

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS, REPLACEMENT OF
PORTAGE CREEK CULVERT, HIGHWAY
124, TOWNSHIP OF MCDOUGALL,
ONTARIO, CONTRACT NO. 2009-5129
G.W.P. 5176-06-00, GEOCRETS NO. 41H-70**

D. M. Wills Associates Limited

Project: SPT1230
June 16, 2009

June 16, 2009

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

Attention: Mr. Michael Lang, P.Eng.

Dear Sirs:

**RE: Foundation Investigation and Design Reports, Replacement of Portage Creek Culvert,
Highway 124, Township of McDougall, Ontario Contract No. 2009-5129, G.W.P. 5176-06-00,
GEOCRES No. 41H-70**

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

For and on behalf of Coffey Geotechnics Inc.



Ramon Miranda, P.Eng.
Manager, Transportation Division

Attachment A: Attachments

**FOUNDATION INVESTIGATION
REPLACEMENT OF PORTAGE CREEK
CULVERT, HIGHWAY 124,
TOWNSHIP OF MCDOUGALL, ONTARIO,
CONTRACT NO. 2009-5129
G.W.P. 5176-06-00, GEOCREC NO. 41H-70**

D. M. Wills Associates Limited

Project: SPT1230
June 16, 2009

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**FOUNDATION INVESTIGATION
REPLACEMENT OF PORTAGE CREEK CULVERT, HIGHWAY 124,
TOWNSHIP OF MCDOUGALL, ONTARIO
CONTRACT NO. 2009-5129
G.W.P. 5176-06-00**

1 INTRODUCTION

The Portage Creek culvert is located on Highway 124 about 2.9 km east of Highway 69 in the Geographical Township of McDougall.

Coffey Geotechnics Inc. (Coffey) was retained by D. M. Wills Associates Limited to carry out a foundation investigation at the site of the proposed replacement of the existing Portage Creek culvert under Highway 124. The culvert provides for water flow in the Portage Creek beneath Highway 124.

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

The findings of the investigation are presented in this report.

2 SITE DESCRIPTION AND PHYSIOGRAPHY

The Portage Creek culvert is located on Highway 124 about 2.9 km east of Highway 69 in the Township of McDougall.

The topography near the site is of a rolling nature, with occasional knobs resulting from bedrock outcrops but the topography close to the Portage Creek culvert, which connects the water flow of Portage Lake, is relatively flat.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, this project site is located within the Physiographic Region known as the Georgian Bay Fringe. This Physiographic Region is characterized by very shallow soil and bare rock knobs and ridges. However, the distribution and thickness of glacial drift in the Parry Sound area varies greatly. Glaciolacustrine deposits consist of valley-fill sands and gravel and fine-grained sediment rhythmites and are scattered throughout the Parry Sound area. The Quaternary deposits found near to the project site are mostly prodeltaic lake bottom silt, clay and sand (laminated to varved) from glaciolacustrine origin. As well, organic deposits consisting of peat, mucks and wetlands are found at the site. Bedrock outcrop is also visible near the site.

According to Geological Highway Map, Southern Ontario (Map 2418), the bedrock underlying this area consists of middle or late Precambrian Metavolcanic rocks (i.e. approximately 1300 million years old), primarily flows, tuff, agglomerate, breccia, minor iron formation and metasediments.

3 FIELD AND LABORATORY WORK

The fieldwork for this project was performed during the periods of September 18 to 19, 2008 and January 11 to 15, 2009 and, as requested by MTO, consisted of drilling and sampling seven boreholes (F1, F3, F4, F5, F6, F7 and F8).

On September 18 to 19, 2008, one borehole (F6) was drilled about 5 m west of the existing culvert, to a depth of 28.0 m (El. 167.6 m) below the existing ground surface. Borehole F6 was started using hollow stem continuous flight auger but rock fill was encountered at a depth of 1.3 m below the existing ground surface and the borehole was further advanced by NQ coring to the end of the rock fill to a depth of 5.6 m below the ground surface. Below 5.6 m the borehole was advanced by wash boring using N-size casing. Dynamic Cone Penetration Tests (DCPT) were performed from the bottom of the borehole to refusal at a depth of 32.7 m below the existing ground surface. Landcore Drilling of Chelmsford, Ontario carried out the drilling, testing and sampling work, under the direction and supervision of a Professional Engineer from Coffey. Upon its completion, borehole was backfilled with a mixture of auger cuttings and hole-plug, as per MTO procedures.

During the period of January 11 to 15, 2009, six boreholes (F1, F3, F4, F5, F7 and F8) were put down. These six boreholes were advanced from the water surface of the creek from a raft by wash boring methods and using H-size casing to depths of 13.1 to 18.8 m below the top of the raft. Walker Drilling of Barrie, Ontario carried out the drilling, testing and sampling work, under the direction and supervision of technical personnel from Coffey.

The locations of the boreholes are shown on the Borehole Location Plan Drawing No. 1.

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

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

As mentioned above, a Dynamic Cone Penetration Test (DCPT) was performed from the bottom of Borehole F6 at 28.0 m to refusal at a depth of 32.7 m (El. 162.9 m). In this test, a 51 mm diameter, 60-degree apex cone, screw attached to the tip of an A-size rod, is driven into the ground, using the same driving energy as the SPT method. By recording the number of blows of the hammer to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of soil compactness condition is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT 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 force effects which in some cases affect the SPT results.

The borehole locations were established in the field by Coffey engineering staff, in relation to the existing features and the elevation differences of creek water from the existing culvert obvert elevation were measured by Coffey engineering staff as well during the second investigation period. Geodetic elevations of the existing culvert obvert and Borehole F6 are provided to us by client. Based on these measured elevation differences and the obvert geodetic elevation of the existing culvert, elevations of boreholes on a raft at the time of investigation were determined.

Water level observations in the open boreholes (or casing) were made during the drilling and at completion of each borehole.

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

4 SUMMARIZED SUBSURFACE CONDITIONS

In the culvert area, the elevation of the centreline of Highway 124 is at approximately 202 m some 400 m south of the culvert location, gradually dropping to El. 195.7 m on top of the culvert and then rising northerly to about El. 202 m some 300 m beyond the culvert location. At the time of our investigation, the water in the Portage Creek was at about El. 192.5-192.6 m in January 2009. Borehole F6 was drilled from the shoulder of the Highway adjacent to the culvert while the remaining six boreholes were put down from a raft in the creek. The ground elevations at the bottom of the Creek at the locations of these boreholes were estimated to range from about 191.6 m (Borehole F8) to about 190.6 m (Borehole F4). The depth of water in the Creek ranged from about 0.9 m (Borehole F8) to 2.0 m (Borehole F4).

Borehole F6, drilled from the top of the embankment, contacted an embankment fill which extends to a depth of 5.6 m below the top of the road shoulder or to El. 190.0 m. The upper 1.3 m of the embankment fill consists of granular materials underlain by rock fill.

In the boreholes drilled from the creek, organic soils were contacted which extended to variable depths ranging from about 0.2 m to 1.9 m below the creek bottom. In general, below the embankment fill in Borehole F6 and the organic soils in the remaining boreholes, the site is underlain by a major silty clay deposit with some clayey silt and silt interbeds. The deposit typically extends to about El. 177-176 m or about 14 m below the bottom of the creek and is underlain in Borehole F6 by fine-grained granular soils (i.e. silts, sandy silt to silty sands). The borehole was terminated at a depth of 28.0 m below the road level (or about 23 m below the bottom of the creek at El. 167.6 m, below which depth a Dynamic Core Penetration test (DCPT) was performed). The test gave evidence of the presence of a more competent stratum at about El. 163 m where refusal was encountered. Water level in the borehole was recorded upon its completion at about El. 193 m.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. An inferred stratigraphic section and profile of each side of embankment are shown in Drawing Nos. 2, 3 and 4. The following description of the individual soil strata is to assist the designers

of the project with an understanding of the anticipated subsurface conditions underlying the site. It should be noted that the soil conditions may vary in between and beyond borehole locations.

4.1 Fill

4.1.1 Granular Fill

Borehole F6, which was put down from the top of the embankment (road shoulder), contacted a 1.3 m thick granular fill (typically sand with some gravel underlain by sand and gravel). Standard Penetration tests performed in the granular fill recorded N-values which ranged from 12 to 48 blows/0.3 m, indicating a compact to dense relative density.

A grain-size distribution analysis performed on a sample from the lower zones of the granular fill yielded the following results, as shown in Figure B-1 in Appendix B:

Gravel:	44%
Sand:	46%
Silt & Clay:	10%

4.1.2 Rock Fill

Underlying the granular fill, Borehole F6 contacted an embankment rock fill which extends to a depth of 5.6 m below the road shoulder level or to El. 190.0 m.

4.2 Organic Soils

Underlying the Creek bed, Boreholes F1 and F3 contacted a 1.9 m thick organic silt layer, extending to El. 189.0 and 188.8 m, respectively. This deposit is considered to be a basically fine-grained granular soil and N-values of 0 (i.e. the sampler sank under its own weight and the weight of the drill rods) to 2 blows/0.3 m were recorded, indicating a very loose relative density. Natural moisture contents of samples from the deposit were measured to range from 32 to 160%.

Due to their organic nature, these materials can be considered to be weak and highly compressible, as well as exhibiting 'secondary consolidation.'

In addition, a 0.2 m thick layer of organic silt to clayey silt was contacted in Boreholes F7 and F8, immediately below the creek bed. As well the top 0.3 ±m of the soil in most of the remaining boreholes was found to be organic rich.

4.3 Silty Clay

Underlying the organic silt or organic rich soils in the creek bed and the rock fill (in Borehole F6), the boreholes show that the site is underlain by a massive silty clay deposit. This cohesive deposit was contacted at depths 0.2 to 1.9 m below the creek bed and was found to extend to depths of about 14.5 and 15.5 m below the creek bottom or to El. 176.1 and 176.6 m in Boreholes F4 and F6, respectively. The remaining boreholes were terminated within this deposit at depths ranging between about 11 and 15 m below the creek bottom or at El. 179.8 – 176.6 m.

The deposit contains some clayey silt, silt, sandy silt and silty sand layers/seams/interbeds and pockets as will be further discussed.

The grain-size distribution of seven samples from the clay deposit is given in an envelope in Figure B-2 in Appendix B. The following grain-size distribution is indicated:

Gravel:	0 %
Sand:	1 – 6 %
Silt:	28-54 %
Clay:	44-68 %

Atterberg limits tests performed on 15 samples in the laboratory indicated the following index values:

Liquid Limit:	26 – 51 %
Plastic Limit:	17 – 27 %
Plasticity Index:	9 - 27

These values are characteristic of clayey soils of low to high but generally of medium plasticity, as shown in Figure B-3 in Appendix B. The measured natural moisture contents are generally in excess of the measured liquid limits indicating a typically weak and compressible soil type.

Standard Penetration tests conducted in silty clay deposit gave N-values which typically range from 0 to 4 blows/0.3 m. Undrained shear strengths as measured by field vane tests varied from about 14 kPa to about 100 kPa but typically between 16 and 40 kPa indicating a soft to firm material with some stiff zones. Variation of undrained shear strengths as measured by field vane tests with elevation is given in Figure C-1 in Appendix C.

The measured natural moisture contents were typically about 60% and two bulk unit weight values were determined as 15.5 and 15.7 kN/m³.

Two consolidation (oedometer) tests were carried out in our laboratory and the results are given in Figures B-4 and B-5, in Appendix B. These indicate probable pre-consolidation pressures of about 5 and 17 kPa in excess of the existing overburden pressures (P_o'). The one-dimensional consolidation tests also indicate C_c values of 0.38 and 0.50 (i.e. compressible structure).

As was mentioned before, the deposit contains some clayey silt, silt, sandy silt and silty sand layers/seams/interbeds and pockets. In particular an approximately 1.0 to 1.5 m thick primarily silt zone was contacted in Boreholes F1, F4 and F5 starting at about 2.0 to 2.5 m below the creek bottom (i.e. at about El. 188.6 m to 188.4 m). The grain-size distribution of a sample from the silt deposit from the upper zones (i.e. from about El. 188 m) from Borehole F4 is given in Figure B-6 in Appendix B. The curve indicates 0% gravel, 8% sand, 82% silt and 10% clay particle sizes. Figure B-7 presents the results of Atterberg limit tests performed on the same sample.

Figure B-8 presents the grain-size distribution of a layer encountered in Borehole 6 at about El. 180 m. Less prominent layers/seams/interbeds or lenses were also contacted in many of the boreholes. The grain-size distribution curves for such layers from Boreholes F3, F5 and F8 (samples from Elevations 180.5, 180.0 and 178.8 m, respectively) are given in Figure B-9 in Appendix B. The significance of such layers/interbeds would be, if continuous, in increasing the rate of consolidation of the silty clay deposit under additional stresses.

4.4 Silt/Silty Sand

The frequency of the silt/sandy silt/silty sand seams in the clay deposit was found to increase with increasing depth and in Boreholes F4 and F6 the clay deposit is underlain by silt at about 14.5 to 15.5 m below the bottom of the creek or at El. 176.1 and 176.6 m, respectively. The remaining boreholes were terminated at Elevations ranging from 179.8 to 176.6 m.

Borehole F4 was terminated in the silt deposit at a depth of 16.5 m below the creek bottom level or at El. 174.1 m while in Borehole F6, which was extended deeper, the silt is underlain by silty sand to silty sand to sandy silt deposits to the termination elevation of 167.6 m.

The grain-size distribution of a sample from these fine-grained granular soils is presented in Figure B-10 in Appendix B.

These units are considered to have higher permeability (i.e. more pervious) in comparison with the overlying practically impervious silty clay deposit. Standard Penetration tests performed yielded N-values which range from 7 to 14 blows/0.3 m which indicate a loose to compact condition.

4.5 Groundwater Conditions

As Boreholes F1, F3, F4, F5, F7 and F8 were put down in the creek from a raft by washboring methods, no groundwater level observations could be made in these boreholes.

In Borehole F6, which was put down from the top of the embankment, the water level was measured upon its completion at a depth of 2.4 m or El. 193.2 m. This may not however represent the stabilized water level.

In our opinion, the groundwater level at the site would be controlled by the water level in the creek, which, at the time of our investigation in January 2009 was at about El. 192.5 m. It should however be pointed out that this would be subject to fluctuations.

It should also be mentioned that while drilling in Borehole F6, soil back-up was noted in the casing starting at about El. 171.5 m (and below) indicating a mild uplift (i.e. excess hydrostatic) pressure in the fine-grained granular soils underlying the clay deposit.

For and on behalf of Coffey Geotechnics Inc.



Gwangha Roh, Ph.D.



Ramon Miranda, P.Eng.



Zuhtu Ozden, P.Eng.



Drawings

METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No. 2009-5129

GWP: 5176-06-00

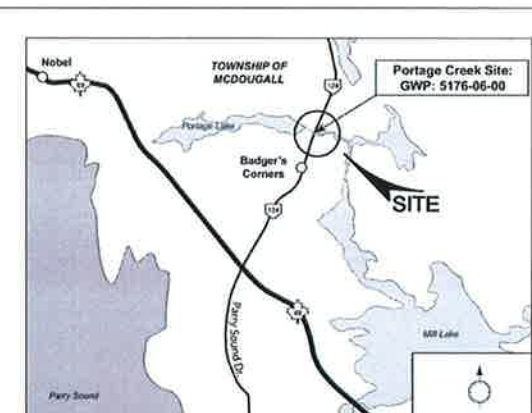
REPLACEMENT OF PORTAGE
CREEK CULVERT
HIGHWAY 124 (STATION 12+827)
BOREHOLE LOCATION PLAN



SHEET

15

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Borehole & Cone

No.	ELEV.	STATION No.	OFFSET
F-1	193.0*	12+847	14.7m Lt C/L
F-3	192.8*	12+844	14.4m Rt C/L
F-4	192.9*	12+824.9	14.4m Rt C/L
F-5	192.9*	12+830.8	13.4m Lt C/L
F-6	195.6	12+823.2	4.1m Lt C/L
F-7	192.9*	12+812	14.6m Lt C/L
F-8	192.8*	12+807	14.7m Rt C/L

*borehole elevation measured @ top of the raft

NOTE

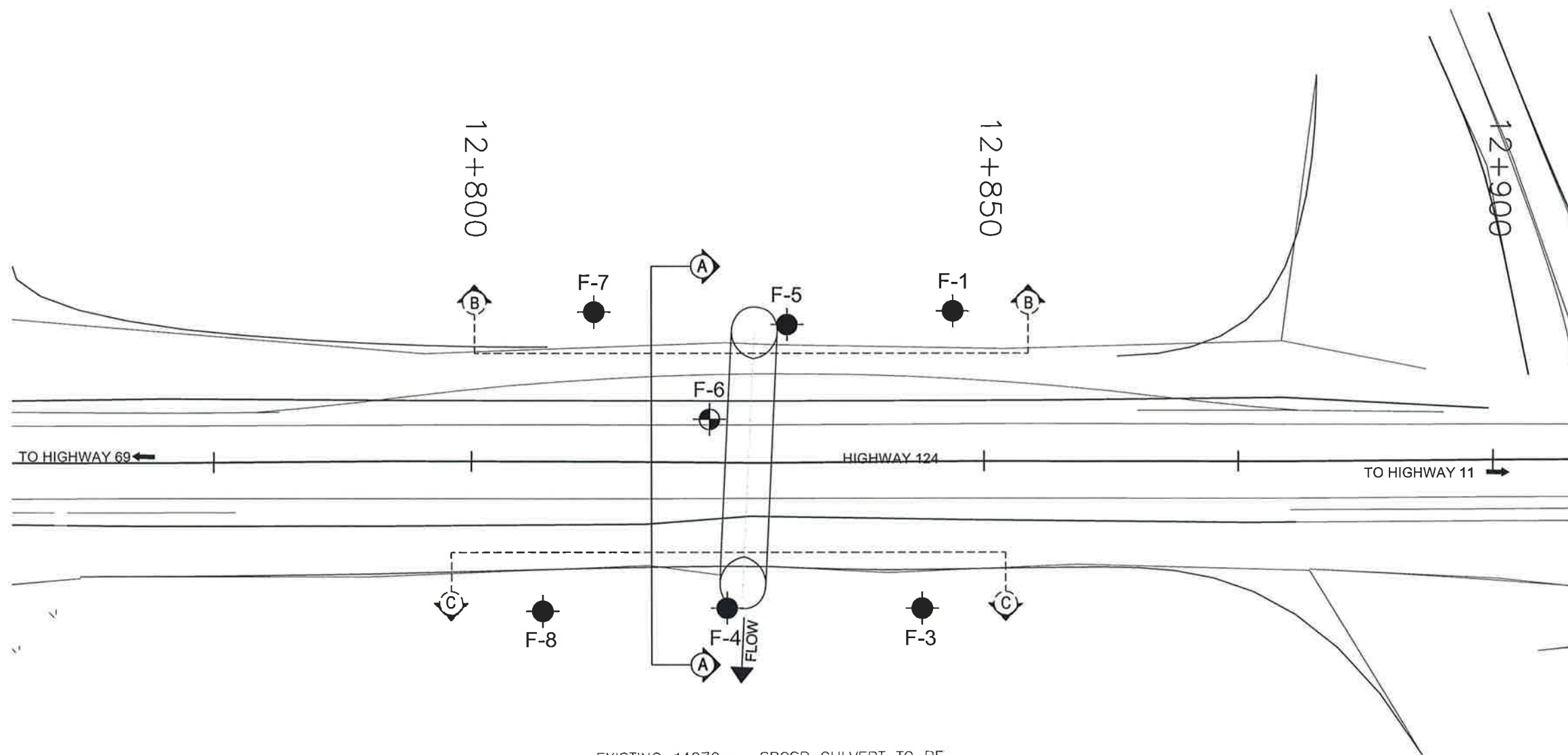
The boundaries between soil strata have been established only at Borehole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

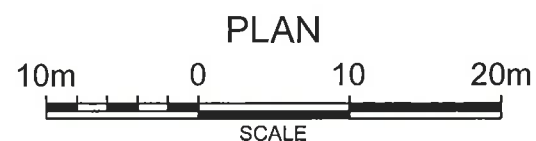
REV.	DATE	BY	DESCRIPTION

Geocres No. 41H-70

SPT 1230			DIST	52
SUBMD	CHECKED	DATE Feb. 2009	SITE	
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG	1



EXISTING $\phi 4270$ mm SPCSP CULVERT TO BE
REPLACED WITH ALUMINUM ALLOY
CORRUGATED PLATE PIPE $\phi 4270$ mm
STA. 12+825



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. 2009-5129

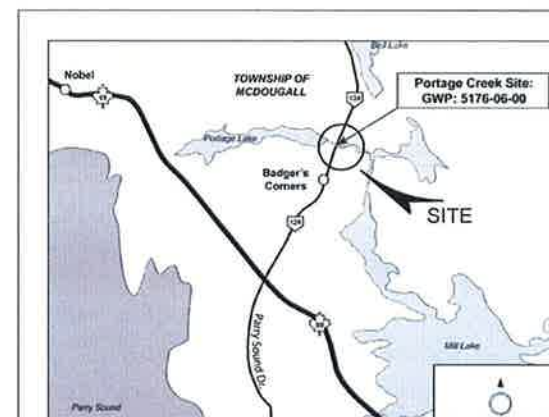
GWP: 5176-06-00

REPLACEMENT OF PORTAGE
CREEK CULVERT
HIGHWAY 124 (STATION 12+827)
CROSS SECTION

SHEET

16

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

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

No.	ELEV.	STATION No.	OFFSET
F-4	192.9*	12+824.9	14.4m Rt C/L
F-5	192.9*	12+830.8	13.4m Lt C/L
F-6	195.6	12+823.2	4.1m Lt C/L

*borehole elevation measured @ top of the raft

NOTE

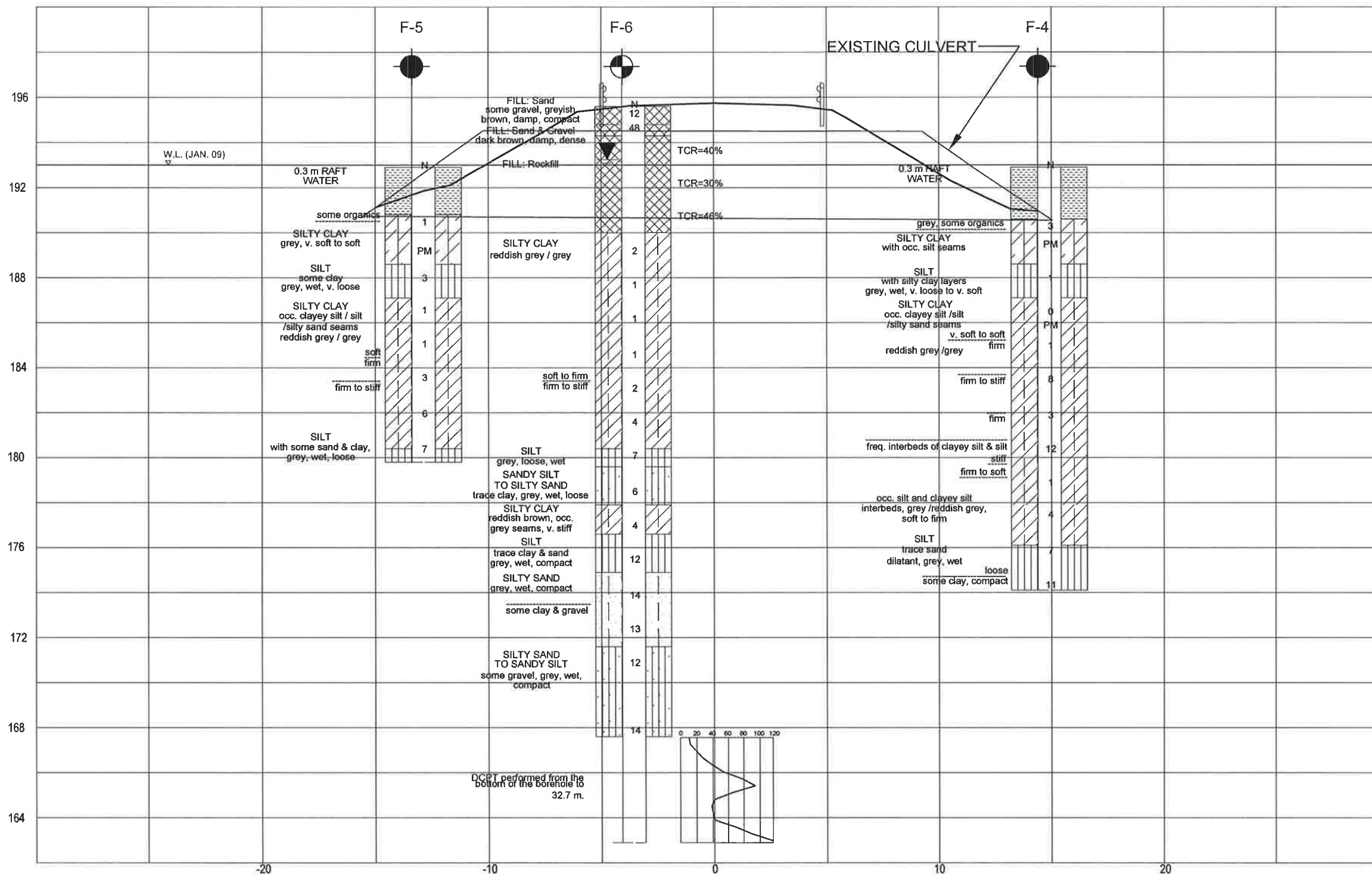
The boundaries between soil strata have been established only at Borehole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

REV.	DATE	BY	DESCRIPTION
------	------	----	-------------

Geocres No. 41H-70

SPT 1230				DIST	52
SUBMD	CHECKED	DATE Feb. 2009	SITE		
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 2		



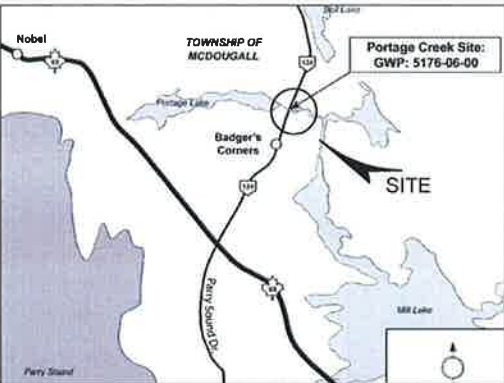
SECTION A-A



HORIZONTAL SCALE



NOTES:
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.



KEY PLAN
N.T.S.

LEGEND

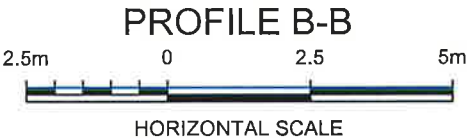
- Borehole & Cone
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	STATION No.	OFFSET
F-1	193.0*	12+847	14.7m Lt C/L
F-5	192.9*	12+830.8	13.4m Lt C/L
F-7	192.9*	12+812	14.6m Lt C/L

*borehole elevation measured @ top of the raft

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

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



REV.	DATE	BY	DESCRIPTION
Geocres No. 41H-70			
SPT 1230			DIST 52
SUBMD	CHECKED	DATE Feb. 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 3

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.

18

KEY PLAN
N.T.S.

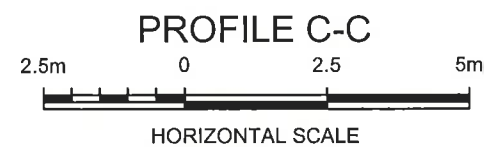
Diagram illustrating the components of a borehole and the locations for water level measurements:

- Borehole**
- 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.	ELEV.	STATION No.	OFFSET
F-3	192.8*	12+844	14.4m Rt C/L
F-4	192.9*	12+824.9	14.4m Rt C/L
F-8	192.8*	12+807	14.7m Rt C/L

≡NOTE≡

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



REV.			
	DATE	BY	DESCRIPTION

Geocres No. 41H-70

SPT 1230			DIST 52
SUBM'D	CHECKED	DATE Feb. 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 4

Appendix A

Record of Borehole Sheets



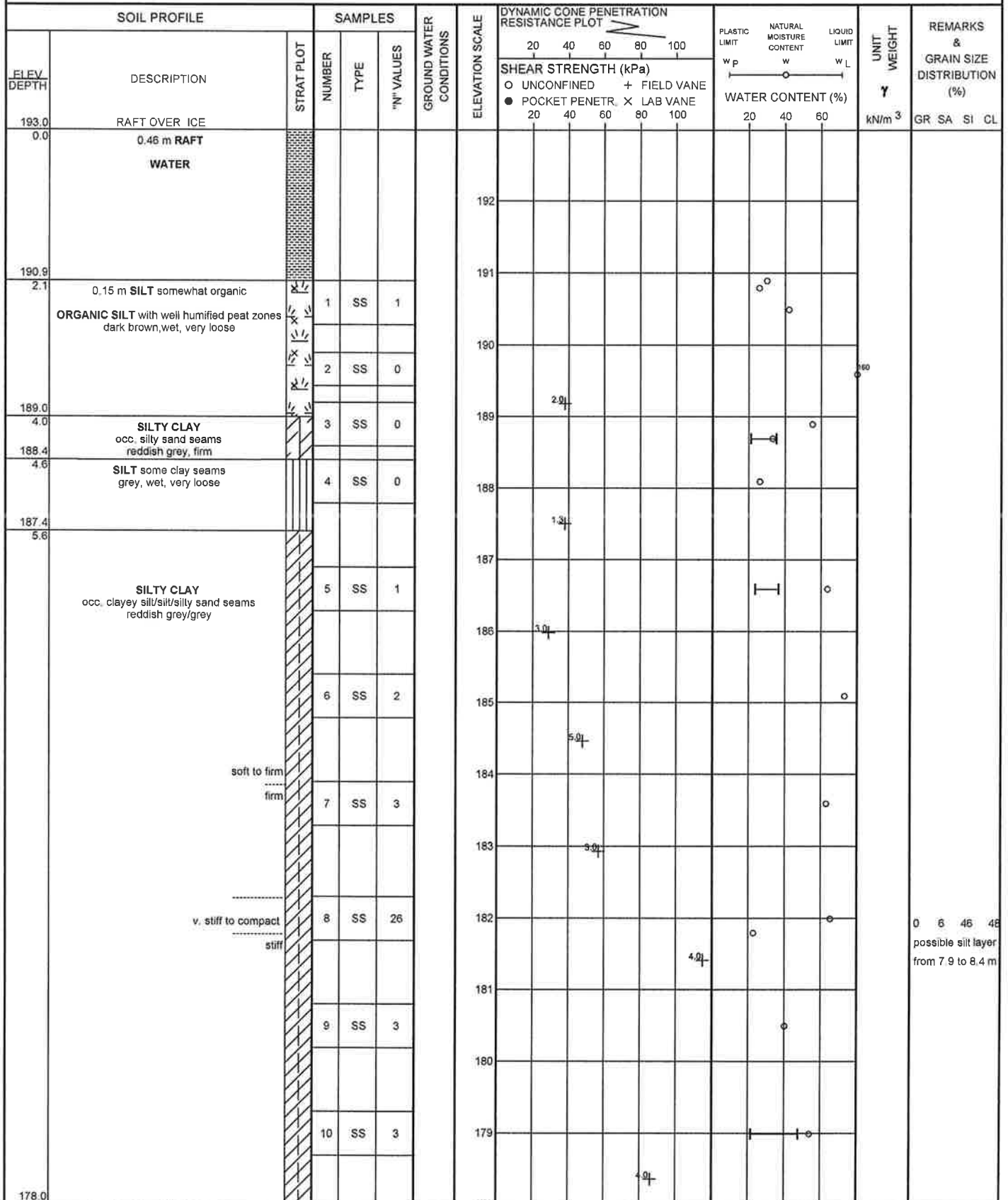
SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-1

1 OF 2

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+847) 14.7 m Lt C/L of HWY 124 (D = -2.4m) ORIGINATED BY Z.I
DIST 52 HWY 124 BOREHOLE TYPE H-Type Wash Boring COMPILED BY Z.I
DATUM Geodetic DATE 1/15/2009 CHECKED BY ZO



+ 3 x 3 Numbers refer to Sensitivity

20 15 10 5 (%) STRAIN AT FAILURE


SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-1

2 OF 2

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+847) 14.7 m Lt C/L of HWY 124 (D = -2.4m) ORIGINATED BY Z.I
DIST 52 HWY 124 BOREHOLE TYPE H-Type Wash Boring COMPILED BY Z.I
DATUM Geodetic DATE 1/15/2009 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE							PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L			
178.0 15.0	SILTY CLAY reddish brown, stiff		11	SS	4													
176.8 16.2																		
	End of Borehole																	

+³, X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

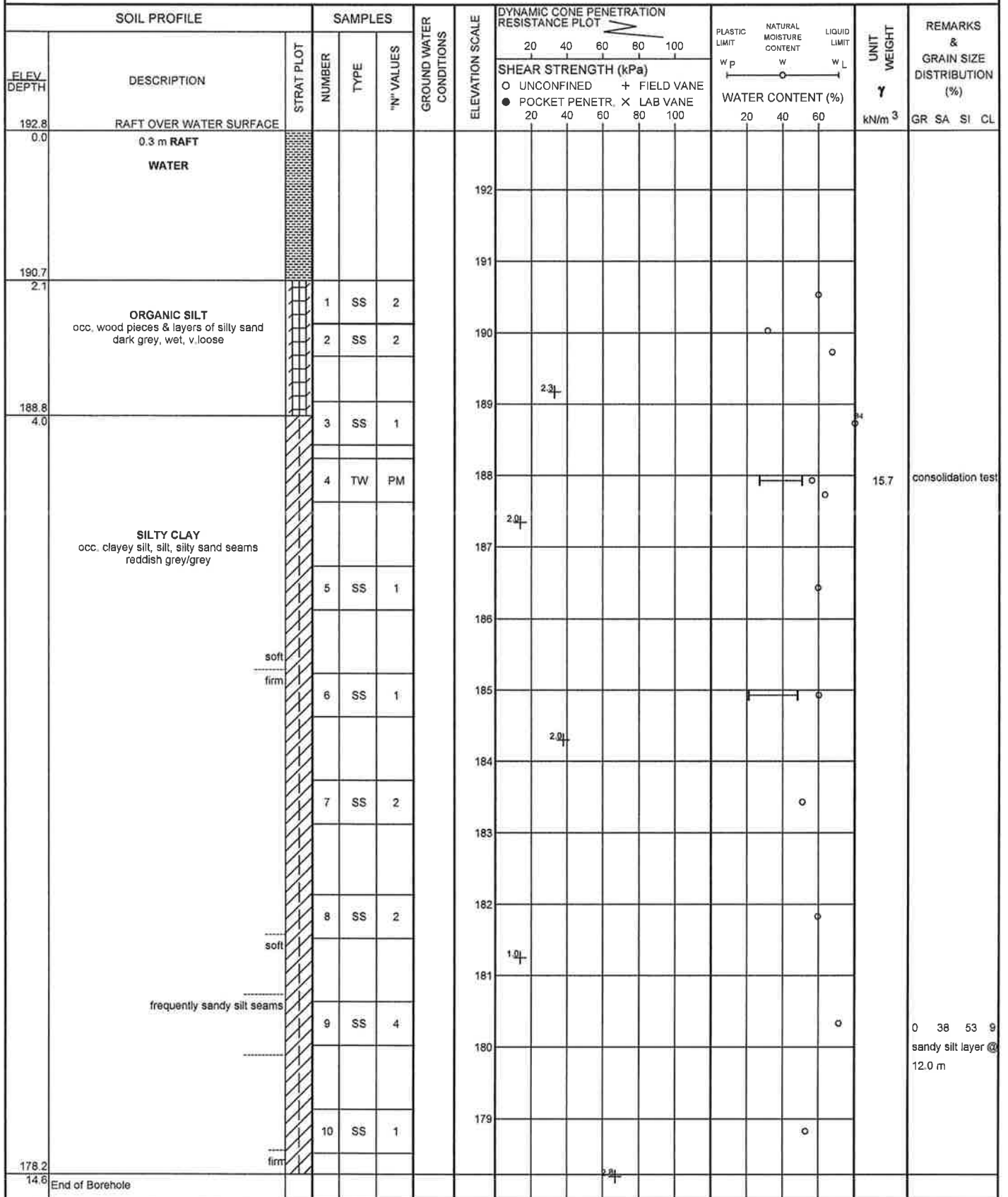
SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-3

1 OF 1

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+844) 14.4 m Rt C/L of HWY 124 (D = -2.4m) ORIGINATED BY Z.I
DIST 52 HWY 124 BOREHOLE TYPE H-Type Wash Boring COMPILED BY Z.I
DATUM Geodetic DATE 1/13/2009 1/14/2009 CHECKED BY ZO



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

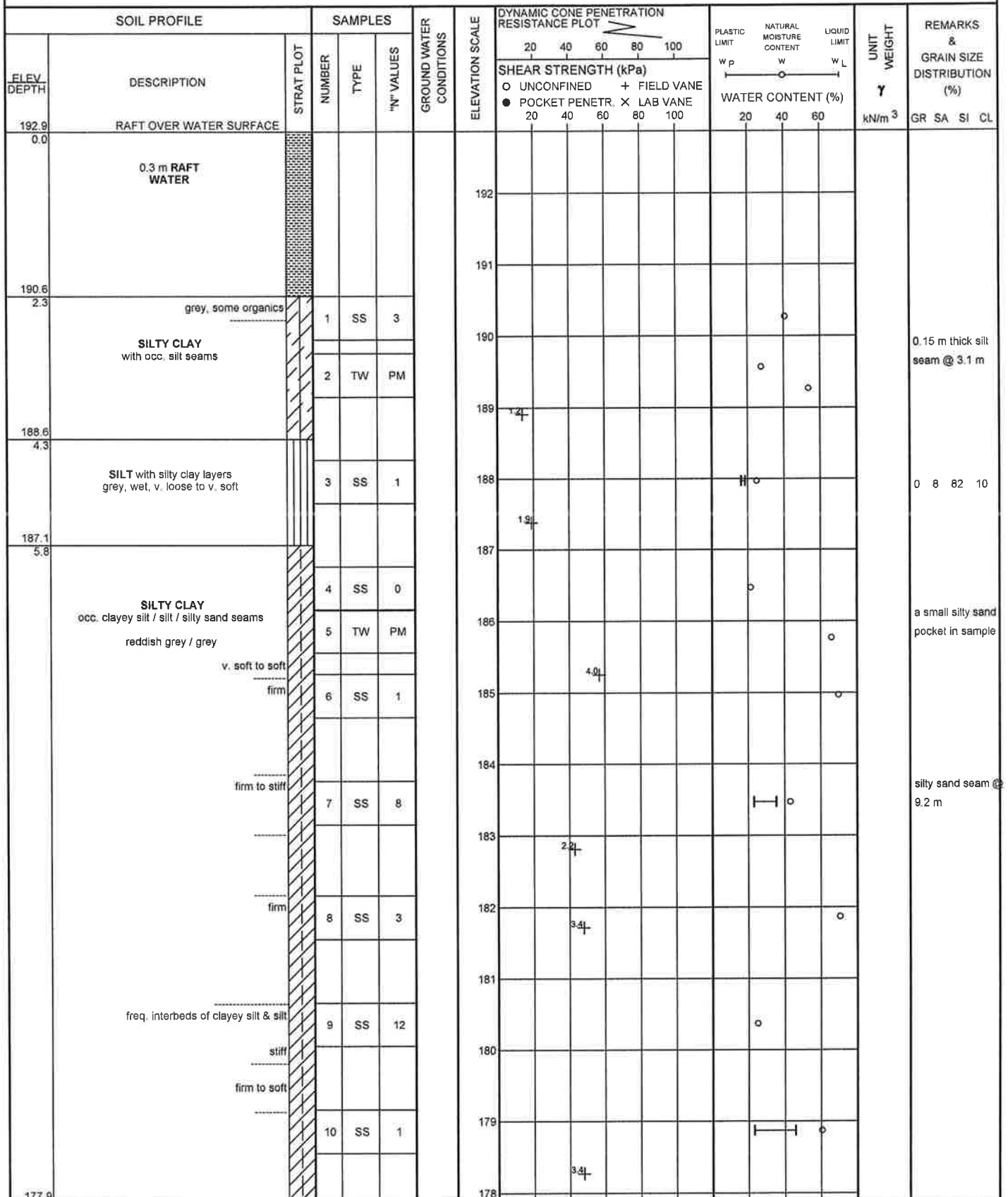
SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-4

1 OF 2

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+824.9) 14.4 m Rt C/L of HWY 124 (D = -2.4m) ORIGINATED BY ZI
DIST 52 HWY 124 BOREHOLE TYPE H-Type Wash Boring COMPILED BY ZI
DATUM Geodetic DATE 1/11/2009 1/12/2009 CHECKED BY ZO



Continued Next Page

+ 3, X 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE





SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-4

2 OF 2

METRIC

GWP 5178-06-00 LOCATION (Sta: 12+824.9) 14.4 m RI C/L of HWY 124 (D = -2.4m) ORIGINATED BY Z.I
DIST 52 HWY 124 BOREHOLE TYPE H-Type Wash Boring COMPILED BY Z.I
DATUM Geodetic DATE 1/11/2009 1/12/2009 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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)									
177.9 15.0	SILTY CLAY occ. silt and clayey silt interbeds grey / reddish grey, soft to firm		11	SS	4		20	40	60	80	100		20	40	60		
176.1 16.8																	
174.1 18.8																	
	SILT trace sand dilatant, grey, wet loose some clay, compact		12	SS	7									20	40	60	
			13	SS	11												
174.1 18.8	End of Borehole																

+³, X³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-5

1 OF 1

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+830.8) 13.4 m Lt C/L of HWY 124 (D = -2.4m) ORIGINATED BY ZI
DIST 52 HWY 124 BOREHOLE TYPE H-Type Wash Boring COMPILED BY ZI
DATUM Geodetic DATE 1/8/2009 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)		WATER CONTENT (%)				
192.9 0.0	RAFT OVER WATER SURFACE						20 40 60 80 100	20 40 60	W _P W W _L				GR SA SI CL	
	0.3 m RAFT WATER													
190.8 2.1	some organics SILTY CLAY grey, very soft to soft		1	SS	1									
			2	TW	PM									
188.6 4.3	SILT some clay grey, wet, very loose		3	SS	3									
187.1 5.8	SILTY CLAY occ. clayey silt / silt / silty sand seams reddish grey / grey		4	SS	1									
	soft ----- firm		5	SS	1									
	firm to stiff		6	SS	3									
			7	SS	6									
180.4 12.5	SILT with some sand with some sand & clay, grey, wet, loose		8	SS	7									
179.8 13.1	End of Borehole													

+³ X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-6

1 OF 3

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+823.2) 4.1 m Lt. C/L of HWY 124 (Sh) ORIGINATED BY SK
 DIST 52 HWY 124 BOREHOLE TYPE Hollow Stem Augering & NW Casing & NQ coring COMPILED BY SS
 DATUM Geodetic DATE 8/18/2008 8/19/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE				
195.6	GROUND SURFACE						20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L		
0.0	FILL: Sand, some gravel greyish brown, damp, compact		1	SS	12							
194.8			2	SS	48							44 46 (10)
0.8	FILL: Sand & Gravel dr. brown, damp, dense											
194.3												
1.3	FILL: Rockfill		3	RC	TCR=40%							
			4	RC	TCR=30%							
			5	RC	TCR=46%							
190.0												
5.6	SILTY CLAY reddish grey / grey		6	SS	2							0 1 52 47
			7	SS	1							
			8	SS	1							
			9	SS	1							0 4 28 68
			10	SS	2							
			11	SS	4							
180.6												

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-6

2 OF 3

METRIC

GWP: 5176-06-00 LOCATION: (Sta: 12+823.2) 4.1 m Lt C/L of HWY 124 (Sh) ORIGINATED BY: SK
 DIST: 52 HWY: 124 BOREHOLE TYPE: Hollow Stem Augering & NW Casing & NQ coring COMPILED BY: SS
 DATUM: Geodetic DATE: 9/18/2008 9/19/2008 CHECKED BY: ZO

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 80 100					
							20 40 60 80 100					
							20 40 60 80 100					
180.6 15.0	SILTY CLAY						180					
180.4 15.2	SILT grey, loose, wet		12	SS	7							0 13 79 8
179.6 16.0	SANDY SILT TO SILTY SAND tr. clay, grey, wet, loose						179					
			13	SS	6							
177.9 17.7	SILTY CLAY red, brown, occ. grey seams, v. stiff						178					
			14	SS	4							
176.6 19.0	SILT tr. clay & sand grey, wet, compact						177					
			15	SS	12							
174.9 20.7	SILTY SAND grey, wet, compact some clay & gravel						176					
			16	SS	14							20 44 (36)
							175					
			17	SS	13							
171.6 24.0	SILTY SAND TO SANDY SILT some gravel, grey, wet, compact						174					0.2 m soil back-u
			18	SS	12							
							173					
							172					
							171					
							170					
							169					
							168					0.3 m soil back-u
167.6 28.0	End of Borehole Water level in open borehole @ 2.4 m (not stabilized)* upon completion. Borehole caved in @ 3.4 m.						167					
							166					

Continued Next Page

+ 3 x 3 Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-6

3 OF 3

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+823.2) 4.1 m Lt. C/L of HWY 124 (Sh) ORIGINATED BY SK
DIST 52 HWY 124 BOREHOLE TYPE Hollow Stem Augering & NW Casing & NQ coring COMPILED BY SS
DATUM Geodetic DATE 9/18/2008 9/19/2008 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 (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH (kPa)	WATER CONTENT (%)			
165.5							20 40 60 80 100	20 40 60				
165												
164												
163												
162.9												
32.7	End of Dynamic Cone Penetration Test (DCPT) DCPT performed from the bottom of the borehole to 32.7 m.											

+³, X³: Numbers refer to Sensitivity

20
15
10
(%) STRAIN AT FAILURE

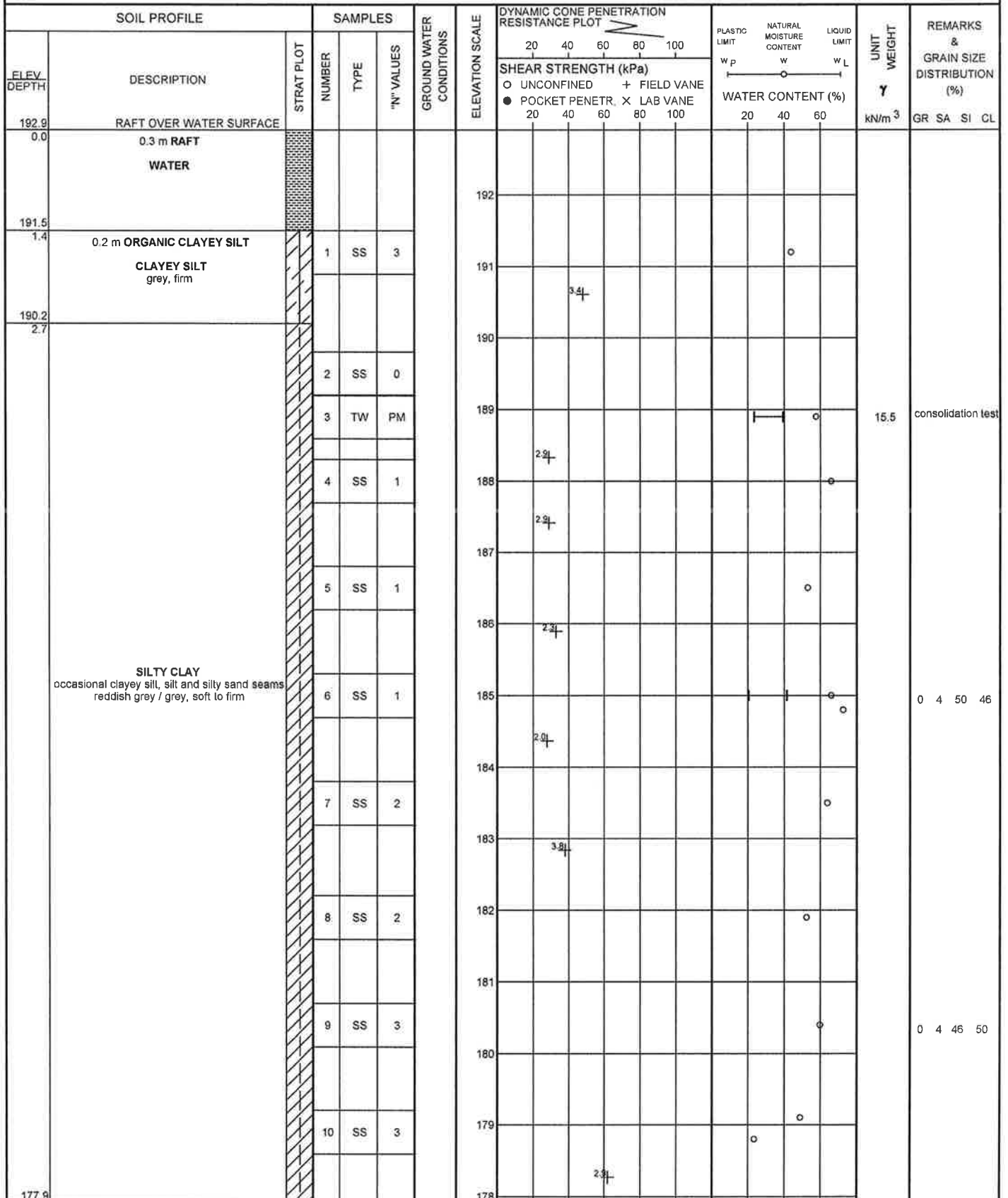
SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-7

1 OF 2

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+812) 14.6 m Lt C/L of HWY 124 (D = -2.4m) ORIGINATED BY Z.I
DIST 52 HWY 124 BOREHOLE TYPE H-Type Wash Boring COMPILED BY Z.I
DATUM Geodetic DATE 1/10/2009 CHECKED BY ZO



Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-7

2 OF 2

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+812) 14.6 m Lt C/L of HWY 124 (D = -2.4m) ORIGINATED BY Z.I
DIST 52 HWY 124 BOREHOLE TYPE H-Type Wash Boring COMPILED BY Z.I
DATUM Geodetic DATE 1/10/2009 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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W P W W L	20 40 60			
177.9 15.0	SILTY CLAY occasional seams of silt / sand grey / reddish grey, firm to stiff		11	SS	4									
176.7 16.2														
16.2	End of Borehole													

+³.X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

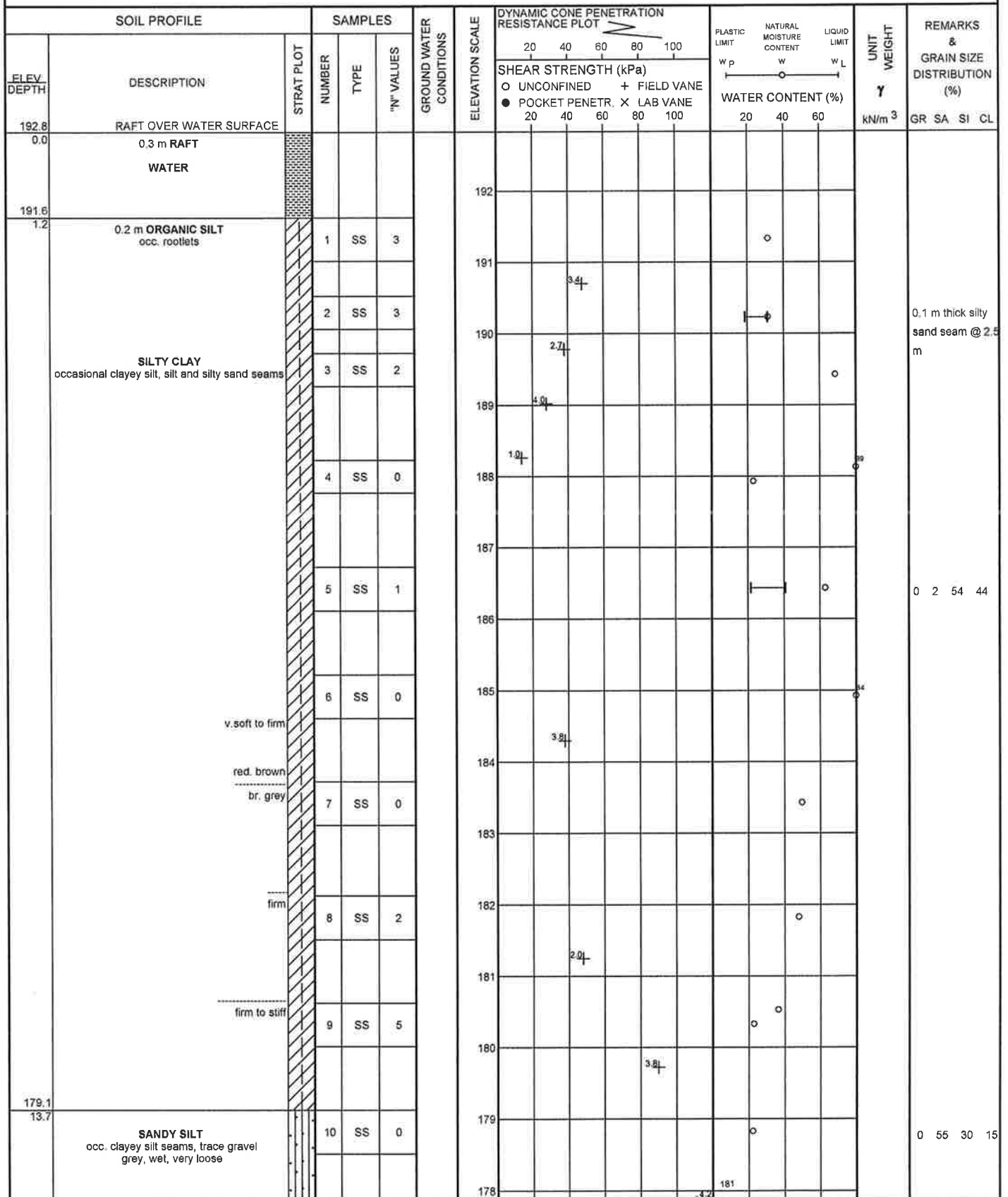
SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-8

1 OF 2

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+807) 14.3 m Rt C/L of HWY 124 (D = -2.4m) ORIGINATED BY Z I
DIST 52 HWY 124 BOREHOLE TYPE H-Type Wash Boring COMPILED BY Z I
DATUM Geodetic DATE 1/12/2009 1/13/2009 CHECKED BY ZO



Continued Next Page

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

SPT1230 : Highway 124 (Portage Creek)

RECORD OF BOREHOLE No F-8

2 OF 2

METRIC

GWP 5176-06-00 LOCATION (Sta: 12+807) 14.3 m Rt C/L of HWY 124 (D = -2.4m) ORIGINATED BY Z.I
DIST 52 HWY 124 BOREHOLE TYPE H-Type Wash Boring COMPILED BY Z.I
DATUM Geodetic DATE 1/12/2009 1/13/2009 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 (%) 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 (%) w _p w w _L				
177.8								20	40	60	80	100			
177.3	SANDY SILT grey, loose, wet		11	SS	5										
15.5	SILTY CLAY reddish grey, firm to stiff														
176.6															
16.2	End of Borehole														

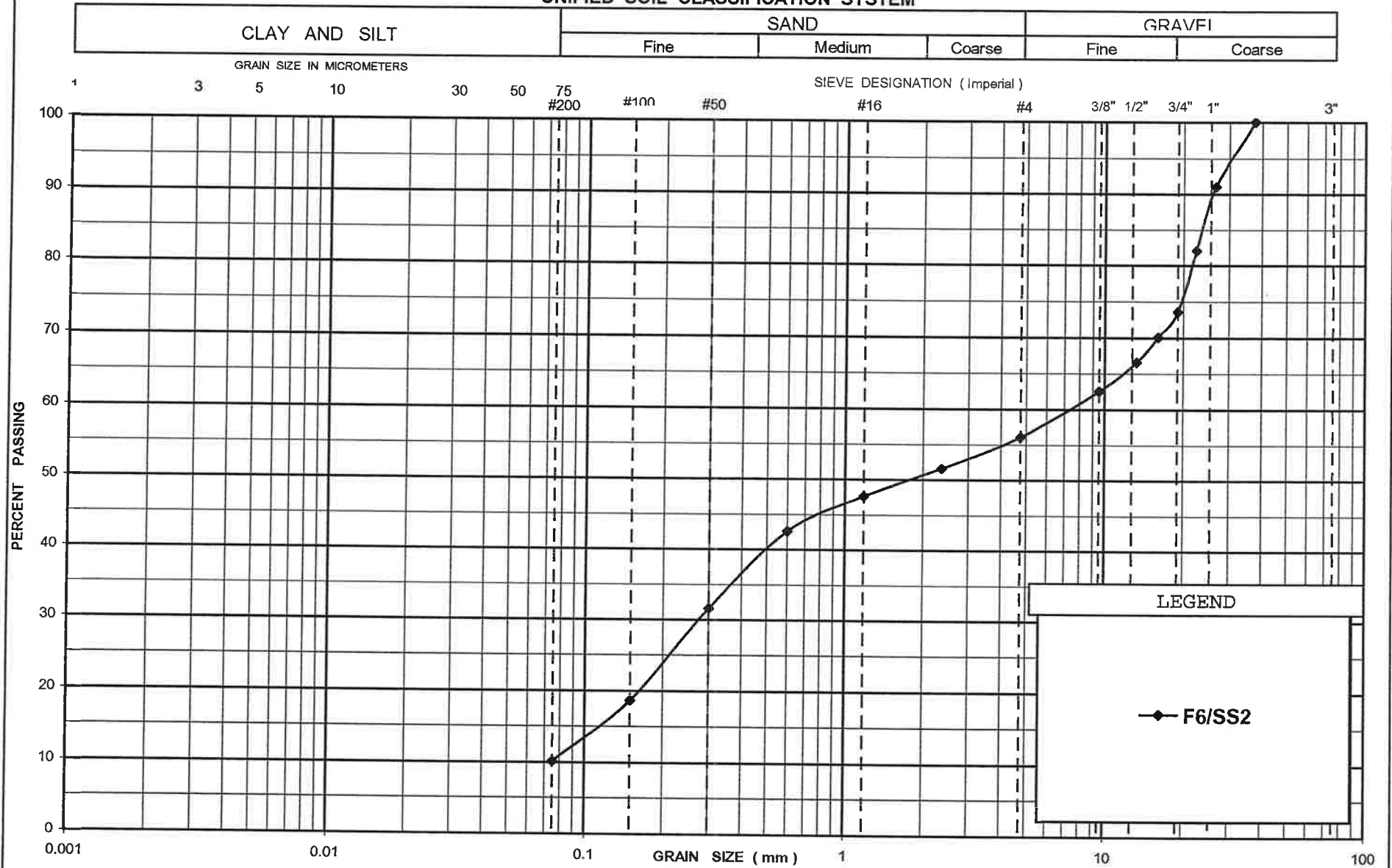
+³, X³: Numbers refer to
Sensitivity

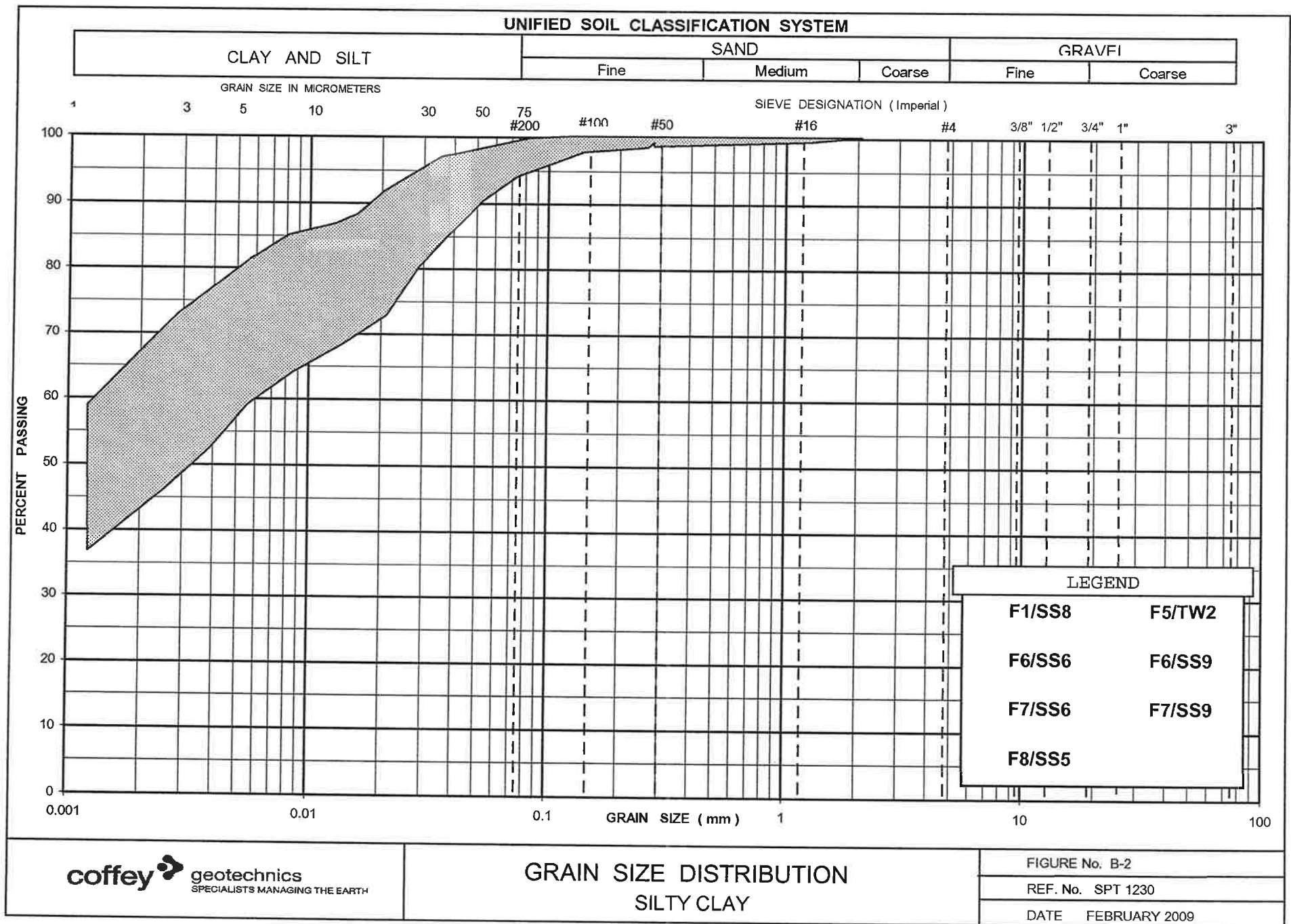
20
15 5
10 (%) STRAIN AT FAILURE

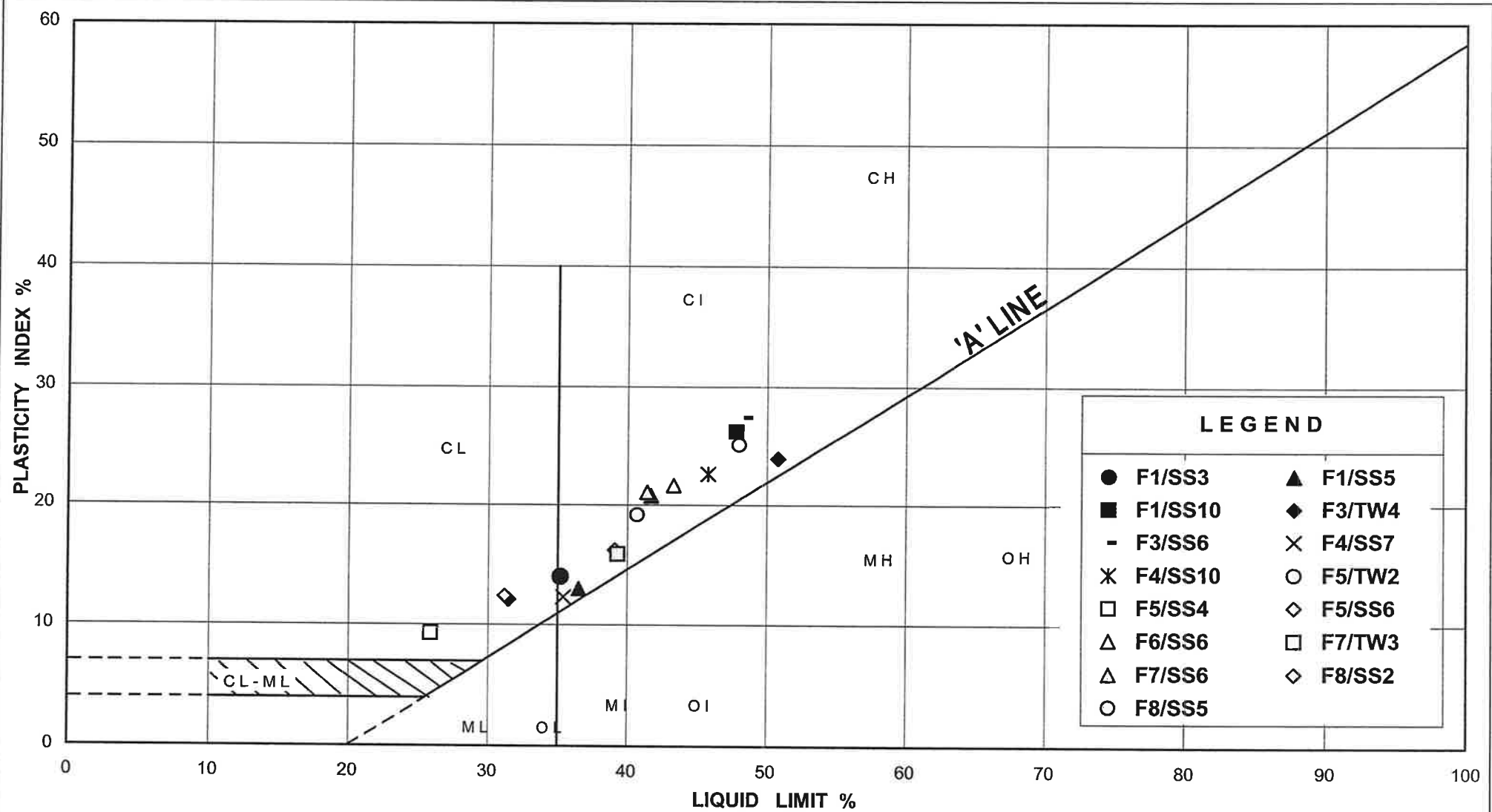
Appendix B

Laboratory Test Results

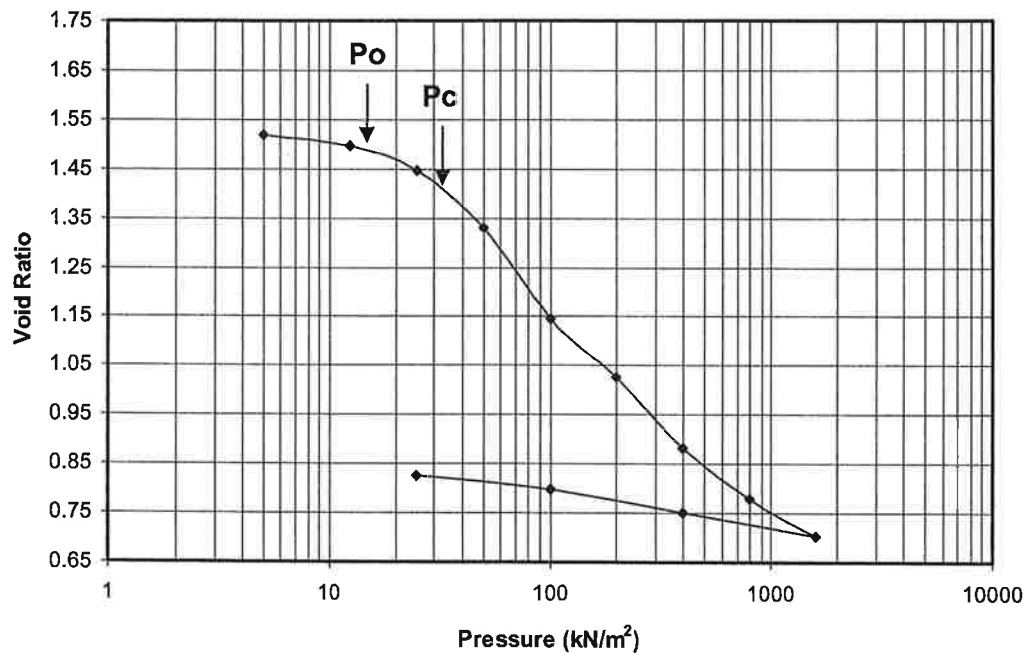
UNIFIED SOIL CLASSIFICATION SYSTEM







Void Ratio versus Pressure



Coefficient of Consolidation vs. Pressure

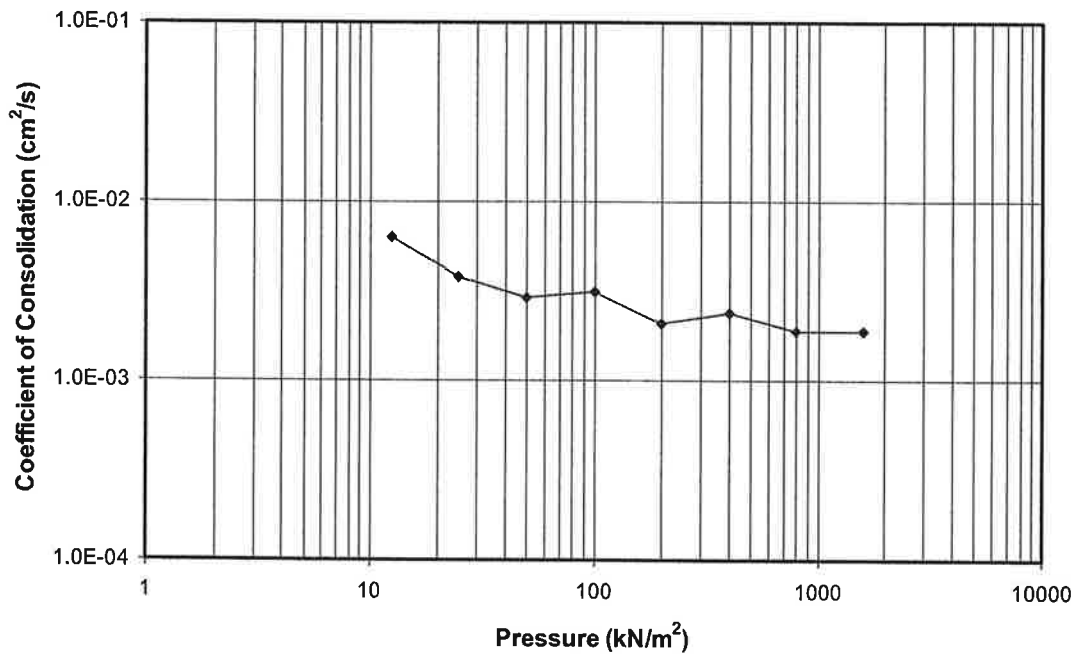


Figure B-4

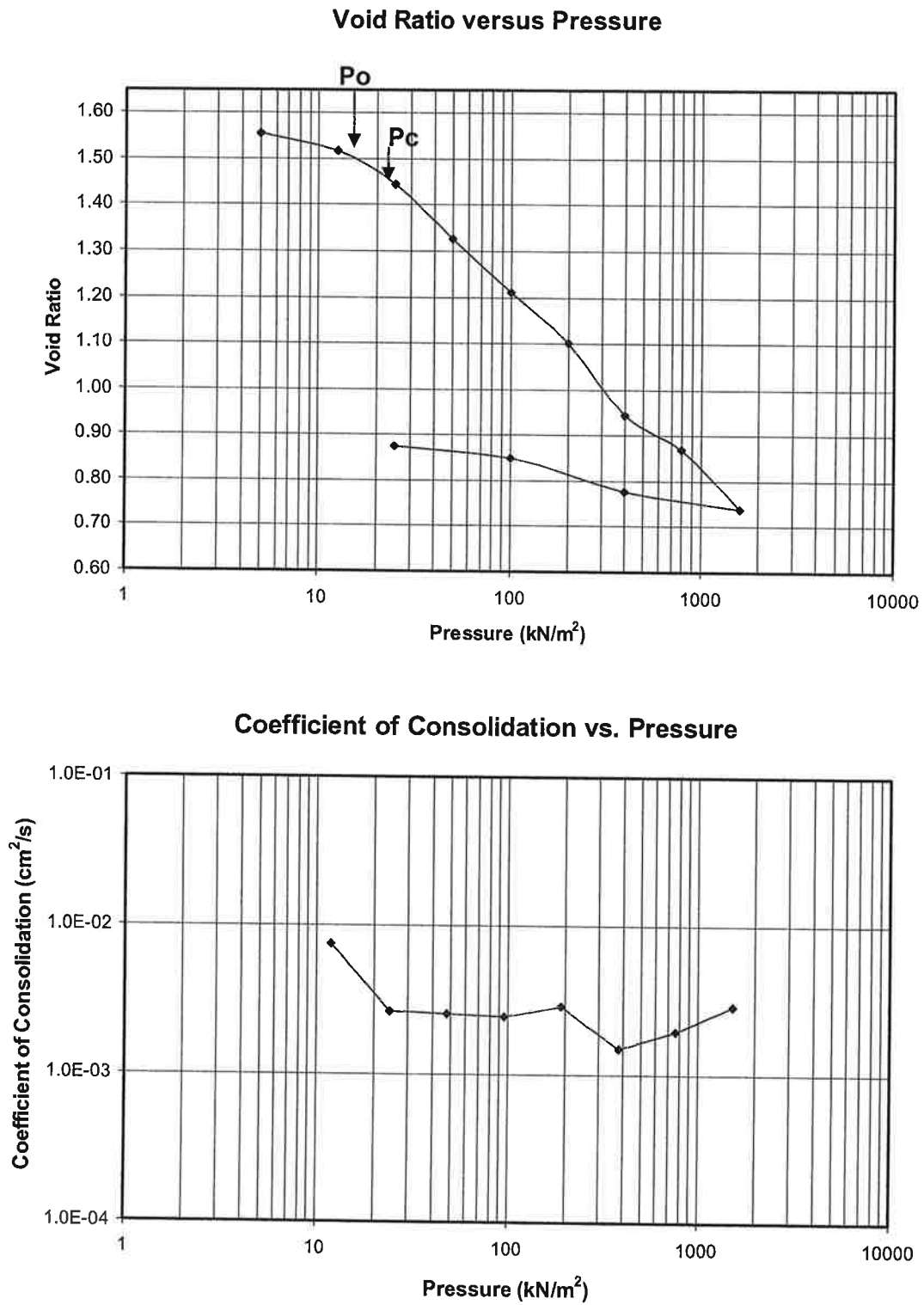
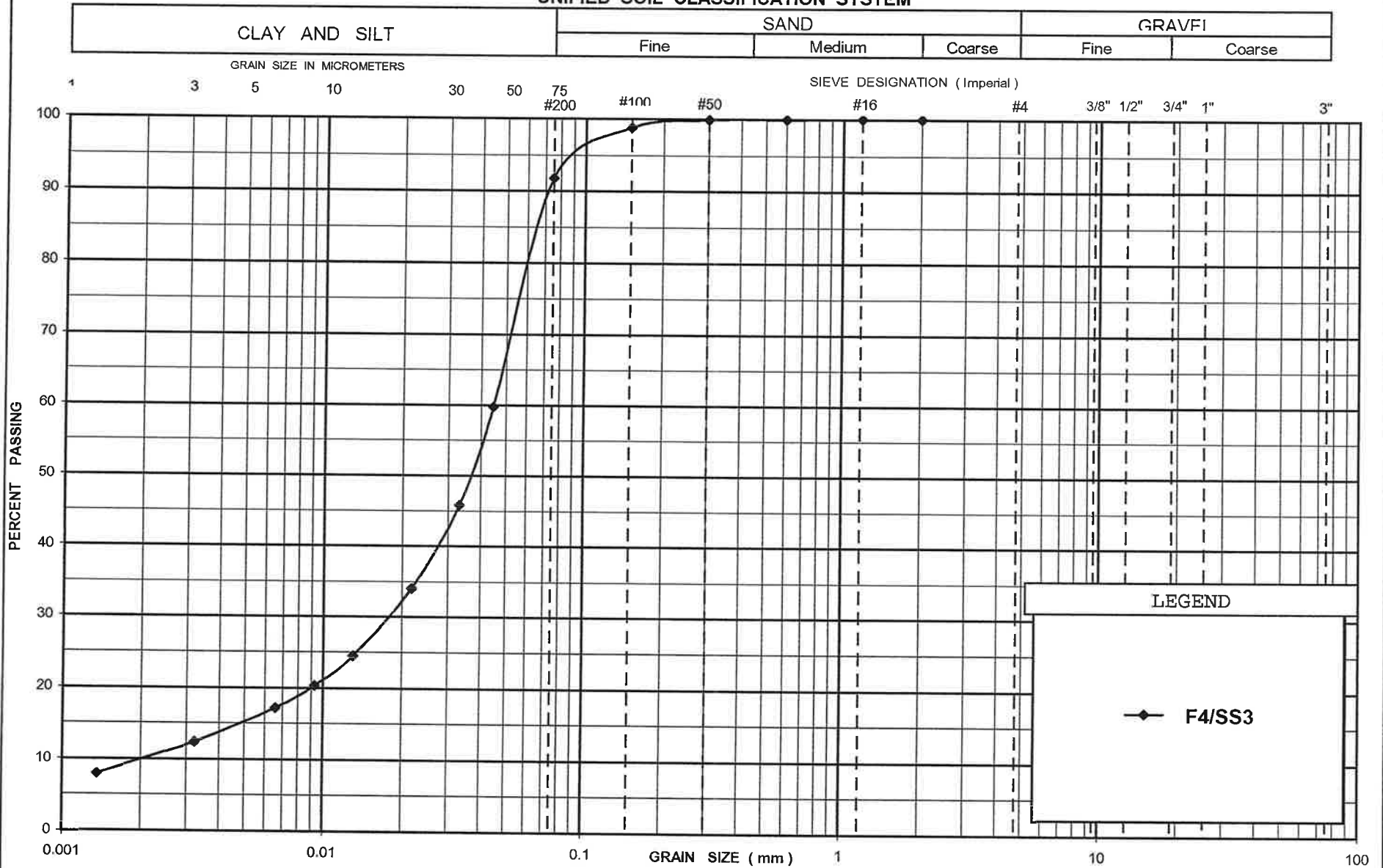
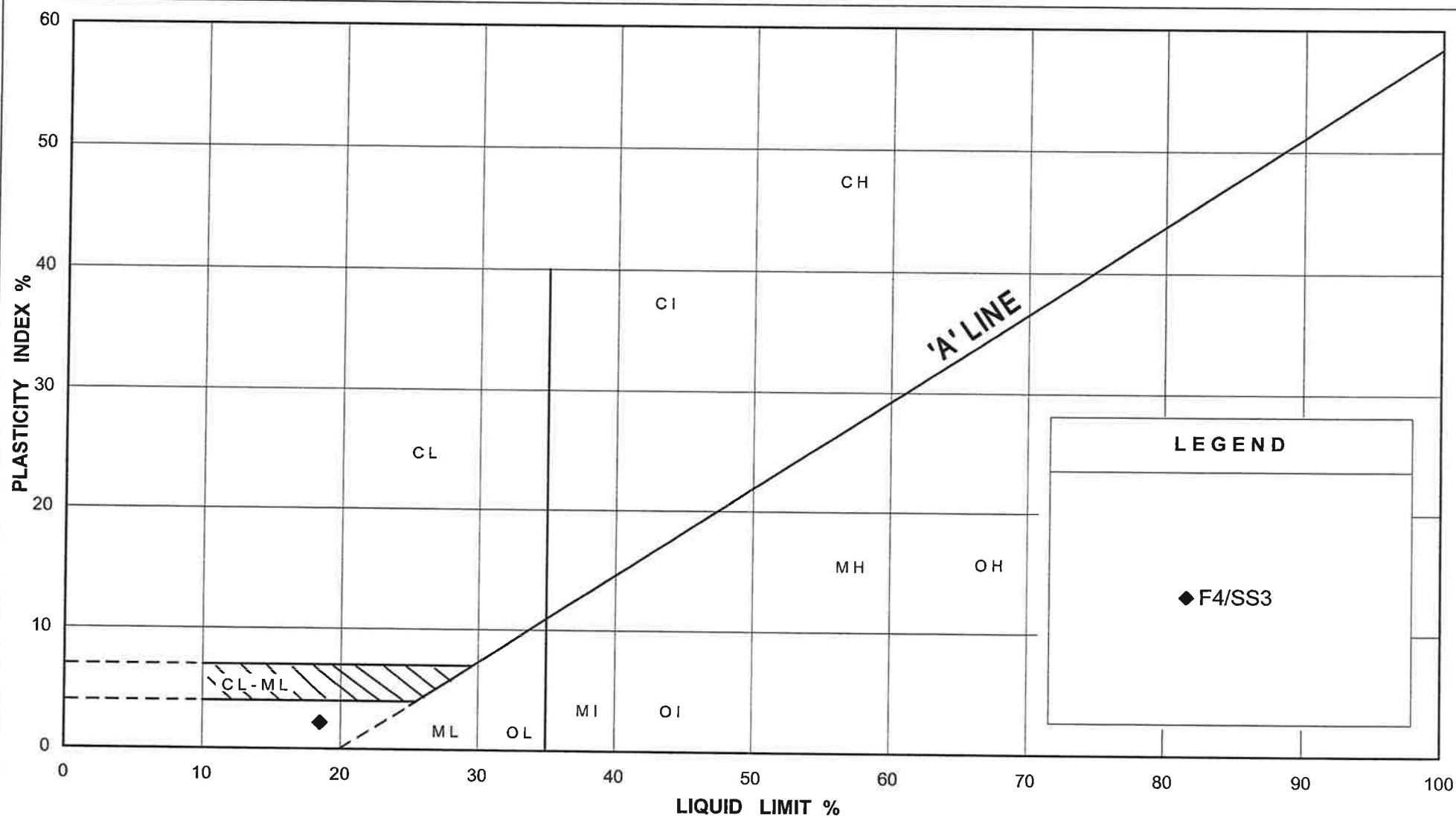
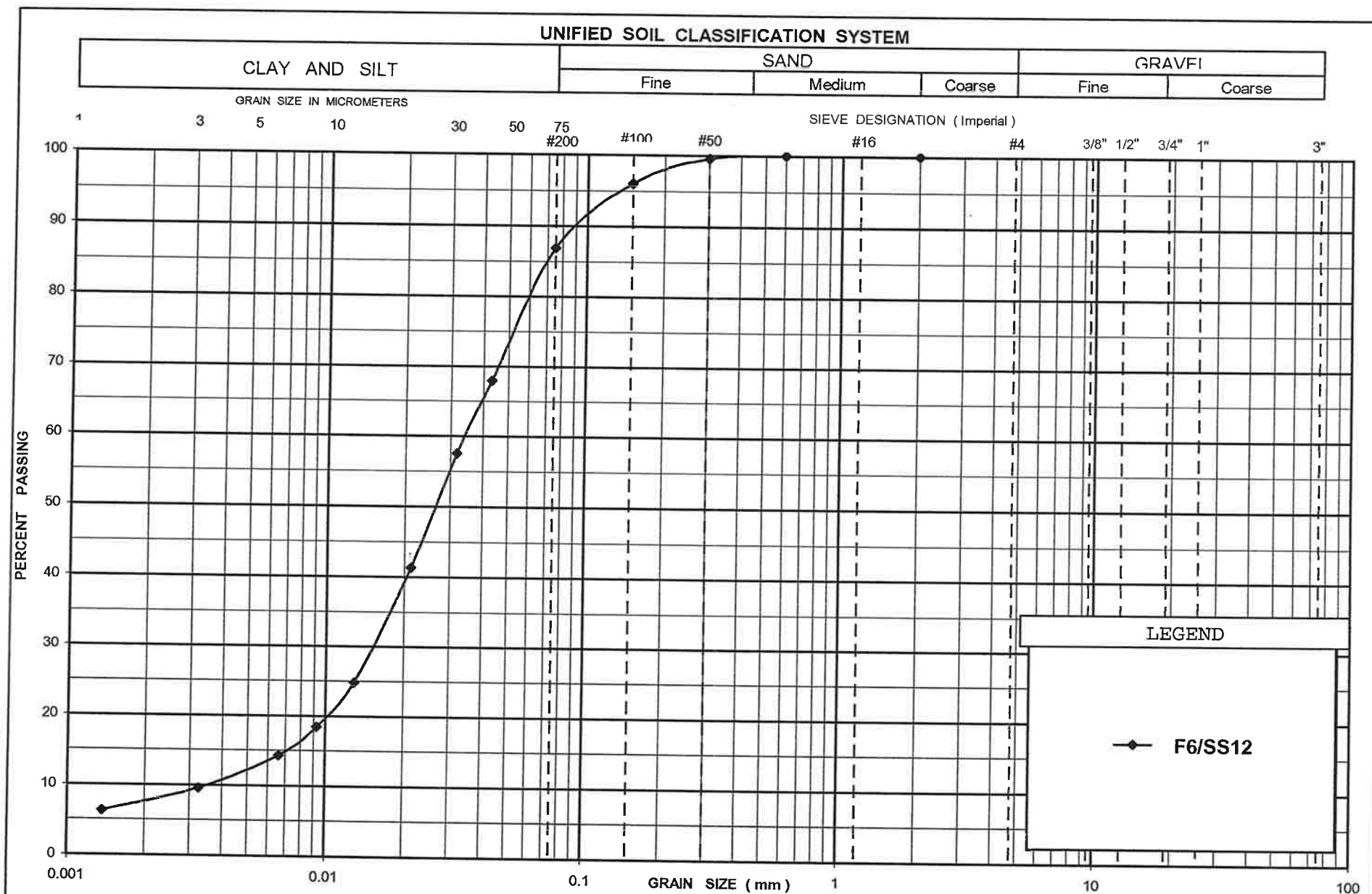


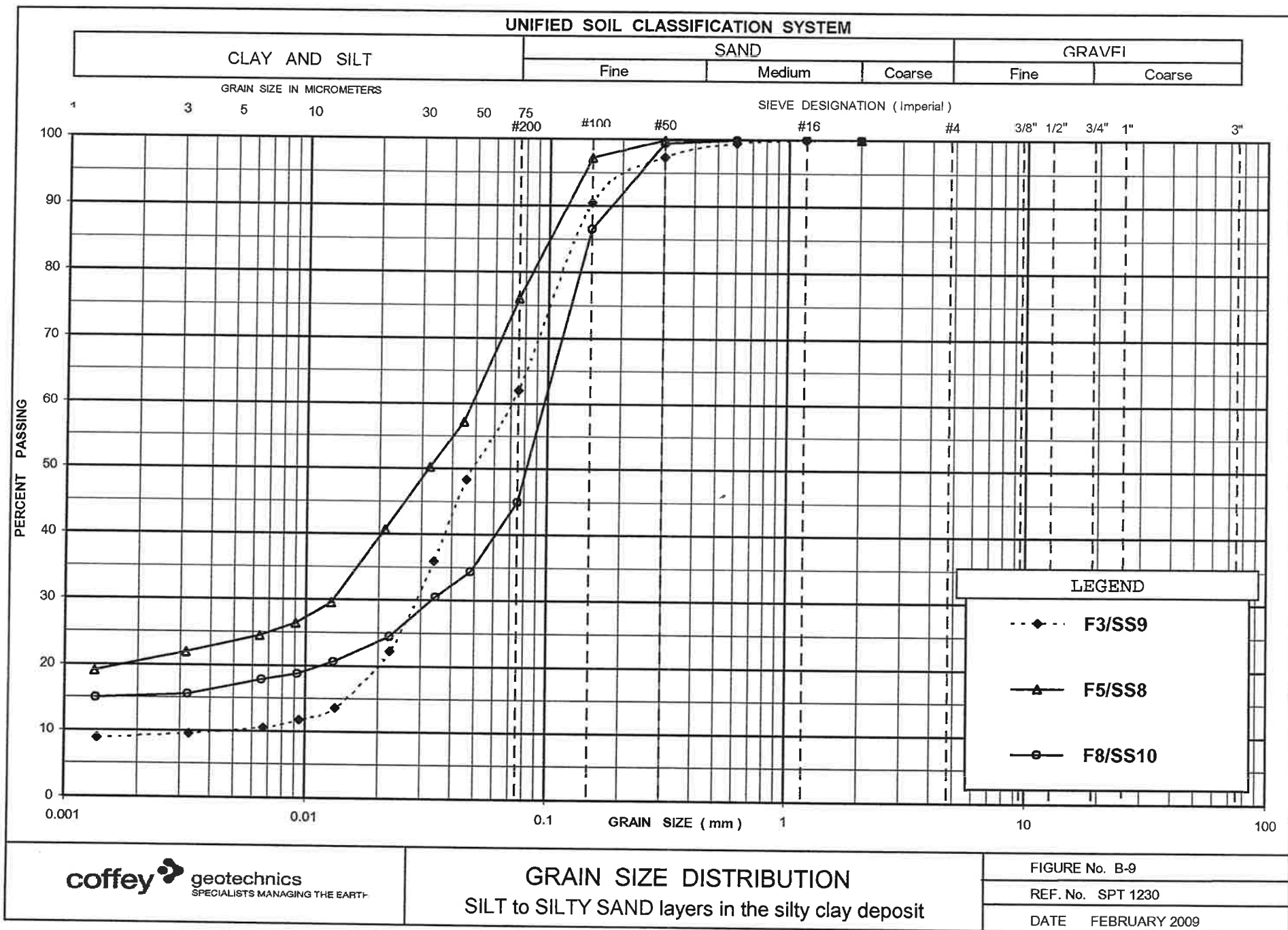
Figure B-5

UNIFIED SOIL CLASSIFICATION SYSTEM

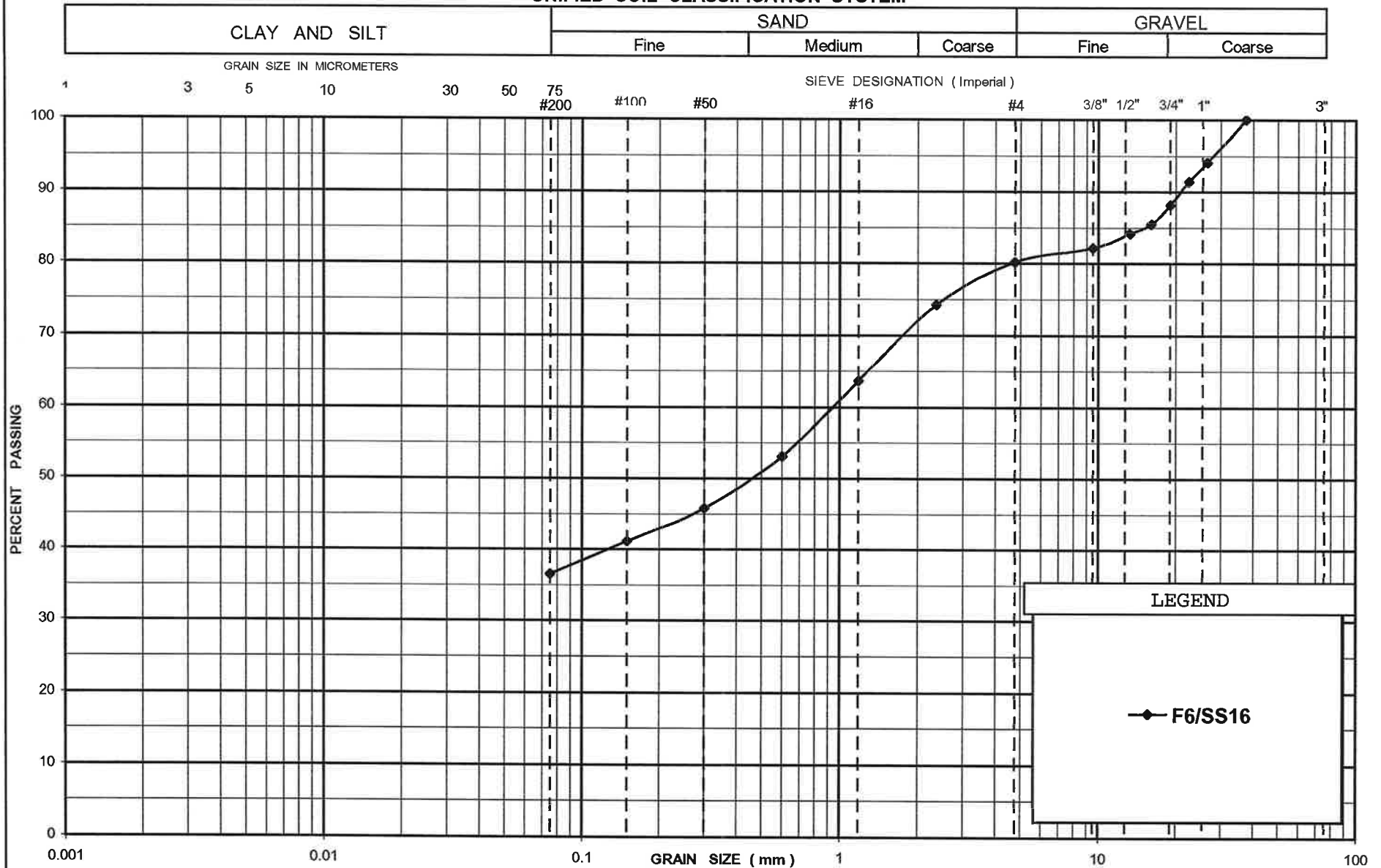








UNIFIED SOIL CLASSIFICATION SYSTEM



Appendix C

Field Vane Test Result

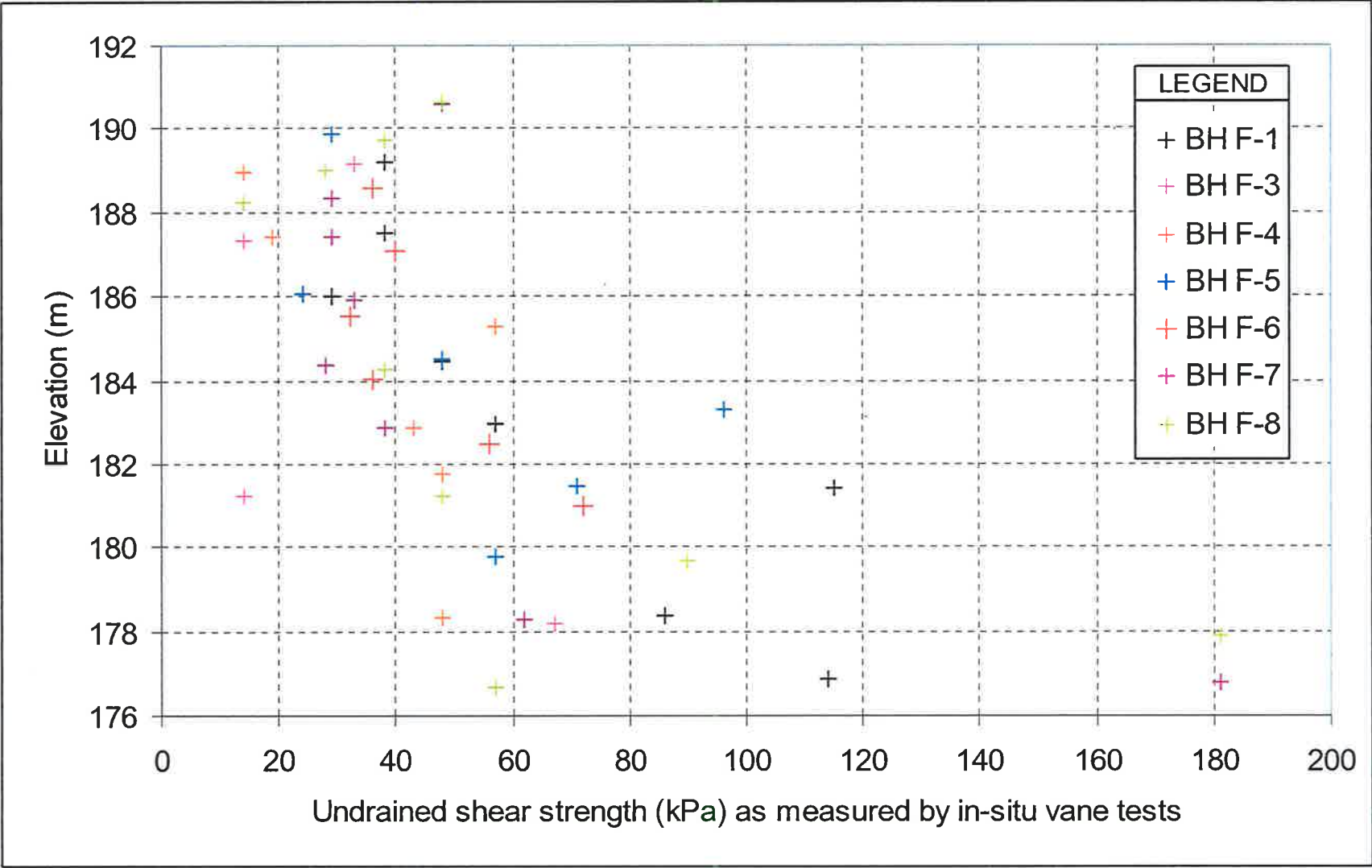


Figure C-1

Appendix D

Site Photographs



Photograph 1 Highway 124 at culvert location (looking south)



Photograph 2 Looking left side of embankment from boat launching area



Photograph 3 Left side of embankment (looking south)



Photograph 4 Right side of embankment (looking south)



Photograph 5 Right side of embankment (looking north)

Appendix E

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

**FOUNDATION DESIGN REPORT
REPLACEMENT OF PORTAGE CREEK
CULVERT, HIGHWAY 124,
TOWNSHIP OF MCDOUGALL, ONTARIO,
CONTRACT NO. 2009-5129
G.W.P. 5176-06-00, GEOCRES NO. 41H-70**

D. M. Wills Associates Limited

Project: SPT1230
June 16, 2009

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Appendices

Appendix F: Staging Plans

Appendix G: Slope Stability Analyses

Appendix H: Limitations of Report

**FOUNDATION DESIGN REPORT
REPLACEMENT OF PORTAGE CREEK CULVERT
HIGHWAY 124, TOWNSHIP OF MCDOUGALL, ONTARIO
CONTRACT NO. 2009-5129; G.W.P. 5176-06-00**

5 DISCUSSION AND RECOMMENDATIONS

The existing culvert beneath Highway 124, about 2.9 km east of Highway 69, provides water flow in the Portage Creek from north (left) to south (right). The 4.27 m diameter, approximately 30 m long SPCSP culvert, which is showing signs of excessive deformations, will be replaced with an aluminium alloy corrugated plate pipe of the same diameter. The new culvert will be lengthened by about 4 m on the right side. The existing culvert invert elevation is about 190.7 m and the new culvert invert is expected to be at about El. 190.2 m.

Seven boreholes were advanced for this investigation, one from the top of highway embankment (from the shoulder) and six from a raft in the creek. In the boreholes put down from the raft, the creek bottom elevation was estimated to range from about 191.6 m to about 190.6 m. At the time of our investigation, the depth of the water in the creek ranged from about 0.9 to 2.0 m. Two of the six boreholes put down from the creek showed the presence of 1.9 m thick organic silt/clayey silt, while in the remaining four the thickness of the upper organic zone was found to be 0.2 to 0.3 m.

In the borehole drilled from the top of the embankment, a granular pavement fill was contacted to a depth of 1.3 m underlain by rock fill to 5.6 m depth or to El. 190.0 m. It is of interest to note that this elevation is 0.6 to 1.5 m lower than the measured approximate creek bottom elevation.

Underlying the embankment fill or the organic soils, the site is underlain by a silty clay deposit. The silty clay deposit is about 15 m thick and contains some clayey silt, silt, sandy silt and silty sand layers, seams or lenses. Below about El. 177-176 m, the site appears to be underlain by fine-grained granular soils comprised of silt, sandy silt and silty sand. Based on the field test results, the silty clay is considered to have a very soft to stiff consistency but typically firm consistency and the underlying fine-grained granular soils are of loose to compact relative density.

5.1 Foundations

The proposed 4.27 m diameter aluminium alloy corrugated plate pipe culvert should be supported on at least 300 mm thick granular bedding. To prepare the subgrade all the organic soil should be removed to the surface of the inorganic subgrade (i.e. all the organic or otherwise unsuitable soils should be removed from beneath the pipe and extending at least 0.8 m beyond the footprint of the pipe on each side). After the inorganic subgrade is exposed if the silty clay appears to be very soft or soft, then a 100 to 150 mm size rock fill should be pushed into the subgrade to strengthen it. Normally a well-graded granular bedding such as Granular 'A' or Granular 'B' Type II material would be used for the construction of the minimum 300 mm thick bedding. If however the site was not fully unwatered and some free water prevails above the stripped subgrade level, the placement of a well-graded soil (which is desirable) would not be practical, as in this case the fines would separate during the placement and rise to the surface. In this event or if it is anticipated that the work would proceed with a water level not depressed to or below the stripped subgrade

elevation, then the use of a poorly graded bedding material would be necessary for practical purposes. Therefore, while every effort should be made to depress the water level to below the subgrade level, if, however, this is found to be impractical then the bedding material can consist of a 150 mm thick layer of 50 mm size clear stone overlain by 150 mm of 19 mm size clear stone, bringing the total thickness of the bedding to 300 mm over the silty clay subgrade. While all seven boreholes show that after stripping the organic soils the subgrade should consist of silty clay, if the subgrade during the construction is found to be other than clayey soils (e.g. silt or fine sand) then clear stone bedding cannot be used. In this event, the site should be properly dewatered and a well-graded bedding should be used.

Borehole F6, which was advanced from the top of the road embankment showed the presence of rock fill to about El. 190.0 m (i.e. about 0.2 m to 0.3 m below the proposed invert elevation). This means that there is a chance that rock fill may be present underneath the existing culvert (i.e. if rock fill was used to strengthen the subgrade before placing the pipe). In this event, the existing bedding can be left in place and used as the bedding for the new pipe, making sure that the rock fill is sufficiently covered by finer materials to a sufficient elevation, to prevent any damage to the new pipe due to any protrusions in the rock fill, which may exist, including any future shifts in the bedding. Again, if the rock fill is found to be below the level of 'the bedding' then a graded granular bedding can be used over a suitable geotextile or alternatively, if the water level is high, a clear stone bedding can be used to prevent the separation of fines below water. In this instance, if the bedding to be placed is less than 200 mm then it could consist of 19 mm size clear stone or if the bedding to be placed is 200 mm or thicker then it could consist of 50 mm size clear stone overlain by 19 mm size clear stone.

Another scenario (possibility) is the presence of at least 300 mm granular bedding under the pipe. In this event the existing bedding can be left in place to support the new culvert.

For subgrade prepared in the manner described above a Factored Bearing Resistance at U.L.S. of 100 kPa and a Geotechnical Resistance at S.L.S. equal to 25 kPa can be assigned.

Under the embankment, the recommended value at S.L.S. is less than the existing embankment loading. This, however, is not considered to be a problem, since the silty clay stratum under the existing embankment would have consolidated under the stresses imposed by the embankment. Since there will be no grade raise, theoretically there should be negligible additional settlements under the existing embankment. However, a settlement of about 25 mm should be allowed for due to rebound during the construction. Based on this, if the founding subgrade is undisturbed during the construction the settlements should be tolerable for the proposed pipe culvert and cambering is considered unnecessary.

These comments, however, do not apply to the widened portions which will be discussed separately in the following paragraphs.

5.1.1 Widening on the Left Side

We understand that on the left side the length of the new pipe will be the same as the existing but the existing side slope will be steepened to 1.25H:1V using rock fill. This will widen the top of the embankment by about 3 m, as shown in Figure F-2 in Appendix F. Assuming that the existing embankment fill consists of rock fill, this is considered feasible but the process can be expected to cause some settlements. Based on the borehole data along with consolidation test results presented in Appendix B, the anticipated

settlement due to the consolidation of the clay under the stresses imposed by the steepened slope using rock fill is about 7 cm. This consolidation settlement can be expected to take place during a time period of about three years. As the rock fill to steepen the slope will be placed before the installation of the new pipe, a small portion (about 1 cm) of the settlement can be expected to take place within a period of about two weeks, resulting in an estimated settlement of about 6 cm after the new pipe is constructed. This is not expected to cause problems with the 4.27 m diameter pipe but you may wish to consider a camber of about 5 cm, starting under the existing road shoulder gradually increasing to 5 cm about 2 m before the end of the pipe and remaining at that level to the end of the pipe.

However, if the rock fill placed during the staging will essentially be removed and culvert extension is also removed, as shown on the staging plan in Appendix F, cambering is not required.

5.1.2 Widening on the Right Side

As shown in Appendix F, the existing embankment will also be steepened to 1.25H:1V, from its present configuration of about 1.7H:1V, similar to the left side, discussed in Section 5.1.1. In addition, a further widening of about 4 m is proposed. While Borehole F4 which is located closest to the existing culvert location on this side shows the presence of little or no organics, a 1.9 m thick organic layer was contacted in Borehole F3, also located on this side but further north. As mentioned before in Section 5.1, to prepare the subgrade, all organic soils should be removed to the surface of the inorganic subgrade (i.e. all the organic or otherwise unsuitable soils should be removed from beneath the pipe and extending at least 0.8 m beyond the footprint of the pipe on each side and 1.5 m from the outlet of the pipe). After the inorganic subgrade is exposed it should be inspected and if the soil is found to be soft or very soft then a 100 to 150 mm size rock fill should be pushed into the subgrade to strengthen it. For practical purposes, pushing the rock fill into the silty clay can be attempted and if the rock fill penetrates it then more can be pushed to a depth of about 0.6 m. If no penetration occurs, then there is no problem since this means that the soil is of sufficient strength. Normally, Granular 'A' or Granular 'B' Type II would be used to provide a minimum 300 mm thick bedding over the strengthened subgrade. If, however, the site was not fully unwatered and the construction must be carried out below free-standing water, then with the well-graded soils there is a good possibility that the fines would separate (i.e. rise to the top). In this case, there would be no option but to use clear stone as the bedding material. If this is the case, the bedding should consist of a 150 mm thick layer of 50 mm clear crushed stone overlain by at least 150 mm thick 19 mm clear crushed stone. If the soil removal is to be more than about 0.3 m then the preparation of the bedding should be carefully conducted in order to prevent a slope failure (e.g. in narrow sections of about 3 m width, each and backfilled). If rather thick organic soils are encountered (i.e. in excess of 0.5 m) then the removal of all organic soil may be impractical and rather than full removal, rock fill can be used to penetrate and displace the organic soils to the bottom of the bedding level. After placing the rock fill, at least 300 mm thick bedding can be placed as discussed above.

The widening will cause considerable settlements in the foundation soils. Based on the borehole and consolidation test data, the anticipated settlements are about 0.5 m, over a period of about 3 years. For this reason, a permanent widening of the embankment is not recommended; as such settlements would be detrimental to the integrity of the pipe, as well as possibly causing settlements and pavement cracking in the roadway. We recommend therefore that immediately after the installation of the new pipe on this side the rock fill be removed to the surface of the low water level in the creek, at a slope of 1 1/4H:1V (similar to

the left side). This will provide a widening of about 3 m. With this configuration, the rock fill below the low water level will permanently remain in place and assuming that the excess rock fill will be removed within ten days of its placement, the anticipated settlement is about 18 cm over a period of about three years. While this is not considered detrimental to the structural integrity of the new culvert, we recommend that consideration be given to providing a camber of about 12 cm, as follows. The cambering would start under the existing road shoulder and gradually increase to 12 cm at about 3 m beyond towards the right, remaining at this level further right to the outer edge of the pipe.

5.2 Backfilling

The bedding and embedment material should be extended along the sides and the top to cover the pipe. The selection and placing of the backfill should be in accordance with OPSD-802.010 and OPSD-802.014. The backfill should consist of free-draining, non-frost susceptible granular materials such as Granular 'A' or 'B' Type II (OPSS-1010). If the bedding material consists of clear stone then a suitable geotextile separator should be placed between the bedding and the backfill material. Bedding and backfilling should be inspected by the QVE, in accordance with SP 902S01. All granular backfill materials should be placed in thin lifts (i.e. not exceeding 300 mm before compaction) and each lift should be compacted to at least 95% of the material's Standard Proctor Maximum Dry Density (SPMDD). The Granular 'A' base and Granular 'B' sub-base courses should be compacted to not less than 98% of the material's SPMDD.

The bedding should be compacted in accordance with SP 105S10.

We would like to point out that the performance of flexible pipe culverts is dependent on the side support provided by the backfill and the adjacent soils. The use of adequate backfill material and especially good compaction are, therefore, necessary for proper side support. For the same reason, the organic soils should be removed within a suitable distance from the footprint of the culvert (e.g. about 1.0 m). The use of heavy compaction equipment should be avoided immediately adjacent and above the pipe, as per MTO practice. During backfill placement, the height of the backfill should be maintained at approximately same level on both sides of the pipe, to avoid lateral displacement of the pipe.

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

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

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

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

Unit weight = 22 kN/m³

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.27$	$K_a=0.34$	$K_a=0.40$
$K_b=0.35$	$K_b=0.44$	$K_b=0.50$

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_o=0.43$	$K_o=0.56$	$K_o=0.62$
$K^*=0.45$	$K^*=0.60$	$K^*=0.66$

Compacted Granular 'B' Type I

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

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a=0.33$	$K_a=0.42$	$K_a=0.54$
$K_b=0.41$	$K_b=0.52$	$K_b=0.64$
$K_o=0.50$	$K_o=0.66$	$K_o=0.76$
$K^*=0.57$	$K^*=0.74$	$K^*=0.86$

Note: K_a is the coefficient of active earth pressure

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

K_o is the coefficient of earth pressure at rest

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

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

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movement can occur such that the active state of earth pressure can develop. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients. The use of vibratory compaction equipment behind the culvert and the retaining walls should be restricted in size as per current MTO practice.

5.3 Embankment Slope Stability

The steepening of the permanent slopes to 1.25H:1V from the present configuration of about 1.7H:1V, may cause foundation slope instability. Similarly, the temporary widening may also cause instability due to the presence of weak soils and these aspects need to be looked into. Slope stability analyses were therefore carried out.

For the slope stability analyses, a soil profile was prepared assuming that existing embankment consists of 1.3 m of granular pavement fill underlain by rock fill to a depth of 5.6 m or to El. 190.0 m, as evidenced by Borehole F6. For the underlying soils the results of all seven boreholes were used. Where the soil consists of organic materials it was assumed that these will be displaced under the footprint of the widened section by rock fill. The bottom of existing creek profile was determined by probing from a boat. For the widening, the thickness of the granular soils overlying the rock fill was assumed to be 0.6 m.

Stability analyses were carried out by means of limit equilibrium using the computer program Slope/W. In our analyses, Morgenstern-Price method was utilized, both for short-term (undrained) and long-term (drained) analyses calculations. Sliding block type limit equilibrium analysis was also performed using Spencer method.

The soil parameters used in our analysis for the soil strata are given in the following table.

Table 5.3.3: Soil Parameters Used for Slope Stability Analysis

Material	Short Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m ³)	ϕ (degrees)	C (kPa)	γ (kN/m ³)
Pavement Fill (sand & gravel)	35	0	22	35	0	22
Embankment fill	42	0	20	42	0	20
Organic soils	0	10	14	20	0	14
Silty clay	0	22-60	15.5-16.0	22	2	15.5-16.0
Silt	28	0	18	28	0	18

As mentioned before, it was assumed that the widening will consist of rock fill (with no more than about 0.6 m thick granular shoulder pavement fill on top). If thicker granular soils are to be used this will adversely affect the factor of safety. The results of the slope stability analyses are presented in Appendix G.

5.3.1 Left Side

As shown in Figures G1, G2, G3, G4 (undrained analysis); G5, and G6 (drained analysis), the calculated minimum factor of safety is 1.3 which is normally acceptable to MTO.

5.3.2 Right Side – Temporary Condition

As shown in Figures G7, G8, F9 and F10 (undrained analysis); G11 and G12 (drained analysis), the calculated minimum factor of safety is 1.3 which is normally acceptable to MTO.

5.3.3 Right Side – Permanent Condition

Figures G13, G14, G15 show typical results for undrained analysis and G16 for drained case and these indicate a minimum factor of safety of 1.5, which is normally acceptable to MTO.

Based on these results and assuming that the embankments will be constructed as discussed, a foundation failure should not occur. As mentioned before, these results are based on the assumption that the existing embankment consists of rock fill and that rock fill will be used for the widening.

5.4 Construction Comments

We understand that the new culvert will be located at the exact location of the existing culvert. To maintain the flow of water across the highway during the construction, either a new temporary culvert can be constructed, which would be removed after the construction or the area can be enclosed with a temporary

dyke and the collected water can be pumped across the highway from a pipe buried in the upper granular portion of the pavement fill.

As the natural, inorganic subgrade is expected to consist of silty clay, the water level can be lowered just below the level of excavation (i.e. subgrade will not be easily disturbed, unlike for example, dilatent silt subgrade soils, which would require a deeper drawdown).

We recommend that the contractor be asked to submit their method of diversion and unwatering to the CA for information purposes.

In addition, if at all possible, the construction should be carried out during a dry season.

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

SP 105 S19 – Protection Systems

SP 902 S01 – Excavation and Backfilling to Structures

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

Rock Fill	Type 3 Soil
Organic Soils	Type 4 Soil
Silty Clay, stiff	Type 3 Soil
Silty Clay, very soft to firm	Type 4 Soil

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

An NSSP should be included in the Contract Documents alerting the Contractor of the subsurface and groundwater conditions and that the groundwater control requirements should be planned accordingly by the Contractor prior to construction.

When abutting into the existing rock fill, proper benching is recommended. If this is impractical, then the surface of the existing rockfill side slope to be widened should be scratched to remove any soils/vegetation etc to provide a good bond between the existing rock fill and the rock fill to be placed for the widening. Before placing any granular fill over the rock fill in the widened section, proper chinking should be applied. Alternatively, a suitably robust geotextile can be placed for separation purposes. As well, the thickness of granular fill over the widened section should not exceed 0.6 m.

We understand that the construction will be carried out in open-cut and shoring will not be required. This is the preferred choice in our opinion since shoring can be expected to be very costly due to the presence of rock fill as revealed by Borehole F6. However, for the sake of completeness the following parameters are provided for the soil types encountered in the boreholes.

Table 5.4.1: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m ³)
Organic Silt	0.50	0.70	1.4	14.0
Silty clay to El. 184 m	0.48	0.65	2.0	15.5
El. 184-180 m	0.45	0.62	2.2	15.8
Below El. 180 m	0.42	0.60	2.4	16.2
Lower Silt	0.40	0.55	2.6	17.5
Lower Silty Sand/Silty Sand to Sandy Silt	0.35	0.52	2.9	18.0

It is our opinion that depths of piles are anticipated to be extensive and dead-man and anchor system will likely be required through the rock fill. If roadway protection system is required, we recommend additional investigation to decide the design parameters of shoring system.

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

The design should be carried out by a Professional Engineer experienced in this type work.

5.5 Erosion Protection

We recommend that the existing erosion and scour measures be evaluated for their sufficiency. If the observations show that they are sufficient, they can be duplicated or extended provided that the reconstruction will not adversely affect erosion and scour potentials. If the existing measures are found to be deficient or if the flow regime (i.e. erosion and scour potentials) will change then further measures will be necessary. The following is a discussion of possible erosion measures.

Erosion and scour protection should be provided at the culvert inlet and outlet (including the slopes and sides). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. The following are some general suggestions, considering that the base of the watercourse and of the channel sides consist of organic soils underlain by silty clays.

We recommend that a concrete cut-off (apron) be constructed both at the inlet and outlet to prevent seepage beneath and around the culvert, especially through the granular bedding and granular backfill around the culvert. Beneath the culvert, the concrete cut-off wall should extend to a suitable depth (e.g. below any possible scour depth). Consideration may also be given to an impervious seal at the inlet and outlet.

At the inlet, consideration may also be given to the use of a clay seal. The purpose of the clay seal is to ensure that water flow is channeled through the culvert and does not seep through the backfill around the structure and from beneath the structure. The clay seal should therefore be continuous and typically 0.6 m thick. It should comply with the material specifications given in OPSS 1205. It should be extended around the culvert from at least 0.3 m above the high water level in the watercourse down to the channel bed and up the other side in a continuous manner. It should be ensured that it extends to cover all the granular

backfill materials to prevent any seepage through them. Typically, the clay seal is protected by laying a 0.6 m thick rock protection over it. The clay seal would generally be extended to about 8 m beyond the inlet.

At the outlet as well as at the inlet (if clay seal is not used), in addition to the concrete cut-off and/or impervious seal or in conjunction with these, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered. This would generally be extended about 8 m along the channel and the sides (to at least 0.3 m above the high water).

However, clayey seal and concrete cut-off may not be required, if clear stone bedding and backfill up to 0.3 m above the high water level is placed.

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

5.6 Frost Protection

Design frost protection for the general area is 1.7 m. A permanent soil cover of at least 1.7 m or its thermal equivalent is therefore required for frost protection. In case of riprap (rock fill), only one half of the rock fill thickness should be assumed to be effective in providing frost protection.

6 CLOSURE

We recommend that once the details of the project are finalized, our recommendations be reviewed for their specific applicability.

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

For and on behalf of Coffey Geotechnics Inc.


Ramon Miranda, P.Eng.





Zuhtu Ozden, P.Eng.



Appendix F

Staging Plans

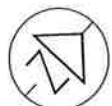
PR-2-207 88-25
MINISTRY OF TRANSPORTATION, ONTARIO

PORTAGE CREEK



METRIC

CONT 2009-5129
GWP 5176-06-00

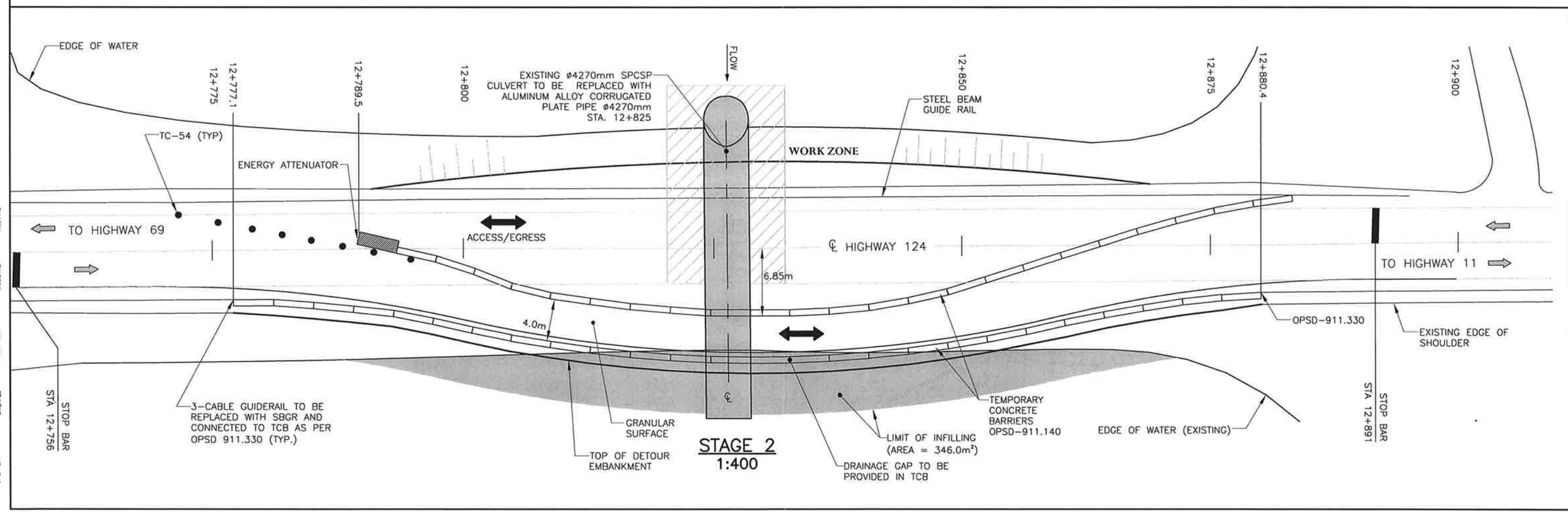
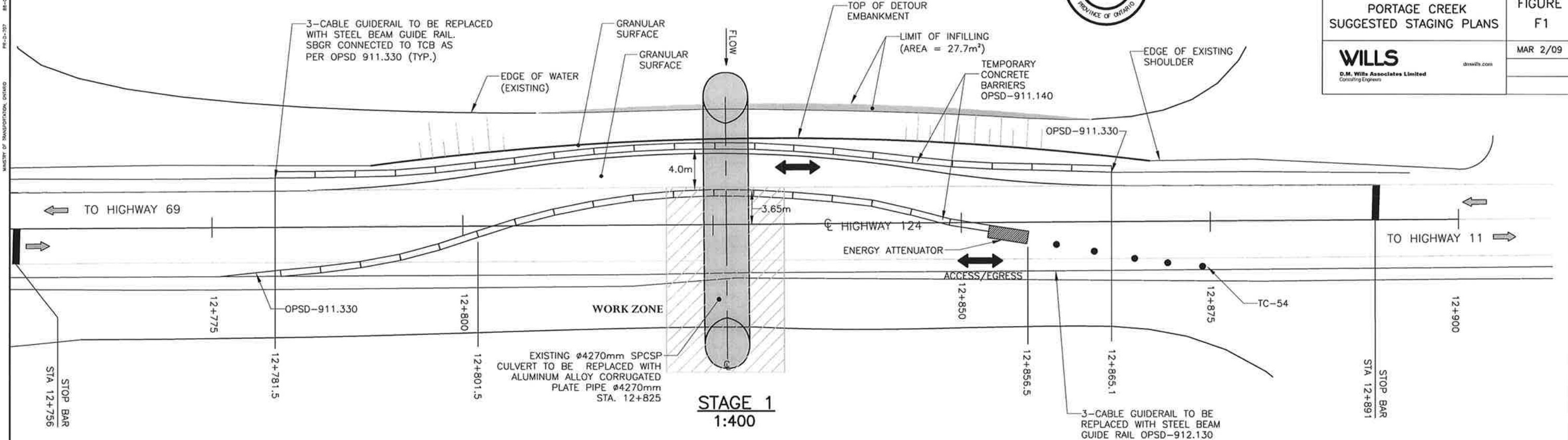


PORTAGE CREEK
SUGGESTED STAGING PLANS

FIGURE
F1

WILLS
D.M. Wills Associates Limited
Consulting Engineers

MAR 2/09



DRAWING NAME:
CREATED:
MODIFIED:
MODDATE:
MODTIME:

METRIC

CONT 2009-5129
WP 5176-06-00

PORTAGE CREEK
SUGGESTED STAGING
SECTIONS

FIGURE
F2

WILLS

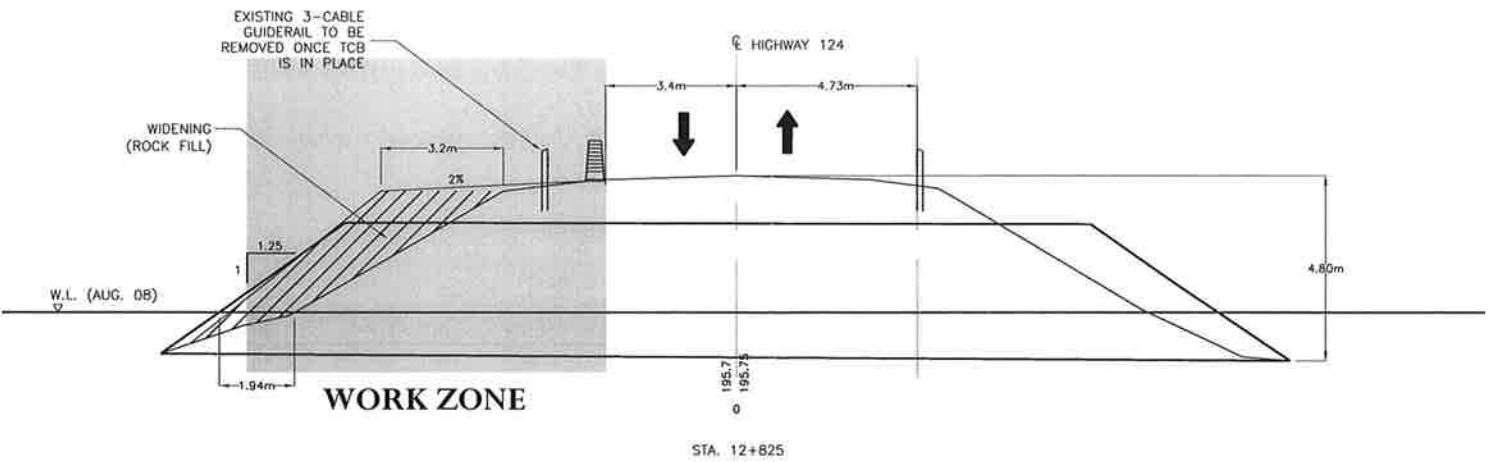
D.M. Wills Associates Limited

dowills.com

FEB 24/09

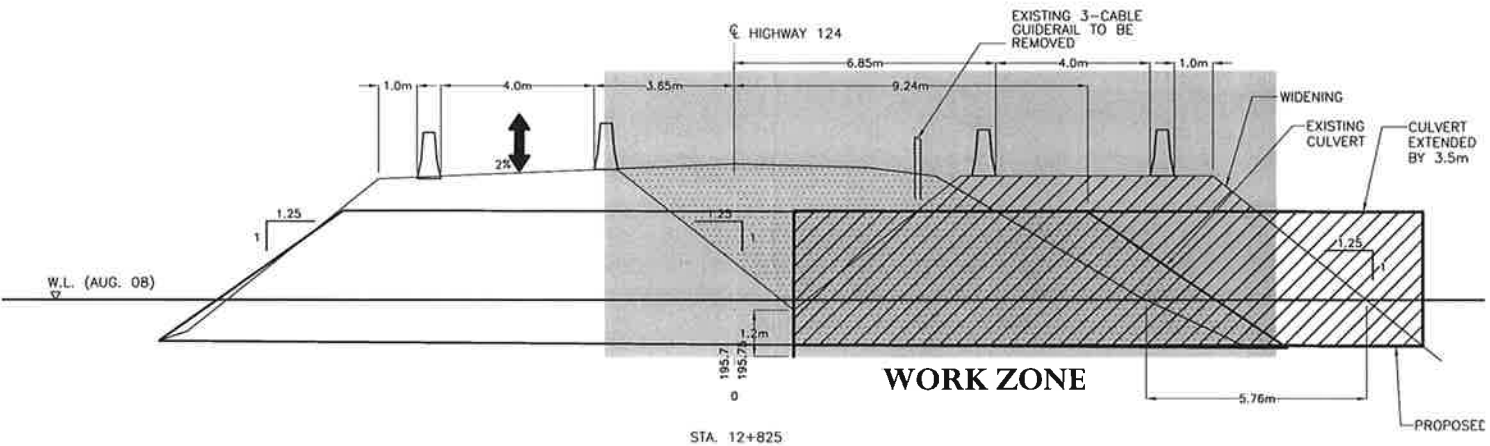
PRE-STAGE 1

WIDENING TO THE NORTH,
TWO-WAY TRAFFIC ON
EXISTING ROADWAY



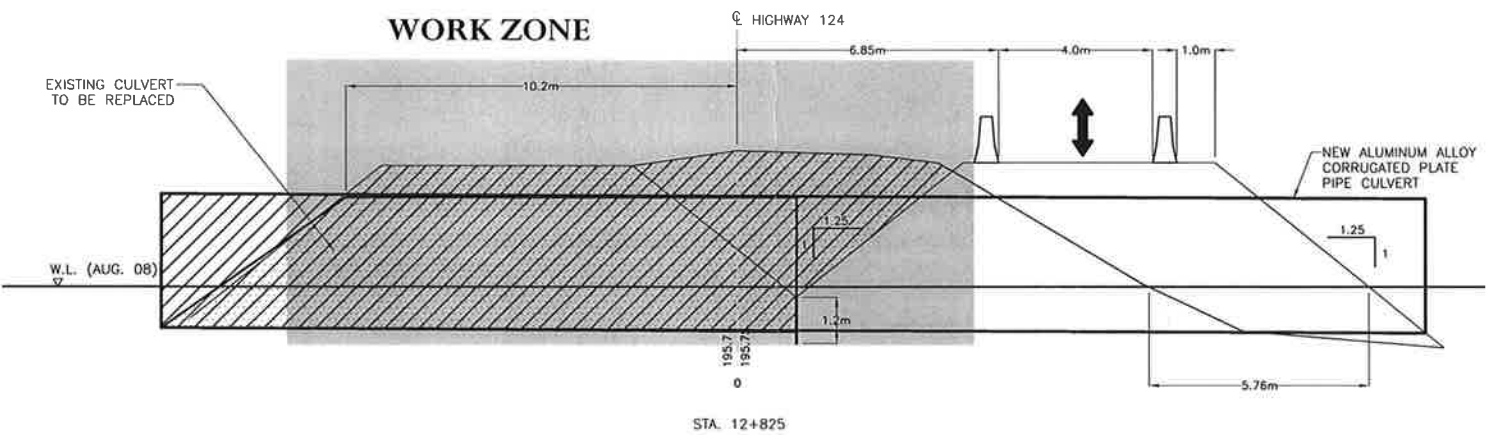
STAGE 1

ONE WAY TRAFFIC SHIFTED TO
DETOUR ON NORTH SIDE,
CULVERT AND DETOUR
CONSTRUCTION ON SOUTH SIDE



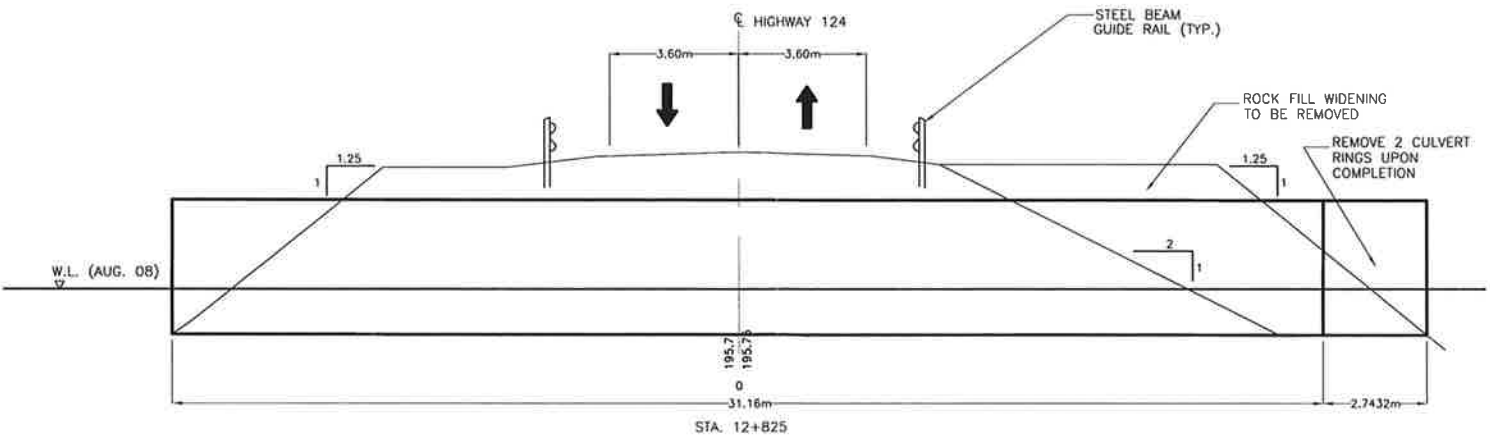
STAGE 2

ONE WAY TRAFFIC SHIFTED TO
DETOUR ON SOUTH, CULVERT
CONSTRUCTION ON NORTH SIDE



FINAL CONFIGURATION

TWO-WAY TRAFFIC RETURNS TO
EXISTING ALIGNMENT



REMOVALS/EXCAVATION
NEW CONSTRUCTION

SCALE
0 1 2 3 4
metres

Appendix G

Slope Stability Analyses

Figure G-1 Left side Undrained analysis

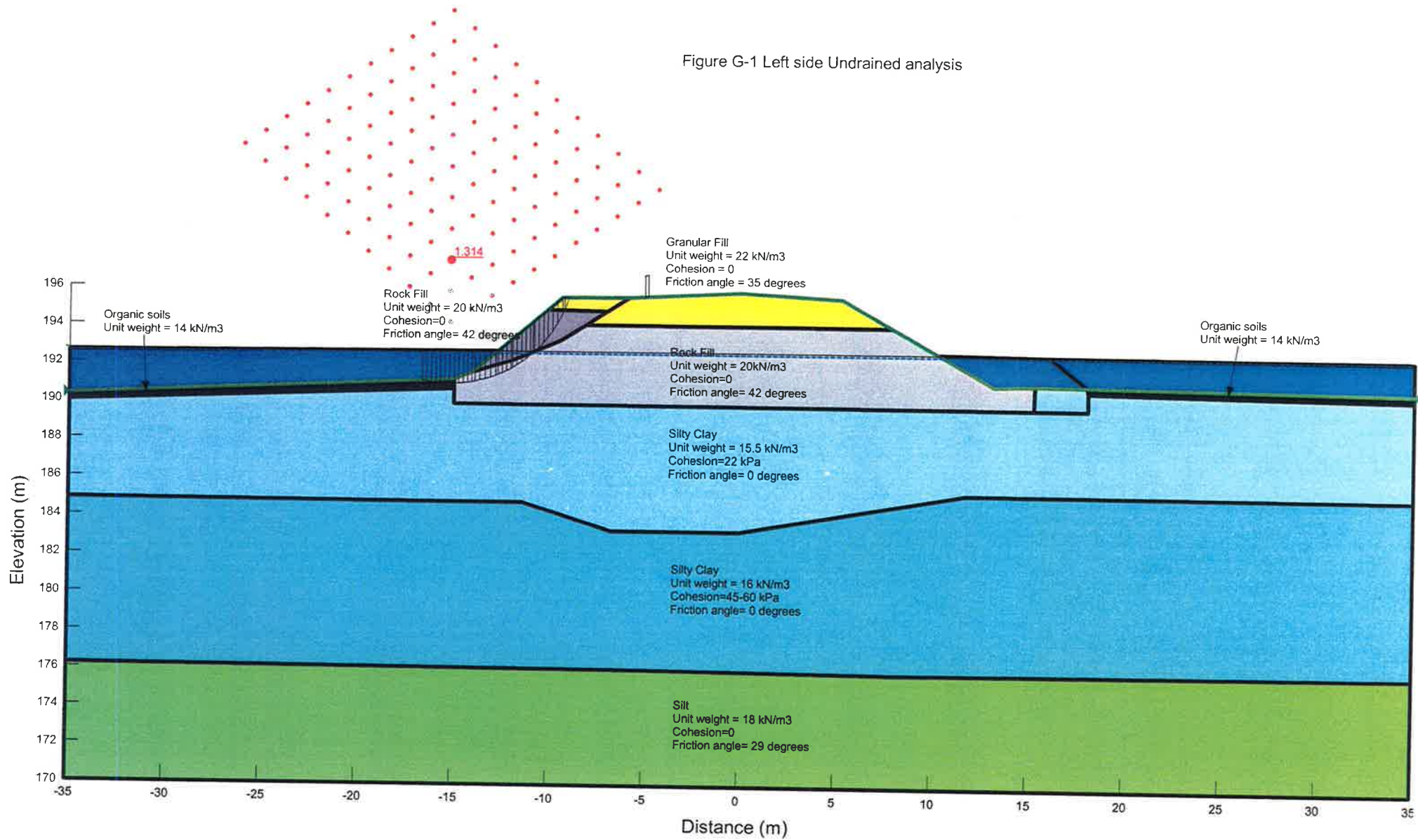


Figure G-2 Left side Undrained analysis -Deep

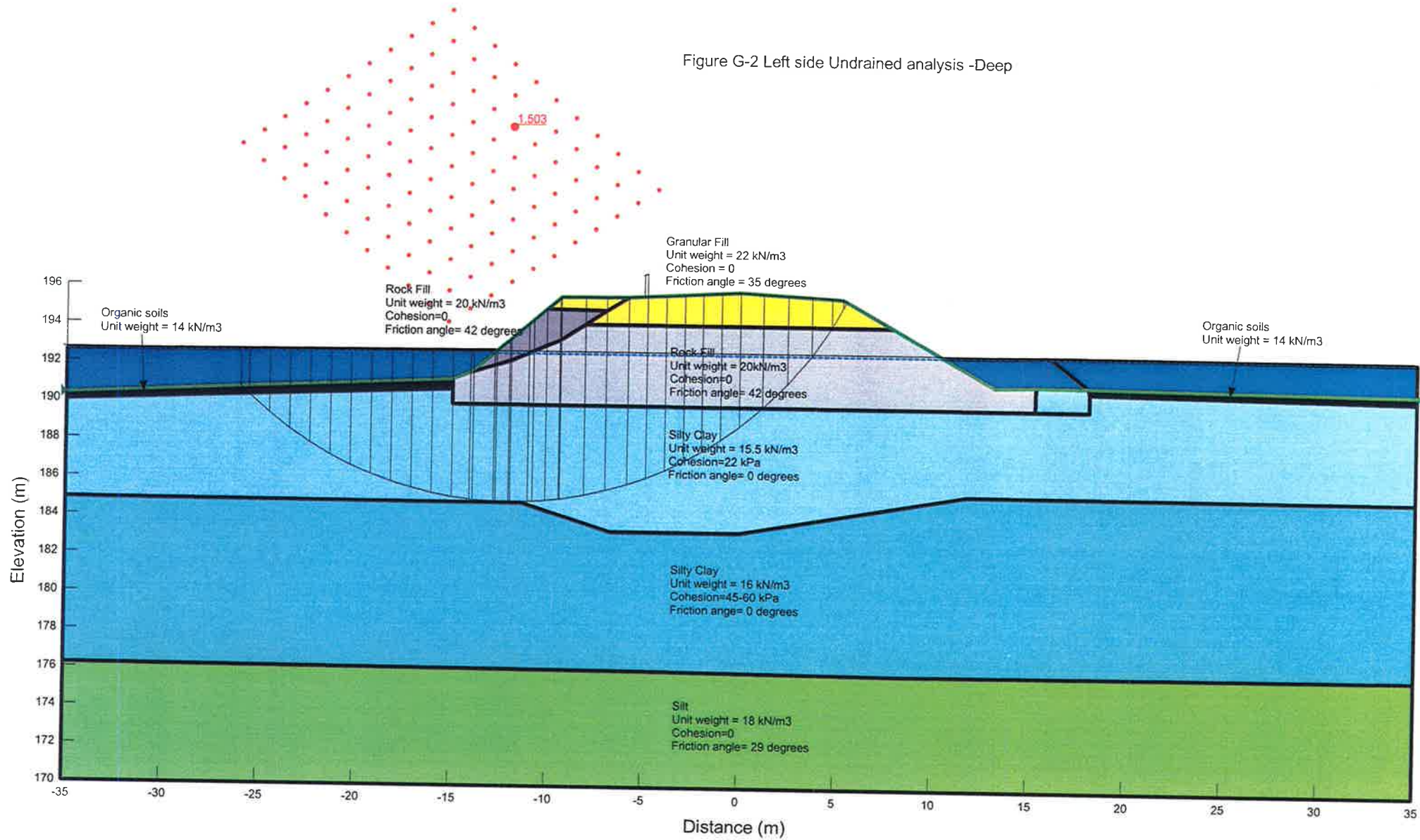
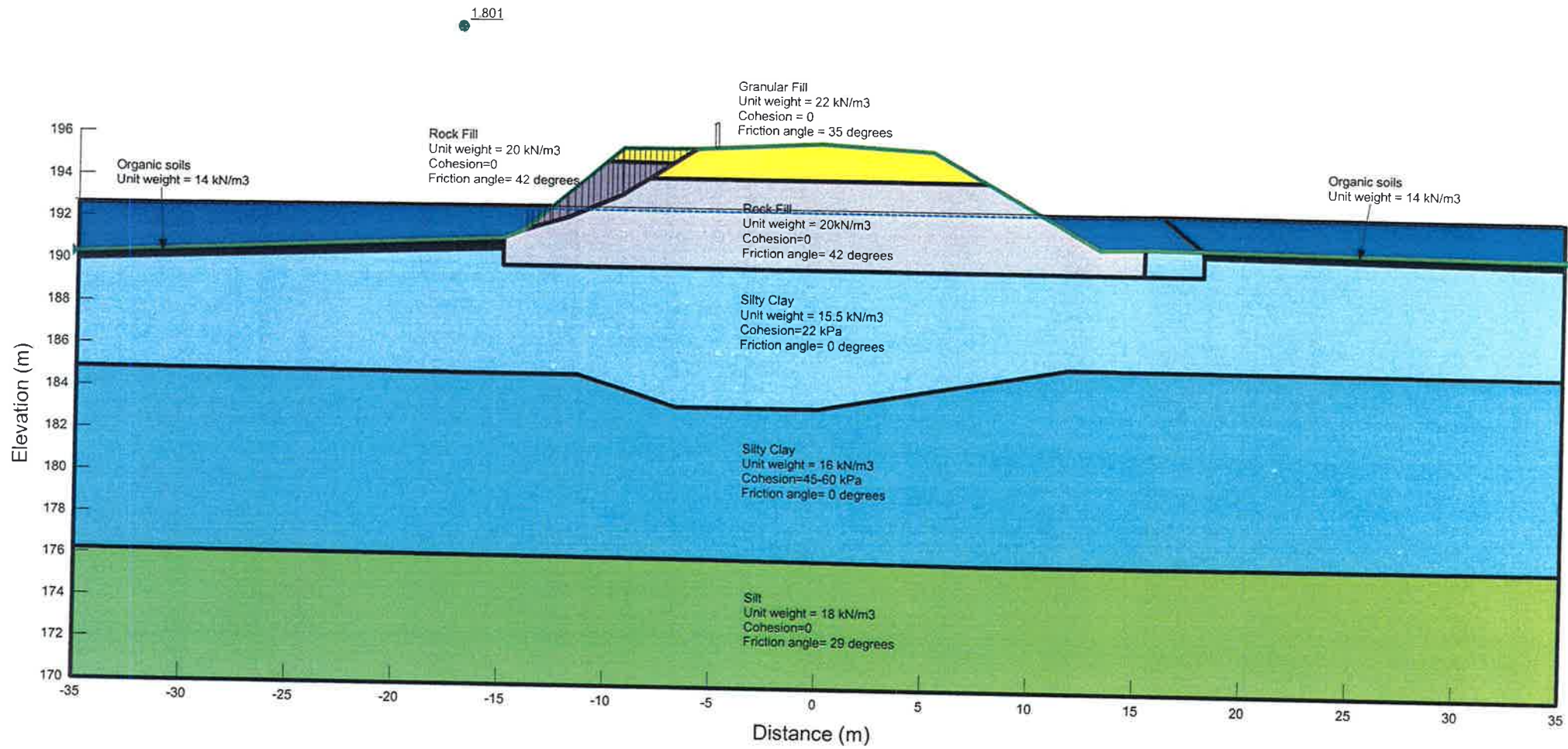


Figure G-3 Left side Undrained analysis -Block



1.423

Figure G-4 Left side Undrained analysis (Staging plan)-Block

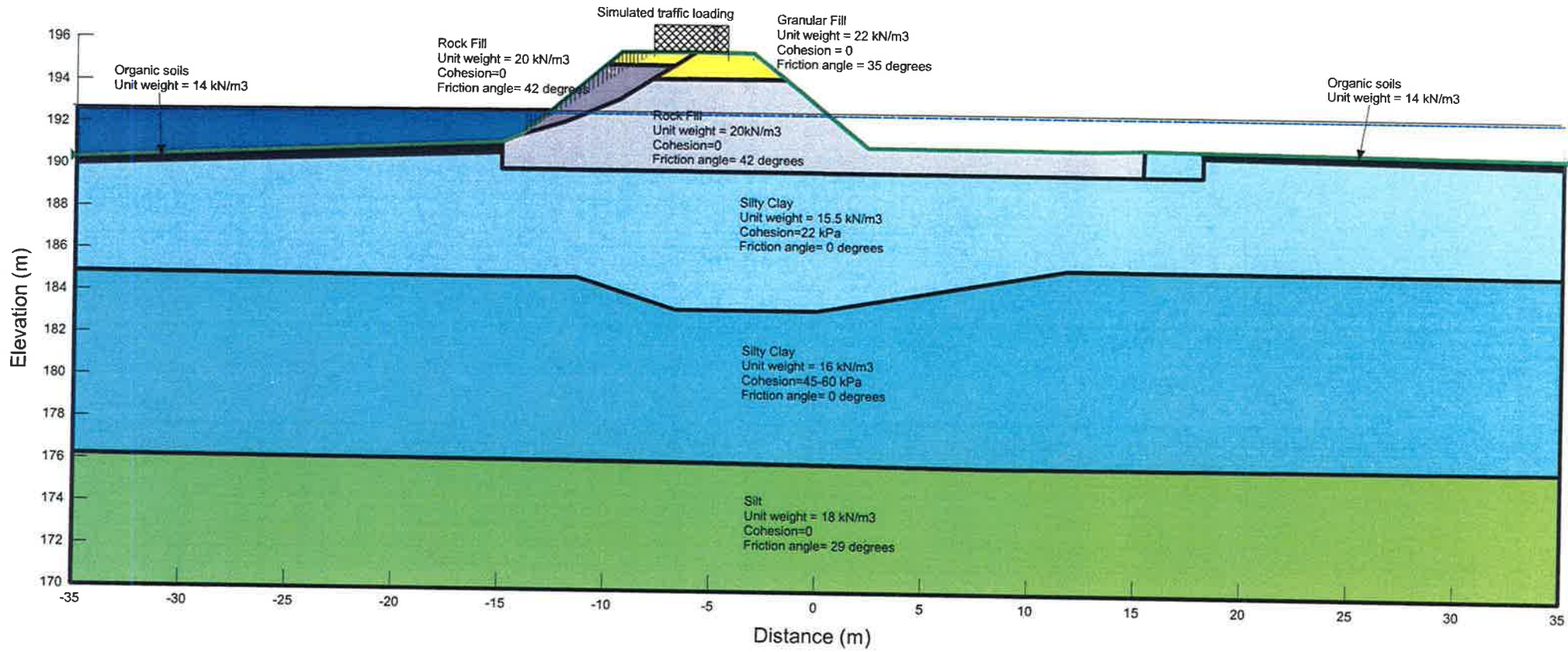


Figure G-5 Left side Drained analysis -Deep

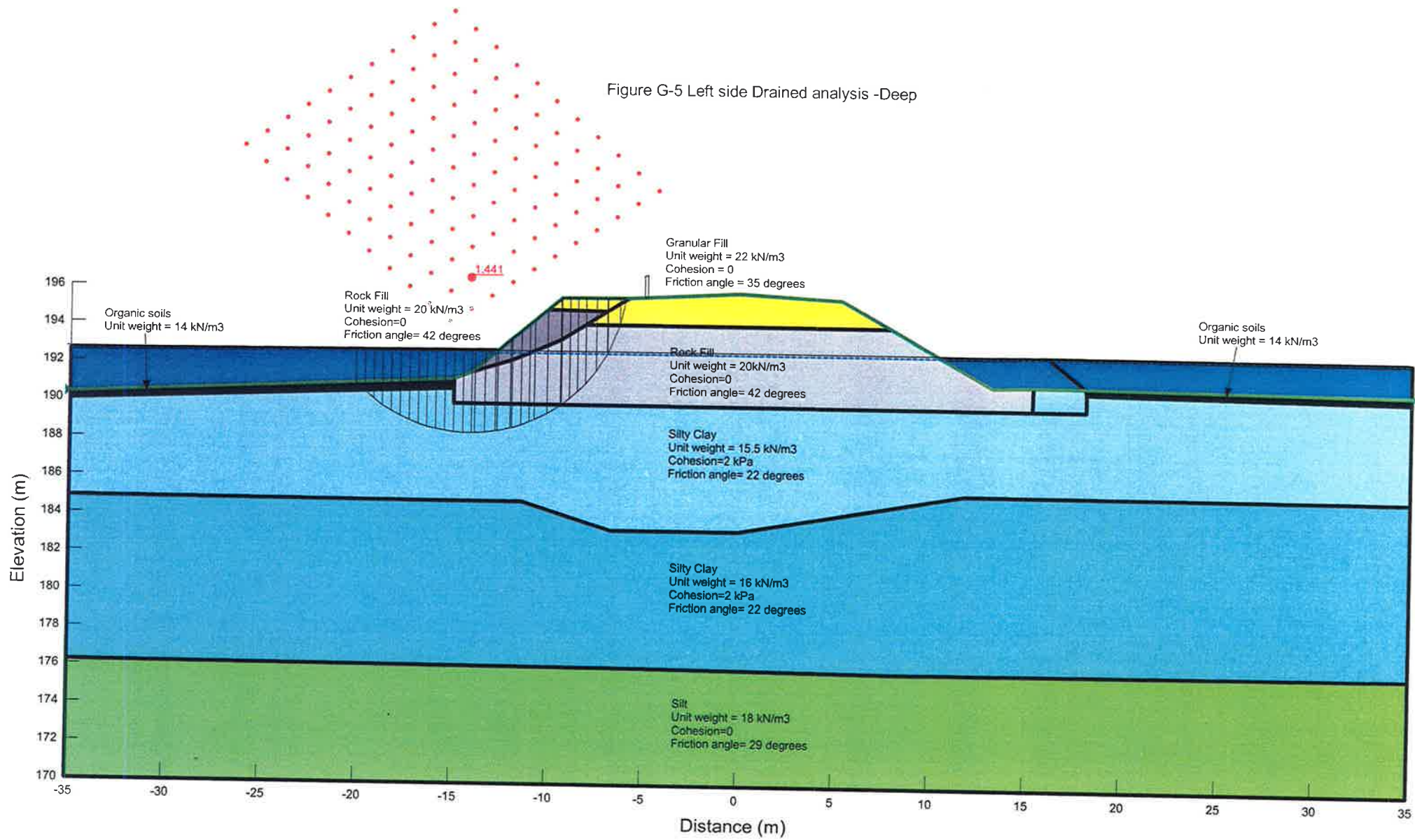


Figure G-6 Left side Drained analysis (Staging plan)-Deep

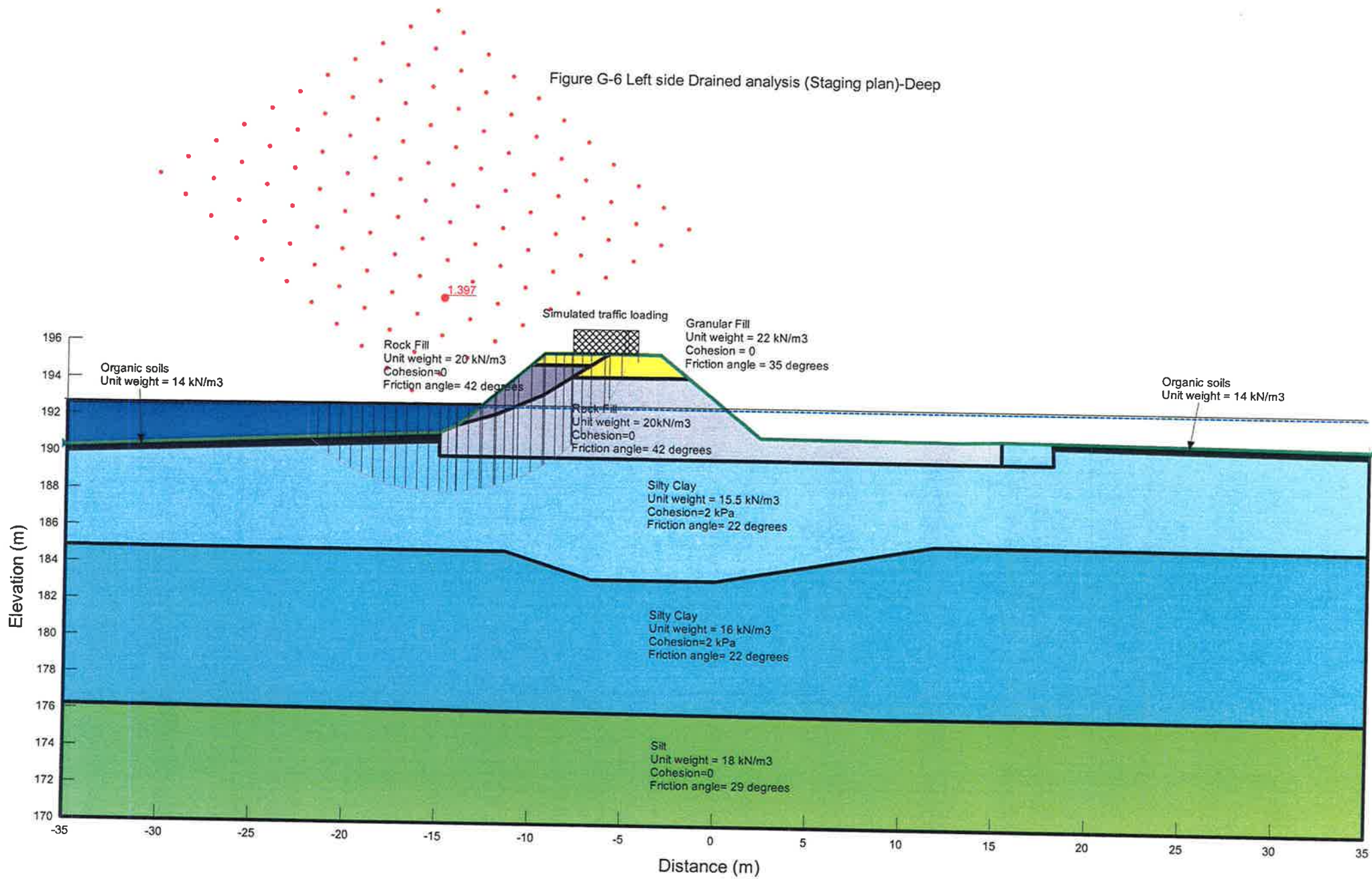


Figure G-7 Right side Undrained analysis

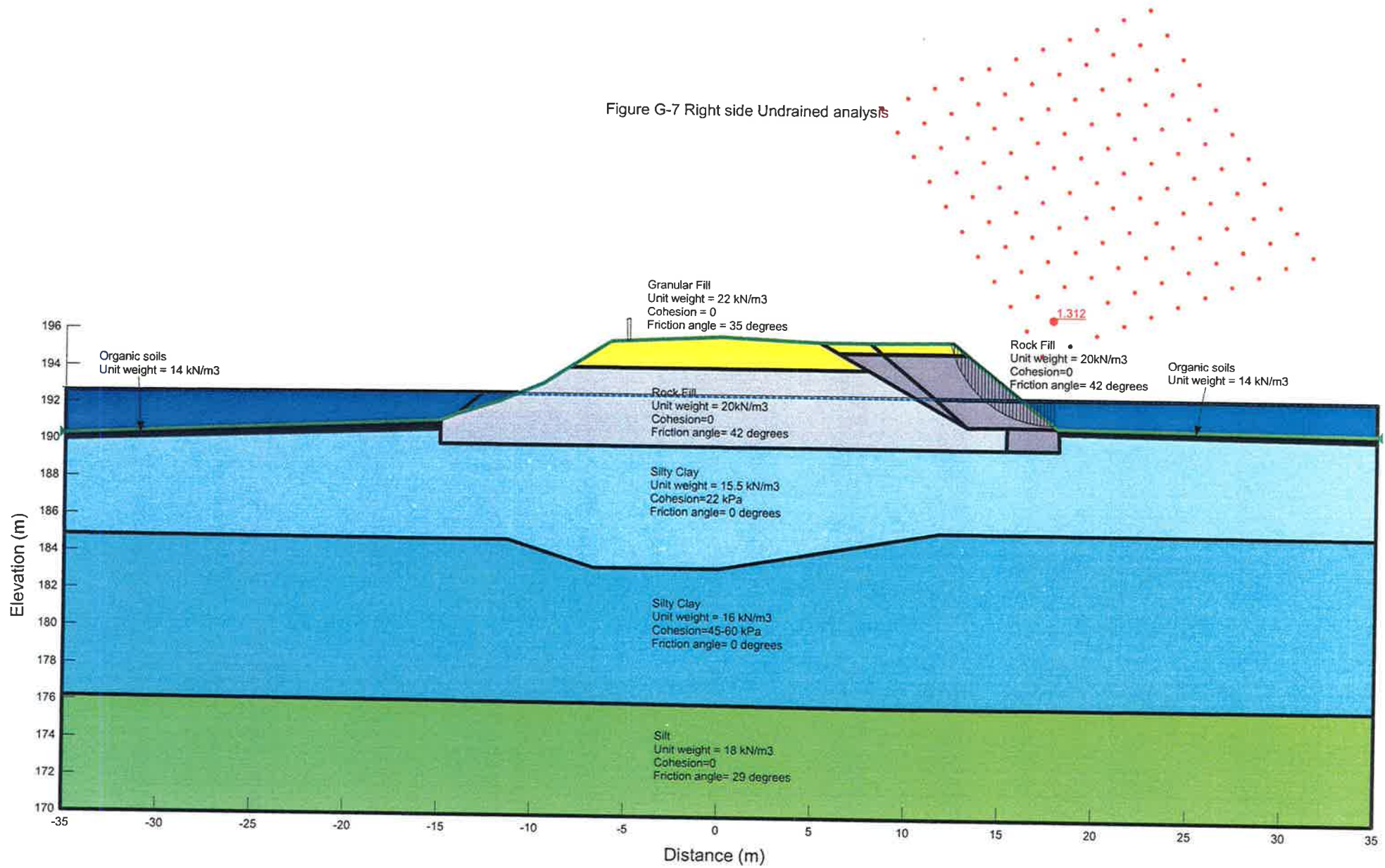


Figure G-8 Right side Undrained analysis -Deep

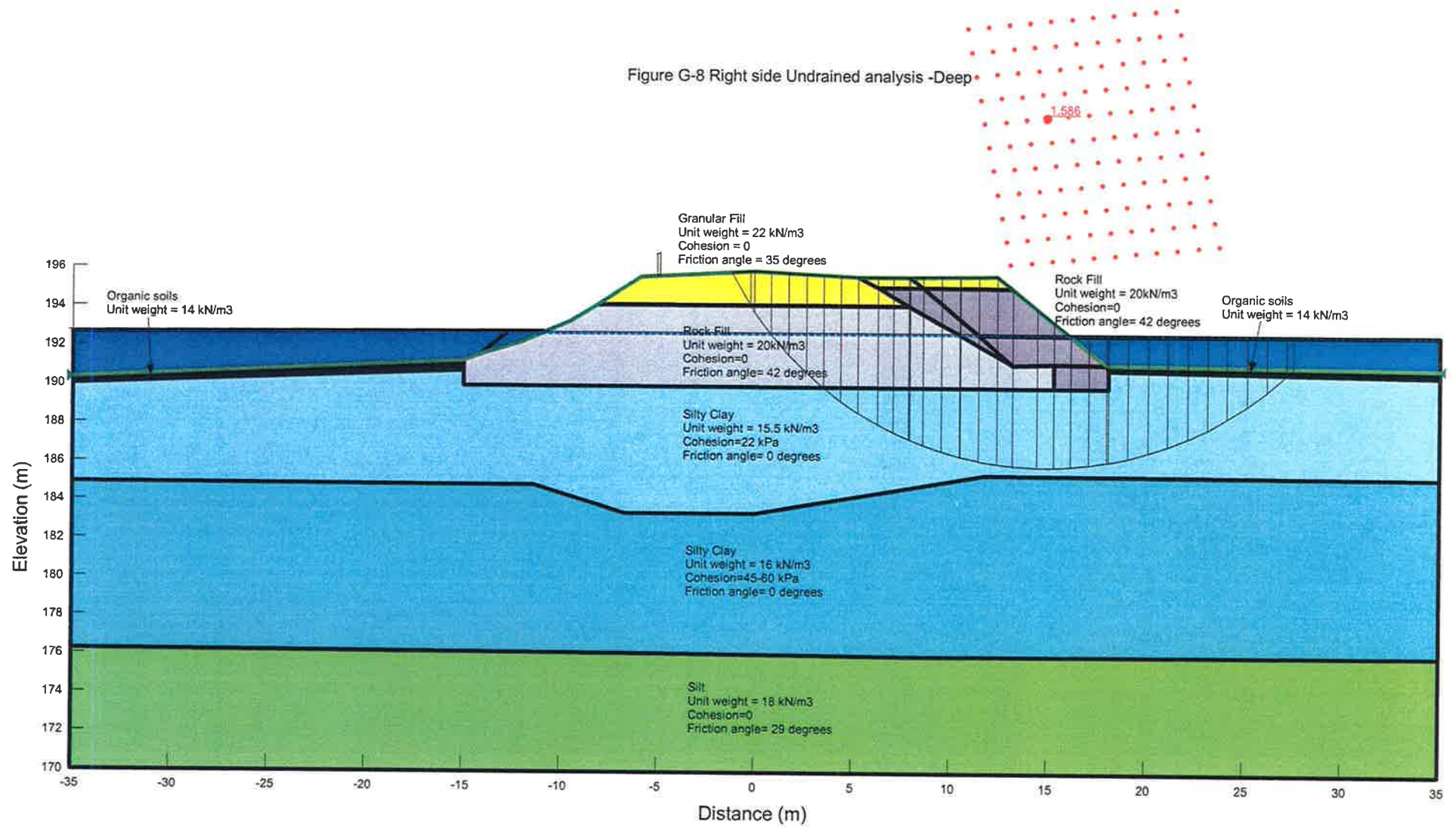


Figure G-9 Right side Undrained analysis -Block

2.524

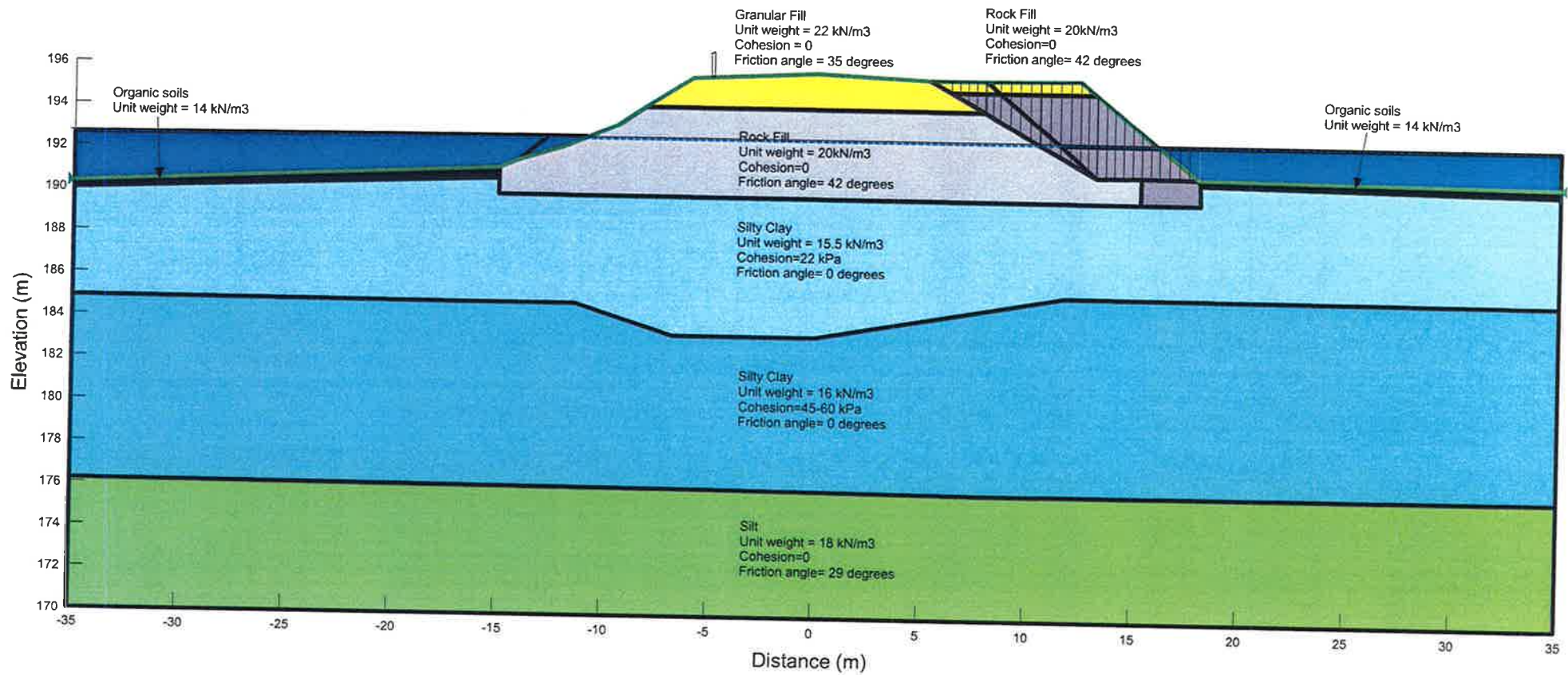


Figure G-10 Right side Undrained analysis (Staging plan)-Block

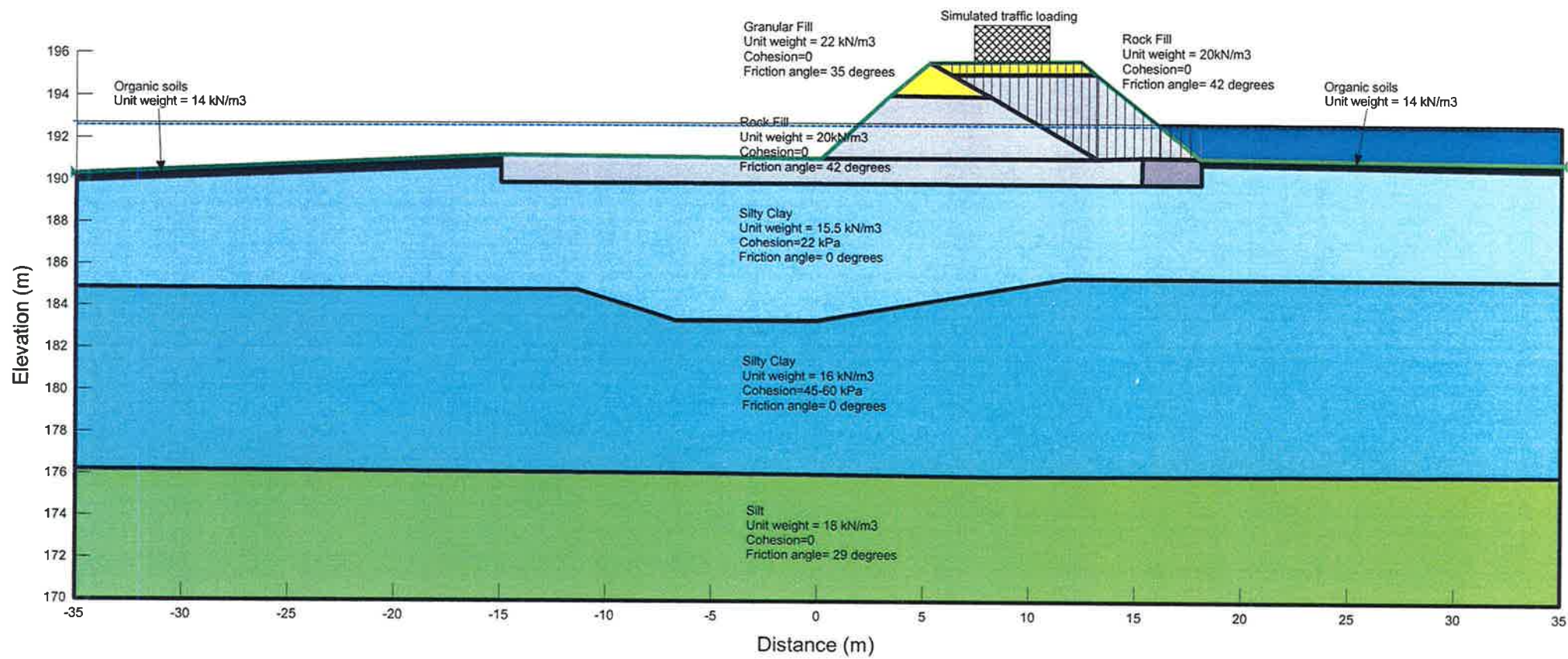


Figure G-11 Right side Drained analysis -Deep

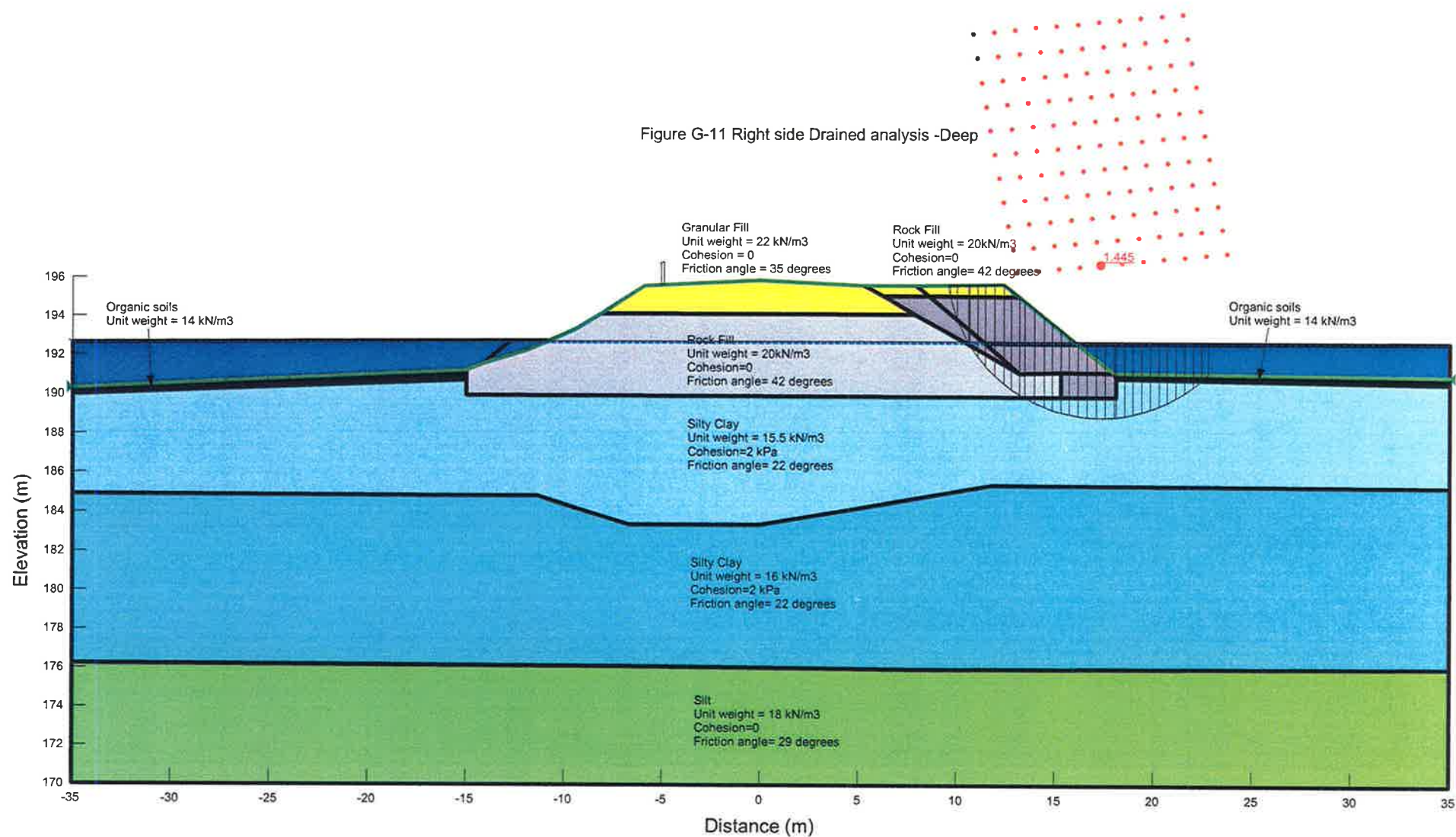


Figure G-12 Right side Drained analysis (Staging plan)-Deep

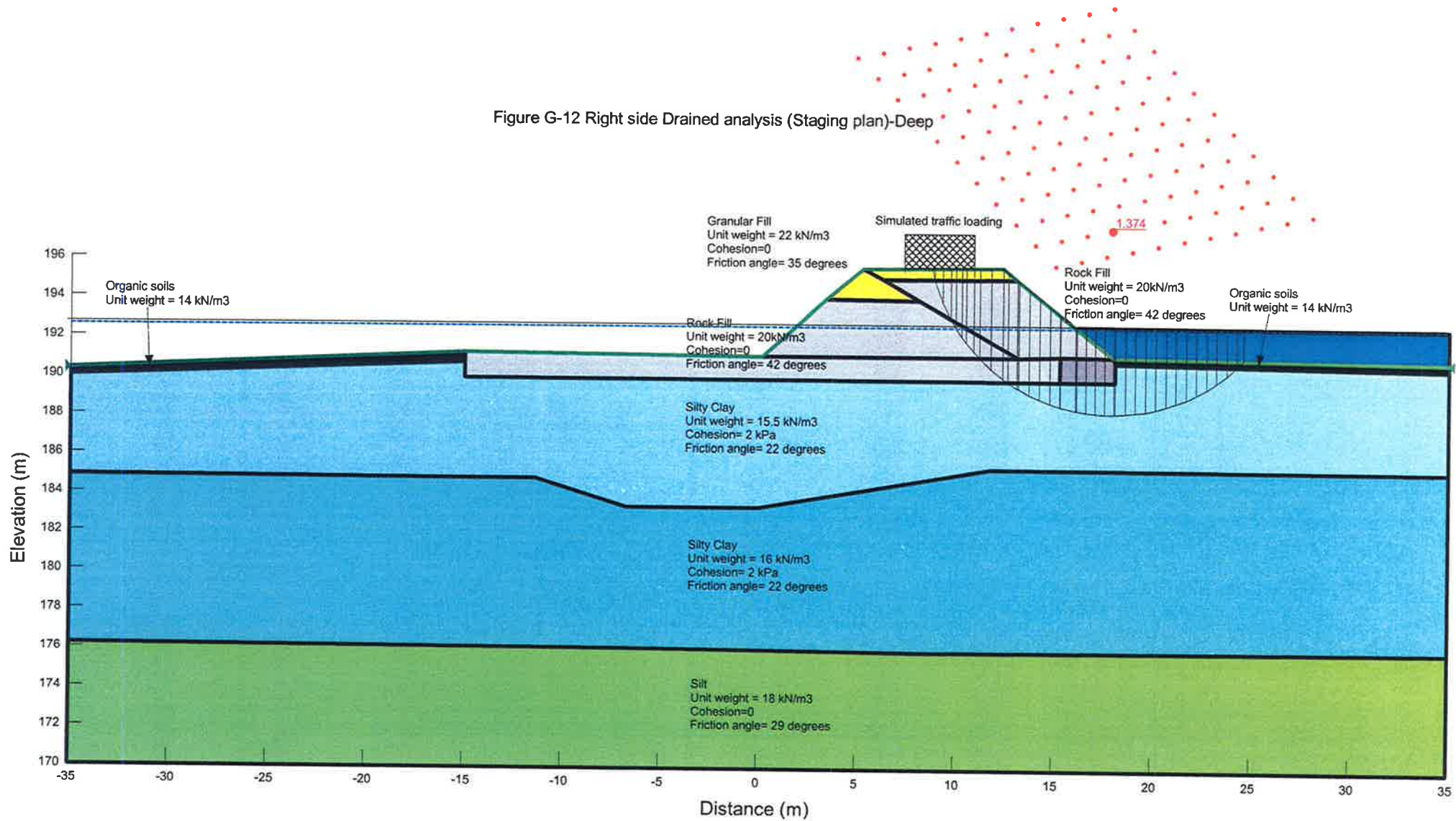


Figure G-13 Right side Unrained analysis

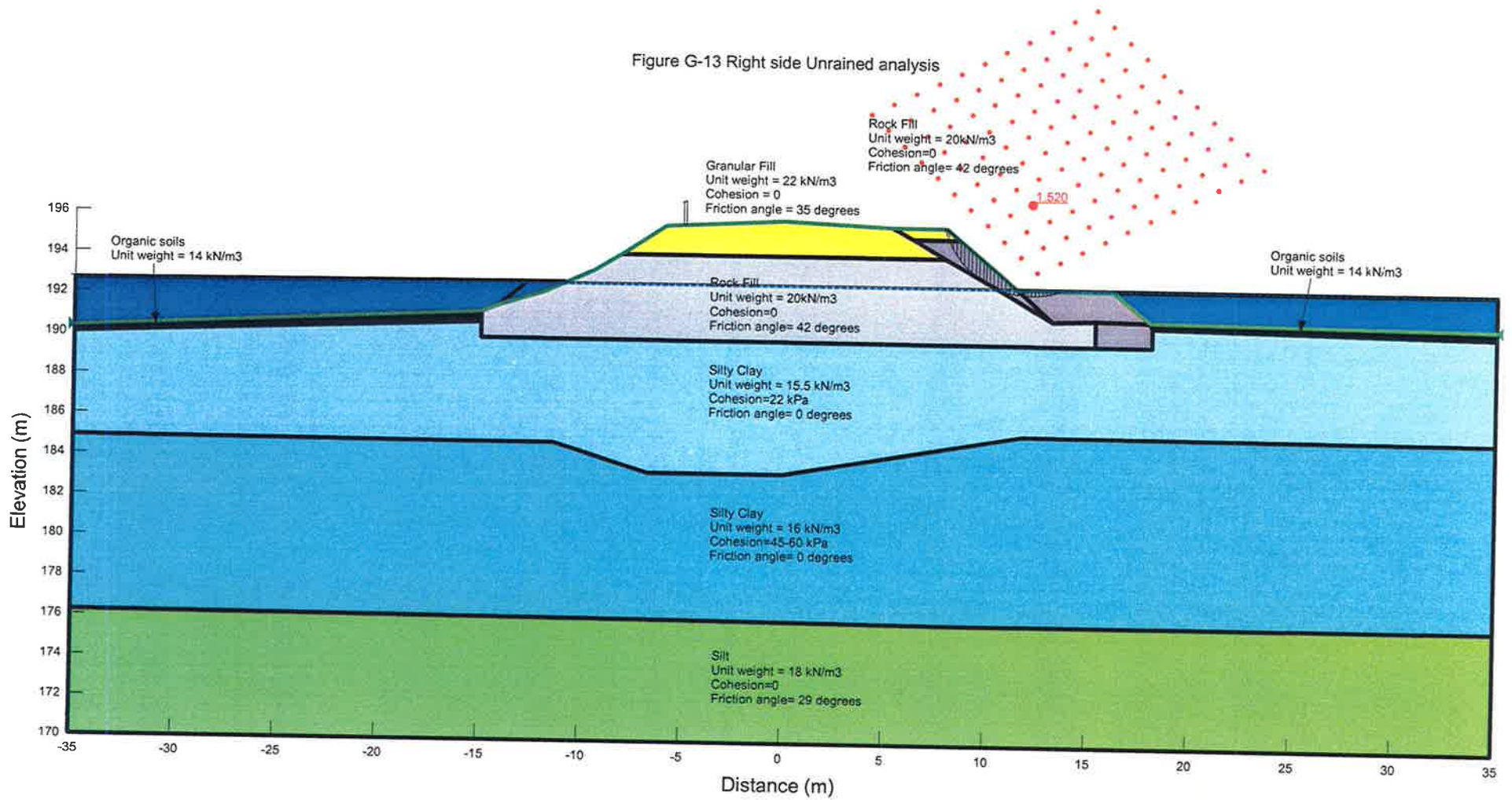


Figure G-14 Right side Undrained analysis -Deep

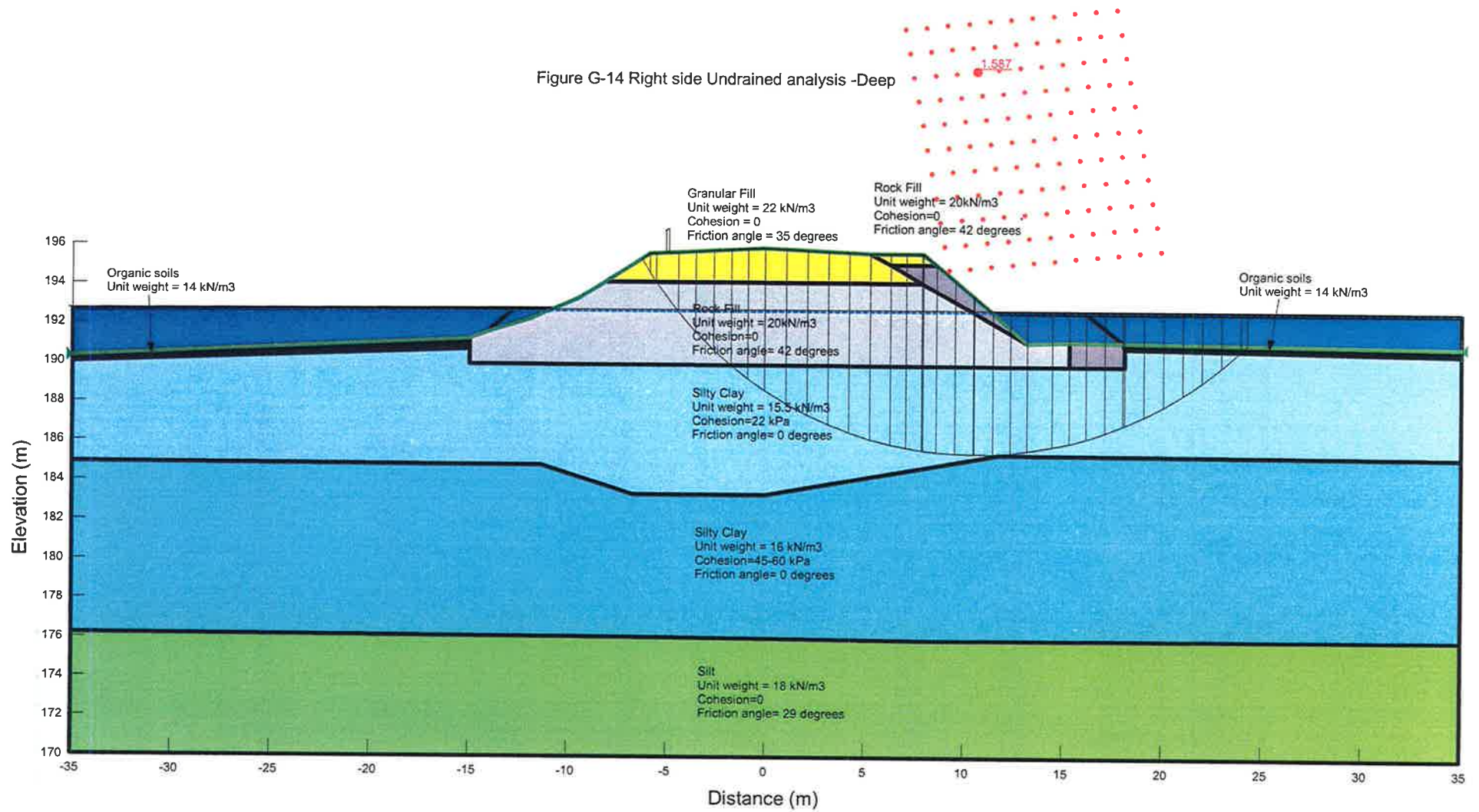


Figure G-15 Right side Undrained analysis -Block

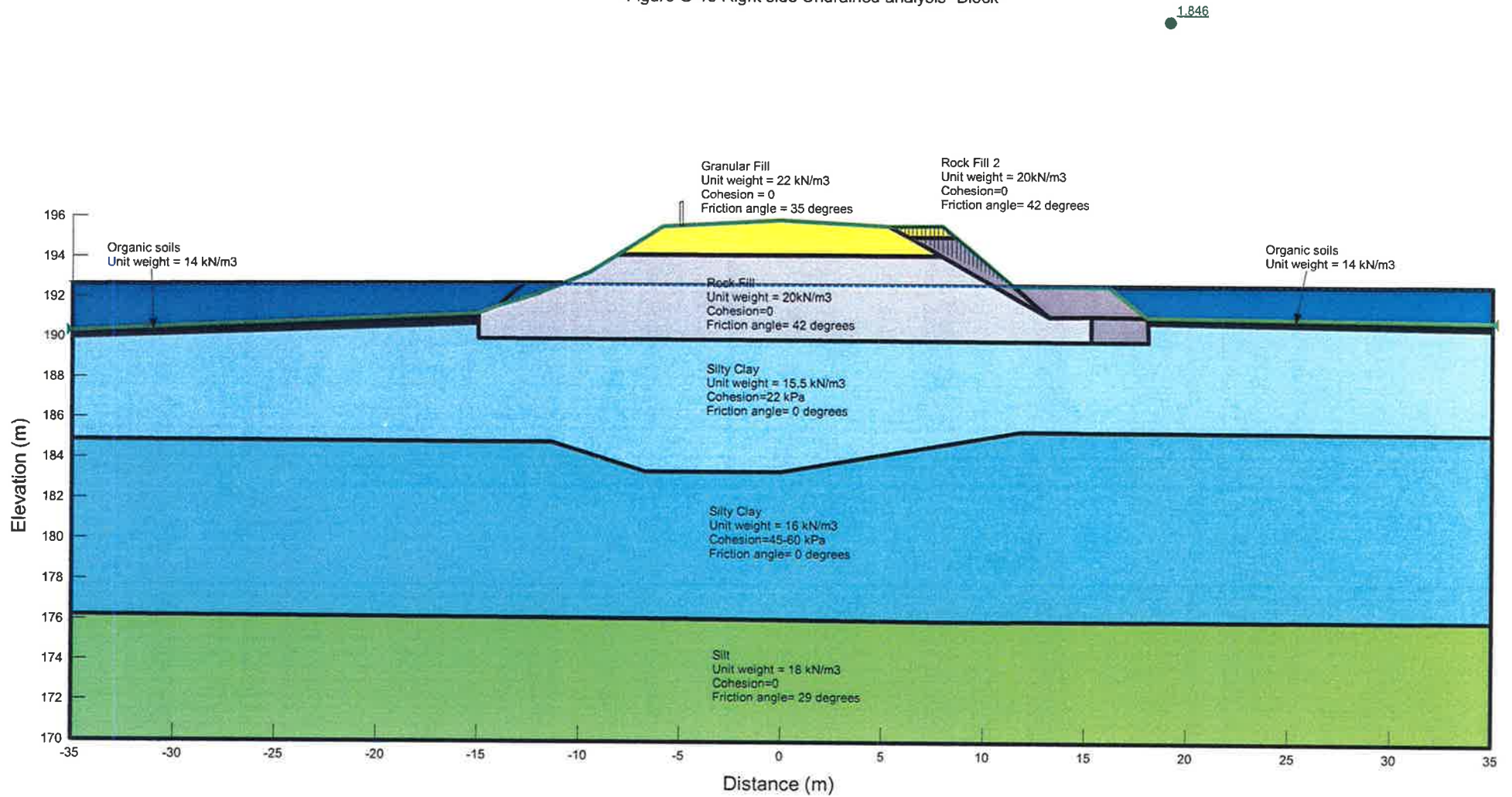
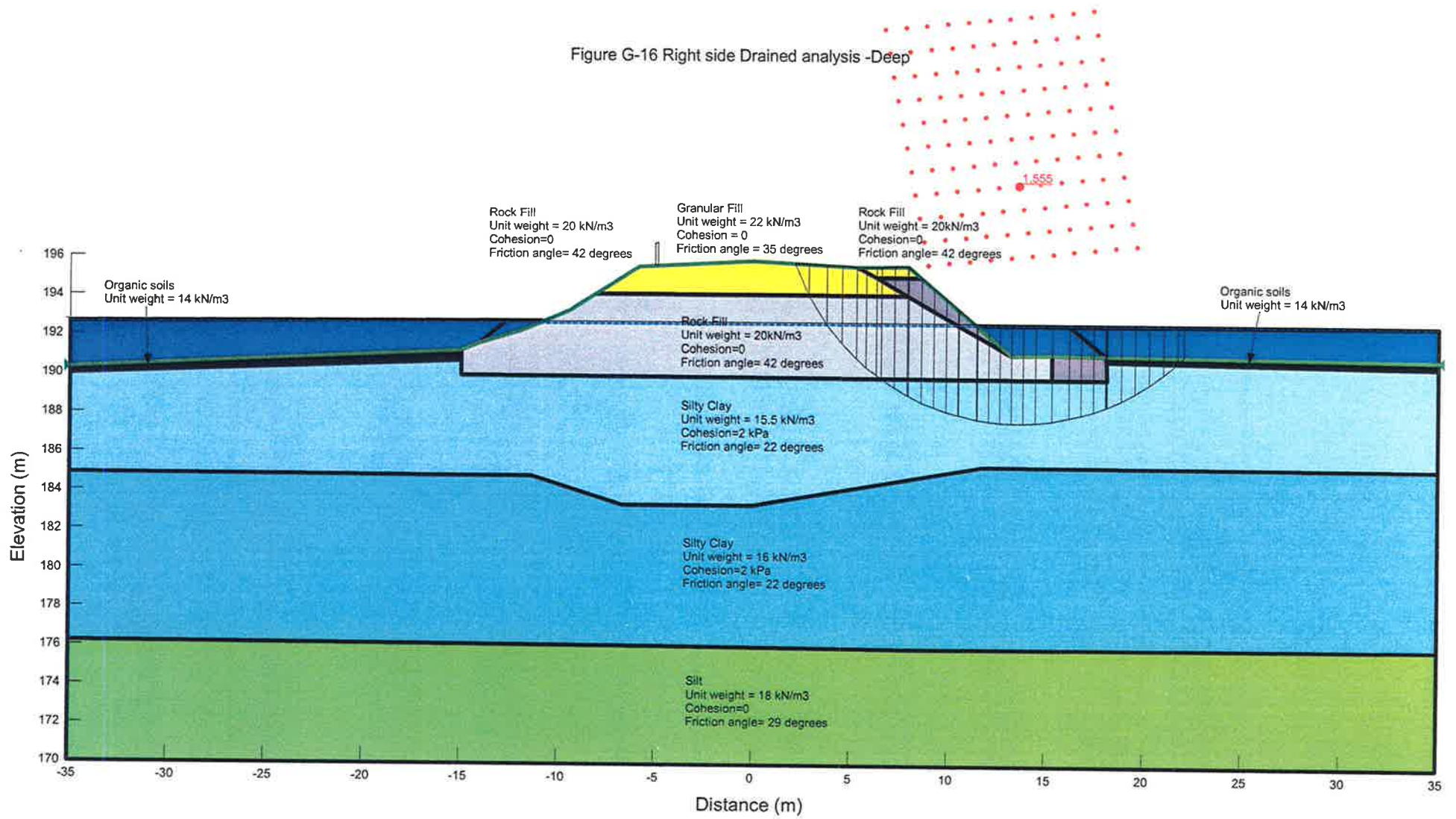


Figure G-16 Right side Drained analysis -Deep



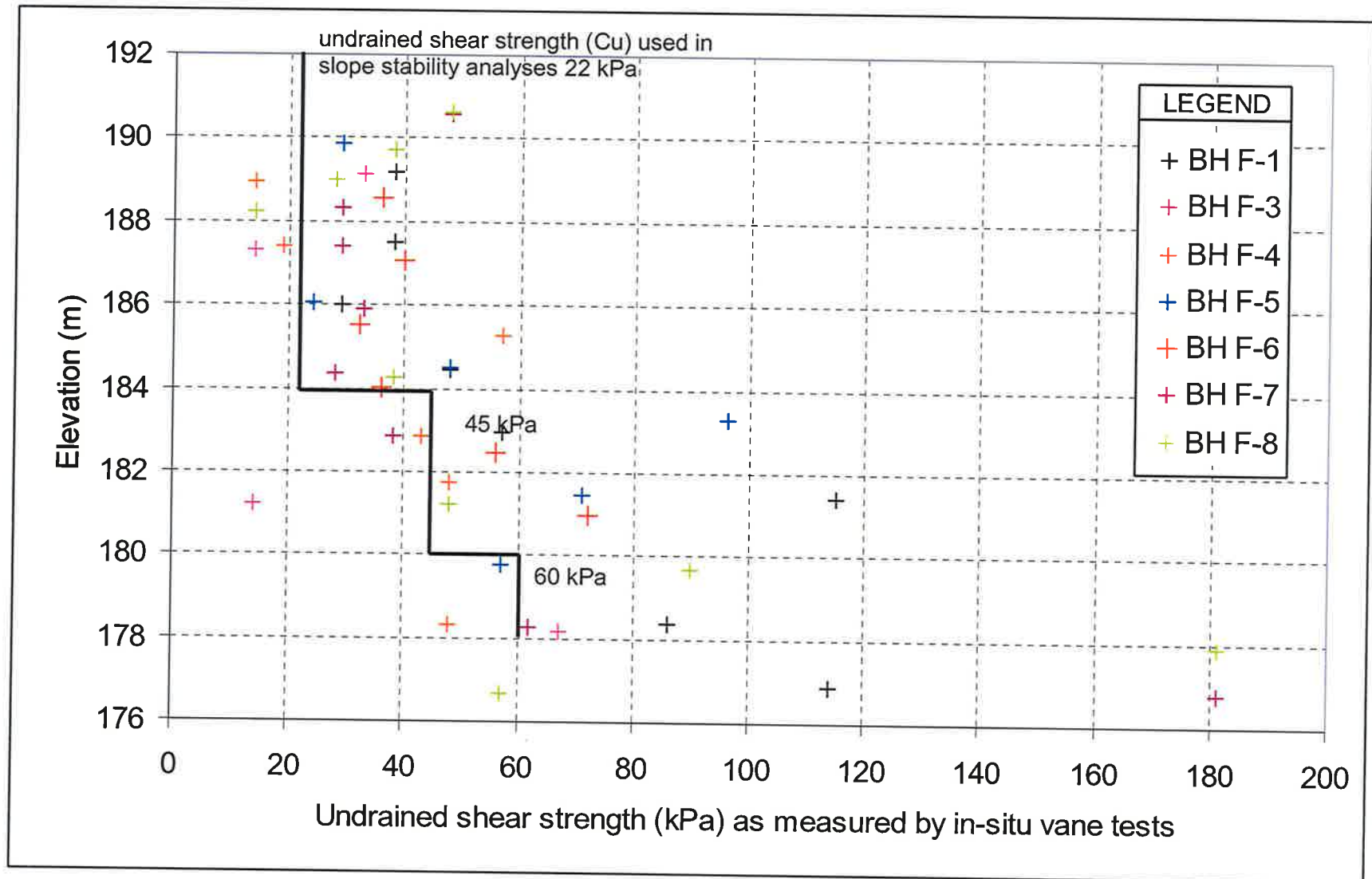


Figure G-17

Appendix H

Limitations of Report

LIMITATIONS OF REPORT

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

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

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

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

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