

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS, REPLACEMENT OF
SHASHAWANDAH CREEK BRIDGE AT
STATION 17+140.696
HIGHWAY 21, NORTH OF TOWN OF
FOREST, LAMBTON COUNTY, ONTARIO
SITE NO. 14-2, G.W.P. 339-97-00
GEOCRES NO. 40P4-48**

Stantec Consulting Ltd.

Project: TRANETOB01210AA
November 3, 2009

November 3, 2009

Stantec Consulting Ltd.
1400 Rymal Road East
Hamilton, Ontario
L8W 3N9

Attention: Mr. Adam Barg, P.Eng.

Dear Sirs:

RE: Foundation Investigation and Design Reports, Replacement of Shashawandah Creek Bridge at Station 17+140.696, Highway 21, North of Town of Forest, County of Lambton, Ontario, Site No. 14-2 and G.W.P. 339-97-00, Geocres No. 40P4-48

Enclosed are the Foundation Investigation and Design Reports for Shashawandah Creek Bridge, north of Town of Forest, Country of Lambton, Ontario, Site No. 14-2 and G.W.P. 339-97-00, Geocres No. 40P4-48.

For and on behalf of Coffey Geotechnics Inc.



Ramon Miranda, P.Eng.
Manager, Transportation Division

Attachment A: Attachments

**FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF SHASHAWANDAH CREEK
BRIDGE AT STATION 17+140.696
HIGHWAY 21, NORTH OF TOWN OF FOREST
LAMBTON COUNTY, ONTARIO
SITE NO. 14-2, G.W.P. 339-97-00
GEOCRES NO. 40P4-48**

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Appendix D: Rock Core Photographs

Appendix E: Explanation of Terms Used in Report

**FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF SHASHAWANDAH CREEK BRIDGE AT STATION 17+140.696
HIGHWAY 21, NORTH OF TOWN OF FOREST, COUNTY OF LAMBTON, ONTARIO,
SITE NO. 14-2, G.W.P. 339-97-00, Geocres No. 40P4-48**

1 INTRODUCTION

The Shashawandah Creek Bridge is located on Highway 21, about 1.5 km north of County Road 6 (Thomson Line), near the Town of Forest in the County of Lambton.

The existing bridge is a single span structure with two lanes including paved shoulders having a length of about 11.3 m. It is our understanding that the existing bridge will be replaced with a new structure.

Coffey Geotechnics Inc. (Coffey), formerly Shaheen & Peaker Limited, was retained in 2007 by Stantec Consulting Ltd. to carry out a preliminary foundation investigation at the site for the proposed replacement of the existing Shashawandah Creek Bridge on Highway 21. In this previous investigation, two deep boreholes (Boreholes 1 and 2) were put down by Coffey to approximate depth of 25.3 m below the existing ground surface. These boreholes were drilled in November 2007 and the findings of the investigation were presented in our Preliminary Foundation Investigation Report (dated April 13, 2009).

This present detail geotechnical investigation consists of four additional boreholes (Borehole 3 to Borehole 6) drilled to an approximate depth of 29 m close to the proposed abutment locations and to about depth of 8 m for the approach embankments. These boreholes were drilled during the period of July 21 to 24, 2009.

The purpose of the new investigation was to provide additional information about the subsurface conditions at the proposed bridge site including verification of the inferred bedrock encountered in the preliminary investigation.

The findings of the current and the previous investigations are presented in this report.

2 SITE DESCRIPTION AND PHYSIOGRAPHY

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the Shashawandah Creek Bridge is located within the Physiographic Region of Huron Slope. It is essentially clay plain modified by a narrow strip of sand and by the twin beaches of glacial Lake Warren which flank the moraine. Below the Warren beach the surface has been smoothed but the deposition of lacustrine clay seldom amounts to more than about 1m and the till often comes to the surface. The till is generally formed from brown calcareous clay, containing a minimum of pebbles and boulders.

According to Bedrock Geology of Ontario Map 2544 and Geological Highway Map 2418, the bedrock underlying this area is of Middle Devonian age (i.e. approximately 370 million years old) and is generally composed of grey shale and limestone. The bedrock in this area belongs to Hamilton Group Formation.

The existing approach embankments 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 FIELD AND LABORATORY WORK

The fieldwork for the detail investigation for the proposed bridge was performed on July 21 to 24, 2009. The fieldwork consisted of drilling and sampling of four boreholes (Borehole 3 through Borehole 6) to depths 28.5 m to 29.6 m for the proposed bridge abutments (Boreholes 4 and 5) and to 8.1 to 8.4 m for approach embankment (Boreholes 3 and 6). In Boreholes 4 and 5, the bedrock was cored more than 3 m. The locations of the new boreholes (Borehole 3 to Borehole 6) as well as the previous boreholes (Boreholes 1 and 2) are shown on the Borehole Location Plan, Drawing No.1. The following table summarizes the borehole locations and drilling depths.

Table 3.1: Borehole Locations and Drilling Depths

Borehole No.	Location	Depth of Borehole Below Existing Ground Surface (m)	Remarks
Borehole 1	17+ 150 (North Abutment)	25.3	-
Borehole 5	17+150 (North Abutment)	28.5	Piezometer and rock coring
Borehole 6	17+170 (North Approach)	8.4	-
Borehole 2	17+128 (South Abutment)	25.2	-
Borehole 4	17+128 (South Abutment)	29.6	Rock coring
Borehole 3	17+108 (South Approach)	8.1	-
Note: Offset Distances from the existing centre line of Highway 21 are indicated on the appropriate Record of Borehole Sheets.			

London Soil Test of London, Ontario carried out the drilling, testing and sampling work in both phases of investigation, under the direction and supervision of Coffey technical personnel. The boreholes were advanced using truck mounted drilling rig, outfitted with tools and equipment for soil sampling and testing. In the overburden material, the boreholes were advanced using continuous flight hollow/solid-stem augers and below overburden material, NQ coring method was used to advance the boreholes into the bedrock and to obtain core samples of the bedrock.

During drilling, sampling in the boreholes was conducted 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 granular (cohesionless) soils (gravels and sands) or the consistency of cohesive soils (clays and clayey soils). Several thin walled Shelby tube samples were also obtained.

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

Groundwater conditions in the boreholes were observed during drilling and upon completion in the open boreholes. In addition, a piezometer was installed in Borehole 5 for monitoring of the groundwater level over a prolonged period of time without interference from surface water. The remaining boreholes were backfilled as per MOE Reg.903, as amended by Reg.372/07. The top portion of boreholes on the existing road pavement was sealed with emulsified asphalt.

London Soil Test of London, Ontario was engaged on Aug.18, 2009 for decommissioning of a piezometer at the location of Borehole 5 in accordance with Ontario Regulation 903.

The borehole locations in the field were established in relation to the existing road chainage (stations) by the engineering staff of Coffey Geotechnics Inc. The geodetic elevations of the ground at the borehole locations were determined by the client's surveyors and provided to us.

The soil and rock samples from the field were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme consisting of natural moisture content determinations for all retrieved soil samples, twenty grain size analyses (twelve from the new investigation and eight from the previous investigation) and twelve Atterberg Limits tests (including eight tests from the new investigation and four from the previous investigation) on selected representative samples were performed. Two rock core samples from Boreholes 4 and 5 were forwarded to the laboratory of Golder Associates where the samples were tested for their unconfined compressive strength (UCS), bulk and dry densities. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B (Figures B1 to B5).

4 SUMMARIZED SUBSURFACE CONDITIONS

The existing top of embankment at the borehole locations varies with elevations ranging from 199.2 (Borehole 2, Borehole 5 and Borehole 6) to 199.8 m (Borehole 3).

The material encountered in the boreholes consisted of pavement fill (granular sub-base and sub-base courses overlain by asphaltic concrete), embankment fill (clayey silt to silty clay) underlain by a massive silty clay to clayey silt till deposit extending to shale bedrock with occasional layers/seams of limestone.

The sub-surface conditions were explored at six boreholes in total (see Table 3.1 in Section 3 for detail location) for this project during the preliminary and detail investigations. The plan location of the boreholes is shown on Drawing No.1 while the stratigraphical profile and sections along boreholes are presented on Drawings 2 to 4. 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. The following description of the individual soil strata is to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site. It should be noted that the soil and groundwater conditions may vary between and beyond the borehole locations.

4.1 Pavement

The pavement structure at the location of the boreholes (Borehole 1 through Borehole 6) was composed of approximately 200 to 370 mm asphalt followed by 300 to 600 mm of granular pavement materials generally composed of sand and gravel.

The grain size distribution of the granular materials from four selected samples (Borehole 1-AS2, Borehole 2-AS2, Borehole 3-AS1 and Borehole 6-AS1) encountered under the asphaltic concrete is given on Figure B1 in Appendix B and is summarised as follows:

Gravel:	30–46%
Sand:	43-53%
Silt & Clay:	3-17%

At the location of Borehole 3, an about 200 mm thick layer of fine sand was contacted at the bottom of granular material. The grain size distribution of the sand layer (Borehole 3-AS2) encountered only at Borehole 3 is shown on Figure B2 in Appendix B and this indicates the following particle size distribution.

Gravel:	1%
Sand:	83%
Silt:	10%
Clay:	6%

This pavement fill is basically granular material.

4.2 Embankment Fill

Underlying the pavement structure, a 0.9 to 2.8 m thick layer of fill was encountered in the boreholes, extending to depths varying from 1.5 m in Borehole 6 to 3.6 m in Borehole 2, corresponding to elevations from 197.7 to 195.6 m. The explored fill is generally composed of clayey silt to silty clay with traces of sand and gravel. The embankment fill contains wood pieces in the upper portion in Borehole 2, asphalt pieces in the lower portion in Borehole 3 and pockets of topsoil/organics at the top portion of this unit in Borehole 5.

The typical grain size distribution of the clayey fill (Borehole 2-SS4 and Borehole 5-SS1) is given on Figure B3 in Appendix B and shows the following gradation.

Gravel:	0-2%
Sand:	10-15%
Silt:	46-48%
Clay:	35-44%

This embankment fill is basically a cohesive material.

Standard Penetration tests performed in this deposit generally yielded N-values between 3 and 13 blows/0.3m, indicating that the consistency of embankment fill can be described as soft to stiff. The results indicate that the fill has received some compaction but has not received systematic compaction. It should be noted that the thickness of fill could vary between and beyond borehole locations.

4.3 Silty Clay to Clayey Silt Till

The native soil in the boreholes is composed of silty clay to clayey silt till with occasional seams/ pockets of sand and this extends to depths varying from 25.1 to 25.3 m in the deep boreholes (Boreholes 1, 2, 4 and 5), corresponding to elevations from 174.3 to 173.9 m. The deposit extended to a maximum explored depth of about 8.3 m in the remaining two relatively shallow boreholes drilled at the location of approach embankment on each side of the existing bridge. The deposit is brown in the upper portion and becomes grey at depths varying from 3.0 to 4.3 m below the existing grade. The lower portion of the till contains weathered shale and/or limestone fragments. Due to the mode of deposition, the presence of cobbles and boulders could be sporadically and widely dispersed throughout the till deposit.

The grain-size distribution of thirteen selected samples (Borehole 1-SS9, Borehole 1-SS16, Borehole 2-SS10, Borehole 2-SS15, Borehole 2-SS18, Borehole 3-SS5, Borehole 4-SS5, Borehole 4-SS16, Borehole 4-SS19, Borehole 5-SS4, Borehole 5-SS9, Borehole 5-SS16, and Borehole 6-SS4) from the glacial deposit is given in an envelope form in Figure B4 attached in Appendix B, which indicates the following gradation.

Gravel:	0-8%
Sand:	4-19%
Silt:	42-52%
Clay:	30-47%

Atterberg limits tests were carried out on twelve samples (Borehole 1-SS5, Borehole 1-SS15, Borehole 2-SS7, Borehole 2-SS16, Borehole 3-SS5, Borehole 4-SS5, Borehole 4-SS16, Borehole 4-SS19, Borehole 5-SS4, Borehole 5-SS9, Borehole 5-SS16 and Borehole 6-SS4) of the silty clay to clayey silt till deposit. The results are given on the individual Record of Borehole Sheets attached in Appendix A and also shown on Figure B5 in Appendix B. The tests yielded the values presented in the following Table 4.3.1.

Table 4.3.1: Atterberg Limits of Silty Clay to Clayey Silt Till

Sample No.	Depth Below the Existing Ground Surface (m)	Plastic Limit – PL (%)	Liquid Limit – LL (%)	Plasticity Index PI
BH1- SS5	2.3-2.8	20.0	34.0	14.0
BH1– SS15	13.7- 14.2	20.2	35.5	15.3
BH2-SS7	3.8 - 4.3	18.1	33.2	15.1
BH2-SS16	15.2 – 15.7	17.7	30.0	12.3
BH3-SS5	2.3-2.8	19.4	32.6	13.2
BH4-SS5	3.8 - 4.3	17.4	31.6	14.2
BH4-SS16	15.2 – 15.7	17.4	31.3	13.9
BH4-SS19	21.3 -21.8	17.4	29.5	12.1

Sample No.	Depth Below the Existing Ground Surface (m)	Plastic Limit – PL (%)	Liquid Limit – LL (%)	Plasticity Index PI
BH5-SS4	3.1 -3.5	17.3	30.4	13.1
BH5-SS9	6.7 – 7.2	17.3	29.9	12.6
BH5-SS16	16.8-17.2	16.2	27.7	11.5
BH6-SS4	1.5 – 2.0	19.5	33.3	13.8
Average Value		18.2	31.6	13.4

The liquid limit (LL) values range from 27.7% to 35.5%, with an average value of 31.6%. The plasticity index (PI) values range from 11.5 to 15.3, with an average value of 13.4. As shown on the plasticity chart on Figure B5 in Appendix B, the clay deposit is generally classified as inorganic clay of low plasticity.

As shown on the borehole logs, the moisture content of clay deposit selected for Atterberg limits tests ranged from about 15.6 % to 21.9%, with an average value of 18%. This indicates that the moisture content is generally at the plastic limits (i.e. the deposit has undergone some degree of pre-consolidation).

Standard Penetration tests performed in this deposit gave 'N' values ranging from 8 to 52 blows/0.3 m. The silty clay to clayey silt deposit is described as stiff to hard in consistency but typically stiff to very stiff. The till deposit is hard near the surface of bedrock surface excluding BH4, where hard layer was not encountered.

4.4 Bedrock

In two of the boreholes (Boreholes 4 and 5) bedrock was penetrated and was proven by NQ coring at starting depths varying from 25.1 to 25.3 m or El. 173.9 to 174.3 m, where grey shale with occasional limestone layers/seams was encountered. Auger refusal on probable bedrock surface was encountered in two boreholes of the preliminary investigation (i.e. Boreholes 1 and 2) at approximate depth of 25.3 m below the existing grade, corresponding to approximate elevation 174.0 m. The bedrock was not cored in Boreholes 1 and 2, as this was not within the terms of reference of the preliminary investigation. The depths and elevations of the bedrock surface and/or inferred bedrock surface at the four borehole locations are presented in Table 4.4.1

Table 4.4.1: Bedrock elevation and condition

Borehole No.	Ground Surface Elevation (m)	Depth Below Ground Surface/Elevation of the Bedrock Surface (m)	T.C.R. (%)*	R.Q.D. (%)**
Borehole 1	199.4	25.3 / 174.1***	-	-
Borehole 2	199.2	25.2 / 174.0***	-	-
Borehole 4	199.4	25.1 / 174.3	5-90	0-37
Borehole 5	199.2	25.3 / 173.9	57-100	0-97

* T.C.R. =Total Core Recovery

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

*** Bedrock surface was inferred from auger refusal.

At the borehole locations, the surface of the bedrock was contacted at Elevations ranging from 174.3 m (Borehole 4) to 173.9 (Borehole 5). From these results and from the regional geology the surface of the bedrock appears to be relatively flat in this project area. It should be also noted that bedrock surface elevation between and beyond the borehole location could vary.

Boreholes 4 and 5 were advanced into the bedrock for a vertical depth of 3.2 m (Borehole 5) and 4.5 m (Borehole 4) by NQ coring. As indicated on the Record of Borehole Sheet for Borehole 4 at south abutment location, a core barrel malfunction occurred in the first three runs (RC21, RC22 and RC23) in about the top 2 m of the rock coring, as was evidenced by the low total core recovery (TCR). However, the core barrel was subsequently fixed and a higher TCR of 90% was obtained in the next run (RC24). The minimum rock coring as per the terms of reference was 3 m but Coffey extended it to 4.5 m in Borehole 4 to obtain more representative cores. Based on the above issue, the recorded rock quality designation (RQD) values within the top 2 m in Borehole 4 are therefore considered not reliable. The RQD values are more reliable starting at about 2 m below the bedrock surface in Borehole 4 and are in the range of 35 to 37%.

Based on our visual examination of the rock core samples from Boreholes 4 and 5 in the laboratory, the upper portion of the shale bedrock (i.e. 1.8 m in Borehole 4 and 1.4 m in Borehole 5), corresponding to Elevation 172.5 m is highly weathered (Borehole 4) to weathered (Borehole 5) and fractured but this portion contains some relatively harder limestone layers/seams as well. In Borehole 4, the top 1.8 m of bedrock is highly weathered; however, it is likely that the recorded very low TCR and RQD values are partially due to core barrel malfunction as described above. Below the weathered portion in Borehole 4, the rock is relatively more competent than indicated by the reported TCR and RQD values and more closely resembles the rock cores obtained in Borehole 5 at the north abutment location.

The percentage of total core recovery was 57% to 100% except for the very low recovery portion due to above mentioned malfunction. The RQD values vary from zero to 97% which indicate a "very poor to excellent" rock quality based on Deere's classification. The RQD values were zero near the bedrock surface and increased generally from 35 to 97 % with depth.

Two unconfined compression tests were performed in the laboratory on selected intact rock core samples from Borehole 4 and Borehole 5 and the tests yielded unconfined compression strengths (UCS) of 13.4

MPa and 24.1 MPa, as shown on Table 4.4.2. The laboratory test results for rock core samples are attached in Appendix B.

Table 4.4.2: UCS Test Data

Borehole & Sample No.	Approximate Depth (m)	Approximate Elevation (m)	Bulk Density (KN/m ³)	Dry Density (KN/m ³)	UCS (MPa)
Borehole 4 –RC24	27.6	171.8	25.18	25.07	24.1
Borehole 5 –RC22	26.8	172.4	25.25	24.95	13.4

The UCS results indicate that the rock can be classified as weak under the conventions of the International Society for Rock Mechanics (ISRM).

4.5 Groundwater Conditions

Groundwater conditions were observed in the open boreholes during drilling and upon completion of each borehole. In Boreholes (Boreholes 4 and 5) where NQ coring was performed (i.e. water was 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 Table 4.5.1.

Table 4.5.1: Groundwater conditions

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Piezometer
Borehole 1	199.4		24.1*/175.3	Nov.6, 2007	
Borehole 2	199.2		-	Nov.7, 2007	
Borehole 3	199.8		-	July 21, 2009	
Borehole 4	199.4		1.2*/198.2	July 24, 2009	
Borehole 5	199.2	25.3/173.9	1.5/197.7	Aug.18, 2009	Yes
Borehole 6	199.2		-	July 21, 2009	

*groundwater table not stabilized

From the water level measured in piezometer installed at the location of Borehole 5 (about 4 weeks after drilling) and the change in soil colour from brown to grey, it is our opinion that groundwater level is about 1.5 to 3 m below the existing ground surface, corresponding to approximate Elevations 196 to 198 m, similar to Shashawandah Creek water level.

It should be noted that the groundwater level can vary and is subject to seasonal fluctuations and in response to major weather events. In addition, a perched water condition can occur due to the accumulation of surface water in the more pervious fill overlying the less pervious till deposit, especially during wet periods.

For and on behalf of Coffey Geotechnics Inc.

Hafiz Muneeb Ahmad, M.Eng., P.Eng.

for
Ramon Miranda, P.Eng.



Zuhtu Ozden, P.Eng.



Drawings

METRIC

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

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

CONT No.
GWP: 339-97-00

HIGHWAY 21
SHASHAWANDAH CREEK BRIDGE
BOREHOLE LOCATION PLAN



SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

- Recent Borehole (drilled July 2009)
- Previous Borehole (drilled November 2007)

No.	ELEVATION	STATION	OFFSET
BH 1	199.4	17+150	4.8m Rt C/L
BH 2	199.2	17+128	5.0m Lt C/L
BH 3	199.8	17+108	3.4m Rt C/L
BH 4	199.4	17+128	4.9m Rt C/L
BH 5	199.2	17+150	4.0m Rt C/L
BH 6	199.2	17+170	3.5m Rt C/L

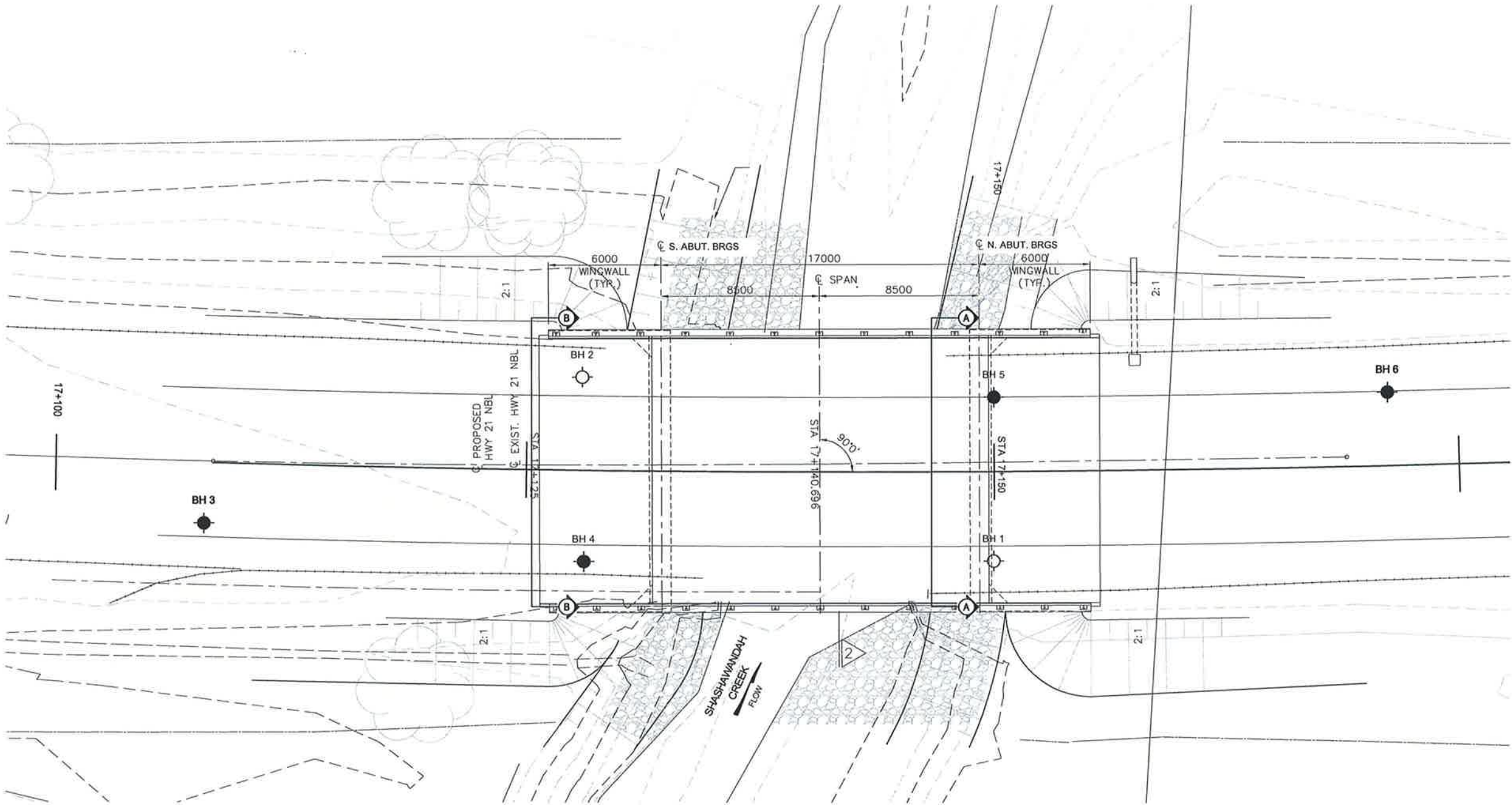
-NOTE-

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 40P4-48	TRANETO01210AA	DIST
SUBMD	CHECKED	DATE Oct. 2009
DRAWN PHK	CHECKED RM	APPROVED ZO



PLAN
SCALE



METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.

GWP: 339-97-00

HIGHWAY 21
SHASHAWANDAH CREEK BRIDGE
PROFILE

SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

- Recent Borehole (drilled July 2009)
- Previous Borehole (drilled November 2007)
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEVATION	STATION	OFFSET
BH 1	199.4	17+150	4.8m Rt C/L
BH 2	199.2	17+128	5.0m Lt C/L
BH 3	199.8	17+108	3.4m Rt C/L
BH 4	199.4	17+128	4.9m Rt C/L
BH 5	199.2	17+150	4.0m Rt C/L
BH 6	199.2	17+170	3.5m Rt C/L

-NOTE-

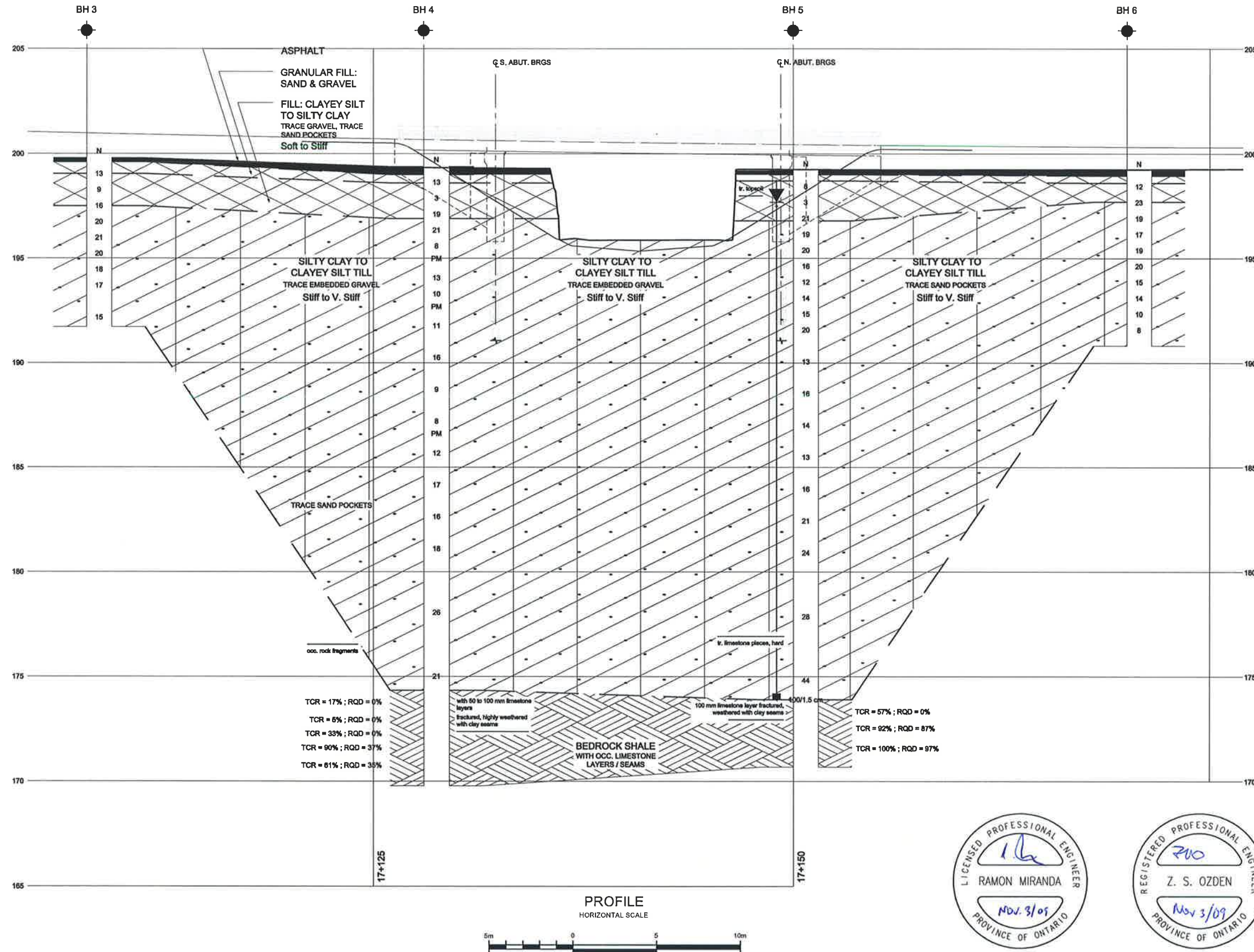
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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

REVISIONS	DATE	BY	DESCRIPTION

Geocore No 40P4-48

TRANET0801210AA				DIST
SUBMD	CHECKED	DATE	Oct 2009	SITE 14-2
DRAWN	PHK	CHECKED	RM	APPROVED ZO DWG 2



METRIC

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

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

CONT No.
GWP: 339-97-00

HIGHWAY 21
SHASHAWANDAH CREEK BRIDGE
SECTION (NORTH ABUTMENT)

SHEET



KEY PLAN
N.T.S.

LEGEND

- Recent Borehole (drilled July 2006)
- Previous Borehole (drilled November 2007)
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEVATION	STATION	OFFSET
BH 1	199.4	17+150	4.8m Rt C/L
BH 5	199.2	17+150	4.0m Lt C/L

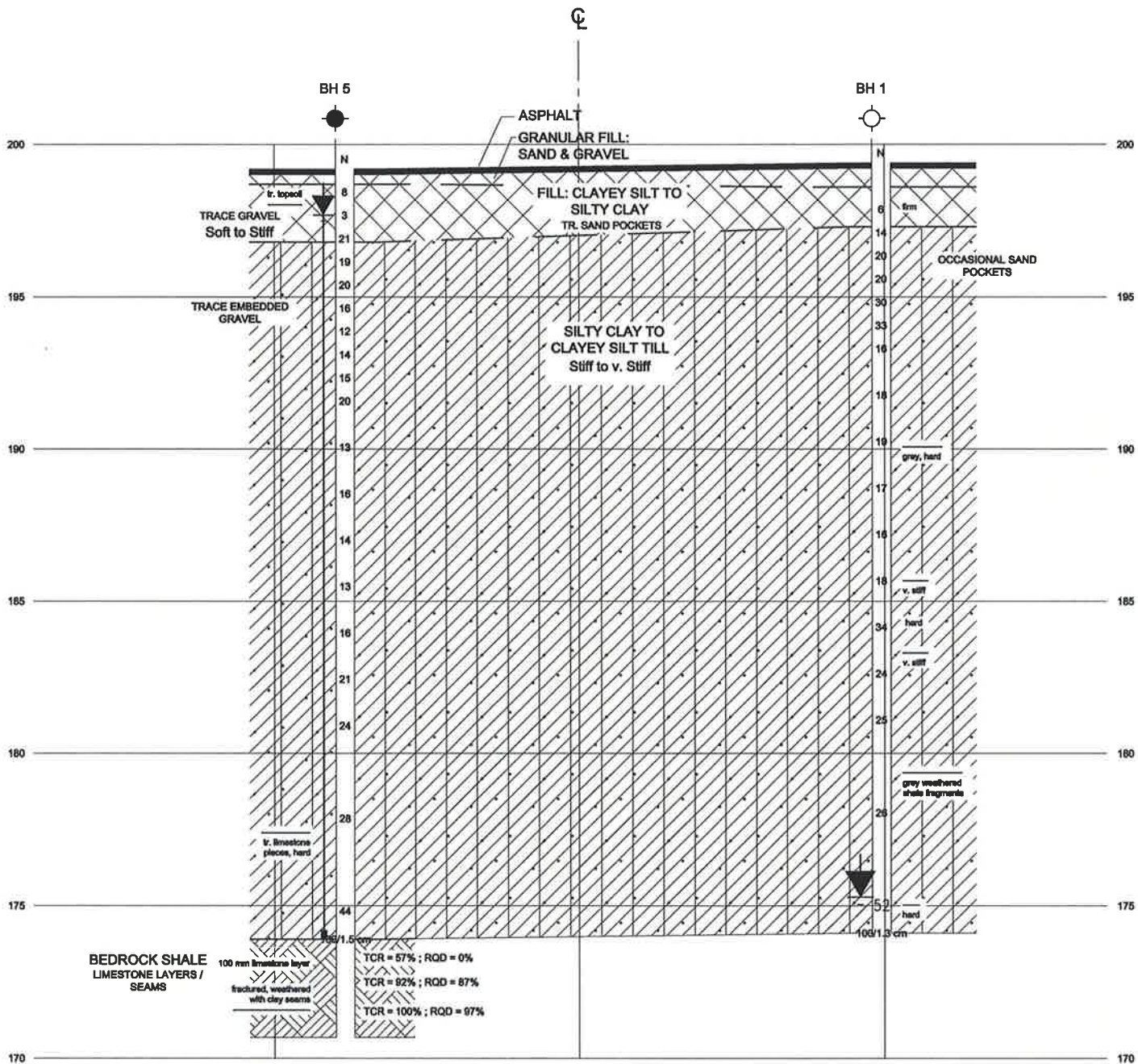
NOTE

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

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 40P4-48				TRANETOB01210AA		DIST	
SUBMD	CHECKED	DATE	Oct. 2009	SITE	14-2		
DRAWN	PHK	CHECKED	RM	APPROVED	ZO	DWG	3



SECTION A-A (NORTH ABUTMENT)

HORIZONTAL SCALE



METRIC

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

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

CONT No.
GWP: 339-97-00

HIGHWAY 21
SHASHAWANDAH CREEK BRIDGE
SECTION (SOUTH ABUTMENT)

SHEET



KEY PLAN
N.T.S.

LEGEND

- Recent Borehole (drilled July 2009)
- Previous Borehole (drilled November 2007)
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEVATION	STATION	OFFSET
BH 2	199.2	17+128	5.0m Lt C/L
BH 4	199.4	17+128	4.9m Rt C/L

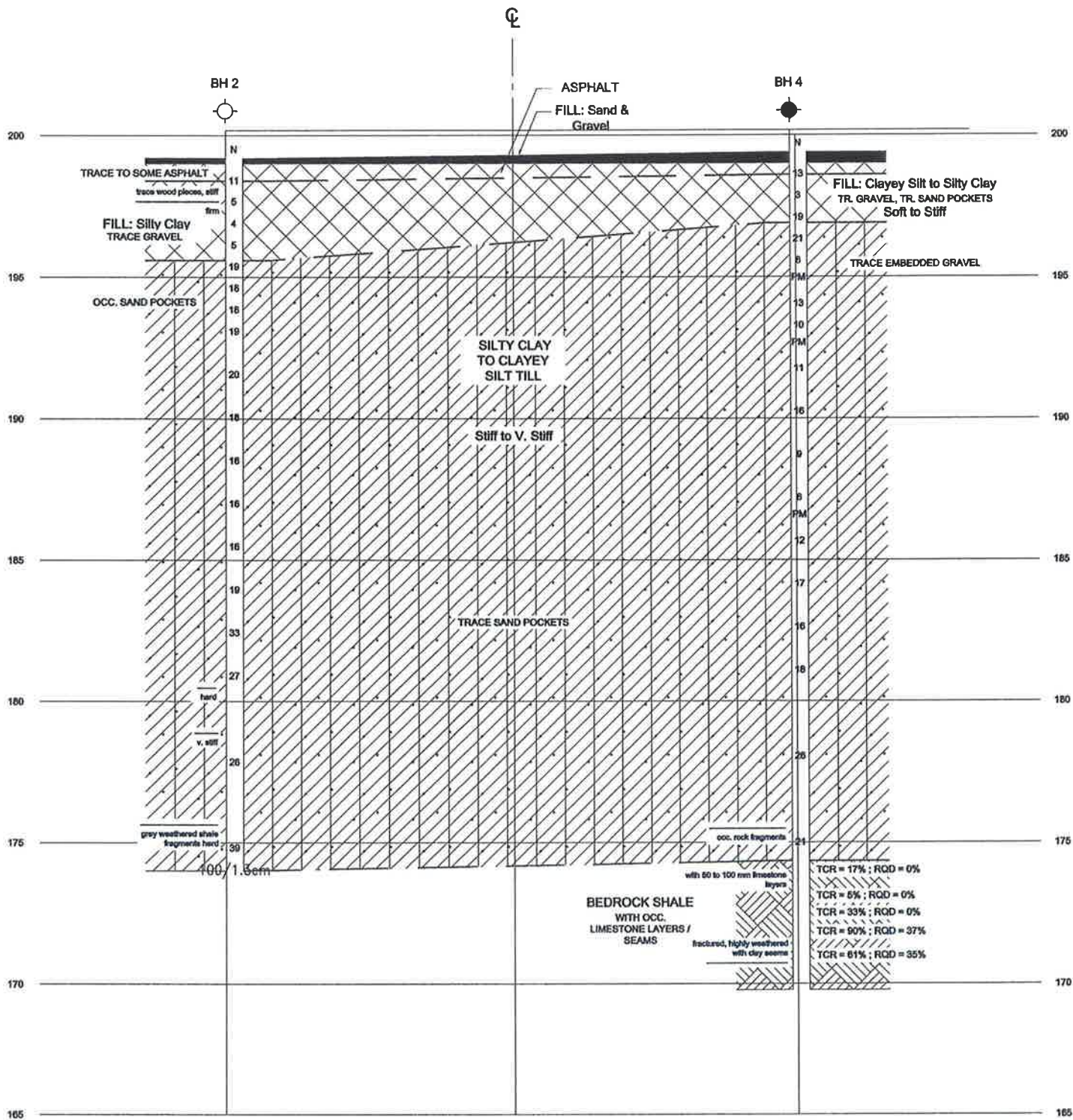
NOTE

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.

DATE	BY	DESCRIPTION

Geocres No 40P4-48				DIST	
TRANETO601210AA				SITE	
SUBMD	CHECKED	DATE	Oct. 2009	14-2	
DRAWN	PHK	CHECKED	RM	APPROVED	DWG



SECTION B-B (SOUTH ABUTMENT)

HORIZONTAL SCALE



Appendix A

Record of Borehole Sheets

SPT 1210: Shashawandah Creek Bridge

RECORD OF BOREHOLE No 1

1 OF 2

METRIC

GWP 339-97-00 LOCATION Sta : 17+150, 4.8 m Rt C/L on Paved Shoulder ORIGINATED BY NE
 DIST HWY 21 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SK
 DATUM Geodetic DATE 11/5/2007 11/6/2007 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)	WATER CONTENT (%)					
199.4														
199.0	200 mm ASPHALT		1	AS										
0.2	FILL: Sand & Gravel brown, damp		2	AS										
198.6			3	AS										
0.8	FILL: Silty Clay tr. sand pockets brown, moist, firm		4	SS	6									
197.3			5	SS	14									
2.1	SILTY CLAY TO CLAYEY SILT TILL occasional sand pockets, moist		6	SS	20									
			7	SS	20									
	brown, stiff to v. stiff		8	SS	30									
	grey, hard		9	SS	33									
			10	SS	16									
	v. stiff		11	SS	18									
			12	SS	19									
			13	SS	17									
			14	SS	16									
			15	SS	18									
184.4														

Continued Next Page

+³ × 3³: Numbers refer to
Sensitivity

20
15 10
(%) STRAIN AT FAILURE

SPT 1210: Shashawandah Creek Bridge

RECORD OF BOREHOLE No 1

2 OF 2

METRIC

GWP 339-97-00 LOCATION Sta : 17+150, 4.8 m Rt C/L on Paved Shoulder ORIGINATED BY NE
 DIST HWY HWY 21 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SK
 DATUM Geodetic DATE 11/5/2007 11/6/2007 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE						
184.4 15.0		hard	16	SS	34		184							1 19 45 35
		v. stiff	17	SS	24		183							
			18	SS	25		182							
	SILTY CLAY TO CLAYEY SILT TILL occ. sand pockets, moist		19	SS	26		181							
			20	SS	52		180							
	gray weathered shale fragments		21	SS	430/1.3 cm		179							Augering hard @ 20.1 m
		hard	22	SS	52		178							
			23	SS	52		177							
			24	SS	52		176							
174.1 25.3	End of Borehole Auger refusal @ 25.3 m, possibly on bedrock Water level @ 24.1 m (not stabilized)* upon completion		25	SS	52		175							Augering very hard @ 25.3 m on possible bedrock

+³, ×³ Numbers refer to
Sensitivity

20
15-5
10 (%) STRAIN AT FAILURE

SPT 1210: Shashawandah Creek Bridge

RECORD OF BOREHOLE No 2

1 OF 2

METRIC

GWP 339-97-00 LOCATION Sta : 17+128, 5.0 m Lt C/L on Paved Shoulder ORIGINATED BY NE
DIST HWY HWY 21 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SK
DATUM Geodetic DATE 11/6/2007 11/7/2007 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							WATER CONTENT (%)
								○ UNCONFINED ● POCKET PENETR.	+ FIELD VANE X LAB VANE						
199.2							20 40 60 80 100		10 20 30					GR SA SI CL	
198.0	200mm ASPHALT		1	AS											
0.2	FILL: Sand & Gravel trace to some asphalt brown, damp		2	AS										36 52 (12)	
198.4			3	SS	11										
0.8	trace wood pieces, stiff														
	FILL: Silty Clay trace Gravel brown, moist		4	SS	5									0 10 46 44	
			5	SS	4										
			6	SS	5										
195.6			7	SS	19										
3.6			8	SS	18										
	SILTY CLAY TO CLAYEY SILT TILL occ. sand pockets, grey, moist, very stiff		9	SS	18										
			10	SS	19									1 7 48 44	
			11	SS	20										
			12	SS	18										
			13	SS	16										
			14	SS	16										
			15	SS	16									1 4 48 47	
184.2															

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

SPT 1210: Shashawandah Creek Bridge

2 OF 2

METRIC

GWP	339-97-00	LOCATION	Sta : 17+128, 5.0 m Lt C/L on Paved Shoulder	ORIGINATED BY	NE
DIST		HWY	HWY 21	BOREHOLE TYPE	Hollow Stem Auger
DATUM	Geodetic	DATE	11/6/2007 11/7/2007	COMPILED BY	SK
				CHECKED BY	RM

+³, ×³: Numbers refer to Sensitivity

SPT 1210: Shashawandah Creek Bridge

RECORD OF BOREHOLE No 3

1 OF 1

METRIC

GWP 339-97-00 LOCATION Sta : 17+108, 3.4 m Rt C/L on Edge of Pavement ORIGINATED BY ZI
 DIST HWY 21 BOREHOLE TYPE Solid Stem Auger COMPILED BY SK
 DATUM Geodetic DATE 7/21/2009 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE								WATER CONTENT (%)		
199.8							20	40	60	80	100				GR	SA	SI	CL
198.6	200 mm ASPHALT																	
0.2	FILL: Sand & Gravel with Silt		1	AS											30	53	(17)	
199.3	FILL: Sand		2	AS											1	83	10	6
0.8	FILL: Clayey Silt to Silty clay tr. sand pockets brown, moist, stiff		3	SS	13	199												
199.0			4	SS	9	198												
0.8	asphalt pieces																	
197.5			5	SS	16	197									0	9	52	39
2.3	SILTY CLAY TO CLAYEY SILT TILL		6	SS	20													
	trace embedded gravel																	
	grey, moist, v. stiff		7	SS	21	196												
			8	SS	20	195												
			9	SS	18													
			10	SS	17	194												
						193												
191.7			11	SS	15	192												
8.1	End of Borehole @ 8.1 m Borehole dry (not stabilized) & open upon completion																	

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1210: Shashawandah Creek Bridge

RECORD OF BOREHOLE No 4

1 OF 3

METRIC

GWP 339-97-00 LOCATION Sta: 17+128, 4.9 m Rt C/L on Paved Shoulder ORIGINATED BY ZI
DIST HWY HWY 21 BOREHOLE TYPE Hollow Stem Auger, NQ Coring COMPILED BY SK
DATUM Geodetic DATE 23/07/2009 24/07/2009 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE						
199.4								20 40 60 80 100						
0.0	370 mm ASPHALT													
199.0														
0.4	FILL: Sand & Gravel brown, moist													
198.6														
0.8			1	SS	13									
	FILL: Clayey Silt to Silty Clay tr. gravel, tr. sand pockets brown, moist soft to stiff		2	SS	3									
196.9			3	SS	19									
2.5			4	SS	21									
			5	SS	8									
			6	TW	PM									
	SILTY CLAY TO CLAYEY SILT TILL													
	trace embedded gravel		7	SS	13									
	moist stiff to v. stiff		8	SS	10									
			9	TW	PM									
			10	SS	11									
			11	SS	16									
			12	SS	9									
			13	SS	8									
			14	TW	PM									
			15	SS	12									
184.4														

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1210: Shashawandah Creek Bridge

RECORD OF BOREHOLE No 4

2 OF 3

METRIC

GWP 339-97-00 LOCATION Sta : 17+128, 4.9 m Rt C/L on Paved Shoulder ORIGINATED BY ZI
DIST HWY HWY 21 BOREHOLE TYPE Hollow Stem Auger, NQ Coring COMPILED BY SK
DATUM Geodetic DATE 23/07/2009 24/07/2009 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED 20 40 60 80 100	+ FIELD VANE 20 40 60 80 100							
184.4 15.0	SILTY CLAY TO CLAYEY SILT TILL trace sand pockets grey, moist very stiff		16	SS	17	184								4 17 45 34		
							183									
					17	SS	16	182								
								181								
			18	SS	18	180										
						179										
						178										
			19	SS	26	177										
						176										
						175										
						174										
						173										
						172										
						171										
						170										
															</	

Continued Next Page

+ 3, X 3; Numbers refer to
Sensitivity

20
15 10 5
10 (%) STRAIN AT FAILURE

METRIC

GR SA SI CL

(%) STRAIN AT FAILURE

SPT 1210: Shashawandah Creek Bridge

RECORD OF BOREHOLE No 5

1 OF 2

METRIC

GWP 339-97-00 LOCATION Sta : 17+150, 4.0 m Lt C/L on Paved Shoulder ORIGINATED BY ZI
DIST HWY HWY 21 BOREHOLE TYPE Hollow Stem Auger, NQ Coring COMPILED BY SK
DATUM Geodetic DATE 7/21/2009 7/23/2009 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
199.2								20 40 60 80 100						GR SA SI CL
199.0	200 mm ASPHALT													
0.2	FILL: Sand & Gravel													
198.7														
0.5														
	tr. topsoil		1	SS	8		199							2 15 48 35
	FILL: Clayey Silt to Silty Clay tr. gravel, tr. sand pockets brown, moist, soft to stiff		2	SS	3		198							
196.8			3	SS	21		197							
2.4														
	brown													
	grey		4	SS	19		196							0 7 47 46
	SILTY CLAY TO CLAYEY SILT TILL													
	trace embedded gravel		5	SS	20		195							
	moist stiff to v. stiff		6	SS	16		194							
			7	SS	12		193							
			8	SS	14		192							6 6 47 41
			9	SS	15		191							
			10	SS	20		190							
			11	SS	13		189							
			12	SS	16		188							
			13	SS	14		187							
			14	SS	13		186							
184.2							185							

Continued Next Page

+ ³ . X ³ Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1210: Shashawandah Creek Bridge

RECORD OF BOREHOLE No 5

2 OF 2

METRIC

GWP 339-97-00 LOCATION Sta : 17+150, 4.0 m Lt C/L on Paved Shoulder ORIGINATED BY ZI
DIST HWY HWY 21 BOREHOLE TYPE Hollow Stem Auger, NQ Coring COMPILED BY SK
DATUM Geodetic DATE 7/21/2009 7/23/2009 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
184.2 15.0	SILTY CLAY TO CLAYEY SILT TILL grey, moist very stiff		15	SS	16		184							8 17 45 30
							183							
			16	SS	21		182							
							181							
			17	SS	24		180							
	tr. limestone pieces, hard						179							auger refusal @ 25.3 m and hole advanced by NQ Coring
							178							
			18	SS	28		177							
							176							
							175							
173.9 25.3	100 mm limestone layer		20	SS	44		174							auger refusal @ 25.3 m and hole advanced by NQ Coring
	fractured, weathered with clay seams		21	RC			173							
	BEDROCK grey, shale, with occ. limestone layers / seams						172							
			22	RC			171							
170.7 28.5	End of borehole @ 28.5 m Piezometer installed to 25.3 m Water level Records in Piezometer: (not stabilized) July 23, 2009 0.9 m July 24, 2009 0.7 m (Possibly due to coring) Aug 18, 2009 1.5 m													

+³, X³: Numbers refer to
Sensitivity

20
15-20
10 (%) STRAIN AT FAILURE

SPT 1210: Shashawandah Creek Bridge

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

GWP 339-97-00 LOCATION Sta : 17+170, 3.5 m Lt C/L on Edge of Pavement ORIGINATED BY ZI
 DIST HWY HWY 21 BOREHOLE TYPE Solid Stem Auger COMPILED BY SK
 DATUM Geodetic DATE 7/21/2009 CHECKED BY RM

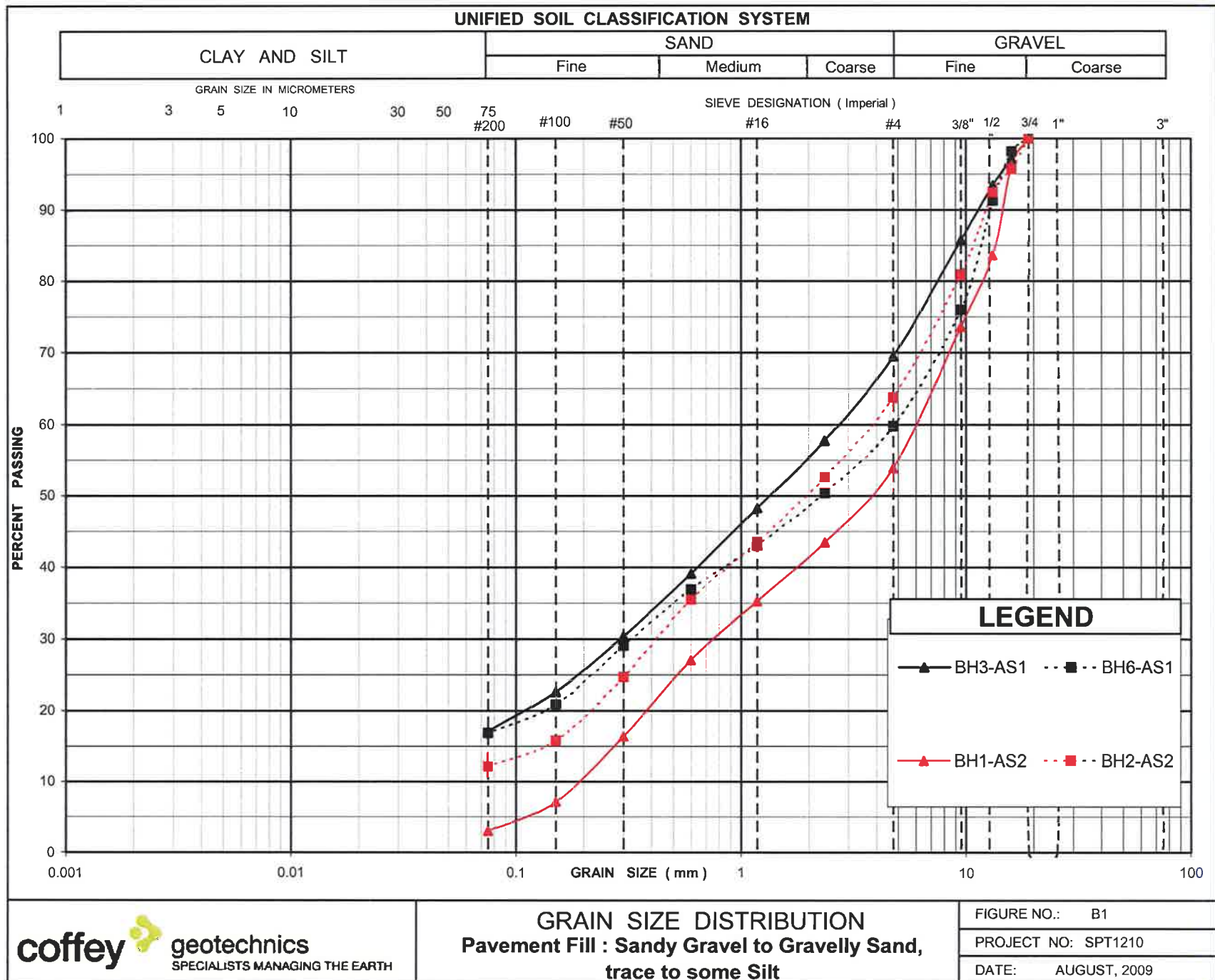
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)								WATER CONTENT (%)
								○ UNCONFINED + FIELD VANE								
								● POCKET PENETR. × LAB VANE								
199.2						20	40	60	80	100						
199.8	250 mm ASPHALT															
0.3	FILL: Sand & Gravel		1	AS											40 43 (17)	
198.6			2	AS												
0.6	FILL: Clayey Silt to Silty Clay tr. gravel grey, moist, stiff		3	SS	12										no recovery	
197.7			4	SS	23										0 6 51 43	
1.5	SILTY CLAY TO CLAYEY SILT TILL trace sand pockets grey, moist stiff to v. stiff		5	SS	19											
			6	SS	17											
			7	SS	19											
			8	SS	20											
			9	SS	15											
			10	SS	14											
			11	SS	10											
			12	SS	8											
190.8																
8.4	End of borehole @ 8.4 m Hole dry (not stabilized) & open upon completion															

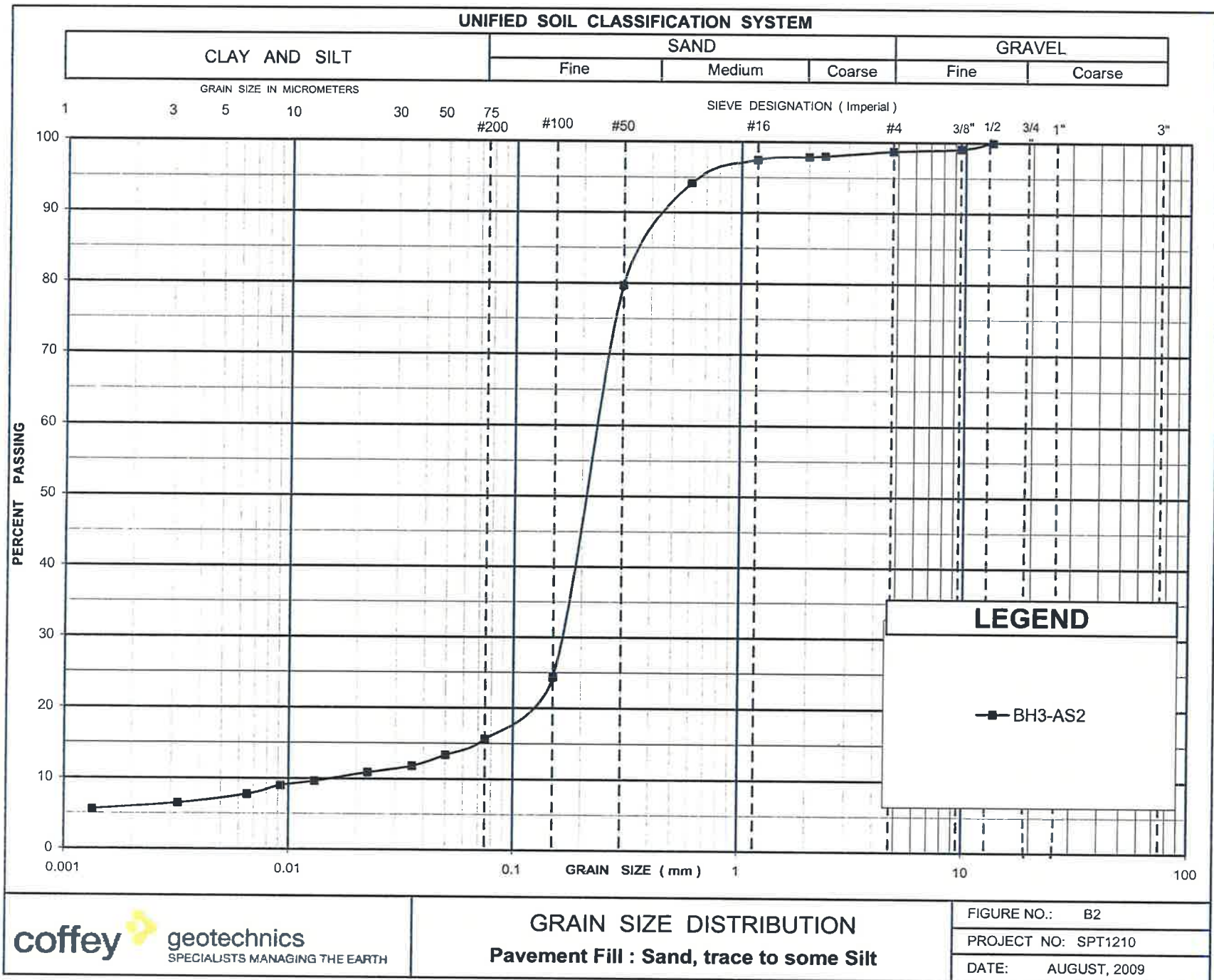
+³, ×³: Numbers refer to
Sensitivity

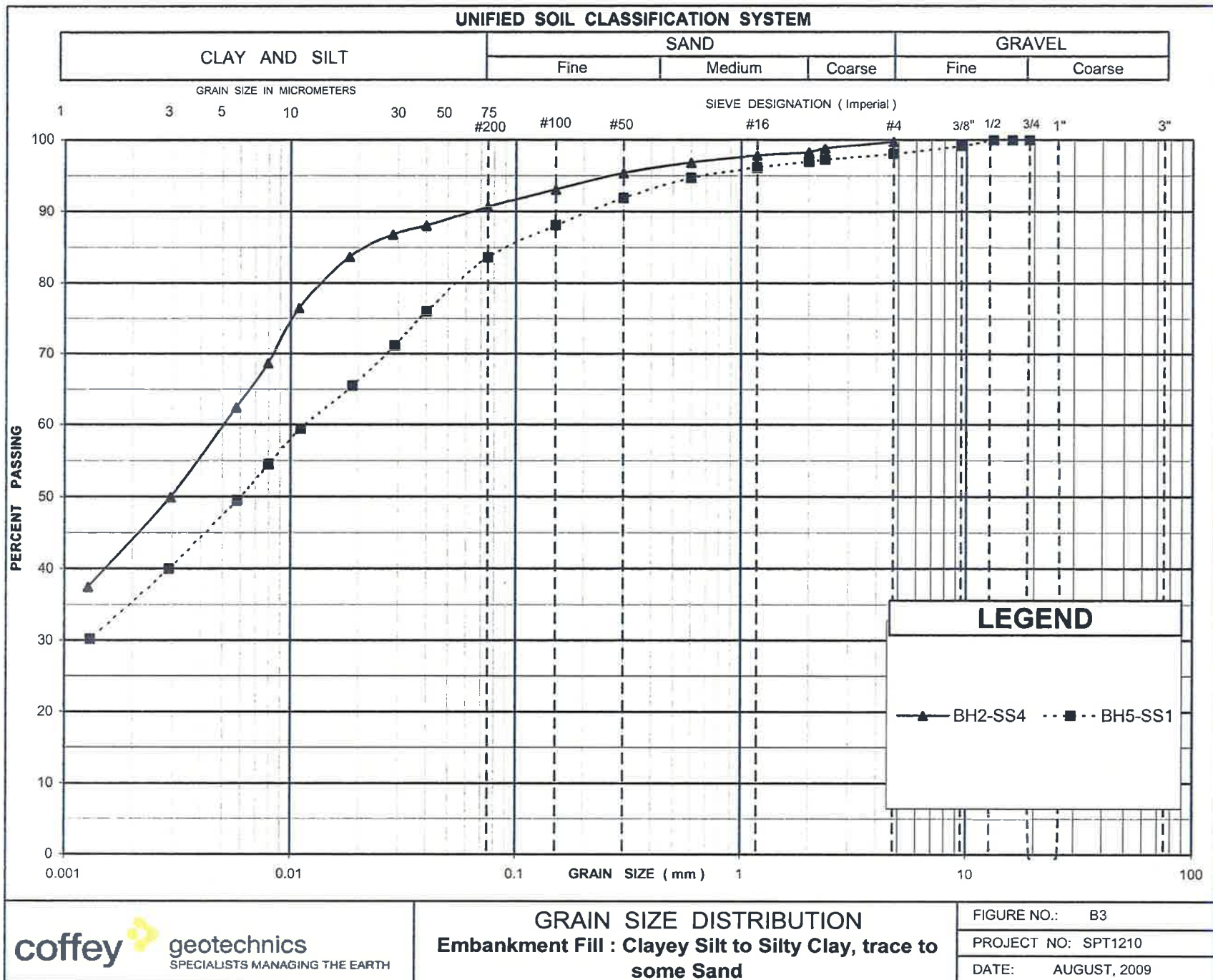
20
15 10 5
(%) STRAIN AT FAILURE

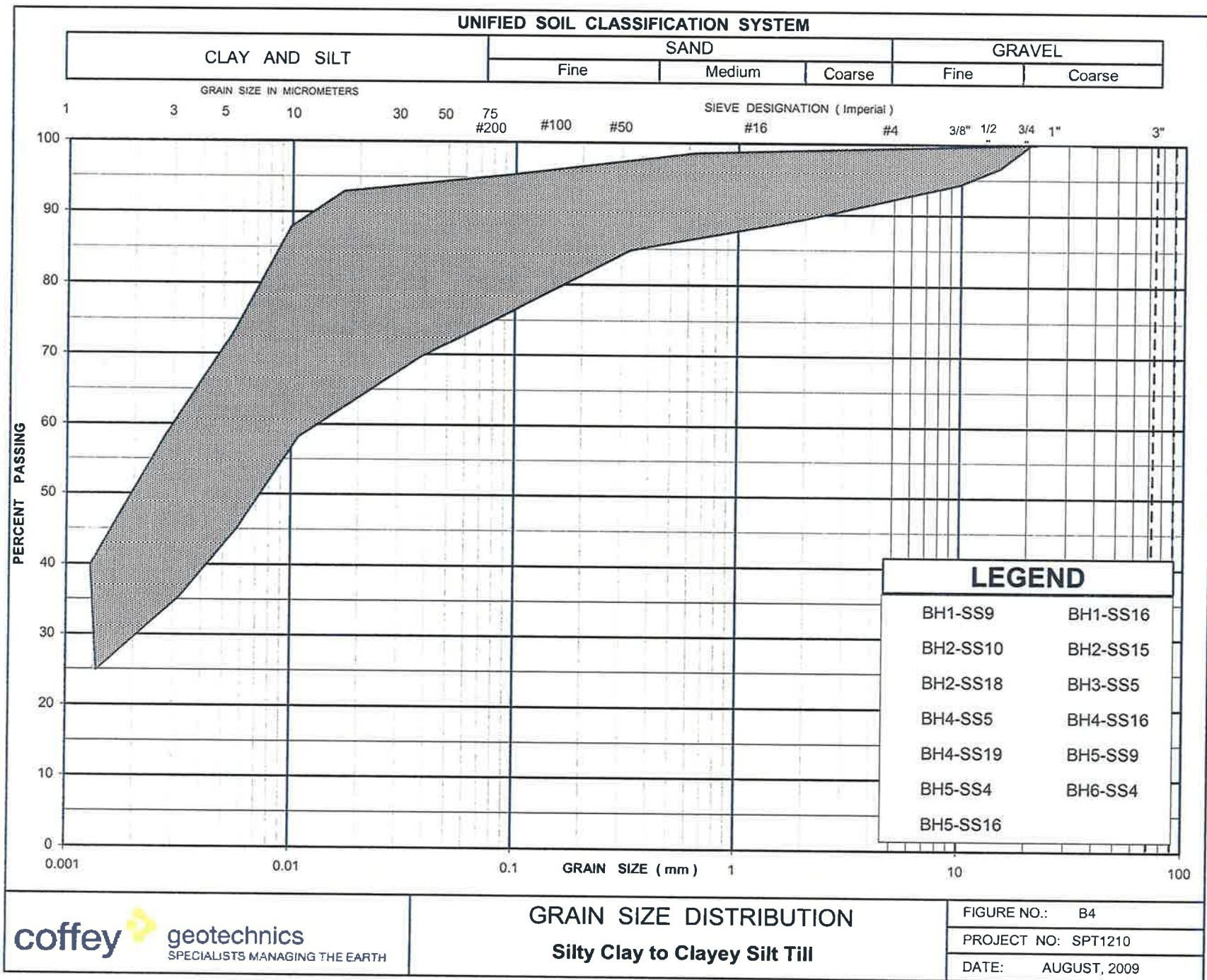
Appendix B

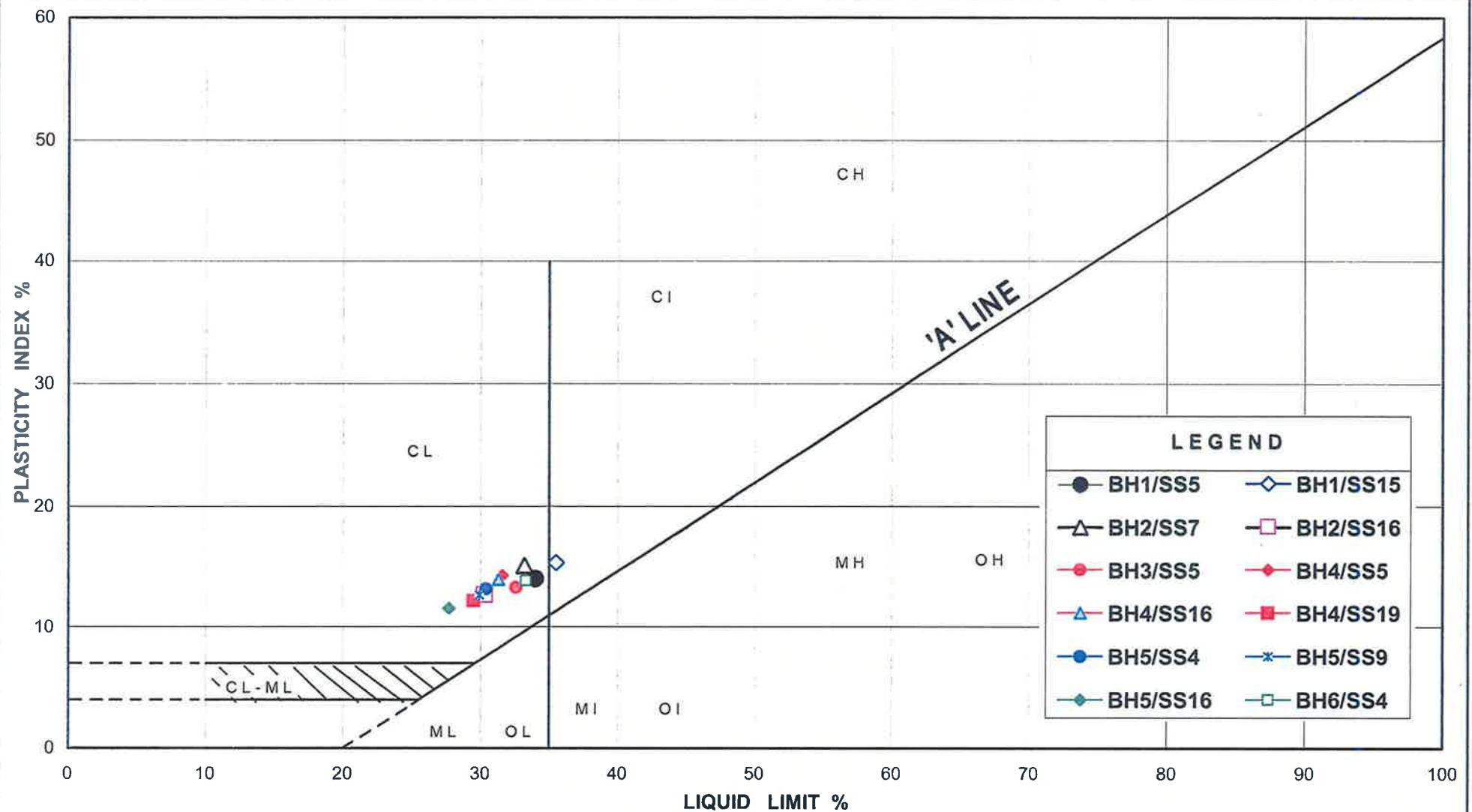
Laboratory Test Results











Appendix C

Site Photographs

Photo 1: Shashawandah Bridge (looking north from a distance)



Photo 2: Close view of Shashawandah Bridge (looking south)



Photo 3: Bridge abutments and water flowing in the Creek (looking north)



Photo 4: The existing pond near the Creek on east side of the Bridge (looking north)



Appendix D

Rock Core Photographs

Photo 1: Rock Core samples from borehole 4



Photo 2: Rock Core samples from borehole 5



Appendix E

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICALL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

**FOUNDATION DESIGN REPORT
REPLACEMENT OF SHASHAWANDAH
CREEK BRIDGE AT STATION 17+140.696
HIGHWAY 21, NORTH OF TOWN OF FOREST
LAMBTON COUNTY, ONTARIO
SITE NO. 14-2, G.W.P. 339-97-00
GEOCRES NO. 40P4-48**

Stantec Consulting Ltd.

Project: TRANETOB01210AA
November 3, 2009

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**FOUNDATION DESIGN REPORT
REPLACEMENT OF SHASHAWANDAH CREEK BRIDGE AT STATION 17+140.696
HIGHWAY 21, NORTH OF TOWN OF FOREST, COUNTRY OF LAMBTON, ONTARIO
SITE NO. 14-2, G.W.P. 339-97-00, Geocres No. 40P4-48**

5. DISCUSSION AND RECOMMENDATIONS

The existing Shashawandah Creek Bridge is a 11.3 m long single span structure with two lanes, including the paved shoulder. It is our understanding that the existing bridge will be replaced with a 17 m long single span structure. The width of the proposed structure will be 15.2 m which will be approximately 2.5 m wider than the existing bridge deck. The longitudinal centre line of the new bridge will be located about 0.4 m east of the centreline of the existing bridge. The proposed bridge will carry two lanes of traffic, similar to the existing one. It is our understanding that staged construction utilizing single lane closures is proposed (i.e. one half of the existing bridge will be demolished, then one half of the new bridge will be constructed while a single lane traffic with a temporary traffic signal will be maintained on the remaining half of the existing bridge. Once the construction of the one half of the new bridge is completed, the traffic will be diverted to the newly built half bridge, after which the remaining half of the existing bridge will be demolished and the other half of the new bridge will be constructed) requiring roadway protection.

The existing grades at the location of boreholes vary from about 199.2 m (Boreholes 2, 5 and 6) to 199.8 m (Borehole 3). The proposed elevation of the top of the bridge structure will be raised by approximately 1.4± m from the existing grades.

The materials encountered in the boreholes consisted of asphalt pavement structure, embankment fill (clayey silt to silty clay) and the underlying silty clay to clayey silt till extending to the surface shale bedrock which has occasional layers/seams of limestone. Underlying the pavement structure, a 0.9 to 2.8 m thick layer of clayey silt to silty clay embankment fill was encountered in boreholes, extending to depths varying from 1.5 m in Borehole 6 to 3.6 m in Borehole 2, corresponding to elevations from 197.7 to 195.6 m. The embankment fill contains wood pieces in the upper portion in Borehole 2, asphalt pieces in the lower portion in Borehole 3 and pockets topsoil/organics at the top portion of this embankment fill in Borehole 5. The native soil in the boreholes is composed of silty clay to clayey silt till with occasional seams/ pockets of sand and this extended to depths varying from 25.1 to 25.3 m in deep boreholes (Boreholes 1, 2, 4 and 5), corresponding to elevations from 174.3 to 173.9 m. The clayey deposit extended to maximum explored depth of about 8.3 m in the remaining two relatively shallow boreholes drilled at the location of approach embankments on each side of the existing bridge. The native deposit is typically brown colour in the upper portion and becoming grey at depths varying from 3.0 to 4.3 m below the existing grade. The lower portion of the till contains weathered shale and/or limestone fragments.

Grey shale bedrock with occasional limestone layers/seams was encountered in two boreholes of the detail investigation (Boreholes 4 and 5) and was proven by NQ coring at depths varying from 25.1 to 25.3 m below the existing road grade or El. 173.9 to 174.3 m. Auger refusal on inferred bedrock surface was encountered in the two deep boreholes of preliminary investigation (Boreholes 1 and 2) at approximate depth of 25.3 m below the existing grade, corresponding to approximate Elevation 174.0 m. The bedrock was not cored in Boreholes 1 and 2, as this was not within the terms of reference of the preliminary geotechnical investigation. From these results and from the regional geology, the surface of the bedrock

appears to be relatively flat in this project area. Based on our visual examination of the rock core samples, the upper portion of the bedrock (i.e. 1.8 m in Borehole 4 and 1.4 m in Borehole 5), corresponding to Elevation 172.5 m is highly weathered (Borehole 4) to weathered (Borehole 5) and fractured but contains some limestone layers/seams which are harder than the typical shale bedrock encountered at the site.

From the water level measured in piezometer installed at the location of Borehole 5 (about four weeks after the completion of drilling) and the change in soil colour from brown to grey at the borehole locations, it is our opinion that groundwater level is about 1.5 to 3 m below the existing ground surface, corresponding to approximate Elevation 196 to 198 m similar to Shashawandah Creek water level. It should be noted that the groundwater level can vary and is subject to seasonal fluctuations and in response to major weather events. In addition, a perched water condition can occur due to the accumulation of surface water in the more pervious fill overlying the practically impervious silty clay to clayey silt till deposit, especially during wet periods.

5.1 Bridge Foundations

Based on the available drawing, provided to us by Stantec, the existing bridge abutments are supported on conventional spread footings founded on the native soil. Based on the results of investigation, we have considered a number of foundation options varying from normal spread footings to deep foundations which include drilled caissons and driven steel piles in view of following factors.

- Existing sub surface conditions
- Cost effectiveness
- Reliability
- Constructability

The use of normal spread footings was considered for the proposed abutments. The geotechnical resistance at the founding levels in some boreholes (e.g. Borehole No. 4) is rather low. Moreover, this option will necessitate extensive excavations adjacent to the existing bridge foundations. As well, integral type abutment design is the preferred option for this bridge which requires driven steel H-piles for the support of the abutments. For these reasons, it is our opinion that the use of driven steel H-piles is the preferred option for this project.

The summary of various foundation options are summarized in Table F1 in Appendix F.

The available foundation options for the proposed bridge structure are discussed in the following paragraphs.

5.1.1 Steel H-Piles

The boreholes show that the geotechnical conditions at the site are suitable for the use of driven steel H-piles to support the proposed Shashawandah Creek Bridge at Highway 21. As mentioned in Table F1, attached in Appendix F, driven steel H-piles are considered to be the preferred option at this site, especially if the option of integral abutments is used, as shown on Drawing No. 1. Because of the presence of possible cobbles and boulders in the till deposits and shale and limestone fragments encountered above

the shale bedrock, low displacement piles (i.e. steel H-piles) are better suited in comparison with 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. These should be flush-mount type. Care must be taken to avoid overdriving and damaging the pile tip. It should be noted that for piles driving to bedrock, the current version of SP 903S01 requires the Contractor to adequately seat the pile on bedrock without damaging the pile and rock points will penetrate into the rock.

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 (ULS) and Axial Resistance at the Serviceability Limit State (SLS) need not be considered. Alternatively, for HP 310x125 steel H-piles which are driven to practical refusal in the bedrock, normally a value of 2250 kN (per pile) is utilized at ULS. In this case, however, the upper portion of the bedrock, about 1.8 to 1.4 m in depth, corresponding to Elevation 172.5 m is highly weathered to weathered and fractured but contains some limestone layers/seams which are harder than the typical shale bedrock encountered at the site. In view of this fact, we recommend that for design purposes 1800 kN/pile (HP 310x110) factored geotechnical resistance at ULS be used for piles driven to practical refusal in the bedrock and SLS will not govern. The heavier H-pile section such as HP 310X125 can be used for 2000 kN/pile factored geotechnical resistance at ULS and SLS need not be considered.

Based on our experience steel H-piles can often penetrate into the highly weathered zone of shale bedrock near the bedrock surface before 'setting-up' during the pile driving process. Based on this, the estimated pile tip elevations are provided in Table 5.1.1.1.

Table 5.1.1.1: Anticipated Pile Tip Depths/Elevations

Support Location	Borehole No.	Existing Ground Surface Elevation (m)	Estimated Pile Cut-Off Elevation (m)	Estimated Depth of Pile Tip Below Existing Ground Surface (m)	Estimated Pile Tip Elevation (m)	Estimated Approximate Pile Length (m)
North Abutment	Borehole 1	199.4	196.4*	26.7	172.7	23.7
	Borehole 5	199.2		26.5	172.7	23.7
South Abutment	Borehole 2	199.2		26.5	172.7	23.7
	Borehole 4	199.4		26.7	172.7	23.7

* Pile cut-off elevation is estimated about 0.6 m higher than the proposed design base elevation for abutments (El. 195.8 m).

The pile tip elevations provided in above table are for estimating purposes only. Due to possible variations in the bedrock surface elevation and the presence of the hard limestone layers/seams in the bedrock, the actual pile tip elevation will vary. The contract should allow for some variations in pile lengths and this aspect should be taken into consideration when ordering the piles. The possibility of piles encountering potential cobbles and boulders in the till or shale and limestone fragments above the bedrock surface

should be anticipated. An NSSP should be provided in the Contract Documents to warn the Contractor the possible presence of cobbles and boulders in the overburden and of large rock fragments/slabs immediately above the bedrock. The suggested NSSP wording is attached in Appendix G.

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 to avoid possible damage to the piles. In this regard, for piles driven to the refusal in the bedrock, it is generally accepted practice at MTO 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 is recommended 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 SP 903S01. Re-striking should be done as per SP 903S01. 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 SP903 S01, to check that relaxation has not occurred. If it occurs, then all the piles should be re-tapped.

While pile heave 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.

For frost protection, all pile caps should have a permanent earth cover of at least 1.2 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. Reference may be made to Section C6-8.7.1 of the Canadian Highway Bridge Design Code (2006), CHBDC, for assessing lateral pile resistance.

In cohesionless soils (these soils were not encountered below the approximate pile cut-off elevation at the borehole locations, as indicated in Table 5.1.1.2), the coefficient of horizontal sub-grade reaction can be estimated from:

$$k_s = n_h z / d$$

Where k_s = coefficient of horizontal sub-grade reaction

z = depth

d = pile width

n_h = coefficient related to soil density as given in Table 5.1.1.2.

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 (as encountered generally at the site at borehole locations), 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 5.1.1.2: Recommended Design Values

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m ³)	Angle of Internal Friction (ϕ) Degrees	Recommended n_h Value (MN/m ³)	Recommended Undrained Shear Strength, c_u (kPa)
North Abutment/ Borehole 1	196.4-184.3	Silty clay to clayey silt till	20.0	-	-	140
	184.3-175.3	Silty clay to clayey silt till	20.5	-	-	180
	175.3 -174.1	Silty clay to clayey silt till	21.5	-	-	220
North Abutment/ Borehole 5	196.4-184.3	Clayey silt to silty clay till	20.0	-	-	120
	184.3-176.3	Silty clay to clayey silt till	20.5	-	-	180
	176.3-173.9	Silty clay to clayey silt till	21.5	-	-	220
South Abutment/ Borehole 2	195.6-182.5	Silty clay to clayey silt till	20.5	-	-	120
	182.5 -174.0	Silty clay to clayey silt till	21.5	-	-	180
South Abutment/ Borehole 4	196.4-184.3	Clayey silt to silty clay till	20.0	-	-	75
	184.3-174.3	Clayey silt to silty clay till	21.0	-	-	150

For preliminary design purposes, the recommended horizontal resistances for HP 310 x 110 or HP 310 x 125 steel H-piles are as follows:

Horizontal Resistance at ULS = 200 kN/pile

Horizontal Resistance at SLS = 110 kN/pile

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

In accordance with MTO structural office requirements (Report SO-96-01), 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 5.1.1.2: 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 Steel Tube Piles

In general, tube piles will provide lower resistances in comparison with H-piles as they will not be driven as deep as H-piles, but the lower resistances may be compensated to some extent by the relatively shorter pile lengths. As mentioned in Table F1, attached in Appendix F, steel tube piles can not be used for integral abutment which we understand is the preferred foundation option for the proposed bridge. These piles have the advantage that they can be inspected after driving and prior to pouring the concrete for possible damage that may have incurred while driving. They should have sufficient wall thickness and base plate thickness to minimize potential damage caused by the expected hard driving conditions. The end plates should not be wider than the base area of the piles (i.e. should not project beyond the circumference of the pile) so that adhesion/friction is not adversely affected. Tube piles will need to be filled with concrete after their installation and inspection for possible damage.

Steel tube piles of 300 mm nominal diameter (e.g. 324 mm x 9.4 mm) driven to approximate Elevation 174 m can be designed to provide a Factored Axial Resistance at ULS of 1000 kN/pile and an Axial Resistance at SLS equal to 700 kN/pile. The piles will need to be driven using a suitably heavy hammer capable of delivering a rated energy of at least 60 kilojoules/blow, but not more than 70 kilojoules/blow. The anticipated pile tips depths/elevations of steel tube piles (324 mm x 9.4 mm) are given in the Table 5.12.1.

Table 5.1.2.1: Anticipated Pile Tip Depths/Elevations

Support Location	Borehole No.	Existing Ground Surface Elevation (m)	Estimated Pile Cut-Off Elevation (m)	Estimated Depth of Pile Tip Below Existing Ground Surface (m)	Estimated Pile Tip Elevation (m)	Estimated Approximate Pile Length (m)
North Abutment	Borehole 1	199.4	196.4*	25.3	174.1	22.3
	Borehole 5	199.2		25.1	174.1	22.3
South Abutment	Borehole 2	199.2		25.2	174.0	22.4
	Borehole 4	199.4		25.1	174.3	22.1

*Pile cut-off elevation is assumed about 0.6 m higher than the proposed design base elevation for abutments (El. 195.8 m).

5.1.3 Drilled Caissons

As explained in the Table F1 in Appendix F, the drilled caissons are not preferred, considering the option of integral type of abutment structure at the site. The proposed bridge structure can be supported on drilled caissons extending about 1.5 m into shale bedrock (El. 172.4 m) for factored geotechnical resistance of 3000 kPa at ULS and Axial Resistance at SLS need not be considered. Drilled caissons founded at least 0.3 m into sound bedrock (i.e. at or below El. 172.1 m) can be designed for a geotechnical resistance of 4000 kPa at ULS. Caisson foundations on weathered bedrock, extended at least 0.3 m below the surface of bedrock (i.e. at or below El.173.6 m) can be designed for a geotechnical axial resistance of 2400 kPa at ULS, and SLS need not be considered. However, the use of drilled caisson foundations is not recommended due to a deep drilling (i.e. based on cost factor).

5.1.4 Spread Footing Foundations

The proposed bridge structure can be supported by conventional spread footings founded on the undisturbed native deposit similar to the existing bridge foundations. For the north abutment (i.e. Boreholes 1 and 5), the recommended factored geotechnical resistance at ULS is 300 kPa and the available factored geotechnical resistance at SLS is 200 kPa at or below El. 196.6 m. At the south abutment location (Boreholes 2 and 4), the recommended factored ULS value is 230 kPa and on SLS value equal to 150 kPa can be used at or below El. 196.9 m. However, the fill encountered in Borehole 2 extending to El. 195.6 m must be removed to the surface of the silty clay to clayey silt till deposit and replaced with mass concrete. All footing excavations must be inspected, evaluated and approved by a geotechnical engineer appointed by the QVE. The geotechnical resistances and the corresponding founding elevations at the borehole locations are summarized on the Table 5.1.4.1.

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.

Under inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with Clause 6.7.4 of the CHBDC (Canadian Highway Bridge Design Code-CAN/CSA-S6-06).

Sliding resistance can be provided by utilizing the sliding resistance between the concrete footing base and the founding soil. For the evaluation of the sliding resistance of the foundation (CHBDC. 6.7.5) the ultimate angle of friction between the underside of the foundations and the clayey silt to silty clay till (or between concrete surfaces) can be taken as 26° .

Table 5.1.4.1:

Geotechnical Resistances & Founding Levels of Spread Footings Founded on Native Soil

Borehole No.	Material	Geotechnical Resistance at SLS (kPa)	Factored Geotechnical Resistance at ULS (kPa)	Minimum Depth below Existing Grade (m)	Founding Level At or Below Elevation (m)	Note
Borehole 1	Silty clay to clayey silt till	200	300	2.8	196.6	North abutment
Borehole 5	Silty clay to clayey silt till	200	300	2.6	196.6	
Borehole 2	Silty clay to clayey silt till	150	230	3.6	195.6	South abutment
Borehole 4	Silty clay to clayey silt till	150	230	2.5	196.9	

The selection of footing depths will require careful evaluation of the foundation levels for the existing bridge. We recommend that footings of the new structure be placed at approximately the same elevations as the existing foundations. In addition, frost and scour depths should be given considerations.

Spread footings on compacted Granular 'A' pad are not recommended based on impracticability of construction adjacent to the one half of the existing bridge footings which will be retained for staged construction.

Foundations designed to the geotechnical resistance at SLS are expected to settle not to exceed 25 mm total and 20 mm differential, provided that the footings are placed on undisturbed, approved, natural soil. However, as discussed later in Section 5.4 of this report, the existing embankment will be widened and raised. In addition to structural loads which are expected to induce about 25 mm settlement, the settlements due to the grade raise and widening must be taken into consideration, bringing the total maximum settlement to about 65 mm.

Spread footing foundations are unlikely to be suitable for the support of this highway bridge due to low geotechnical resistance and high settlements and due to the fact that excavations adjacent to the existing

structure footings will be required. As well, the use of integral abutments require driven H-pile foundations as was discussed before. For these reasons, we do not recommend the use of spread footings for this project. The recommended foundation support for this project is therefore steel H-piles driven to the surface of bedrock.

5.2 Retaining Wall Foundations

Foundation conditions for normal spread foundations were discussed in the previous section 5.1.4 of this report and will not be repeated here. For the design of spread footing foundations for proposed retaining walls, the geotechnical resistances and the corresponding founding elevations at the borehole locations are summarized on the Table 5.2.1.

Consideration can be given to a Retained Soil System (RSS) for integral abutment, where the design details permit this type of wall. The RSS should be designed and constructed as per SP 599S22 and SP 599S23. These specifications are attached in Appendix F.

Table 5.2.1: Geotechnical Resistances & Founding Levels of Spread Footings for Retaining Walls Founded on Native Soil

Borehole No.	Material	Geotechnical Resistance at SLS (kPa)	Factored Geotechnical Resistance at ULS (kPa)	Minimum Depth below Existing Grade (m)	Founding Level At or Below Elevation (m)	Note
Borehole 1	Silty clay to clayey silt till	200	300	2.8	196.6	North-east wing wall
Borehole 5	Silty clay to clayey silt till	200	300	2.5	196.7	North-west wing wall
Borehole 2	Silty clay to clayey silt till	180	240	3.7	195.5	South-west wing wall
Borehole 4	Silty clay to clayey silt till	150	230	2.5	196.9	South-east wing wall

5.3 Lateral Earth Pressures

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free draining granular materials in accordance with the Ontario Ministry of Transportation and Ontario Provincial Standards.

Granular backfill to be placed behind the abutment walls and wing walls should conform to the minimum requirements illustrated in OPSP 3101.150. The granular backfill should conform to OPSS 1010 for either

Granular 'A' or 'B' Type I and Type II. To maintain free draining characteristics in these granular fill materials, the maximum percentage passing the No. 200 sieve (75 μ m) should be limited to 5%.

The backfill should be placed in accordance with SP 105S10. A perforated subdrain should be installed behind the base of the walls as shown in OPSD 3101.150 to maintain the granular fill in a drained condition. The subdrain should be directed to a highway drainage system.

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

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

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

Unit weight = 22 kN/m³

Coefficients of Lateral Earth Pressure:

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

Compacted Granular 'B' Type I or Granular 'B' Type III

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

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.31$	$K_a=0.42$	$K_a=0.54$
$K_b=0.41$	$K_b=0.52$	$K_b=0.64$
$K_o=0.47$	$K_o=0.66$	$K_o=0.76$
$K^*=0.57$	$K^*=0.74$	$K^*=0.86$

NOTE:

K_a is the coefficient of active earth pressure

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

K_o is the coefficient of earth pressure at rest

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

The Granular 'B' Type III should meet the gradations as per Amendment to OPSS1010, April 2004

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

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. In the case of a rigid frame structure yielding is unlikely and therefore at rest pressures should be used, as per Clause 6.9.2 of CAN/CSA-S6-06 CHBDC. The effect of compaction during construction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6.9.2 of CAN/CSA-S6-06 CHBDC. The use of vibratory compaction equipment behind the retaining walls should be restricted in size as per current MTO and municipal practice. Vibration generated by traffic should also be considered in the selection of appropriate earth pressure coefficients.

5.4 Embankment Stability and Settlements

By estimating from Drawing No.1, the existing embankment grade will be raised by about 1.2 to 1.4 m and will be widened by approximately 2.5 m.

Based on the findings of investigation, the existing fill has not received a systematic compaction and has a soft to stiff consistency. It is recommended that a portion of the existing fill in the approach embankments which has a lack of compaction should be removed under the footprint of the approach embankments before placing the additional fill. From the borehole investigation, it was found that the existing fill was not properly compacted at the location of Boreholes 1, 2, 4 and 5. The compaction of the existing fill as inferred in the remaining two boreholes, at the location of approach embankments, on each side of the existing bridge was considered to be acceptable. Based on interpretation between the boreholes, the existing fill from the approach embankments, about 10 m on each side of the proposed abutments, should be removed and replaced with compacted engineered fill. The excavated fill free from topsoil/organics and construction debris such wood pieces can be re-used to raise the grade, if weather permits. No stability problems are anticipated due to foundation conditions for the proposed height of embankments (i.e. up to approximately 4.0 to 4.5 m). Conventional embankment slopes of 2H:1V will be stable, provided the subgrade is properly prepared.

All organic and otherwise unsuitable soils should be removed within an envelope given by an imaginary slope no steeper than 1H:1V from the toe of the proposed embankment, as per MTO standard procedures. After stripping, the exposed subgrade should be inspected and approved. It should then be compacted, where feasible, from the surface using a suitable compactor.

Proper benching of existing embankment slopes should be implemented when widening any existing embankment, as per MTO procedures and in accordance with OPSD 208.010, as illustrated in Appendix G.

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

Assuming that the uncompacted earth fill is removed, about $1.4\pm$ m of grade raise would likely result in a settlement of the order of 40 mm due to the settlement of foundation soils and of the existing embankment. About one-third of this settlement would take place within one month, with the majority of the remaining within the next ten years.

In addition, the settlement of the embankment fill under their own weight can be expected to occur. For a properly constructed, about $1.4\pm$ m additional high embankment on the top of the existing about 3 m high embankment, this should not exceed 15 mm. The time rate will depend on the material used for construction. This settlement will largely occur during construction for granular materials. However, if SSM or granular soils are used, about half of this settlement should be completed within two months and the remaining half substantially completed within one year. This will bring the total anticipated settlements to 55 mm (i.e. 40 mm + 15 mm). It is our opinion that with these settlements neither surcharging nor preloading is required. However, it is recommended that additional fill to raise the approach embankments be placed at the site, at least six weeks prior the placement of granular pavement fill. It is recommended that asphalt paving of the road be delayed as long as the construction schedule permits.

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

In addition, the forward slopes should be protected from erosion and scour.

5.5 Construction Comments

We understand that a staged construction is proposed utilizing single lane closures requiring roadway protection. With this approach, shoring will be required and no change in the existing side slopes is anticipated for staging.

No major problems are anticipated during foundation excavations as the soils are generally cohesive and amount of water entering into the excavation can be controlled by gravity drainage and conventional pumping.

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

SP 105 S19 – Protection Systems

SP 902 S01 – Excavation and Backfilling to Structures

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

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

Silty clay to clayey silt till (stiff): Type 3 soil

Silty clay to clayey silt till (very stiff to hard): Type 2 soil

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

Temporary shoring will be required to support the excavations. In Ontario, shoring typically consists of soldier pile and timber lagging or sheet piling (with or without bracing / rakers). The shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this case, the required performance level is considered 2. The shoring system should be designed by a Professional Engineer, experienced in this type of work.

The coefficient of lateral earth pressures given in Table 5.5.1 can be used for the design of the temporary shoring system, based on the borehole results.

Table 5.5.1: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m ³)
Granular Fill	0.33	0.50	3.0	21.5
Silty Clay (Embankment Fill)	0.40	0.60	-	18.0
Silty clay to clayey silt till	0.36	0.53	2.8	21.0

We recommend that an NSSP be issued specifying that shoring piles will be cut-off approximately 1.2 m below grade.

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

Materials that may impede the driving of the piles should not be used in the affected areas. It is also recommended the vibrations be monitored during the driving of the piles to ensure that the foundations of the existing bridge (half) are not adversely affected. Special provision for vibration monitoring is given in the Appendix G. An NSSP should be issued in this respect.

5.6 Frost Protection

Design frost protection depth for the general area is 1.2 m. Therefore, a permanent soil cover of 1.2 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 Limitation of Report, as quoted in Appendix H, are an integral part of this report.

For and on behalf of Coffey Geotechnics Inc.



Hafiz Muneeb Ahmad, M.Eng., P.Eng.



Ramon Miranda, P.Eng.



Zuhtu Ozden, P.Eng.



Appendix F

Summary of Foundation Options

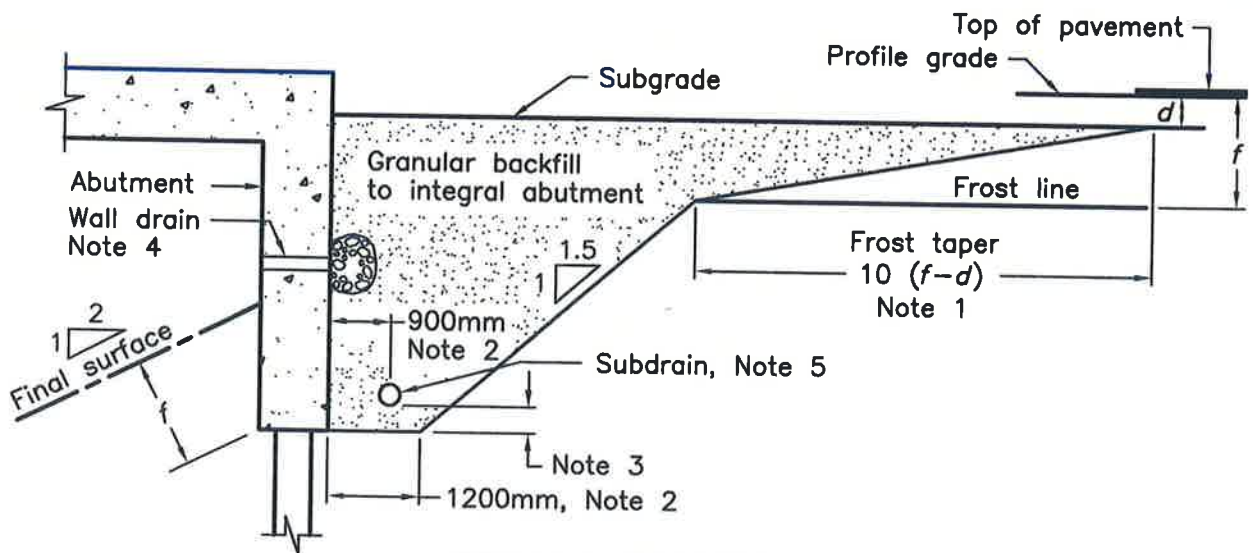
Table F1: Summary of Foundation Options

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Spread Footings	Cost effective. Low geotechnical resistance (may not be feasible to carry imposed loads). Not suitable for integral abutment design	Rather large excavations adjacent to the existing footings of the one half of the existing bridge which will be retained for staged construction will be required; may be risky, if footing elevations do not sufficiently match; extensive settlements will likely occur	Moderate	Unlikely to be suitable for this project. Not recommended
Spread Footings on Compacted Granular 'A' pad	Cost effective	Not feasible	Low to moderate	Not recommended based on impracticability of construction, adjacent to the existing bridge foundations
Timber Piles	Cost effective. Commercial length is about 15 m. After driving about 15 m, the clay deposit is very stiff in consistency may not provide sufficient support for a bridge structure	Considered unreliable under a highway bridge (may have limited useful life span). May not drive to full 15 m depth without being damaged	Low to moderate	Not recommended based on reliability factor
Drilled concrete caissons extending into bedrock	Reliable but expensive and not suitable for integral abutments	Reliable	High	Not recommended based on cost and unsuitability for integral abutment design

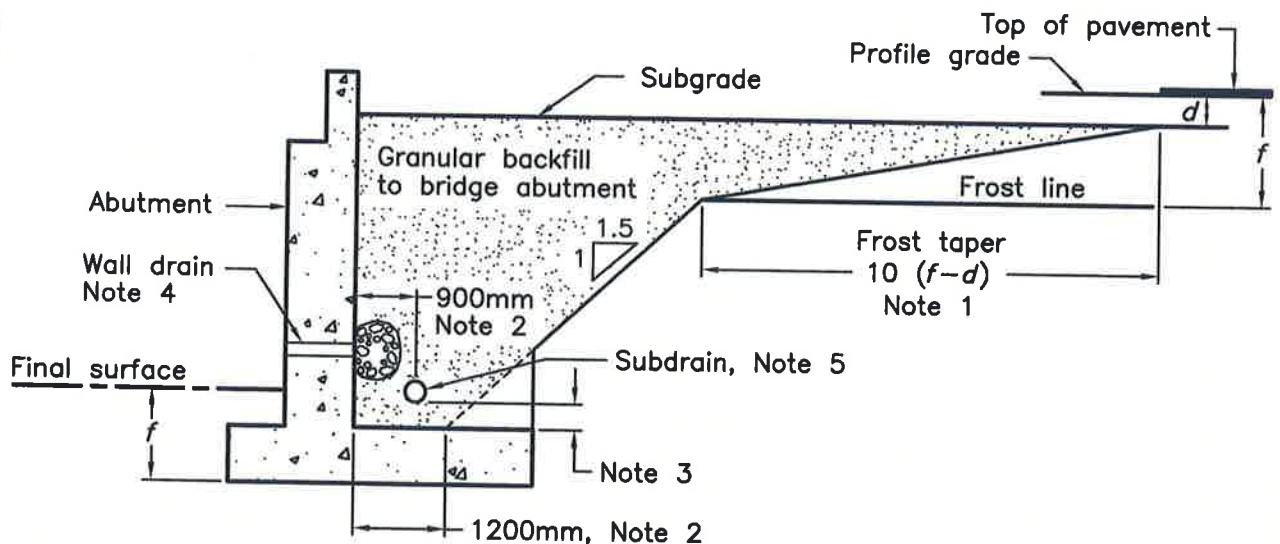
Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Steel Tube Piles	Not suitable for integral abutment design. Being relatively high displacement piles as compared with steel H-piles, they are less suitable for this project	Reliable but not as reliable as steel H-piles for this project.	Moderate to high	Not a preferred option based on reliability in comparison with steel H-piles. As well not suitable for integral abutment design
Steel H-Piles	Suitable for integral abutment design	Reliable	Moderate to high	Preferred option based on a combination of reliability and cost factors, as well as suitability for integral abutment design

Appendix G

OPSD & SP



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.

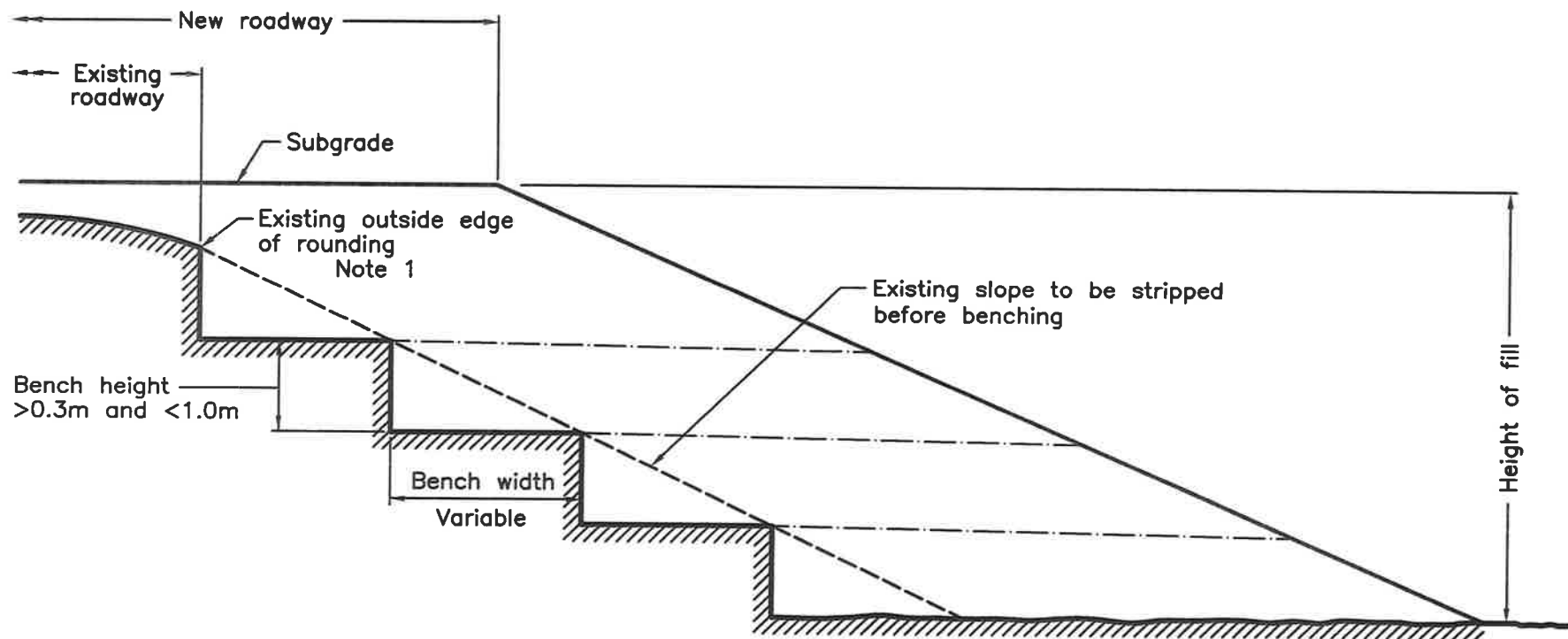
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 Rev 0

**WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT**



OPSD - 3101.150



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

CSP FOR INTEGRAL ABUTMENT - Item No.

Special Provision

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.	± 25 mm
Maximum deviation from specified spacing between inner and outer CSP's.	± 25 mm
Maximum deviation of any point on the top perimeter of the CSP's from the specified Elevation.	± 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.

903.07.03.07 Vibration Monitoring

The vibration monitoring equipment shall be placed on the existing structure such that it will not be disturbed. The location should be as close as possible to the piling works.

The vibrations at the existing structure shall not exceed 100 mm/s (peak particle velocity).

The Contractor shall take readings on the first pile in each pile group (i.e. at each corner of the abutment), starting with the pile furthest away from the existing structure. As a minimum, the readings should be taken and recorded during the first 3 m of driving and during seating of the pile onto the bedrock.

The results shall be certified by the Quality Verification Engineer as being accurate and meeting the requirements of the specification. The results shall be submitted to the Contract Administrator prior to continuing with the remaining piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the results are acceptable, the Contractor may continue with the remaining piles with readings taken during driving of each pile. Subsequent vibration readings should be taken for each pile during bedrock seating. The results of the subsequent piles should be certified by the Quality Verification Engineer as being accurate and meeting the requirements of the specifications. The results shall be submitted to the Contract Administrator at the end of each day.

If the readings are not within the limits stated above, the Contractor must alter his driving procedures until the vibrations on the existing structure are within acceptable levels. The above process must be repeated for each pile.

RETAINED SOIL SYSTEM, TRUE ABUTMENT - Item No.
RETAINED SOIL SYSTEM, FALSE ABUTMENT - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, HIGH PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, MEDIUM PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, LOW PERFORMANCE - Item No.
RETAINED SOIL SYSTEM WITH FINISHING CAP, WALL/SLOPE, HIGH PERFORMANCE -
Item No.
RETAINED SOIL SYSTEM WITH FINISHING CAP, WALL/SLOPE, MEDIUM
PERFORMANCE - Item No.
RETAINED SOIL SYSTEM WITH TRAFFIC BARRIER, WALL/SLOPE, HIGH
PERFORMANCE - Item No.
RETAINED SOIL SYSTEM WITH TRAFFIC BARRIER, WALL/SLOPE, MEDIUM
PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, ROADBASE EMBANKMENT - Item No.

Special Provision No. 599S22

March 2001

1.0 SCOPE

This special provision covers the requirements for the design, supply and construction of Retained Soil Systems (RSS).

Additional requirements for RSS precast concrete facing elements shall be as specified elsewhere in the Contract.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, General:

OPSS 180 Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Construction:

OPSS 501 Compaction
OPSS 539 Protection Schemes

Ministry of Transportation Publications

MTO Designated Sources of Materials (DSM)
Generic Requirements for Retained Soil Systems for DSM
Ontario Highway Bridge Design Code 1991 - 3rd Edition (OHBDC)

3.0 DEFINITIONS

For the purposes of this special provision the following definitions apply:

Approved Product Drawings: means the documentation for an RSS which has been submitted to the Ministry by the Manufacturer for approval and listing in the DSM, in accordance with the Generic Requirements for Retained Soil Systems for DSM.

Associated Backfill: means all backfill other than engineered backfill necessary to construct the RSS, and to reinstate the excavation for the RSS.

Design Engineer: means the Engineer who produces the working drawings; the Design Engineer shall be certified by the Manufacturer as having the appropriate experience and expertise to provide design services for the Manufacturer's RSS.

Design Check Engineer: means the Engineer who checks the original design; the Design Check Engineer shall be certified by the Manufacturer as having the appropriate experience and expertise to provide design services for the Manufacturer's RSS.

Engineered Backfill: means all backfill that is part of the engineered materials comprising the RSS and/or the RSS foundation.

External Stability: means the stability of the foundation and slope/embankment on which the RSS relies for support during and after construction.

Internal Stability: means the stability of the engineered materials comprising the RSS.

Manufacturer: means the party who supplies and/or specifies the design, materials and components for the RSS selected by the Contractor.

Quality Verification Engineer: means an Engineer recognized by the Manufacturer as having demonstrated experience and expertise to provide quality verification services for the Manufacturer's RSS. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and to issue Certificates of Conformance.

Retained Soil System (RSS): means a proprietary system which uses mechanical soil stabilization to retain horizontal loads in excess of 2 m in height for applications such as true and false abutment structures, retaining walls and steep slopes; or, to retain vertical loads for applications such as embankments over soft ground.

Stamped: means working drawings that have been reviewed and stamped "Conforms with Contract Documents". The stamp shall include the date and signature of the Quality Verification Engineer

4.0 SUBMISSION AND DESIGN REQUIREMENTS

4.1 Submissions

4.1.1 Working Drawings

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

The Contractor shall submit working drawings for the design, fabrication and construction of the RSS to the QVE for review and stamping.

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

At least two weeks prior to commencement of construction of the RSS, the Contractor shall submit to the Contract Administrator three (3) sets of the stamped working drawings. The Contract Administrator will forward one set of the stamped working drawings to the Pavement and Foundation Section, Ministry of Transportation, Downsview, for information purposes.

4.1.2 Working Drawing Requirements

Working drawings shall include at least the following:

All design, fabrication and construction drawings and specifications for the RSS;
Details of all excavation, unwatering, drainage and backfilling required to construct the RSS, including type and source of associated backfill;
Details at joints and connections to other structures where shown in the Contract Drawings
Details of all protection systems;
Statement of bearing resistance required by the RSS foundation, and the bearing resistance provided in accordance with the OHBDC;
Statement of satisfactory internal and external stability;
All design, fabrication and construction drawings and specifications for traffic barriers and base, and finishing caps, where applicable;
Details of how all relevant Operational Constraints and Environmental Constraints, as specified elsewhere in the Contract, will be adhered to.
A copy of the Approved Product Drawings covering material and construction details

4.1.3 Certificate of Conformance

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of each of the following operations and prior to commencement of each subsequent operation:

- foundation base preparation
- on-site delivery of manufactured and fabricated components
- alignment of RSS as per contract documents
- backfill material

The Certificates of Conformance shall state that the materials and work have been supplied and installed in general conformance with the stamped working drawings and Contract documents.

Upon completion of the RSS installation, the Contractor shall submit to the Contract Administrator a final Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the RSS has been constructed in general conformance with the stamped working drawings and Contract documents.

4.1.4 Warranty

The Contractor shall submit an unconditional warranty to the Owner, to implement all repair and maintenance requirements to the RSS related to design, materials and workmanship for a period of three (3) years from the date of certification of completion of the Contract.

4.2 Design

4.2.1 General

The Contractor shall verify the existing site conditions and ground elevations before preparing the working drawings, and notify the Contract Administrator immediately if site conditions differ from those described in the Contract.

The Application, Performance, and Appearance requirements for the RSS shall be as specified elsewhere in the Contract.

The geometric requirements of the RSS, including alignment and profiles, typical cross-sections, and location of traffic barriers and/or finishing caps, as well as other constraints influencing the design of the RSS, shall be as specified elsewhere in the Contract.

4.2.2 RSS Selection

The Contractor may select any RSS designated as A (Accepted) or as DE (Demonstration) on the DSM List that meets the specified Contract requirements. RSS qualified as DE (Demonstration) status will require inspection, instrumentation, monitoring and reporting by the Manufacturer, in accordance with the Generic Requirements for Retained Soil Systems for DSM.

The RSS selected and designed by the Contractor shall meet all of the requirements for the RSS specified in the Contract.

4.2.3 Foundation Investigation Report

A Foundation Investigation Report that describes the subsurface conditions at the RSS is available, as specified elsewhere in the Contract. The Owner warrants that the information provided in the Foundation Investigation Report can be relied upon with the following limitations and exceptions:

Any interpretations of data or opinions expressed in the report are not warranted; and

Although the raw measured data presented is warranted, the Contractor must satisfy himself as to sufficiency of the information presented and obtain any updating or additional information, and perform any studies, analyses or investigations the Contractor deems necessary in order to prepare his design, at no additional cost to the Owner.

4.2.4 Protection Systems

Where the stability, safety or function of an existing roadway, railway, and other works can be impaired by an excavation or temporary slope, the Contractor shall provide protection systems as required, including sheet-piling, shoring, and the driving of piles where necessary, to prevent damage to such works.

Design of protection systems shall be in accordance with SP 539S01.

5.0 MATERIALS

5.1 General

All materials for the selected RSS shall conform to Approved Product Drawings for that RSS.

5.2 Associated Backfill

Associated backfill shall be suitable for the particular application, and be approved by the Design Engineer as compatible with the RSS.

7.0 CONSTRUCTION

7.1 General

The work shall include the construction of the RSS, with traffic barriers and finishing caps where specified, and all excavation, unwatering, drainage and backfilling required to construct the RSS.

Associated backfill shall be compacted in accordance with OPSS 501.

7.2 RSS

The RSS shall be constructed in conformance with the stamped working drawings.

7.3 Protection Systems

Protection systems shall be constructed in accordance with the stamped working drawings.

Protection systems shall be removed in accordance with SP 539S01.

7.4 Management of Excess Materials

Excess materials resulting from carrying out the work shall be removed and managed as specified elsewhere in the Contract.

8.0 QUALITY ASSURANCE

The Contractor shall submit representative samples of the RSS components to the Contract Administrator when requested.

10.0 BASIS OF PAYMENT

Payment at the contract price for the above tender item(s) shall be full compensation for all labour, equipment and material to do the work.

RETAINED SOIL SYSTEM, TRUE ABUTMENT - Item No.
RETAINED SOIL SYSTEM, FALSE ABUTMENT - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, HIGH PERFORMANCE - Item No.
RETAINED SOIL SYSTEM, WALL/SLOPE, MEDIUM PERFORMANCE - Item No.
RETAINED SOIL SYSTEM WITH FINISHING CAP, WALL/SLOPE, HIGH PERFORMANCE -
Item No.
RETAINED SOIL SYSTEM WITH FINISHING CAP, WALL/SLOPE, MEDIUM PERFORMANCE -
Item No.
RETAINED SOIL SYSTEM WITH TRAFFIC BARRIER, WALL/SLOPE, HIGH PERFORMANCE -
Item No.
RETAINED SOIL SYSTEM WITH TRAFFIC BARRIER, WALL/SLOPE, MEDIUM
PERFORMANCE - Item No.

Special Provision No. 599S23

March 2006

1.0 SCOPE

This special provision covers the requirements for materials, quality control and quality assurance testing and acceptance criteria for precast concrete facing elements including panels.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Material:

OPSS 1002, Material Specification for Aggregates - Concrete
OPSS 1350, Material Specification for Concrete - Materials and Production

Ministry of Transportation Publications

MTO Laboratory Testing Manual: Tests

LS- 412, Method of Test for Scaling Resistance of Concrete Surfaces Exposed to De-icing Chemicals

Canadian Standards Association

CSA A 23.1- Concrete Materials and Methods of Concrete Construction
CSA A23.2-3C, Making and Curing Concrete Compression and Flexure Test Specimens
CSA A23.2-4C, Air Content of Plastic Concrete by the Pressure Method
CSA A23.2-5C, Slump of Concrete
CSA A23.2-9C, Compressive Strength of Cylindrical Concrete Specimens
CSA A23.2-14C Obtaining and Testing Drilled Cores for Compressive Strength Testing

American Society of Testing Materials

ASTM C457, Microscopical Determination of Parameters of the Air Void System in Hardened Concrete

3.0 DEFINITIONS – Not Used

4.0 SUBMISSION REQUIREMENTS

For precast concrete facing elements, the following information shall be submitted to the Contract Administrator at least four weeks prior to the use of the precast concrete facing elements:

- concrete mix design,
- test data on aggregates documenting conformance with OPSS 1002.
- Manufacturer's production quality control data on compressive strength and air void—system parameters, less than 6 months old
- manufacturer's production quality control data on salt scaling resistance less than 12 months old.

Testing shall be carried out in conformance with test methods specified in this Special Provision.

If, due to the product's physical characteristics, the product cannot be tested for scaling resistance in conformance with MTO Laboratory Testing Manual, LS- 412, Method of Test for Scaling Resistance of Concrete Surfaces Exposed to De-icing Chemicals, the Owner will request alternative testing.

5.0 MATERIALS

Concrete shall conform to OPSS 1350 except for the following:

Air void system parameters shall be a minimum of 3% air content and the average spacing factor obtained on a minimum of two cores per structure shall be no more than 0.200mm maximum with no individual test result greater than 0.230 mm.

Concrete shall conform to requirements for scaling resistance. The average maximum scaling mass loss shall be 0.8 kg/m².

6.0 QUALITY CONTROL AND QUALITY ASSURANCE

The quality control and quality assurance results will be used for determining acceptance of the product supplied to this contract.

6.01 Quality Control of Precast Concrete Facing Units

Copies of all quality control tests required shall be provided to the Contract Administrator as soon as they are available unless otherwise specified in this Special Provision.

The Contractor shall submit quality control test data on concrete air content, slump, temperature, compressive strength, air void system parameters analysis and cover over reinforcing steel.

Testing for air content, slump, temperature and compressive strength shall be carried out per each 30 m³ of concrete produced or per each day of production whichever is more frequent. For acceptance purposes, the Contractor shall test a minimum of three sets of 150 x300 mm compressive strength cylinders each representing different batches of concrete, at a laboratory acceptable to the Owner. Each set shall consist of two cylinders. The cylinders made for acceptance purposes shall be made and cured in conformance with CSA A23.2-3C under standard moisture and temperature conditions and tested in conformance with CSA A23.2-9C. The cylinders shall be made by a concrete field testing technician certified by the Canadian Standard Association or by the American Concrete Institute. This person shall have successfully completed, as part of the certification requirement, written and practical examinations within the last five years verifying his/her competence to carry out field testing of concrete, and have in his/her possession, at all times testing is to be performed, a card issued by the certifying agency verifying the currency of the individual's certification.

Air void system parameters analysis shall be carried out by the Contractor on a minimum of two 100 mm diameter cores per structure removed from precast concrete facing elements at locations determined by the Contract Administrator. Individual cores shall be taken from different panels. Cores shall not contain embedded steel. For air void system parameters testing, the Contractor shall use a laboratory that is on the Ministry's list of approved laboratories and operators for this testing. The cores shall be cut lengthwise into two halves with one half to be tested by the Contractor and the other forwarded to the Ministry (Manager, Concrete Section, Room 15, Building C, 1201 Wilson Avenue, Downsview, Ontario, M3M 1J8). Air void analysis results shall be submitted to the Contract Administrator within 21 days of delivery of the precast elements to the job site.

Concrete cover measurements shall be carried out by the Contractor on reinforced concrete facing elements supplied to the contract, before they are installed. A minimum of 30 measurements per structure shall be carried out. Measurements shall be carried out at locations and on precast elements randomly selected by the Contract Administrator. Cover measurement shall be carried out with a covermeter or a method acceptable to the Contract Administrator. The depth of cover, to the nearest millimetre, shall be determined to the outermost reinforcing steel. Concrete cover measurement results shall be submitted to the Contract Administrator at least 2 days prior to the installation of the reinforced concrete facing elements represented by the test results.

Testing shall be carried out in conformance with the following:

- Slump: CSA A23.2-5C, Slump of Concrete
- Air Content: CSA A23.2-4C, Air Content of Plastic Concrete by the Pressure Method
- Compressive Strength: CSA A23.2-9C, Compressive Strength of Cylindrical Concrete Specimens and CSA A23.2-3C, Making and Curing Concrete Compression and Flexure Test Specimens
- Obtaining Cores: CSA A23.2-14C Obtaining and Testing Drilled Cores for Compressive Strength Testing
- Air Void System Parameters: ASTM C457, Microscopical Determination of Parameters of the Air Void System in Hardened Concrete

6.02 Quality Assurance

The Contractor shall obtain two 300 mm x 300 mm specimens per structure for testing of scaling resistance by the Ministry of Transportation. The specimens shall be obtained from finished precast concrete facing elements randomly selected by the Contract Administrator. The Contractor shall deliver the samples to the Ministry (Manager, Concrete Section, Room 15, Building C, 1201 Wilson Avenue, Downsview, Ontario, M3M 1J8). Testing for scaling resistance will be carried out in conformance with MTO Laboratory Testing Manual, LS- 412, Method of Test for Scaling Resistance of Concrete Surfaces Exposed to De-icing Chemicals.

6.03 Acceptance of Precast Concrete Facing Elements

The acceptance of precast concrete facing elements will be based on quality control test results obtained by the Contractor and on salt scaling results obtained by the Owner.

Acceptability of air void system parameters will be based on individual core results for air content, and on the average result from two cores per structure for spacing factor. Precast concrete facing elements on a structure represented by a pair of cores which fails to meet the requirements for air void system parameters will be considered unacceptable.

Acceptability of concrete compressive strength will be based on the following:

- the average of all sets of compressive strength tests shall be equal to or greater than the specified strength
- no individual strength test shall be more than 15% below the specified strength.

When the compressive strength specimens fail to meet these requirements, the precast concrete facing panels supplied to the contract will be considered unacceptable.

Acceptability of concrete cover over reinforcing steel will be based on the percentage of satisfactory measurements. The concrete cover over reinforcing steel shall be within ± 10 mm of the design concrete cover. When 10 % or more of the total number of measurements per structure is outside the specified limits, the panels in a structure represented by these measurements will be considered unacceptable.

Acceptability of salt scaling resistance will be based on average of results obtained on two 300 mm x 300 mm specimens representing a structure. When the specimens fail to meet the requirements for salt scaling resistance the precast concrete facing units represented by the specimens will be considered unacceptable.

Unacceptable concrete facing elements shall be removed and replaced at the Contractor's expense. The Contractor may elect to submit a proposal for remedial action to the Contract Administrator.

SUGGESTED NSSP WORDING

All pile driving should be carried out in accordance with SP 903S01. The contract document should also contain a NSSP containing the following wording:

Cobbles and Boulders

"The Contractor is informed that the soils at this site may contain cobbles and boulders and large rock fragments/slabs immediately above the bedrock that may impede the progress of driving piles. The soil conditions are described in the Foundation Investigation Report prepared for this site. Reference should be made to this report for a description of the soil conditions. The factual data presented on the Record of Borehole Sheets governs any interpretation of the site conditions."

Shale Bedrock

"The Contractor is informed that shale bedrock with occasional limestone layers/seams will be encountered at this site. The rock conditions are described in the Foundation Investigation Report prepared for this site. Reference should be made to this report for a description of the rock conditions. Care must be taken to avoid overdriving and damaging the pile tip."

Appendix H

Limitations of Report

LIMITATIONS OF REPORT

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

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

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

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

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