appears to be highly weathered in this area due to the frost penetration into the bedrock during the glacial period.

Unconfined compression tests were performed on selected intact rock samples and the tests yielded unconfined compressive strength of about 37 MPa. These results indicate that the rock samples tested can be classified as being “medium strong”.

An about 0.2 to 0.3 m clayey seam within the bedrock were noted at about Elevation 62.5 and 61.0 m in Boreholes B2 and P2, respectively.

At the east abutment borehole locations the surface of the bedrock was inferred at Elevations ranging from 66.7 m (Borehole B1) to 64.9 m (Borehole B11). From these results the bedrock surface generally appears to dip towards north at about 3 % slope at east abutment location of the proposed bridge.

At the west abutment borehole locations the surface of the bedrock was inferred at Elevations ranging from 68.8 m (Borehole B9) to 67.2 m (Borehole B8). From these results the bedrock surface generally appears to dip towards south at about 2.6 % at west abutment location of the proposed bridge.

At the Pier No. 2 borehole locations the surface of the bedrock was inferred at Elevations of 65.7 m (Borehole P2) and 66.2 m (Borehole P1). From the findings the bedrock surface generally appears to dip towards north at about 2.4 % at Pier No. 2 location of the proposed bridge.

At the Pier No. 1 borehole locations the surface of the bedrock was inferred at Elevations of 66.7 m (Borehole P3) and 66.5 m (Borehole P4). From the findings the bedrock surface generally appears to be relatively flat at Pier No. 1 location of the proposed bridge.

Along the proposed bridge site longitudinally, the bedrock surface appears to dip from the west towards the Pier No. 2 location at the site and then remain relatively level towards the east abutment location.

## 4.9 Groundwater Conditions

Groundwater conditions were observed in the open boreholes while drilling and upon completion of each borehole. In the deep boreholes, where NQ coring was used (i.e. water introduced into the boreholes) the on-completion water levels may not be reliable. The observations made in the boreholes are shown on the individual Record of Borehole Sheets in Appendix A and are summarized in the following table.

**Table 4.9.1: Groundwater condition**

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Piezometer
B1	75.8		Dry*	Upon completion	No
B2	75.5	14.3/61.2	7.5/68.0	Oct 3, 2008	Yes**
B3	74.9		Dry* wet & caved in @ El. 67.4 m	Upon completion	No
B4	75.8		6.7/69.1*	Upon completion	No
B5	76.2	12.3/63.9	6.4/69.8*	Upon completion	Yes
B6	75.7		Dry* caved in @ El. 68.1 m	Upon completion	No
B8	69.2		Dry*	Upon completion	No
B9	71.8		Dry*	Upon completion	No
B10	70.4		Dry*	Upon completion	No
B11	69.0		4.9/64.1*	Upon completion	No

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Piezometer
P1	69.7		Dry*	Upon completion	No
P2	69.4		1.2/68.2*	Upon completion	No
P3	69.4		1.5/67.9*	Upon completion	No
P4	69.3		1.3/68.0*	Upon completion	No

\*groundwater table not stabilized

\*\*damaged

The piezometers were installed in the bedrock, because in our experience, sometimes an upward gradient occurs emanating from within the bedrock. From the measured values, the groundwater level below original grade at the time of investigation was at about El 68 to 70 m. While a perched water condition could possibly be encountered at the site due to the accumulation of the surface water in the fill materials, the measured water levels at the site do not represent a perched condition.

It should be pointed out that the water observed levels represent the conditions at the time of our investigations and that they would be subject to fluctuations, both seasonally and in response to major weather events.

For and on behalf of Coffey Geotechnics Inc.



**Gwangha Roh, Ph.D.**




**Ramon Miranda, P.Eng.**



**Zuhtu Ozden, P.Eng.**



Drawings



METRIC

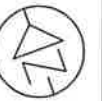
NOTES:

FOR DETAILED SUBSURFACE CONDITIONS  
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.  
GWP: 4011-06-00

HIGHWAY 417 / CYRVILLE ROAD BRIDGE  
OTTAWA, ONTARIO  
BOREHOLE LOCATION PLAN  
AND SOIL STRATA



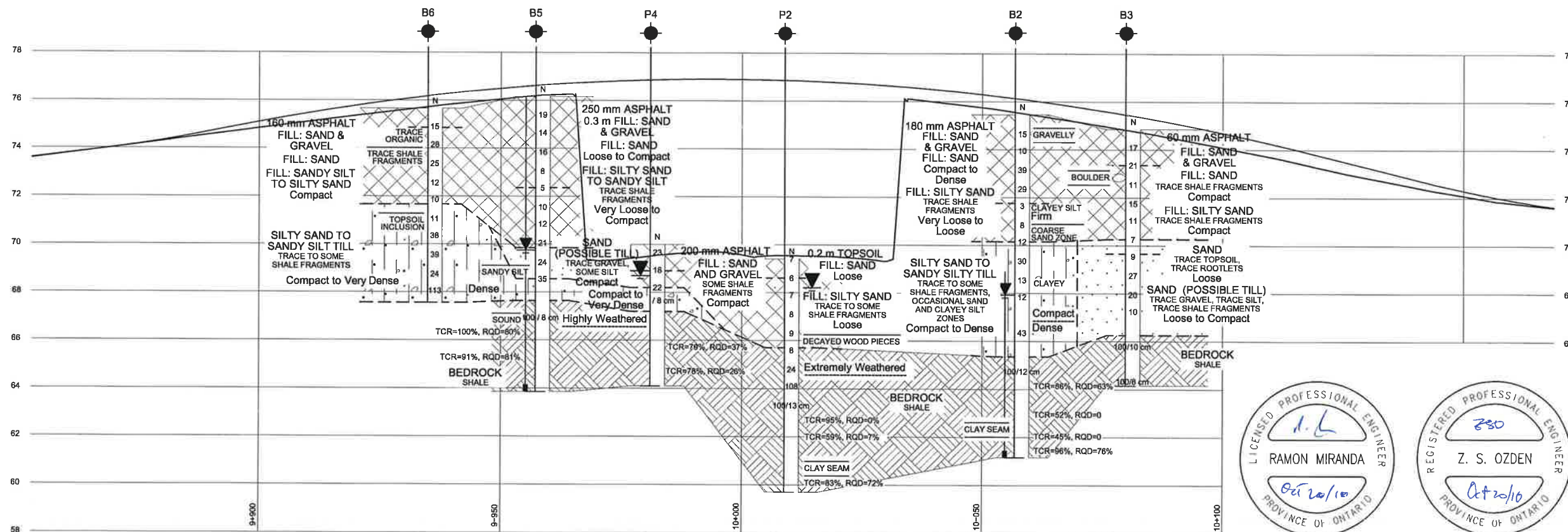
SHEET

coffey geotechnics  
SPECIALISTS MANAGING THE EARTH



KEY PLAN  
N.T.S.

PLAN



PROFILE

HORIZONTAL SCALE



LEGEND

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

No.	ELEVATION (m)	EASTING	NORTHING
B1	75.8	5031701.47	373328.80
B2	75.5	5031705.98	373344.06
B3	74.9	5031698.98	373366.20
B4	75.8	5031736.44	373234.57
B5	76.2	5031741.01	373250.87
B6	75.7	5031748.91	373229.97
B8	69.2	5031713.19	373217.70
B9	71.8	5031762.63	373264.24
B10	70.4	5031684.44	373316.65
B11	69.0	5031726.39	373351.80
P1	69.7	5031715.14	373274.22
P2	69.4	5031727.98	373301.12
P3	69.4	5031724.70	373250.83
P4	69.3	5031737.09	373274.42

-NOTE-

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

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31G5-229

TRANETO01228AA		DIST	
SUBMD	CHECKED	DATE	Oct 20, 2010
DRAWN	SH	CHECKED	RM
APPROVED	ZO	DWG	1





METRIC

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

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

CONT No.  
GWP: 4011-06-00

HIGHWAY 417 / CYRVILLE ROAD BRIDGE  
OTTAWA, ONTARIO  
SOIL STRATA (SECTIONS AT ABUTMENTS)

SHEET

coffey geotechnics  
SPECIALISTS MANAGING THE EARTH



KEY PLAN  
N.T.S.

LEGEND

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

No.	ELEVATION (m)	EASTING	NORTHING
B1	75.8	5031701.47	373328.80
B2	75.5	5031705.98	373344.06
B4	75.8	5031736.44	373234.57
B5	76.2	5031741.01	373250.87

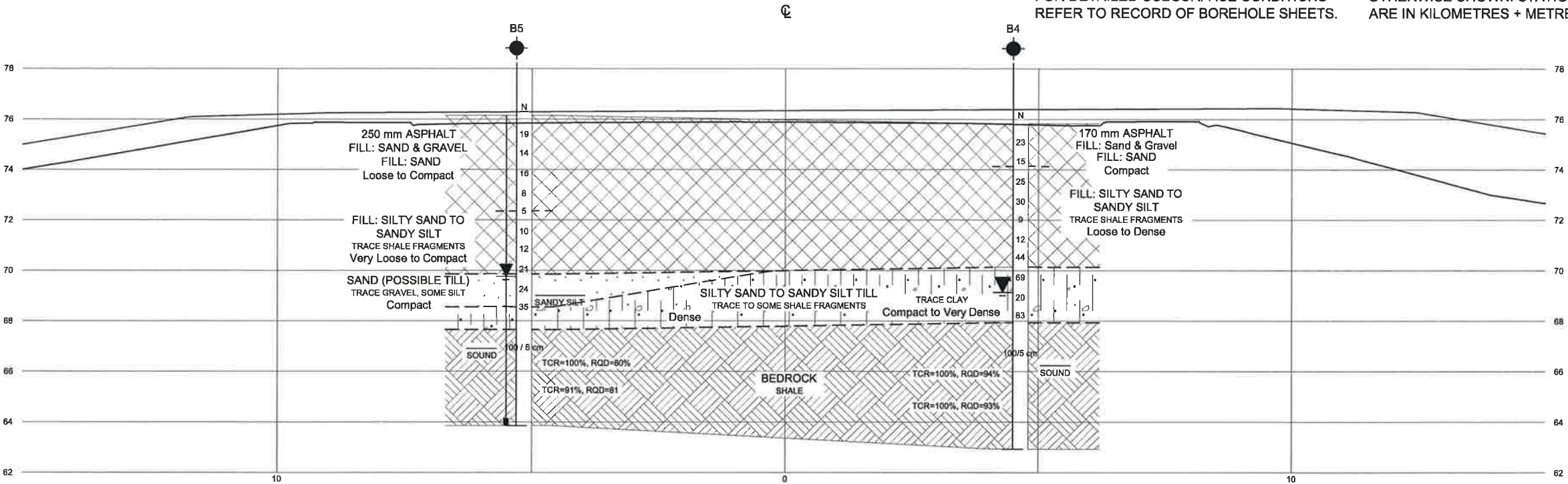
-NOTE-

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

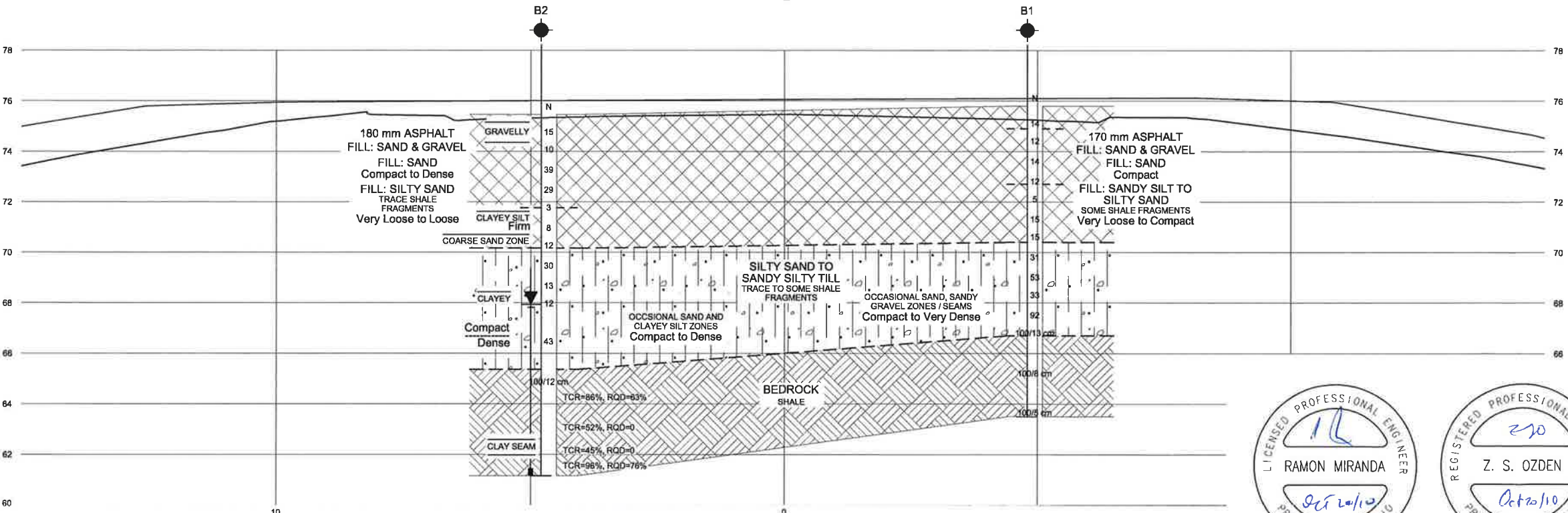
NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31GS-229				TRANET001226AA		DIST	
SUBMD	CHECKED	DATE	Oct 20, 2010	SITE			
DRAWN	SH	CHECKED	RM	APPROVED	ZO	DWG	2



SECTION A-A (WEST ABUTMENT)



SECTION B-B (EAST ABUTMENT)





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: 4011-06-00

HIGHWAY 417 / CYRVILLE ROAD BRIDGE  
OTTAWA, ONTARIO  
SOIL STRATA (SECTIONS AT PIERS)

SHEET

coffey geotechnics  
SPECIALISTS MANAGING THE EARTH



KEY PLAN  
N.T.S.

LEGEND

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

No.	ELEVATION (m)	EASTING	NORTHING
P1	69.7	5031715.14	373274.22
P2	69.4	5031727.98	373301.12
P3	69.4	5031724.70	373250.83
P4	69.3	5031737.09	373274.42

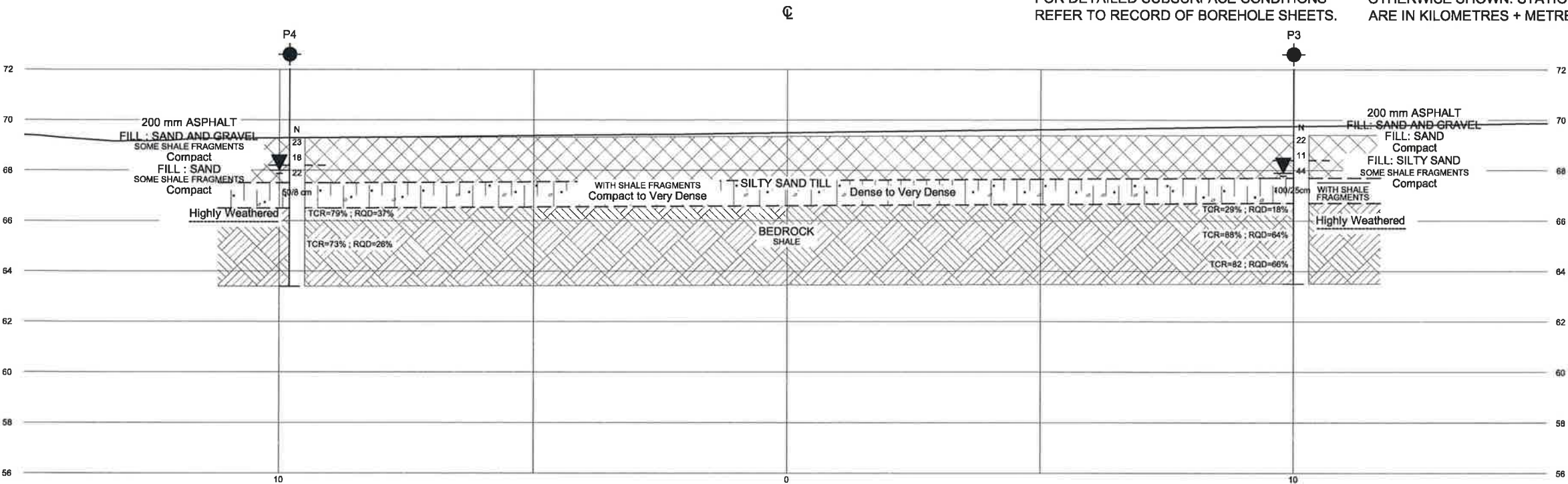
-NOTE-

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

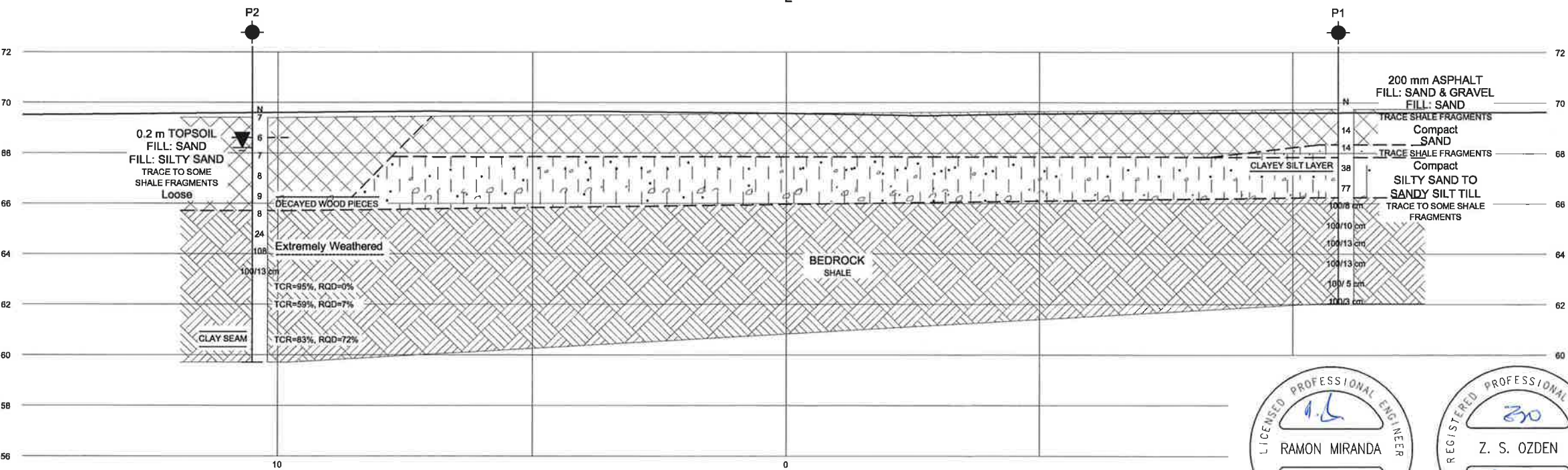
NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31GS-229	TRANETO801226AA	DIST
SUBMD	CHECKED	DATE Oct 20, 2010
DRAWN SH	CHECKED RM	APPROVED ZO
		SITE DWG
		3



SECTION C-C (PIER NO. 1)



SECTION D-D (PIER NO. 2)



# Appendix A

## **Record of Borehole Sheets**

TRANETO01226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No B1

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031701.5 N, 373328.8 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 9/29/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
75.8 0.0	GROUND SURFACE																
	170 mm ASPHALT		1	AS													13 67 (20)
	0.2 m FILL: Sand & Gravel		2	AS													
	FILL: Sand, tr gravel, some silt, brown																
74.9 0.9			3	SS	14		75										
	FILL: Sand, tr. to some silt																
	brown, moist, compact		4	SS	12		74										0 89 (11)
			5	SS	14												
							73										
72.7 3.1	FILL: Sandy Silt to Silty Sand		6	SS	12												
	tr. gravel, some shale fragmens, tr. clay																
	brown to dark brown, v. loose to compact,		7	SS	5		72										
	moist																
			8	SS	15		71										20 48 22 10
																	gravel @ spoon
																	tip
70.4 5.4	SILTY SAND TO SANDY SILT TILL		9	SS	15		70										
	tr. shale fragments, occ. sand, sandy gravel																
	zones/seams		10	SS	31												
	compact to v. dense																
							69										augering hard
			11	SS	53												below 6.6 m
			12	SS	33		68										
			13	SS	92												
							67										
66.7 9.1			14	SS 100 / 13 cm													spoon bouncing
	BEDROCK						66										
	dark grey to black, shale																
			15	SS 100 / 8 cm			65										
							64										
63.5 12.3			16	SS 100 / 5 cm													
	End of borehole																
	Borehole open & dry (not stabilized) upon																
	completion																

+<sup>3</sup> × 3<sub>2</sub>

Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

TRANETO01226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No B10

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031684.4 N, 373316.7 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger & NW Casing COMPILED BY SD  
DATUM Geodetic DATE 10/6/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE							
70.4	GROUND SURFACE							20 40 60 80 100							
0.0	0.2 m TOPSOIL						70								
70.0	FILL: Silty Sand		1	SS	14										
0.4	tr. organic, dark grey, compact, moist														
	FILL: Sandy Silt to Silty Sand		2	SS	22										
	tr. gravel, tr. shale fragments														
	dark grey, compact, moist														
	some organic		3	SS	18		69								sewer smell
	possible cobbles		4	SS	18		68								Auger grinding
			5	SS	15		67								
66.5			6	SS	100/10 cm										Augering hard below 3.8 m
3.9															
	BEDROCK														
	dark grey to black shale														Spoon bouncing
			7	SS	100/10 cm		66								
64.9			8	SS	100/8 cm		65								
5.5															
	End of borehole														
	Borehole dry (not stabilized) and open upon completion.														

+ 3, × 3, ÷ 3

Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

TRANETOBO1226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No B11

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031726.4 N, 373351.8 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 9/22/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE								WATER CONTENT (%)
69.0 0.0	GROUND SURFACE						69	20	40	60	80	100	10	20	30	
	0.2 m TOPSOIL		1	SS	7								○			
		loose														
		compact	2	SS	10		68						○			
	FILL: Silty Sand to Sandy Silt some gravel, tr. shale fragments dark brown, moist		3	SS	14								○			
66.7 2.3							67									
	SILTY SAND TO SANDY SILT TILL tr. shale fragment, dark grey	v. loose	4	SS	4									○		
		compact	5	SS	28		66						○			
		v. dense	6	SS	125											
64.9 4.1	BEDROCK dark grey to black shale		7	SS	100/8 cm		65						○			
63.6 5.4	End of borehole Water level in borehole @ 4.9 m (not stabilized)* upon completion Borehole caved-in @ 4.9 m upon completion		8	SS	100/5 cm		64									

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

20  
15 10 5  
(%) STRAIN AT FAILURE

TRANETOBO1226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No B2

1 OF 2

METRIC

GWP 4011-06-00 LOCATION 5031706.0 N, 373344.1 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger & NW Casing & NQ coring COMPILED BY SD  
DATUM Geodetic DATE 10/1/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m <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		WATER CONTENT (%) w <sub>P</sub> w   w <sub>L</sub>				
75.5 0.0	GROUND SURFACE							20   40   60   80   100						
	180 mm ASPHALT		1	AS										
	0.3 m FILL: Sand & Gravel		2	AS										
	FILL: Sand tr. gravel, brown	gravelly	3	SS	15									
	compact to dense, moist		4	SS	10									
			5	SS	39									
			6	SS	29									
71.8 3.7	FILL: Silty Sand		7	SS	3									
	tr. to some gravel, tr. shale fragments													
	tr. to some clay, brown to dark brown	clayey silt, firm	8	SS	8									0 49 27 24
	v. loose to loose, moist	coarse sand zone												
70.2 5.3	SILTY SAND TO SANDY SILT TILL		9	SS	12									
	tr. to some shale fragments													
	occ. sand and clayey silt zones		10	SS	30									
	compact to dense													
		moist	11	SS	13									spoon wet
		wet												
		clayey	12	SS	12									14 42 30 14 spoon wet
		compact												auger grinding augering hard below
		dense	13	SS	43									
65.4 10.1	BEDROCK		14	SS	100/120									Augering v. hard below 10.1 m
	dark grey to black shale													
			15	RC	TCR=86% RQD=63%									Auger refusal @ 11.0 m
			16	RC	TCR=52% RQD=0									
		clay seam												
			17	RC	TCR=45% RQD=0									
61.2 14.3	End of borehole		18	RC	TCR=96% RQD=76%									
	Water level @ 6.7 m (not stabilized)* upon completion													

Continued Next Page

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







TRANETO01226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No B3

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031699.0 N, 373366.2 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 10/2/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
74.9 0.0	GROUND SURFACE		1	AS													GR SA SI CL
	60 mm ASPHALT 0.1 m FILL: Sand & Gravel FILL: Sand, some gravel, some silt, tr. to some clay, tr. shale fragments brown, compact, moist		2	AS													
73.4 1.5			3	SS	17												
			4	SS	21												15 50 21 12
			5	SS	11												borehole moved 1.5 m south from original borehole location due to possible boulder @ 2.1 m
	FILL: Silty Sand tr. shale fragments, some clay brown to grey, compact, moist	boulder	6	SS	15												
			7	SS	11												
70.3 4.6	SAND tr. topsoil, tr. rootlets brown, loose, moist		8	SS	7												
69.7 5.2			9	SS	9												
	SAND (Possible Till) tr. gravel, tr. silt, tr. shale fragments grey to dark grey, loose to compact		10	SS	27												
			11	SS	20												12 81 (7) spoon wet
		moist	12	SS	10												
		wet															
66.3 8.6	BEDROCK dark grey to black, shale		13	SS	100/10 cm												augering hard below 8.5 m spoon bouncing
64.2 10.7	End of borehole Borehole caved-in @ 7.5 m* upon completion Borehole dry (not stabilized) upon completion		14	SS	00/8 cm												

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

TRANETO01226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No B4

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031736.4 N, 373234.6 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger & NW Casing & NQ coring COMPILED BY SD  
DATUM Geodetic DATE 9/30/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
75.8 0.0	GROUND SURFACE																
	170 mm ASPHALT		1	AS													
	0.3 m FILL: Sand & Gravel		2	AS													
	FILL: Sand		3	SS	23												
	tr. gravel, tr. silt																
	brown, compact, damp																
74.1 1.7			4	SS	15												
	FILL: Silty Sand to Sandy Silt		5	SS	25												
	tr. gravel, tr. shale fragments		6	SS	30												
	tr. clay, occ. sand layer		7	SS	9												
	brown to blackish brown		8	SS	12												
	loose to dense		9	SS	44												
70.1 5.7	SILTY SAND TO SANDY SILT TILL		10	SS	69												
	tr. to some shale fragments, tr. clay		11	SS	20												
	brown to blackish brown		12	SS	83												
	compact to v. dense, moist to wet																
67.9 7.9	BEDROCK		13	SS	100/5 on												
	dark gray to black shale		14	RCTCR=100%													
				RQD=94%													
			15	RCTCR=100%													
				RQD=93%													
62.9 12.9	End of borehole																
	Water level @ 6.7 m (not stabilized)* upon																
	completion																
	Borehole caved-in @ 8.4 m upon completion																

+ 3, x 3

Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

TRANETO01226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No B5

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031741.0 N, 373250.9 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger & NW Casing & NQ coring COMPILED BY SD  
DATUM Geodetic DATE 10/3/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
76.2 0.0	GROUND SURFACE																
	250 mm ASPHALT 0.3 m FILL: Sand & Gravel FILL: Sand tr. to some gravel, tr. silt loose to compact, moist		1	AS			76										
			2	AS													
			3	SS	19		75										
			4	SS	14		74										
			5	SS	16		73										
			6	SS	8		72										
72.4 3.8	FILL: Silty Sand to Sandy Silt tr. gravel, tr. shale fragments occ. sand zones, brown to dark brown v. loose to compact, moist		7	SS	5		71										
			8	SS	10		70										
			9	SS	12		69										
69.9 6.3	SAND (Possible Till) tr. gravel, some silt, greyish brown, compact	moist wet	10	SS	21		68										
68.6 7.6	SANDY SILT tr. shale fragments, dense, moist	sandy silt	11	SS	24		67										
67.7 8.5	SILT SAND TO SANDY SILT TILL tr. shale fragments, dense, moist		12	SS	35		66										
			13	SS	100/8 cm		65										
	BEDROCK dark grey to black shale	sound	14	RC	TCR=100% RQD=80%		64										
			15	RC	TCR=91% RQD=80%												
63.9 12.3	End of borehole Water level @ 6.4 m (not stabilized)* upon completion Piezometer installed to 12.3 m																

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

TRANETOBO1226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No B6

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031748.9 N, 373230.0 E ORIGINATED BY ZI  
 DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
 DATUM Geodetic DATE 10/2/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
75.7 0.0	GROUND SURFACE													
74.9 0.8	160 mm ASPHALT 0.3 m FILL: Sand & Gravel FILL: Sand, some gravel													
	tr. organic tr. shale fragments		1	SS	15									
			2	SS	28									
	FILL: Sandy Silt to silty sand trace organics, dark brown moist, compact		3	SS	25									14 58 16 12
			4	SS	12									
71.7 4.0	topsoil inclusion		5	SS	10									
	SILTY SAND TO SANDY SILT TILL tr. to some shale fragments, brown to dark brown compact to v. dense, moist		6	SS	11									spoon wet
			7	SS	38									
			8	SS	39									augering hard below 5.8 m
			9	SS	24									23 42 23 12
67.6 8.1	End of borehole Auger refusal on possible bedrock @ 8.1 m Borehole dry upon completion borehole caved-in @ 7.6 m* upon completion		10	SS	113									

TRANETOBO1226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No B8

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031713.2 N, 373217.7 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 9/24/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
69.2 0.0	GROUND SURFACE																
	0.5 m TOPSOIL (dark brown sandy silt) FILL: Sandy Silt tr. shale fragments, tr. organic grey to dark grey, v. loose to compact, moist		1	SS	4		69										
68.0 1.2			2	SS	16		68										
67.2 2.0	SILTY SAND TO SANDY SILT TILL tr. shale fragments, compact to v. dense		3	SS	72												
			4	SS	100/5 cm		67										
	BEDROCK dark grey to black shale		5	SS	100/13 cm		66										
			6	SS	100/15 cm		65										
			7	SS	100/10 cm												
63.8 5.4			8	SS	100/10 cm		64										
	End of borehole Borehole open & dry (not stabilized) upon completion.																

+ 3 × 3

Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE


TRANETOB01226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No B9

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031762.6 N, 373264.2 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SD  
DATUM Geodetic DATE 9/26/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							
71.8 0.0	GROUND SURFACE														
71.3 0.5	0.2 m TOPSOIL SAND tr. gravel		1	SS	7										
70.7 1.1	SILT tr. to some sand loose to compact, wet		2	SS	13										
	SILTY SAND TO SANDY SILT TILL trace shale fragments, occ. sand pockets dark grey, compact to dense, moist		3	SS	39										
			4	SS	39										
68.8 3.0			BEDROCK dark grey to black shale	5	SS	100/10 cm									
	6			SS	100/10 cm										
	7			SS	100/8 cm										
	8			SS	100/13 cm										
65.6 6.2	End of borehole Borehole dry (not stabilized) & open upon completion	9	SS	100/8 cm											

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

20  
15 5  
10 (%) STRAIN AT FAILURE

TRANETOB01226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No P1

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031715.1 N, 373274.2 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger COMPILED BY ZI  
DATUM Geodetic DATE 9/25/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
69.7 0.0	GROUND SURFACE																
	200 mm ASPHALT		1	AS													
	0.4 m FILL: Sand & Gravel		2	AS													
	FILL: Sand																
	some gravel, tr. shale fragments		3	SS	14												
68.3 1.4																	
67.8 1.9	SAND		4	SS	14												
	tr. shale fragments																
	brown, compact, moist																
	clayey silt layer, we		5	SS	38												
	SILTY SAND TO SANDY SILT TILL		6	SS	77												
	tr. to some shale fragments																
66.2 3.5			7	SS	100/8 cm												
	BEDROCK		8	SS	100/10 cm												
	dark grey to black shale		9	SS	100/13 cm												
			10	SS	100/13 cm												
			11	SS	100/5 cm												
			12	SS	100/8 cm												
62.0 7.7	End of borehole																
	Borehole dry (not stabilized) & open upon completion																

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

20  
15  
10  
5  
(%) STRAIN AT FAILURE

TRANETO01226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No P2

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031728.0 N, 373301.1 E ORIGINATED BY ZI  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Auger & NW Casing & NQ Coring COMPILED BY ZI  
DATUM Geodetic DATE 9/24/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
69.4 0.0	GROUND SURFACE																
68.6 0.8	0.2 m TOPSOIL FILL: Sand, some silt, some gravel, tr. organics brown, loose		1	SS	7		69										
			2	SS	6		68										
	FILL: Silty Sand tr. gravel, tr. to some shale fragments, tr. clay, brown to dark brown loose, wet		3	SS	7												
			4	SS	8		67										
			5	SS	9		66										
65.7 3.7	decayed wood pieces		6	SS	8		65										
			7	SS	24		64										
	extremely weathered		8	SS	108		63										
			9	SS	100/13.0m		62										
	BEDROCK dark grey to blackish shale		10	RC	TCR=95% RQD=0%		61										
			11	RC	TCR=59% RQD=7%		60										
	clay seam		12	RC	TCR=83% RQD=72%												
59.7 9.7	End of borehole Water level @ 1.2 m (not stabilized)* upon completion borehole caved-in @ 4.3m upon completion																

+ 3, × 3, Numbers refer to  
Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE



TRANETOBO1226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No P3

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031724.7 N, 373250.8 E ORIGINATED BY RK  
 DIST HWY 417 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SK  
 DATUM Geodetic DATE 3/30/2010 CHECKED BY RM

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 (%)
FLEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
69.4 0.0	GROUND SURFACE																
	200 mm ASPHALT		1	AS													
	0.6 m FILL : sand and gravel		2	SS	22		69										41 54 (5)
68.4 1.0	FILL : silty sand		3	SS	11												13 45 33 9
67.7 1.7	some gravel, some shale fragments		4	SS	44		68										32 41 22 5
	grey, compact, moist																
	SILTY SAND TILL		5	SS	100/25 cm		67										spoon wet
66.7 2.7	grey, dense to v. dense, moist																36 47 12 5
	with shale fragments		6	RC	TCR=23% ; RQD=18%		66										auger refusal and
	dark grey to black, wet																NQ Coring began
	highly weathered		7	RC	TCR=88% ; RQD=64%		65										@ 2.7 m
	BEDROCK		8	RC	TCR=82% ; RQD=66%		64										
	dark grey to black shale																
63.5 5.9	End of Borehole																
	Water level @ 1.5 m (not stabilized)* upon																
	completion																
	Borehole caved-in @ 1.8 m upon completion																

TRANETOB01226AA : Highway 417, Cyrville Bridge, Ottawa

# RECORD OF BOREHOLE No P4

1 OF 1

METRIC

GWP 4011-06-00 LOCATION 5031737.1 N, 373274.4 E ORIGINATED BY RK  
DIST HWY 417 BOREHOLE TYPE Hollow Stem Augers, NQ Coring COMPILED BY SK  
DATUM Geodetic DATE 3/31/2010 CHECKED BY RM

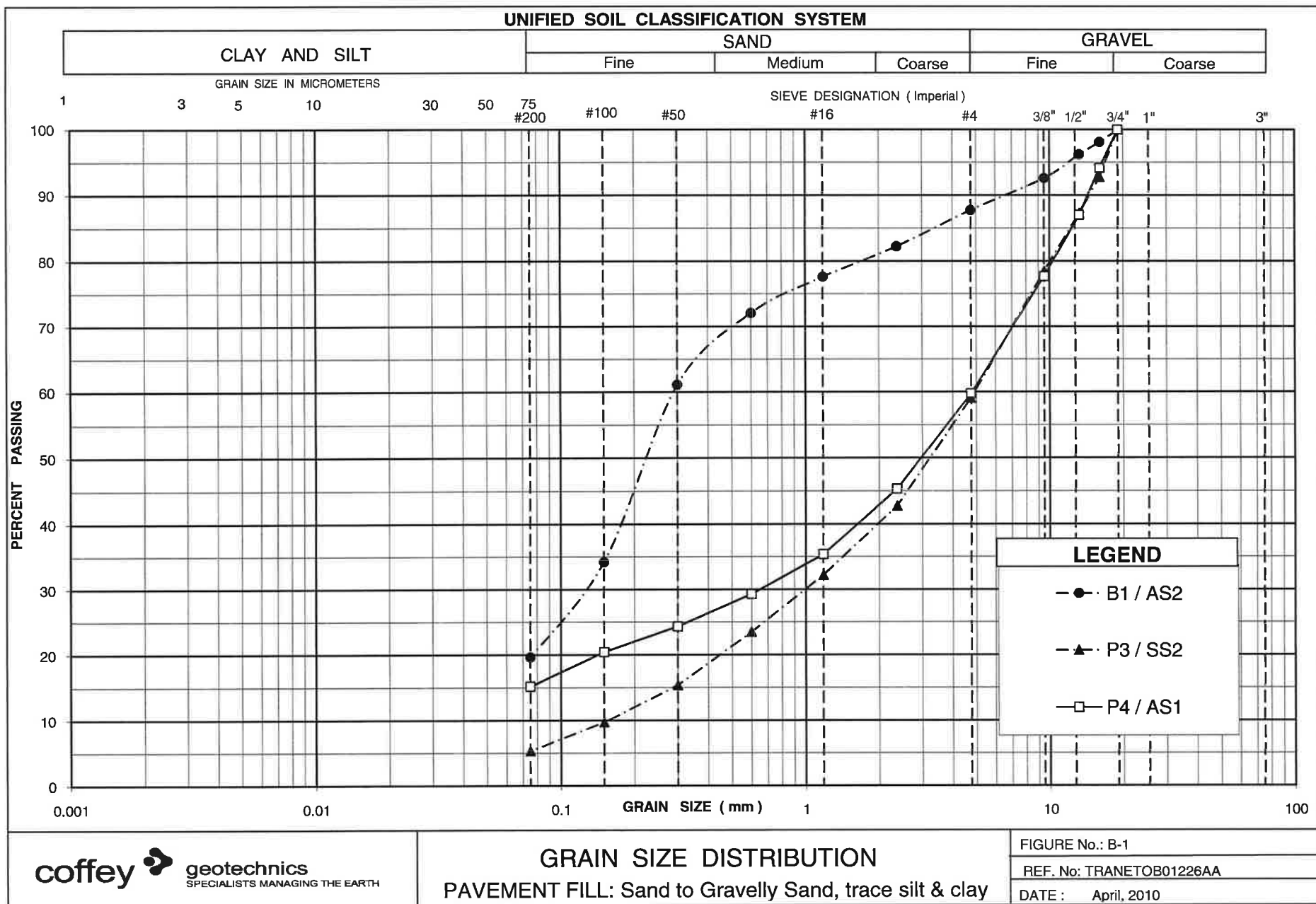
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
69.3	GROUND SURFACE													
0.0	200 mm ASPHALT		1	AS			69							40 45 (15)
	0.6 m FILL: Sand and Gravel		2	SS	23									
68.2	FILL: Sand and Gravel													
1.1	some silt, some shale fragments		3	SS	18		68							spoon wet
67.5	grey, compact, moist to wet		4	SS	22									24 60 (16)
1.8			5	SS	50/8 cm		67							spoon wet
66.5	SILTY SAND TILL													32 47 16 5
2.8	with shale fragments													augering
	grey, compact to v. dense, wet to moist													continued slowly
66.5														up to refusal @
2.8	highly weathered		6	RC			66							2.8m
	BEDROCK													NQ Coring began
	dark grey to black shale		7	RC			65							@ 2.8 m
63.4							64							
5.9	End of borehole													
	water level @ 1.3 m (not stabilized)* upon													
	completion													
	Borehole caved-in @ 1.7 m upon completion													

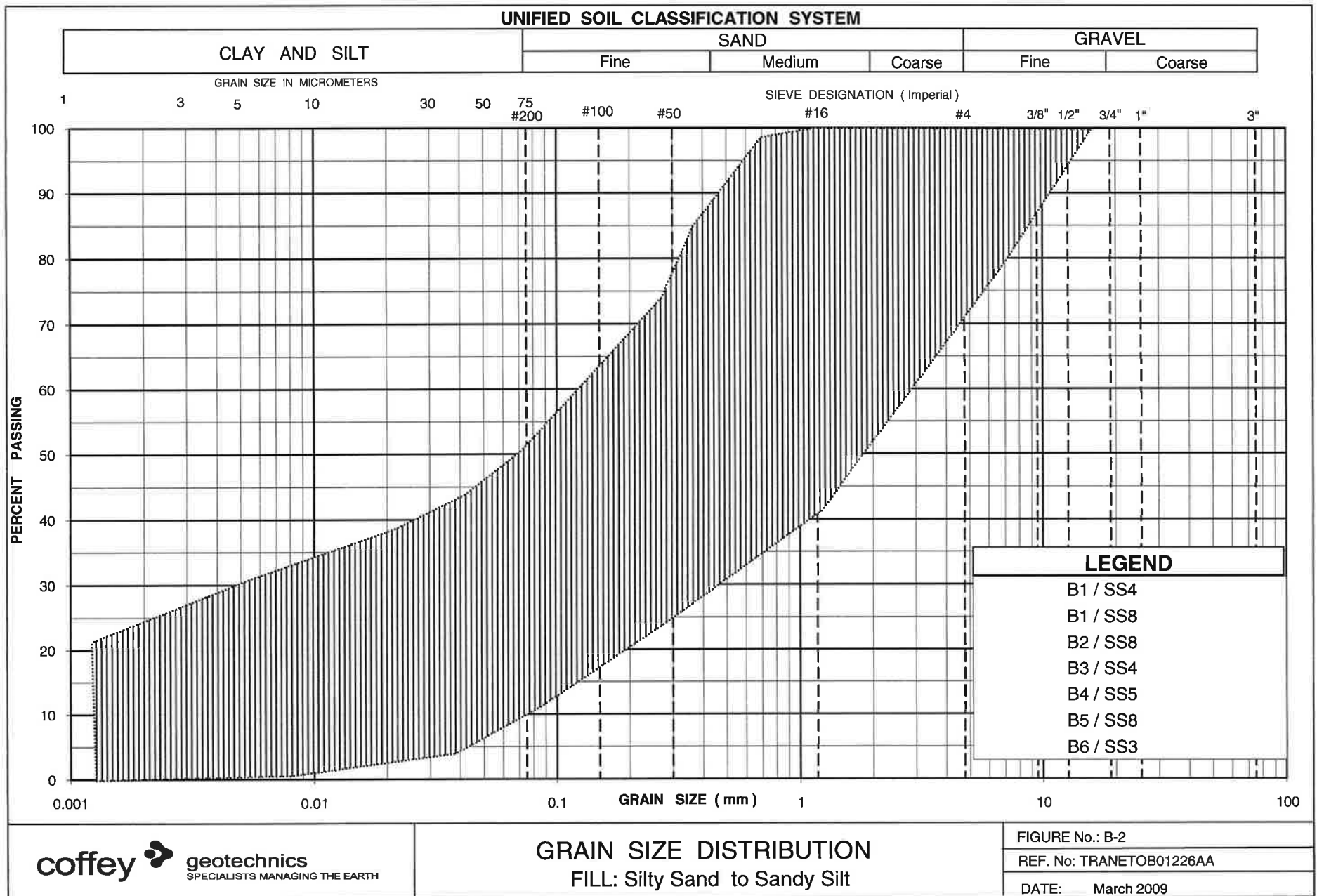
+ 3, x 3 Numbers refer to  
Sensitivity

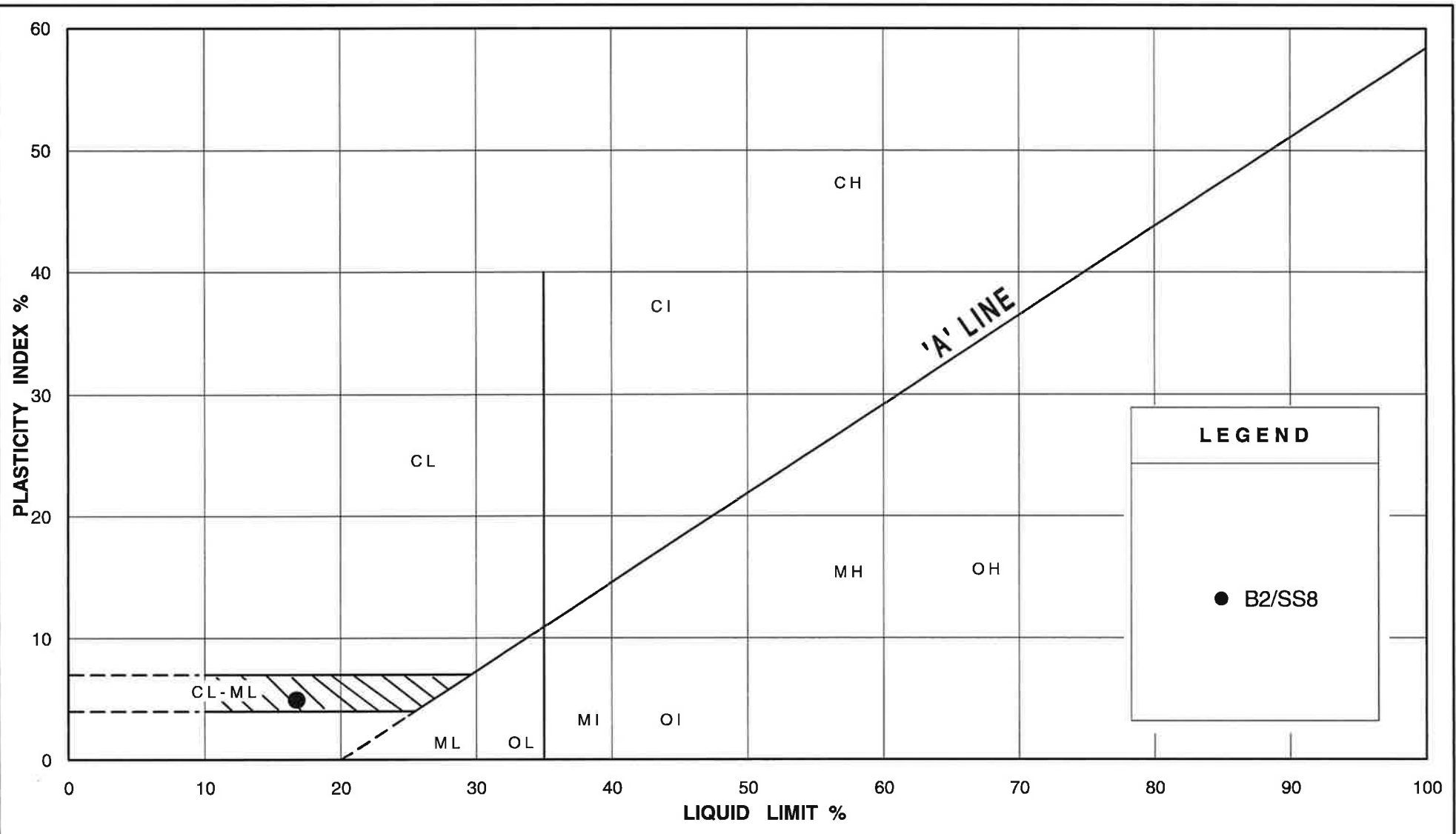
20  
15 5  
10 (%) STRAIN AT FAILURE

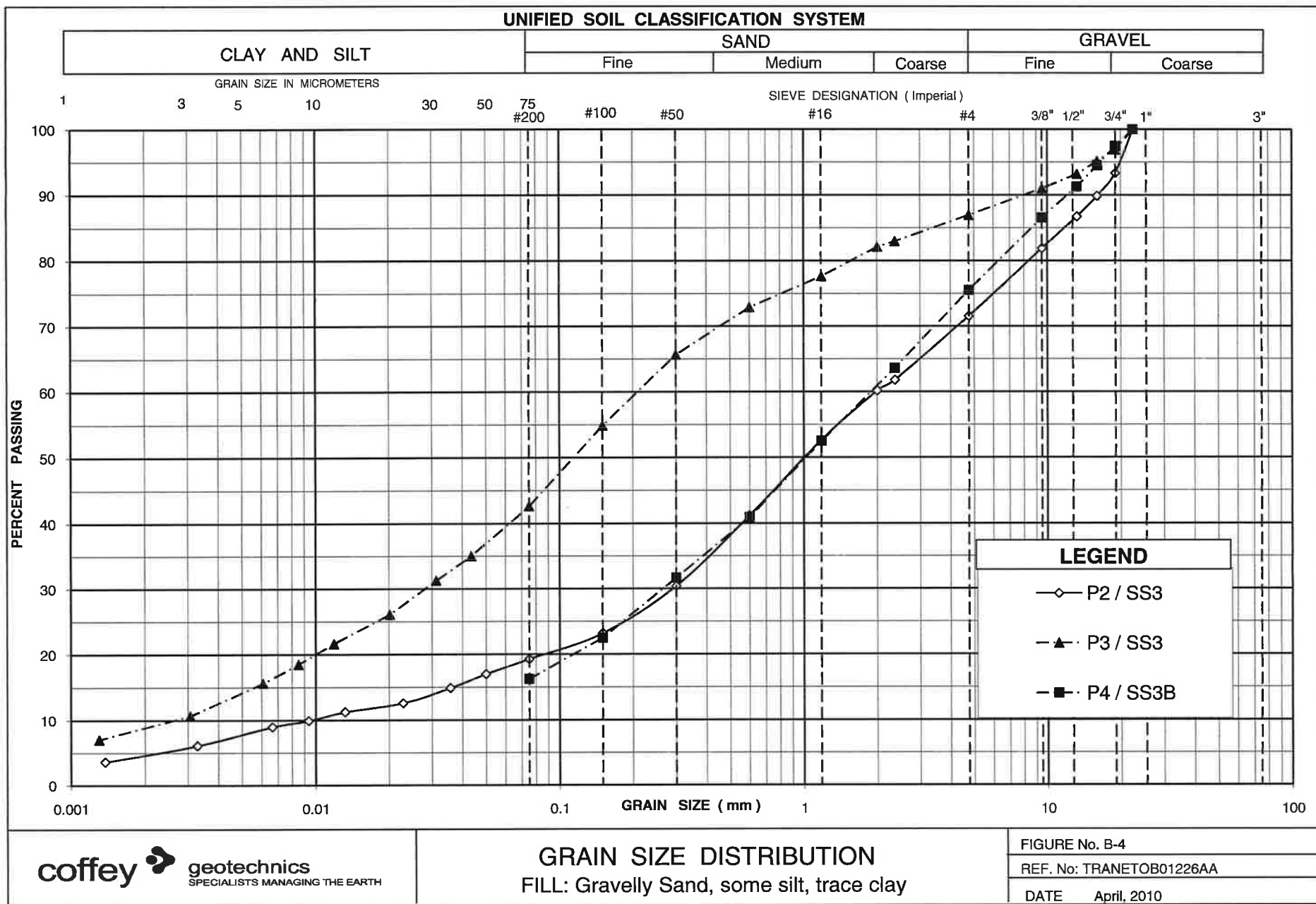
# Appendix B

## Laboratory Test Results

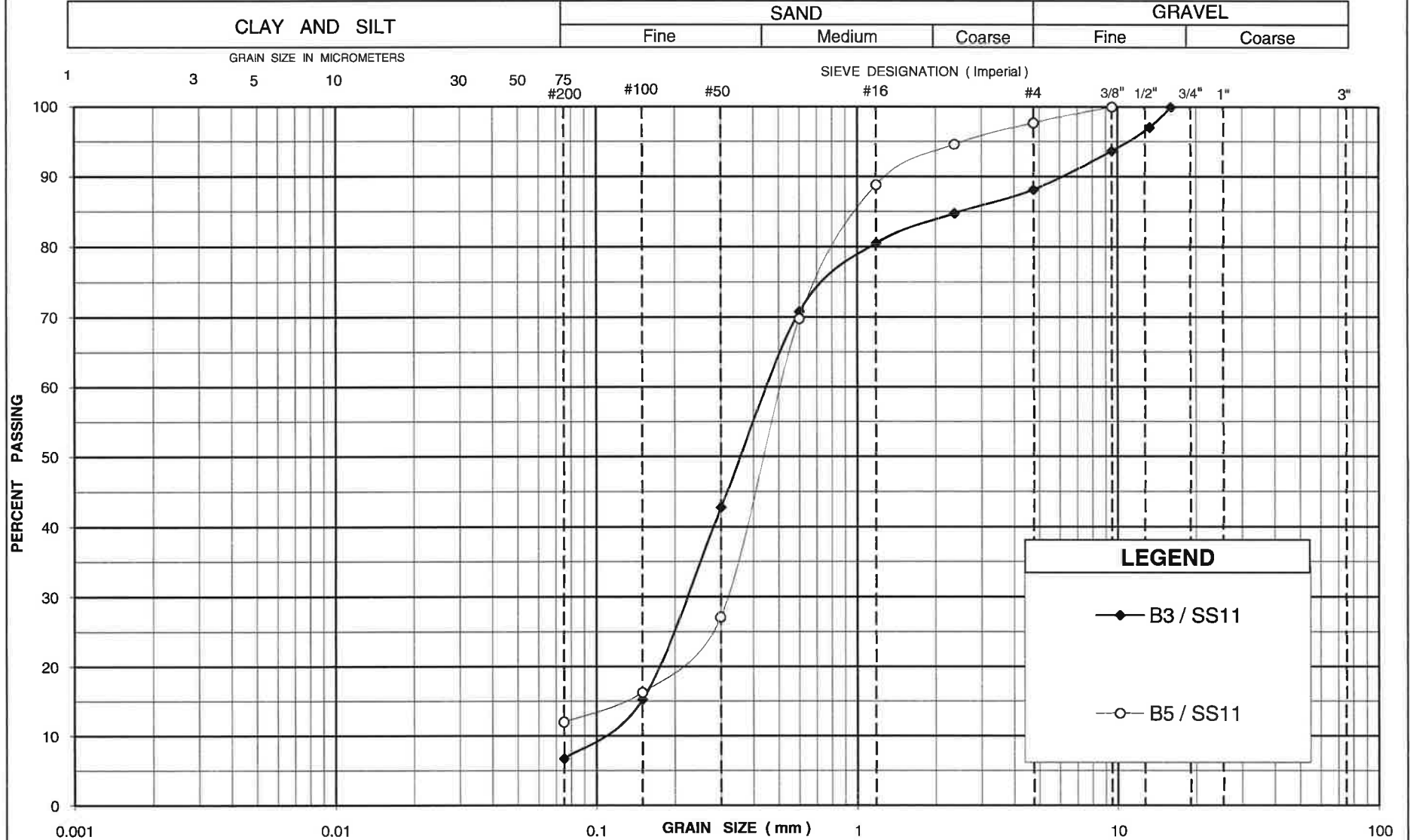




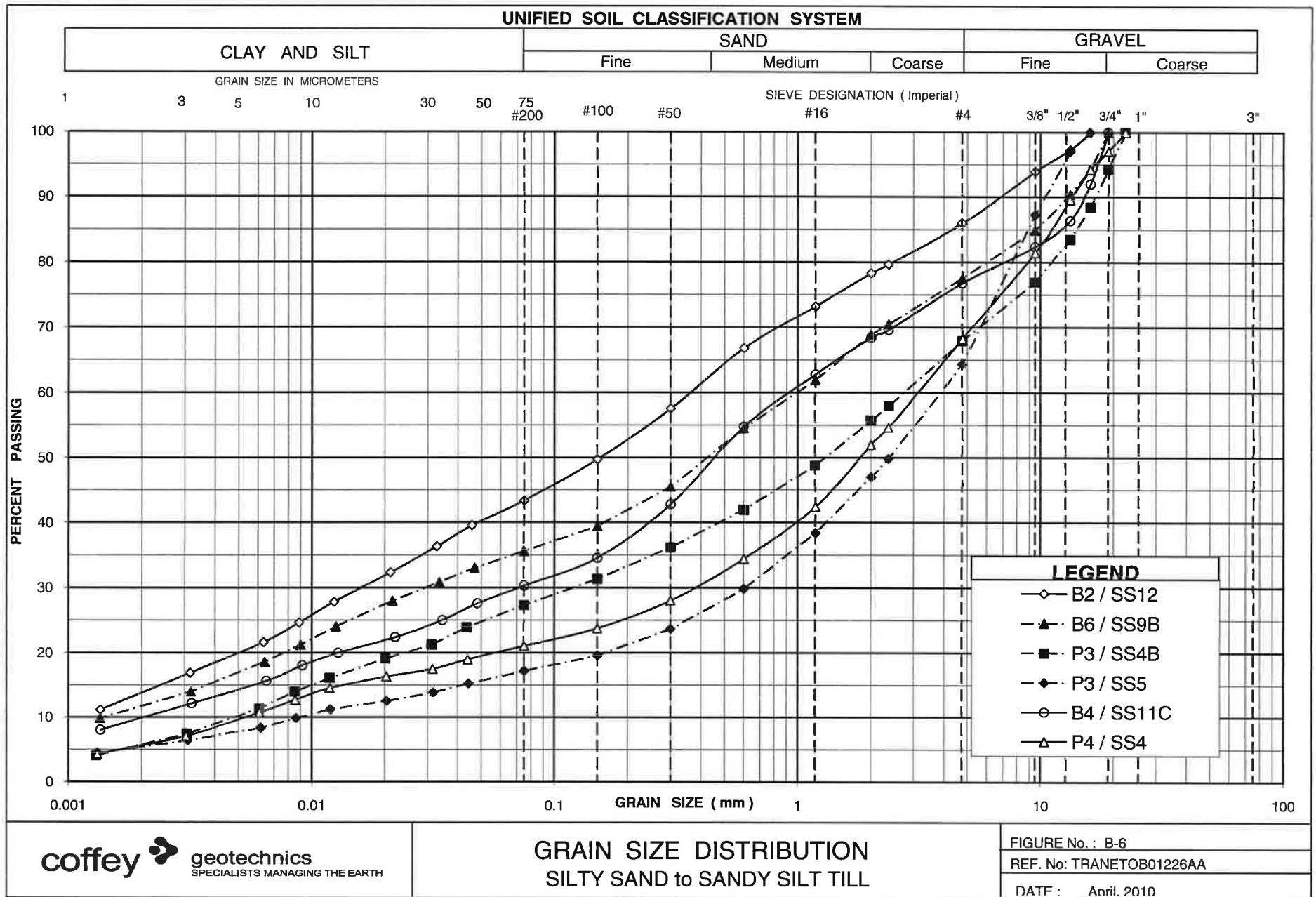


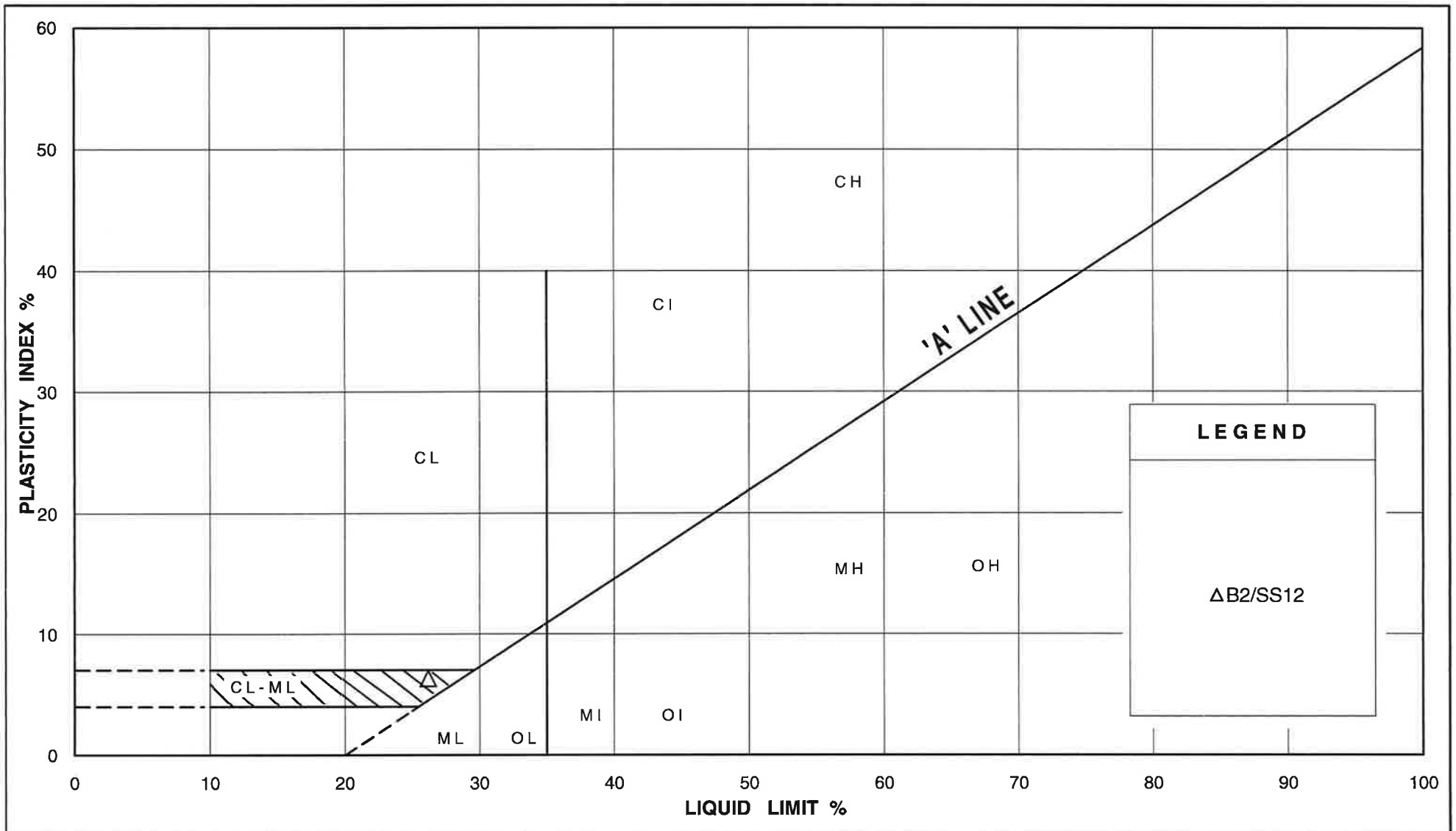


# UNIFIED SOIL CLASSIFICATION SYSTEM









## PLASTICITY CHART

SILTY SAND to SANDY SILT TILL (zone with relatively higher clay content)

FIGURE No.: B-7

REF. No.: TRANETOB01226AA

DATE: March, 2009

# UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-04

## SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1116-0005	SAMPLE NUMBER	RC15
BOREHOLE NUMBER	B2	SAMPLE DEPTH, m	11.40-11.53

## TEST CONDITIONS

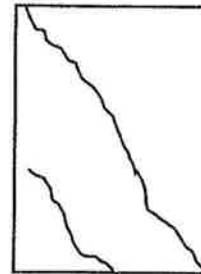
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.36

## SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	11.22	WATER CONTENT, (specimen) %	1.60
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m <sup>3</sup>	24.20
SAMPLE AREA, cm <sup>2</sup>	17.68	DRY UNIT WT., kN/m <sup>3</sup>	23.82
SAMPLE VOLUME, cm <sup>3</sup>	198.41	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	489.80	VOID RATIO	0.11
DRY WEIGHT, g	482.09		

## VISUAL INSPECTION

## FAILURE SKETCH



## TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	37.7
----------------------	---	-------------------------	------

REMARKS: Existing cracks, very fragile.

DATE: 2/27/2009

Checked By: *hli*

**Golder Associates**

# UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-04

## SAMPLE IDENTIFICATION

PROJECT NUMBER	09-1116-0005	SAMPLE NUMBER	RC15
BOREHOLE NUMBER	B4	SAMPLE DEPTH, m	11.60-11.93

## TEST CONDITIONS

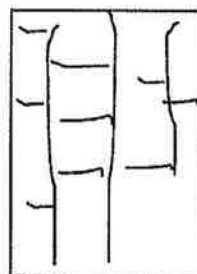
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.32

## SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.97	WATER CONTENT, (specimen) %	1.36
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m <sup>3</sup>	24.58
SAMPLE AREA, cm <sup>2</sup>	17.53	DRY UNIT WT., kN/m <sup>3</sup>	24.25
SAMPLE VOLUME, cm <sup>3</sup>	192.35	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	482.40	VOID RATIO	0.09
DRY WEIGHT, g	475.93		

## VISUAL INSPECTION

## FAILURE SKETCH



## TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	37.3
----------------------	---	-------------------------	------

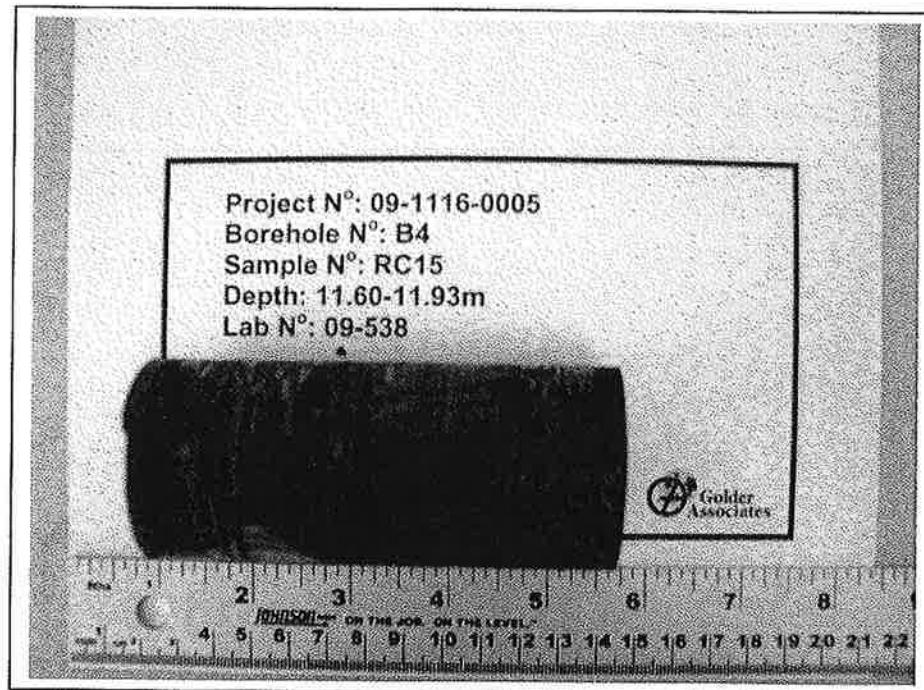
REMARKS:	Existing cracks, very fragile.	DATE:	2/27/2009
----------	--------------------------------	-------	-----------

Checked By: *MM*

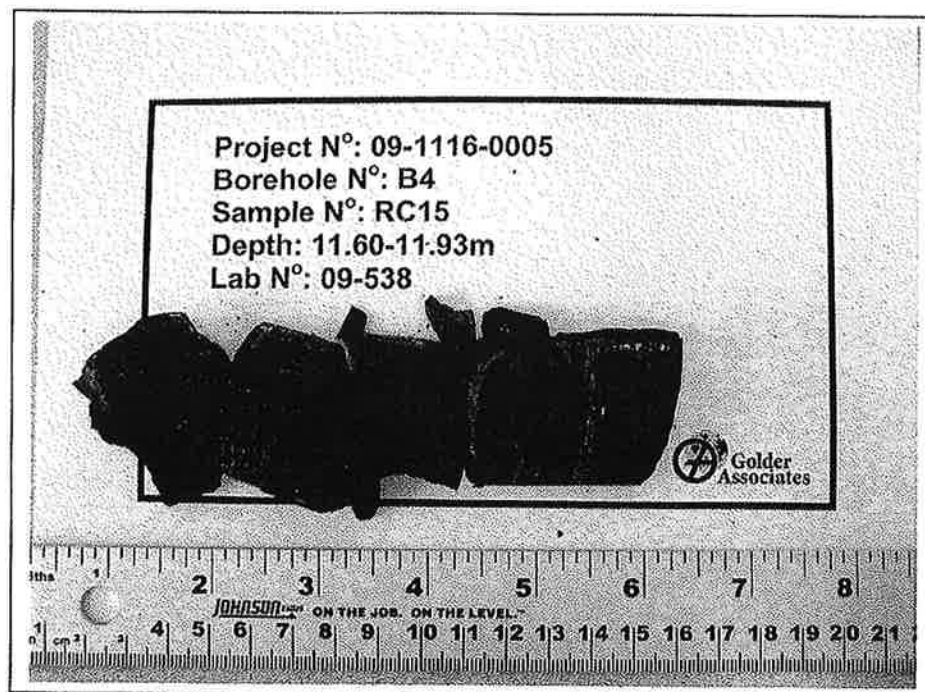
Golder Associates

**UNCONFINED COMPRESSION TEST**  
ASTM D2166-98A

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

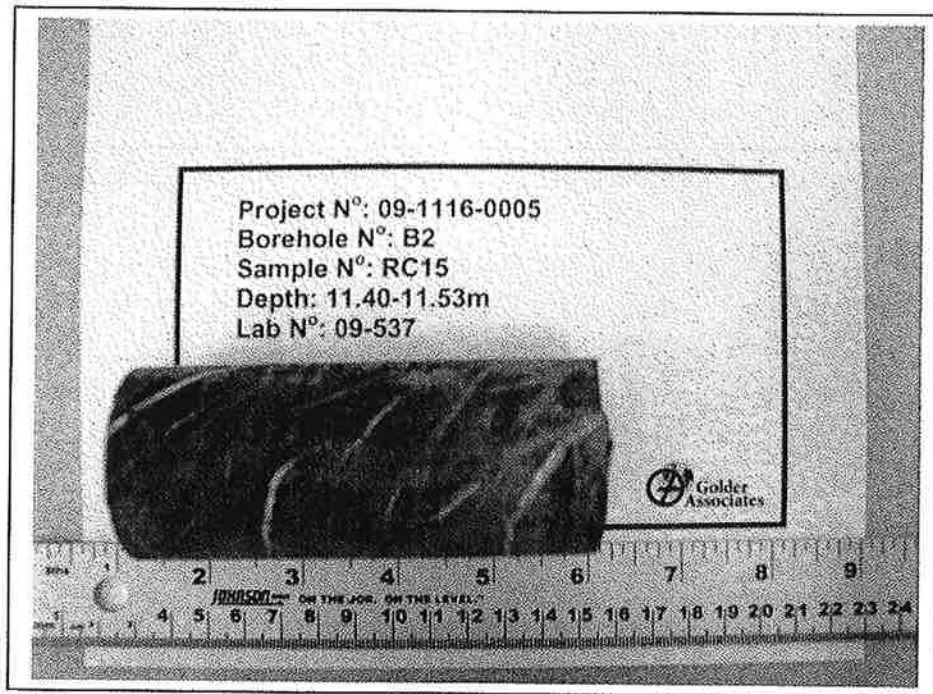
Date 2/27/2009  
Project 09-1116-0005

**Golder Associates**

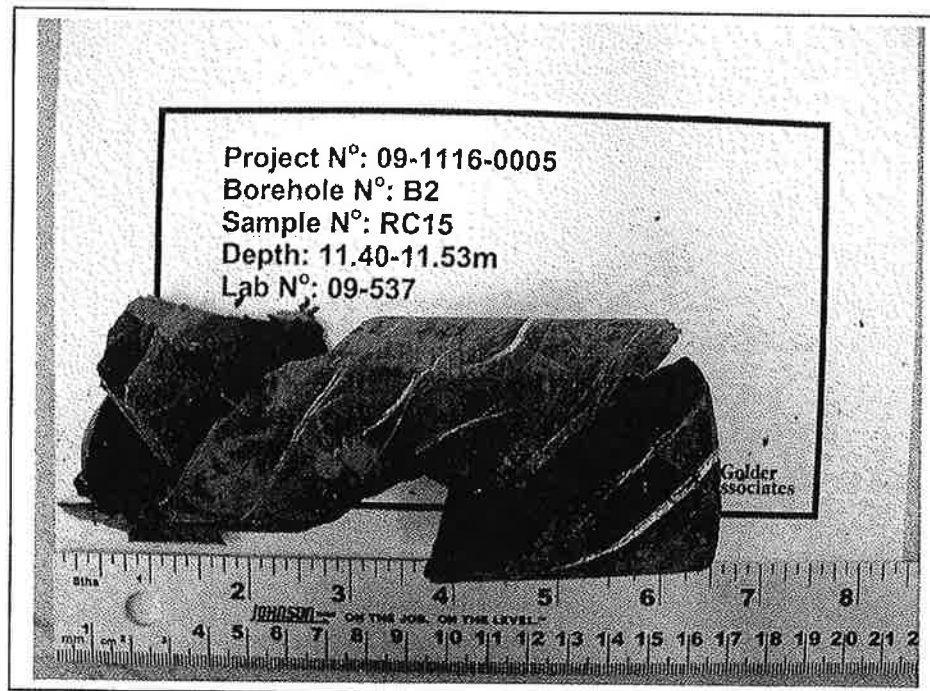
Drawn AH  
Chkd. *ah*

**UNCONFINED COMPRESSION TEST**  
ASTM D2166-98A

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 2/27/2009  
Project 09-1116-0005

**Golder Associates**

Drawn AH  
Chkd. JLB

# Appendix C

## Site Photographs





Photograph 1 Cyrville Road Bridge over Highway 417 (looking East)



Photograph 2 Highway 417 (looking North)





Photograph 3 North East Side of Approach Embankment (looking East)



Photograph 4 South East Side of Approach Embankment (looking South)

# Appendix D

## Rock Core Photographs



BH # P3 / Rock Core RC6, Depth (2.7 – 4.0) m



BH # P3 / Rock Core RC7 & RC8, Depth (4.0 – 5.1) m and (5.1 – 5.9) m resp.





BH # P4 / Rock Core RC6, Depth (2.8 – 4.4) m



BH # P4 / Rock Core RC7, Depth (4.4 – 5.9) m

# Appendix E

**Explanation of Terms Used in Report**

## EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

**JOINT AND BEDDING:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

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

### MECHANICALL PROPERTIES OF SOIL

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

## PHYSICAL PROPERTIES OF SOIL

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

**FOUNDATION DESIGN REPORT  
HIGHWAY 417 / CYRVILLE ROAD BRIDGE,  
CITY OF OTTAWA, ONTARIO  
G.W.P. 4011-06-00  
GEOCRES NO. 31G5-229**

AECOM

Project: TRANETOB01226AA  
October 27, 2010

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**FOUNDATION DESIGN REPORT  
HIGHWAY 417/CYRVILLE ROAD BRIDGE  
CITY OF OTTAWA, ONTARIO  
G.W.P. 4011-06-00**

## **5 DISCUSSION AND RECOMMENDATIONS**

The existing Cyrville Road bridge over Highway 417 is to be replaced with a three span structure. The proposed bridge will be about 25 m longer than the existing bridge and will carry the future four lanes Cyrville Road traffic (i.e. total bridge deck width of about 24.7 m). The new bridge will incorporate a pier No. 2 (central pier) in the unpaved median of Highway 417 and will have an additional pier (Pier No. 1) between Highway 417 and westbound ramp from Highway 174 to St Laurent Boulevard. As shown on Drawing No. 1, Pier No. 1 will be located about 33 m west from the proposed Pier No. 2. Similar to the existing bridge structure, the proposed three span underpass structure will be skewed at about 43° to the existing Highway 417 centreline.

As shown in Drawing No. 1, the anticipated height of the proposed bridge over the existing Highway 417 grade is about 7 m at abutment locations. From information supplied to us, the existing bridge is a two span structure with a total length of about 67 m and its foundations are supported on normal spread footing foundations.

The sub-surface conditions were explored at fourteen (14) boreholes (see Table 3.1 in Section 3 of the foundation investigation section of this report) in the two stages, due to a change in the design, as was discussed earlier. In general, beneath some embankment and other fill materials and a veneer of topsoil and some surficial sand and silt layers, the boreholes show the presence of a silty sand to sandy silt glacial till deposit overlying the dark grey to black shale bedrock. Boreholes contacted the bedrock at about Elevations of 64.9 to 68.8 m or about 0.7 to 4.6 m below the Highway 417 elevation.

The water level at the time of investigation was contacted at Elevations in between 68 and 70 m, based on the measurements of water level in open boreholes and the two piezometers that were installed. It should be noted that the groundwater table can expected to be subject to seasonal fluctuations and in response to major weather events.

### **5.1 Foundations**

We understand that the proposed bridge will be constructed at the same location as the existing bridge, with the same longitudinal centreline but about 1 m higher than the existing bridge. It is our understanding that the proposed three span underpass structure will be about 25 m longer and 8 m wider than the existing bridge. Two construction options are under consideration: The first option is full road closure and the other option is staged construction. For staged construction, we understand that the construction will start with the demolition of one half of the existing bridge and the remaining single lane of the existing bridge will retain the Cyrville Road traffic during the construction of the half of the proposed structure. After this, the newly built half bridge will carry the traffic of the Cyrville Road during the construction of the other half of the proposed bridge.

Based on the results of our investigation, we have considered a number of foundation options varying from normal spread footings to deep foundations which include drilled caissons, micropiles and driven steel piles.

### 5.1.1 Abutments

We understand that semi-integral abutment design is anticipated for the structure. In this case, the preferred option for the support of the abutments is the use of caisson (drilled and cast-in-place) foundations. If necessary, spread footing foundations can also be used with full road closure option, although this may not be the preferred option. In case the new bridge incorporates integral abutments, the use of driven H-piles would be the suitable option for abutment support.

The use of spread footings to support the abutments was looked into but the foundations will need to extend to considerable depths below the groundwater table. Furthermore, deep excavations are not desirable immediately adjacent to the existing and a newly built half of the proposed bridge if staged construction is proposed. As well, considerable shoring effort will be required to retain the existing embankment in order to effect the excavation for the footings. For these reasons spread footing foundations are not considered to be a good alternative, especially for the staged construction option. In addition, because of the closeness to the existing and newly built structures, supporting the foundation elements on compacted granular pad is considered impractical for this project, if staged construction is proposed.

The advantages and disadvantages of various foundation support types at the abutment locations are summarized in Appendix F.

The following paragraphs present a further discussion on these options.

#### 5.1.1.1 Spread Footing Foundations

If necessary, the abutments can be supported on spread footing foundations similar to the existing bridge foundation placed on the shale bedrock (west abutment) or very dense till (east abutment). However, bridge structure supported by spread footing on different materials is not desirable. In consideration of this, spread footing foundations placed on the shale bedrock at both abutment locations are preferred. The following table summarizes the recommended resistances and footing depths. If higher bearing resistance is required, spread footing foundations at deeper depths can also be considered.

**Table 5.1.1.1 Spread Footing Foundations for Abutments**

Location	Applicable Boreholes	Recommended Footing Elevation (Bottom of Footing) (m)	Recommended Bearing Resistance at S.L.S. (kPa)	Recommended Factored Bearing Resistance at U.L.S. (kPa)	Subgrade Material
East Abutment	B1	69.0	400	600	Dense to V. Dense Till
		67.1	500	750	V. Dense Till
		66.0	Will not govern	1000	Shale Bedrock
	B2	67.0	400	600	Dense Till
		65.7	500	750	V. Dense Till

Location	Applicable Boreholes	Recommended Footing Elevation (Bottom of Footing) (m)	Recommended Bearing Resistance at S.L.S. (kPa)	Recommended Factored Bearing Resistance at U.L.S. (kPa)	Subgrade Material
		65.0	Will not govern	1000	Shale Bedrock
West Abutment	B4	69.0	250	450	Compact Till
		68.3	400	600	V. dense till
		67.0	Will not govern	1000	Shale Bedrock
	B5	69.5	250	450	Compact Till
		68.3	400	600	Dense to V. Dense Till
		67.0	Will not govern	1000	Shale Bedrock

With the recommended S.L.S. values, the total and differential settlements should not exceed 25 mm and 20 mm, respectively, provided the subgrade material is not unduly disturbed during the construction.

In most cases, different elevations were provided for support of the foundations under a single footing element. In this case, two approaches can be taken. The footing bottom can be designed and constructed for the lower elevation (e.g. at the east abutment location, the footing can be constructed using the lower elevation of 67.0 m, using the following resistance, S.L.S.= 400 kPa and U.L.S.= 600 kPa). Alternatively, the footing can be designed for elevation 69.0 m and the elevation can be lowered as directed by the Geotechnical Engineer appointed by the QVE, as the conditions warrant. The grade can then be raised to El. 69.0 m (underside of footing) using mass concrete. Obviously, the latter is the more cost effective of the two approaches.

In any event, allowance should be made to place a 120 mm thick concrete mud mat (i.e. skim coat) in the footing excavation as soon as possible (not more than four hours) after excavating to the bearing grade. The footing excavation should be inspected and approved by the Geotechnical Engineer prior to pouring the concrete mud mat.

It should be noted that in between and beyond borehole locations, the bedrock surface and the depth to the suitable founding surface may vary considerably. Additionally, bedrock surface elevations were estimated based on the samples and augering speed/efforts and thus the actual bedrock surface may be different than the given elevations in Table 4.8.1 in Section 4.8 of the foundation investigation section of this report.

For the use of spread footing foundations on the bedrock, all loose or weathered rock under the footprint of the footing should be removed and replaced with concrete. All footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer who is familiar with the findings of this investigation. This is important for this site, since the bedrock appears to be highly weathered to variable depths. Mass concrete may be placed to raise the grade to the founding level, where necessary.

Under the inclined loading conditions the Factored Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6.7.4 of C.H.B.D.C.

Sliding resistance can be provided by utilizing the sliding resistance between concrete and compact to very dense till or bedrock surface. For the evaluation of the sliding resistance of the foundation (C.H.B.D.C. 6.7.5) the ultimate angle of friction (unfactored) between the underside of the foundations and the bedrock

surface (or between concrete surfaces) can be taken as  $28^\circ$ , while the ultimate angle of friction (unfactored) between the underside of the foundations and generally very dense glacial till can be taken as  $27^\circ$ . If additional horizontal resistance is required or if the rock surface is not sufficiently level, dowelling or keying-in into the bedrock can be considered. Such measures would be required if the rock surface is smooth and/or inclined.

If there are net uplift forces which are to be resisted by rock anchors, the factored rock/grout bond resistance at U.L.S. can be taken as 500 kPa and resistance at S.L.S. can be taken as 300 kPa. The upper 1.0 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 2.0 m into the sound rock (embedded length in the sufficiently sound rock). The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked.

For frost protection the footing should have a permanent earth cover of at least 1.8 m.

As can be seen from above table, deep excavations will be required, which may extend below the water table. These will necessitate shoring and dewatering. In addition, the excavations can be expected to extend to variable depths. As well, spread footing foundations are not suitable for the support of integral abutments. For these reasons, the use of spread footing foundations is not recommended, unless full road closure during construction is implemented.

For the full road closure case, if the proposed pile cap bottom elevations are similar to the proposed founding elevations of spread footing, use of spread footings instead of deep foundations can be considered feasible because similar shoring efforts will be required for both pile cap and spread footing construction. However, a greater dewatering effort will be required for the spread footing foundations in comparison with the pile cap construction because the subgrade for the spread footing should be carefully prepared ensuring that the exposed subgrade is not disturbed.

For spread footing foundations, all footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer appointed by QVE who is familiar with the findings of this investigation. This is important for this site, since the bedrock appears to be highly weathered to variable depths. It is recommended that probing be performed below the proposed footing level to ensure that a highly weathered soft rock layer(s) does (do) not underlie the hard rock layer beneath the footing. Probing or star drilling may be considered for this purpose.

#### **5.1.1.2 Spread Footings on Compacted Fill**

In case of full road closure and demolishing of the existing bridge, consideration can be given to using spread footings on compacted granular fill. The engineered fill could consist of Granular 'A' type material, compacted in thin layers to at least 100% of the material's Standard Proctor Maximum Dry Density (SPMDD). The thickness of the Granular 'A' pad supporting the spread footing foundations should be at least 2 m. Prior to the placement of the engineered fill, the upper variable, weak and otherwise unsuitable zones of the existing subgrade should be stripped to the surface of the competent stratum. The suggested highest subgrade elevations at the borehole locations are given in Table 5.1.1.2.1.

**Table 5.1.1.2.1 Recommended Subgrade elevations for Engineered Fill**

Location	Borehole No.	Existing ground elevation (m)	Recommended stripping (base of granular pad) depth (m)	Recommended stripping (base of granular pad) elevation (m)	Soil type
East Abutment	B1	75.8	5.5	70.3	Silty and to sandy silt till
	B2	75.5	5.8	69.7	Silty and to sandy silt till
West Abutment	B4	75.8	5.8	70.0	Silty and to sandy silt till
	B5	76.2	6.4	69.8	Sand (possible till)

After stripping, the exposed subgrade should be inspected and approved by the Geotechnical Engineer. The approved subgrade may then need to be proof-rolled, depending on the site and subgrade conditions, as directed by the Geotechnical Engineer.

The construction of the Granular 'A' pad and of the earth fill should meet the minimum requirements as per MTO, as shown in Appendix H. The Granular 'A' pad supporting the spread footing foundations should be at least 2 m thick.

For footings satisfying these requirements, a factored vertical bearing resistance at U.L.S. (factored) equal to 850 kPa and a bearing resistance at S.L.S. of 350 kPa (for 25 mm settlement) can be utilized. This should, however, be reviewed before the embankment and foundation details are finalized.

The unfactored horizontal resistance against sliding between concrete and properly compacted Granular 'A' fill can be calculated using an angle of friction of 35 degrees.

### 5.1.1.3 Drilled Caisson Foundations

The use of augered and cast-in-place concrete foundations (drilled caissons) can be a feasible foundation option if semi-integral abutment is proposed.

Caissons extended at least 1.2 m into the relatively sound shale bedrock can be designed for an axial geotechnical resistance of 2000 kPa at U.L.S. (factored) and bearing resistance at S.L.S. need not be considered. This resistance value can be increased to 2500 kPa at U.L.S. (factored) with 2.0 m socketing into the rock. Higher resistance of caisson is not recommended at this site due to the presence of soft zones in the shale bedrock and the clayey seams observed within bedrock during the investigation. The following table summarizes the anticipated caisson bottom elevations at the borehole locations.

**Table 5.1.1.3.1: Caisson Foundations**

Location	Recommended bottom of caisson elevation (m)	Recommended Bearing Resistance at S.L.S. (kPa)	Recommended Factored Bearing Resistance at U.L.S. (kPa)	Subgrade Material
East Abutment	63.5	Not applicable	2000	Shale bedrock
West Abutment	65.0	Not applicable	2000	Shale bedrock

\* S.L.S. is for a settlement not exceeding 25 mm

These design values are applicable to commonly used caisson sizes in Ontario (i.e. between 0.76 and 1.8 m diameter) provided the minimum caisson length is 4.0 m below the bottom of the pile cap. However, the

use of relatively smaller caisson sizes (i.e. between 0.76 and 1.5 m) would be preferable as these are relatively easier and more efficient to install. For example, a 0.9 m diameter caisson will have a base area of  $r^2\pi=(0.9/2)^2 \times 3.1416=0.64 \text{ m}^2$ . When designed for a U.L.S. (factored) value of 2000 kPa, the caisson would be capable of carrying an axial load of  $0.64 \text{ m}^2 \times 2000 \text{ kN/m}^2 = 1280 \text{ kN/caisson}$  at U.L.S. (factored). Similarly, if a 1.2 m diameter caisson is used, then the caisson resistance at U.L.S. (factored) would be  $(1.2/2)^2 \times 3.1416 \times 2000 \text{ kN/m}^2 = 2260 \text{ kN/caisson}$ .

As was mentioned before, these resistance values assume a minimum of 1.2 m socket into the relatively sound shale bedrock. This aspect must be verified during the installation of the caissons by the Geotechnical Engineer appointed by the QVE, who would also inspect the base of the caissons and approve them. We recommend that an NSSP be issued to cover this requirement.

The minimum caisson diameter is 0.76 m to enable the cleaning and inspection of the base of the caisson. The clear distance between any two adjacent caissons should be at least two diameters (edge to edge).

Difficulties may arise during the installation of the caissons due to the basically cohesionless nature of the till and especially the sand (possible till) layers/pockets below the groundwater table, as well as the possible presence of cobbles, boulders and shale fragments in the till along with possible hard layers in the shale bedrock. Some dewatering is expected to be necessary to intercept and remove surface water and to pump out any perched water. Dewatering of the glacial till and especially the sand (possible till) deposit may also be necessary. Temporary steel casing will be required during the construction of the caisson holes to prevent caving. The casing/liner would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking'. Concrete must be poured expeditiously after the preparation and approval of the base of the caisson to prevent the deterioration of the shale bedrock. Even though these are standard aspects of caisson installation operations, we recommend that they be 'red-flagged' in the contract documents to reduce the possibility of claims for 'extras' by the contractor, including the possible presence of cobbles, boulders and shale fragments in the glacial till deposit. An NSSP should be issued to alert the contractor of cobbles, boulders, shale fragments in the overburden and the possible hard zones in the shale bedrock, as well as possible dewatering requirement.

The tremie concrete method can be used, if desired or required, to reduce the degree of dewatering during the installation of the caissons. However, based on the borehole data, the use of the tremie concrete method is unlikely to be necessary.

The anticipated caisson elevations at the borehole locations, as given in the table above can be used for design purposes, with interpolation in between and beyond the borehole locations. Actual caisson depths in the field would be decided during their installation, ensuring at least 1.2 m socket into the relatively sound shale bedrock. This is important for this project since the bedrock appears to be highly weathered to variable depths below the rock/overburden interface. The sockets may have to be advanced by rock coring or churn drilling since the shale bedrock at the site contains medium strong layers, as evidenced by the unconfined compression tests performed on two intact pieces of the rock core obtained.

#### 5.1.1.4 Steel H-pile Foundations

Driven steel H-pile foundations are the recommended option for the support of the proposed bridge abutments, if integral abutments are proposed. The borehole data indicate that the granular overburden

and the underlying weathered shale bedrock at the bridge site are suitable for the use of driven steel H-piles at the abutment locations. Steel H-piles are preferable to other types of driven piles, such as precast concrete piles, steel tube piles, etc, since steel H-piles are low displacement piles in comparison with precast concrete or steel tube piles. It is recommended that a steel H-pile with a relatively heavy section, such as HP 310 x 110 or HP 310X125, be used with a suitable bearing point (i.e. Titus standard rock bearing points or APF hard bite or approved equivalent) to prevent damage to the pile during the anticipated heavy driving conditions and to ensure adequate seating of the piles in the rock. Care must be taken to avoid overdriving and damaging the pile tip.

Based on local experience, steel H-piles can often penetrate the upper 1 to 2 m highly weathered zone of the Billings Formation before 'setting-up' during the pile driving process. Based on this, the estimated pile tip elevations are provided in Table 5.1.1.4.1.

For HP 310x110 steel H-piles, which are driven to practical refusal in the bedrock. MTO practice is to use a value of 2000 kN (per pile) for Factored Axial Resistance at the Ultimate Limit State (U.L.S.) and Axial Resistance at the Serviceability Limit State (S.L.S.) need not be considered. In this instance, however, due to the weak nature of the bedrock, we recommend an axial resistance of 1800 kN/pile at U.L.S. (factored) and S.L.S. need not be considered. For HP 310X125 the U.L.S. (factored) value can be increased to 2000 kN/pile and S.L.S. need not be considered.

**Table 5.1.1.4.1: Estimated Tip Elevations for Steel H-Pile Foundation**

Location	Borehole No.	Existing Ground Elevation at Borehole Location (m)	Probable Pile Top Elevation (m)	Estimated Pile Tip Depth/Elevations (m)	Estimated Pile Length (m)
10+055 (east abutment)	B1	75.8	71.9	65.2	6.7
10+068 (east abutment)	B2	75.5	71.9	63.9	8.0
9+954 (west abutment)	B4	75.8	71.9	65.7	6.0
9+968 (west abutment)	B5	76.2	71.9	66.7	5.2

The estimated pile refusal depth (i.e. tip elevations) in between and beyond the borehole locations can be interpolated or alternatively, an average single elevation can be quoted as follows, east abutment: about El. 64 m, west abutment: about El. 66 m.

According to a drawing provided to us by AECOM at the previous two span bridge design stage, the elevation for the pile tops will be approximately 71.9 m and therefore length of the piles based on the borehole data can be expected to range from about 5.2 m to 8.0 m. However, the actual pile lengths may vary considerably, as the tip elevations given above are for general guidance purposes only. We recommend that consideration be given to this aspect when ordering the piles. The possibility of piles encountering potential cobbles and boulders in the till should be anticipated. An NSSP should be provided in the Contract Documents to warn the Contractor the possible presence of cobbles, boulders and shale fragments in the overburden.



It should be noted that the Hiley Formula is not applicable for piles driven to refusal on bedrock. The pile termination or set criteria will depend on the pile driving hammer type, helmet, selected pile and length of pile. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid possible damage to the piles. In this regard, for pile driven to the refusal on the bedrock, it is generally accepted practice to reduce hammer energy after abrupt peaking is met on the bedrock surface, and then gradually increase the energy over a series of blows to seat the pile.

Alternatively, a refusal criterion of 5 blows for 6 mm for three consecutive sets can be maintained for practical refusal on bedrock, based on our pile driving experience in Ontario. As well, 16 blows for 20 mm or 20 blows for 25 mm penetration can also be used. These values are based on typical hammer energy of 60 kilojoules/blow, with an energy transfer (efficiency) of 40%.

If the piles encounter refusal before sufficiently penetrating to top of bedrock, then pile capacities may need to be revisited and alternative measures sought. Therefore, pile driving records should be kept and if refusal is met above the recommended bearing zone, the Foundation Design Engineer and the Bridge Design Engineer should be consulted to assess axial resistance and the minimum pile length requirements. It is also possible that the piles may be driven some distance below the estimated pile tip elevations.

All pile driving should be carried out in accordance with SP903S01. Re-striking should be done as per SP903S01. After each pile is installed, an elevation should be taken of the pile top or on a suitable mark on the side of the pile. This elevation should be checked periodically to confirm that the pile has not heaved as a result of the driving of adjacent piles. Piles that are heaved must be re-driven to the required resistance as required by the engineer. At least 10% of the piles (but not less than two piles) driven at each support element should be re-tapped not less than 24 hours after the driving of the pile, as per SP903S01, to check that relaxation has not occurred. If it has, then all the piles should be re-tapped.

While pile heave/relaxation is not anticipated, if it is observed, it may be necessary to stagger the driving of the piles. The use of light-weight (e.g. HP 310 x 79) piles is not recommended as lighter piles are more vulnerable to damage. Consideration should be given to provide an NSSP to alert the contractor of the possible presence of cobbles, boulders and shale fragments in overburden and possible heavy driving requirements through the dense to very dense strata.

For frost protection, all pile caps should have a permanent earth cover of at least 1.8 m or be provided with an equivalent thickness of extruded rigid exterior-grade polystyrene insulation.

Eccentric loading on piles and the required pile spacing should be considered as per the latest Canadian Highway Bridge Design Code S6-06. Reference may be made to Section C6-8.7.1 of the Canadian Highway Bridge Design Code S6-06, for assessing lateral pile resistance.

In cohesionless soils, the coefficient of horizontal subgrade reaction can be estimated from:

$$k_s = n_h z / d$$

Where  $k_s$  = coefficient of horizontal subgrade reaction

$z$  = depth

$d$  = pile width

$n_h$  = coefficient related to soil density as given in Table 5.1.1.4.2.

Also as presented in the same table are estimated values for angle of internal friction and bulk unit weights.

Where the soil is primarily cohesive, the undrained shear strength of the soil is given. In this case,

$$k_s = 67 c_u / d$$

Where  $k_s$  = coefficient of horizontal subgrade reaction

$c_u$  = undrained shear strength

$d$  = width of pile

**Table 5.1.1.4.2 Anticipated  $n_h$  and  $c_u$  Values**

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m <sup>3</sup> )	Angle of Internal Friction ( $\phi$ ) Degrees	Recommended $n_h$ Value (kN/m <sup>3</sup> )	Recommended Undrained Shear Strength, $c_u$ (kPa)	Groundwater Elevation (m)
B1	71.9-70.4	embankment fill	19.0	31	4500	-	Dry upon completion (estimated at El. 69.0 m)
	70.4-68.0	Glacial till	21.0	33	6600	-	
	68.0-66.7	Glacial till	21.5	34	11000	-	
B2	71.9-70.2	embankment fill	18.5	30	2200	-	68.0
	70.2-68.0	Glacial till	20.5	32	5800	-	
	68.0-67.0	Glacial till	20.5	30	1300	-	
	67.0-65.4	Glacial till	21.5	34	11000	-	
B4	71.9-70.1	embankment fill	19.5	32	5000	-	69.0
	70.1-69.0	Glacial till	21.5	34	18000	-	
	69.0-68.5	Glacial till	21.0	33	4400	-	
	68.5-67.9	Glacial till	21.5	34	11000	-	
B5	71.9-69.9	embankment fill	19.0	31	4500	-	70.0
	69.9-68.6	Sand (possible till)	20.5	32	3600	-	
	68.6-67.7	Glacial till	21.5	33	11000	-	

\* assumed pile top elevation based on the AECOM drawing is about El. 71.9 m

For preliminary estimating, the recommended horizontal resistances for HP 310 x 110 steel H-piles are as follows:

Horizontal Resistance at U.L.S. = 120 kN/pile

Horizontal Resistance at S.L.S.\* = 50 kN/pile

\*for a lateral displacement of 10 mm at the pile head with reference to Section C6.8.7.1 of CHBDC

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone.

MTO structural office requirements (Report SO-96-01) indicate that the flex zone can be provided by augering a 600 mm diameter hole 3000 mm deep and filling with uniform sand. A special provision should be included in the contract specifying the supply and installation of the CSP's, including the gradation of the sand. The special provision is given in Appendix G; the required gradation of the uniform sand is presented in the following Table.

**Table 5.1.1.4.3 Sand Gradation required for the Flex Zone**

Sieve Size	Percentage Passing
2 mm	100 %
600 µm	80-100 %
425 µm	40-80 %
250 µm	4-25 %
150 µm	0-6 %

### 5.1.2 Pier No. 2 (Central Pier) Foundations

The new proposed bridge will be a three span structure with Piers No. 1 (north pier) and No. 2 (central pier). Pier No. 2 is located within the existing median of Highway 417.

We understand that the new footing will be at the existing footing location. To construct the new support, the existing footing can either be removed before constructing the new support or left in place. If the existing footing is to be left in place, caissons or micropiles could be installed through it (with full road closure). However, difficulties will likely occur during the installation due to the presence of thick concrete and reinforcing bars and for this reason, this option may not be feasible. You may wish to discuss this aspect with specialized contractors. Alternatively, caissons or micropiles can be installed around the existing footing and a new pile cap around and over the existing footing can be constructed.

If spread footing must be used (with full road closure) then the existing footings will have to be removed with full shoring support and dewatering. For the construction of the footing where backfill is required we recommend the use of weak concrete for this purpose, to prevent the seepage of water onto the supporting Billings Shale which may be softened by water.

Boreholes P1 and P2, drilled within the median area of Highway 417, indicate, beneath an approximately 0.2 m thick asphalt (Borehole P1) and a veneer of topsoil (Borehole P2), the presence of fill materials such as sand, sand & gravel and silty sand. The fill material (possible sewer trench backfill or backfill from original construction), encountered in Borehole P2, extends to a depth of 3.7 m below the ground surface or to El. 65.7 m and is in loose condition. Underlying the fill material in Borehole P1, the borehole encountered a 0.5 m thick sand layer which is in turn underlain by a silty sand to sandy silt till. Below the fill (Borehole P2) and the silty sand to sandy silt till (Borehole P1), Boreholes P1 and P2 show the presence of weathered shale bedrock at depths of 3.5 and 3.7 m or at El. 66.2 and 65.7 m, respectively. Top 2 m portion of the shale bedrock in Borehole P2 appears to be extremely weathered as evidenced by the recorded N-values of 8 and 24 blow/0.3 m and visual examination of the recovered samples.

For the pier No. 2 location, above mentioned subsurface conditions indicate that if normal spread footings are to be utilized, they must be extended to El. 66.0 to 63.7 m or about 3.7 m (Borehole P2) to 5.7 m (Borehole P1) below the existing grade in the median area. This means excavations extending several metres below the estimated groundwater level. In addition, extensive excavation adjacent to the existing and new structure is required, if staged construction is proposed. As such, the use of spread footing foundations at this site is not recommended, especially if a staged construction is to be implemented. The recommended options are drilled caisson foundations and micropiles, as discussed later in this section of the report. The use of driven pile foundation is not recommended due to the short lengths.

A summary of various foundation options for the pier No. 2 foundation supports is given in Appendix F.

#### 5.1.2.1 Spread Footing Foundations

As mentioned before, the use of spread footing foundations for the support of the pier No. 2 is not a recommended option. However, if spread footing foundations must be used, the depth to the surface of the sufficiently sound bedrock from the existing ground surface (to support spread footing foundations) at Borehole P1 and P2 locations is given in the following table.

**Table 5.1.2.1.1 Spread Footing Foundations for Pier No. 2**

Location	Applicable Borehole (ground elevation) (m)	Recommended Footing Depth (Bottom of Footing) (m)	Recommended Footing Elevation (Bottom of Footing) (m)	Recommended Bearing Resistance at S.L.S. (kPa)	Recommended Factored Bearing Resistance at U.L.S. (kPa)	Subgrade Material
Pier No. 2	P1 (69.7)	3.7	66.0	Not applicable	900	shale bedrock
		5.7	64.0	Not applicable	1200	shale bedrock
	P2 (69.4)	5.7	63.7	500	750	shale bedrock
		6.2	63.2	Not applicable	900	shale bedrock
		7.4	62.0	Not applicable	1200	shale bedrock

Provided that the subgrade material is not unduly disturbed during the construction, with the recommended S.L.S. values, the total and differential settlements should be not exceed 25 mm and 20 mm, respectively.

Under inclined loading conditions, the bearing resistance at U.L.S. should be reduced in accordance with CHBDC S6-06.

Sliding resistance can be provided by utilizing the sliding resistance between concrete and clean bedrock surface. For the evaluation of the sliding resistance of the foundation (C.H.B.D.C. 6.7.5), the ultimate angle of friction (unfactored) between the underside of the foundations and the clean, intact bedrock surface (or between concrete surfaces) can be taken as  $28^\circ$ . If additional horizontal resistance is required or if the rock surface is not sufficiently level, dowelling or keying-in into the bedrock can be considered. Such measures would be required if the rock surface is smooth and/or inclined.

If there are net uplift forces which are to be resisted by rock anchors, the factored rock/grout bond resistance at U.L.S. can be taken as 500 kPa and the resistance at S.L.S. can be taken as 300 kPa. The upper 2 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 2.5 m into the sound rock (embedded length in the rock). The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked.

All footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer who is familiar with the findings of this investigation. This is important for this site, since the bedrock appears to be extremely weathered near its surface to variable depths. As mentioned before, all footings should be founded on sufficiently sound bedrock. For this purpose, all loose or weathered rock under the footprint of the footing and some distance beyond should be removed to the surface of the sufficiently sound bedrock and replaced with concrete. Mass concrete may be placed to raise the grade to the founding level, where necessary. In this event, the geometry of the footing

excavation will need to be determined after some of the details are known and we will be pleased to further discuss this aspect.

During the construction, the following approach can be taken. The footing can be placed at the lower of the two elevations recommended. For example, if designed for the resistance of 900 kPa, the entire pier No. 2 footing excavation can be carried out to El. 63.2 m (i.e. to about 6.2 to 6.5 m below the existing grade). The footing can be placed at this depth subject to inspection, elevation and approval of the Geologist or Geotechnical Engineer. Alternatively, the excavation can be carried out El. 66.0 m and then the grade further lowered as directed by the Geologist or the Engineer. The anticipated excavation elevation at Borehole P2 location is 63.2 m. The excavation would then be approved by the Geologist or the Engineer, as discussed before. The grade can then be raised to the footing design elevation (i.e. El. 66.0 m), using mass concrete at the proposed pier No. 2 location. As mentioned before the actual geometry will need to be determined (e.g. size of excavation at bottom, side slopes etc.)

Allowance should be made to place a 120 mm thick concrete mud mat (i.e. skim coat) in the footing excavation as soon as possible (not more than four hours) after excavation. The footing excavation should be inspected and approved by the Geotechnical Engineer prior to pouring the concrete mud mat.

As can be seen from the table provided, relatively deep excavations extending to below water table will be required. As the footings should be constructed in the dry, dewatering, as well as a temporary shoring system, will be required (due to the proximity to the existing structure, especially if staged construction is proposed).

For frost protection the footing should have a permanent earth cover of at least 1.8 m.

For the reasons discussed above, spread footing foundation option is not recommended for the Pier No. 2 location, especially if a staged construction is proposed for this project, due to the closeness to the existing and the newly built structures.

For spread footing foundations, all footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer appointed by QVE who is familiar with the findings of this investigation. This is important for this site, since the bedrock appears to be highly weathered to variable depths. It is recommended that probing be performed below the proposed footing level to ensure that a highly weathered soft rock layer(s) does (do) not underlie the hard rock layer beneath the footing. Probing or star drilling may be considered for this purpose.

#### **5.1.2.2 Micropile Foundations**

An alternative which may be considered is the use of micropiles to support the Pier No. 2, especially if a staged construction is proposed.

A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can withstand axial and/or lateral loads, and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in access-restrictive environments and in most soil and rock types and ground conditions. Micropiles can be installed at any angle below the horizontal using the same type of equipment

used for ground anchor and grouting projects. Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to enhance the support of existing structures. Micropile structural capacities, by comparison, primarily rely on high capacity steel elements to resist most or all of the applied loads. These steel elements have been reported to occupy as much as one-half of the whole volume. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout and ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter (typically 160 to 260 mm), any end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e., pressure grouting or gravity feed. The role of the drilling method is also influential, although less well quantified.

Axial resistances of up to about 900 kN/micropile are available (at U.L.S.) and S.L.S. will typically not govern. In this present case, up to a similar resistance would be available depending on the diameter and penetration into the sound bedrock. The lateral resistances would also depend on the diameter, as well as to a lesser extent on the socket length into the bedrock.

The use of micropiles may be less economical than spread footing foundations and caissons due to the fact that the installation requires a more specialized installer for the micropiles than the many contractors who are able to routinely install caissons. However, it is advantageous if low overhead is necessity and/or interference of new foundation support with the existing pile foundations is a concern. As was mentioned before, geotechnical resistances will also depend on such factors as diameter, method of installation, socket lengths, etc. Typically, the geotechnical resistance is calculated by multiplying the circumferential area (i.e. circumference x length) by bond strength. For preliminary estimating purposes, the bond strength between the micropile and the sound shale bedrock can be taken as 300 kPa at S.L.S. and 500 kPa at U.L.S. (factored), but the contribution from the upper, relatively fractured 1.5 m should be disregarded. A special provision will need to be developed for this project.

In summary, consideration may given to the use of micropiles, especially if a staged construction is required, particularly if low overhead is available for the operation of equipment and there is interference between the existing and new foundations when spread footings or caissons are used (which are discussed in the next section of this report).

The axial and horizontal resistances of micropiles and other details regarding the design of micropiles can be discussed with specialist contractor and we will be pleased to expand on this further should you wish to pursue this option.

### **5.1.2.3 Drilled Caisson Foundations**

Augered and cast-in-place concrete foundations (drilled caissons) can be considered for the support of the Pier No. 2 and is the recommended alternative, where feasible.

For caissons socketed at least 1.2 m into the sufficiently sound shale (i.e. to El. 62.5 m at Borehole P1 and to El. 60.5 m at Borehole P2), a factored geotechnical resistance value of 1600 kPa at U.L.S. can be used and bearing resistance at S.L.S. need not be considered. This design value is applicable to commonly used caisson sizes in Ontario (i.e. between 0.76 and 1.8 m diameter) provided the minimum caisson length is

about 3.0 m below the bottom of the pile cap. However, the use of relatively smaller caisson sizes (i.e. between 0.76 and 1.5 m) would be preferable as these are relatively easier and more efficient to install. For example, a 0.9 m diameter caisson will have a base area of  $r^2\pi=(0.9/2)^2\times3.1416=0.64\text{ m}^2$ . When designed for a U.L.S. value of 1600 kPa, the caisson would be capable of carrying an axial load of  $0.64\text{ m}^2 \times 1600\text{ kN/m}^2 = 1024\text{ kN/caisson}$  at U.L.S. (factored). Similarly, if a 1.2 m diameter caisson is used, then the caisson resistance at U.L.S. (factored) would be  $(1.2/2)^2 \times 3.1416 \times 1600\text{ kN/m}^2 = 1810\text{ kN/caisson}$ .

Higher resistances are also available at greater depths (e.g. a factored bearing resistance at U.L.S. 2000 kPa at El. 59.9 m and a factored bearing resistance at U.L.S. of 2200 kPa at El. 59.6 m) for about 0.9 to 1.2 m diameter caissons, which we understand will be used for this project.

These resistance values assume that the caissons will be socketed at least 1.8 m into the sufficiently sound bedrock, for a factored bearing resistance of 2000 kPa at U.L.S. and at least 2.1 m for a factored bearing resistance of 2200 kPa at U.L.S. Geotechnical resistance at S.L.S. need not be considered. This aspect must be verified during the installation of the caissons by the Geotechnical Engineer appointed by the QVE, who would also inspect the base of the caissons and approve them. We recommend that an NSSP be issued to cover this requirement.

At the location of Boreholes P1 and P2, the anticipated caisson depths (below the existing grade) and base elevations are 9.8 m/59.9 m and 9.5 m/59.9 m, respectively. Assuming a frost protection depth of 1.8 m below existing grade, the probable caisson length would be about 8 m.

The minimum caisson diameter is 0.76 m to enable the cleaning and inspection of the base of the caisson. The clear distance between any two adjacent caissons should be at least two diameters (edge to edge).

Dewatering may be required to facilitate the installation of the caisson units. As well, some difficulty should be anticipated due to the presence of possible cobbles, boulders and shale fragments in the till, as well as due to the presence of hard layers in the shale bedrock. These conditions are however not uncommon for such applications, as discussed below.

Difficulties may arise during the installation of the caissons due to the basically cohesionless nature of the till and especially the sand (including the possible till) below the groundwater table, as well as the presence of cobbles, boulders and shale fragments in the till. Some dewatering is expected to be necessary to intercept and remove surface water. As well, dewatering may be required to prevent the disturbance of the base of the caisson before pouring the concrete. Temporary steel casing will be required during the construction of the caisson holes to prevent caving. The casing/liner would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking.' Concrete must be poured expeditiously after the preparation and approval of the base of the caisson to prevent the deterioration of the shale bedrock. Even though these are standard aspects of caisson installation operations, we recommend that they be 'red-flagged' in the contract documents to reduce the possibility of claims for 'extras' by the contractor, including the possible presence of cobbles, boulders and shale fragments in the glacial till deposit and the presence of hard layers in the shale bedrock. An NSSP should be issued to alert the contractor of cobbles, boulders and shale fragments in the overburden. Within this NSSP, the Contractor should be alerted that cohesionless soil deposits submerged below the groundwater table are subject to conditions of unbalanced hydrostatic head that will cause caving while installing the caisson and that the Contractor should select an appropriate dewatering scheme to effect the installation without soil

caving and/or necking. As well, water ingress from within the bedrock must be prevented. In addition the possible presence of cobbles, boulders and rock slabs in the overburden and hard layers in the shale bedrock should be red-flagged.

The tremie concrete method can be used, if desired or required, to reduce the degree of dewatering during the installation of the caissons. However, based on the borehole data, the use of the tremie concrete method is unlikely to be necessary.

The anticipated caisson depths/elevations at the borehole locations, as given before, can be used for design purposes, with interpolation in between and beyond the borehole locations. Actual caisson depths in the field would be decided during their installation, ensuring sufficient caisson into the sufficiently sound shale bedrock. This is important for this project since the bedrock appears to be highly weathered to variable depths below the rock/overburden interface. The sockets may have to be advanced by rock coring or churn drilling since the relatively intact zones in the shale bedrock at the site is medium strong, as determined by the unconfined compression tests performed on two intact rock core pieces.

#### **5.1.2.4 Driven Piles**

Driven steel H-piles can be considered to support the Pier No. 2 but this is not a recommended option, since the surface of the bedrock was contacted at about 2 m below the probable pile top elevation and the penetration of the piles into the shale bedrock may be erratic. Some piles may find refusal near the surface of bedrock on a relatively 'hard' layer in the bedrock, while others may penetrate considerably deeper. In addition some piles may 'walk' on the surface of the hard layer. The use of other types of driven piles than H-piles, including steel tube piles, is considered an even poorer choice.

For these reasons, and with due consideration of vibration which will be generated during the installation, the use of driven piles at the pier No. 2 location is not recommended.

#### **5.1.3 Pier No. 1 Foundations**

Pier No. 1 will be located about 33 m west of the proposed Pier No. 2 (central) to accommodate a westbound ramp from Highway 174 to St. Laurent Boulevard.

Boreholes P3 and P4, drilled for the Pier No. 1, indicate the presence of fill to depths of 1.7 and 1.8 m respectively, corresponding to El. 67.7 and 67.5 m. The fill is underlain by a 1.0 m thick silty sand till deposit which is in turn underlain by highly weathered shale bedrock at depths/elevations 2.7/66.7 m and 2.8/66.5 m. As such, the subsurface conditions at this location appear to be more uniform than the conditions encountered at Boreholes P1 and P2 at the Pier No. 2 location.

##### **5.1.3.1 Spread Footing Foundations**

If normal spread footing foundations are to be utilized, we recommend that the footings be extended into the relatively sound bedrock for consistency between the Pier No. 1 and Pier No. 2 and to avoid excessive differential settlements. The following table details the recommended footing depths/elevations at the borehole locations.



**Table 5.1.3.1.1 Spread Footing Foundations for Pier No. 1**

Location	Applicable Borehole (ground elevation) (m)	Recommended Footing Depth (Bottom of Footing) (m)	Recommended Footing Elevation (Bottom of Footing) (m)	Recommended Bearing Resistance at S.L.S. (kPa)	Recommended Factored Bearing Resistance at U.L.S. (kPa)	Subgrade Material
Pier No. 1	P3 (69.4)	3.1	66.3	400	600	shale bedrock
		3.6	65.8	Not applicable	900	shale bedrock
		4.1	65.3	Not applicable	1200	shale bedrock
	P4 (69.3)	3.0	66.3	400	600	shale bedrock
		3.5	65.8	Not applicable	900	shale bedrock
		5.0	64.5	Not applicable	1200	shale bedrock

With the recommended S.L.S. values, the total and differential settlements should not exceed 25 mm and 20 mm, respectively, provided that the founding subgrade is not unduly disturbed during the construction.

Under inclined loading conditions, the bearing resistance at U.L.S. should be reduced in accordance with CHBDC S6-06.

Sliding resistance can be provided by utilizing the sliding resistance between concrete and clean bedrock surface. For the evaluation of the sliding resistance of the foundation (C.H.B.D.C. 6.7.5), the ultimate angle of friction (unfactored) between the underside of the foundations and the clean, intact bedrock surface (or between concrete surfaces) can be taken as  $28^\circ$ . If additional horizontal resistance is required or if the rock surface is not sufficiently level, dowelling or keying-in into the bedrock can be considered. Such measures would be required if the rock surface is smooth and/or inclined.

As can be seen from table provided, relatively deep excavations extending below the water table will be required. As the footing should be constructed in the dry, dewatering, as well as temporary shoring will likely be required.

For frost protection the footing should have a permanent earth cover of at about 1.8 m.

If there are net uplift forces which are to be resisted by rock anchors, the factored rock/grout bond resistance at U.L.S. can be taken as 500 kPa and the resistance at S.L.S. can be taken as 300 kPa. The upper 1.5 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 2.5 m into the sound rock (embedded length in the rock). The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked.

All footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer who is familiar with the findings of this investigation. This is important for this site, since the bedrock appears to be highly weathered to variable depths. As mentioned before, all footings should be founded on sufficiently sound bedrock. For this purpose, all loose or weathered rock under the footprint of the footing and some distance beyond should be removed to the surface of the sufficiently sound bedrock and replaced with concrete. Mass concrete may be placed to raise the grade to the founding level, where necessary. In this event the geometry of the footing excavation will need to be determined after some of the depths are known and we will be pleased to further discuss this aspect.

Allowance should be made to place a 120 mm thick concrete mud mat (i.e. skim coat) in the footing excavation as soon as possible (not more than four hours) after excavation. The footing excavation should be inspected and approved by the Geotechnical Engineer prior to pouring the concrete mud mat.

The use of normal spread footing foundations to support the Pier No. 1, while more feasible in comparison with the Pier No.2, is not the recommended option.

For spread footing foundations, all footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer appointed by QVE who is familiar with the findings of this investigation. This is important for this site, since the bedrock appears to be highly weathered to variable depths. It is recommended that probing be performed below the proposed footing level to ensure that a highly weathered soft rock layer(s) does (do) not underlie the hard rock layer beneath the footing. Probing or star drilling may be considered for this purpose.

#### **5.1.3.2 Micropile Foundations**

Another alternative which may be considered is the use of micropiles foundations for the support of the pier No.1. The details for this option were discussed for the Pier No. 2 in section 5.1.2.2 of the report. As similar conditions apply the discussion will not be repeated here for the sake of brevity.

#### **5.1.3.3 Drilled Caisson Foundations**

Augered and cast-in-place concrete foundations (drilled caissons) is another alternative which can be considered to support the Pier No. 1.

For caissons socketed at least 1.2 m into the sufficiently sound shale (i.e. to El. 64.1 m at both Boreholes P3 and P4), a factored geotechnical resistance value of 1600 kPa at U.L.S. can be used. In this instance the geotechnical resistance at S.L.S. need not be considered. This resistance value is applicable to commonly used caisson sizes in Ontario (i.e. between 0.76 and 1.8 m diameter) provided the minimum caisson length is about 3.0 m below the bottom of the pile cap. However, the use of relatively smaller caisson sizes (i.e. between 0.76 and 1.5 m) would be preferable as these are relatively easier and more efficient to install. For example, a 0.9 m diameter caisson will have a base area of  $r^2\pi=(0.9/2)^2 \times 3.1416 = 0.64 \text{ m}^2$ . When designed for a U.L.S. value of 1600 kPa, the caisson would be capable of carrying an axial load of  $0.64 \text{ m}^2 \times 1600 \text{ kN/m}^2 = 1024 \text{ kN/caisson}$  at U.L.S.. Similarly, if a 1.2 m diameter caisson is used, then the factored resistance at U.L.S. would be  $(1.2/2)^2 \times 3.1416 \times 1600 \text{ kN/m}^2 = 1810 \text{ kN/caisson}$ .

Higher resistances are also available at greater depths (e.g. factored bearing resistance at U.L.S. of 2000 kPa at El. 63.3 m and factored resistance at U.L.S. of 2200 kPa at El. 63.0 m) for about 0.9 to 1.2 m diameter caissons, which we understand will be used for this project.

When recommending these resistances it is assumed that the caissons will be socketed at least 2.0 m into the sufficiently sound bedrock for a factored bearing resistance of 2000 kPa at U.L.S. and at least 2.3 m for a factored bearing resistance of 2200 kPa at U.L.S.. Geotechnical resistance at S.L.S. will not govern. This aspect must be verified during the installation of the caissons by the Geotechnical Engineer appointed by the QVE, who would also inspect the base of the caissons and approve them. We recommend that an NSSP be issued to cover this requirement.

At the location of Boreholes P3 and P4, the anticipated caisson depth (below the existing grade) at a base elevation of 63.3 m, is about 6.0 m. Assuming a frost protection depth of 1.8 m below existing grade, the probable caisson length would be about 4.2 m.

The minimum caisson diameter is 0.76 m to enable the cleaning and inspection of the base of the caisson. The clear distance between any two adjacent caissons should be at least two diameters (edge to edge).

Dewatering may be required to facilitate the installation of the caisson units. As well, some difficulty should be anticipated due to the presence of possible cobbles, boulders and shale fragments in the till, as well as due to the presence of hard layers in the shale bedrock. These conditions are however not uncommon for such applications, as discussed below.

Difficulties may arise during the installation of the caissons due to the basically cohesionless nature of the till below the groundwater table, as well as the presence of cobbles, boulders and shale fragments in the till. However, as no sand layers were encountered, the dewatering at this location can be expected to be less difficult in comparison with the Pier No.2 location, based on the findings of Boreholes P3 and P4. Some dewatering is expected to be necessary to intercept and remove surface water. As well, dewatering may be required to prevent the disturbance of the base of the caisson before pouring the concrete. Temporary steel casing will be required during the construction of the caisson holes to prevent caving. The casing/liner would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking.' Concrete must be poured expeditiously after the preparation and approval of the base of the caisson to prevent the deterioration of the shale bedrock. Even though these are standard aspects of caisson installation operations, we recommend that they be 'red-flagged' in the contract documents to reduce the possibility of claims for 'extras' by the contractor, including the possible presence of cobbles, boulders and shale fragments in the glacial till deposit and the presence of hard layers in the shale bedrock. An NSSP should be issued to alert the contractor of cobbles, boulders and shale fragments in the overburden. As was mentioned for the Pier No.2, we recommend that within this NSSP, the Contractor be alerted that cohesionless soil deposits submerged below the groundwater table are subject to conditions of unbalanced hydrostatic head that will cause caving while installing the caisson and that the Contractor should select an appropriate dewatering scheme to effect the installation without soil caving and/or necking. As well, water ingress from within the bedrock must be prevented. In addition, the possible presence of cobbles, boulders and rock slabs in the overburden and hard layers in the shale bedrock should be red-flagged.

The tremie concrete method can be used, if desired or required, to reduce the degree of dewatering during the installation of the caissons. However, based on the borehole data, the use of the tremie concrete method is unlikely to be necessary.

The anticipated caisson depth/elevation at the borehole locations, as given before, can be used for design purposes. The actual caisson depths in the field would be decided during their installation, ensuring that each caisson is socketed into the sufficiently sound shale bedrock. This is important for this project since the bedrock appears to be highly weathered to variable depths below the rock/overburden interface. Penetration into the bedrock may have to be advanced by rock coring or churn drilling since the relatively intact zones in the shale bedrock at the site is medium strong, as determined by the unconfined compression tests performed on two intact rock core pieces.

The use of caisson foundations at the Pier No.1 location is a preferred option from a foundation engineering viewpoint.

#### 5.1.3.4 Driven Piles

Similar to the Pier No. 2, the surface of the bedrock was contacted in Boreholes P3 and P4 at shallow depths (i.e. about 1 m below the probable pile top elevation) and the penetration of the piles into the bedrock is likely to be erratic.

The use of driven piles to support the Pier No.1 is considered undesirable, based on reliability.

## 5.2. Lateral Earth Pressures

Backfill behind abutments should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation Standards and the requirements of OPSP 3101.150, as given in Appendix G.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B', OPSS 1010) and the provision of drains pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with CHBDC S6-06. For design purposes, the following static parameters can be used.

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

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

Unit Weight =  $22 \text{ kN/m}^3$

Coefficient of Lateral Earth Pressure:

$$K_a = 0.27 \qquad K_b = 0.35$$

$$K_0 = 0.43 \qquad K^* = 0.45$$

### Compacted Granular 'B' Type I

Angle of Internal Friction,  $\phi = 32^\circ$  (unfactored)

Unit Weight =  $21 \text{ kN/m}^3$

Coefficient of Lateral Earth Pressure:

$$K_a = 0.31 \qquad K_b = 0.41$$

$$K_0 = 0.47 \qquad K^* = 0.57$$

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 and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding (e.g. supported on bedrock), then at rest pressures should be used in accordance with CHBDC S6-06. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of CHBDC S6-06.

For unrestrained wing walls (if any), the intermediate earth pressure coefficient  $K_b$  may be adopted. In the determination of degree of wall displacement or rotation to mobilize the fully active earth pressure state, Section C6.9 of the CHBDC S6-06 Commentary can be consulted.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

## **5.2.1 Seismic Design Data**

### **5.2.1.1 Site Coefficient**

The subsurface conditions encountered at the site are represented by Soil Profile Type I (see Clause 4.4.6.2 of CHBDC CAN/CSA-S6-06). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient,  $S$ , for the site is 1.0.

### **5.2.1.2 Seismic Zone and Zonal Acceleration Ratio (A)**

Table A3.1.1 of the CHBDC provides a zonal Acceleration Ratio ( $A$ ) of 0.20 and Velocity Related Seismic Zone ( $Z_v$ ) of 2 for Ottawa. As site coefficient ( $S$ ) is 1.0, and the zonal acceleration is 0.20, the design zonal acceleration ratio for the site can be taken as  $A=0.20$ .

### **5.2.1.3 Seismic Earth Pressures**

Seismic (earthquake) loading should be taken into account in the design in accordance with Section 4.6 of the CHBDC.

In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, the horizontal seismic coefficient,  $k_h$ , used in the calculation of the seismic active pressure coefficient, is taken as  $k_h=0.30$ . The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration coefficient,  $k_v$ . Three discrete values of vertical acceleration coefficient are typically selected analysis, corresponding to  $k_v = +2/3 k_h$ ,  $k_v = 0$ , and  $k_v = -2/3 k_h$ .

The following seismic active pressure coefficients ( $K_{AE}$ ) may be used in design; these coefficients reflect the maximum  $K_{AE}$  obtained using the  $k_h$ , and three values of  $k_v$  as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

### Seismic Active Pressure Coefficients

Active Earth Pressure Coefficient	Granular 'A' ( $\phi = 35^\circ$ - unfactored)	Granular 'B' Type II ( $\phi = 32^\circ$ - unfactored)
Non-Seismic, $K_a$	0.27	0.31
Seismic, $K_{AE}$	0.55	0.61

In the calculation of  $K_{AE}$ , the effect of the friction between the wall and the soil is not considered (i.e.  $\delta=0$ ).

#### 5.2.1.4 Liquefaction Potential

If the proposed structure is supported by deep foundations (driven piles or caissons) or spread footings founded in/on the shale bedrock or compacted fill /dense till, the founding materials are considered not liquefiable.

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C4.6.2 of the CHBDC Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, and assuming a ground surface acceleration of 0.20 g, a factor of safety of greater than 1.0 against liquefaction is obtained for magnitude 7.5 earthquake events under the approach embankment.

### 5.3 Retained Soil System (RSS) Walls

It is our understanding that retained soil system (RSS) will be utilized at the proposed bridge abutment locations. All the RSS walls should be constructed as per SP599S22/S23. The existing approach embankment should be properly benched as per MTO standards (OPSD 208.010) prior to the RSS construction. After the excavation, the exposed subgrade needs to be inspected and all the unsuitable soils need to be replaced with properly compacted, acceptable engineered fill.

We understand that for this project, a typical compacted Granular 'A' pad will be placed on each bench. Based on the information provided to us by AECOM, a vertical face RSS wall (high performance-high appearance) will be founded on about 1 m thick compacted Granular 'A' pad, which will be placed at elevations of about 67.5 m (east abutment) and 68.0 m (west abutment). At these elevations, the boreholes show the presence of variable conditions, ranging from fill (e.g. Boreholes B10 and B11), compact to very dense till (Boreholes B1, B2, B5, B6 and B8) to loose to compact wet sand (Borehole B3) as well as bedrock (Boreholes B4 and B9). For this reason, when the excavation is carried to the required level, the exposed subgrade should be carefully inspected and where fill, very loose or soft or otherwise unsuitable soils are encountered, they should be removed and replaced with well compacted granular soils, such as Granular 'A' materials. We also recommend that even if the material below the footing elevation consists of bedrock, the subgrade should be sub-excavated to the required granular pad level (i.e. at least 1.0 m thick) in order to provide a uniform support. Where excavations extend below the water level, dewatering would be required to avoid disturbance of the natural soils and to facilitate the compaction of the granular pad. The recommended minimum degree of compaction is 98 % of Standard Proctor Maximum Dry Density (SPMDD). Provided the subgrade is prepared in the manner described and the granular soils are properly and uniformly compacted, a factored bearing resistance at U.L.S. of 180 kPa and a bearing resistance at S.L.S. of 120 kPa can be used for the footing supporting the facing wall. The S.L.S. value

corresponds to a settlement of about 25 mm, depending on the details.

Associated backfill should be suitable for the particular application and be approved by the Design Engineer as compatible with the RSS. All the backfill should be compacted in accordance with OPSS 501. Frost protection (typically 1.8 m at this site) needs to be considered.

While no major problems are anticipated based on the prevailing subsurface conditions at the site, the RSS is typically a patented method and the provider of the system normally guarantees its stability.

## 5.4 Approach Embankments

Based on the information provided to us by AECOM, the grade at the west and east abutment locations will be raised by about 1.0 m above the grade of the existing embankments or to about El. 76.5 to 77.0 m. As well, widening of the embankment with side slope of 2H:1V or flatter (flatter than 2H:1V on the cross-section drawings provided to us by AECOM) is proposed to accommodate the proposed wider new structure to replace the existing bridge and that this will involve a grade raise of up to about 7.5 m over the existing grades (o.g.).

Based on the available borehole data, foundation failures are not anticipated for approach embankments constructed with normal 2H:1V side slopes or flatter, provided that all unsuitable materials will be removed as per MTO standards, prior to placing the embankment fills, as per standard MTO procedures. The anticipated stripping depths/elevations at the borehole locations are as follows:

**Table 5.3.1 Recommend Stripping Depth at Borehole Locations**

Borehole No.	Existing Ground Elevation at the Borehole Location (m)	Recommended Stripping Depth/ Elevation (m)
B8	69.2	0.5/68.7
B9	71.8	0.2/71.6
B10	70.4	0.2/70.2
B11	69.0	0.3/68.7

After stripping, the exposed subgrade should be inspected, approved and properly compacted (i.e. proof rolled) from the surface, using a heavy compactor. If necessary, the groundwater table should be lowered to at least 0.7 m in below the subgrade level, before any proof rolling and the application of significant compaction effort. This dewatering can be achieved by gravity drainage and pumping from strategically placed sumps and, if necessary, ditches.

Assuming properly compacted, acceptable inorganic earth fill materials are utilized 2H:1V side slopes can be used for the construction of the approach fills. Proper erosion control measures should be implemented by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

The existing embankment side slopes should be properly benched as per MTO standards (OPSD 208.010) where the embankment widening is proposed.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Select Subgrade Materials - OPSS1010). Fill used for construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of

SP105S10 and OPSS 206. Construction should be in accordance with SP206S03. Quality assurance should be provided as per MTO standard 501.08 (OPSS 501).

Based on the findings of the boreholes, the anticipated foundation settlements under the stresses generated by the approximately up to 7.5 m grade raise for up to 6 m widening section are approximately 25 mm, while another 15 mm of settlement can occur due to settlement of the widened embankment under its own weight, bringing the total anticipated settlements to about 40 mm. The anticipated total settlements are therefore not more than 40 mm, which, in our opinion, will not necessitate surcharging, especially since some of these settlements would take place immediately after construction. It should be however noted that the settlements are anticipated under the widened section, while the anticipated settlements under the existing portion of the embankment, where only about 1.0 m grade raise is anticipated, are about 10 mm. This may lead to differential settlements of the order of 30 mm (i.e. 40 mm – 10 mm = 30 mm). While differential settlements of this magnitude are not unreasonable, they may cause some longitudinal cracking of the pavement along the widened and the existing portion of the embankments. It is therefore recommended that the paving of the road be delayed as long as practically possible in order to effect some of these differential settlements before paving. As well, it is recommended that any excessive differential settlements should be observed during the construction and if necessary they can be rectified (e.g. any cracking or down drag of the existing embankment), due to stress superposition. An NSSP may be issued for this aspect to alert the Contractor.

The foundation settlements should be substantially completed within a period of about three months 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. We recommend that in order to minimize differential settlements immediately adjacent to the new bridge structure, the approach embankment widening be constructed to the subgrade elevation (i.e. bottom of granular pavement fill). The paving of the road itself (i.e. after placing the sub-base and base granular courses) be delayed by at least about four weeks, after the placement of the granular pavement fills.

## 5.5 Construction Comments

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

### OPSS 539 – Protection Systems

#### SP 902S01 – Excavation and Backfilling to Structures.

The boreholes show that the excavations can be expected to extend through some fill material and surficial sand, sand (possible till) and silt deposits which are underlain by silty sand to sandy silt till layers. These soils can be classified as follows:

Granular Embankment (Pavement) Fill	Type 3 soil
Embankment Fill	Type 3 soil above water level
(Sandy silt to silty sand, gravelly sand and sand)	Type 4 soil below water level



Silt, Sand, Sand (possible till)	Type 3 soil above water level
	Type 4 soil below water level
Glacial Till (Dense to very dense)	Type 2 soil above water table
	Type 4 soil below water table, if the soil was not dewatered
Glacial Till (Loose to compact)	Type 3 soil above water table
	Type 4 soil below water table, if the soil was not dewatered

Dewatering will be required during the construction since the groundwater table at the time of our investigation was typically around the o.g. level (about El. 69 m). If at the pier locations normal spread footings are to be utilized, then more substantial dewatering will be required. It should be also noted that the groundwater table can expected to be subject to seasonal fluctuations and in response to major weather events) based on the observations in the boreholes, including wetness of the soil samples during the investigation and old investigation records. As excavations must be carried out in the dry, dewatering will be required. This may consist of deep wells/deep filtered sumps along with perimeter ditches (to intercept and dispose of surface/perched water). Based on the information provided to us by AECOM, the foundations of the existing bridge including the Pier No. 2, are supported on spread footing foundations. Due to the extent of excavation, shoring will be required at the pier locations. As well, shoring will likely be required for west abutment, Pier No. 1 and west approach embankment construction, if staged construction is proposed. The shoring system should be designed by a Professional Engineer, experienced in this type of work. All shoring should be in accordance with OPSS 539.

If the existing foundations need to be removed at the Pier No.2 location for accommodating new caisson foundations, similar dewatering and shoring system as mentioned above spread footing foundations are required. If caisson foundations can be installed through the existing spread footing foundation (e.g. using jack hammer or cutting and coring to accommodate the new caissons within the existing foundations), some dewatering may be required due to the essentially non-cohesive nature of the overburden soils above the existing foundations, together with the recorded high water table to retain the integrity of the base of the caisson excavations, especially at the Pier No.2 location.

Some minor dewatering will also be required to facilitate stripping and the construction of the new embankment fills, which, should it be necessary, can consist of gravity drainage and pumping from strategically placed sumps.

In Ontario, shoring typically consists of soldier pile and timber lagging or sheet piling (with or without bracing / rakers). In this instance, tiebacks will also likely be required. The soldier piles can be expected to extend into the shale bedrock. Tiebacks would extend through the very dense till or shale bedrock depending on the depth of shoring.

The shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this case, the required performance level is considered 2. The shoring system should be designed by a Professional

Engineer, experienced in this type of work. As mentioned before all shoring should be in accordance with SP 105S19.

**Table 5.4.1: Recommended Unfactored Parameters for Temporary Shoring Design**

Soil Type	$K_a$	$K_o$	$K_p$	$\gamma$ (kN/m <sup>3</sup> )
Granular Embankment Fill (typically upper 1.2 m)	0.32	0.49	3.1	21.0
Lower Embankment Fill	0.33	0.50	3.0	20.5
Other Fill	0.38	0.55	2.7	18.0
Topsoil	0.55	0.72	1.0	14.0
Sand/Sand (possible till)/Silt	0.45	0.62	2.2	18.0
Silty Sand to Sandy Silt Till (loose to compact)	0.33	0.50	3.0	20.5
Silty Sand to Sandy Silt Till (dense to very dense)	0.29	0.45	3.4	22.0
Weathered shale	0.26	0.42	3.6	22.0

It should be pointed out that the presence of cobbles, boulders and shale fragments can be expected within the overburden, as well possibly in the embankment and other fill layers. These can be expected to cause problems during the installation of shoring units. This aspect should be 'red-flagged' in the contract documents.

As was mentioned before, materials that may impede the driving of the piles should not be used as backfill in the affected areas.

If pile foundations are selected to support the bridge abutment foundations such as integral type bridge abutment, vibration monitoring for the pile driving will be required. Special provision for vibration monitoring is given in the Appendix G. An NSSP should be issued in this respect.

It should be pointed out that the existing watermain adjacent to the existing east abutment of Cyrville Road bridge may need to be relocated. Details are however not available. We recommend that the Contractor be alerted to protect this watermain during the excavation if it remains at the same location.

It should also pointed out that temporarily excavated slopes steeper than 2H:1V adjacent to the existing highway need to be monitored during the construction.

## 5.6 Frost Protection

Design frost protection depth for the general area is 1.8 m. Therefore, a permanent soil cover of 1.8 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. 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

The Limitations of Report, as quoted in Appendix I, are an integral part of this report.

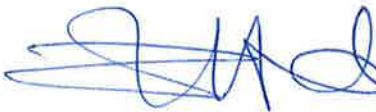
For and on behalf of Coffey Geotechnics Inc.



**Gwangha Roh, Ph.D.**



**Ramon Miranda, P.Eng.**



**Zuhtu Ozden, P.Eng.**



# Appendix F

## **Summary of Foundation Alternatives**

### Summary of Foundation Alternatives for West Abutment

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	<p>Cost effective (if full road closure is proposed)</p> <p>Moderate to high cost (if staged construction is proposed)</p> <p>Will require extensive excavations.</p> <p>Will require shoring and dewatering</p>	<p>High risk due to extensive excavation and dewatering.</p> <p>Not recommended especially for staged construction due to the closeness to the existing and newly built structures.</p>	Moderate to High Cost	<p>Not recommended due to extensive excavation, very close to the existing and newly built structure especially if staged construction is proposed.</p> <p>Not suitable for integral abutment design but may be suitable for semi-integral abutment design.</p> <p>Considered feasible if full road closure is allowed.</p>
Spread footings on compacted Granular 'A' pad	<p>Impractical to implement considering the closeness of the existing and newly built structures if staged construction is proposed.</p> <p>Will require extensive excavations.</p>	<p>Moderate bearing resistance and moderate to high settlements can be expected. Considered impractical for staged construction due to closeness to the existing and newly built structures.</p>	Low to moderate Cost	<p>Not recommended based on economics and practicality, if staged construction is required.</p> <p>Not suitable for integral abutment design but may be suitable for semi-integral abutment design.</p>
Steel H-piles	<p>Low displacement piles; relatively short in some locations but adequate depth; suitable for integral abutment design.</p> <p>Vibration monitoring is essential if staged construction is proposed.</p>	<p>Cobbles, boulders and shale fragments may be encountered during the installation, which may present problems.</p> <p>Care must be taken to drive the piles into the bedrock</p>	Moderate cost	<p>Suitable for integral abutment design.</p> <p>Recommended based on suitability, economics and reliability, especially if an integral abutment design is necessary.</p>
Steel Tube Piles	<p>Higher displacement piles in comparison with steel H-piles. Not suitable for integral abutment design.</p>	<p>The presence of cobbles, boulders and shale fragments. Hard to drive into the bedrock.</p>	Moderate cost	<p>Not suitable for integral abutment design and they are considered less reliable than Steel H-piles for this project. Not recommended.</p>

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Drilled and cast-in-place Concrete piles (drilled caissons)	<p>Less vibration created than driven piles.</p> <p>Not suitable for integral abutment design but feasible for semi-integral abutment.</p>	The presence of cobbles, boulders and shale fragments may present problems during the installation of drilled caisson foundations.	Moderate cost	<p>Not suitable for integral abutment design may be suitable for semi-integral abutment design.</p> <p>Recommended from reliability point of view.</p>

#### Summary of Foundation Alternatives for East Abutment

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	<p>Cost effective (if full road closure is proposed)</p> <p>Moderate to high cost (if staged construction is proposed)</p> <p>Will require extensive excavations.</p> <p>Will require shoring and dewatering</p>	<p>High risk due to extensive excavation and dewatering.</p> <p>Not recommended especially for staged construction due to the closeness to the existing and newly built structures.</p>	Moderate to High Cost	<p>Not recommended due to extensive excavation, very close to the existing and newly built structure if staged construction is proposed.</p> <p>Not suitable for integral abutment design but may be suitable for semi-integral abutment design.</p> <p>Considered feasible if full road closure is allowed.</p>
Spread footings on compacted Granular 'A' pad	<p>Impractical to implement considering the closeness of the existing and newly built structures if staged construction is proposed.</p> <p>Will require extensive excavations.</p>	Moderate bearing resistance and moderate to high settlements can be expected. Considered impractical for staged construction due to closeness to the existing and newly built structures.	Low to moderate Cost	<p>Not recommended based on economics and practicality, if staged construction is required.</p> <p>Not suitable for integral abutment design but may be suitable for semi-integral abutment design.</p>
Steel H-piles	<p>Low displacement piles; relatively short in some locations but adequate depth; suitable for integral abutment design.</p> <p>Vibration monitoring is essential if staged construction is proposed.</p>	<p>Cobbles, boulders and shale fragments may be encountered during the installation, which may present problems.</p> <p>Care must be taken to drive the piles into the bedrock</p>	Moderate cost	<p>Suitable for integral abutment design.</p> <p>Recommended based on suitability, economics and reliability, especially if an integral abutment design is necessary.</p>

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Steel Tube Piles	Higher displacement piles in comparison with steel H-piles. Not suitable for integral abutment design.	The presence of cobbles, boulders and shale fragments. Hard to drive into the bedrock.	Moderate cost	Not suitable for integral abutment design and they are considered less reliable than Steel H-piles for this project. Not recommended.
Drilled and cast-in-place Concrete piles (drilled caissons)	Less vibration created than driven piles. Not suitable for integral abutment design but feasible for semi-integral abutment.	The presence of cobbles, boulders and shale fragments may present problems during the installation of drilled caisson foundations.	Moderate cost	Not suitable for integral abutment design may be suitable for semi-integral abutment design.  Recommended from reliability point of view.

#### Summary of Foundation Alternatives at Pier No. 2 (central pier)

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	Normally least costly, but in this instance based on the results of Boreholes P1 and P2 will necessitate a rather deep excavation extending below the groundwater table, immediately adjacent to the existing spread footing foundations if staged construction is proposed. Will require shoring and dewatering.	Possible deformation of the existing pier foundation, due to the adjacent excavation.	May be uneconomical due to extensive excavation required for spread footing construction	Not recommended due to extensive excavation (partially extending below the groundwater table), close to the existing Highway 417.  If however spread footings are to be used, they should only be considered in conjunction with full road closure.
Steel H-piles	Little or no shoring required; minimizes dewatering. Some piles may encounter refusal at very shallow depth.	The piles will be short and may be extremely short if boulders are encountered during their driving cause vibration.	Moderate	Not a good option based on reliability.
Steel Tube Piles	Higher displacement piles in comparison with Steel H-piles; vulnerable to damage due to the expected hard driving condition. Less suitable than H-piles. No shoring required, minimizes dewatering.	Considered unsuitable for the prevailing subsurface conditions, may be too short.	Moderate	Not a good option based on reliability.

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Drilled and Cast-in-place Concrete Piles (Drilled Caissons)	Minimizes vibrations; little or no shoring required; some dewatering will be required, provides suitable resistances.  If the existing spread footing foundations need to be removed, excavation with dewatering will be required.	Some problems may arise during the construction due to the presence of cobbles, boulders and shale fragments, as well as hard layers in the shale which may increase cost.	Moderate	A recommended option.
Micropile Foundations	Minimizes vibrations and dewatering.  Can be installed in low overhead conditions which may be advantageous for a staged construction option.  If the existing spread footing foundations need to be removed, excavation with dewatering will be required.	Problems may arise during the construction due to cobbles, boulders and shale fragments.	Expensive due to less competitive pricing.	A feasible option especially for staged construction but more expensive than drilled caissons.

#### Summary of Foundation Alternatives at Pier No. 1

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	Least costly, but will necessitate a rather deep excavation extending below the groundwater table.  Will require shoring and dewatering.	Moderate risk due to extensive excavation and dewatering.	May be uneconomical due to relatively deep excavation required for spread footing construction	A feasible option but not recommended due to relatively deep excavation (partially extending below the groundwater table), close to the existing Highway 417.  If however spread footings are to be used, they should only be considered in conjunction with full road closure.
Steel H-piles	No shoring required, minimizes dewatering.  Some piles may encounter refusal at very shallow depth.	The piles will be short and may be extremely short if boulders are encountered during their driving cause vibration.	Moderate	Not a good option based on reliability.



Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Steel Tube Piles	Higher displacement piles in comparison with Steel H-piles; vulnerable to damage due to the expected hard driving condition. Less suitable than H-piles. No shoring required, minimizes dewatering.	Considered unsuitable for the prevailing subsurface conditions, may be too short.	Moderate	Not a good option based on reliability.
Drilled and Cast-in-place Concrete Piles (Drilled Caissons)	Minimizes vibrations; little or no shoring required; some dewatering will be required, provides suitable resistances.  Excavation with dewatering will be required for the existing spread footing foundations removal.	Some problems may arise during the construction due to the presence of cobbles, boulders and shale fragments, as well as hard layers in the shale which may increase cost.	Moderate	A recommended option.
Micropile Foundations	Minimizes vibrations and dewatering.  Can be installed in low overhead conditions which may be advantageous for a staged construction option.  If the existing spread footing foundations need to be removed, excavation with dewatering will be required.	Problems may arise during the construction due to cobbles, boulders and shale fragments.	Expensive due to less competitive pricing.	A feasible option but less cost effective than drilled caissons.

# Appendix G

## List of Standard Specifications

## List of Standard Specifications

### OPSD

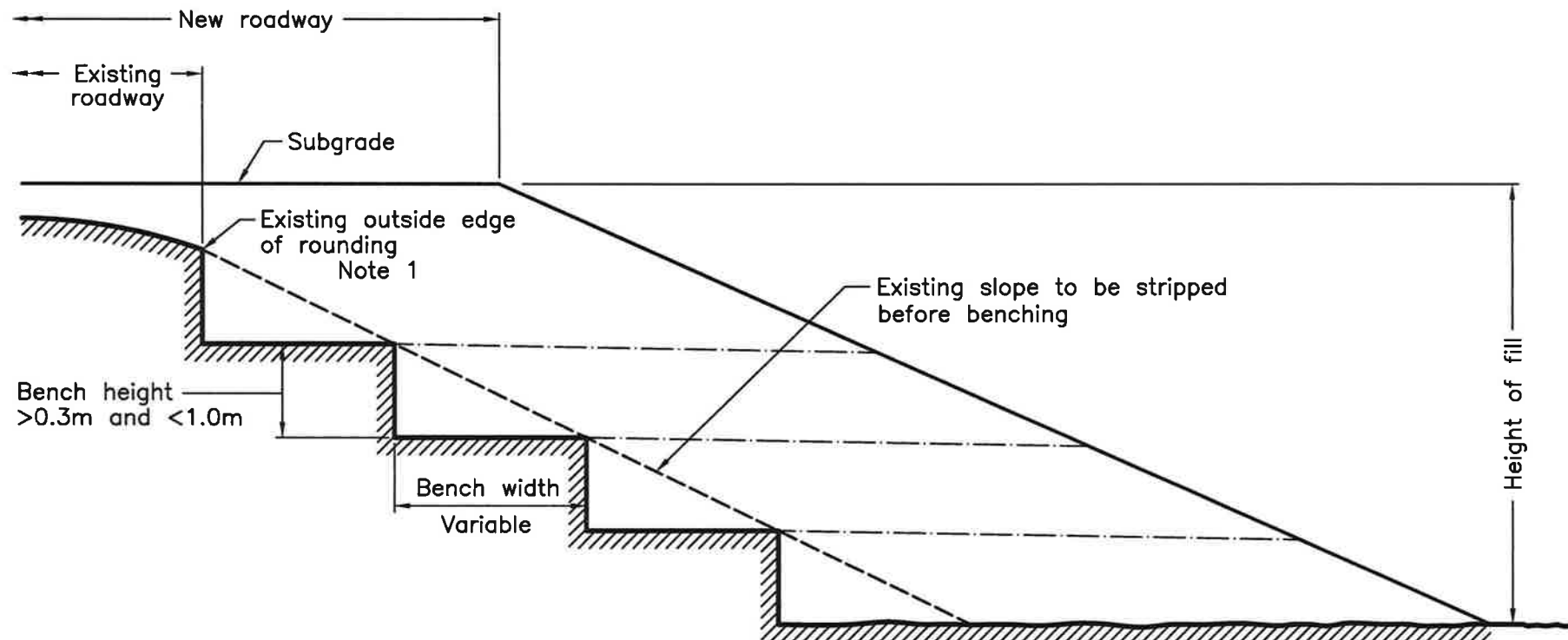
- 208.010 BENCHING OF EARTH SLOPES (included)
- 3101.150 WALLS, ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT (included)

### OPSS

- 206 CONSTRUCTION SPECIFICATION FOR GRADING
- 212 CONSTRUCTION SPECIFICATION FOR BORROW
- 501 CONSTRUCTION SPECIFICATION FOR COMPACTING
- 539 CONSTRUCTION SPECIFICATION FOR TEMPORARY PROTECTION SYSTEMS
- 571 CONSTRUCTION SPECIFICATION FOR SODDING
- 572 CONSTRUCTION SPECIFICATION FOR SEED AND COVER
- 1010 MATERIAL SPECIFICATION FOR AGGREGATES - BASE, SUBBASE, SELECT SUBGRADE, AND BACKFILL MATERIAL

### SP

- SP903S01 PILING
- SP902S01 EXCAVATION AND BACKFILLING TO STRUCTURES
- SP206S03 EARTH EXCAVATION, GRADING, EXCAVATION FOR PAVEMENT WIDENING, ROCK EXCAVATION, GRADING, ROCK FACE, ROCK EMBANKMENT
- SP599S22 RETAINED SOIL SYSTEM
- SP599S23 RETAINED SOIL SYSTEM
- CSP FOR INTEGRAL ABUTMENT (included)



**NOTES:**

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

A Benching is not required on existing slopes flatter than 3H:1V.

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

ONTARIO PROVINCIAL STANDARD DRAWING

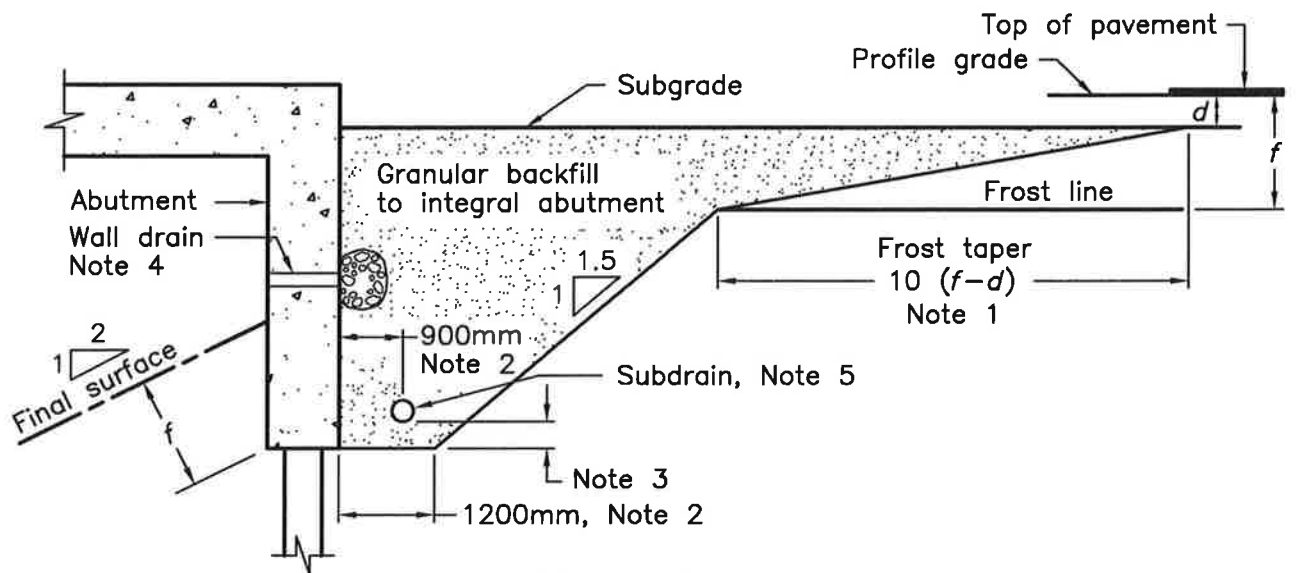
Nov 2003

Rev 1

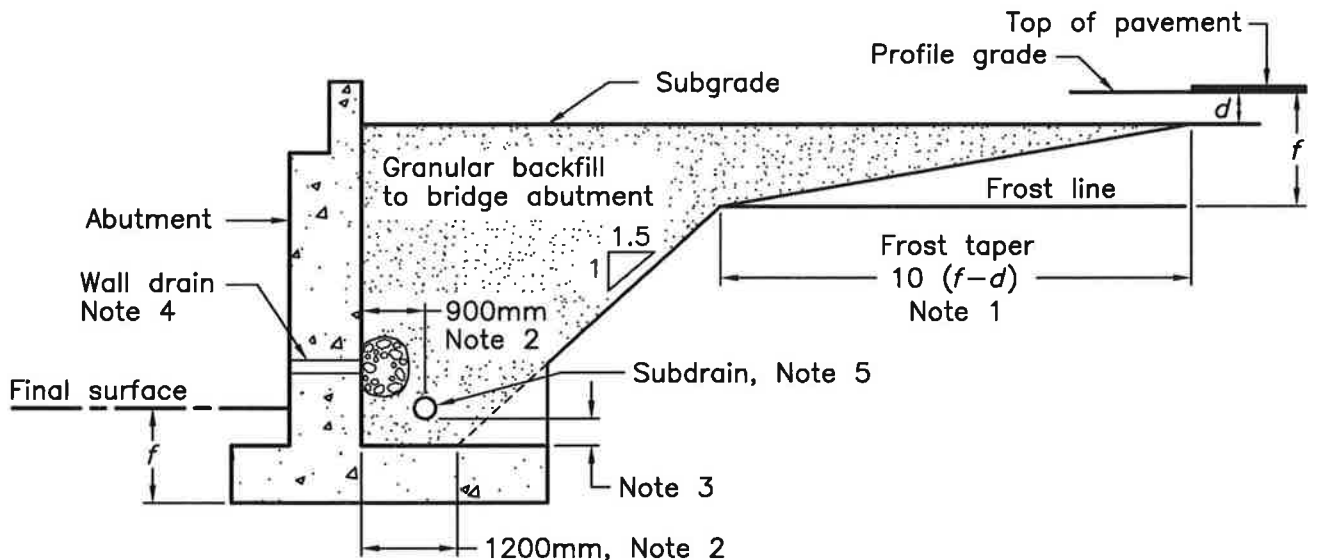
**BENCHING OF EARTH SLOPES**



**OPSD – 208.010**



### INTEGRAL ABUTMENT



### ABUTMENT

#### NOTES:

- 1  $d$  = depth of combined base and subbase courses.  
 $f$  = roadbed depth of frost penetration as specified.
- 2 Dimensions perpendicular to back face of abutment.
- 3 Height to be consistent with positive drainage of subdrain as specified.
- 4 Where specified, wall drains shall be installed according to OPSD-3190.100.
- 5 150mm dia perforated pipe subdrain wrapped with geotextile.
- A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the fill unless interrupted by the retaining wall or wingwall.
- B Sections shown are parallel to centreline of roadway.
- C Subdrain to be installed with a 2% gradient behind wall.
- D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005

Rev 0



**WALLS**  
**ABUTMENT, BACKFILL**  
**MINIMUM GRANULAR REQUIREMENT**

**OPSD - 3101.150**

## **CSP FOR INTEGRAL ABUTMENT - Item No.**

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### **Special Provision**

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#### **Scope**

This specification covers the requirements for the installation of the CSP's, including sand fill and polystyrene sheets, at the integral abutments.

#### **References**

This specification refers to the following standards, specifications or publications:

##### **Ontario Provincial Standard Specifications, Construction:**

OPSS 906      Structural Steel  
OPSS 909      Prestressed Concrete - Precast Members

##### **Ontario Provincial Standard Specifications, General:**

OPSS 180      Management and Disposal of Excess Materials

##### **Ontario Provincial Standard Specifications, Material:**

OPSS 1605      Expanded Extruded Polystyrene  
OPSS 1801      Corrugated Steel Pipe Products

##### **Canadian Standards Association Standards:**

CSA G164-M    Galvanizing of Irregularly-Shaped Articles

##### **Ministry of Transportation Publications**

MTO Manual of Designated Sources of Materials

#### **Definitions**

For the purposes of this specification, the following definitions apply:

**Abutment Stem:** means the cast-in-place concrete component of the abutment placed over the top of the piles and forming the bearing seat for the girders.

**CSP:** means helical corrugated steel pipe.

**Design Engineer:** means the Engineer who produces the design and/or working drawings, and who has a minimum of five (5) years in the design and/or construction of bridges.

## **Submission and Design Requirements**

### **Submissions**

All submissions shall bear the seal and signature of the Design Engineer.

At least two weeks prior to commencement of installation of the abutment, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times.

### **Working Drawing Requirements**

Working drawings shall include at least the following:

1. Layout and Elevations of the CSP's;
2. Source of the sand fill, and description of placing method and equipment;
3. Location and details of all temporary bracing, including permanent and temporary spacers, for the piles, CSP's and abutment stems;
4. Detailed construction sequence for the work, including installation and removal of the temporary bracing.

### **Design Requirements**

The Contractor shall be responsible for the complete detailed design of the construction sequence for the work, including the installation and removal of all temporary bracing. The general sequence of construction shall be as shown on the Contract drawings.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including temporary and permanent spacers, required to maintain the piles, CSP's, abutment stems and girders in their specified positions through all stages of construction until concrete in deck has reached a compressive strength of 25 MPa. All temporary bracing, except spacers identified as permanent on the Contract drawings, shall be removed.

Temporary bracing for prestressed, precast girders shall meet the requirements of OPSS 909. Temporary bracing for structural steel girders shall meet the requirements of OPSS 906.

## **Material**

### **Corrugated Steel Pipe**

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM # 4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

### **Permanent Spacers and Associated Hardware**

Permanent spacers and associated hardware left in place shall not consist of wood and corrodible material.

## **Sand Fill**

The sand fill for backfilling the inner CSP shall meet the gradation requirements of Table 1 below:

Table 1 - Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	# 10	100 %
600 µm	# 30	80 % to 100 %
425 µm	# 40	40 % to 80 %
250 µm	# 60	5 % to 25 %
150 µm	# 100	0 % to 6 %

## **Expanded Extruded Polystyrene**

Expanded extruded polystyrene shall be in accordance with OPSS 1605, and shall be from a supplier listed under DSM # 3.30.30.

## **Construction**

### **General**

The sequence of construction for installing the concrete pads, CSP's, sand fill and abutment stems, including the installation and removal of the temporary bracing, shall be in accordance with the working drawings.

The Contractor shall not proceed with the abutment backfill above the level of the bottom of the CSP's without written permission from the Contract Administrator.

### **Corrugated Steel Pipe**

CSP's shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSP's will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSP's shall be in accordance with the manufacturer's recommendations. Damaged CSP's shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSP's shall be repaired by two coats of zinc-rich paint.

The Contractor shall set the inner and outer CSP over each pile in the abutment into the concrete pad, following the batter of the pile, while the concrete in the concrete pad is still plastic. The CSP's shall extend at least 150 mm into the concrete pad.

The Contractor shall ensure the full perimeter of the tops of all CSP's at each abutment are at the elevation shown on the working drawings.



After the CSP's have been set, the Contractor shall take all measures necessary to prevent the ingress of water, backfill and debris into the CSP's.

### **Sand Fill**

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the inner CSP and pile. No additional compaction effort other than the action of placing the sand fill itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP's.

After the sand fill has been placed to the top of each inner CSP, the Contractor shall take all measures necessary to prevent the ingress of water and other liquids into the sand fill until after the concrete in the abutment stem has been placed and cured.

### **Expanded Extruded Polystyrene**

The expanded extruded polystyrene sheets shall completely cover the area under the abutment stem as shown on the Contract drawings. The sheets shall be placed in one piece for the width of the abutment stem, with butt joints perpendicular to the centre-line of abutment bearings. The minimum length of sheet shall be 500 mm.

Joints between sheets within 500 mm of a pile centre-line will not be permitted. At each pile location, a minimum 1000 mm long sheet shall be centred on the pile and a 500 mm diameter hole neatly cut in the sheet so as to fit over the pile in one piece, fully spanning the annular space between the double CSP's.

The Contractor shall adjust the backfill to ensure full and uniform contact of the sheets with the backfill and the full perimeter of the tops of the CSP's. The vertical step at joints between sheets shall not exceed 5 mm.

The Contractor shall protect the sheets from damage during installation of the reinforcing for the abutment stem, and shall secure the sheets from "floating" during placing of the concrete in the abutment stem. Only hardware approved by the Owner shall be used to secure the sheets. All hardware used to secure the sheets shall be installed so as not to project above the top surface of the sheets into the abutment stem.

### **Temporary Bracing**

Temporary bracing shall be installed and removed in accordance with the working drawings.

The temporary bracing shall not distort, nor pierce the walls of, the CSP's. Welding to the CSP's will not be permitted.

Concrete anchors shall be removed and the holes filled with non-shrink grout.

### **Tolerances**

The CSP's at each pile shall be constructed to the following tolerances:

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of inner and outer CSP from pile centroid.	$\pm 25$ mm
Maximum deviation from specified spacing between inner and outer CSP's.	$\pm 25$ mm
Maximum deviation of any point on the top perimeter of the CSP's from the specified Elevation.	$\pm 10$ mm

### **Quality Assurance**

Prior to placing the CSP's, the Contractor shall establish reference points at each abutment and determine the location of the centroid of each pile in the abutment with respect to these reference points. The Contractor shall maintain the reference points until written permission to proceed with the backfill above the level of the bottom of the CSP's has been given by the Contract Administrator.

### **Measurement for Payment**

There will be no measurement for this item.

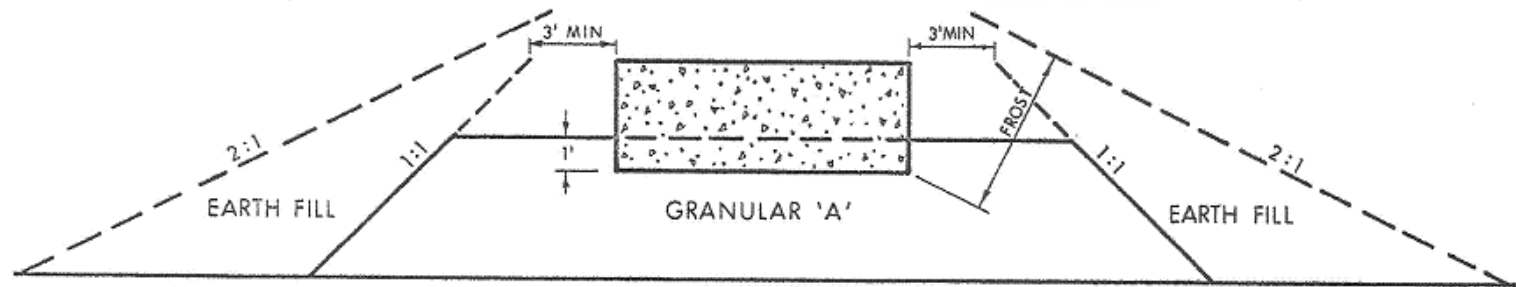
### **Basis of Payment**

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material required to do the work.

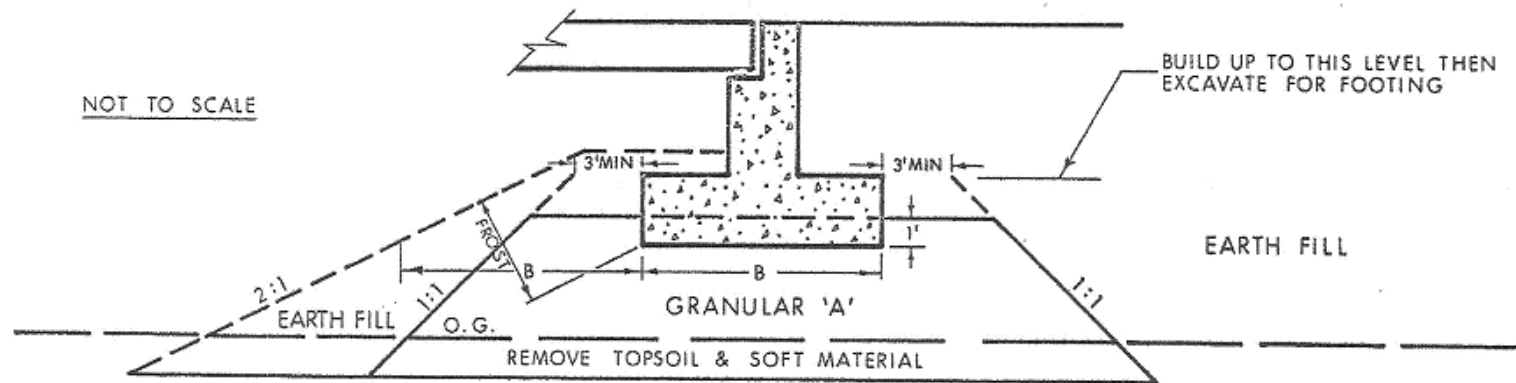
# Appendix H

**MTO Drawing**

# ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



X SECTION



LONGITUDINAL SECTION

## NOTES:

- 1 - REMOVE TOPSOIL &/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' & EARTH FILL.
- 2 - PLACE GRANULAR 'A' & EARTH FILL TO TOP OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.C. STANDARDS.
- 3 - EXCAVATE COMPACTED GRANULAR 'A' & EARTH FILL FOR FOOTING.

# 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.