

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
SHEWFELT BRIDGE RESTORATION,
GOULAIS BAY ROAD, 3 KM WEST OF
HIGHWAY 17, DISTRICT OF ALGOMA,
ONTARIO, W.P. 5139-10-01, SITE 38S-031**

LEA Consulting Limited

Project: TRANETOB01156AB
October 12, 2010

October 12, 2010

LEA Consulting Limited
625 Cochrane Drive, Suite 900
Markham, Ontario
L3R 9R9

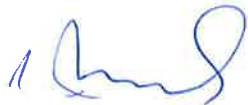
Attention: Peter Ojala, P. Eng.

Dear Sirs:

RE: Foundation Investigation and Design Reports, Shewfelt Bridge Restoration, Goulais Bay Road, 3 Km West of Highway 17, District of Algoma, Ontario, W.P. 5139-10-01, Site 38S-031

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

For and on behalf of Coffey Geotechnics Inc.



Ramon Miranda, P. Eng.
Manager, Transportation Division

**FOUNDATION INVESTIGATION REPORT
SHEWFELT BRIDGE RESTORATION,
GOULAIS BAY ROAD, 3 KM WEST OF
HIGHWAY 17, DISTRICT OF ALGOMA,
ONTARIO, W.P. 5139-10-01, SITE 38S-031**

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**FOUNDATION INVESTIGATION REPORT
SHEWFELT BRIDGE RESTORATION, GOULAIS BAY ROAD
3 KM WEST OF HIGHWAY 17, DISTRICT OF ALGOMA ONTARIO
W.P. 5139-10-01, SITE 38S-031**

1 INTRODUCTION

Coffey Geotechnics Inc. (Coffey) was retained by LEA Consulting Limited (LEA) to carry out a foundation investigation at the site of the proposed rehabilitation of Shewfelt Bridge over the Goulais River on Goulais Bay Road between Highway 552 and Pine Shores Road, in the Township of Fenwick, approximately 3 km west of Highway 17. The site is located within the District of Algoma and has MTO Site Number 38S-031.

The existing Shewfelt Bridge is a four-span bridge, with a total length of 84.7 m and contains a two-span single lane Bailey bridge (63.1 m) with a timber deck, and two steel girder end spans with concrete deck and timber deck at the west and east spans, respectively, each 10.8 m in length (see site photographs in Appendix C). It is understood that the performance of the existing bridge is affected by the problems of bridge foundation settlement and rotation, slope stability, active erosion and riverbank slumping (upstream of the existing bridge).

Previously, Shaheen & Peaker Limited (now known as Coffey Geotechnics Inc.) was retained by LEA Consulting Limited (LEA) to carry out an advance foundation investigation in 2006. The findings of that investigation were presented in our report entitled "Advance Foundation Investigation, Shewfelt Bridge Replacement, Goulais Bay Road, 3 km west of Highway 17, District of Algoma, Ontario, G.W.P. 5290-04-00, Site 38S-031," dated May 24, 2006, Project No. SPT1156A.

In 2008, the locations of a new bridge and approaches (some 125 m away from the existing bridge location) were finalized and Coffey was retained by LEA to carry out a detail investigation for the detail design of the bridge and approach roads. The findings of that investigation were presented in our report entitled "Foundation Investigation and Design Reports, Shewfelt Bridge Replacement, Goulais Bay Road, 3 km west of Highway 17, District of Algoma, Ontario, G.W.P. 5290-04-00, Site 38S-031, GEOCRES NO. 41K-82" dated December 09, 2009, Project No. TRANETOB01156AA.

Since then the rehabilitation of the existing bridge (replacement of the existing bridge at the same location) was proposed by Ministry of Transportation Ontario (MTO) and Coffey was retained by LEA to conduct a foundation investigation.

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

The findings of the investigation are presented in this report.

2 SITE DESCRIPTION AND GEOLOGY

The Goulais River is located in a deep and wide valley (the Goulais River Valley) north of Sault Ste. Marie. In the general vicinity of the project site, the area is referred to as the Goulais River Beach Ridges, which

are described as ancient beach ridges of an alluvial plain. The river meanders on its way toward Lake Superior and numerous oxbow lagoons are evident.

The Goulais River in the vicinity of the project site has steep banks, with bank failures having occurred in many areas. It is evident that the Goulais River is continuing to undercut its banks at turns in the river, resulting in slope failures and re-alignment of the river channel. It is noted that a section of the existing Old Goulais Bay Road located to the north of the existing bridge near the west bank of the river is in proximity to such a bend in the present river channel.

Based on available information, the Goulais River Valley was probably cut by a major pre-glacial river. At the time of the retreat of the last glaciations, a river flowed in the Goulais Valley carrying glacial materials into Glacial Lake Algonquin, resulting in deep glacial deposits. As well, it appears that deep clays were deposited and followed by sands and silts deposited by the present river itself.

3 INVESTIGATION PROCEDURES

The fieldwork at the existing Shewfelt Bridge location was performed during the period of July 6, 2010 through July 15, 2010. As requested, the fieldwork consisted of drilling and sampling two boreholes (Boreholes 201 and 202) for the bridge pier rehabilitation. The plan location of the boreholes is shown in 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	Drilling Depth Below the Goulais River* (m)	Dynamic Cone Penetration Tests	Piezometer
BH 201	Central pier area	39.3	Yes (from the borehole bottom)	No
BH 202	East pier area	37.8	Yes (from the borehole bottom)	No

*below the water level in the River

Walker Drilling of Utopia, Ontario carried out the drilling, testing and sampling work, under the direction and supervision of technical personnel (Mr. Gwangha Roh) from Coffey. Boreholes were located in the river and these boreholes were therefore drilled from a raft. The boreholes were advanced by wash boring methods, using N casing. A pressure gauge was installed at the connection of the water supply hose to the casing swivel to enable to monitor the water pressure inside the casing (in anticipation of the expected artesian condition which would increase the water pressure in this closed system) could be monitored during the drilling. Drilling mud was also utilized to counter-balance the hydrostatic uplift condition below the major clay deposit.

Samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. This test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS – split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value

of the soil which is indicative of the compactness condition of cohesionless granular soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

In cohesive (clayey) deposits, where the consistency of the soil permitted, relatively undisturbed samples (TW) were taken with 50 mm diameter thin-walled (Shelby) tubes which were manually pushed into the bottom of the borehole. Where the consistency of the clay permitted, field vane tests were performed to measure in-situ undrained shear strength of clayey soil.

Dynamic Cone Penetration tests were performed at the end of the boreholes, to refusal. In Dynamic Cone Penetration Test (DCPT), a 51 mm diameter, 60 deg. apex cone point, screw-attached to the tip of A-size rods, is driven into the ground using the same driving energy as in the SPT method. By recording the number of blows to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of relative density/consistency is obtained. Although the interpretation of the test results is difficult because no samples can be obtained by the DCPT method and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination/reduction of unbalanced hydrostatic effects which in many cases affect the SPT values, especially when fine-grained granular soils or cobbles/boulders are encountered.

Groundwater conditions in the boreholes were observed during the investigation and upon completion. The boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

The borehole locations were established by our field supervisor in relation to the existing bridge structure. Based on the provided survey information (done by MTO), the Geodetic river water surface elevations at the borehole locations were measured daily by our field supervisor.

The soil samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural water content determinations, grain size analyses, Atterberg Limits tests, consolidation tests and unit weight determinations was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

4 SUBSURFACE CONDITIONS

The sub-surface conditions were explored at two boreholes (see Table 3.1 in Section 3) during the current investigation.

The plan locations of the boreholes are shown in Drawing No. 1. Details of sub-surface conditions encountered at each borehole location for the current 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. A stratigraphic profile at bridge section is shown on Drawing No. 1. Detailed laboratory test results are enclosed in Appendix B.

In general, the sub-surface stratigraphy comprises very loose to loose cohesionless sand at the river bed (only encountered at Borehole 202) which is in turn underlain by a 29 to 32 m thick deposit of silty clay to clay. The upper and lower 1 to 2 meters of the silty clay deposit was found to have more silt content. The silty clay in Borehole 201 is further underlain by a sand deposit, followed by a sand and gravel deposit, while the silty clay to clay in Borehole 202 is further underlain by a clayey silt deposit, followed by a sand

and gravel deposit. Both boreholes were terminated within the sand and gravel deposit after 0.6 to 3.2 m penetration into the sand and gravel deposit due to the observed severe artesian condition.

Reference can also be made to the previous investigation results, as detailed in Section 1, for a broader understanding of the conditions throughout the general area.

4.1 Upper Gravelly Sand

Borehole 202, drilled adjacent to the existing bridge east pier, shows the presence of a gravelly sand deposit below the river bed. This non-cohesive (granular) deposit was found to extend to a depth of 1.5 m below the river bed or to El. 179.6 m.

The grain-size distribution of a combined sample from the deposit is given in Figure B-1, in Appendix B. The analyses results indicate the following grain-size distribution.

Gravel:	22 %
Sand:	74 %
Silt & Clay:	4 %

N-values recorded in this deposit are 3 and 9 blows/0.3 m which indicate a very loose to loose condition.

4.2 Silty Clay

Underlying the gravelly sand at the depth of 1.5 m below the river bottom in Borehole 202 and from the river bed in Borehole 201, the boreholes contacted a massive cohesive deposit. The deposit was found to extend to depths of 32.0 and 30.5 m below the river bed (or to El. 148.0 m and 150.6 m) in Boreholes 201 and 202, respectively. The deposit consists of a reddish grey to reddish silty clay to clay with occasional to frequent silt and clayey silt seams. The upper portion of the deposit (typically about upper 1 to 2 m) was found to have relatively more silt content and thus the upper zone typically resembles a mixture of clay and silt material. At depths of 20 to 23 m into the deposit, more frequent silt and clayey silt seams were observed with increasing depth. In Borehole 202, an about 2 m thick lower zone in this deposit was found to have more silt content and was classified as a clayey silt deposit.

The grain-size distribution of ten samples from the deposit is given in Figure B-2. The results of the tests show the following grain-size distribution:

Gravel:	0 %
Sand:	0-2 %
Silt:	18-48 % (average 30 %)
Clay:	52-82 % (average 70 %)

When analysing these grain-size results, it should be kept in mind that the samples tested were a mixture of several or more individual interbeds.

The Atterberg limits tests performed on thirteen samples is given in Figures B-3 in Appendix B. These tests yielded the following index values:

Liquid Limit:	31-69 % (Average 53%)
Plastic Limit:	19-33 % (Average 26%)
Plasticity Index:	11-37 (Average 27)
Natural water content	35-69 % (Average 55%)

These results indicate clayey soils of low to high but typically high plasticity. The lower plasticity index values are generally from the upper or lower more silty zones. There is also some variation within each zone, where annual deposition shows a range from more plastic (i.e. fatter) to relatively less plastic (i.e. leaner) clay content, akin to a varved clay. As shown on the individual Record of Borehole Sheets, the measured natural water contents are generally near or in excess of the measured liquid limits which indicate the likelihood of a normally consolidated soil deposit, or only a slight pre-consolidation.

Standard Penetration tests conducted in the silty clay deposit gave N-values which typically range from 0 to 12 blows/0.3 m which indicate a very soft to stiff consistency. The higher N-values were typically recorded at deeper depths which contain more frequent clayey silt and silt seams.

The undrained in-situ shear strengths of the deposit were measured in the field by means of field vane tests. The measured values range from 48 to in excess of 200 kPa, indicating a firm to hard consistency. Again the higher undrained shear strengths were measured in the lower zones of the deposit, as shown in Figure D1 (Appendix D) where the variation of the measured in-situ vane strength values (i.e. in-situ undrained shear strengths) versus elevation is presented. The figure indicates a typical undrained shear strength of 50 kPa near the top of the deposit, gradually increasing to about 100 kPa at about El. 164 m (or about 16 m into the deposit). Below this elevation, the measured undrained shear strength typically range from 100 to 200 kPa. Also plotted on the same figures are the effective overburden stress (P'_o), as well as the plot of $0.23 P'_o$ with elevation. It is commonly acknowledged that with Ontario clays if the measured undrained shear strengths are in excess of $0.23 P'_o$ line, the deposit may be somewhat over-consolidated, perhaps due to removal of previously existing overburden.

A total of two oedometer (one dimensional consolidation) tests were performed in the laboratory on 50 mm diameter Shelby tube (TW) samples (one from Borehole 201 and one from Borehole 202). The results are presented in Figures B-4 and B-5, in Appendix B. These show a possible pre-consolidation pressure in excess of existing overburden pressure $P'_c - P'_o$ in order of 80 to 120 kPa. It should be pointed out that the presence of silty seams was noted in samples TW 3 (Borehole 201) and TW 5 (Borehole 202) and this is expected to have affected consolidation test results because the silty portions of the sample can be expected to be less compressible than the clay zones.

The measured bulk unit weight of the TW samples range from 16.2 to 17.9 kN/m³.

4.3 Clayey Silt

In Borehole 202 the bottom 2 m of the silty clay deposit appear to be of lower plasticity and was identified as clayey silt. This deposit was found to extend to 32.5 m below the river bed or to El. 148.6 m.

The grain-size distribution of a sample from the clayey silt gave the following results:

Gravel:	0 %
Sand:	2 %
Silt:	76 %
Clay:	22 %

as shown in Figure B-6 in Appendix B.

The Atterberg limits tests performed on the same sample is given in Figures B-7 in Appendix B. These tests yielded the following index values:

Liquid Limit:	27 %
Plastic Limit:	20 %
Plasticity Index:	7

These results indicate clayey soils of low plasticity (i.e. clayey silt).

Two Standard Penetration tests performed in this deposit yielded N-values of 14 and 25 blows/0.3 m which indicate a stiff to very stiff consistency.

4.4 Lower Sand

Underlying the silty clay deposit, Borehole 201 contacted a 3.5 m thick sand layer at a depth of 32.0 m below the river bed or at El. 148.0 m. This deposit was found to extend to 35.5 m below the river bed or to El. 144.5 m. The sand deposit contains some silt and gravel size particles including reddish sandstone fragments.

The grain-size distribution of a sample from the deposit gave the following results:

Gravel:	14 %
Sand:	68 %
Silt & Clay:	18 %

See Figure B-8 in Appendix B.

The material is classified as a granular (non-cohesive) soil type.

Two Standard Penetration tests conducted in the sand deposit gave N-values of 23 and 53 blows/0.3 m which indicate a compact to very dense relative density.

4.5 Sand and Gravel

Underlying the sand in Borehole 201 and the clayey silt in Borehole 202, a sand and gravel deposit was contacted at depths of 35.5 m and 32.5 m or at El. 144.5 m and 148.6 m in Boreholes 201 and 202,

respectively. Borehole 201 was terminated within this deposit at a depth of 36.1 m below the river bed or El. 143.9 m or after penetrating it by 0.6 m, due to an artesian condition which was observed 3 m above the river water level. Borehole 202 was also terminated within this deposit at a depth of 35.7 m below the river bed or El. 145.4 m or after penetrating it by 3.2 m, also due to an artesian condition which was observed 3.3 m above the river water level. This deposit was noted to contain reddish sandstone fragments.

The grain-size distribution of ten samples from the deposit is given in Figure B-9. The results of the analyses show the following grain-size distribution:

Gravel:	33-48 %
Sand:	41-48 %
Silt & Clay:	10-21 %

The deposit is considered to be a granular (non-cohesive) soil type. During drilling the presence of cobbles and boulders was inferred in the deposit.

Measured SPT N-values in this unit were 41 and 48 blows/0.3 m (Borehole 202) and 95 blows/0.3 m penetration (Borehole 201), indicating a dense to very dense relative density. The measured natural water content of soils sample were 10 to 14 %.

Dynamic Cone Penetration Tests (DCPT) were carried out at the bottom of Boreholes 201 and 202. The DCPT encountered refusal at a depth of 36.4 m below river bed in Borehole 201 (El. 143.6 m) and at a depth of 37.5 m below the river bed in Borehole 202 (El. 143.6 m).

4.6 Groundwater Conditions

Groundwater conditions were observed in the open boreholes while drilling and upon completion of each borehole. Groundwater level measurements during the drilling were recorded but not included in our Record of Borehole Sheets because groundwater level measurements may not be reliable due to wash boring method (i.e. water introduced into the boreholes).

While drilling, in Borehole 201, slight artesian conditions were observed at 10, 15, 16 and 19 m below the bottom of river or at El. 170, 165, 164 and 161 m. These were observed while drilling in the silty clay deposit and probably emanate from the silt and sand interbeds in the clay deposit. As such they are likely to be localized. When the drilling advanced to the sand and gravel deposit at about 36 m below the bottom of the river or to about El. 144 m, an aggressive artesian condition was noted to reach 6.2 m above the bottom of the river or to El. 186.2 m (i.e. about 3 m above water level in the river).

In Borehole 202, a slight artesian condition was observed at 4 m below the river bottom level or at El. 177 m followed by a 6 m artesian condition above the bottom of the river (or 4 m above the water level in the River) when the drilling was advanced to 10 m below the bottom of the river or to El. 171 m. This was followed by slight artesian conditions when the drilling advance to about 11 m (El. 170 m) and to 20 m (El. 161 m), in the silty clay deposit. In this borehole, when the borehole was advanced to 33 m below the bottom of the River (El. 148 m) an approximately 5 m artesian condition was noted (El. 186.2 m) in the sand and gravel deposit followed by a 5.4 m artesian condition (El. 186.5 m) when the drilling reached approximately 36 m or to about El. 145 m.

From these observations, it can be concluded that the lower granular deposits underlying the silty clay deposit exhibit high artesian conditions. Intermittent artesian conditions were also observed while drilling in the overlying clayey deposits, probably emanating from the localized silt and sand interbeds within the silty clay/clayey silt.

It should be pointed out that the water levels observed represent the conditions at the time of our investigation and that they would be subject to seasonal fluctuations as well as fluctuations due to major weather events and the water level in the Goulais River.

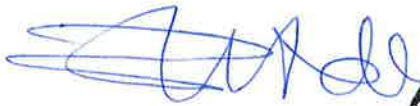
For and on behalf of Coffey Geotechnics Inc.



Gwangha Roh, Ph.D.



Ramon Miranda, P.Eng.



Zuhtu Ozden, P.Eng.



Drawing

WP: 5139-10-01



SHEET

**SHEWFELT BRIDGE REPLACEMENT
BOREHOLE LOCATION PLAN
AND SOIL STRATA**

coffey  **geotechnics**
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND



Borehole & Cone

1

Blows/0.3m (Std. Pen. Test, 475 J/blow)

Water Level at Time of Investigation
(W.L. Not Stabilized)

Hea



Artesian Water



Field Vane Shear

No.	WATER ELEVATION	STATION	OFFSET
201	183.2	10+340	8.3 m LT
202	183.2	10+311	6.4 m LT
No.	GROUND ELEVATION	NORTHING	EASTING
1*	183.3	5175908.7	275614.1
102*	187.7	5175838.2	275525.9

* Boreholes 1 and 102 were drilled about 140 m and 115m south from the existing bridge, respectively.

-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.

[illegible]

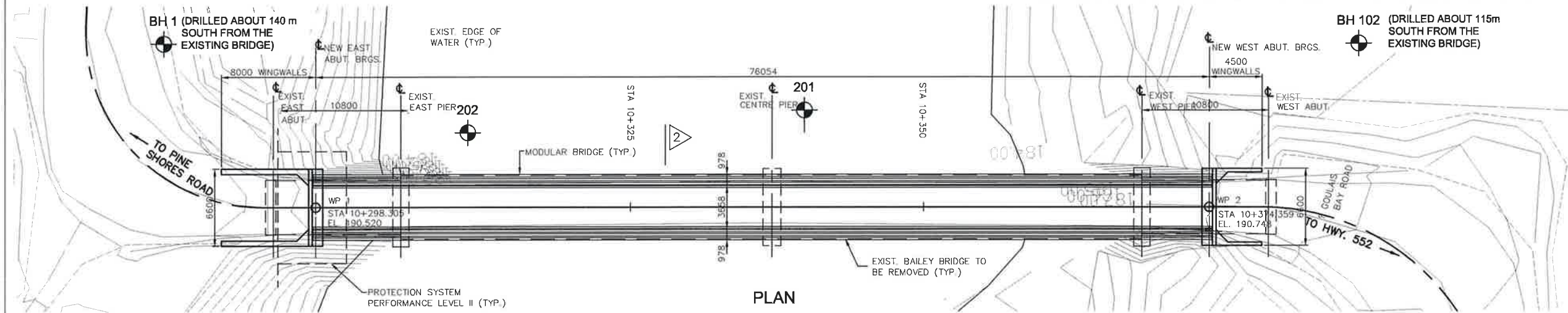
TRANETOBO1156AB						DIST
SUBMD		CHECKED		DATE Sep1 03, 2010		SITE 38S-031
DRAWN SH		CHECKED RM		APPROVED ZO		DWG 1

METRIC

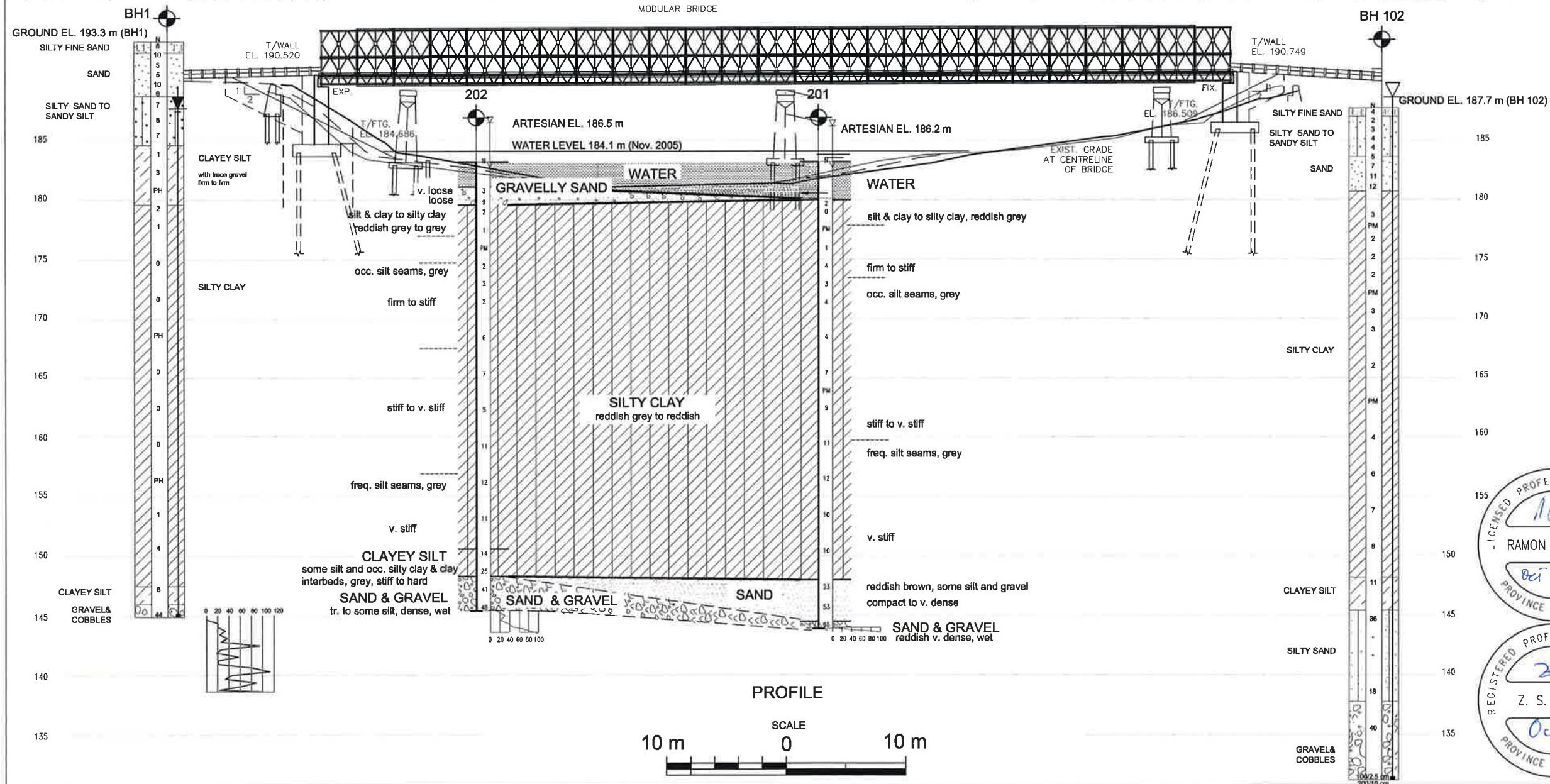
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.



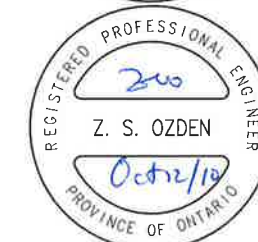
PLAN



PROFILE

SCALE

A horizontal scale bar with a central point labeled '0'. To the left of '0' is a tick mark labeled '10 m'. To the right of '0' is a tick mark labeled '10 m'. The bar is divided into segments by vertical lines.



Appendix A

Record of Borehole Sheets

TRANETOB01156AB

RECORD OF BOREHOLE No 201

1 OF 3

METRIC

GWP 5290-04-01 LOCATION Shewfelt Bridge Restoration ORIGINATED BY G.R.
DIST HWY BOREHOLE TYPE N casing, wash boring COMPILED BY W.C.
DATUM DATE 7/7/2010 11/7/2010 CHECKED BY Z.O.

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	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
183.2 0.0	WATER SURFACE											
	WATER											
180.0 3.2	GROUND SURFACE											
	silt & clay to silty clay reddish grey		1	SS	2		183					
			2	SS	0		182					
							181					
							180					
							179					0 1 40 59
			3	TW	PM		178	40				consolidation tes
							177	22				
							176					
	SILTY CLAY reddish grey to reddish, firm to stiff		4	SS	1		175	15				
							174	20				
			5	SS	4		173	3.2				0 0 32 68
							172	3.5				
	occ. silt seams, grey		6	SS	3		171	3.1				
							170	1.8				
			7	SS	4		169	2.7				
								3.0				water pressure in the casing - up and down (slight artesian condition)
168.2			8	SS	4							0 0 19 81

Continued Next Page

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

TRANETOB01156AB

RECORD OF BOREHOLE No 201

2 OF 3

METRIC

GWP 5290-04-01 LOCATION Shewfelt Bridge Restoration ORIGINATED BY G.R.
 DIST HWY BOREHOLE TYPE N casing, wash boring COMPILED BY W.C.
 DATUM DATE 7/7/2010 11/7/2010 CHECKED BY Z.O.

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	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
168.2 15.0								SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE				
								30 60 90 120 150		20 40 60		
			9	SS	7		168	2.8				
							167	1.8				
							166					
			10	TW	PM		165	3.5				
							164	3.1				
			11	SS	9		163					
							162	3.3				
							161	3.5				
			12	SS	11		160					
							159	3.2				
							158	3.5				
			13	SS	12		157					
							156	>>				
							155	>>				
			14	SS	10		154					

SILTY CLAY
reddish grey to reddish, stiff to v. stiff

freq. silt seams, grey

water pressure in
the casing - up
and down (slight
artesian condition)
water pressure in
the casing - up
and down (slight
artesian condition)

0 0 30 70

water pressure in
the casing - up
and down (slight
artesian condition)

0 0 22 78

Continued Next Page

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

TRANETOBO1156AB

RECORD OF BOREHOLE No 201

3 OF 3

METRIC

GWP 5290-04-01 LOCATION Shewfelt Bridge Restoration ORIGINATED BY G.R.
DIST HWY BOREHOLE TYPE N casing, wash boring COMPILED BY W.C.
DATUM DATE 7/7/2010 11/7/2010 CHECKED BY Z.O.

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. × LAB VANE							WATER CONTENT (%)
153.2 30.0	SILTY CLAY reddish grey to reddish, v. stiff														
			15	SS	10										0 2 43 55
148.0 35.2	SAND some silt and gravel reddish brown, wet, compact to v. dense														
			16	SS	23										
			17	SS	53										
144.5 38.7	SAND AND GRAVEL		18	SS	95									48 42 (10)	
143.9 39.3	reddish, v. dense, wet													soil back up 0.3 m	
143.6 39.6	End of Borehole.													at the bottom	
	DCPT performed from the bottom of borehole. End of DCPT @ 39.6 m.													Artesian condition @ 39.3 m depth, water in the casing 3 m above river level (El. 186.2 m)	

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

TRANETOB01156AB

1 OF 3

METRIC

GWP	5290-04-01	LOCATION	Shewfelt Bridge Restoration	ORIGINATED BY	G.R.
DIST	HWY	BOREHOLE TYPE	N casing, wash boring	COMPILED BY	W.C.
DATUM	DATE	11/7/2010	7/14/2010	CHECKED BY	Z.O.

[illegible]

Continued Next Page

+³, ×³; Numbers refer to Sensitivity

TRANETO01156AB

RECORD OF BOREHOLE No 202

2 OF 3

METRIC

GWP 5290-04-01 LOCATION Shewfelt Bridge Restoration ORIGINATED BY G.R.
 DIST HWY BOREHOLE TYPE N casing, wash boring COMPILED BY W.C.
 DATUM DATE 11/7/2010 7/14/2010 CHECKED BY Z.O.

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
FLEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR × LAB VANE	WATER CONTENT (%)					
168.2 15.0							168	2.1						
							167	2.3						
			10	SS	7		166							
							165							0 1 19 80
							164	2.3						
							163	2.9						
			11	SS	5		162							
	SILTY CLAY reddish grey to reddish, stiff to v. stiff						161	>3.2						
							160	>3.4						
			12	SS	11		159							
							158	>3.1						
							157	>3.2						
	freq. silt seams, grey		13	SS	12		156							0 0 18 82
							155	>3.0						
							154	>2.5						

Continued Next Page

+ 3, × 3; Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

TRANETO01156AB

RECORD OF BOREHOLE No 202

3 OF 3

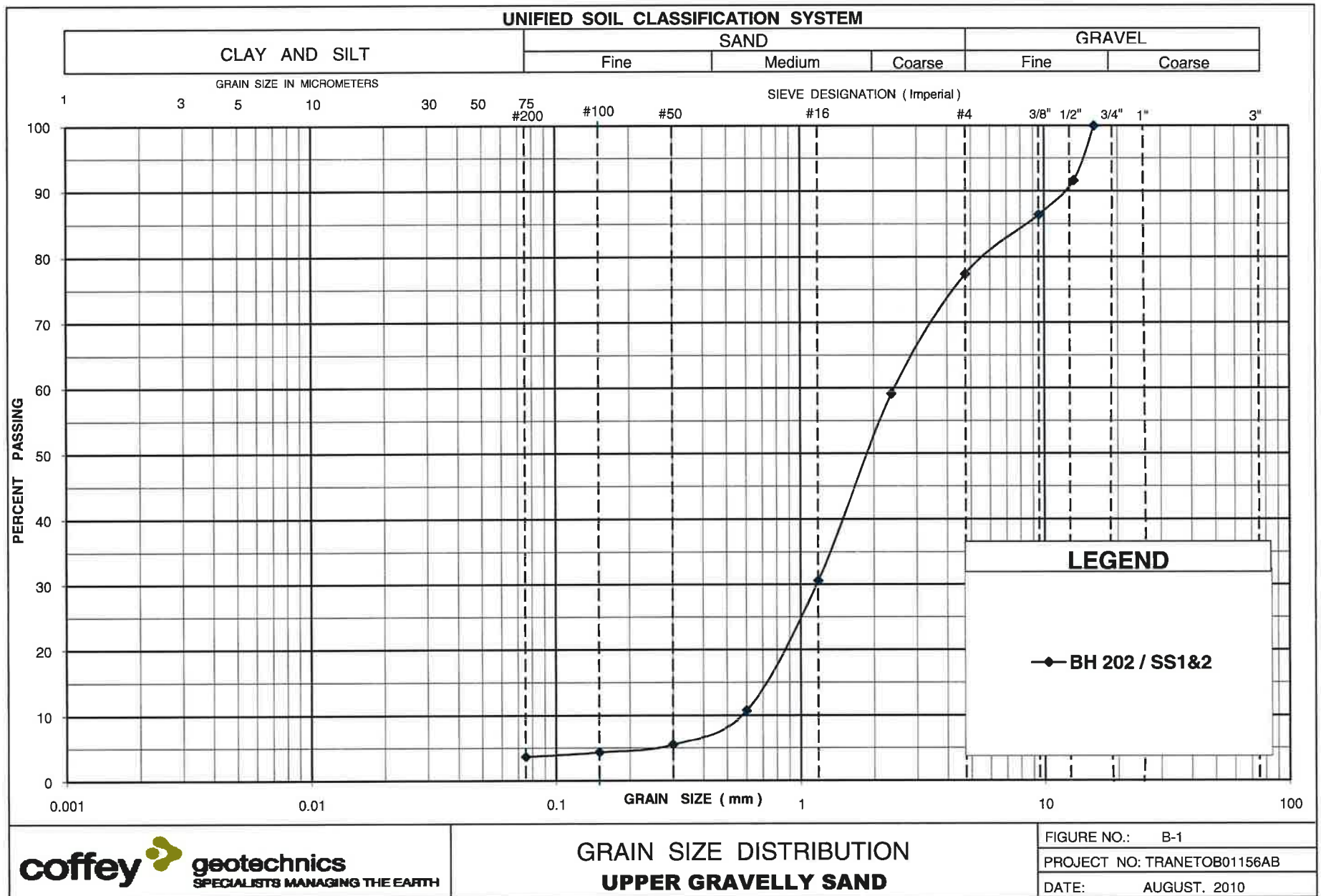
METRIC

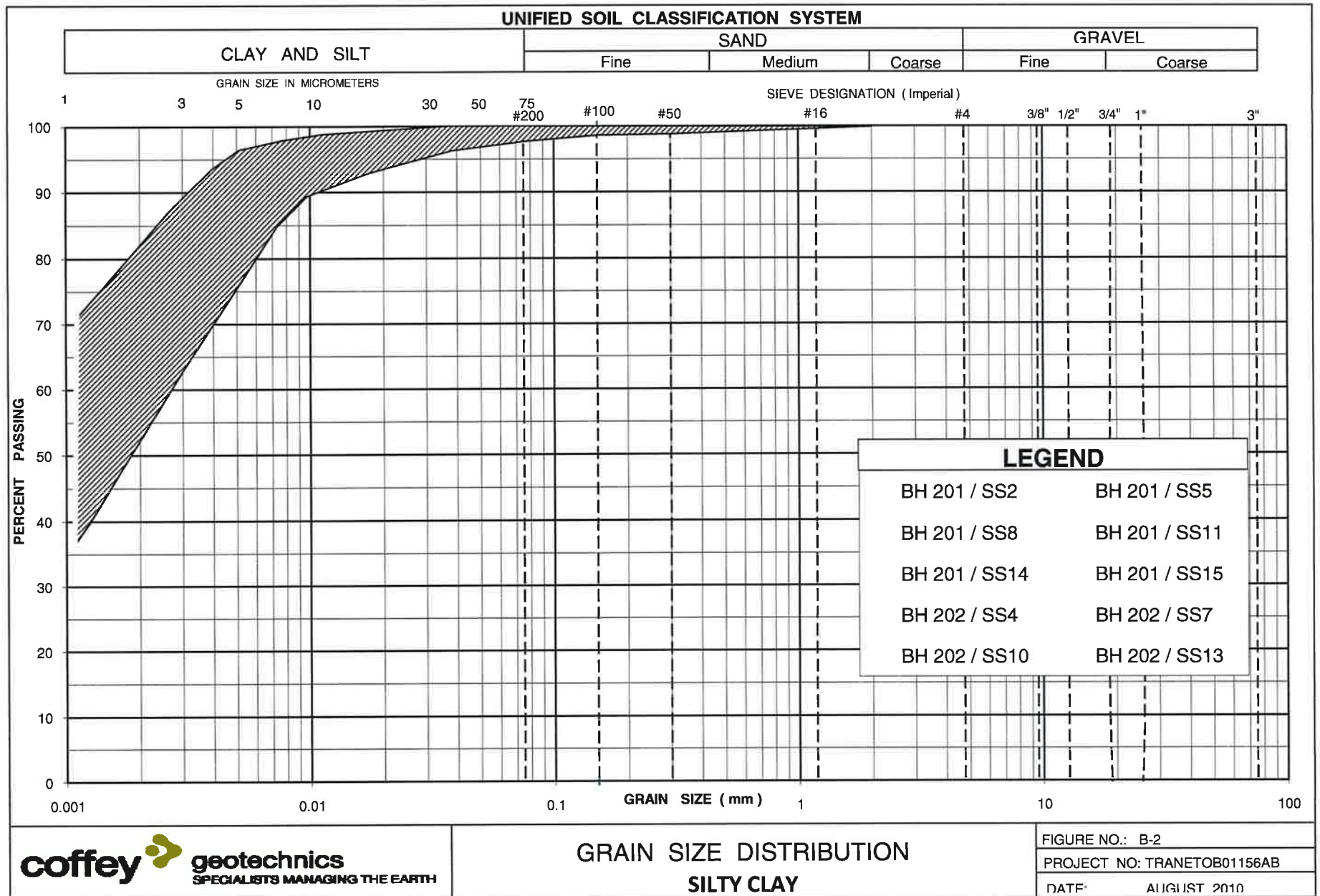
GWP 5290-04-01 LOCATION Shewfelt Bridge Restoration ORIGINATED BY G.R.
DIST HWY BOREHOLE TYPE N casing, wash boring COMPILED BY W.C.
DATUM DATE 11/7/2010 7/14/2010 CHECKED BY Z.O.

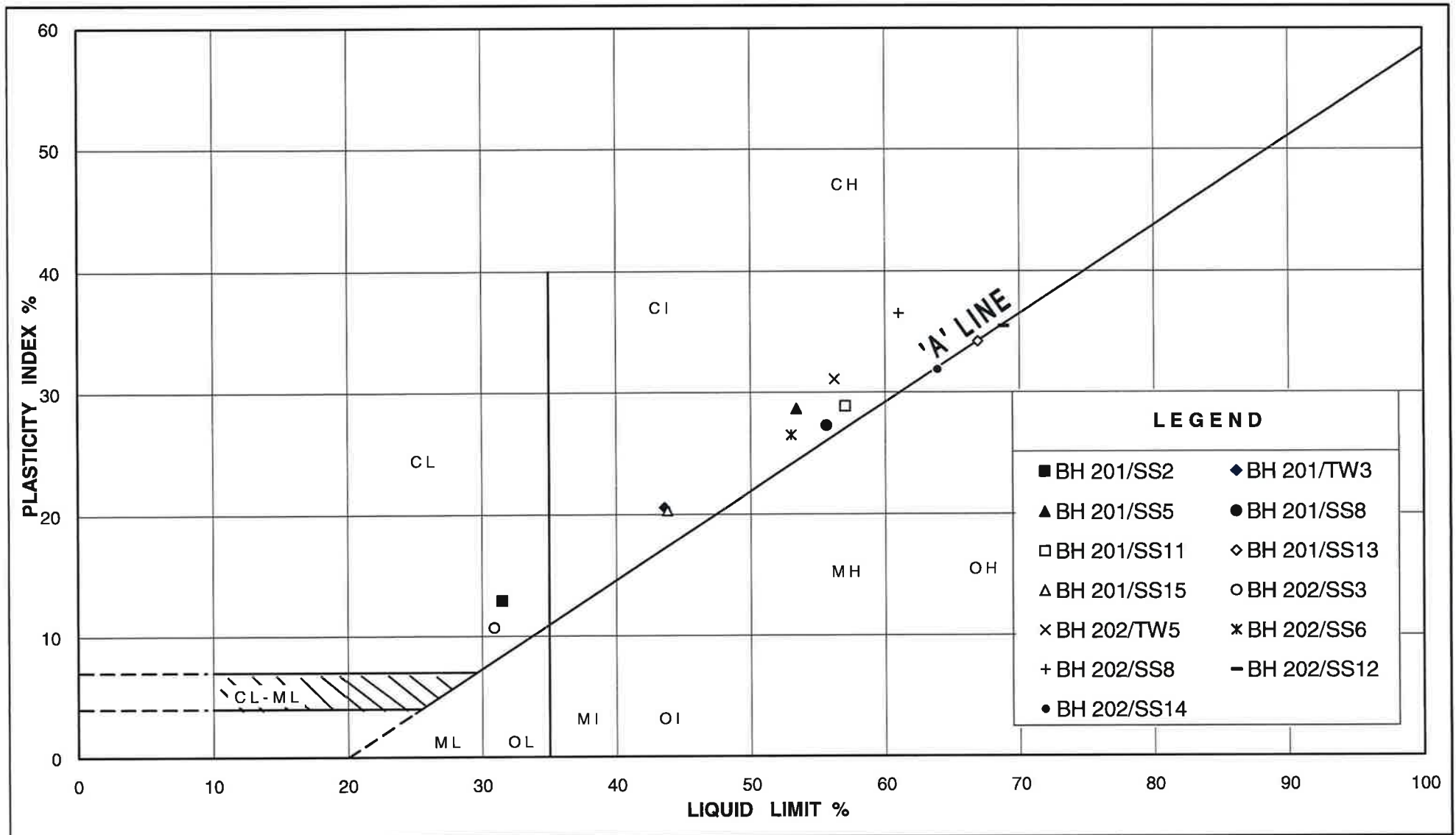
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR x LAB VANE					PLASTIC LIMIT W _P NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L
153.2 30.0	SILTY CLAY reddish grey to reddish, v. stiff		14	SS	11								
150.6 32.6	CLAYEY SILT some silt and occ. silt clay & clay interbeds grey, stiff to hard		15	SS	14								
148.6 34.6	SAND AND GRAVEL tr to some silt reddish to reddish brown, dense, wet		16	SS	25								
			17	SS	41								
			18	SS	48								
145.4 37.6	End of Borehole. DCPT begins @ 37.2 m.												
143.6 39.6	DCPT performed from the bottom of borehole (on backup). End of DCPT @ 39.6 m.												

Appendix B

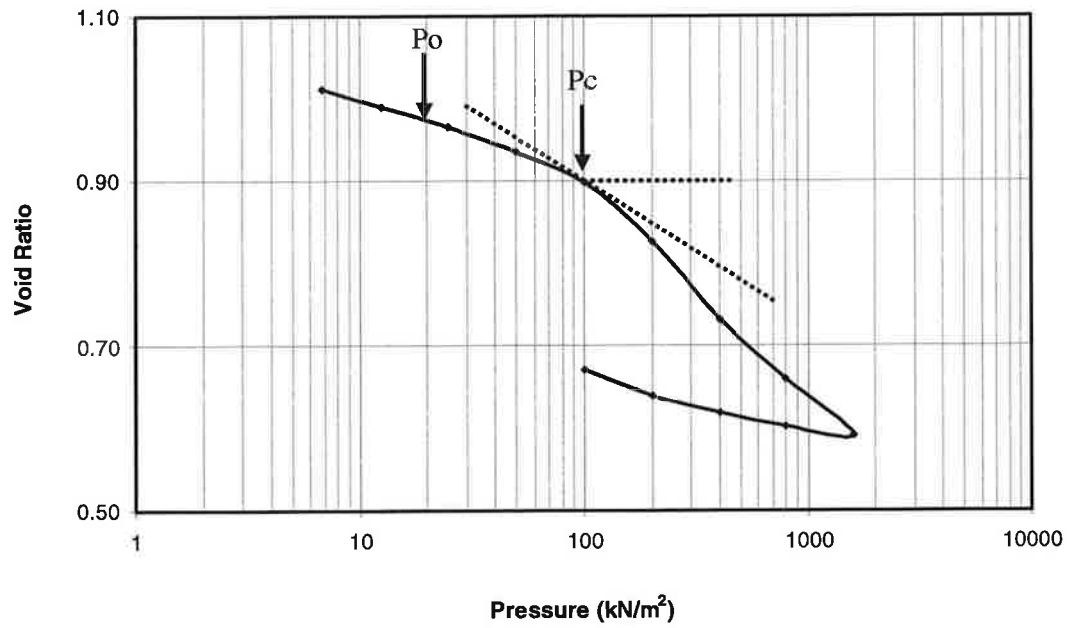
Laboratory Test Results



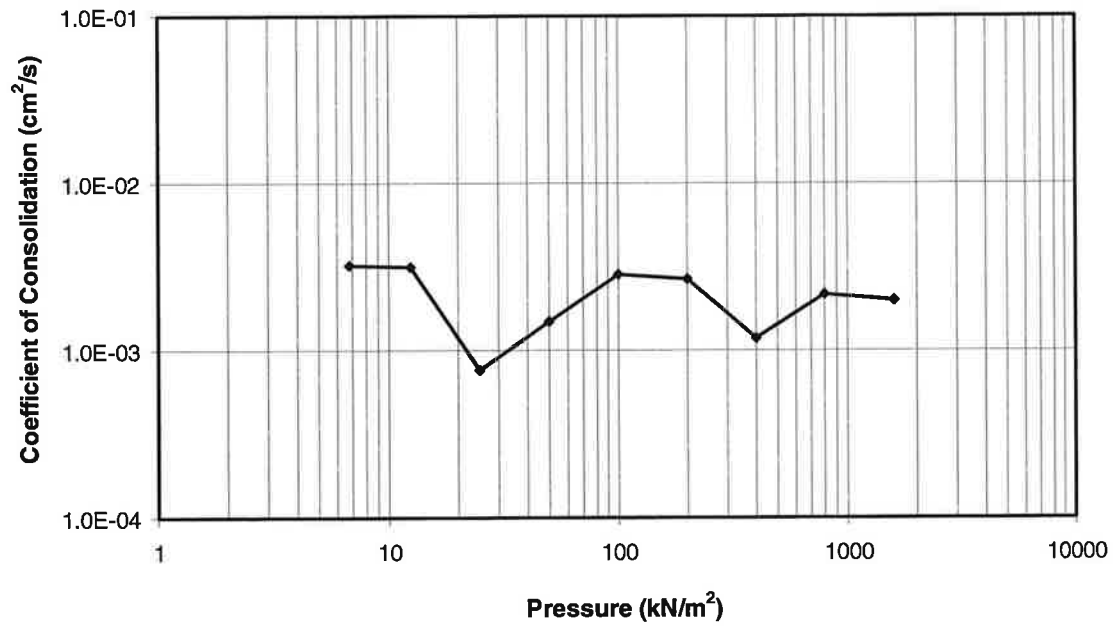




Void Ratio versus Pressure

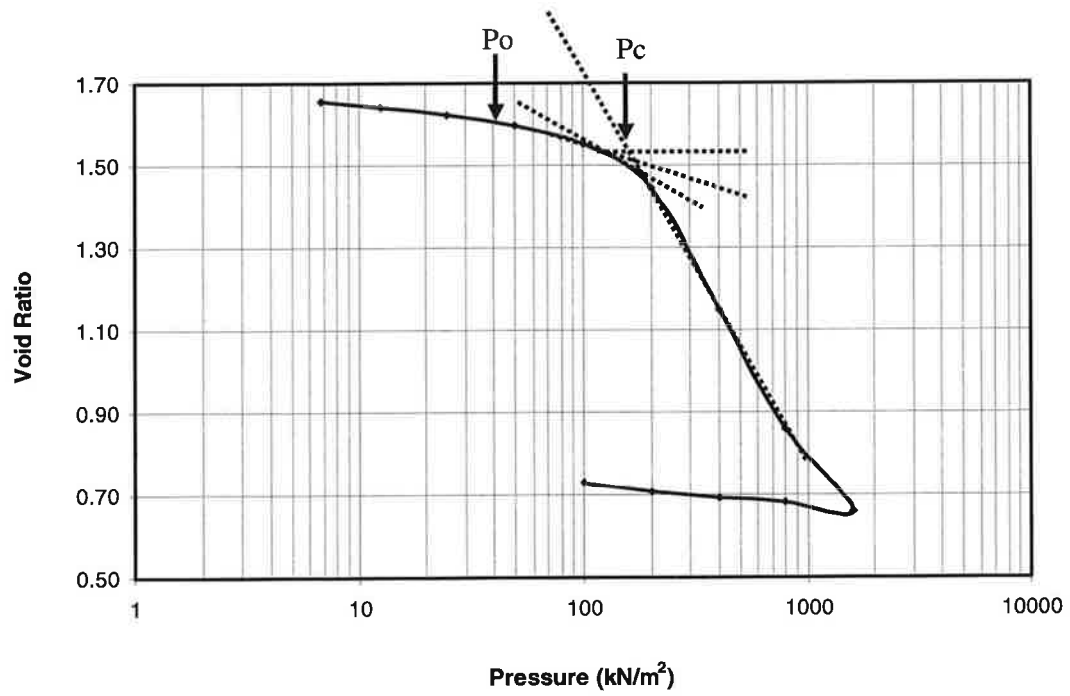


Coefficient of Consolidation vs. Pressure

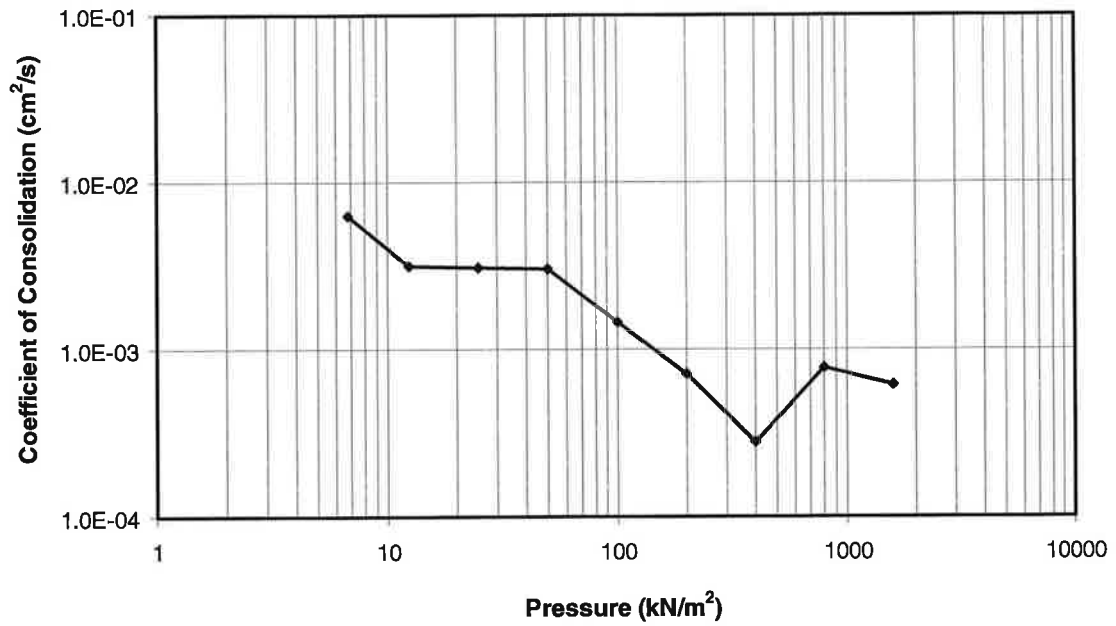


B-4 Borehole B 201 TW3

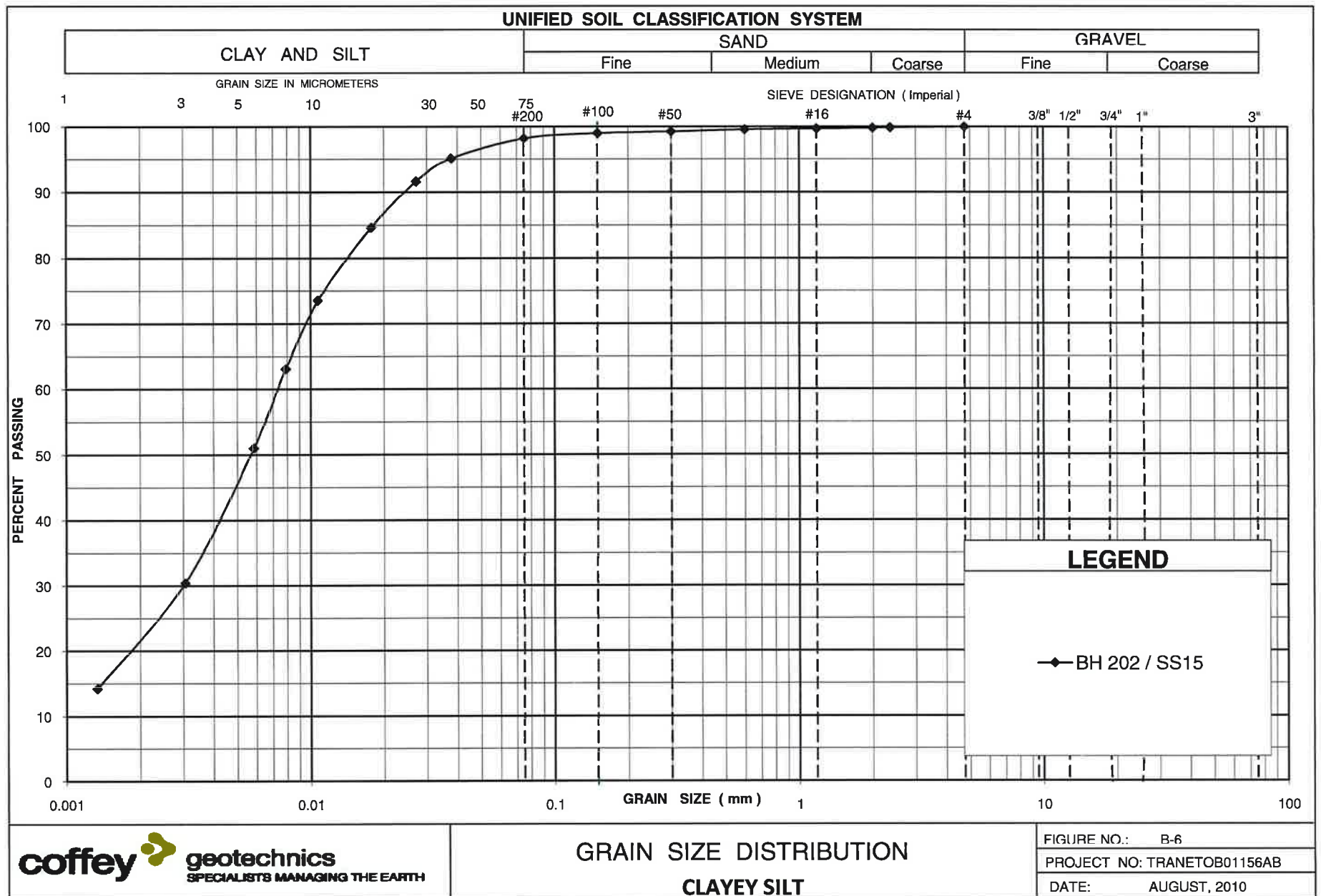
Void Ratio versus Pressure

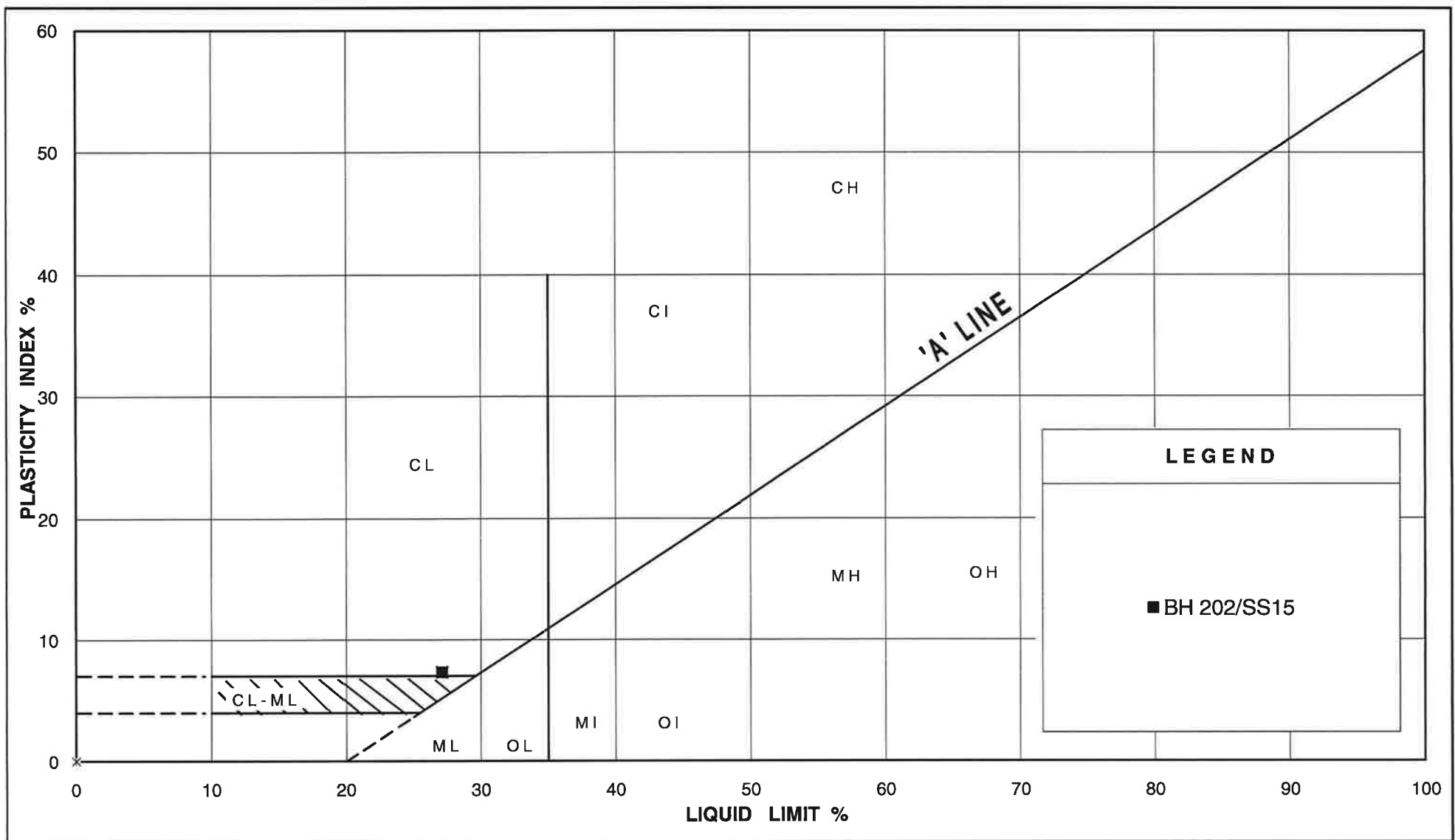


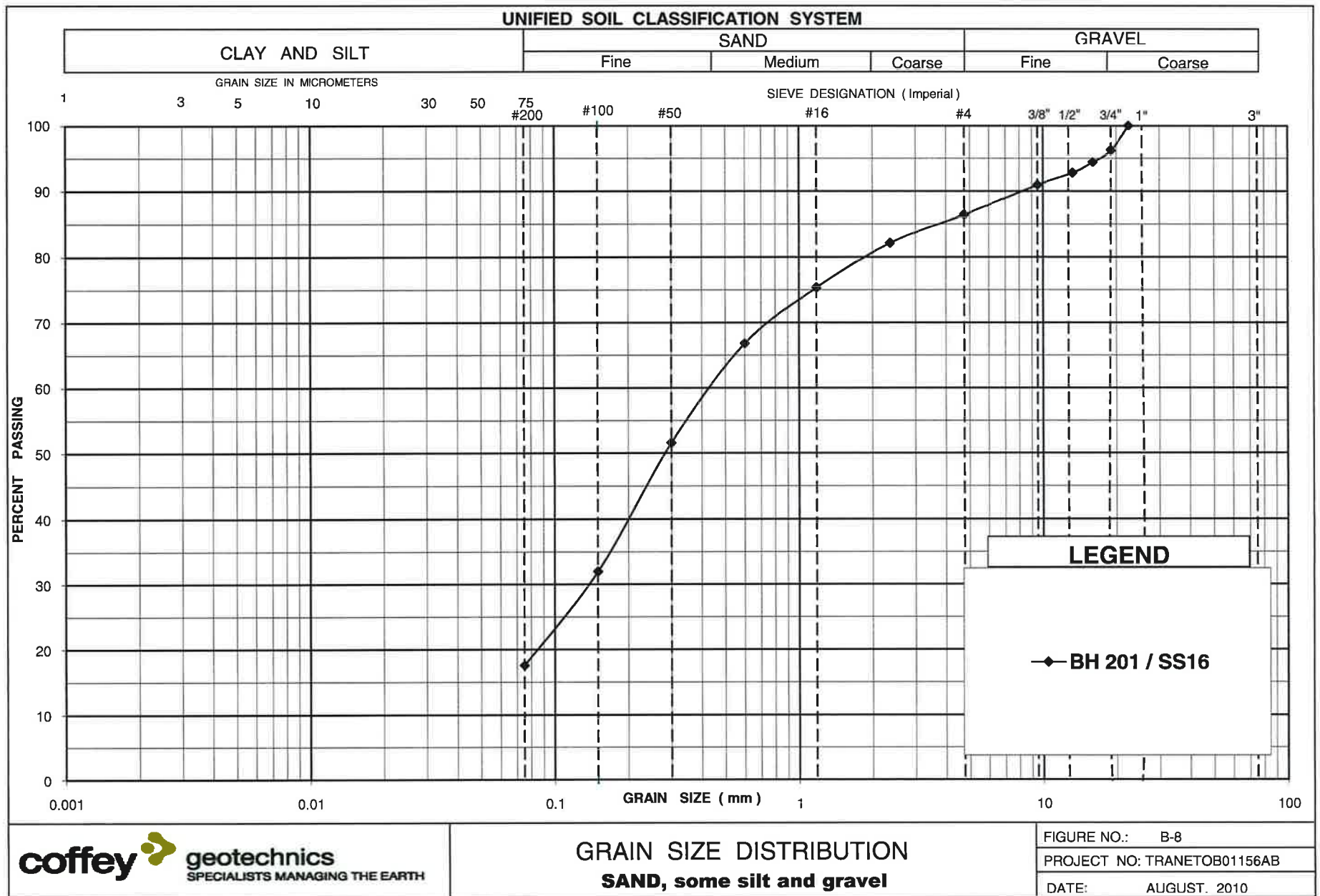
Coefficient of Consolidation vs. Pressure

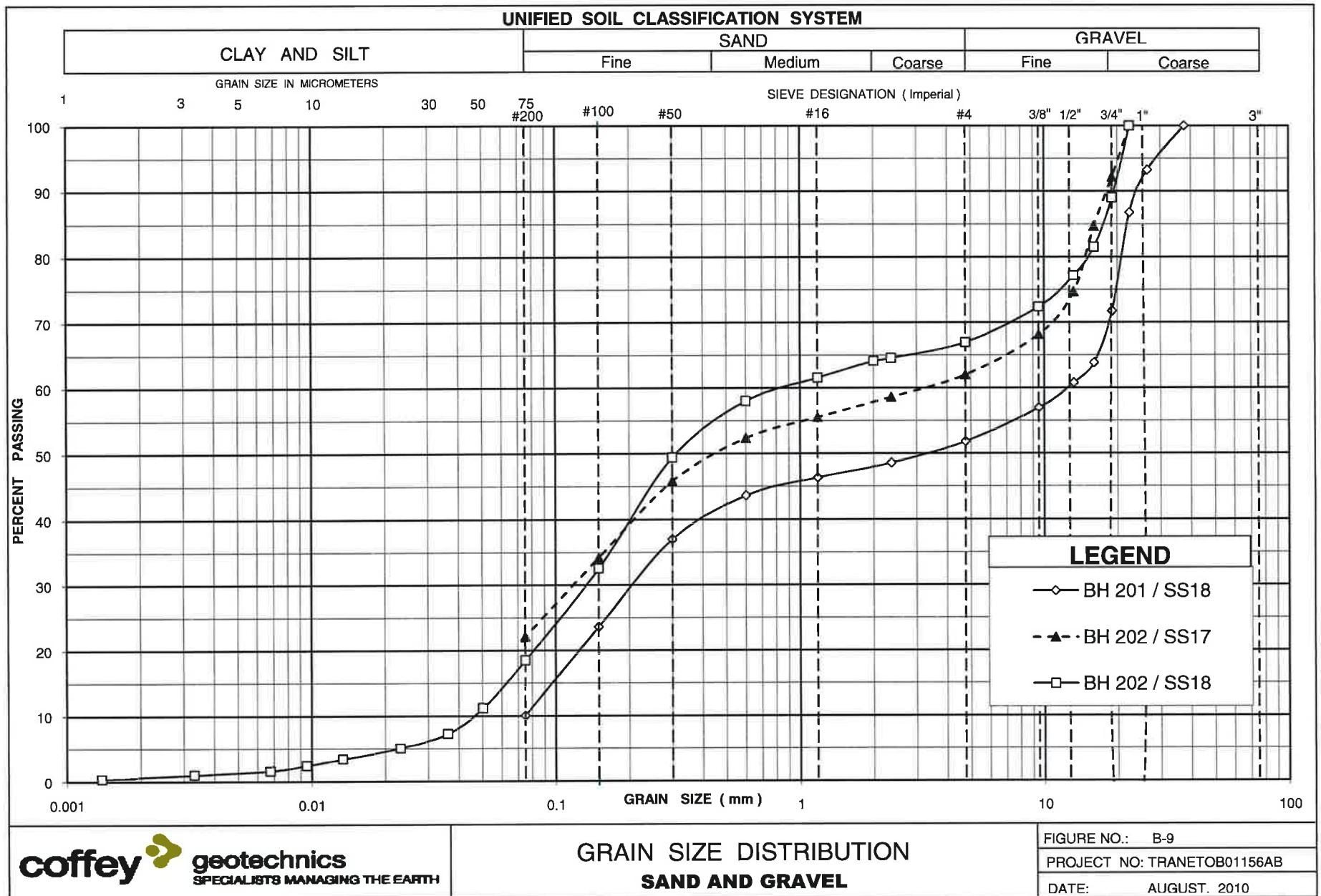


B-5 Borehole B 202 TW5









Appendix C

Site Photographs



Photograph 1: Project site (looking north)



Photograph 2: Project site (looking south)



Photograph 3: Existing bridge east pier

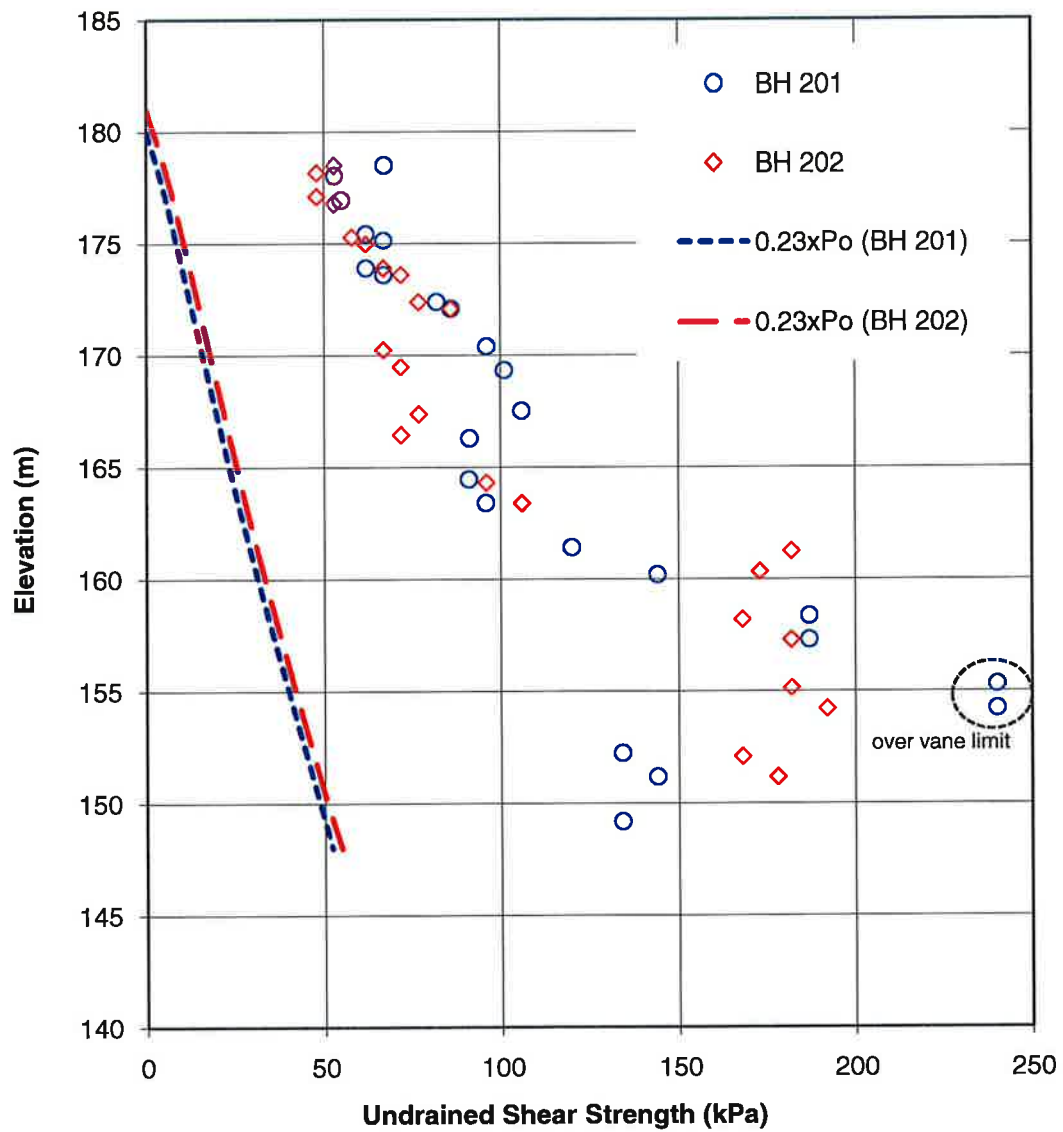


Photograph 4: Existing bridge central pier

Appendix D

Undrained Shear Strength Plot

Figure D-1 Shear Strength Plot



Appendix E

Previous Foundation Investigation Report (2009)

**FOUNDATION INVESTIGATION REPORT
SHEWFELT BRIDGE REPLACEMENT
GOULAIS BAY ROAD, 3 KM WEST OF
HIGHWAY 17,
DISTRICT OF ALGOMA, ONTARIO
G.W.P. 5290-04-00, SITE 38S-031
GEOCRES NO. 41K-82**

LEA Consulting Limited

Project: SPT1156
December 09, 2009

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2	SITE DESCRIPTION AND GEOLOGY	1
3	INVESTIGATION PROCEDURES	2
4	SUBSURFACE CONDITIONS	4
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Drawing 1: Borehole Location Plan

Drawing 2: Stratigraphy

Drawing 3: Borehole Location Plan & Stratigraphy

Drawing 4: Borehole Location Plan & Stratigraphy

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Appendix A: Record of Borehole Sheets

Appendix B: Laboratory Test Results

Appendix C: Undrained Shear Strength Plots

Appendix D: Site Photographs

Appendix E: Rock Core Photographs

Appendix F: Explanation of Terms Used in Report

**FOUNDATION INVESTIGATION REPORT
SHEWFELT BRIDGE PLACEMENT, GOULAIS BAY ROAD
3 KM WEST OF HIGHWAY 17, DISTRICT OF ALGOMA
ONTARIO, G.W.P. 5290-04-00, SITE 38S-031**

1 INTRODUCTION

Coffey Geotechnics Inc. (Coffey) was retained by LEA Consulting Limited (LEA) to carry out a foundation investigation at the site of the proposed replacement of the Shewfelt Bridge over the Goulais River on Goulais Bay Road between Highway 552 and Pine Shores Road, in the Township of Fenwick, approximately 3 km west of Highway 17. The site is located within the District of Algoma and has MTO Site Number 38S-031.

The existing Shewfelt Bridge is a four-span bridge with a total length of 84.7 m, which contains a two-span single lane Bailey bridge (63.1 m) with a timber deck, and two steel girder end spans with a concrete deck, each 10.8 m in length. It is understood that the performance of the existing bridge is affected by the problems of bridge foundation settlements and rotation, slope stability, active erosion and riverbank slumping (upstream of the existing bridge).

In 2006, Shaheen & Peaker Limited (now known as Coffey Geotechnics Inc.) was retained to carry out an advance foundation investigation. During this investigation, a borehole was put down on each side of the river at the proposed new bridge location. Borehole 1 was drilled at the proposed east abutment location and Borehole 2 was advanced at the west abutment location. The findings of that investigation were presented in our report entitled "Advance Foundation Investigation, Shewfelt Bridge Replacement, Goulais Bay Road, 3 km west of Highway 17, District of Algoma, Ontario, G.W.P. 5290-04-00, Site 38S-031," dated May 24, 2006, Project No. SPT1156A.

Since then the location of the proposed bridge was finalized and Coffey was retained by LEA to conduct another investigation for the detail design of the bridge and for the approach roads.

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

The findings of the investigation are presented in this report.

2 SITE DESCRIPTION AND GEOLOGY

The Goulais River is located in a deep and wide valley (the Goulais River Valley) north of Sault Ste. Marie. In the general vicinity of the project site, the area is referred to as the Goulais River Beach Ridges, which is described as ancient beach ridges of an alluvial plain. The river meanders on its way toward Lake Superior and numerous oxbow lagoons are evident.

The Goulais River in the vicinity of the project site has steep banks, with bank failures having occurred in many areas. It is evident that the Goulais River is continuing to undercut its banks at turns in the river,

resulting in slope failures and re-alignment of the river channel. It is noted that a section of the existing Goulais Bay Road located to the north of the existing bridge near the west bank of the river is at close proximity to such a bend in the present river channel. Appendix D provides photographs illustrating the site setting.

Based on available information, the Goulais River Valley was probably cut by a major pre-glacial river. At the time of the retreat of the last glaciations, a river flowed in the Goulais Valley carrying glacial materials into Glacial Lake Algonquin, resulting in deep glacial deposits. As well, it appears that deep clays were deposited and followed by sands and silts deposited by the present river itself.

3 INVESTIGATION PROCEDURES

The fieldwork for the proposed Shewfelt Bridge was performed during the period of September 22, 2008 through October 11, 2008. As agreed with MTO, the fieldwork consisted of drilling and sampling two boreholes (Boreholes 101 and 102) for the bridge structure, six boreholes for the approach fills (Boreholes 103 through 107) and two boreholes for a cut section of the proposed road, some 120 to 150 m east of the proposed bridge location (Boreholes 108 and 109), as well as performing field and Dynamic Cone Penetration tests (DCPT). In addition, a 9.1 m deep borehole was drilled adjacent to Borehole 102 to install two shallow piezometers. As mentioned before, two boreholes (Boreholes 1 and 2) were previously drilled at the site in 2006, for the advance investigation. The plan location of the previous and present boreholes is shown in Drawing Nos. 1, 3 and 4. The following table summarizes the borehole locations and drilling depths.

Table 3.1: Borehole Locations and Drilling Depths

Borehole No.	Location	Drilling Depth Below Existing Ground Surface (m)	Dynamic Cone Penetration Tests	Piezometer
BH1	East side of Goulais River	48.3	19.8 m to 24.3 m 25.9 m to 30.5 m 48.3 m to 54.9 m	1 deep piezometer
BH2	West side of Goulais River	41.6	40.2 m to 41.6 m	-
BH101	In the River	47.9*	38.1* m to 40.8 m* 41.3* m to 42.9 m*	-
BH102	West side of Goulais River	56.5	43.4 m to 53.8 m	1 deep piezometer
BH102A	Adjacent to BH102	9.1	-	2 shallow piezometers
BH103	West side of Goulais River	16.5	-	-
BH104	West side of Goulais River	10.4	-	1 piezometer
BH105	West side of Goulais River	8.8	-	-
BH106	West side of Goulais River	4.4	-	-
BH107	East side of Goulais River	6.7	-	-
BH108	East side of Goulais River	6.7	-	1 piezometer

Borehole No.	Location	Drilling Depth Below Existing Ground Surface (m)	Dynamic Cone Penetration Tests	Piezometer
BH109	East side of Goulais River	8.8	-	1 piezometer

*below the water level in the River

Walker Drilling of Utopia, Ontario carried out the drilling, testing and sampling work, under the direction and supervision of a Professional Engineer from Coffey. Boreholes which were put down from the land were advanced using a truck and a track-mounted drilling rigs, both outfitted with tools and equipment for soil sampling and testing. Drilling was effected using hollow-item augers, however, in Borehole 102 wash boring methods were also utilized below a depth of 9 m. As well coring was effected to advance the borehole through cobbles and boulders.

Borehole 101 was located in the river and this borehole was drilled from a raft. The borehole was advanced by wash boring methods, using NW casing. Coring was also utilized to advance the borehole.

In the deep boreholes (BH1, BH2, BH 101 and BH102) drilling mud was utilized to counter-balance the hydrostatic uplift due groundwater.

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

In cohesive (clayey) deposits, where the consistency of the soil permitted, relatively undisturbed samples (TW) were taken with 50 mm or 70 mm diameter thin-walled (Shelby) tubes which were pushed into the bottom of the borehole by the application of static weight or using hydraulic pressure. The undrained shear strength of the soil was also measured in-situ by Field Vane tests. Where the consistency of clay permitted, a standard MTO Field Vane was used to conduct the tests but when the soil became stiffer this was changed to small Field Vane.

As mentioned in Boreholes 101 and 102 (and also in Boreholes 1 and 2), Dynamic Cone Penetration tests were performed. In Dynamic Cone Penetration Test (DCPT), a 51 mm diameter, 60 deg. apex cone point, screw-attached to the tip of A-size rods, is driven into the ground using the same driving energy as in the SPT method. By recording the number of blows to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of relative density/consistency is obtained. Although the interpretation of the test results is difficult because no samples can be obtained by the DCPT method and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic effects which in many cases affect the SPT values, especially when fine-grained granular soils or cobbles/boulders are encountered.

Groundwater conditions in the boreholes were observed during drilling and upon completion in the open boreholes. In addition to the piezometer that was installed in Borehole 1 during the previous investigation, six additional piezometers were installed during this investigation, as detailed in Table 3.1, to enable

groundwater level monitoring in the boreholes over a prolonged period of time without interference from surface water in Boreholes 102, 102A, 104, 108 and 109. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

The borehole locations were established by our field supervisor in relation to centreline stakes that were previously established in the field by surveyors (retained by LEA) prior to our field crew's arrival at the site. Based on the provided bench mark information, the Geodetic ground surface elevations at the borehole locations were surveyed by our fieldwork supervisor. The benchmark used was HCP 1110 which we understand has Geodetic elevation of 189.48 m. This benchmark, which is a railway spike in a root of a 0.6 m diameter spruce tree is located to the south of the road alignment near the eastern bank of the Goulais River.

The soil and rock samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content and unit weight determinations, grain size analyses, one dimensional oedometer (consolidation) and Atterberg Limits tests, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

4 SUBSURFACE CONDITIONS

The sub-surface conditions were explored at nine boreholes (see Table 3.1 in Section 3) during the current investigation and two boreholes during the 2006 investigation (Advance Foundation Investigation). The plan locations of the boreholes are shown on Drawing Nos. 1, 3 and 4. Details of sub-surface conditions encountered at each borehole location for the current and the prior (2006) 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. A stratigraphic profile at bridge section, cut section and fill section is shown on Drawing Nos. 2, 3 and 4. Detailed laboratory test results are enclosed in Appendix B. Rock core photographs are shown in Appendix E.

In general, the sub-surface stratigraphy comprises surficial topsoil and/or minor fill materials overlying typically very loose to loose cohesionless sand, sandy silt and silty sand to sandy silt deposits, which are in turn underlain by a 33 to 38 m thick deposit of soft to very stiff silty clay. The upper and lower 2 to 4 meters of the deposit was found to contain frequent silt and clayey silt interbeds. In the deep boreholes, the silty clay is further underlain by a clayey silt deposit, followed by silty sand, gravels and cobbles at the bottom of the boreholes. Probable bedrock was encountered in Borehole 101 at a depth of about 46 m/ El. 137 m. Borehole 102 was extended some 6 m below this elevation without encountering bedrock.

4.1 Topsoil

A 0.2 to 0.5 m thick sandy topsoil layer was contacted in Boreholes 1, 2, 103, 104, 105, 106, 108 and 109.

4.2 Surficial Fill

Borehole 107 contacted a 0.15 m thick sand and gravel layer followed by silty fine sand fill extending to a depth of 0.8 m below the ground surface or to El. 193.4 m.

Based on a recorded N-value of 24 blows/0.3 m this basically granular surficial fill is considered compact.

4.3 Silty Fine Sand

A surficial silty fine sand deposit was contacted in Boreholes 1, 2, 102 and 106. This granular (non-cohesive) soil extended to a depth of 0.7 to 0.8 m in Boreholes 1, 2 and 102 and to 1.4 m (El. 186.0 m) in Borehole 106.

Standard Penetration tests performed in this deposit yielded N-values which range from 4 blows/0.3 m (Boreholes 2, 102 and 106) to 8 blows/0.3 m (Borehole 1) which indicate a very loose to loose condition.

4.4 Sand

On the east side of the river (where the grade is approximately 6 m higher than the west side) the boreholes show the presence of a surficial sand deposit. In Boreholes 1, 107, 108 and 109, which were put down on the east side of the river, the surficial sand was contacted immediately underlying the topsoil, fill or surficial silty fine sand at depths of 0.3 to 0.8 m below the ground surface or below Elevations 192.7 to 193.4 m. The thickness of this non-cohesive granular deposit was found to range from 1.9 to 3.9 m and it extended to depths of 2.2 to 4.6 m below the ground surface or to El. 188.8 to 191.4 m.

The grain-size distribution of three samples from the deposit from Boreholes 107 and 109 is given in Figure B-1, in Appendix B. This indicates the following grain-size distribution.

Gravel:	1-9%
Sand:	89-95%
Silt & Clay:	2-4%

From the grain-size distribution, the material is considered to be more pervious than the underlying sandy silt to silty sand deposits, with an estimated coefficient of permeability (k) of the order of 1×10^{-1} to 4×10^{-2} cm/sec.

N-values recorded in this deposit generally range from 5 to 27 blows/0.3 m (typically 8 to 16 blows/0.3 m) which indicate a loose to compact condition.

A similar sand layer was also contacted in Borehole 101 immediately below the river bed, as well as in all the boreholes drilled on the west bank (i.e. Boreholes 2, 102, 103, 104, 105 and 106) at depths of 2.5 to 4.1 m below the ground surface or below El. 183.6 m to 184.9 m. The deposit was also contacted on the east bank in Boreholes 108 and 109 at a depth of 4.4 m below the ground surface or below El. 189.2 and 189.3 m, respectively. The thickness of this granular soil was found to range from 1.6 to 4.5 m and it was found to extend to Elevations ranging from 187.7 m to 179.4 m.

The grain-size distribution of seven samples from this granular deposit is given on Figure B-2 in an envelope form. As shown, the following grain-size distribution is indicated:

Gravel:	0-17%
Sand:	80-95%
Silt & Clay:	2-5%

Standard Penetration tests performed in this sand deposit gave N-values which range from 0 to 22 blows/0.3 m indicating a very loose to compact but typically loose to compact relative density.

4.5 Sandy Silt to Silty Sand

A deposit of sandy silt to silty sand was encountered in all the boreholes drilled at the site, except for Borehole 101. In this borehole, the deposit appears to have been eroded by the meandering river.

In the east bank area where the o.g. levels are higher, this deposit was found underlying the sand deposit at depths ranging from 2.2 m (Borehole 108) to 4.6 m (Borehole 1) or below elevations ranging from 191.4 to 188.8 m. In Borehole 1 the thickness of the deposit was found to be 4.4 m where it extended to El. 184.3 m whereas in Boreholes 107, 108 and 109, the thickness of the deposit was found to range from 1.5 to 2.3 m and it extended to El. 189.3 to below El. 187.5 m. In Boreholes 108 and 109 a second layer of the deposit was found below El. 187.6 and 187.7 m, respectively.

On the west bank area, the deposit was found immediately below the topsoil or the surficial silty fine sand at depths of 0.3 to 1.4 m. It appears that the surficial sand deposit was probably eroded by the meandering river. The thickness of the sandy silt to silty sand deposit in the boreholes drilled on the west side of the river was found to range from about 3.4 to 3.7 m near the river (in Boreholes 2, 102, 103 and 104) to 2.1 to 1.6 m in boreholes 105 and 106, away from the watercourse. The deposit was found to extend to El. 182.6 to 184.9 m.

This is basically sandy silt deposit with some silty fine sand zones. The grain-size distribution of five samples from the deposit is presented in an envelope form in Figure B-3 in Appendix B. The results show the following gradation:

Gravel:	0%
Sand:	33-59%
Silt:	33-58%
Clay:	4-14%

This deposit is considered to be less pervious than the sand deposit which overlies it on the east side and underlies it on the west side of the present watercourse location, but more pervious than the massive silty clay deposit which underlies the entire site.

N-values recorded in this unit ranged from 2 to 24 blows/0.3 m but typically 3 to 9 blows/0.3 m. These results indicate a generally very loose to loose relative density with occasional compact zones (e.g. Borehole 107).

4.6 Silty Clay

Underlying the non-cohesive deposits described in the previous sections, all the deep boreholes contacted a massive cohesive deposit at depths ranging from 3.9 m (El. 179.4 m) in Borehole 101 (below the river bottom) to 9.0 m (El. 184.3). The following table summarizes the top and bottom elevations of the deposit, as encountered in the deep boreholes.

Borehole No.	Depth Below Ground Surface/Elevation of the Top of the Deposit(m)	Depth of Below Ground Surface/Elevation of the Bottom of the Deposit (m)
1	9.0/184.3	47.2/146.1
101	3.9/179.4	37.0/146.3
102	7.0/180.7	42.3/145.4
2	7.6/179.5	40.5/146.6

The deposit consists of a reddish silty clay to clay with occasional clayey silt and grey silt seams. The upper and lower portions of the deposit (typically the upper and lower 2 to 4 m) was found to contain frequent clayey silt and silt seams and thus the upper and lower zones typically resemble a layered clayey silt material.

The grain-size distribution of four samples from the deposit (from Boreholes 101 and 102) is given in Figure B-4. Five samples from Boreholes 1 and 2 were tested during our 2006 investigation and these are included as Figure B-5. The results of the tests on the nine samples show the following grain-size distribution:

Gravel:	0-2%
Sand:	0-2%
Silt:	22-64%
Clay:	32-78%

Figure B-6 from 2006 investigation shows the grain-size distribution of a sample from the upper clayey silt zones of Borehole 1. This indicates 74% silt and 26% clay size particles.

When analysing these grain-size results it should be kept in mind that the samples tested is a mixture of several or more individual interbeds.

The Atterberg limits tests performed during the present (17 samples) and the previous investigation (6 samples) are given in Figures B-7, B-8, B-9 and B-10 in Appendix B. These tests yielded the following index values:

Liquid Limit:	25-79% (Average 55%)
Plastic Limit:	16-31% (Average 24%)
Plasticity Index:	9-55 (Average 31)

These results indicate clayey soils of low to high but typically medium to high plasticity. The lower plasticity index values are from the upper or lower more silty zones, typically above and below Elevations 175 and 153 m, respectively. There is also some variation within each zone, where annual deposition shows a range from more plastic (i.e. fatter) to relatively less plastic (i.e. leaner) clay content. As shown on the individual Record of Borehole Sheets, the measured natural moisture contents are generally near or in

excess of the measured liquid limits which indicate the likelihood of a normally consolidated soil deposit, or only a slight pre-consolidation.

Standard Penetration tests conducted in the silty clay deposit gave N-values which typically range from 0 to 3 blows/0.3 m which indicate a very soft consistency but N-values as high as 22 blows/0.3 m were also recorded indicating firm to very stiff zones. The higher N-values were typically recorded in the clayey silt zones which contain stiffer than silt/clayey silt interbeds (generally in the upper and lower zones of the deposit).

The undrained in-situ shear strengths of the deposit were measured in the field by means of field vane tests, using MTO type field vanes. The measured values range from 12 to in excess of 100 kPa, indicating a very soft to very stiff consistency.

In Figures C1 and C2 (Appendix C) the variation of the measured in-situ vane strength values (i.e. in-situ undrained shear strengths) at each deep borehole versus elevation is presented. Also plotted on each figures are the effective overburden stress (P'_o), as well as the plot of $0.23 P'_o$ with elevation. It is commonly acknowledged that with Ontario clays if the measured undrained shear strengths are in excess of $0.23 P'_o$ line, the deposit may be somewhat over-consolidated, perhaps due to removal of previously existing overburden. A total of four oedometer (one dimensional consolidation) tests was performed in the laboratory on 50 to 70 mm diameter Shelby tube (TW) samples (three from Borehole 102 and one from Borehole 103). The results are presented in Figures B-11 through B-14, in Appendix B. These show a possible pre-consolidation pressure in excess of existing overburden pressure $P'_c - P'_o$ in order of 80 to 150 kPa. It should be pointed out that the presence of silty seams was noted in samples TW 11 (Borehole 102) and TW 12 (Borehole 103) and this is expected to have affected consolidation test results because the silty soil typically be less compressible than the adjacent clay.

The measured bulk unit weight of the TW samples range from 15.3 to 18.3 kN/m³.

4.7 Silty Sand with Gravel (Possible Till)

In Boreholes 101 and 102 the silty clay to clayey silt deposit is underlain at depths/elevations of 37.0/146.3 m and 42.3/145.4 m by a relatively coarse grained soil consisting of silty sand with gravel and occasional cobbles and boulders. From an examination of the soil samples recovered from the deposit, it is likely to be of glacial till origin (i.e. a silty sand till). The deposit was found to extend to about depths/elevations of 40.0 m/ El. 143.3 m and 50.0 m/ El. 137.7 m in Boreholes 101 and 102, respectively.

The grain-size distribution of two samples from the deposit from Borehole 102 gave the following results:

Gravel:	34-40%
Sand:	38-43%
Silt:	16-17%
Clay:	6%

as shown in Figure B-15 in Appendix B.

Two Standard Penetration tests performed in this deposit yielded N-values of 36 and 85 blows/0.3 m and Dynamic Cone Penetration tests (DCPT) gave blow counts which range from 14 to 45 blows/0.3 m. Based on these values, the relative density of the deposit is considered to be typically compact to dense with a very dense zone in the upper portion of Borehole 101.

4.8 Silty Fine Sand

Underlying the silty sand till, Borehole 101 contacted at a depth of 40 m (El. 143.3 m) a relatively fine-grained soil consisting of silty fine sand with some gravel and occasional cobbles and boulders. This deposit was found to extend to 42.0 m below the river bed or to El. 140.1 m.

Based on DCPT test results which range from 25 to 60 blows/0.3 (typically 30 to 40 blows/0.3 m), the relative density of the deposit is described as compact to dense.

4.9 Gravel and Cobbles

Underlying the silty clay, Boreholes 1 and 2 encountered at 47.2 m (El. 146.1 m) and 40.5 m (El. 146.6 m), respectively, a deposit which consists of gravel and cobbles. The presence of some sand in-fill as well as occasional boulders was also noted. These two boreholes were terminated in this coarse grained granular deposit at 48.3 m and 41.6 m (El. 145.0 m and 145.5 m), respectively.

Measured SPT N-values in this deposit ranged from 44 blows per 0.3 m (Borehole 1) to 50 blows per 0.08 m penetration (Borehole 2), indicating dense to very dense relative density. The measured natural moisture content of one soil sample was 11%.

Dynamic Cone Penetration Tests (DCPT) were carried out at the bottom of Boreholes 1 and 2. In Borehole 1, DCPT was carried out from 48.5 m to 54.9 m (El. 144.8 m to El. 138.4 m) below ground surface. As can be seen in the DCPT plots in the Record of Borehole Sheets, the DCPT blow counts had high variation with depth (a "zigzag" curve pattern) with test results varying from 19 to over 200 blows per 0.3 m penetration, and this may be due to the presence of cobbles and/or boulders which obstructed the penetration of the cone/rod assembly. This may also be the result in the bending of the rods and the non-vertical penetration of the cone/rod assembly. The DCPT encountered refusal at 54.9 m (El. 138.4 m) below ground surface. From the results, the relative density of the soil below El. 145.0 m can be surmized to be compact to very dense.

In Borehole 2, Dynamic Cone Penetration Test was carried out from 40.2 m to 41.6 m (El. 146.9 m to El. 145.5 m) and encountered refusal at 41.6 m (El. 145.5 m) below ground surface.

The deposit also contacted in Boreholes 101 and 102 at depths/elevations 43.2 m/140.1 m and 50.0 m/137.7 m, respectively. The boreholes were extended in this deposit by a vertical distance of 2.8 m and 6.5 m, respectively. Due to the presence of boulders and cobbles, frequent coring was resorted to advance the boreholes. Borehole 102 was terminated in this deposit while in Borehole 101, coring results indicate the possible presence of bedrock, underlying this deposit at a depth of 44.8 m below the river bottom, or below El. 137.3 m.

In these two boreholes, reliable N-values could not be obtained due to the presence of boulders and advancing by means of coring. But based on observations made while drilling and DCPT results, the relative density is probably compact to dense.

4.10 Probable Bedrock

In Borehole 101, which was put down by washboring methods from a raft (at the proposed pier location) in the river, a reddish brown colored metamorphosed sandstone was contacted at a depth of 44.8 m below the bottom of the river or at El. 137.3 m. This was cored for a vertical distance of 1.9 m to 46.7 m below the bottom of the river or El. 135.4 m where the borehole was abandoned and plugged due to a severe artesian condition. This possibly represents the bedrock, although Borehole 102 was extended below this elevation to El. 131.2 m without encountering bedrock.

The percentage of recovery was 100% while the RQD value recorded from El. 137.3 to 137.0 m was 48% and below this elevation to El. 135.4 m, it was 72%.

4.11 Groundwater Conditions

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

Table 4.11.1 Summary of Groundwater Level Measurements

Borehole No./Elevation	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Piezometers	Remarks
1 (193.3 m)	47.8/145.5	5.6/187.7	March/06	Yes	Lower hydrostatic level
2 (187.1 m)		3.4/183.7	March/06	No	On completion Upper hydrostatic level
101 (182.1 m)	44.0/138.1 46.7/135.4	+3.3/185.4 +5.5/187.6	Nov/08	No	Artesian condition measured inside casing while drilling in lower hydrostatic level
102 (187.7 m)	56.5/131.2	+1.4 to +2.3/ 189.1 to 190.0	Sept-Oct/08	Yes	Artesian condition Lower hydrostatic level
102A (187.7 m)	6.1/181.6 9.1/178.6	4.2/183.5 4.2/183.5	Sep-Oct/08 Sept-Oct/08	Yes Yes	Upper Hydrostatic level Upper Hydrostatic level
103 (187.6 m)		3.6/184.0	Sept/08	No	On completion Upper hydrostatic level
104 (187.4)	9.1/178.3	3.8/183.6	Sept/08	Yes	Upper hydrostatic level
105		3.8/183.6	Sept/08	No	On completion

Borehole No./Elevation	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Piezometers	Remarks
(187.4 m)					Upper hydrostatic level
106 (187.4 m)		3.5/183.9	Sept/08	No	On completion Upper hydrostatic level
107 (194.2 m)		4.2/190.0	Sept/08	No	On completion Upper hydrostatic level
108 (193.6 m)	6.0/.187.6	4.4/189.2	Sept-Oct 08	Yes	Upper hydrostatic level
109 (193.7 m)	7.6/186.1	4.2/189.5	Oct/08	Yes	Upper hydrostatic level

From the measured values, it appears that there are two distinct water/piezometric levels at the site, namely an upper level and a lower level (below the silty clay deposit).

On the east bank, the upper water level was measured at about El. 189.2-190.0 m while the lower piezometric level deeper in the profile was measured at El. 187.7 m. On the west bank, the upper water level was measured at El. 183.5-184.0 m, while the lower piezometric level deeper in the profile was measured at 189.1-190.0 m (i.e. similar to the water level measured in the piezometer installed in the east bank area). This represents an artesian condition in relation to the ground levels on the west side of the river. In the borehole drilled in the middle (i.e. within the river), an artesian condition was noted which was measured to reach 5.5 m above the bottom of the river or El. 187.6 m. It is likely that this level could have reached El. 189-190 m. It is also of interest to note that when the artesian condition was being measured, the water level in the deep piezometer installed on the west bank (i.e. Borehole 102) was noted to start reacting (i.e. dropping).

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

For and on behalf of Coffey Geotechnics Inc.


Ramon Miranda, P.Eng.





Zuhtu Ozden, P.Eng.



Drawings

Previous Foundation Investigation Report (2009)

METRIC

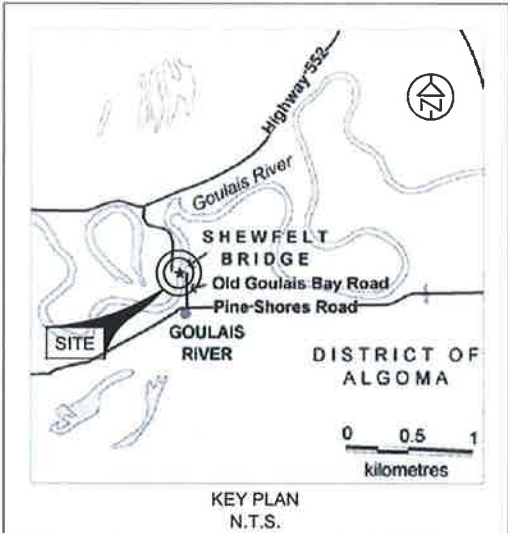
NOTES:
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No.
GWP: 5290-04-00

OLD GOULAIS BAY ROAD
SHEWFELT BRIDGE
BOREHOLE LOCATION PLAN

SHEET

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SPECIALISTS MANAGING THE EARTH



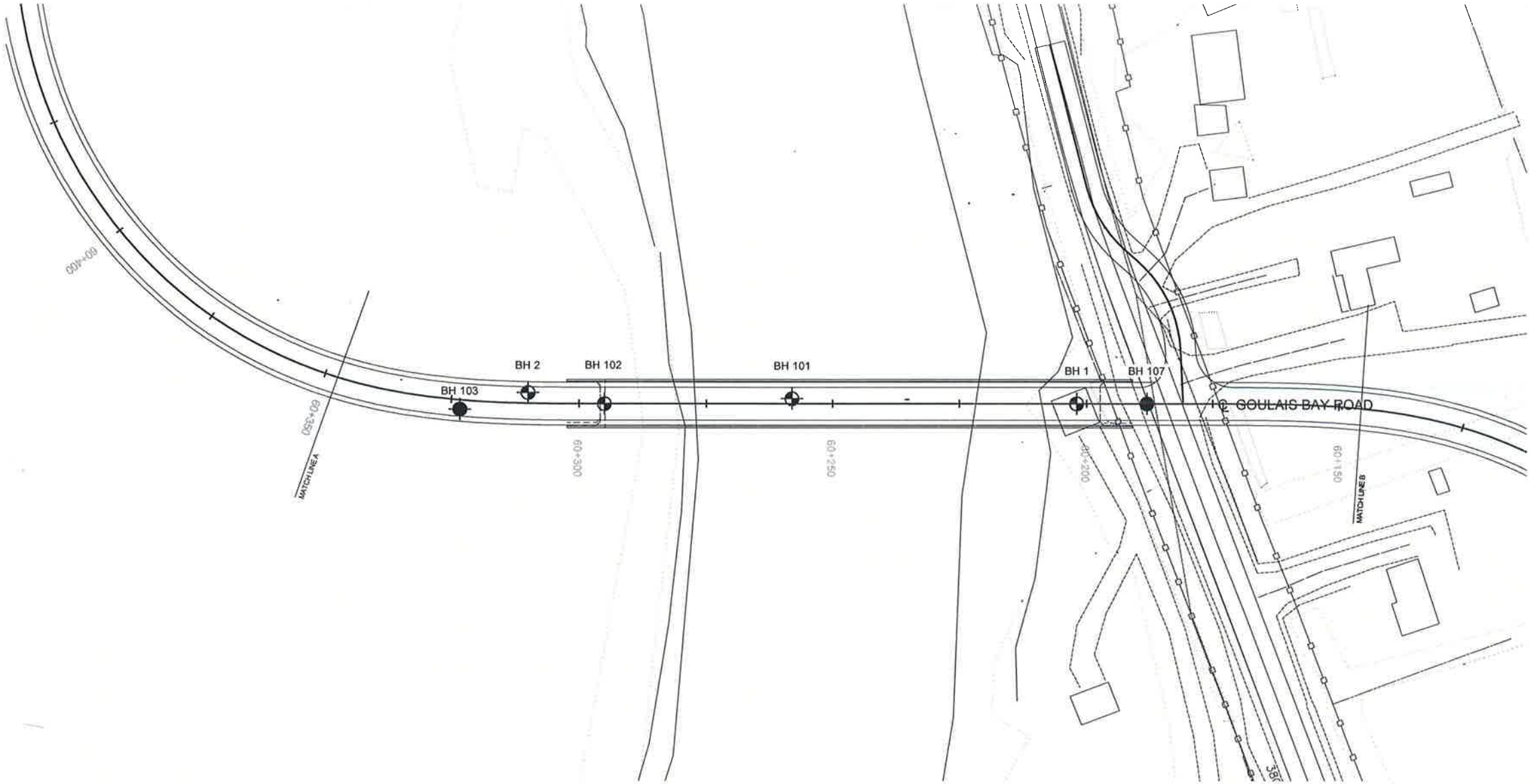
LEGEND			
	Borehole		
	Borehole & Cone		
No.	ELEVATION	NORTHING	EASTING
BH 101	183.3m	5175927.5	275561.3
BH 102	187.7m	5175938.2	275525.9
BH 103	187.6m	5175948.2	275498.7
BH 107	194.2m	5175904.3	275627.4
BH 1	193.3m	5175908.7	275614.1
BH 2	187.1m	5175945.1	275512.4

-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 41K-82			
TRANETO801156AA			DIST
SUBMD	CHECKED	DATE Dec 8, 2009	SITE 38S-031
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1

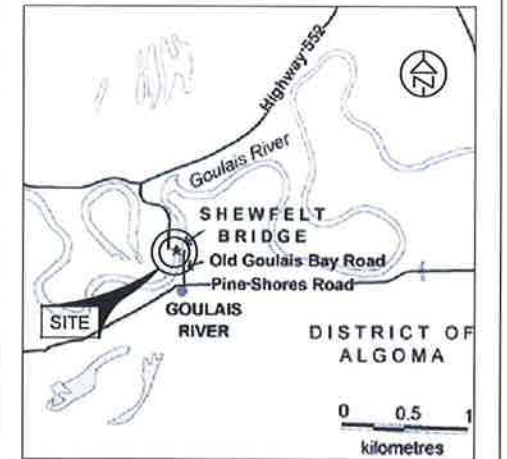


DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.










FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

OLD GOULAIS BAY ROAD
SHEWFELT BRIDGE
SOIL STRATA

coffey  **geotechnics**
SPECIALISTS MANAGING THE EARTH



LEGEND

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

No.	ELEVATION	NORTHING	EASTING
BH 101	183.3m	5175927.5	275561.3
BH 102	187.7m	5175938.2	275525.9
BH 103	187.6m	5175948.2	275498.7
BH 107	194.2m	5175904.3	275627.4
BH 1	193.3m	5175908.7	275614.1
BH 2	187.1m	5175945.1	275512.4

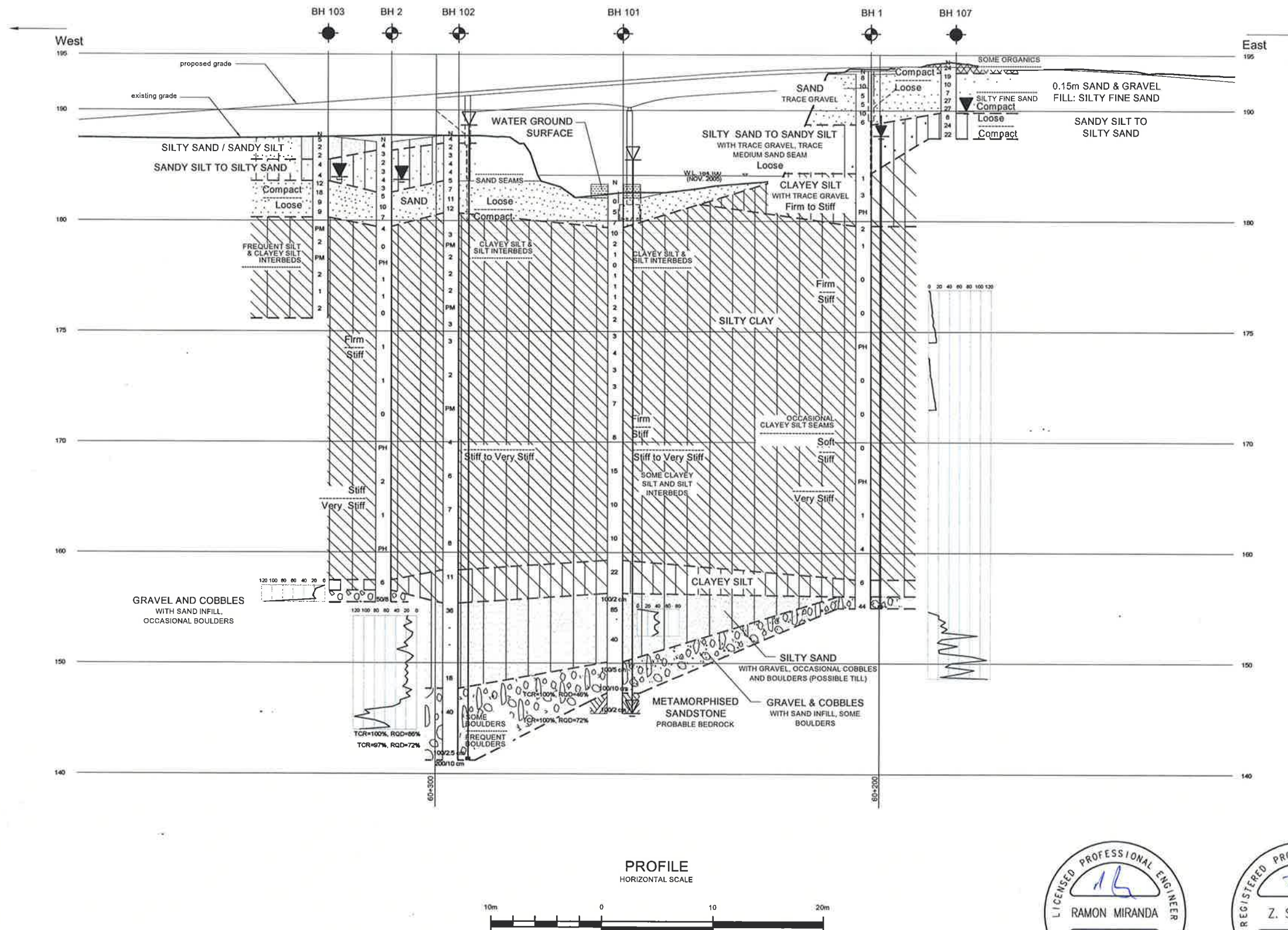
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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

REVISIONS			
	DATE	BY	DESCRIPTION

Geocres No 41K-82

TRANETO01156AA				DIST	
SUBMDD		CHECKED	DATE	Dec 8, 2009	SITE 38S-031
DRAWN	PHK	CHECKED	RM	APPROVED	20 DWG 2



METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.
GWP: 5290-04-00

OLD GOULAIS BAY ROAD
SHEWFELT BRIDGE
BOREHOLE LOCATION PLAN
AND SOIL STRATA



SHEET

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KEY PLAN
N.T.S.

LEGEND

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

No.	ELEVATION	NORTHING	EASTING
BH 104	187.4m	5175959.3	275478.6
BH 105	187.4m	5175975.0	275461.9
BH 106	187.4m	5176017.2	275442.4

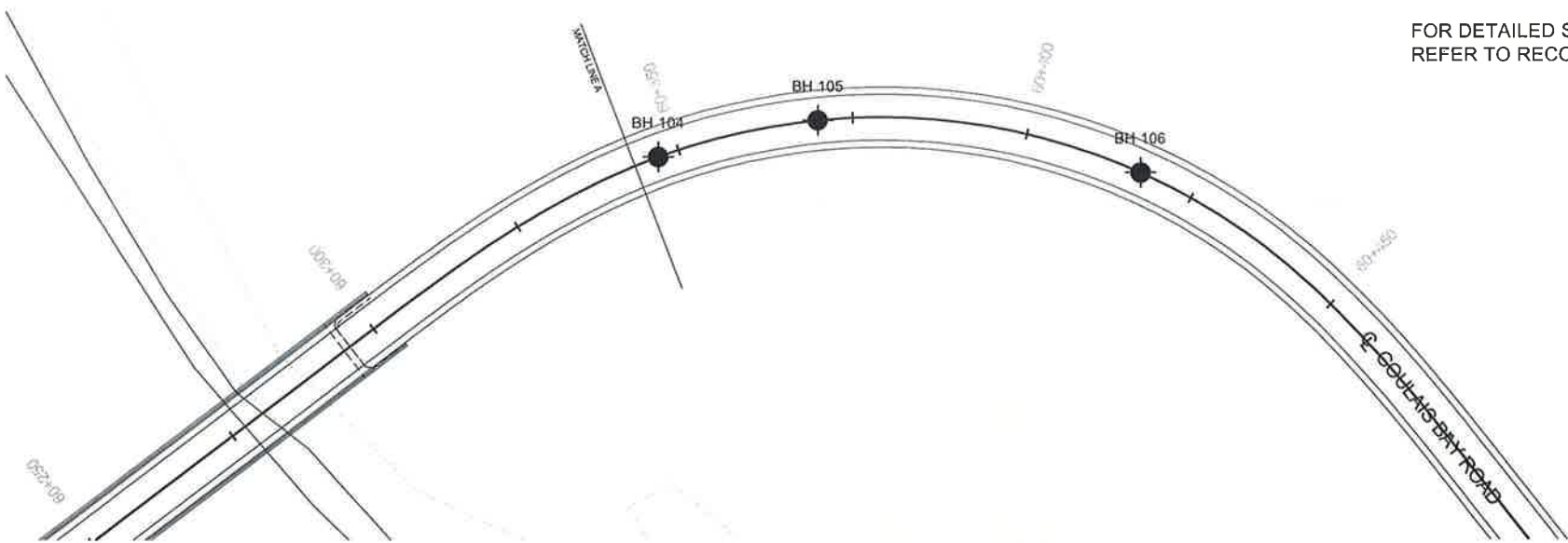
-NOTE-

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

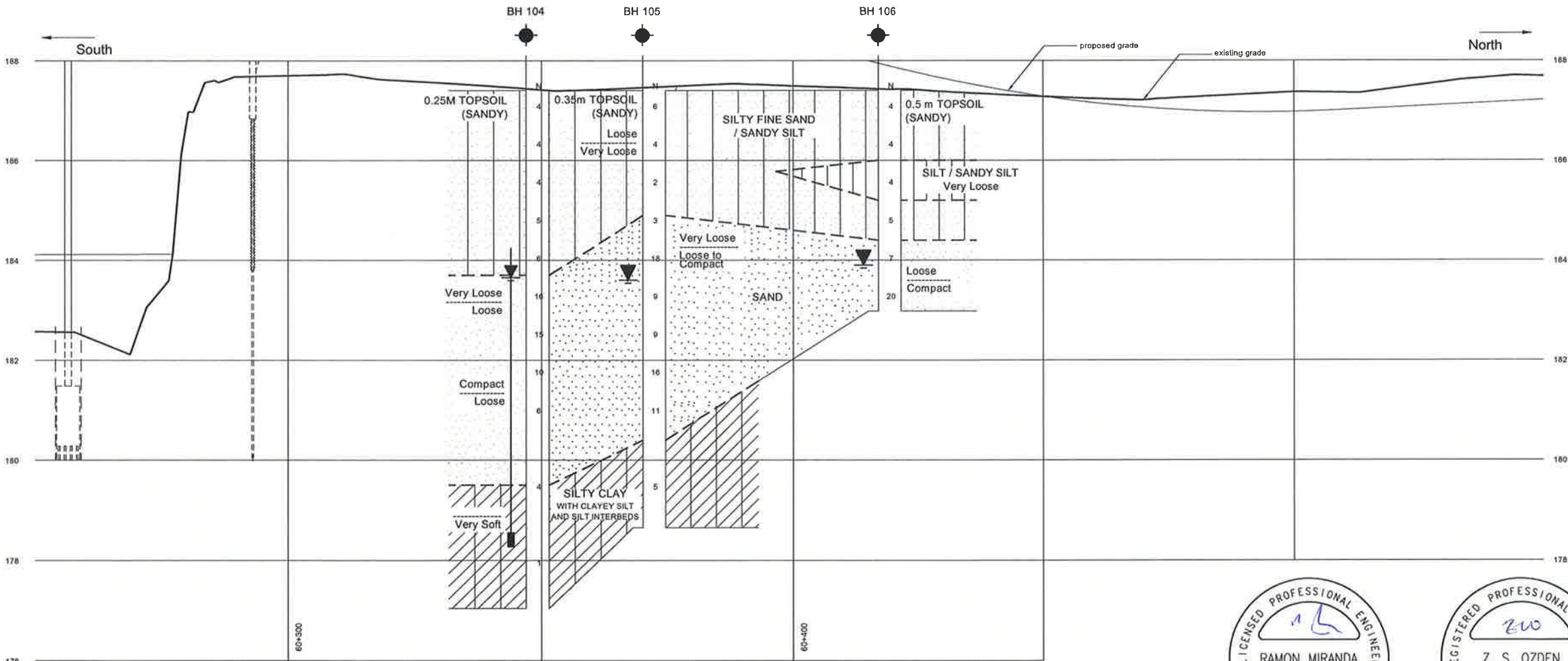
NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 41K-82				
TRANETO01156AA				DIST
SUBM'D	CHECKED	DATE	Dec 8, 2009	SITE 385-031
DRAWN PHK	CHECKED RM	APPROVED	ZO	DWG 3

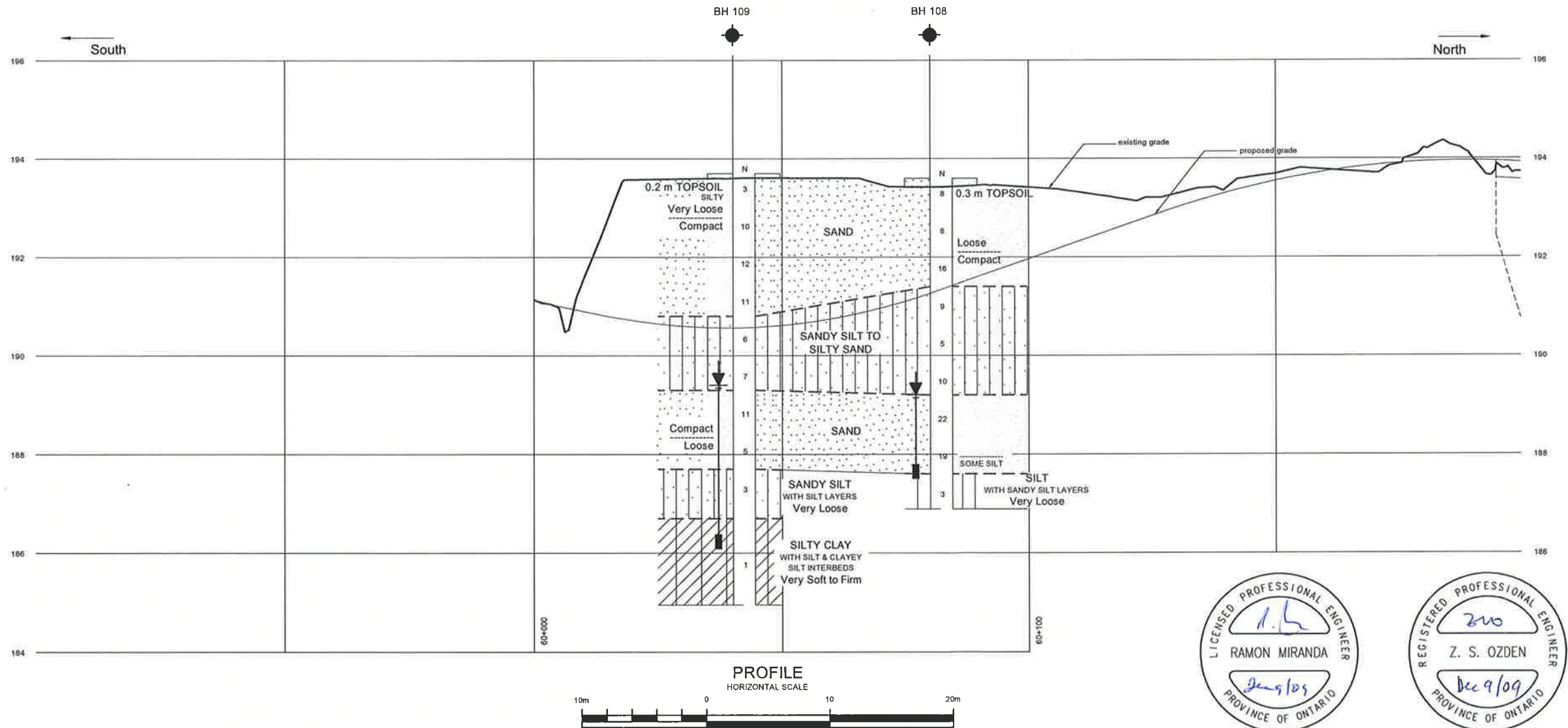
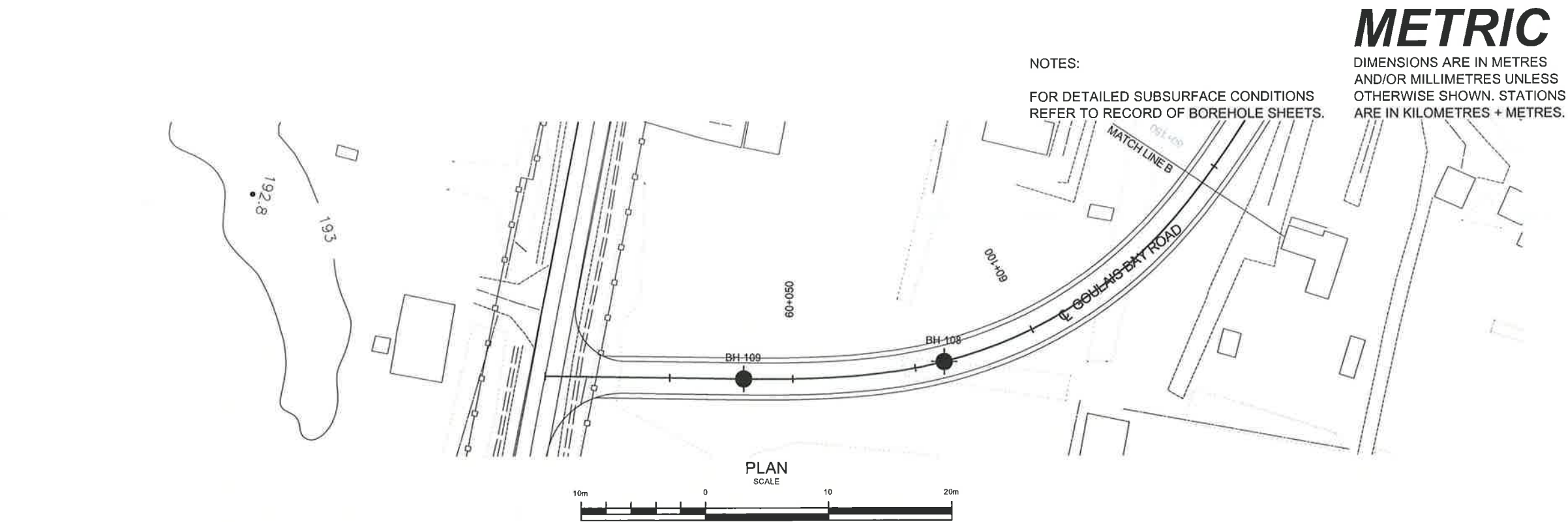


PLAN
SCALE



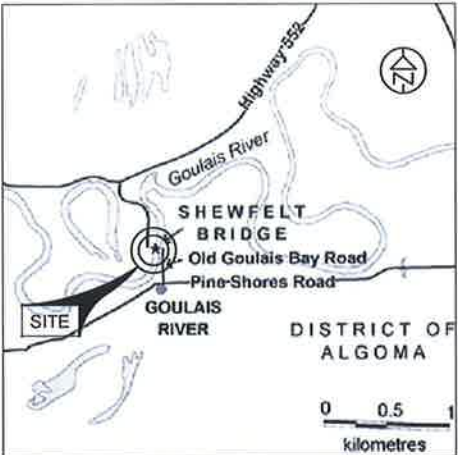
PROFILE
HORIZONTAL SCALE





CONT No.	SHEET
GWP: 5290-04-00	
OLD GOULAIS BAY ROAD SHEWFELT BRIDGE BOREHOLE LOCATION PLAN AND SOIL STRATA	

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

No.	ELEVATION	NORTHING	EASTING
BH 108	193.6m	5175847.1	275713.9
BH 109	193.7m	5175808.2	275726.4

-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 41K-82			
TRANETO01156AA			
SUBMD	CHECKED	DATE Dec 8, 2009	DIST SITE 38S-031
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 4



Appendix A

Record of Borehole Sheets

Previous Foundation Investigation Report (2009)

RECORD OF BOREHOLE No BH1

1 OF 4

METRIC

GWP 5290-04-00 LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 914.4; E 275 624.7 ORIGINATED BY G.J.
DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT COMPILED BY J.Z.
DATUM Geodetic DATE 2/28/2006 CHECKED BY KSH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								20	40	60		
193.3 0.0	Ground Surface											
192.7 0.7	250mm TOPSOIL SILTY FINE SAND with trace rootlets & topsoil dark brown, loose, moist (frozen)		1	SS	8							
			2	SS	10							
	SAND with trace gravel, trace topsoil pockets brown, loose, moist		3	SS	5							
			4	SS	5							
			5	SS	10							
			6	SS	6							
188.8 4.6			7	SS	7							
	SILTY SAND to SANDY SILT with trace gravel brown loose, wet		8	SS	6							
			9	SS	7							
	trace medium sand seam											
184.3 8.0			10	SS	1							
	CLAYEY SILT with trace gravel grey, firm to stiff		11	SS	3							
			12	TW	PH							
179.6 13.7			13	SS	2							
	SILTY CLAY reddish grey, firm											

Continued Next Page

+³, x³: Numbers refer to Sensitivity
20
15 10 5 (%) STRAIN AT FAILURE

50mm diameter
Shelby tube
sample
(0) 79 21

RECORD OF BOREHOLE No BH1

2 OF 4

METRIC

GWP 5290-04-00

LOCATION Shewell Bridge, Goulais River --Coords: N 5 175 914.4; E 275 624.7

ORIGINATED BY G.I.

DIST Algoma HWY 17

BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.Z.

DATUM Geodetic

DATE 2/20/2006

CHECKED BY KSH

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 Y 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	WATER CONTENT (%) 20 40 60					
	occ. clayey silt seams		14	SS	1		176							
							177							
							176							
	SILTY CLAY reddish grey, firm to stiff	firm stiff	15	SS	0		175							
							174							
							173							
							172							N-value not reliable
							171							
							170							
							169							
			17	TW	PH		168						16.6	70mm diameter Shelby tube sample (0) 34 66
							167							
							166							N-value not reliable
			18	SS	0		165							
							164							

Continued Next Page

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

SPT1156A

RECORD OF BOREHOLE No BH1

3 OF 4

METRIC

GWP 5290-04-00

LOCATION Shewell Bridge, Goulais River - Coords: N 5 175 914.4; E 275 624.7

ORIGINATED BY G.I.

DIST Algoma HWY 17

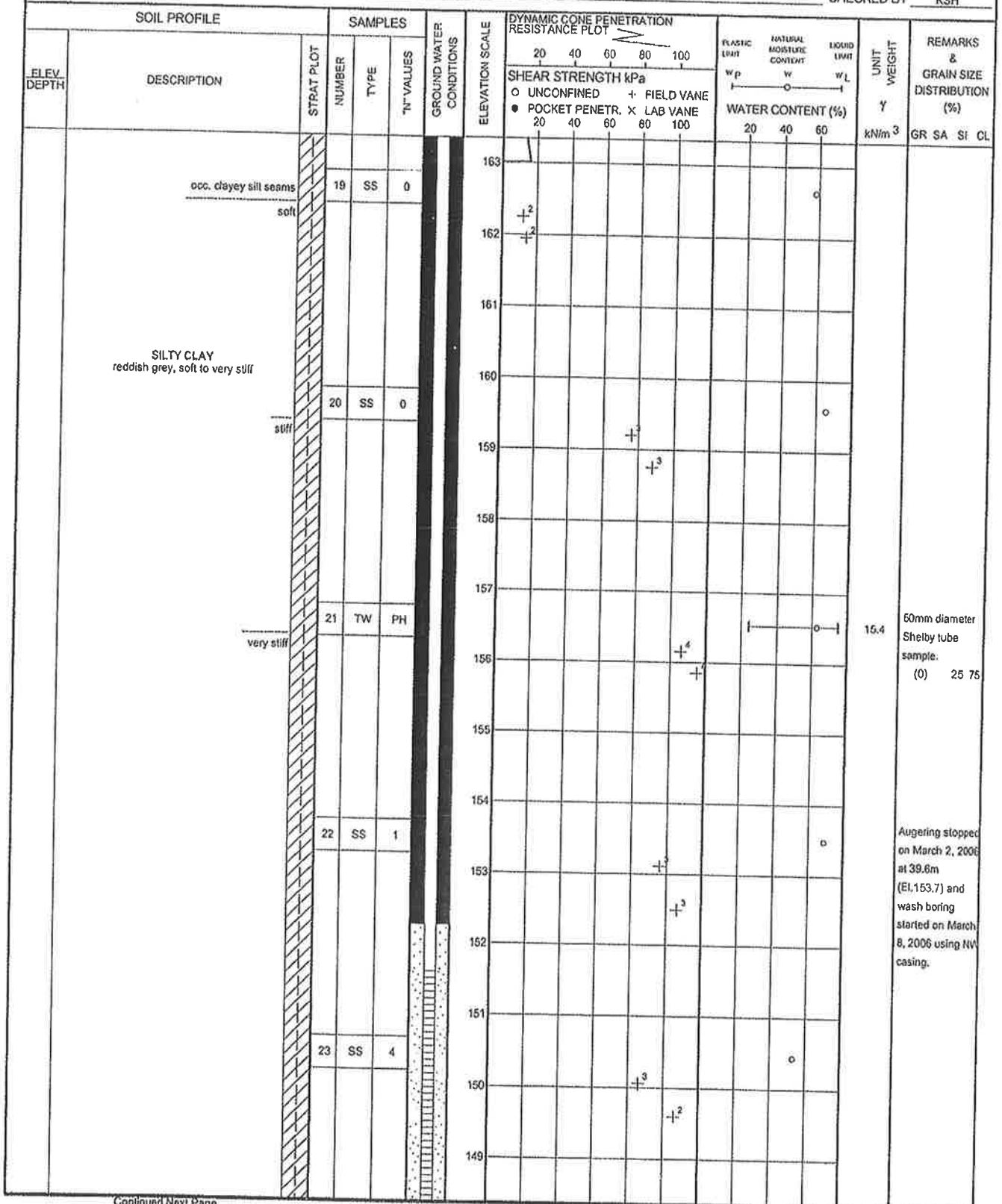
BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.2.

DATUM Geodetic

DATE 2/28/2006

CHECKED BY KSH



Continued Next Page

+³, x³: Numbers refer to Sensitivity 20 15-0.5 10 (%) STRAIN AT FAILURE

SPT1156A

RECORD OF BOREHOLE No BH1

4 OF 4

METRIC

GWP 5290-04-00

LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 914.4; E 275 624.7

ORIGINATED BY G.I.

DIST Algoma HWY 17

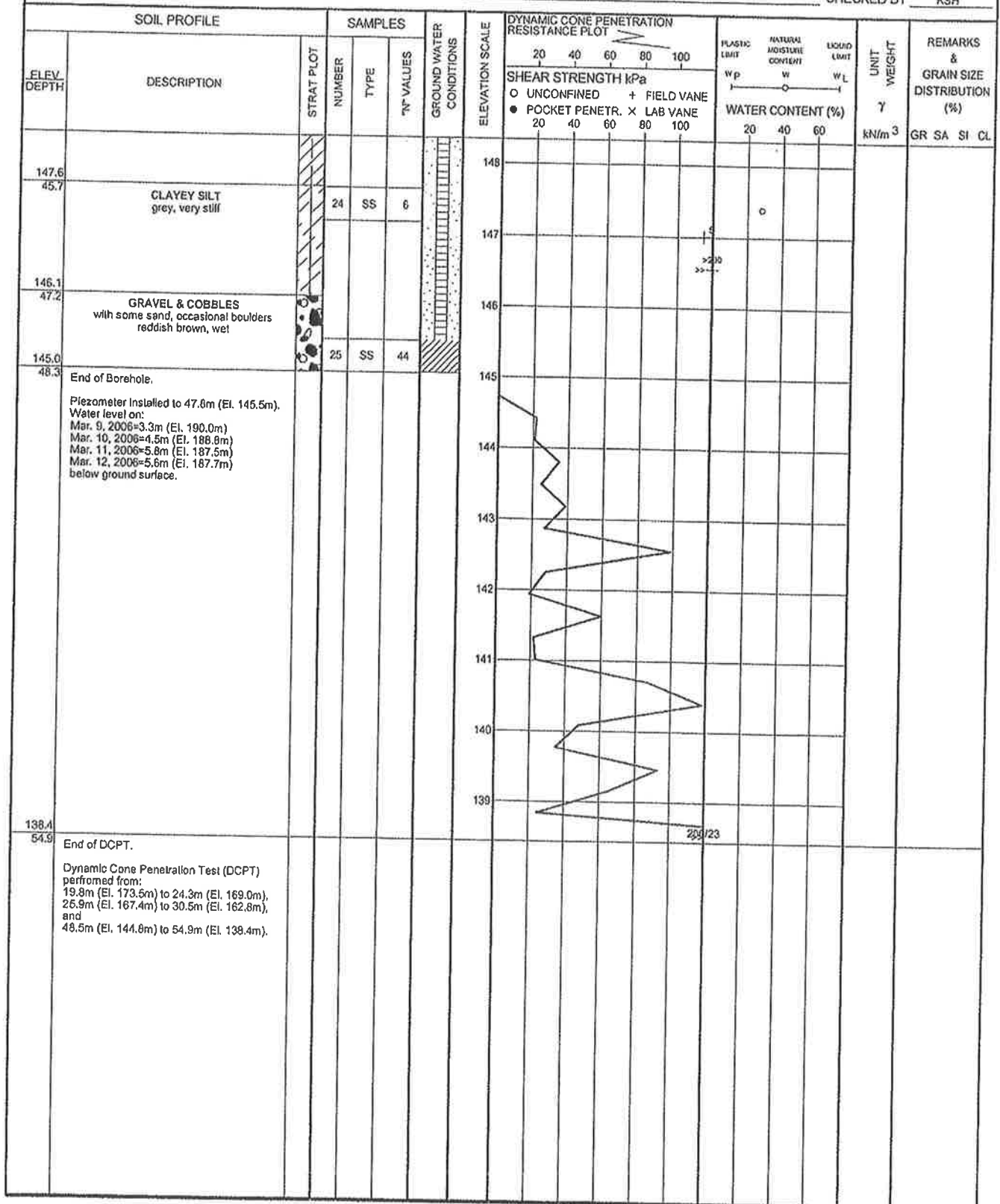
BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.Z.

DATUM Geodetic

DATE 2/28/2006

CHECKED BY KSH



+³, ×³: Numbers refer to Sensitivity
20
15-0-5
10 (%) STRAIN AT FAILURE

SPT1156A

RECORD OF BOREHOLE No BH2

1 OF 3

METRIC

GWP 5290-04-00

LOCATION Shewfelt Bridge, Goulais River - Coords: N 5 175 950.6; E 275 523.1

ORIGINATED BY G.I.

DIST Algoma HWY 17

BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.Z.

DATUM Geodetic

DATE 3/10/2006

CHECKED BY KSH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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. × LAB VANE					
187.1 0.0	Ground Surface													
186.3 0.8	250mm TOPSOIL SILTY FINE SAND with trace topsoil & rootlets brown, very loose, moist		1	SS	4									
	SANDY SILT brown, very loose, moist		2	SS	3									
			3	SS	2									
			4	SS	3									
			5	SS	4									
			6	SS	3									
182.6 4.6	SAND with trace gravel brown, loose, wet		7	SS	5									
			8	SS	10									
			9	SS	7									
179.5 7.6	CLAYEY SILT grey, stiff		10	SS	4									
178.0 9.2		SILTY CLAY reddish grey, firm		11	SS	0								
			12	TW	PH									
			13	SS	1									
			14	SS	1									

Continued Next Page

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

50 mm diameter
Shelby tube
sample.
2 2 64 32

SPT1156A

RECORD OF BOREHOLE No BH2

2 OF 3

METRIC

GWP 5290-04-00

LOCATION Shewfelt Bridge, Goulais River -Coords: N 5 175 950.6; E 275 623.1

ORIGINATED BY G.I.

DIST Algoma HWY 17

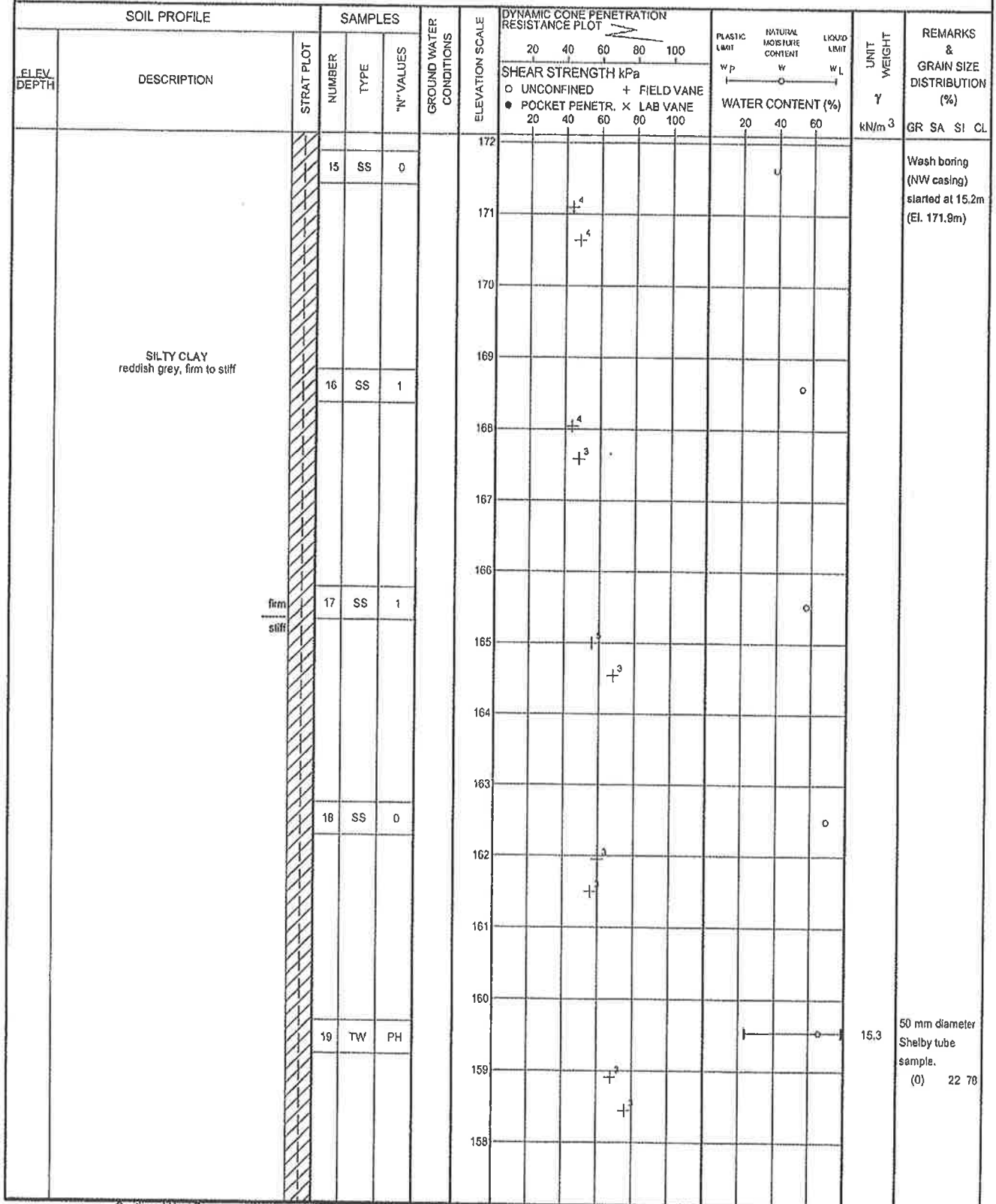
BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.Z.

DATUM Geodetic

DATE 3/10/2006

CHECKED BY KSH



Continued Next Page

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

SPT1156A

RECORD OF BOREHOLE No BH2

3 OF 3

METRIC

GWP 5290-04-00 LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 950.6; E 275 523.1 ORIGINATED BY G.I.
DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT COMPILED BY J.Z.
DATUM Geodetic DATE 3/10/2006 CHECKED BY KSH

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. × LAB VANE	WATER CONTENT (%)					
							20 40 60 80 100	20 40 60						
			20	SS	2									
			21	SS	1									
			22	TW	PH									
147.5 39.6	CLAYEY SILT gray, very stiff		23	SS	6									
146.6 40.5	GRAVEL and COBBLES with sand infill, occasional boulders reddish brown, wet													
145.5 41.6	End of borehole.		24	SS	50/8									
	Dynamic Cone Penetration Test carried out from 40.2m (El. 146.9m) to 41.6m (El. 145.6m). * Water level in open borehole: 3.4m (not stabilized).													

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

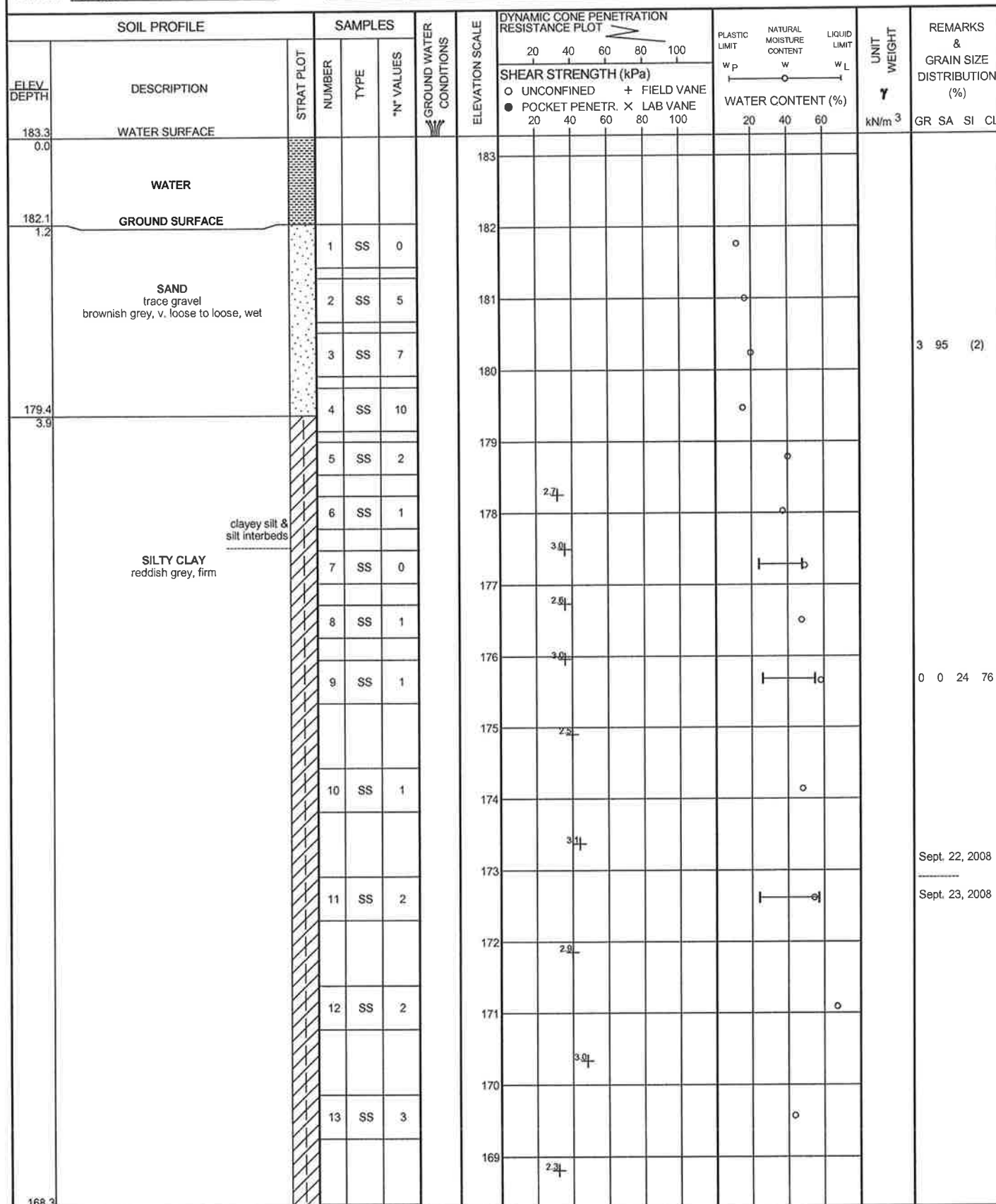
SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 101

1 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+258, N: 5175927.45 E: 275561.25 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE NW Casing and Wash Boring COMPILED BY SS
DATUM Geodetic DATE 22/09/2008 11/10/2008 CHECKED BY ZO



Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

2 OF 4

METRIC

DATUM Geodetic DATE 22/09/2008 11/10/2008 CHECKED BY ZO

[illegible]

coffey geotechnics
SPECIALISTS MANAGING THE EARTH




SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 101

3 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+258, N: 5175927.45 E: 275561.25 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE NW Casing and Wash Boring COMPILED BY SS
DATUM Geodetic DATE 22/09/2008 11/10/2008 CHECKED BY ZO

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. × LAB VANE							WATER CONTENT (%)																													
153.3 30.0	SILTY CLAY reddish grey stiff to v. stiff		21	SS	10	153	20	40	60	80	100	20	40	60	182	152	151	150	149	148	147	146	145	144	143	142	141	140	139	138														
149.3 34.0			22	SS	22																										151	150	149	148	147	146	145	144	143	142	141	140	139	138
146.3 37.0	SILTY SAND with gravel, occ. cobbles and boulders (possible till) reddish grey, compact to v. dense, wet		23	SS	100/2 cm	145	20	40	60	80	100	20	40	60	163	144	143	142	141	140	139	138	137	136	135	134	133	132	131	130														
143.3 40.0			24	RC																											143	142	141	140	139	138	137	136	135	134	133	132	131	130
			25	SS	85																										143	142	141	140	139	138	137	136	135	134	133	132	131	130
140.1 43.2	GRAVEL AND COBBLES with sand infill, some boulders reddish brown, wet		26	SS	40	142	20	40	60	80	100	20	40	60	192	141	140	139	138	137	136	135	134	133	132	131	130	129	128	127														
138.3			27	SS	100/5 cm																										140	139	138	137	136	135	134	133	132	131	130	129	128	127
			28	RC		139	138	137	136	135	134	133	132	131	130	129	128	127	126	125	124	123	122	121	120	119	118	117	116	115	114													

Dynamic Cone Penetration Test (DCPT) performed from 38.4 to 40.8 m when Casing is @ 37.8 m.

DCPT performed from 41.5 m to 43.9 m

Oct. 7, 2008

Continued Next Page

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

Dynamic Cone Penetration Test (DCPT) performed from 38.4 to 40.8 m when Casing is @ 37.8 m.

DCPT performed from 41.5 m to 43.9 m

Oct. 7, 2008


SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 101

4 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+258, N: 5175927.45 E: 275561.25 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE NW Casing and Wash Boring COMPILED BY SS
DATUM Geodetic DATE 22/09/2008 11/10/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. x LAB VANE			WATER CONTENT (%) w _p w w _L				
138.3 45.0	GRAVEL AND COBBLES with sand infill, some boulders reddish brown, wet					138								Artesian condition @ El. 138.1 m 3.3 m above ground (to El. 185.4 m)	
137.3 46.0			29	SS	100/10 cm										
137.0 46.3			METAMORPHISED SANDSTONE (possible bedrock) reddish brown	30	RCTCR=100%			137							
	METAMORPHISED SANDSTONE (probable bedrock) reddish brown	31	SS RQD=48% 100/2 cm												
			32	RCTCR=100% RQD=72%		136									
135.4 47.9	End of Borehole.													Artesian condition @ bottom of borehole 5.5 m above ground (to El. 187.6 m)	

+³, ×³: Numbers refer to Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102

1 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+295, N:5175938.23 275525.92 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger, Wash Boring & Rock Coring COMPILED BY SS
DATUM Geodetic DATE 03/09/2008 10/09/2008 CHECKED BY ZO

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)						
187.7	GROUND SURFACE													
0.0	SILTY FINE SAND organic stained to 0.4 m brown, v. loose, moist		1	SS	4									
187.0			2	SS	2									
0.7	SANDY SILT TO SILTY SAND brown, v. loose, moist		3	SS	3									0 33 54 13
			4	SS	4									
			5	SS	4									
183.6		sand seams	6	SS	5									
4.1	SAND trace gravel wet		7	SS	7									sampler wet @ 4.5 m
		brown loose	8	SS	11									17 80 (3)
		compact grey	9	SS	12									
180.7			10	SS	3									
7.0			11	TW	PM									H/S augering
	SILTY CLAY reddish grey, firm		12	SS	2								18.3	NW casing wash boring consolidation test
		clayey silt & silt interbeds	13	SS	2									0 1 32 67
			14	SS	2									
172.7														

Continued Next Page

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

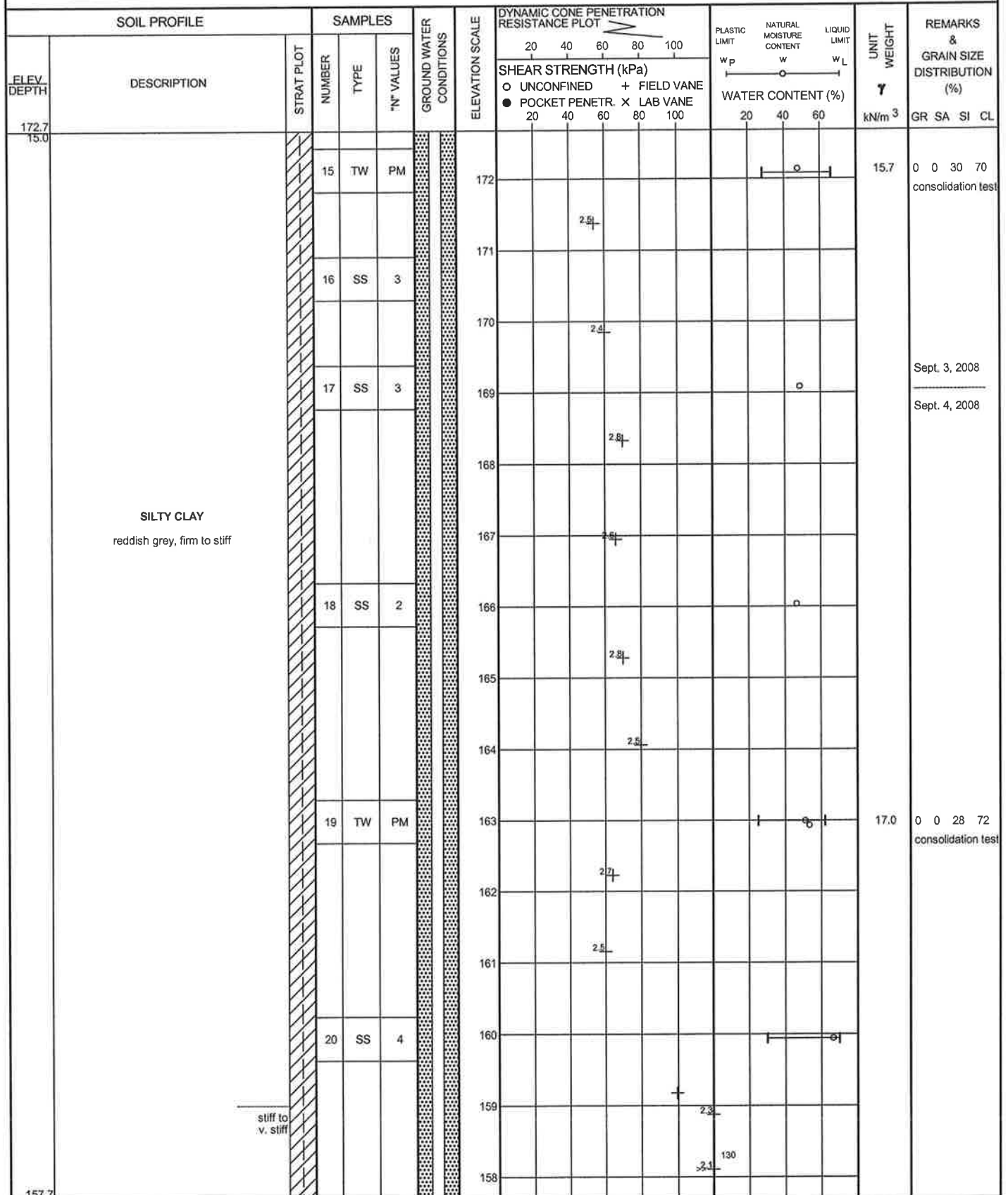
SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102

2 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+295, N:5175938.23 275525.92 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger, Wash Boring & Rock Coring COMPILED BY SS
DATUM Geodetic DATE 03/09/2008 10/09/2008 CHECKED BY ZO



Continued Next Page

+ 3, X 3; Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE





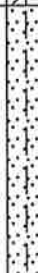
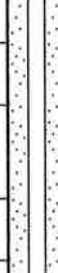
SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102

3 OF 4

METRIC

GWP	5290-04-00	LOCATION	Sta. 60+295, N:5175938.23 275525.92	ORIGINATED BY	GI
DIST	HWY 17	BOREHOLE TYPE	Hollow Stem Auger, Wash Boring & Rock Coring	COMPILED BY	SS
DATUM	Geodetic	DATE	03/09/2008 10/09/2008	CHECKED BY	ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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 (%)	
							<div><div></div><div></div><div></div><div></div><div></div></div>				<div><div></div><div></div><div></div></div>						
157.7 30.0	SILTY CLAY reddish grey stiff to v. stiff		21	SS	6		157										
								156					144				
								155					168				
					22		SS	7	154								
									153					144			
									152					192			
					23		SS	8	151								
									150								
148.4 39.3	CLAYEY SILT with silt seams grey, stiff to v. stiff wet, dilatant		24	SS	11		149										
								148									
145.4 42.3	SILTY SAND with gravel occ. cobbles and boulders (possible till) reddish grey, compact to dense, wet		25	SS	36		147										
								146									
								145									
					26		SS	-	144								
142.7																	

1.5m sand
back-up in casing
Sept. 4, 2008
34 44 16 6

Sept. 5, 2008
Dynamic Cone
Penetration Test
(DCPT) performed
from 43.3m to
54.1m.
40 38 16 6

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

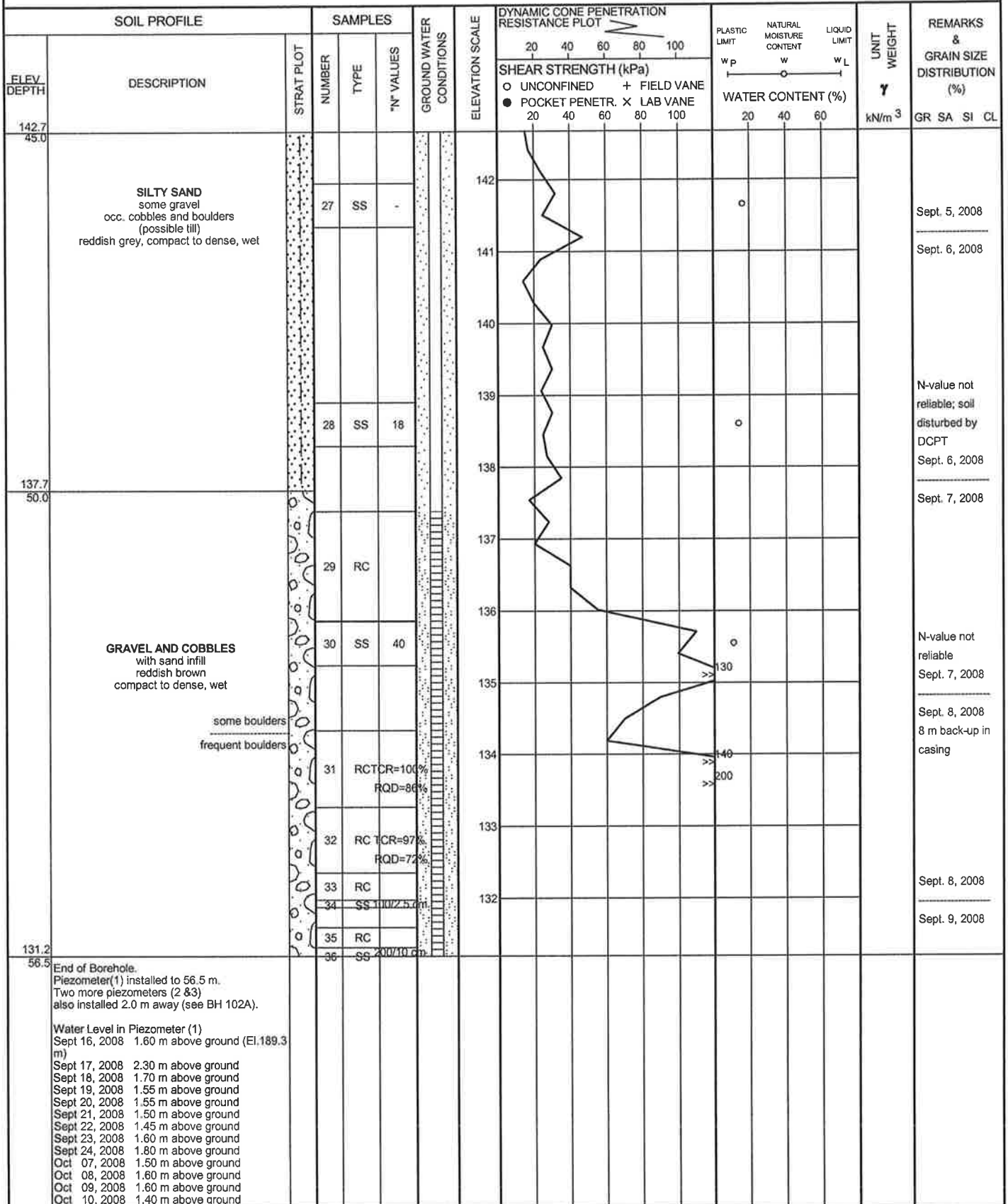
SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102

4 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+295, N:5175938.23 275525.92 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger, Wash Boring & Rock Coring COMPILED BY SS
DATUM Geodetic DATE 03/09/2008 10/09/2008 CHECKED BY ZO



+³, X³: Numbers refer to Sensitivity 20 15 10 (% STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102A

1 OF 2

METRIC

GWP 5290-04-00

LOCATION

Sta. 60+295 : 2m away from BH 102

ORIGINATED BY GI

DIST _____ HWY 17

BOREHOLE TYPE

Hollow Stem Auger

COMPILED BY SS

DATUM Geodetic

DATE _____

9/3/2008	9/10/2008
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CHECKED BY ZO

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+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102A

2 OF 2

METRIC

GWP 5290-04-00 LOCATION Sta. 60+285 : 2m away from BH 102 ORIGINATED BY GI
 DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
 DATUM Geodetic DATE 9/3/2008 9/10/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
172.7	Oct 08, 2008 4.20 m Oct 09, 2008 4.20 m Oct 10, 2008 4.20 m																

+³ ×³ Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 103

1 OF 2

METRIC

GWP 5290-04-00 LOCATION Sta. 60+324, N:5175948.19 E:275498.72 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 17/09/2008 CHECKED BY ZO

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)						
187.6 0.0	GROUND SURFACE							20 40 60 80 100						
	0.3 m TOPSOIL (Sandy) SILTY SAND/SANDY SILT some topsoil inclusions (previously disturbed) brown, v.loose, moist		1	SS	5		187							
			2	SS	2									
			3	SS	2		186							
185.5 2.1	SANDY SILT/SILTY SAND brown, v.loose, moist upto 3.3 m, wet below		4	SS	4		185							0 37 51 12
			5	SS	4		184							
183.6 4.0	SAND wet		6	SS	12		183							
	brown compact		7	SS	18		182							10 87 (3)
	grey loose		8	SS	9		181							
			9	SS	9		180							
180.3 7.3			10	TW	PM		179	23						
			11	SS	2		178							
	SILTY CLAY reddish grey, soft to firm		12	TW	PM		177	24						
	frequent silt & clayey silt interbeds		13	SS	2		176	25						
			14	SS	1		175							
							174	23						
172.6							173	20						

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+³ . ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE


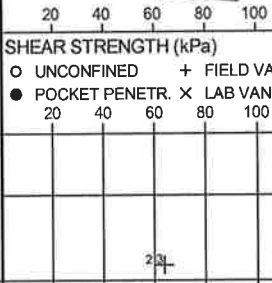
SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 103

2 OF 2

METRIC

GWP 5290-04-00 LOCATION Sta. 60+324, N:5175948.19 E:275498.72 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 17/09/2008 CHECKED BY ZO

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)						
172.6 15.0	SILTY CLAY reddish grey, firm to stiff		15	SS	2	172								
171.2 16.5														
End of Borehole. Water level @ 3.6 m upon completion.														

+³, X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 104

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+347, N:5175959.35 E:275478.64 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 17/09/2008 CHECKED BY ZO

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)							
187.4 0.0	GROUND SURFACE						20	40	60	80	100				
	0.25 m TOPSOIL (Sandy)		1	SS	4										
	SILTY FINE SAND/SANDY SILT brown, moist		2	SS	4										
			3	SS	4										
			4	SS	5										
		v.loose													
		loose	5	SS	6										
183.7 3.7			6	SS	16										
	SAND wet		7	SS	15										
		brown compact	8	SS	10										
		loose grey	9	SS	6										
		v.loose													
179.5 7.9			10	SS	4										
	SILTY CLAY with silt and frequent clayey silt interbeds reddish grey, soft to firm		11	SS	1										
177.0 10.4	End of Borehole: Piezometer installed to 9.1 m Water Level in Piezometer Sept 18, 2008 6.2 m Sept 19, 2008 5.5 m Sept 20, 2008 4.9 m Sept 21, 2008 4.3 m Sept 22, 2008 4.1 m Sept 23, 2008 3.8 m Sept 24, 2008 3.8 m Oct 07, 2008 3.8 m Oct 08, 2008 3.8 m Oct 09, 2008 3.75 m Oct 10, 2008 3.75 m														

+ 3, X 3: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 105

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+370, N:5175974.99 E:275461.86 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 18/09/2008 CHECKED BY ZO

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)							
187.4 0.0	GROUND SURFACE							20	40	60	80	100			
	0.35 m TOPSOIL(Sandy)	loose	1	SS	6		187								
		v. loose	2	SS	4		186								
	SILTY FINE SAND/SANDY SILT brown, damp to moist		3	SS	2		185								0 49 37 14
184.9 2.5			4	SS	3		184								
		v. loose	5	SS	18		183								
	SAND	loose to compact	6	SS	9		182								
	moist to 3.5 m, wet below		7	SS	9		181								7 88 (5)
		brown	8	SS	16		180								
		grey	9	SS	11		179								
180.4 7.0							180								
	SILTY CLAY		10	SS	5		179								
	with clayey silt and silt interbeds reddish grey, firm														
178.6 8.8	End of Borehole. Water level @ 3.8 m upon completion (not stabilized)*														

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 106

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+417, N:5176017.18 E:275442.41 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Solid Stem Auger COMPILED BY SS
DATUM Geodetic DATE 18/09/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
								20 40 60 80 100						
187.4	GROUND SURFACE													
0.0	0.5 m TOPSOIL (Sandy)		1	SS	4		187							
	SILTY FINE SAND brown, v.loose, damp		2	SS	4		186							
186.0														
1.4	SILT/SANDY SILT brown, v.loose, moist		3	SS	4		185							
185.2			4	SS	5		184							
2.2	SILTY FINE SAND brown, loose, moist													
184.4			5	SS	7		184							
3.0	SAND moist to 3.6 m wet below		6	SS	20		183							
183.0														
4.4	End of Borehole. Water level @ 3.5 m and hole caved @ 3.5 m upon completion (not stabilized)*													

+³.X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 107

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+188, N:5175904.27 E:275627.39 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Solid Stem Auger COMPILED BY SS
DATUM Geodetic DATE 20/09/2008 CHECKED BY ZO

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 (%)
194.2	GROUND SURFACE							20 40 60 80 100								
0.0	0.15 m SAND & GRAVEL some organics		1	SS	24	☒	194								9 89 (2)	
193.4	FILL: SILTY FINE SAND brown, compact moist															
0.8	SAND brown, damp to moist		2	SS	19		193									
			3	SS	10		192									
		compact		4	SS		7	191								
		loose		5	SS		27	190								
		silty fine sand compact		6	SS		27	189								
189.8	SANDY SILT TO SILTY SAND wet		7	SS	8		☒	188								1 95 (4)
4.4			8	SS	24											
		loose		9	SS	22										
		compact brown														
	grey															
187.5																
6.7	End of Borehole. Water level @ 4.2 m upon completion (not stabilized)*															

+³ . X³: Numbers refer to Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 108

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+080, N:5175847.1 E:275713.92 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 21/09/2008 CHECKED BY ZO

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)							
						20 40 60 80 100			20 40 60						
						○ UNCONFINED + FIELD VANE									
						● POCKET PENETR. x LAB VANE									
193.6	GROUND SURFACE														
0.0	0.3 m TOPSOIL		1	SS	8		193								
	SAND		2	SS	8										
	brown, dry to moist														
	loose														
	compact		3	SS	16		192								
191.4															
2.2	SANDY SILT TO SILTY SAND		4	SS	9		191								
	loose, wet														
	brown		5	SS	5										
	grey														
			6	SS	10		190								
189.2															
4.4	SAND		7	SS	22		189								
	grey, compact														
	wet														
	some silt		8	SS	19		188								
187.6															
6.0	SILT with sandy silt layers		9	SS	3										
	grey, v.loose														
	wet, dilatant														
186.9															
6.7	End of Borehole.														
	Water level @ 4.5 m on completion.														
	Piezometer installed to 6.0 m														
	Water Level in Piezometer														
	Sept 22, 2008 4.4 m														
	Sept 23, 2008 4.4 m														
	Sept 24, 2008 4.4 m														
	Oct 07, 2008 4.4 m														
	Oct 08, 2008 4.4 m														
	Oct 09, 2008 4.4 m														
	Oct 10, 2008 4.4 m														

+ 3, X 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 109

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+040, N:5175808.21 E:275726.41 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 21/09/2008 CHECKED BY ZO

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)						
193.7	GROUND SURFACE							20 40 60 80 100						
0.0	0.2 m TOPSOIL 0.15 m SANDY SILT org. stained	v. loose silty	1	SS	3			○ UNCONFINED + FIELD VANE						
		compact	2	SS	10			● POCKET PENETR. X LAB VANE						7 91 (2)
	SAND brown, compact, damp to moist		3	SS	12									
			4	SS	11									
190.8			5	SS	6									
2.9	SANDY SILT with silt layers grey, loose, wet, dilatant		6	SS	7									
			7	SS	11									
189.3			8	SS	5									
4.4	SAND brown, wet	compact	9	SS	3									
		loose	10	SS	1									0 95 (5)
187.7														
6.0	SANDY SILT with silt layers grey, v. loose, wet, dilatant													
186.7														
7.0	SILTY CLAY with silt & clayey silt interbeds reddish grey (clay), grey (silt) very soft to firm													
184.9														
8.8	End of Borehole. Piezometer installed to 7.6 m Water Level in Piezometer Sept 22, 2008 6.2 m Sept 23, 2008 5.4 m Sept 24, 2008 4.8 m Oct 07, 2008 4.3 m Oct 08, 2008 4.3 m Oct 09, 2008 4.3 m Oct 10, 2008 4.3 m													

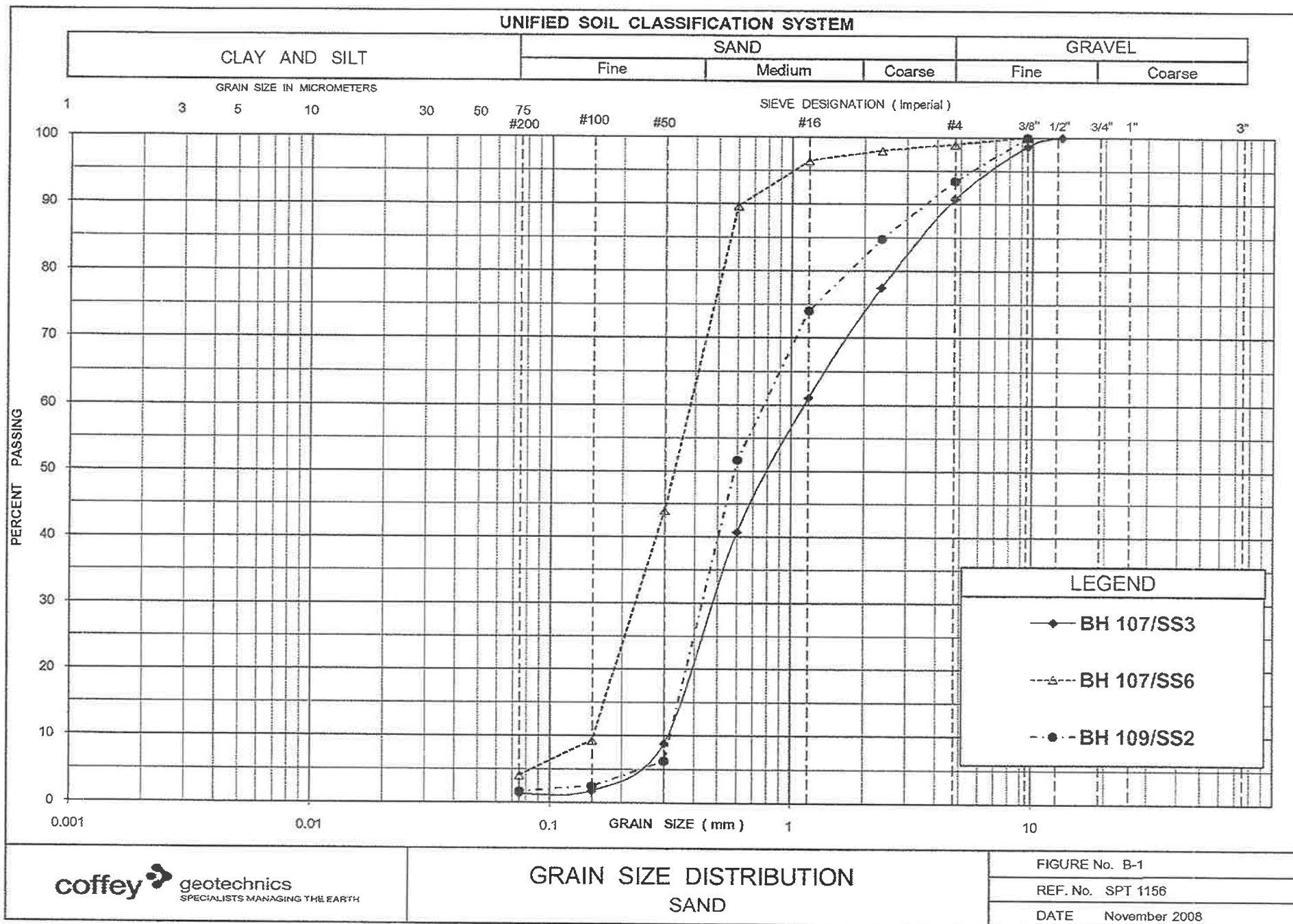
+³, ×³: Numbers refer to
Sensitivity

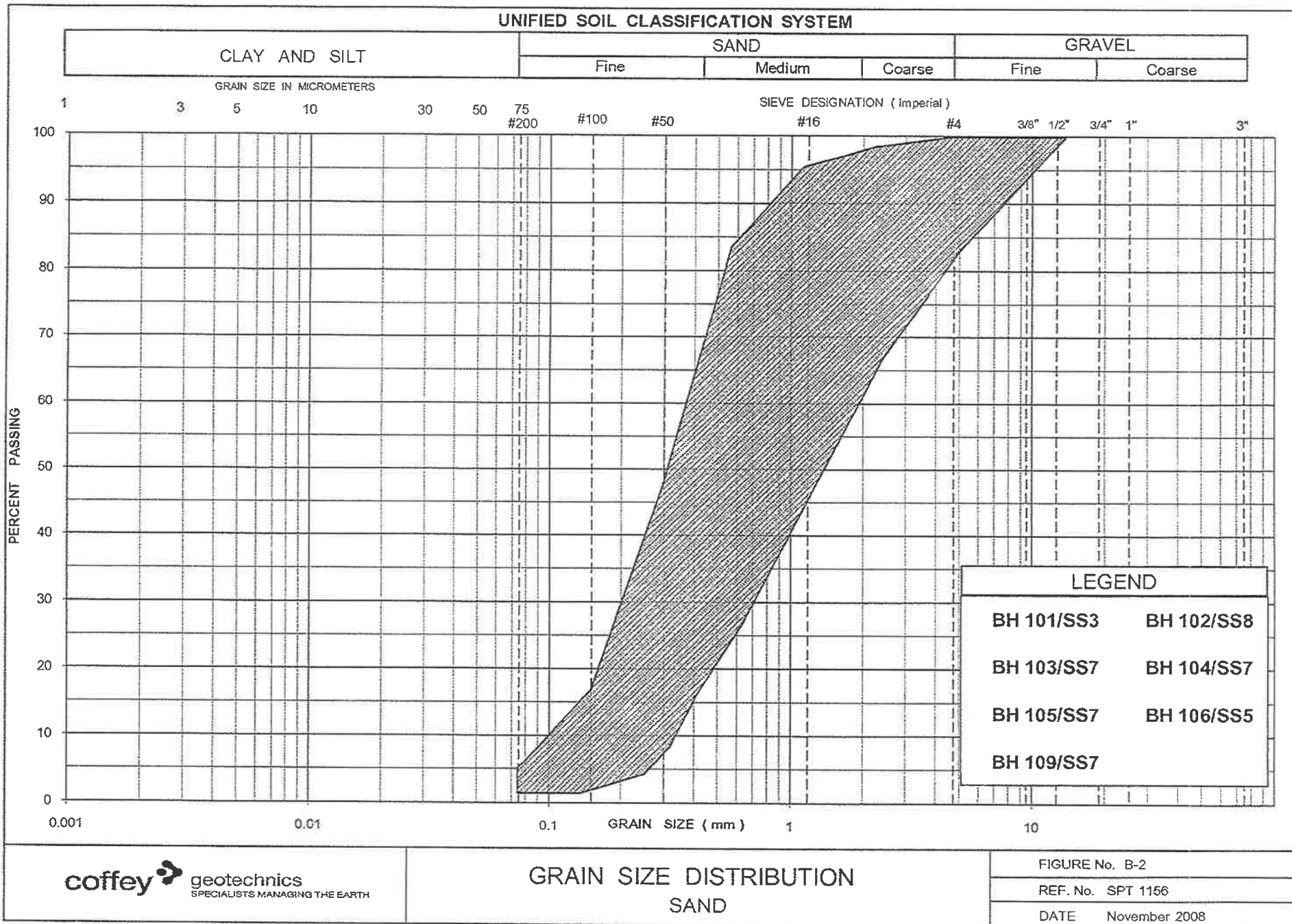
20
15 10 5
(%) STRAIN AT FAILURE

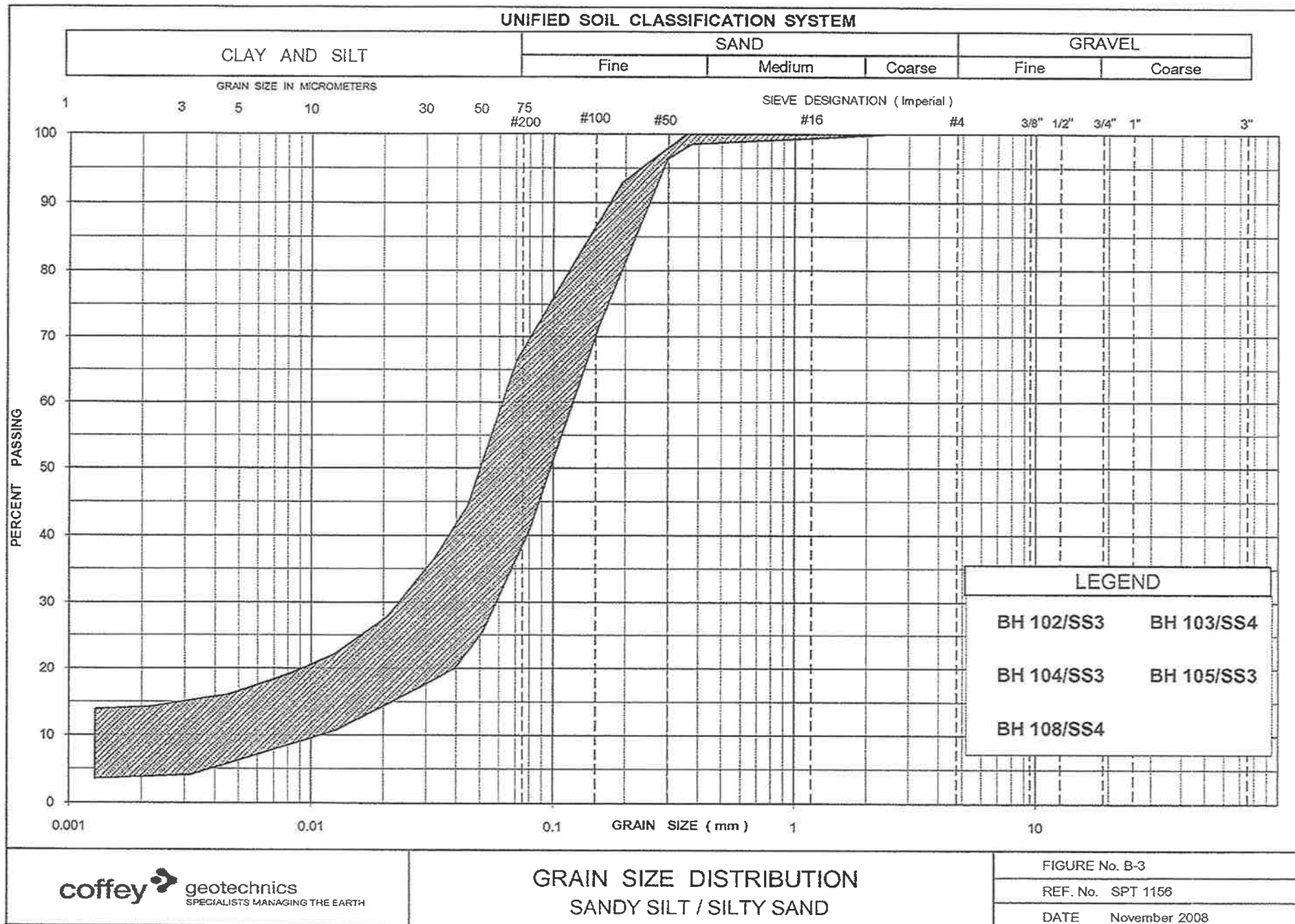
Appendix B

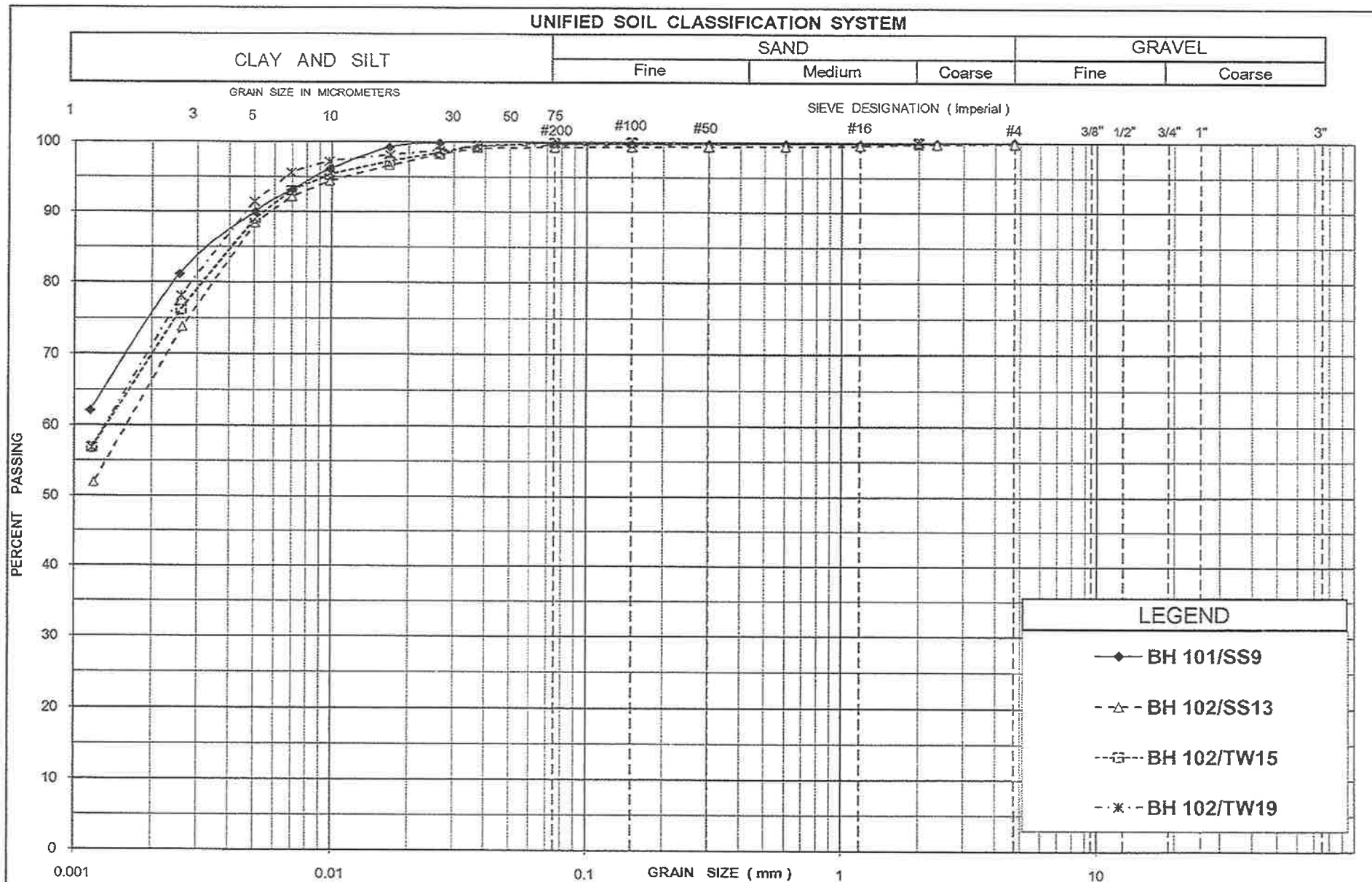
Laboratory Test Results

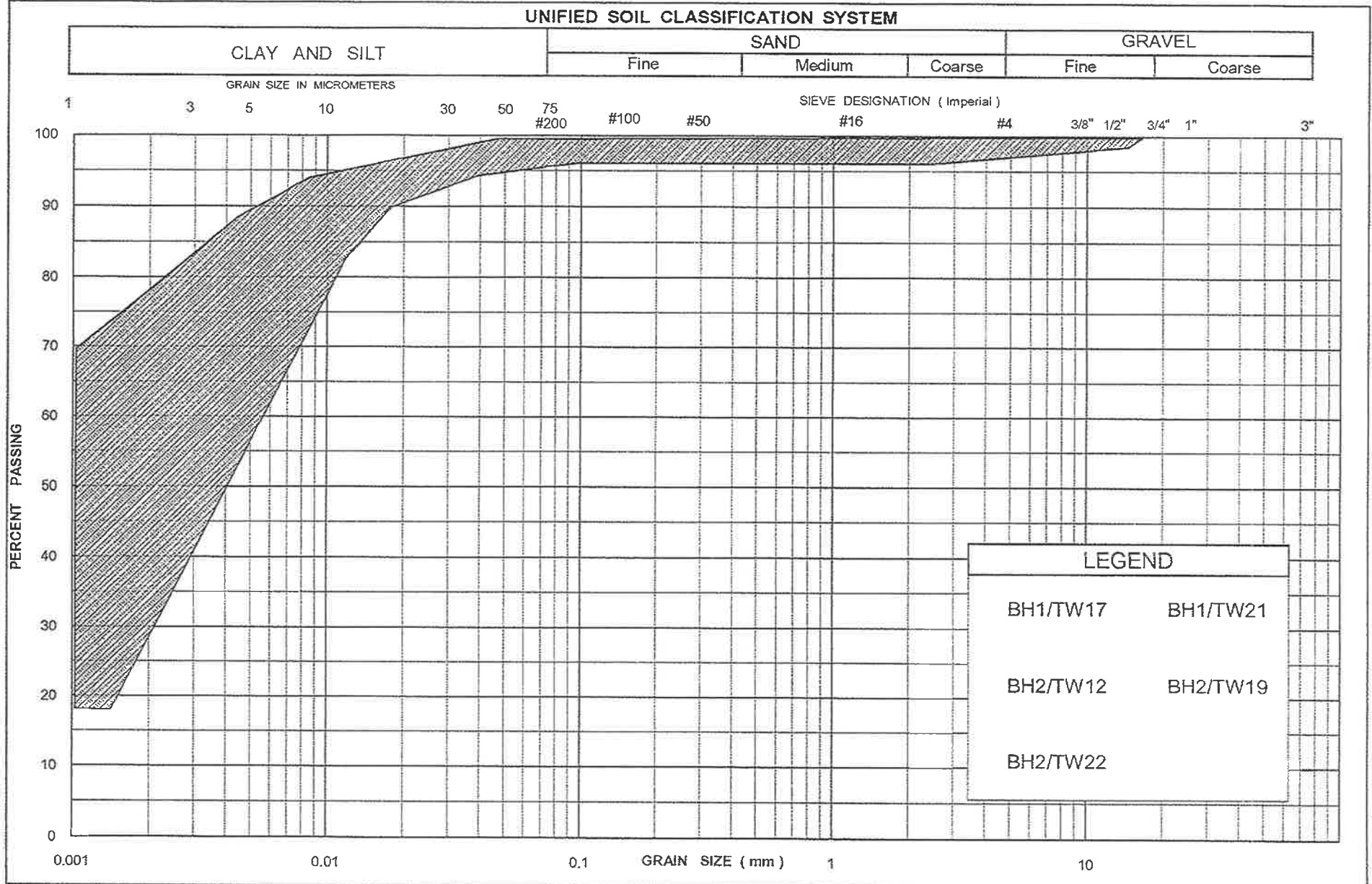
Previous Foundation Investigation Report (2009)

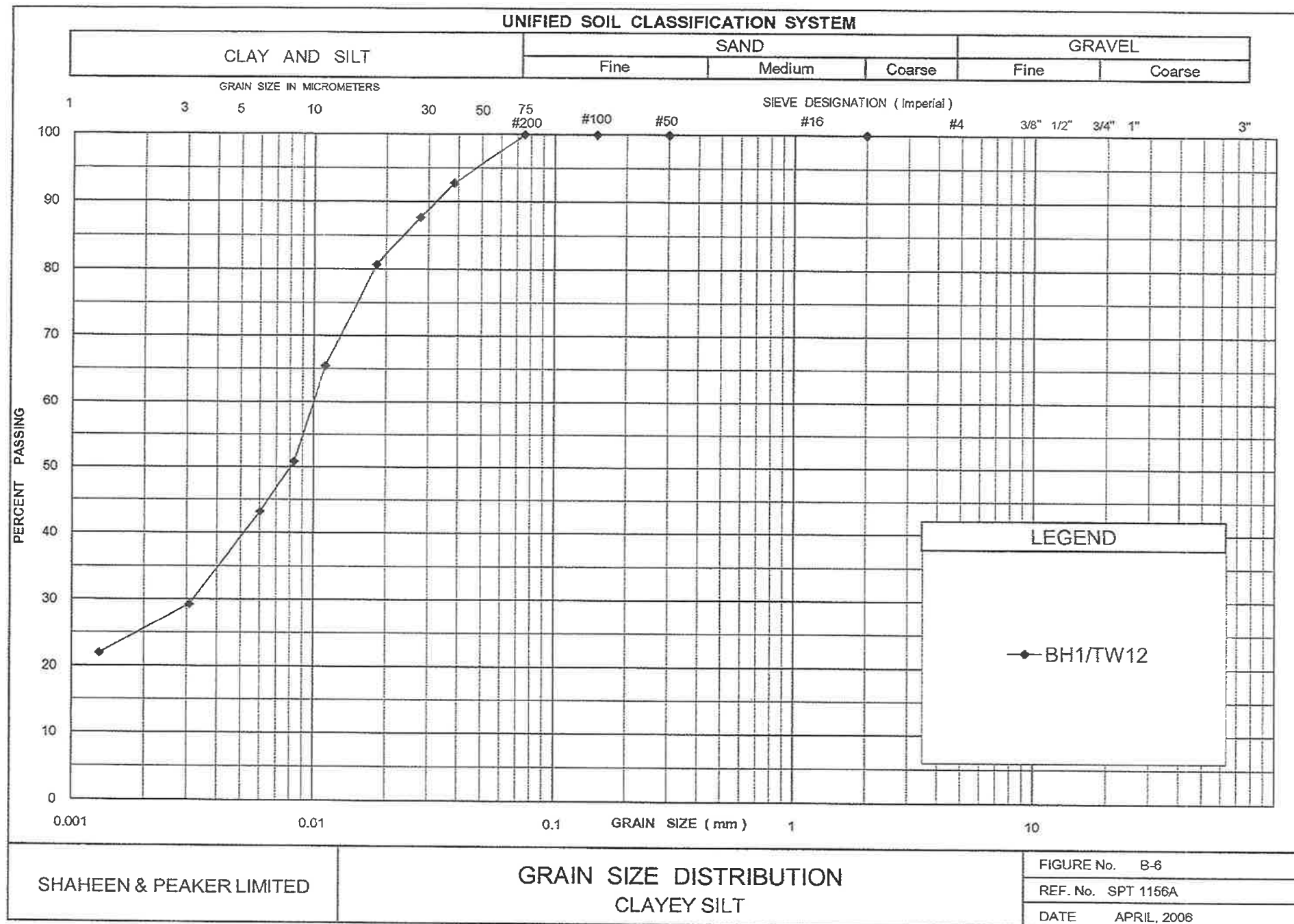


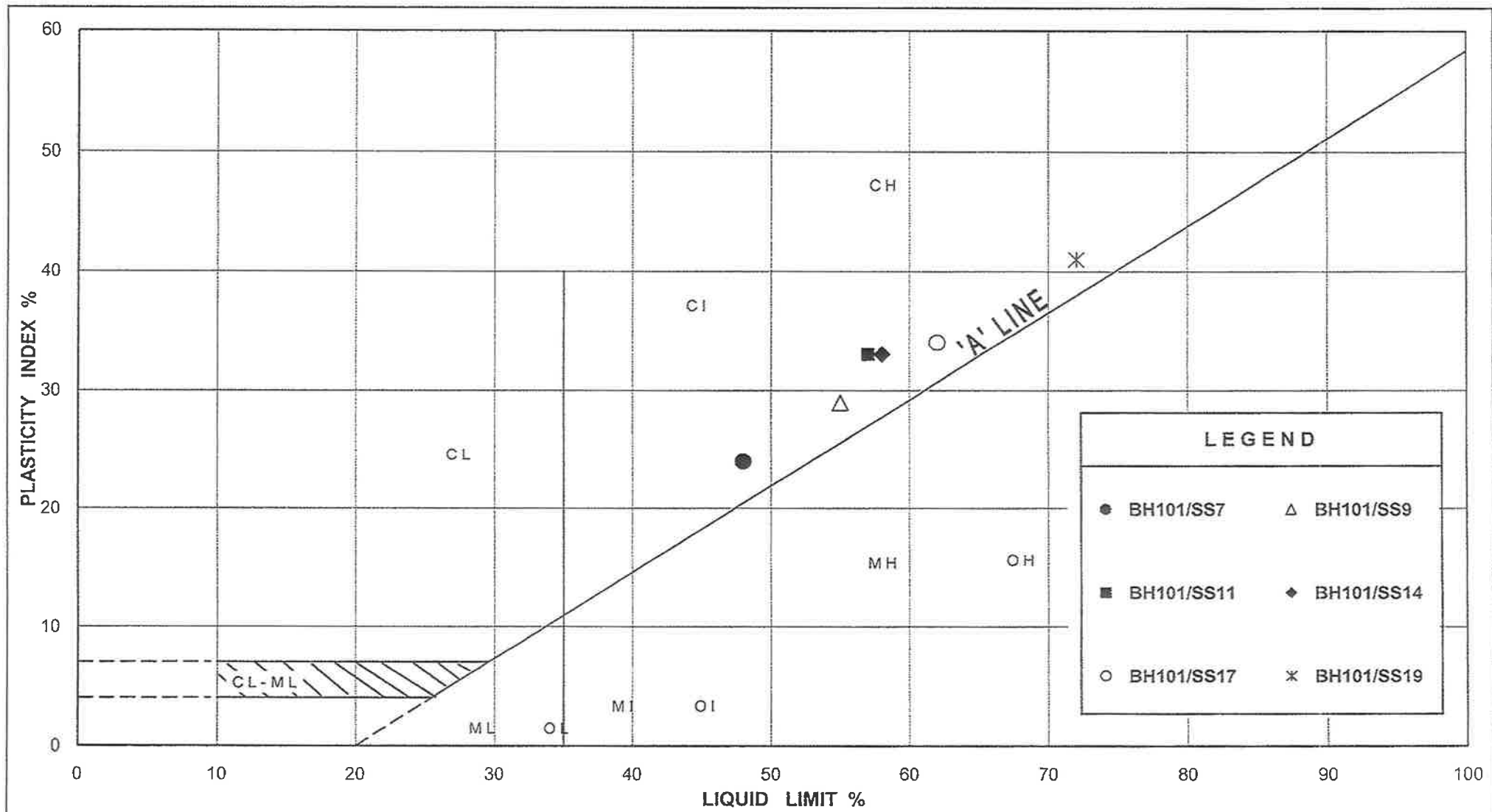


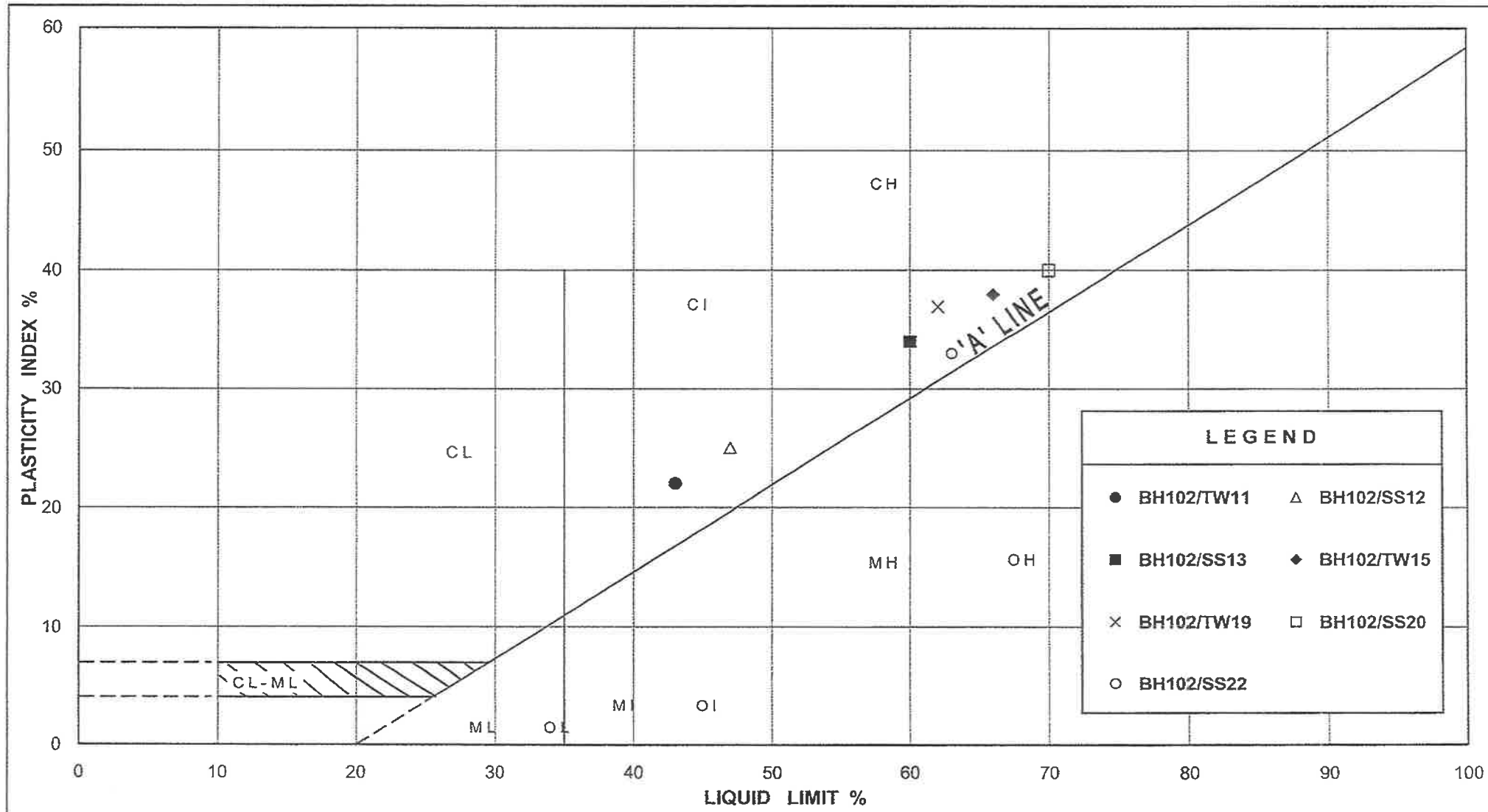


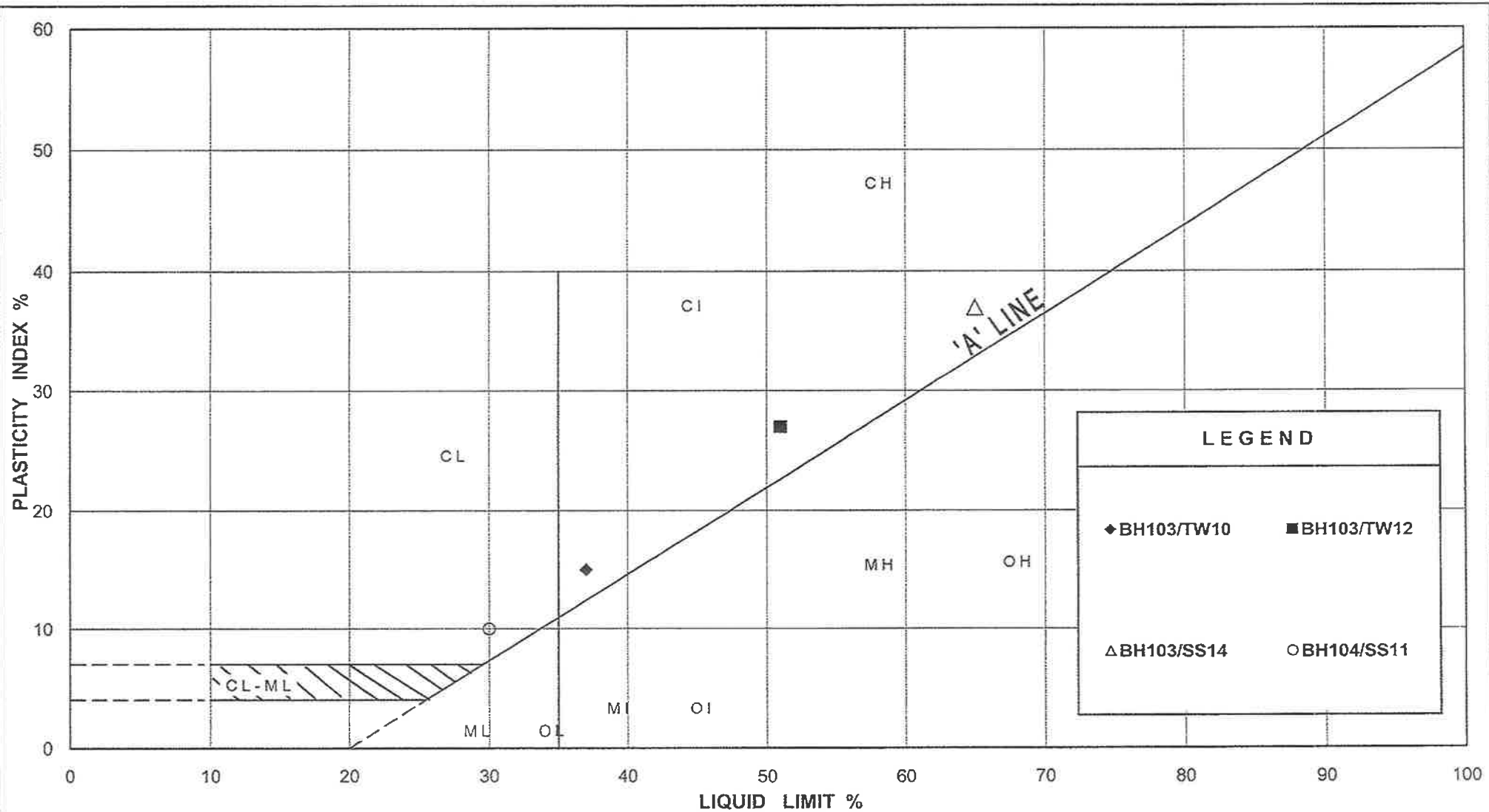


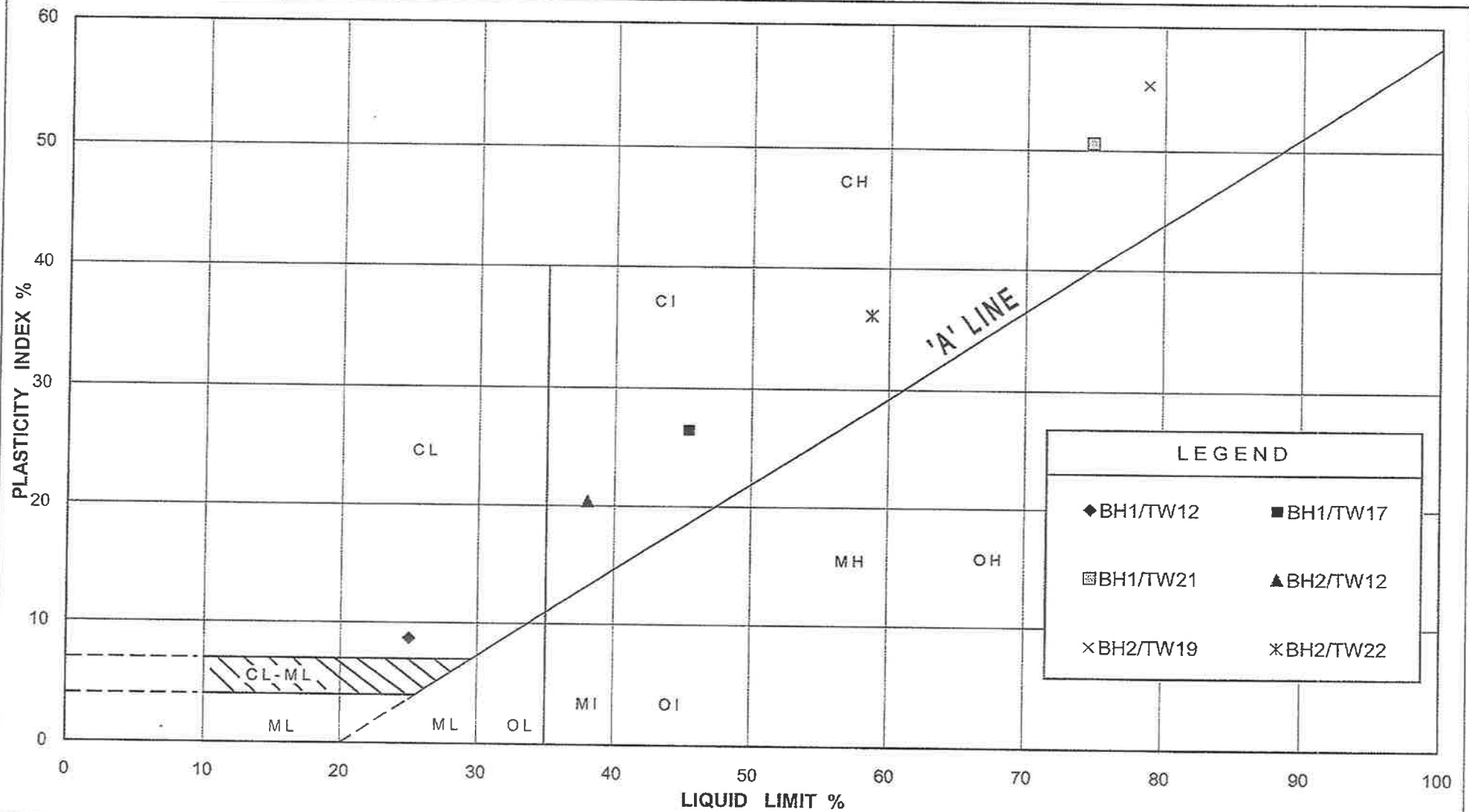












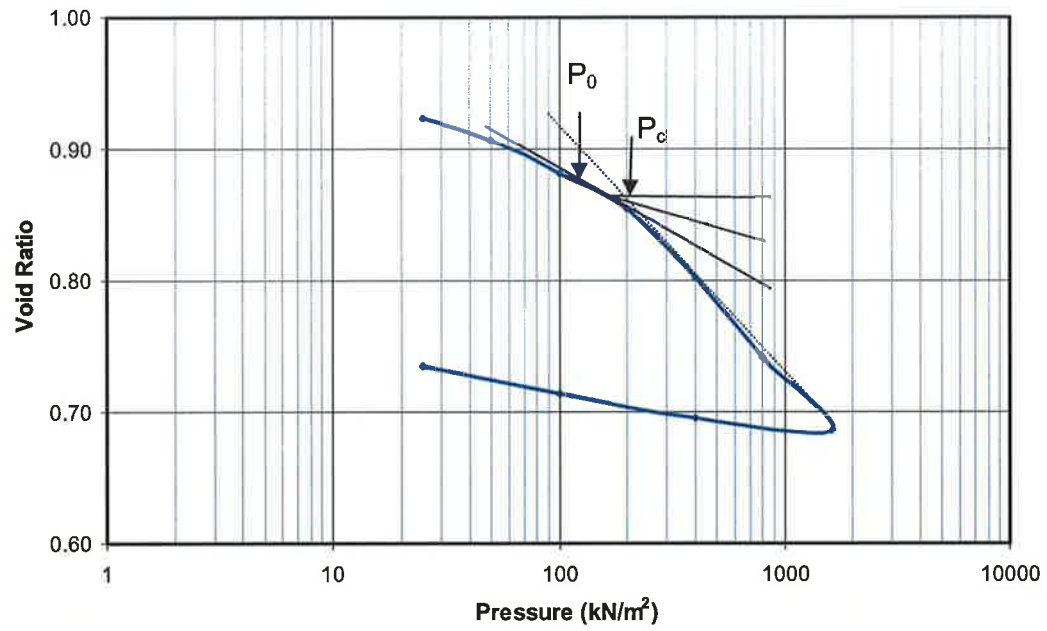
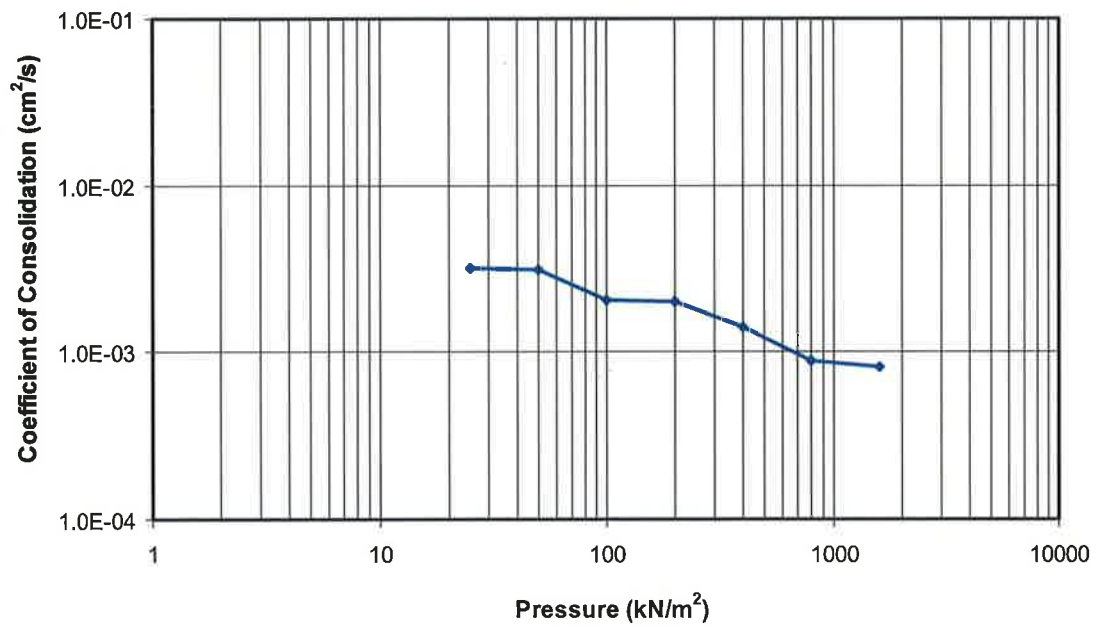
SHAHEEN & PEAKER LIMITED

PLASTICITY CHART
SILTY CLAY

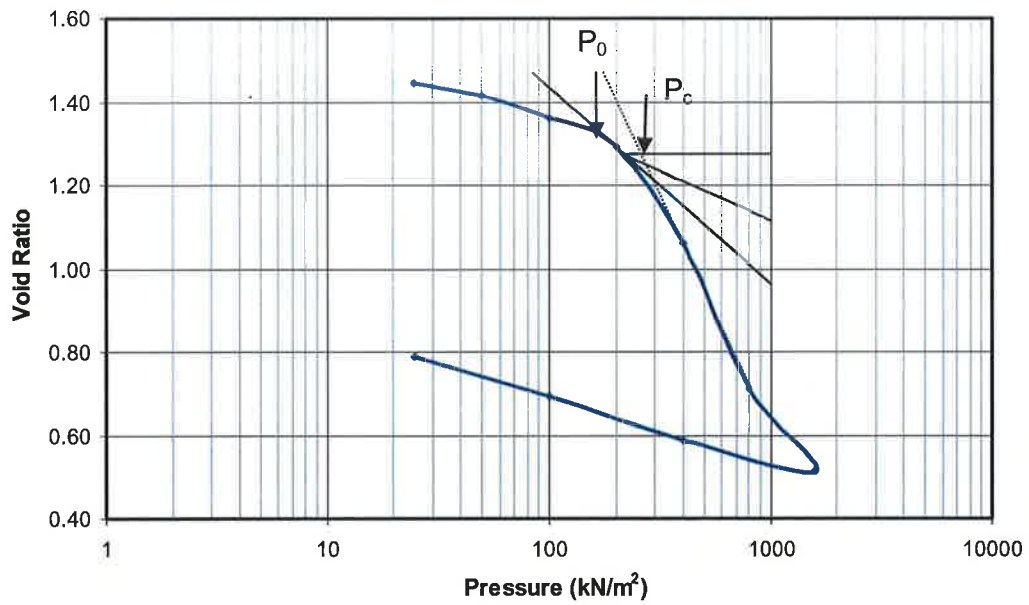
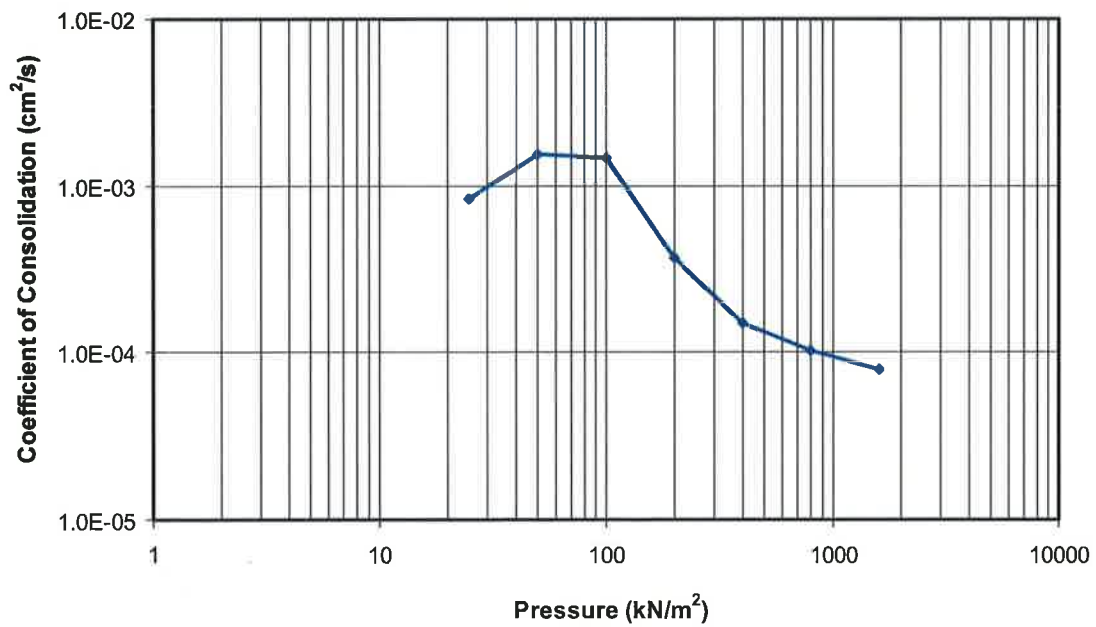
FIG No B-10

G.W.P. 5290-04-00

REF No SPT 1156A

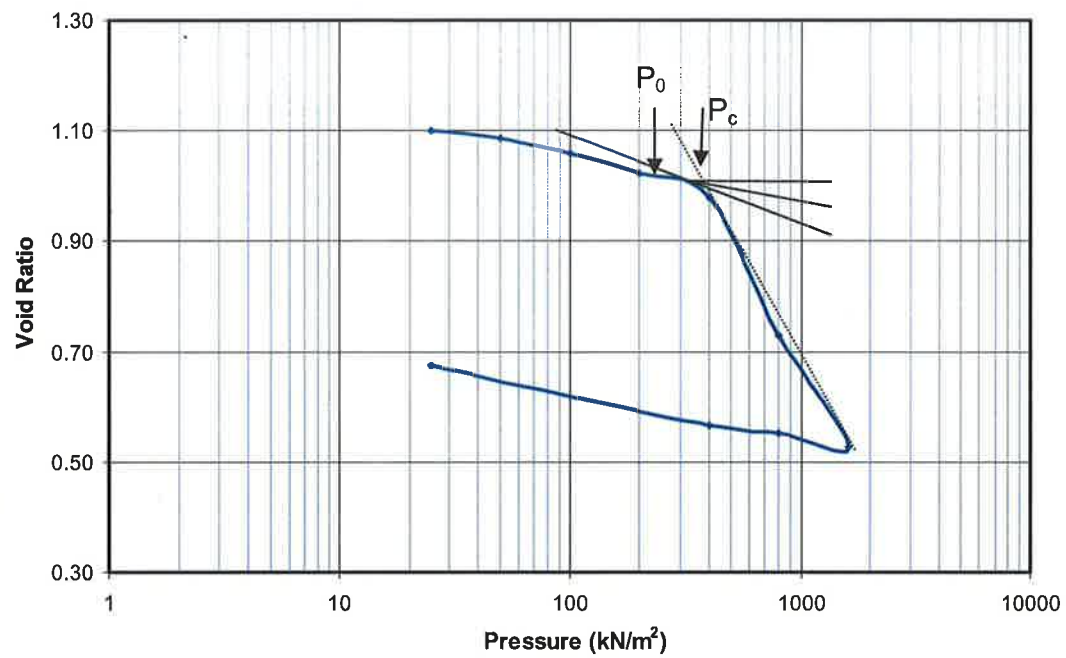
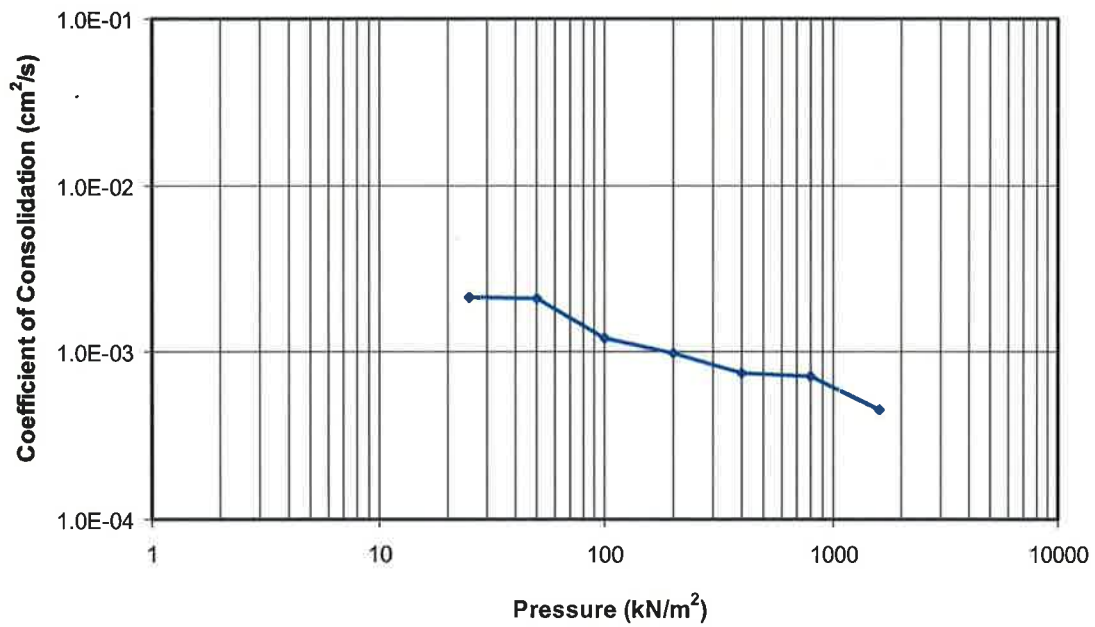
Void Ratio versus Pressure**Coefficient of Consolidation vs. Pressure**

BH 102 TW 11

Void Ratio versus Pressure**Coefficient of Consolidation vs. Pressure**

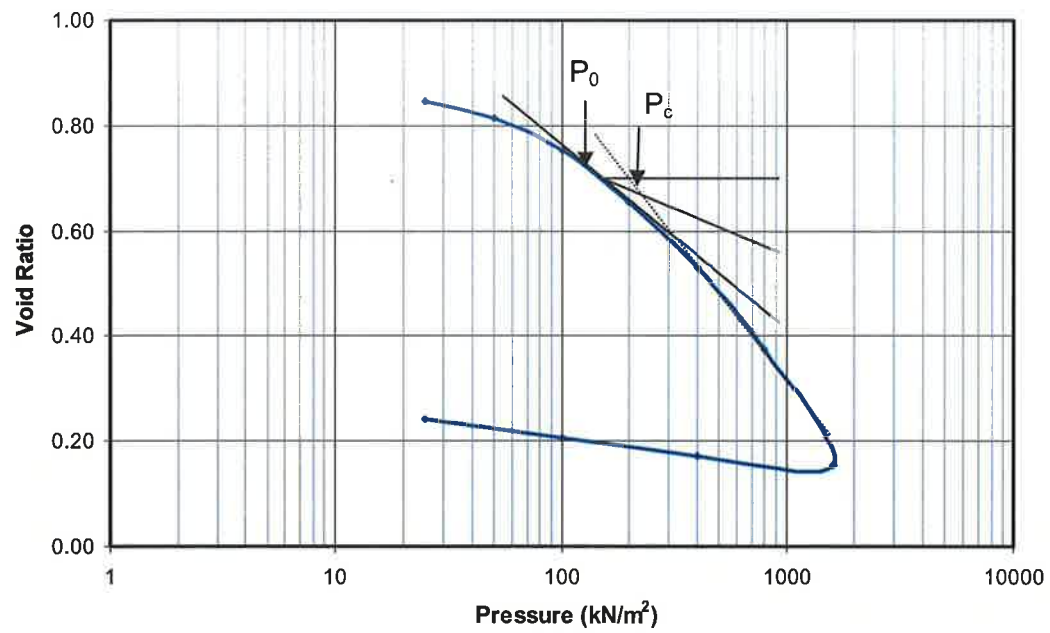
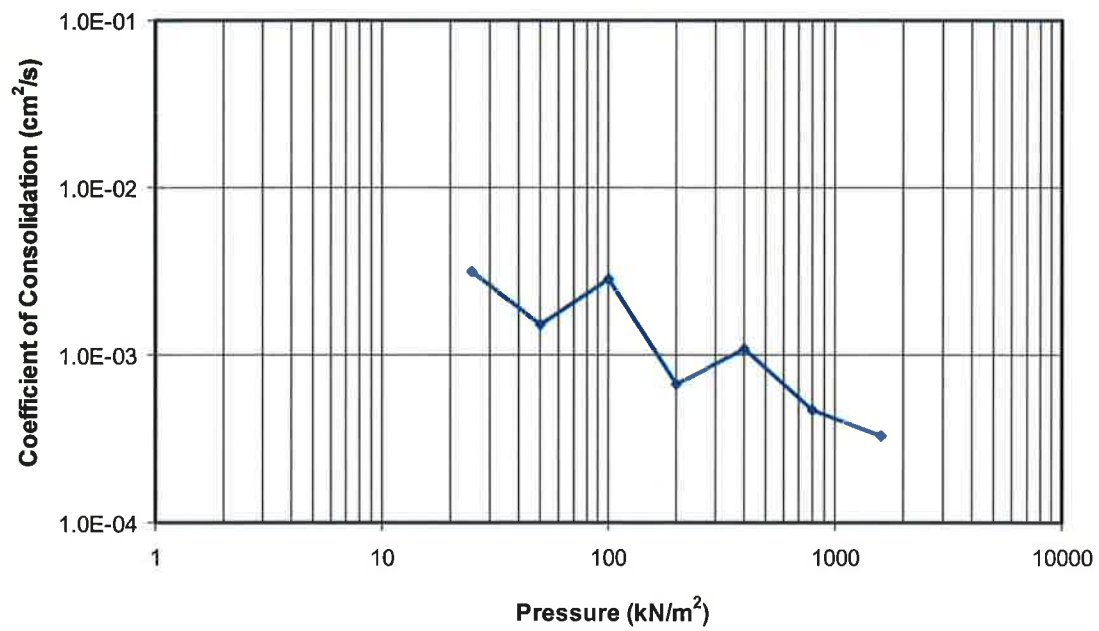
BH 102 TW 15

Figure B-12

Void Ratio versus Pressure**Coefficient of Consolidation vs. Pressure**

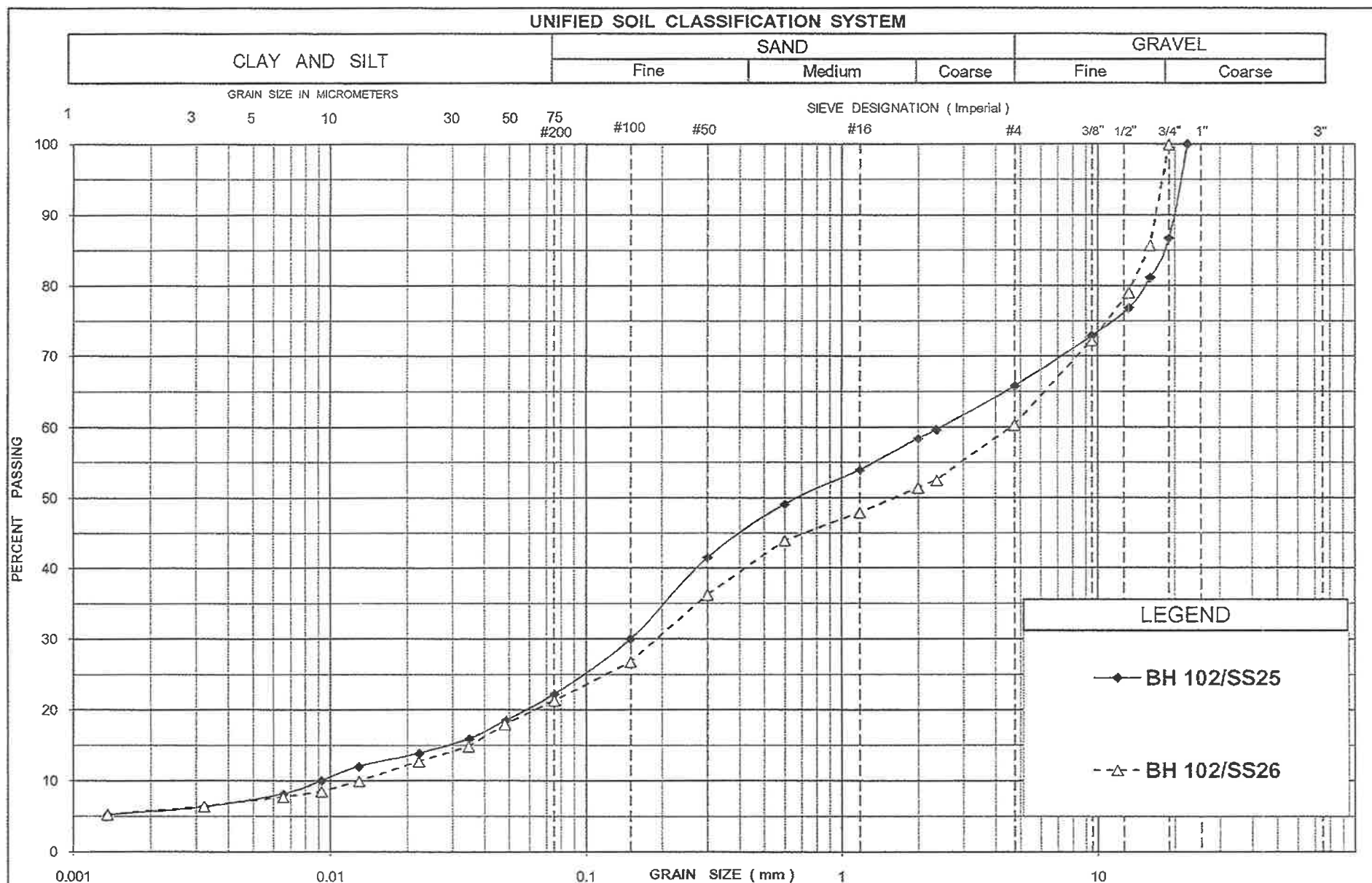
BH 102 TW 19

Figure B-13

Void Ratio versus Pressure**Coefficient of Consolidation vs. Pressure**

BH 103 TW 12

Figure B-14



Appendix C

Undrained Shear Strength Plots

Previous Foundation Investigation Report (2009)

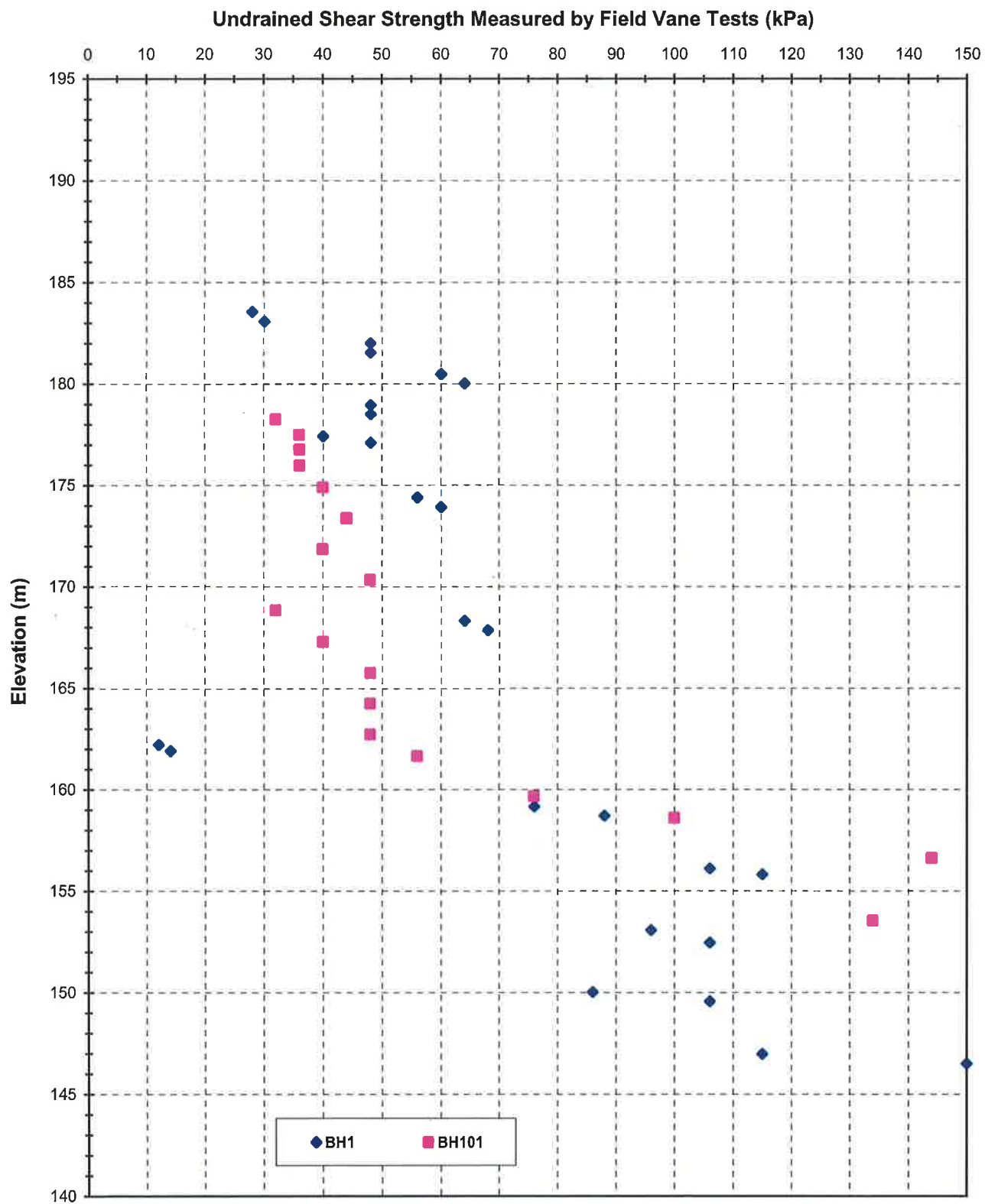


Figure C1 Plot of Undrained Shear Strength with Elevation for Boreholes 1 and 101

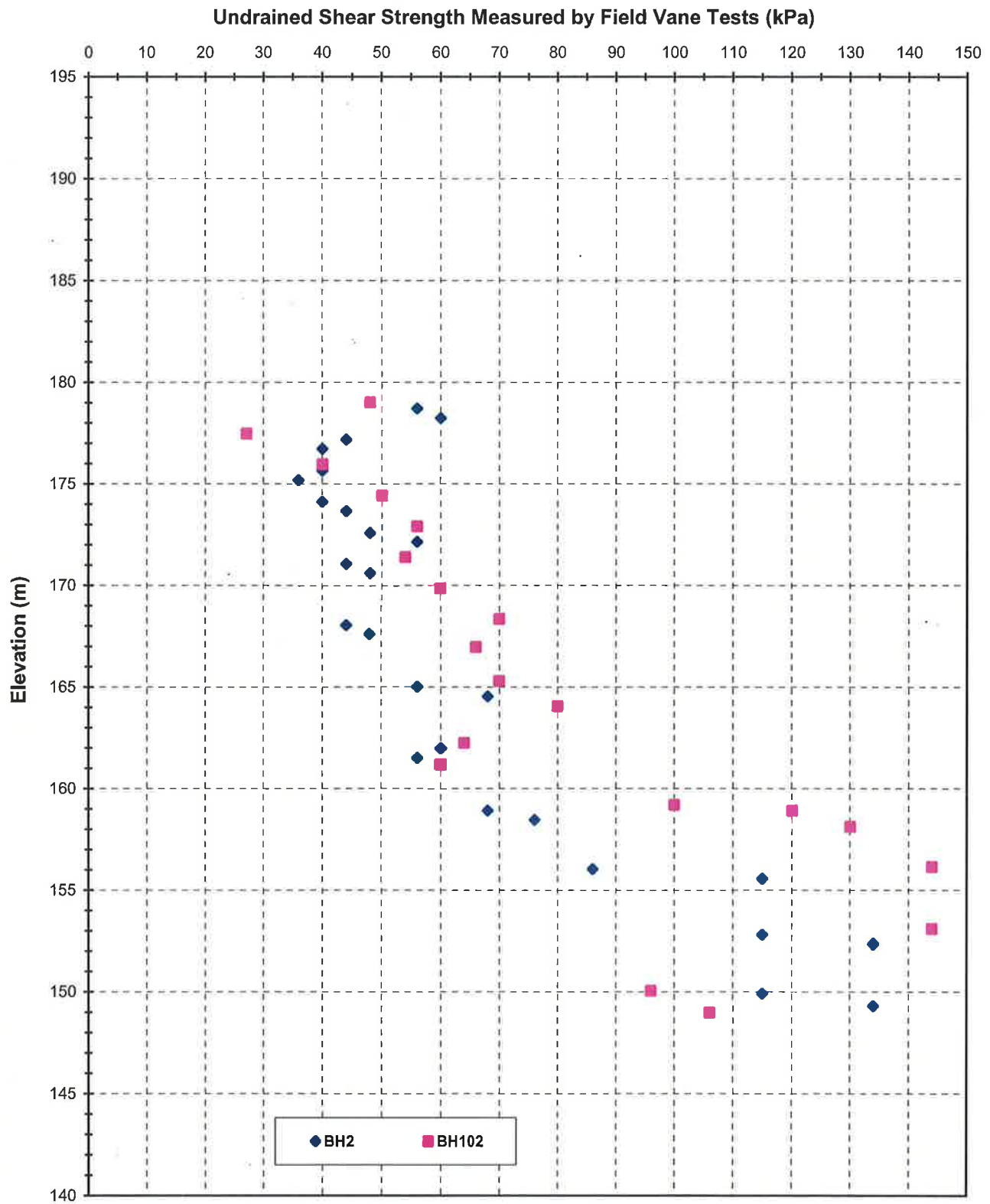


Figure C2 Plot of Undrained Shear Strength with Elevation for Boreholes 2 and 102

Appendix D

Site Photographs

Previous Foundation Investigation Report (2009)



Photograph D-1 Fill section (looking west)



Photograph D-2 West bank of Goulais River



Photograph D-3 East bank of Goulais River



Photograph D-4 Cut section (looking east)A

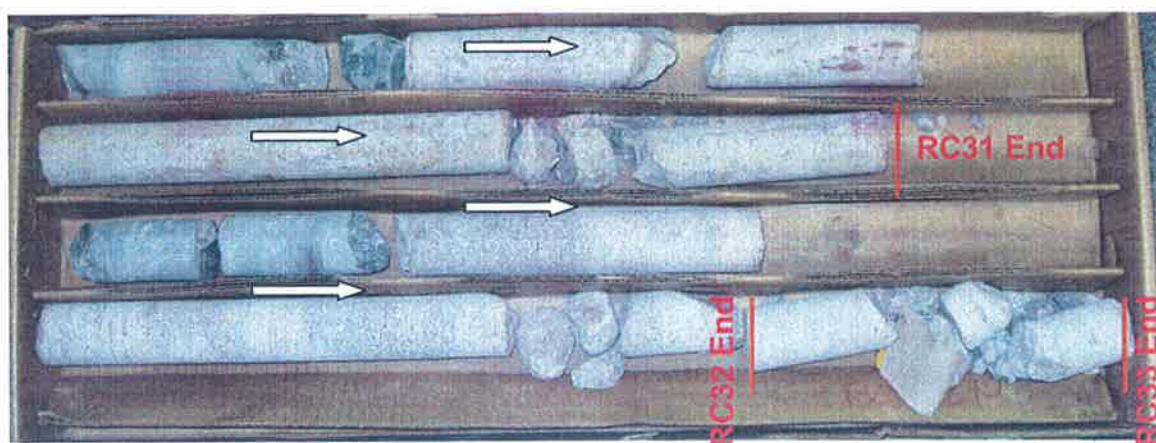
Appendix E

Rock Core Photographs

Previous Foundation Investigation Report (2009)



Borehole 101 RC28, RC30 and RC32



Borehole 102 RC31, RC32 and RC32

Appendix F

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
SHEWFELT BRIDGE RESTORATION,
GOULAIS BAY ROAD, 3 KM WEST OF
HIGHWAY 17, DISTRICT OF ALGOMA,
ONTARIO, W.P. 5139-10-01, SITE 38S-031**

LEA Consulting Limited

Project: TRANETOB01156AB
October 12, 2010

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5.2	New Bridge	10
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Appendices

Appendix G: Cross Section Drawings

Appendix H: Profile Drawings

Appendix I: Results of Slope Stability Analyses

Appendix J: List of Standard Specifications

Appendix K: Limitations of Report

**FOUNDATION DESIGN REPORT
SHEWFELT BRIDGE RESTORATION, GOULAIS BAY ROAD
3 KM WEST OF HIGHWAY 17, DISTRICT OF ALGOMA ONTARIO
W.P. 5139-10-01, SITE 38S-031**

5 DISCUSSION AND RECOMMENDATIONS

5.1 Proposed Works

The original plan was to replace the existing bridge with a new two-span concrete deck on steel girder structure, some 125 m away from the existing site. To this end, in 2006, an advance investigation was carried out, which consisted of drilling two deep boreholes at the proposed new bridge location (Boreholes 1 and 2). This was followed by additional boreholes (100 series boreholes) in 2008. Subsequently, it was decided to rehabilitate the existing bridge. This entailed the repair of the central and east piers of the existing bridge and for this reason, two boreholes (Boreholes 201 and 202) were put down near these support elements. These two boreholes were advanced from the River, using a raft.

These boreholes showed the presence of a very loose to loose sand deposit in Borehole 202, extending to a depth of 1.5 m below the river bed or to El. 179.6 m. Below this elevation in Borehole 202 and immediately at the River bed (El. 180.0 m) in Borehole 201, an approximate 31 to 32 m thick silty clay deposit was contacted (i.e. extending to El. 148.0 m and 148.6 m in Boreholes 201 and 202, respectively). In Borehole 202, the lower 2 m of the deposit was found to consist primarily of clayey silt, with some silt and occasional silty clay and clay interbeds. The presence of occasional silt interbeds was found throughout much of the deposit. While drilling, artesian conditions were noted in this massive basically cohesive deposit, possibly due to the presence of the silt interbeds. This cohesive deposit is underlain below 148.6 to 148.0 m by granular (non-cohesive) deposits. The boreholes were terminated within these basal granular soils after penetrating them for a distance of 3.2 to 4.1 m or at El. 145.4 to 143.9 m, due to severe artesian conditions encountered while drilling. In the deep boreholes drilled in 2006 and 2008, some 120 to 140 m away from the present site, the surficial soils were found to consist of cohesionless sandy silt/silty sand deposits, extending to a typical elevation of 180± m, also underlain by a massive silty clay deposit (changing to clayey silt in the bottom 2 to 4 m) to about El. 146 m. This cohesive deposit was found to be underlain by cohesionless soils exhibiting excess hydrostatic pressure or artesian conditions. These conditions are not dissimilar to the conditions encountered in Boreholes 201 and 202, except that the interface of the massive silty clay deposit with the underlying basal granular soils was contacted at Boreholes 201 and 202 (existing bridge site) at an elevation of about 2 m higher than the previously drilled site. As well, no artesian conditions were encountered in the silty clay deposit while advancing the boreholes at the previously drilled site.

After the field work was completed at the present bridge site, the rehabilitation option was changed to bridge replacement and we were asked to submit our report based on the two available boreholes (i.e. Boreholes 201 and 202) which were drilled for the rehabilitation option. It should be pointed out that due to this, the borehole coverage is not consistent with normal MTO procedures. For example, there are no boreholes at the west abutment location, no boreholes were drilled from the top of the existing embankment, as well, there are no approach holes. In preparing the report and carrying out analyses, we

were asked to make assumptions, but you may wish to carry out an additional investigation to verify the actual conditions.

5.2 New Bridge

The existing single lane bridge is supported on three piers, consisting of a central pier, an east pier and a west pier. The two spans in between the piers measure 31.7 m, each. The abutments are located 10.8 m beyond the piers, bringing the total length to 85.0 m. The structure is known to be supported on timber piles. The new structure will be a 76.2 m long, single lane, single span modular bridge. The new embankments will therefore be located in between the existing abutments and the existing piers. The maximum grade raise is expected to be typically limited to about 1.2 m above the existing road grade at the abutment locations, diminishing to about 0.7 m within a horizontal distance of several meters and then gradually attaining the existing road grade at about El. 190 m.

5.3 Foundations

The silty clay is considered unsuitable to support normal shallow spread footing foundations and, therefore, deep foundations will need to be utilized.

The use of drilled and cast-in-place concrete (caisson) foundations to support the structure is considered impractical due to a lack of a well defined bearing zone within the silty clay and the cohesionless nature of the underlying granular soils, coupled with the artesian conditions encountered. Auger press (auger cast) piles can be extended into the cohesionless soils below the groundwater table but are expected to be uneconomical for this project, as well as offering little resistance to lateral loads. They are, therefore, not recommended based on reliability and cost. Similarly, expanded base concrete piles (Franki-type) and driven concrete piles are not considered to represent cost effective, reliable solutions.

The available borehole data show that, with the prevailing subsurface conditions, the use of a low displacement pile, such as steel H-piles would be better suited than other types, such as steel tube piles. When selecting the pile tip elevations, consideration will need to be given to the presence of artesian conditions within the silty clay deposit and in the underlying granular soils. During the drilling, when, the presence of artesian pressures in the silty clay deposit became evident, the casing was advanced further and the pressure dropped. From this it appears that the artesian probably originates from the silt interbeds within the silty clay and is unlikely to cause major problems while driving the piles. The artesian pressure emanating from the underlying granular deposits, on the other hand, appeared to be much more aggressive. In the view of this there are two scenarios regarding pile tip elevations.

The first scenario is to truncate the driving of the piles within the silty clay/clayey silt. This utilizes adhesion plus very little end bearing. As a result, the geotechnical resistances will be limited but the possibility of problems due to the artesian conditions is also reduced.

The second scenario is to extend the piles a short distance into the granular deposits underlying the silty clay/clayey silt, thus utilizing some end bearing, in addition to adhesion. This will provide higher geotechnical resistances but will increase the potential for problems due to the artesian conditions.

These two scenarios are discussed below.

In the first case, it is recommended that the piles be driven to 1.2 m above the interface of the silty clay/clayey silt and the underlying granular soils. It would be preferable to drive the piles another 0.9 m (i.e. to about 0.3 m above the interface) to pick up additional adhesion and end resistance but since the interface elevation may vary across the site (especially since the available borehole information is somewhat away from the proposed abutment locations where the piles will be driven), a separation of not less than 1.2 m would be desirable. The following table summarizes the recommended pile tip elevations and resistances for this option.

Table 5.3.1 Recommended Pile Tip Elevations and Resistances for Pile Tips in the Silty Clay/Clayey Silt Deposit (Scenario 1)

Borehole No.	Existing Ground Elevation (m) -River bed	Pile Cap (Bottom) Elevation (m)	Recommended Tip Elevation (m)	Corresponding Approximate Pile Length Below Pile Cap (m)	Recommended Pile Resistances for HP 310x110 or 310x125 Steel H-Piles	
					Factored U.L.S (kN/pile)	S.L.S. (kN/pile)
201	180.0	185.6	149.2	36.4	900	600
202	181.1	183.7	149.8	33.9	900	600

As the piles will likely be driven before placing new fill at the abutment locations, the SLS value is based on a settlement of about 40 mm at the back line of piles (i.e. away from the River side) and about 20 mm at the front (near the River frontage). The effects of this magnitude of total and differential settlements can probably be somewhat alleviated by introducing sufficient rigidity to the pile caps. In any event, this aspect should be taken into consideration in the design.

The piles should not be equipped with flange reinforcing as this will adversely affect the adhesion (i.e. pile resistances are primarily derived from adhesion), as well as creating a conduit for upward water and soil particle movement due to artesian conditions which prevail at the site.

We recommend that the piles be driven using a heavy hammer capable of delivering a rated energy of at least 50 kilojoules/blow, but not more than 65 kilojoules/blow (Anna wants to use more than 55 kilojoules/blow). The driving of the piles in the field should be monitored by a recognized pile driving formula such as the Hiley Formula. The estimated ultimate resistance of the piles by the Hiley Formula can be calculated by dividing the recommended axial resistance at U.L.S. by a resistance factor of 0.4 as per current MTO practice. In accordance with this criterion, the recommended ultimate resistance is $900 \div 0.4 = 2250$ kN/pile. The piles may be driven to about 3 m above the design elevation and driving then monitored by employing the Hiley Dynamic Formula in accordance with MTO Standard Drawing SS103-11. Regardless of the resistance obtained we recommend that the piles be driven to the required pile tip elevation. If, however, the ultimate resistance as per Hiley Formula is lower than 2250 kN/pile, the Engineer should be immediately notified.

During the driving process, piles which have already been driven will need to be monitored to determine if they are heaving due to the effects of driving of adjacent piles. If this phenomenon occurs, the affected piles will need to be re-driven. Retapping, to check that relaxation has not occurred, will be necessary in accordance with MTO procedures. Furthermore, it may be necessary to stagger the driving of the piles.

We recommend that consideration be given to pile load test(s) to verify the pile resistances. If this is not possible, we recommend PDA (Pile Driving Analyzer) testing.

The minimum spacing between the piles should be chosen in consideration of the longer than usual pile lengths. The high slenderness ratio of the piles may need to be considered. A heavier section, such as HP 310x125 may be useful in this respect. The design should be in accordance with the Canadian Highway Bridge Design Code S6-06.

We recommend that an NSSP be provided in the contract to warn the Contractor of the presence of artesian conditions and the possibility of encountering cobble and boulders. In addition, mitigating measures can be incorporated in the contract to alleviate the effects of artesian conditions, should it be necessary.

Downdrag forces on the piles need not be considered as the piles will settle under additional loads and thus downdrag forces will be relieved. However, as mentioned before, the back row of piles can be expected to settle somewhat greater than normal values of 25 mm (i.e. up to 40 mm), unless the pile cap rigidity will prevent differential settlements.

The second approach would be to drive the piles into the granular soils underlying the silty clay/clayey silt, thus utilizing additional adhesion and end resistance. As the artesian condition in the boreholes was found to become more aggressive with increased depth, a limited penetration into the granular soils is preferable. The following table summarizes the suggested pile tip elevations and pile resistances for this case.

Table 5.3.2 Recommended Pile Tip Elevations and Resistances for Pile Tips in the Basal Granular Soil Deposits (Scenario 2)

Borehole No.	Existing Ground Elevation (m) -River bed	Pile Cap (Bottom) Elevation (m)	Recommended Tip Elevation (m)	Corresponding Approximate Pile Length Below Pile Cap (m)	Recommended Pile Resistances for HP 310x110 or 310x125 Steel H-Piles	
					Factored ULS (kN/pile)	SLS (kN/pile)
201	180.0	185.6	146.0	39.6	1200	800
202	181.1	183.7	147.0	36.7	1200	800

As mentioned before, due to sequencing, the back row of the piles can be expected to settle (i.e. about 40 mm total settlement). The settlement of the front row of the piles can be expected to be about half this value.

The piles should not be fitted with flange reinforcing as this will reduce resistance due to adhesion, as well as promoting an easier path for artesian water and soil particles to move upwards, due to aggressive artesian conditions encountered. Consideration can be given to using a heavier pile section such as HP 310x125 to minimize the risk due to cobbles and boulders, the presence of which can be expected within the basal granular soils. The heavier section may have the further advantage of increasing lateral load resistance.

The piles will need to be driven using a heavy hammer capable of delivering a rated energy of at least 55 kilojoules/blow, but not more than 70 kilojoules/blow. The driving of the piles in the field should be monitored by a recognized pile driving formula such as the Hiley Formula. The estimated ultimate resistance of the piles by the Hiley Formula can be calculated by dividing the recommended axial resistance at U.L.S. by a resistance factor of 0.4 as per current MTO practice. As the actual driving of the piles in the field will be governed by the Hiley Formula, the pile tip elevations given in the Table 5.3.2 are for general guidance purposes only and the actual pile lengths may be different than the lengths quoted. We

recommend that an NSSP be prepared to inform the Contractor of this possibility. In accordance with the pile driving MTO criterion, the ultimate resistance is $1200 \div 0.4 = 3000$ kN/pile.

With the above criterion, the piles may be driven to about 4 m above the design elevation and the driving then monitored by employing the Hiley Dynamic Formula in accordance with MTO Standard Drawing SS103-11.

It is recommended that the piles not be driven more than about 0.5 m below the given pile tip elevations before notifying the Engineer, due to the significant upward hydraulic gradients encountered in the boreholes.

During the driving process, piles which have already been driven will need to be monitored to determine if they are heaving due to the effects of driving of adjacent piles. If this phenomenon occurs, the affected piles will need to be re-driven. Retapping, to check that relaxation has not occurred, will be necessary in accordance with MTO procedures. Furthermore, it may be necessary to stagger the driving of the piles.

The minimum spacing between the piles should be chosen in consideration of the longer than usual pile lengths. The high slenderness ratio of the piles may need to be considered. A heavier section, such as HP 310X125 may be useful in this respect. The design should be in accordance with the Canadian Highway Bridge Design Code S6-06.

We recommend that an NSSP be provided in the contract to warn the Contractor of the probable presence of cobbles and boulders especially in the basal granular soils, as well as the hydrostatic uplift and artesian conditions. In addition, mitigation measures can be incorporated in the contract to alleviate the effects of artesian conditions, should it be necessary.

Consideration should be given to pile load test(s) to verify the pile resistances. If this is not possible, then PDA (Pile Driving Analyzer) testing should be implemented.

Downdrag forces on the piles need not be considered, as was discussed before. In this instance as in the first scenario case the back line piles can be expected to settle (total settlement up to 40 mm) more than the front row piles, supporting the abutments.

While this particular approach is more reliable from geotechnical resistance point of view, as well as being slightly more economical, it suffers from the fact that there is an increased probability of encountering problems due to the artesian conditions which prevail at the site. For this reason, in our opinion it is not the recommended approach, but rather the termination of the pile tips in the silty clay/clayey silt deposit appears to be less risky regarding the artesian problem, and is the recommended approach.

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

$$k_s = n_h z/d$$

where k_s = coefficient of horizontal subgrade reaction

z = depth

d = pile width

n_h = coefficient related to soil density as given in Table 5.3.3

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

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

$$k_s = 67 c_u/d$$

where k_s = coefficient of horizontal subgrade reaction

c_u = undrained shear strength

d = width of pile

Table 5.3.3

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)
West Abutment Borehole 201	180.0-172.0	Silty Clay	16.5			60
	172.0-162.0	Silty Clay	16.5			90
	162.0-159.0	Silty Clay	17.0			120
	159.0-154.0	Silty Clay	17.0			150
	154.0-148.0	Silty Clay	17.0			130
	148.0-144.5	Sand	19.5	32	6.0	
	144.5-143.6	Sand and Gravel	21.0	34	11.0	
East Abutment Borehole 202	181.1-179.6	Gravelly Sand	19.5	32	1.3	
	179.6-175.0	Silty Clay	16.0			45
	175.0-165.0	Silty Clay	16.5			60
	165.0-162.0	Silty Clay	16.5			90
	162.0-150.6	Silty Clay	17.0			150
	150.6-148.6	Clayey Silt	17.0			130
	148.6-145.4	Sand and Gravel	21.0	34	11.0	

For preliminary estimating purposes, the recommended horizontal resistances for HP310 x 110 steel H-piles are as follows:

Factored Horizontal Resistance at U.L.S. = 130 kN/pile

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

The lateral resistance of the piles can be supplemented, if desired, by horizontal components of battered piles. In this instance, we recommend that the batter be limited to no more than 4:1, as in practice greater batter would be difficult to install.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with C.H.B.D.C. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of C.H.B.D.C.

For unrestrained wing walls (if any), the intermediate earth pressure coefficient K_0 may be adopted. In the determination of degree of wall displacement or rotation to mobilize the fully active earth pressure state, Section C6.9 of the C.H.B.D.C. Commentary can be consulted.

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

5.5 Settlements

We understand that in general the grade raise above the existing grade will be accommodated within approximately 6.6 m wide and 8 m long wing wall enclosures. The grade raise within the wing wall enclosure will typically range from a maximum of 3 m immediately adjacent to the abutment walls, gradually dropping to about 0.7 m. Beyond the wing-wall enclosure the grade raise is expected to be 0.7 m or less. Several cross sections of the proposed embankment configuration were provided to us by LEA and these cross-sections are appended to the report (Appendix G). The wing-walls will be cantilevered to transfer the structural loads to the abutment foundations.

We also understand that where the grade raise is highest (i.e. immediately adjacent to the abutment wall), the height of the fill will be carried by the pile caps (i.e. for a horizontal distance of about 1.8 m beyond the abutment wall). We also understand that this load transfer was taken into consideration by LEA, including additional frictional forces due to settling fill, in determining pile loads.

A settlement analysis was carried out on this basis.

There are no boreholes available from the top of the embankment and we were requested to base our calculations on the subsurface conditions encountered in the previously drilled boreholes from the top of the river bank in the vicinity of the site, along with visual evidence. Since the existing grade at the site is typically 189-188 m and the bottom of the River elevations at Boreholes 201 and 202 are about 181-180 m, the upper 8 m of the existing soils was assumed to consist of basically granular soils of loose to compact relative density.

The consolidation characteristics of the underlying silty clay deposit were investigated in the laboratory by means of two consolidation (oedometer) tests. The results of these tests are presented in Appendix B and these show a possible pre-consolidation pressure of the order of 80 to 140 kPa in excess of the existing overburden pressure. These results are not dissimilar to the test result obtained on samples from previous boreholes in the vicinity of the present site. In addition, as discussed in section 4.2 of this report, the silty clay deposit appears to be over-consolidated based on $0.23 \times P_o'$ relationship. As the proposed embankment is relatively narrow (i.e. approximately 7 m wide), a significant portion of the stress will be distributed within the overlying sand and any existing fill. Based on these premises and using consolidation parameters determined in the oedometer test, from the pre-consolidated range, the maximum settlement in the silty clay deposit was calculated to be about 20 mm. An additional settlement of about 50 mm in the

overlying sandy soils was calculated, bringing the total settlement at the road level to about 70 mm. If the upper natural soils and any fills overlying the silty clay deposit are indeed granular (i.e. sandy), then the settlements in these granular deposits can be expected to take place rather rapidly and can be expected to be substantially completed within about six weeks. Delaying the paving of the road may help in this respect. Alternatively the use of light weight fill, where feasible, can be considered.

Of the anticipated 70 mm settlement, a significant portion will affect the piles, particularly the piles in the back row, as was discussed before. Of this settlement, the consolidation settlement in the silty clay will affect the piles, while of the overlying soils only the settlement of these below the pile cap will affect the piles. This portion can be estimated to be about 20 to 25 mm.

If the existing piles and pile caps (from the existing bridge) are left in place, these can be expected to reduce settlements in the vicinity of the pile caps, by transferring some of the stresses due to the grade raise to the piles. It should however be pointed out that leaving the pile caps in place may lead to some minor differential settlement and/or frost heave.

5.6 Slope stability

Slope stability analyses were carried out using the information provided to us by LEA, as given in Appendix H (forward slope profiles), as well as cross sections given in Appendix G. In our original (2006 and 2008) reports, 3H:1V forward slopes were recommended. We understand however that in this present case, 3H:1V forward slopes are not feasible and since the existing slopes are standing at 2H:1V forward slopes, LEA selected to use 2H:1V forward slopes.

The stability of the forward slopes beyond the abutment and wing wall structures was analysed by the limit equilibrium approach. The analysis was carried out using the commercial two-dimensional slope stability computer program Slope/W (Geostudio 2004) and the Morgenstern and Price method of analysis for both short term (undrained) and long term (drained) analysis calculations.

The soil parameters used for slope stability were based on the previous investigation (in 2008) for soil conditions above El. 181-180 m (since no boreholes were drilled for this present investigation at the abutment locations) and on the current investigation (2010) below these elevations. Based on this and visual observations, the soils above El. 180 m was assumed to be sandy silt underlain by sand to El. 180.0 m. The soil parameters adopted in the analysis are summarized in Table 5.6.1

Table 5.6.1 Soil Parameters for Slope Stability

Soil Type	Unit Weight (kN/m ³)	Shear Strength Parameters			
		Undrained		Drained	
		Shear Strength (kPa)	Angle of internal friction (deg)	Cohesion (kPa)	Effective angle of internal friction (deg)
New Embankment Fill	20.5	-	32	-	32
Sandy Silt/Silty Sand	20.0	-	29	-	29
Sand	20.0	-	30	-	30
Silty Clay	16.5 - 17.0	45 kPa from El. 180.0 to 175.0 m 60 kPa from El. 175.0 to 165.0 m	-	3 kPa from El. 180.0 to 175.0 m 4 kPa from El. 175.0 to 165.0 m	26

As was mentioned before permanent forward slopes no steeper than 3H:1V were recommended for the previously proposed bridge, which is located about 125 m away from the existing bridge. No steeper than 3H:1V permanent slopes were proposed, based on the fact that banks of the River in the general area are known to be highly unstable due to active River erosion and scour.

Based on the information provided to us, the grade raise of approach embankments over and above the existing grades, will be minimal, while an about 1.5 to 2.5 m grade raise is planned immediately beyond the abutment walls. It is our understanding that most of the proposed grade raise will be surrounded on three sides by abutment walls and cantilevered wing wall structures with structural loads transferred to abutment piles. Stability analyses for forward slopes, which will be located beyond the abutment and wing wall structures, with the proposed slope configuration (i.e. 2H:1V), were carried out.

Based on the analyses, as discussed, possible deep seated failures were considered with 2H:1V forward slopes and typical results for the analyses are presented in Appendix I. As shown, the analyses indicate an acceptable factor of safety (i.e. of the order of 1.5) for deep seated failures, provided proper erosion measures are implemented. Shallow failures were not analyzed but we understand that this will be taken care of by implementing proper measures, such as rock (rip-rap) protection.

The stability of side slopes was also looked into. In this case too the existing slopes are standing at approximately 2H:1V side slopes or steeper. Our analysis shows that 2H:1V side slopes would be stable if they are protected from erosion. We recommend, however that where the existing side slopes are steeper than 2H:1V they should be flattened to no steeper than 2H:1V.

5.7 Construction

We understand that the proposed pile cap (bottom) Elevations are 185.6 m and 183.7 m for the west and east abutments, respectively. This means that excavations to construct the abutments will likely have to extend to at least slightly below these elevations (in order to place a layer of granular soil for equipment and construction traffic support). It is envisaged that some dewatering will be required to provide a stable excavation base, especially at the east abutment location. As the water level at the site will likely be

controlled by the water level in the River, the level of dewatering effort will also depend on the water level in the River, at the time of construction.

The water level in the River was recorded at El. 183.2 m in July 2010, while in November 2005, a high level of 184.1 m was recorded. Assuming a water level of about 183.5-183.0 m in the River, a low level of dewatering effort can be expected at the west abutment location, assuming excavation will extend to about El. 185.3 m. Pumping from shallow filtered sumps will probably suffice, if needed. At the east abutment location, with the proposed pile cap elevation, the excavation will probably extend below or very close to the water level in the River. In this instance, depending on the actual soil conditions, closely spaced and strategically spaced filtered, deep sumps will likely be required. If well points are to be used, the position of the silty clay deposit below the granular soils should be considered in the design.

The excavations can be expected to require excavation protection (i.e. shoring).

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

SP 105S19 – Protection Systems

SP 902S01 – Excavation and Backfilling to Structures.

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. The shoring system should be designed by a Professional Engineer, experienced in this type of work.

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

5.8 Scour Protection

The Goulais River is actively eroding its shores and large scale slumping and slope failures are evident in the general area.

We recommend that channel and bridge scour protection and erosion control be designed by an experienced Hydraulic Engineer. The following should be considered for erosion/scour control measures.

- Flow rate
- Water depth
- Type of transported sediments
- Detailed cross section survey
- Stream pattern and alignment
- Channel gradient
- Effects of the constriction of river flow due to the construction of the bridge piers

- Effect of flooding

Based on the previous investigations near this site, the following are some suggestions which would be subject to review and revision during design by an experienced Hydraulic Engineer. The scour/erosion protection can possibly consist of 0.5 m thick R-50 size rock, as per OPSS 1004. A granular filter or a suitable geotextile will be required for separation and filtering purposes. Granular filter can consist of a 150 mm thick layer of concrete Fine Aggregates (Type FA1) underlain by another 150 mm thick layer of Concrete Coarse Aggregates (Group I/20-5). Alternatively, a sufficiently robust geotextile such as Terafix 400R or 800R (or equivalent) can be placed in lieu of the natural filter materials. It should however be pointed out that geotextile may not be stable on slopes steeper than 3H:1V, unless they are permanently secured. All materials will need to be machine placed in a manner to avoid segregation. The scour/erosion system should be placed at least 0.3 m above the 1:100 year storm elevation. It is furthermore recommended that some form of scour/erosion protection be extended at least 10 m into the river bed, in view of the fact that the river is aggressively eroding its banks, in this general area.

5.9 Frost Protection

Design frost protection depth for the general area is about 1.9 m. Therefore, a permanent soil cover of about 1.9 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. In case of rockfill, only one-half of the rockfill thickness should be assumed to be effective in providing frost protection.

6 CLOSURE

As was mentioned before, the investigation was carried out for the rehabilitation option of the existing bridge structure. The investigation falls short of MTO requirements for the replacement option. You may wish to advance additional boreholes to augment the information from this present investigation.

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

For and on behalf of Coffey Geotechnics Inc.


Gwangha Roh, Ph.D.


Ramon Miranda, P.Eng.

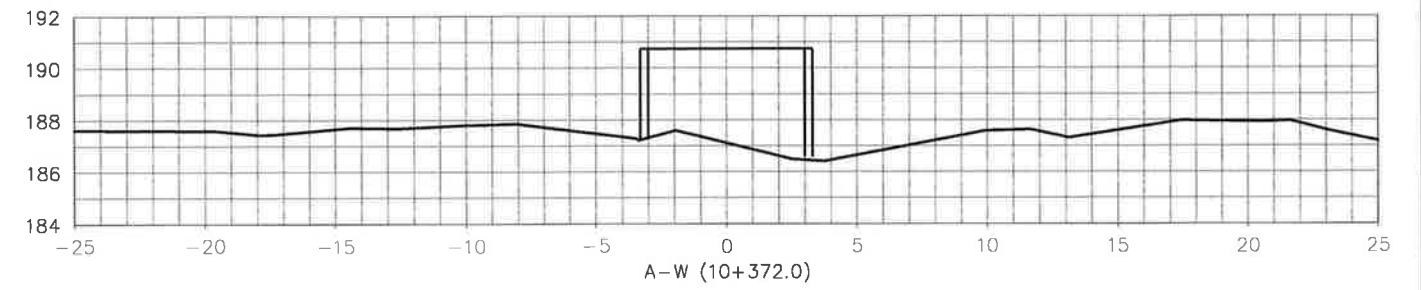
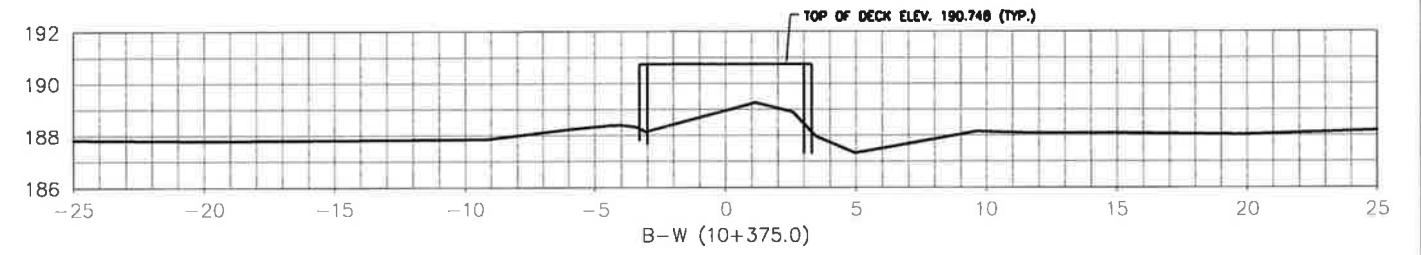
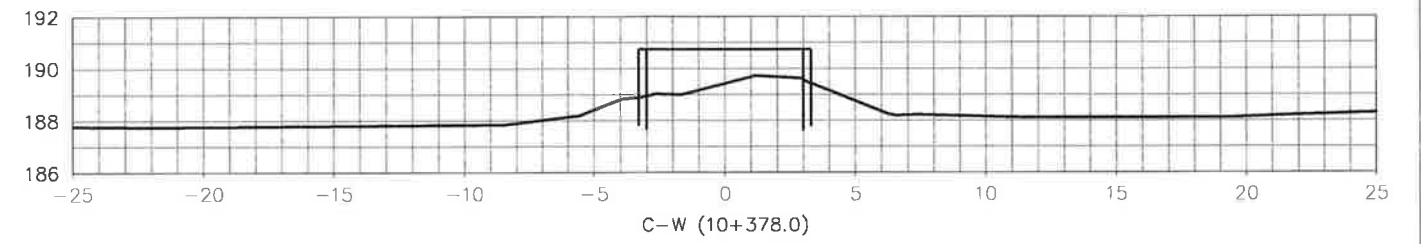
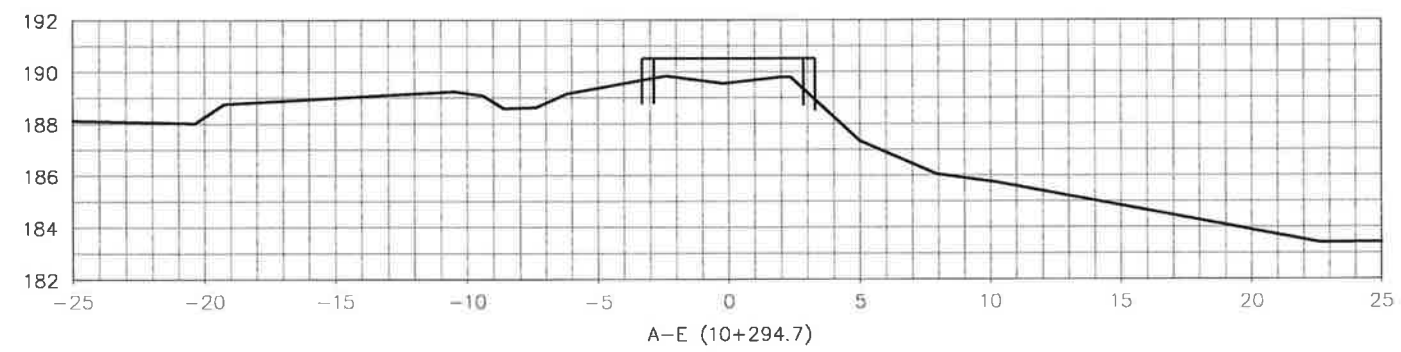
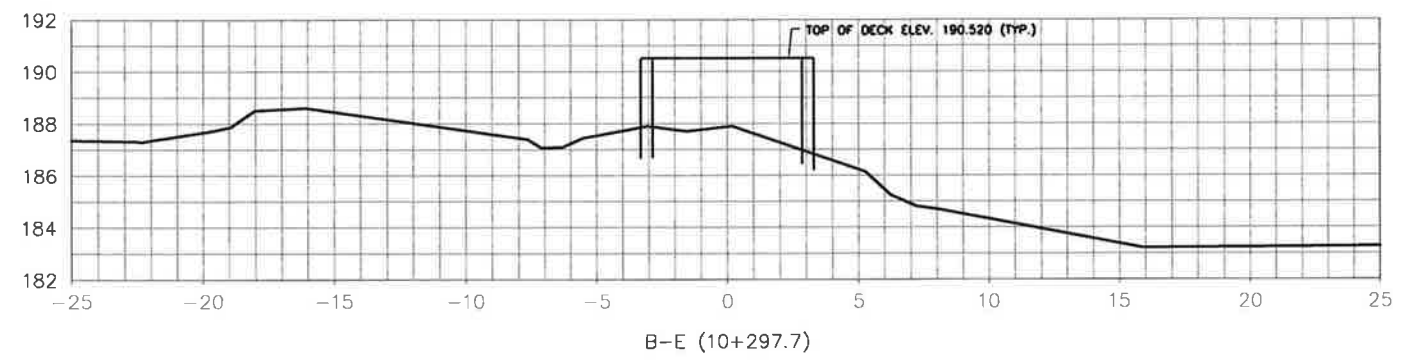
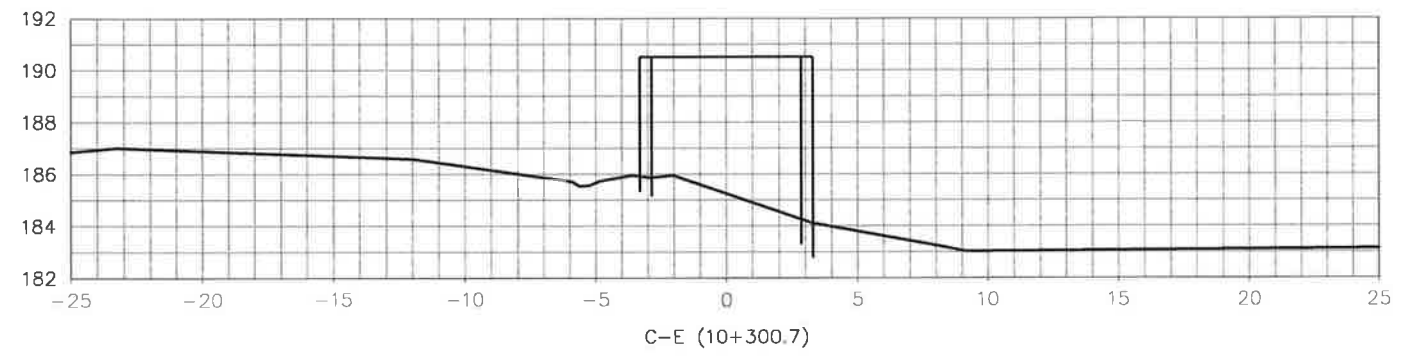



Zuhtu Ozden, P.Eng.



Appendix G

Cross Section Drawings

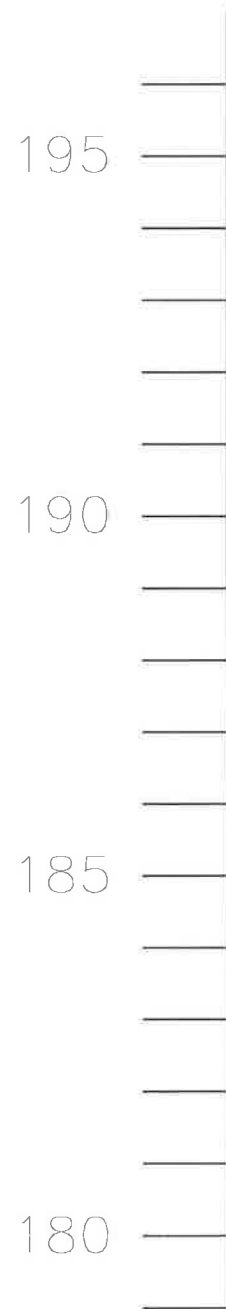


Appendix H

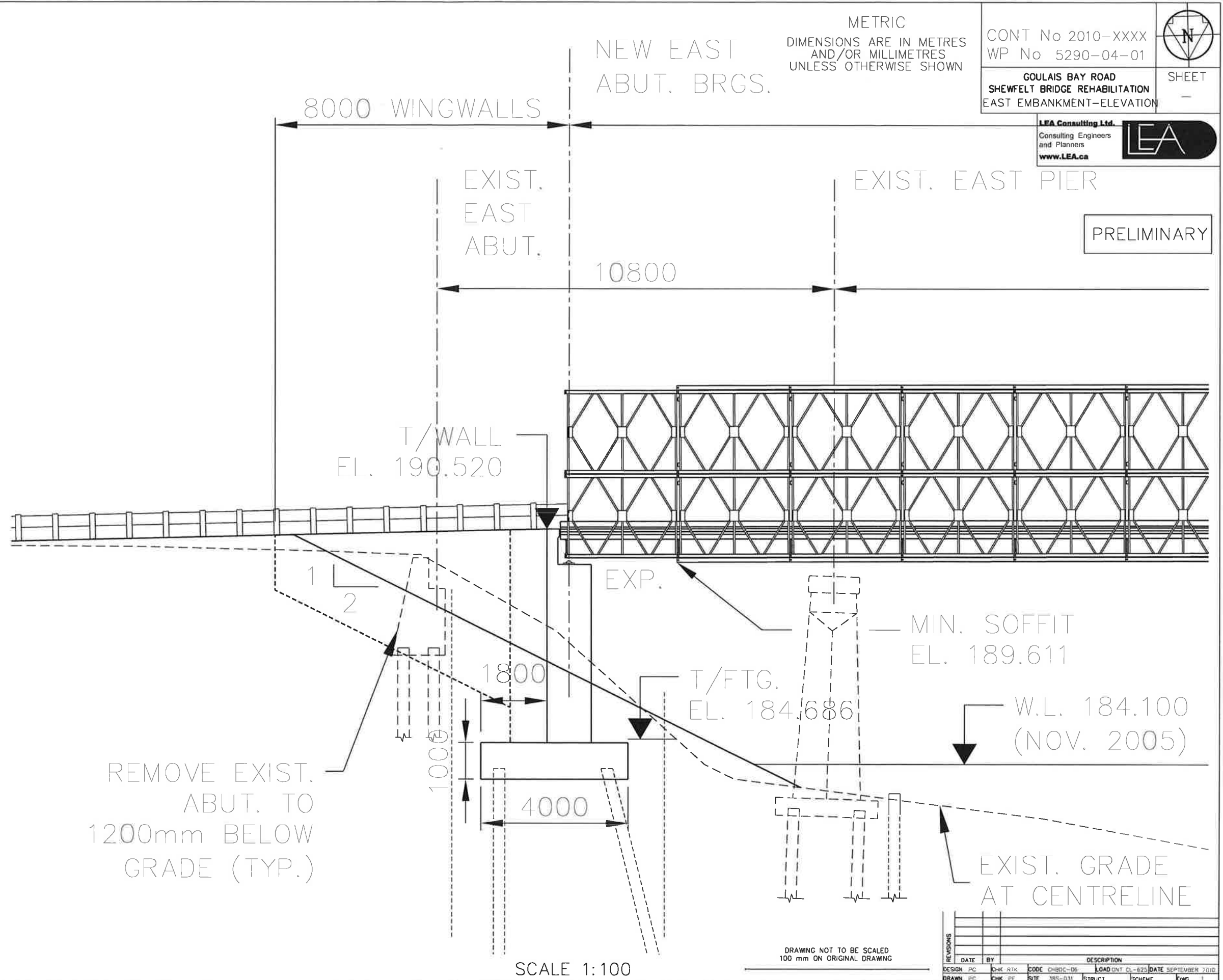
Profile Drawings

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CREATED: Oct 04, 2010-11:37am

MINISTRY OF TRANSPORTATION, ONTARIO
PR-D-707 88-05



REMOVE EXIST.
ABUT. TO
1200mm BELOW
GRADE (TYP.)



SCALE 1:100

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No 2010-xxxx
WP No 5290-04-01

GOULAIS BAY ROAD
SHEWFELT BRIDGE REHABILITATION
EAST EMBANKMENT-ELEVATION

LEA Consulting Ltd.
Consulting Engineers
and Planners
www.LEA.ca



SHEET

PRELIMINARY

REVISIONS		DATE		BY		DESCRIPTION	
DESIGN	PC	CHK	RTK	CODE	CHBDC-06	LOAD	ONT CL-623
DRAWN	PC	CHK	PF	SITE	385-031	STRUCT	SCHEME

DATE SEPTEMBER 2010

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No 2010-XXXX
WP No 5290-04-01

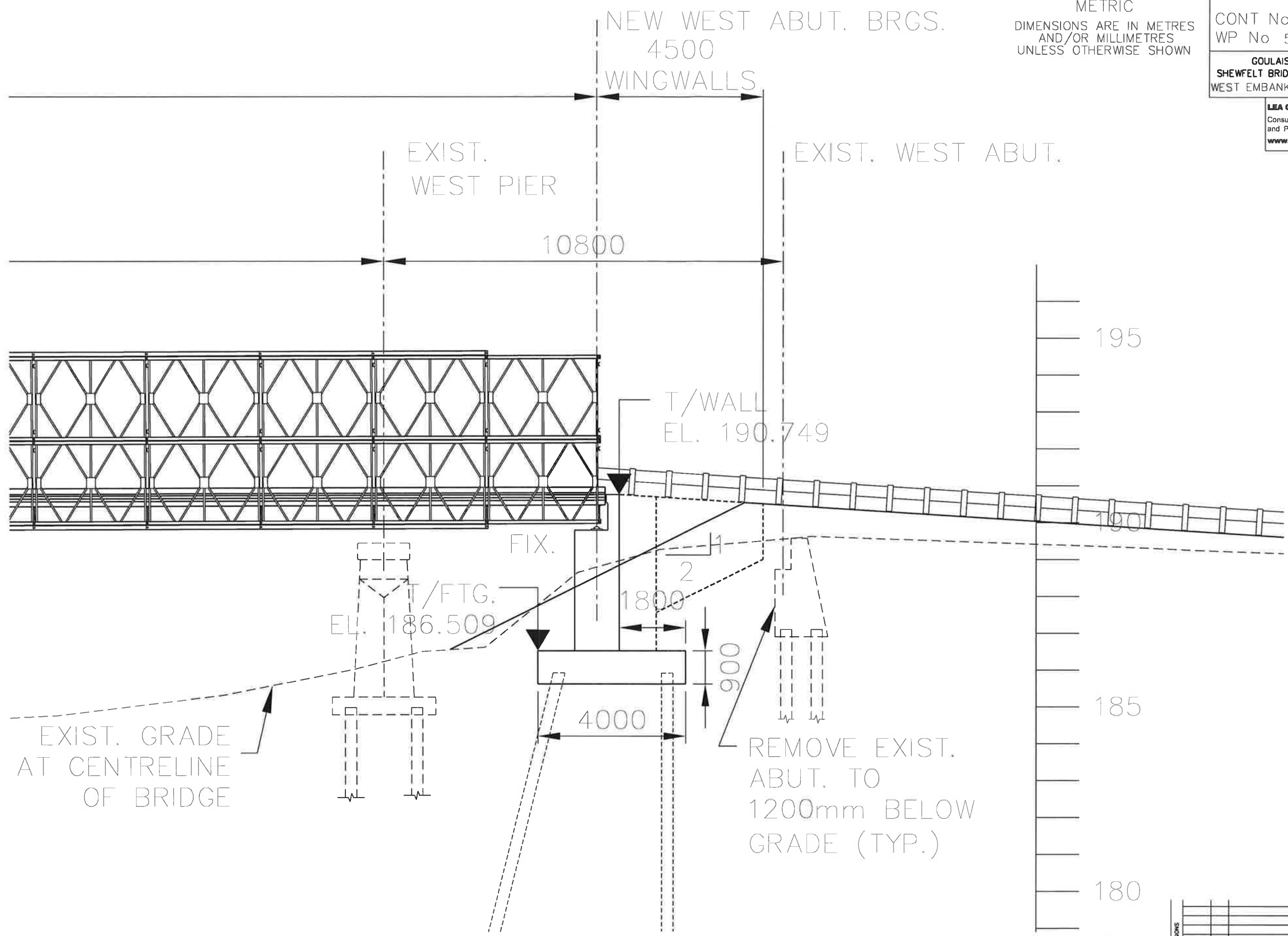
GOULAIS BAY ROAD
SHEWFELT BRIDGE REHABILITATION
WEST EMBANKMENT-ELEVATION

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SHEET
-

PRELIMINARY



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS		DATE		BY		DESCRIPTION	
DESIGN	PC	CHK	RJK	CODE	CHBDC-06	LOAD QNT	CL-B25
DRAWN	PC	CHK	PF	DATE	38S-031	STRUCT	SCHEME

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Appendix I

Results of Slope Stability Analyses

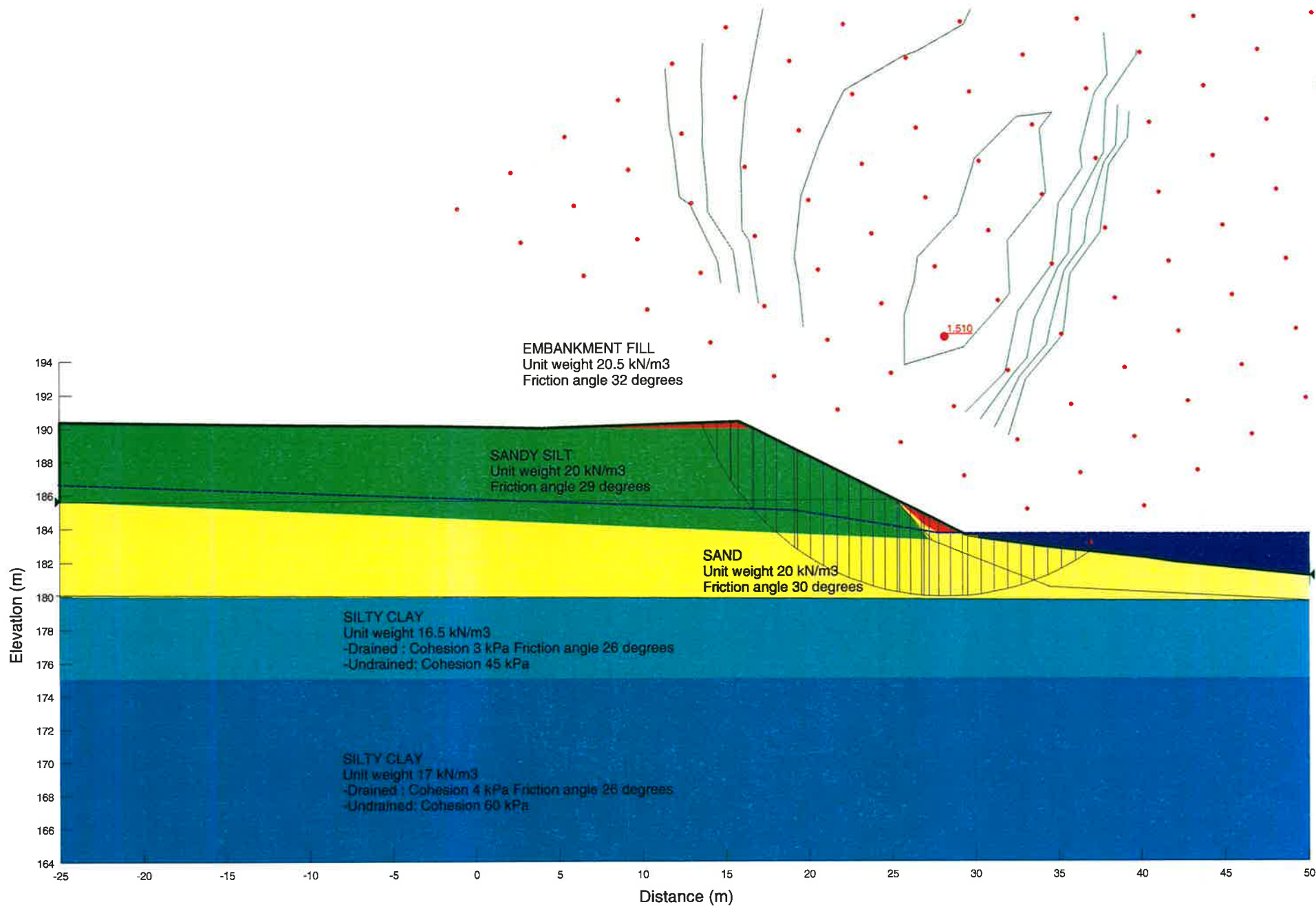


Figure 1. East Abutment -undrained analysis with normal water level

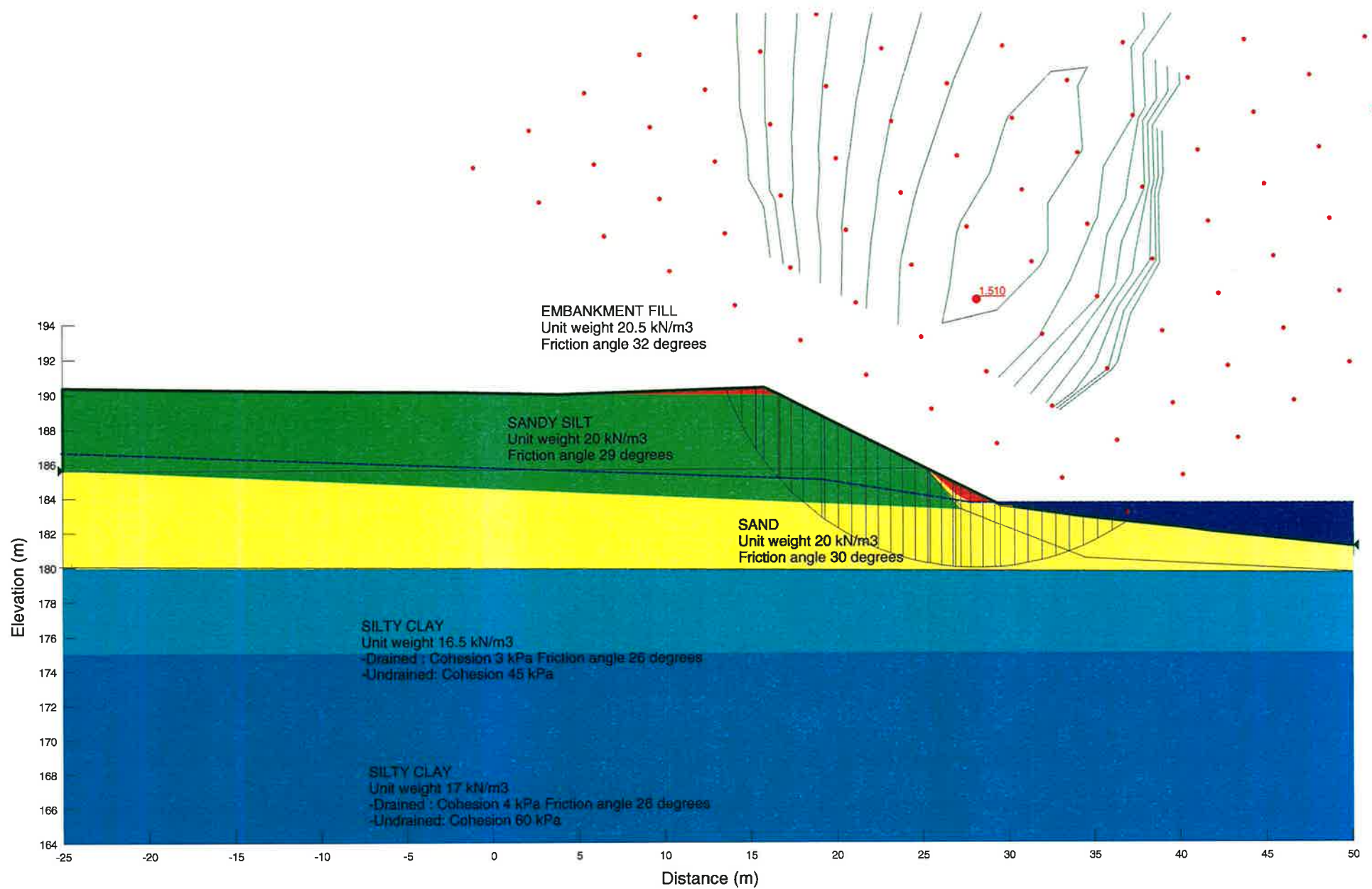


Figure 2. East Abutment -drained analysis with normal water level

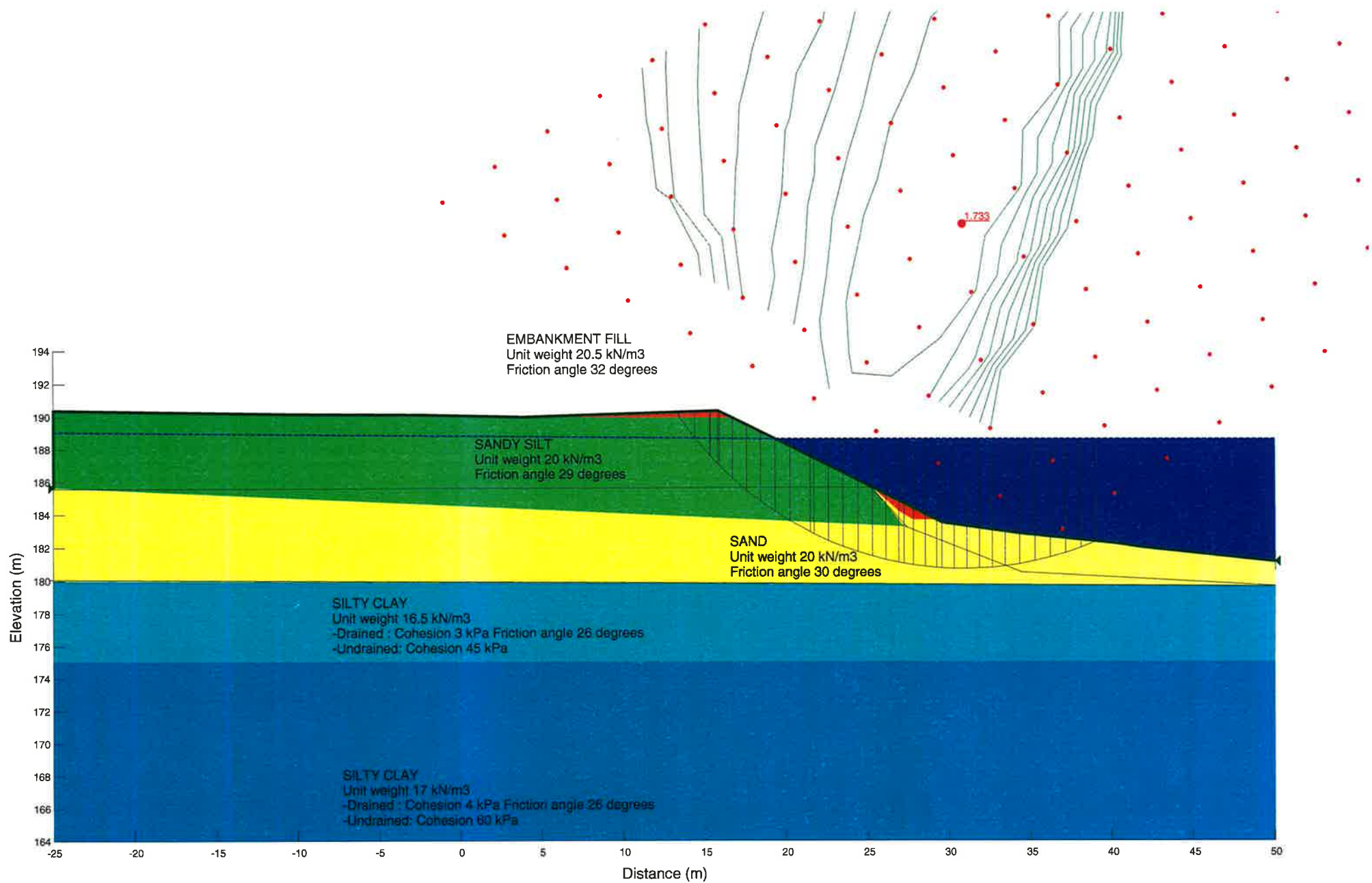


Figure 3. East Abutment -undrained analysis with high water level

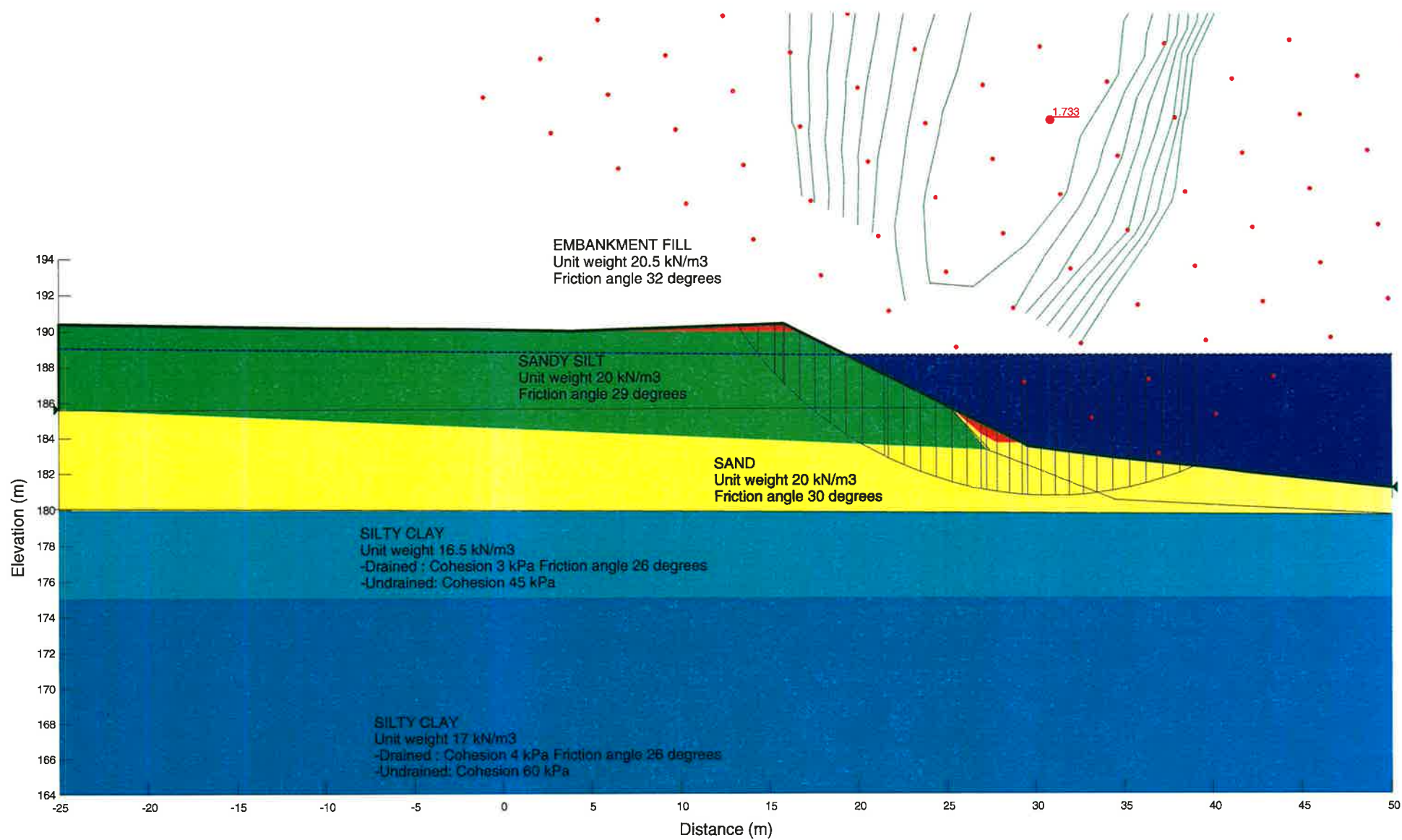


Figure 4. East Abutment -drained analysis with high water level

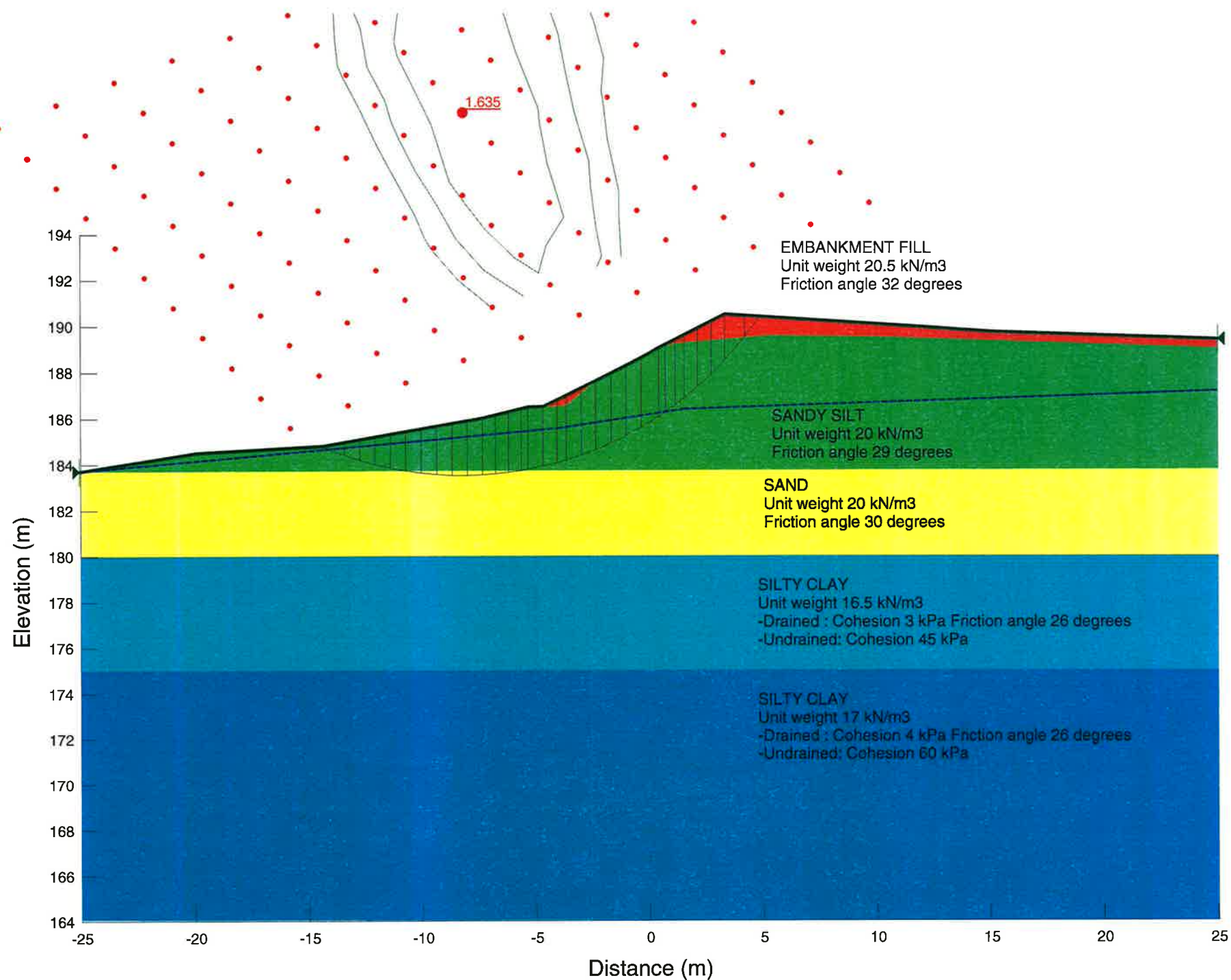


Figure 5. West Abutment - undrained analysis with normal water level

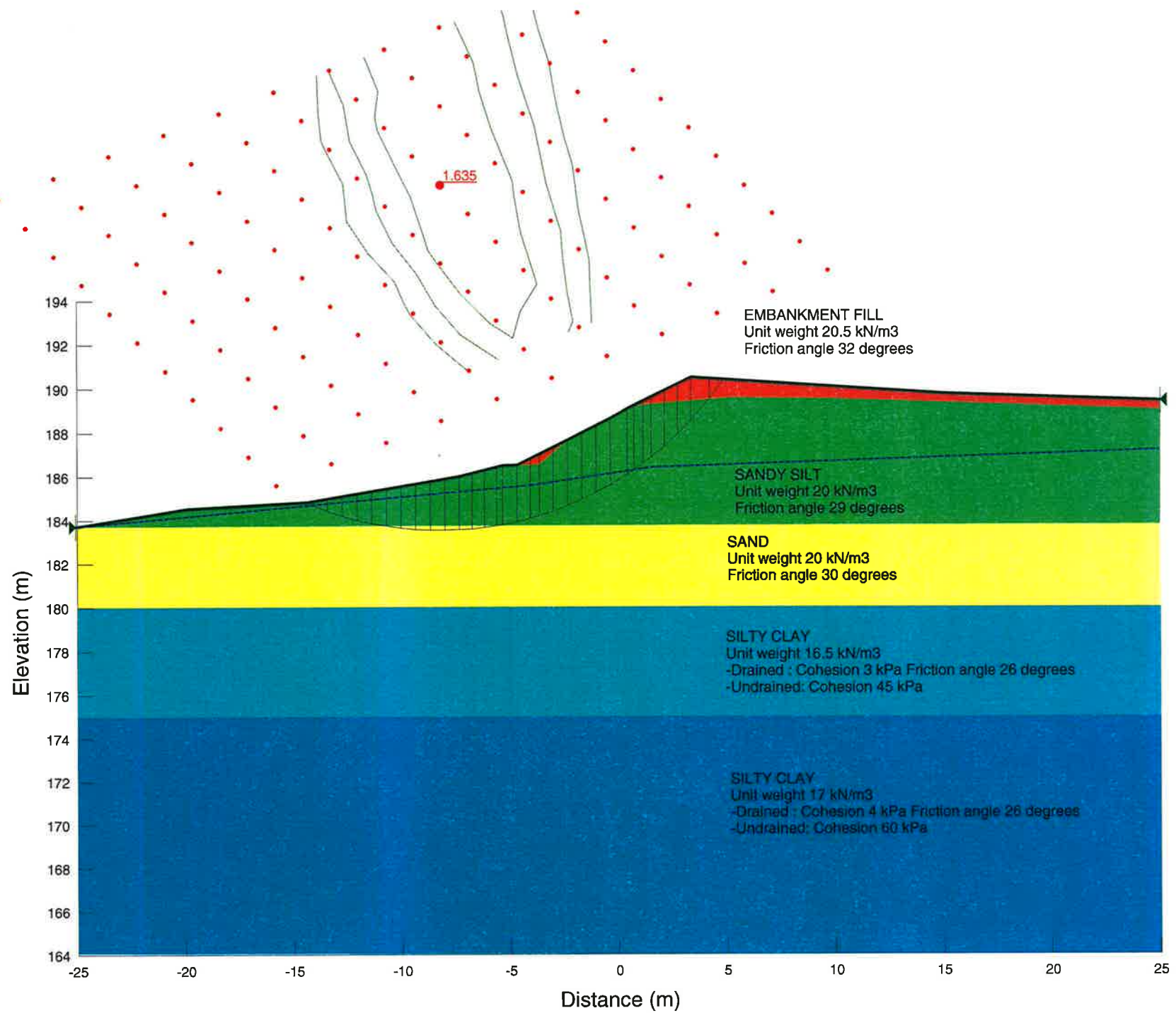


Figure 6. West Abutment - drained analysis with normal water level

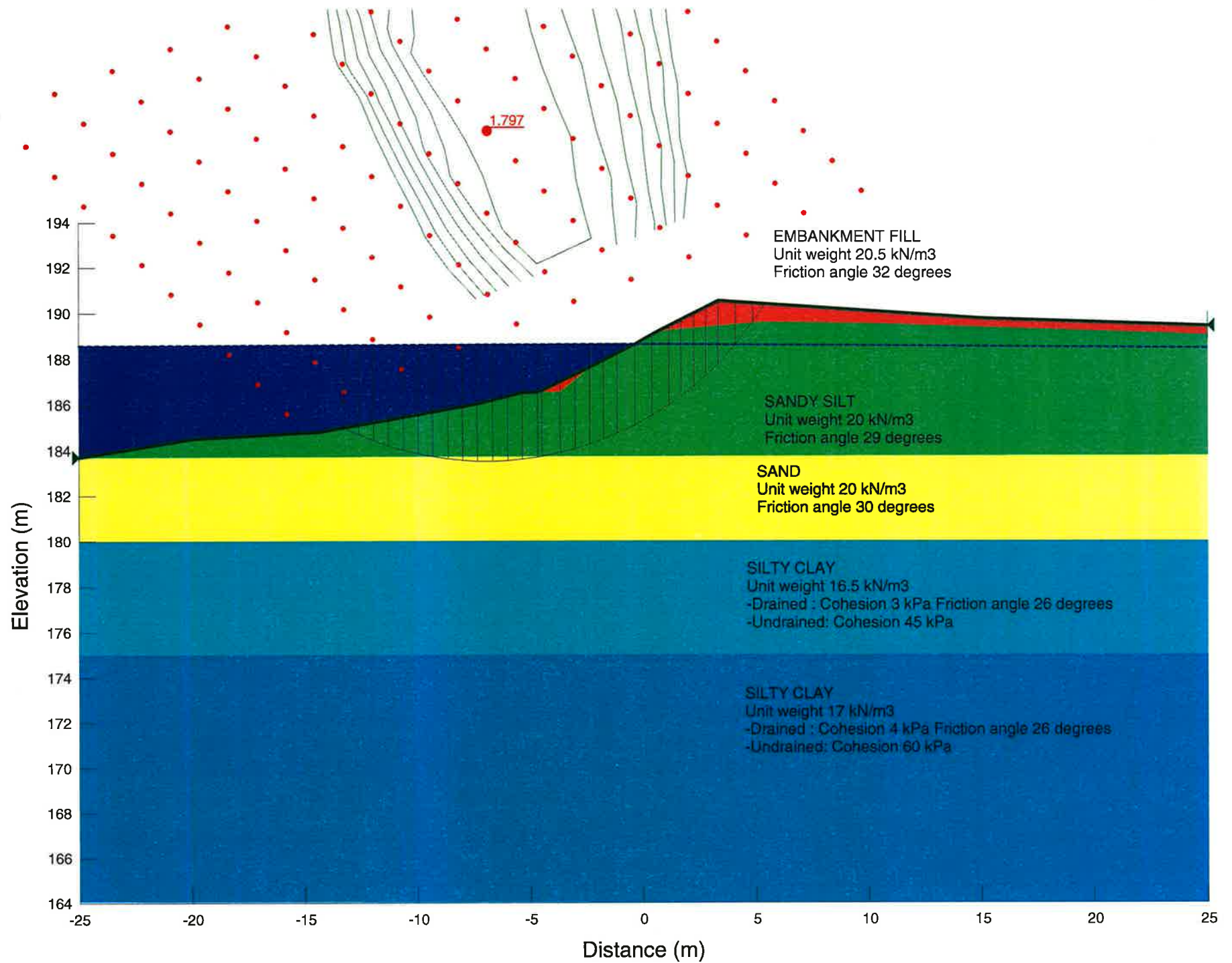


Figure 7. West Abutment - undrained analysis with high water level

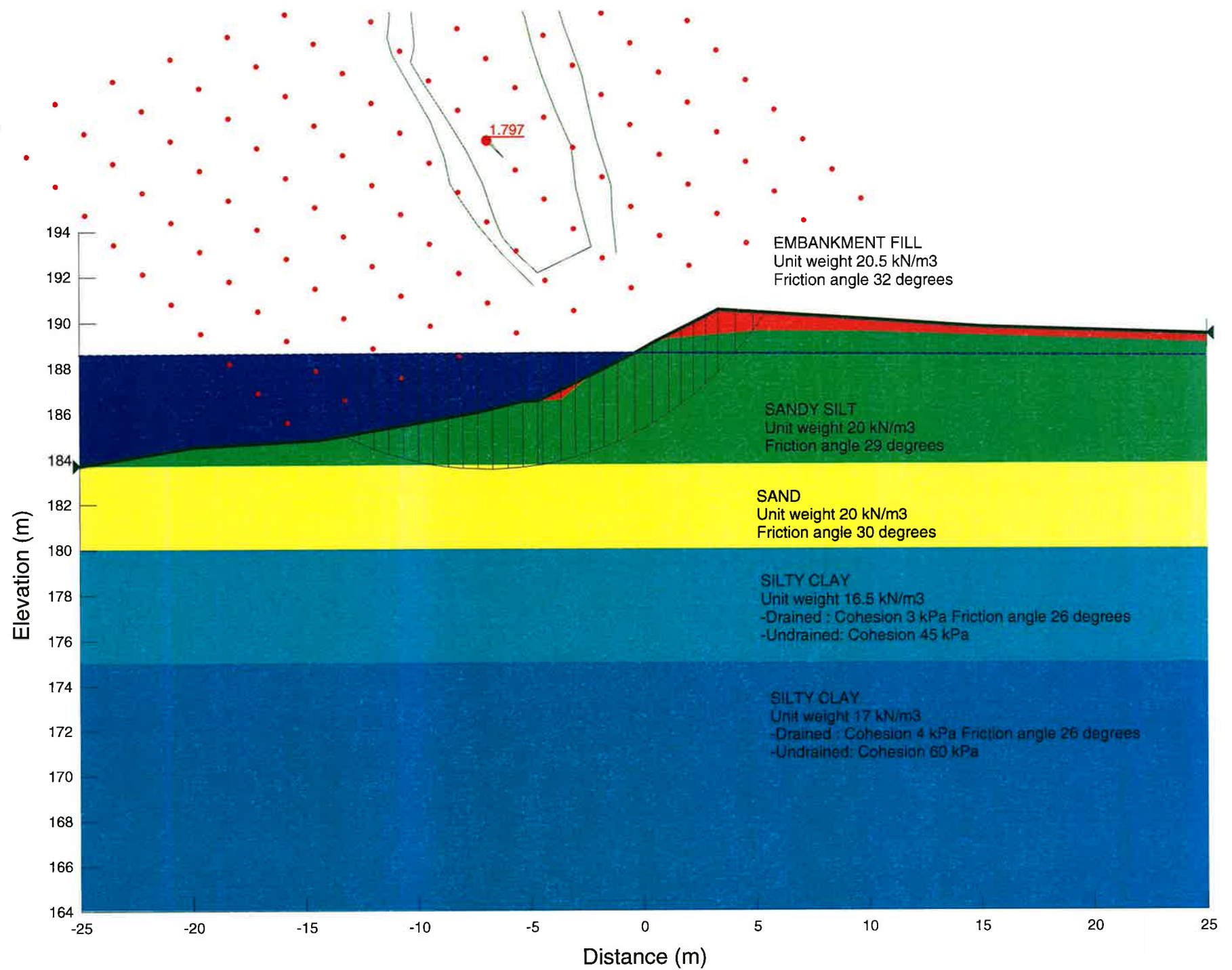


Figure 8. West Abutment - drained analysis with high water level

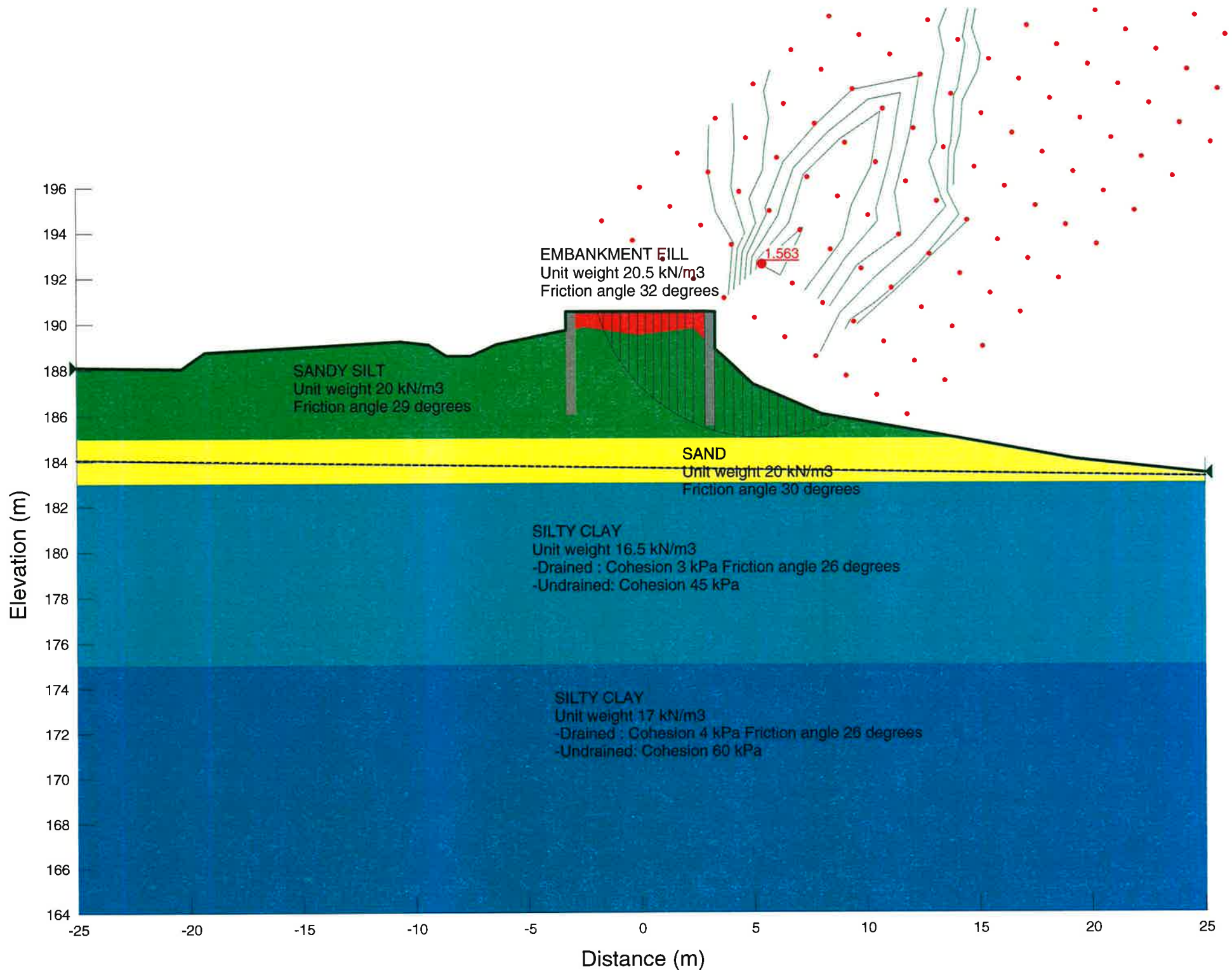


Figure 9. Station 10+295 with normal water level

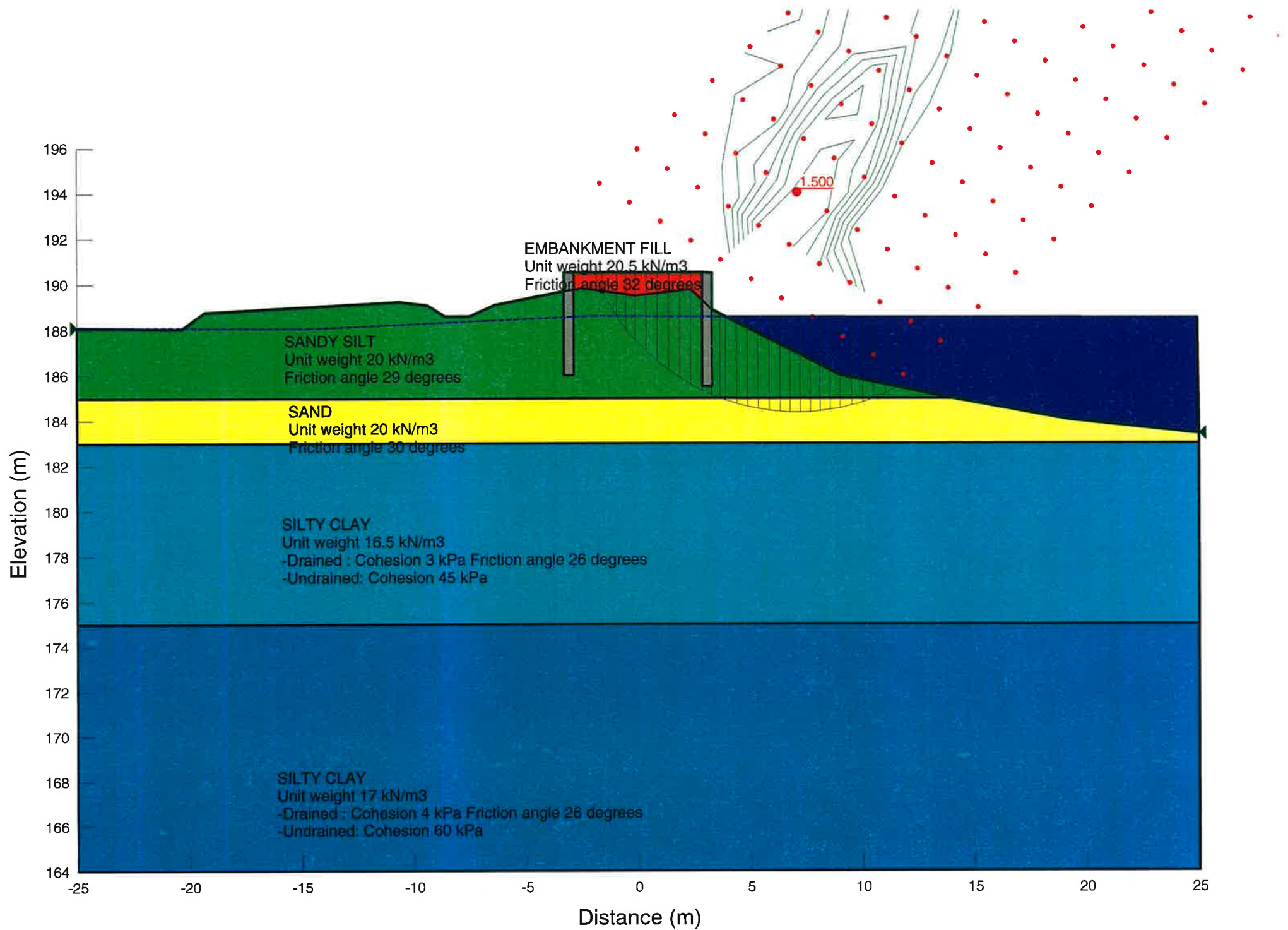


Figure 10. Station 10+295 with high water level (2H:1V side slope)

Appendix J

List of Standard Specifications

OPSD

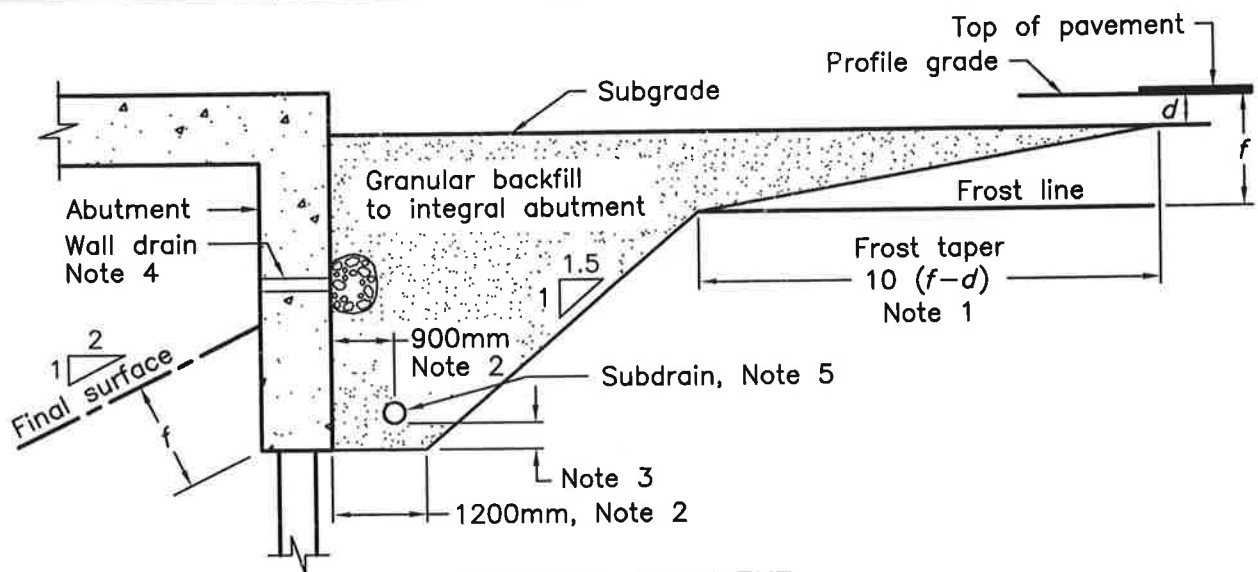
- OPSD 3101.150 – WALLS, ABUTMENT, BACKFILL, MINIMUM GRANULAR REQUIREMENT (included)
- OPSD 3101.200 – WALLS, ABUTMENT, BACKFILL, ROCK (included)

OPSS

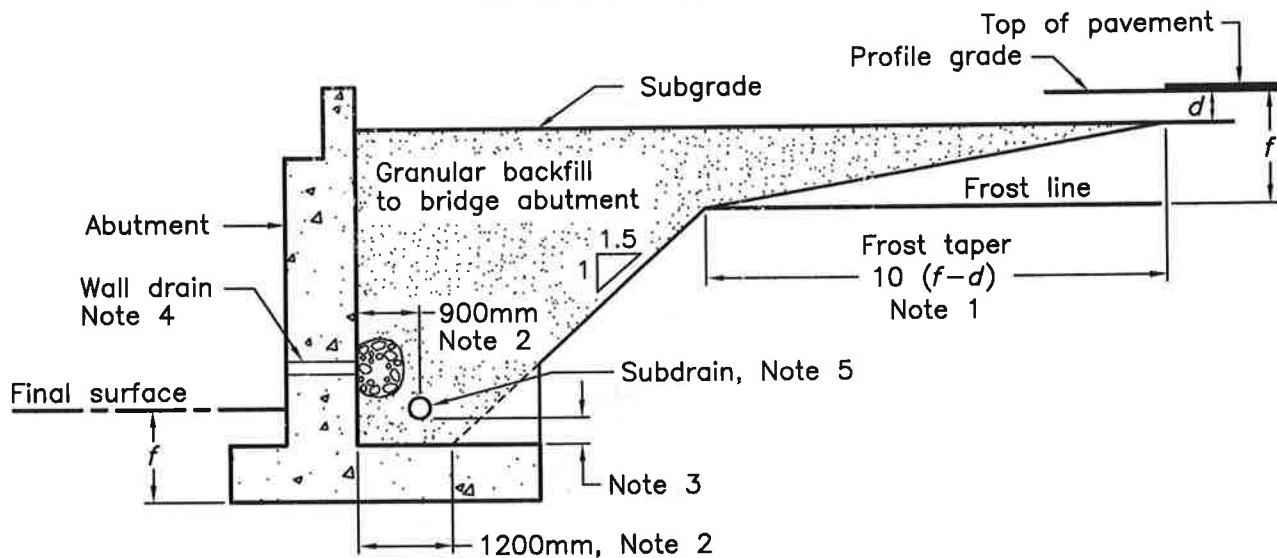
- OPSS 1004 MATERIAL SPECIFICATION FOR AGGREGATES – MISCELLANEOUS (not included)

SP

- SP 105S19 – PROTECTION SYSTEMS (not included)
- SP 902S01 – EXCAVATION AND BACKFILLING TO STRUCTURES (not included)



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.

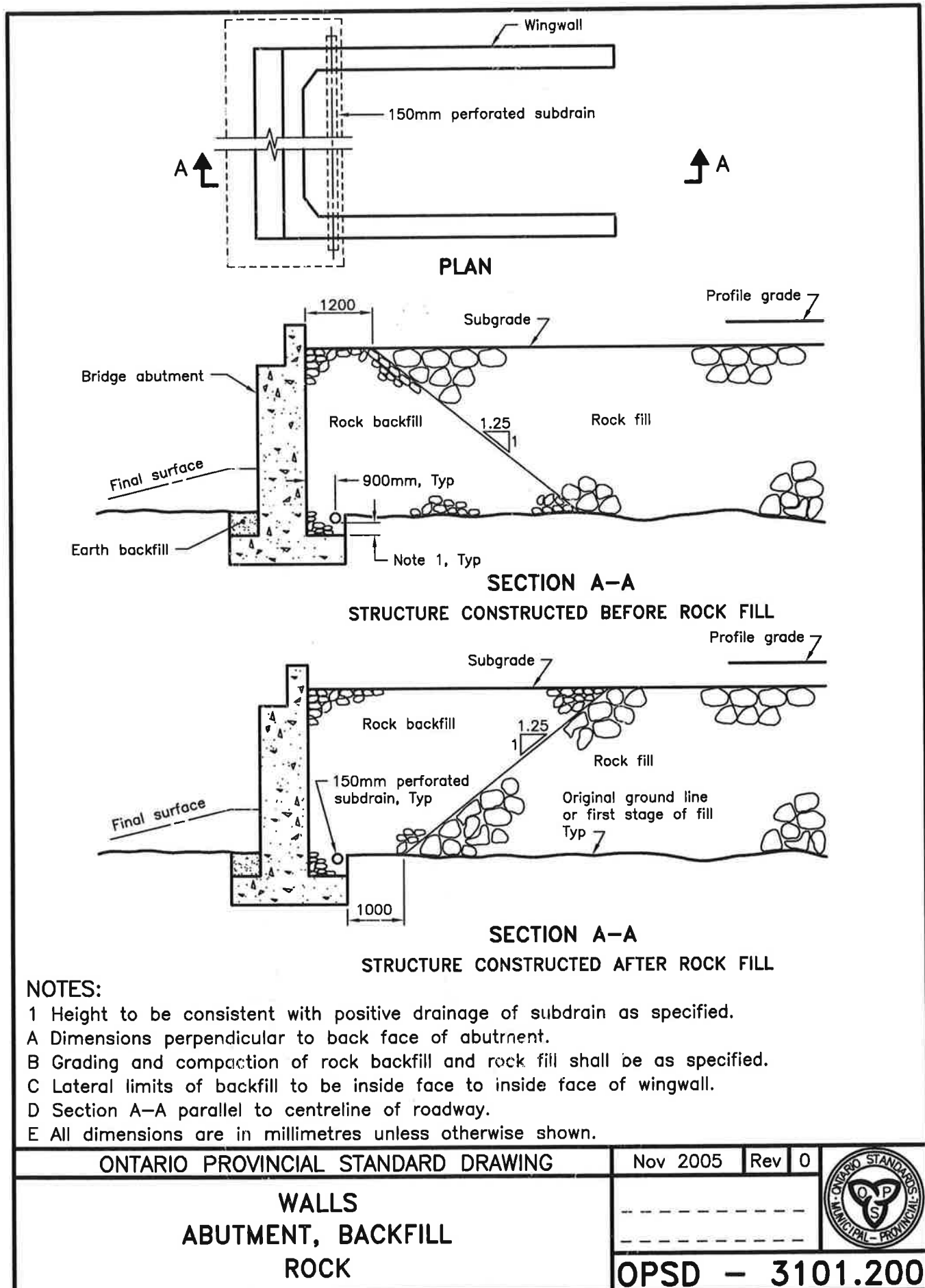
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 Rev 0

WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

OPSD - 3101.150





Appendix K

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.