

**FOUNDATION INVESTIGATION AND DESIGN REPORTS  
HIGHWAY 6 (NEW) AND BUTTER ROAD  
HAMILTON, ONTARIO  
W.P. 605-00-01**

**Prepared For:**

**MINISTRY OF TRANSPORTATION – CENTRAL REGION**

**Prepared by:**

**SHAHEEN & PEAKER LIMITED**

**Project: SPT1120  
December 14, 2009**



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### DRAWING

### DRAWING No.

BOREHOLE LOCATION PLAN & SOIL STRATA

1

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**FOUNDATION INVESTIGATION REPORT  
HIGHWAY 6 (NEW) AND BUTTER ROAD  
HAMILTON, ONTARIO  
W.P. 605-00-01**

## **1. INTRODUCTION**

Shaheen & Peaker Limited (S&P) was retained by the Ministry of Transportation – Central Region to carry out a foundation investigation at the site of proposed Highway 6 (New) Underpass at Butter Road in Hamilton, Ontario.

The site is located at Butter Road about 1 km east of Fiddlers Green Road and about 2 km southwest of the Hamilton Airport in the City of Hamilton.

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

The findings of the investigation are presented in this report.

## **2. SITE DESCRIPTION AND GEOLOGY**

The project site is located on Butter Road, which was, at the time of our investigation, re-aligned for the construction of the proposed Highway 6 underpass structure and roadway. Advance approach fills had been placed to a height of about 7 m above existing grades in order to pre-load the site. Under some portion of the advance fills and in between, the presence of the previously existing Butter Road granular pavement fills can be expected (see site photographs presented in Appendix D). The lands adjacent to the site are generally agricultural. The site is located south of the Niagara Escarpment and in the physiographic region known as the Haldiman Clay Plain. This is a broad, undulating plain of glacio-lacustrine surface sediments that stretches north to south from the edge of the Niagara Escarpment to the Onondaga Escarpment in the south. The plain was submerged under the Lake Warren.

The overburden depth in the general area is generally of the order of 20 m. The underlying bedrock consists of a succession of Paleozoic beds dipping slightly southward and under Lake Erie. The bedrock consists of dolostone of the Guelph Formation, belonging to the Middle Silurian Period of the Paleozoic Era and is approximately 425 million years old.

### 3. METHOD OF INVESTIGATION

The fieldwork for this investigation was performed during the period from January 27 through February 3, 2004 and consisted of drilling and sampling four boreholes at the locations shown on the Borehole Location Plan, Drawing No. 1.

Boreholes 1 and 2 were drilled at the proposed abutment locations, from the existing ground level (i.e. El. 222.3 and 223.0 m, respectively). These boreholes were extended to auger refusal at depths of 21.6 m (El. 200.7 m) and 22.6 m (El. 200.4 m), respectively. Boreholes 3 and 4 were drilled from the top of the existing embankment fill (El. 228.8 m and 228.6 m, respectively). Borehole 3 was extended to a depth of 9.8 m (El. 219.0 m) while Borehole 4 was extended to refusal at a depth of 28.2 m or El. 200.4 m.

The boreholes were advanced using continuous-flight solid-stem or hollow-stem augers. Sampling in the boreholes was effected at frequent intervals of depth by the Standard Penetration Test (SPT) method, as specified in ASTM D1586. This 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 (split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil, which is indicative of the compactness condition of granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

Where the consistency of the soil permitted in cohesive (clayey) deposits, relatively undisturbed samples (TW) were taken with 51 mm and 70 mm diameter thin-walled (Shelby) tubes which were pushed into the bottom of the borehole by the application of static weight by hydraulic pressure. The undrained shear strength of the soil was measured in-situ by means of Field Vane tests. Smaller size Field Vane (51 mm diameter and 102 mm in height) was employed in place of the MTO-Type Field Vane at depths where high undrained shear strength cohesive deposits were encountered.

Drilling and sampling was conducted under the supervision of a geotechnical engineer from S&P.

Groundwater conditions in the open boreholes were observed during the drilling and subsequently, where possible. Upon completion, the boreholes were grouted using a bentonite cement mixture. The recorded water levels are presented on the appropriate Record of Borehole Sheets in Appendix A.

A laboratory testing programme, consisting of natural moisture content measurements, bulk unit weight determination, grain-size analyses, Atterberg Limits tests, one-dimensional consolidation (oedometer) tests (ASTM D2435) and unconsolidated-undrained triaxial compression tests (ASTM D2850) were performed on selected soil samples. The results of

laboratory tests are presented on the appropriate Record of Borehole Sheets and also in Appendix B.

The borehole locations were established in the field by S&P personnel. The geodetic elevations and coordinates of the boreholes were provided by MTO Surveyors.

#### **4. SUBSURFACE CONDITIONS**

Boreholes 1 and 2, which were drilled from the toe of the existing advance fill embankments, show the presence of about 0.4 m and 1.7 m of clayey silt fill, underlain by a 1.6 m and 0.6 m thick layer of crushed limestone layer, respectively (i.e. to El. 220.3 m and 220.7 m, respectively). In Borehole 2, the crushed limestone is underlain by a clayey silt fill which extends to El. 220.1 m. Boreholes 3 and 4, which were put down from the top of the advance approach fill embankment, encountered approximately 6.7 m and 9.3 m clayey silt fill. In Borehole 3, the embankment fill (i.e. clayey silt) was found to be underlain by a 1.1 m thick crushed limestone layer. In these boreholes (i.e. Boreholes 3 and 4), the fill extends to El. 221.0 and 219.3 m, respectively. Underlying the fill and in some of the boreholes, a surficial silt layer, the overburden consists of an extensive clayey silt deposit with some silty clay zones. This deposit is between 18.6 and 19.7 m thick extends to the surface of dolostone bedrock at about El. 200.5 m.

Details of the subsurface conditions encountered in the boreholes drilled for this investigation are presented on the Record of Borehole Sheets in Appendix A. The plan locations of the boreholes, along with a subsurface profile, are presented in Drawing No. 1. The Records of Boreholes, drilled at the site by others in October 2000, are given in Appendix E. The individual strata encountered in the boreholes drilled for the present investigation are briefly described in the following paragraphs.

##### **4.1 FILL**

###### **4.1.1 EMBANKMENT FILL**

Boreholes 3 and 4 were drilled from the top of the advance embankment fill and these two boreholes contacted a clayey silt fill which extends to 6.7 m (El. 222.1) and 9.3 m (El. 219.3 m), respectively.

The grain-size distribution of samples from the fill is given in Figures B-1, B-3 and B-6 in Appendix B. These indicate the following grain-size distribution:

Gravel:	0 – 3%
Sand:	8 – 12%
Silt:	71-80%
Clay:	9-18%

Atterberg limits tests performed in the laboratory on selected samples yielded the following index values, as shown in Figures B-2, B-4 and B-7.

Liquid Limit: 22-26%  
Plastic Limit: 15-18%  
Plasticity Index: 6- 8%

These results are characteristic of clayey soils of low plasticity (i.e. clayey silt to silty clay materials). Visual and tactile examination of the soil samples indicated that the material behaves more like a cohesive silt, rather than a clayey silt/silty clay soil. It is also noted that the material exhibits a relatively high degree of dilatancy.

The measured moisture contents of samples recovered from these advance fill materials are given on the individual Record of Borehole Sheets, as well as in Figure C-1, in Appendix C.

Two Standard Proctor Compaction tests were performed, on bulk samples obtained from the embankment fill materials generally from 1.5 to 3 m below the top of the fill. These laboratory tests yielded the following results:

Standard Proctor Maximum Dry Density = 1825 and 1837 kg/m<sup>3</sup> or  
(17.9 and 18.0 kN/m<sup>3</sup>)  
Optimum Moisture Content = 14.8 and 13.9%

The natural moisture contents of these two samples tested were measured to be 17.6-18.1 and 18.2-18.6%.

Standard Penetration Resistances (N-values) of the fill obtained by Standard Penetration tests in the field, ranged from 3 to 15 blows/0.3 m; the lower values being recorded generally in the upper 3 to 4 m of the fill embankment. Plots of the measured N-values versus depth are given in Figure C-2 in Appendix C.

In Boreholes 1 and 2, which were drilled from near the toe of the advance embankment fill, a clayey silt fill was encountered immediately below the ground surface, extending to depths of 0.4 m and 1.7 m, respectively. This fill is believed to be material placed at the site while placing the advance fills.

#### 4.1.2 GRANULAR FILL

The embankment fill in Borehole 3 and the surficial clayey silt fill encountered in Boreholes 1 and 2 (described in the preceding section) are underlain by a granular fill consisting of gravel and sand. This material consists mostly of crushed limestone and is believed to have constituted the Butter Road base materials (before the road was realigned for the construction of the proposed bridge and Highway 6 (New) under the structure). The thickness of this granular material was found to be 1.6 m/0.6 m/1.1 m at Boreholes 1, 2 and 3, respectively. The recorded N-values in this deposit were 15, 21, 33 and in excess of 50 blows/0.3 m, indicating a compact to very dense condition.

#### 4.2 TOPSOIL

An approximately 0.1 m thick silty topsoil layer was contacted in Borehole 3, underlying the gravel and sand (crushed limestone) fill. In Borehole 4, the presence of some organics and topsoil mixture was noted at about 9 m depth at the base of the existing advance embankment fill and in the underlying silt and clayey silt to a depth of about 10 m depth below the ground surface or from about El. 219.6 to 218.7 m.

#### 4.3 SURFICIAL SILT

Underlying the fill deposits, layers of surficial silt were contacted beneath the advance embankment fills in Boreholes 3 and 4, as follows:

Borehole No.	Depth Below Ground Surface/Elev.	Thickness of Silt Layer
3	7.9/220.9 m	0.4 m
	9.0/219.8 m	0.8 m plus (end of borehole)
4	9.3/219.3 m	0.3 m

The recorded N-values in the surficial silt layers encountered in Borehole 3 were 32 and 72 blows/0.3 m indicating a dense condition in the upper silt layer and very dense condition in the lower silt layer.

#### 4.4 CLAYEY SILT

The predominant overburden underlying the site is a massive clayey silt deposit with some silty clay and occasional thin silt seams. This is a cohesive soil and owing to the presence of occasional embedded coarse sand (grits) and gravel-size particles, the material resembles a glacial till. This deposit was encountered in all the boreholes, except for Borehole 3, which was terminated at a shallow depth before encountering this deposit.



In the deep boreholes (i.e. Boreholes 1, 2 and 4), which were extended to refusal, the deposit was found to extend to the surface of the inferred bedrock except at Borehole 1, where a 0.2 m thick zone of rock fragments in a clayey silt matrix was encountered, immediately above the inferred bedrock surface. The following are the recorded thickness of the deposit in the boreholes.

Borehole No.	Top Elevation of Clayey Silt Stratum	Bottom Elevation of Clayey Silt Stratum	Thickness of Clayey Silt Stratum
1	220.3 m	200.9 m	19.4 m
2	220.1 m	200.4 m	19.7 m
4	219.0 m	200.4 m	18.6 m

Grain-size distribution analysis of six selected samples from the deposit yielded the following particle size distribution.

Gravel:	0 – 2%
Sand:	1 – 16%
Silt:	53 - 87%
Clay:	9 - 46%

The results are presented in an envelope form in Figure B-9 in Appendix B.

As shown in Figure B-10, Appendix B, Atterberg limits tests were performed on 17 samples and these gave the following results.

Liquid Limit:	19-42%
Plastic Limit:	15-22%
Plasticity Index:	3-20%

These results are characteristic of clayey soils of low to medium plasticity, but generally low. The measured natural moisture contents generally range from 15% to 32%. The measured natural moisture contents are generally somewhat closer to the measured plastic limits rather than the liquid limits. This indicates some degree of possible pre-consolidation (i.e. the deposit probably carried in the past higher overburden pressures than the presently existing conditions). A plot of measured plasticity index values versus elevation is given in Figure C-3 in Appendix C.

An unusual feature of these laboratory test results is that with most soils, the measured clay-size percentages are normally associated with higher plasticity index values than the values measured for this deposit (except for Sample 6 from Borehole 2). Another unusual feature is that the samples of the material obtained from the boreholes showed a somewhat higher degree of dilatancy than would be expected from soil containing a relatively high percentage of clay sizes as measured. This rather unusual property can perhaps be

caused by clay-size particles being rather inactive. Chapman and Putnam observed this behaviour many years ago and offered the following hypotheses on similar soils as an explanation, “ . . . Mechanical analyses indicate about 50% clay and 40% silt, but its behaviour is more like that of silt than clay. It is very slippery when wet and inclined to be mealy when dry. It is probably composed of freshly ground rock flour rather than weathered clay materials.”\*

The results of Standard Penetration tests (N-values) recorded in this deposit ranged from 8 to 53 blows/0.3 m in Boreholes 1 and 2 which were drilled at the toe of the existing advance fills. These results indicate firm to hard consistency, but generally very stiff.

In Borehole 4, which was drilled from the top of the existing (approximately 7 to 9 m high) advance approach fills yielded somewhat higher values, which range from 15 to 57 blows/0.3 m. These relatively higher values may possibly reflect the influence of the existing fills, especially immediately beneath the embankment.

A plot of N-values recorded in the clayey silt deposit in Boreholes 1, 2 and 4 is given in Figure C-4 in Appendix C.

The undrained shear strength of the deposit was measured by means of field vane tests as well as quick triaxial compression tests performed in the laboratory (two tests). The field vane tests yielded in-situ undrained shear strengths ranging from 70 to in excess of 240 kPa, while quick triaxial laboratory tests gave values of 173 and 239 kPa. A plot of the measured undrained shear strength versus elevation is given in Figure C-5, Appendix C.

Six one-dimensional consolidation (oedometer) tests were performed on selected thin-walled, open drive Shelby tube (TW) samples. The results of these tests are given in Figures B-11 through B-16 in Appendix B. Five of the six consolidation test curves show probable pre-consolidation pressures of the order of 40 to 120 kPa in excess of the existing overburden pressures (i.e.  $P_c - P_o$ ).

The measured bulk unit weights of samples from the TW samples range from 18.0 to 21.5 kN/m<sup>3</sup>, with an average value of 20.4 kN/m<sup>3</sup>.

#### 4.5 INFERRED BEDROCK

In Boreholes 1, 2 and 4, refusal to further augering was encountered at depths of 21.6 m, 22.6 m and 28.2 m, respectively or at El. 200.7 to 200.4 m in the boreholes. From these and the behaviour of the sampler and drilling rods (i.e. bouncing), it is believed that these depths/elevations represent the surface of the bedrock.

---

\* Chapman, L. J. and Putnam, D.F., “The Physiography of Southern Ontario,” Ontario Geological Survey, Special Volume 2, Ontario Ministry of Natural Resources.

During the previous investigation carried out by others in 2001, the surface of the bedrock was contacted at depths ranging from 20.8 to 21.2 m or El. 200.7 to 200.9 m and the bedrock was cored and was found to consist of dolostone. Core recovery ranged from 93 to 100% while the R.Q.D. values between 67 to 92% were recorded, indicating a fair to excellent but typically good rock quality.

#### 4.6 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling, upon the completion of each borehole and thereafter, whenever possible. Piezometers were not installed in the boreholes because this was not within our terms of reference. The following water levels were measured in the open boreholes.

Borehole No.	Borehole Depth/Elevation	Recorded Water Level Below Existing Ground Surface/Elevation	Comments
1	21.6/200.7 m	1.1/221.2 m	Completion of borehole
2	22.6/200.4 m	22.6/200.4 m 7.6/215.4 m	Completion of borehole One day after completion
3	9.8/219.0 m	Dry/dry to El. 219.0 m	Completion
4	28.2/200.4 m	27.7/200.9	Completion

Because of the presence of clayey soils (i.e. relatively impervious) these observations are not considered to represent the stabilized groundwater levels.

Piezometers were installed in Boreholes 1 and 4 drilled during the previous investigation carried in October 2001 by others. In Borehole 1, which was extended to 6.5 m below the ground surface or to El. 214.7 m, a water level was recorded at 1.0 m below the ground surface or at El. 220.2 m about three weeks after the completion of the borehole. In Borehole 4 which was extended into the bedrock at 24.2 m below the ground surface or El. 197.7 m, the groundwater level was recorded at 0.3 m or El. 221.7 m, also about three weeks after the completion of the borehole. This water level appears to be emanating from the bedrock.

Based on these findings together with change of colour of the soil from brown to grey, the groundwater table at the site is likely to be generally between Elevations 221 and 218 m and would be subject to seasonal fluctuations. In addition, a perched water condition is likely to occur due to the accumulation of surface water in the granular soils overlying the relatively impervious massive clayey silt deposit.

**SHAHEEN & PEAKER LIMITED**



Fanyu Zhu, Ph.D., P.Eng.\*

A handwritten signature in blue ink, appearing to read "Z.S. Ozden", written over a light blue horizontal line.

Z.S. Ozden, P.Eng.

ZO:tr/idrive

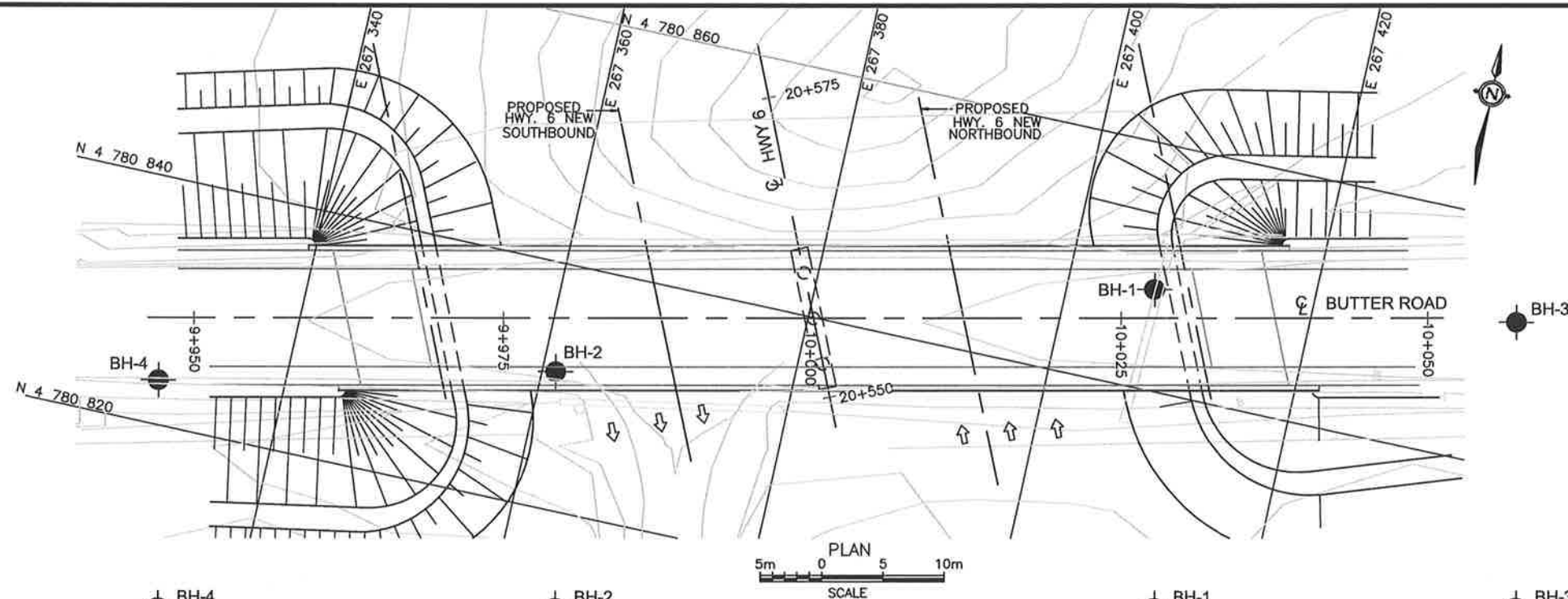
K.R. Peaker, Ph.D., P.Eng.\*\*

\*The original draft report was signed and sealed by Dr. Fanyu Zhu who is no longer with the company.

\*\* The original draft report was signed and sealed by Dr. Ken Peaker who is no longer with the company.

The report was finalized on December 14, 2009 without making any changes to the draft report.

# Drawing



# METRIC

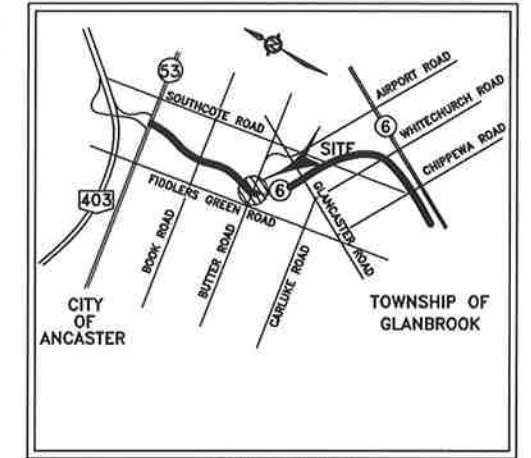
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. STATIONS  
ARE IN KILOMETRES + METRES.

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

CONT No.  
WP: 605-00-01

HIGHWAY 6(NEW) UNDERPASS  
AT BUTTER ROAD  
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITED



KEY PLAN  
N.T.S

## LEGEND

- Bore Hole
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation  
Jan. and Feb., 2004

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
BH-1	222.3	4 780 848.2	267 407.0
BH-2	223.0	4 780 831.2	267 361.1
BH-3	228.8	4 780 851.9	267 436.4
BH-4	228.6	4 780 823.6	267 329.9

## NOTE

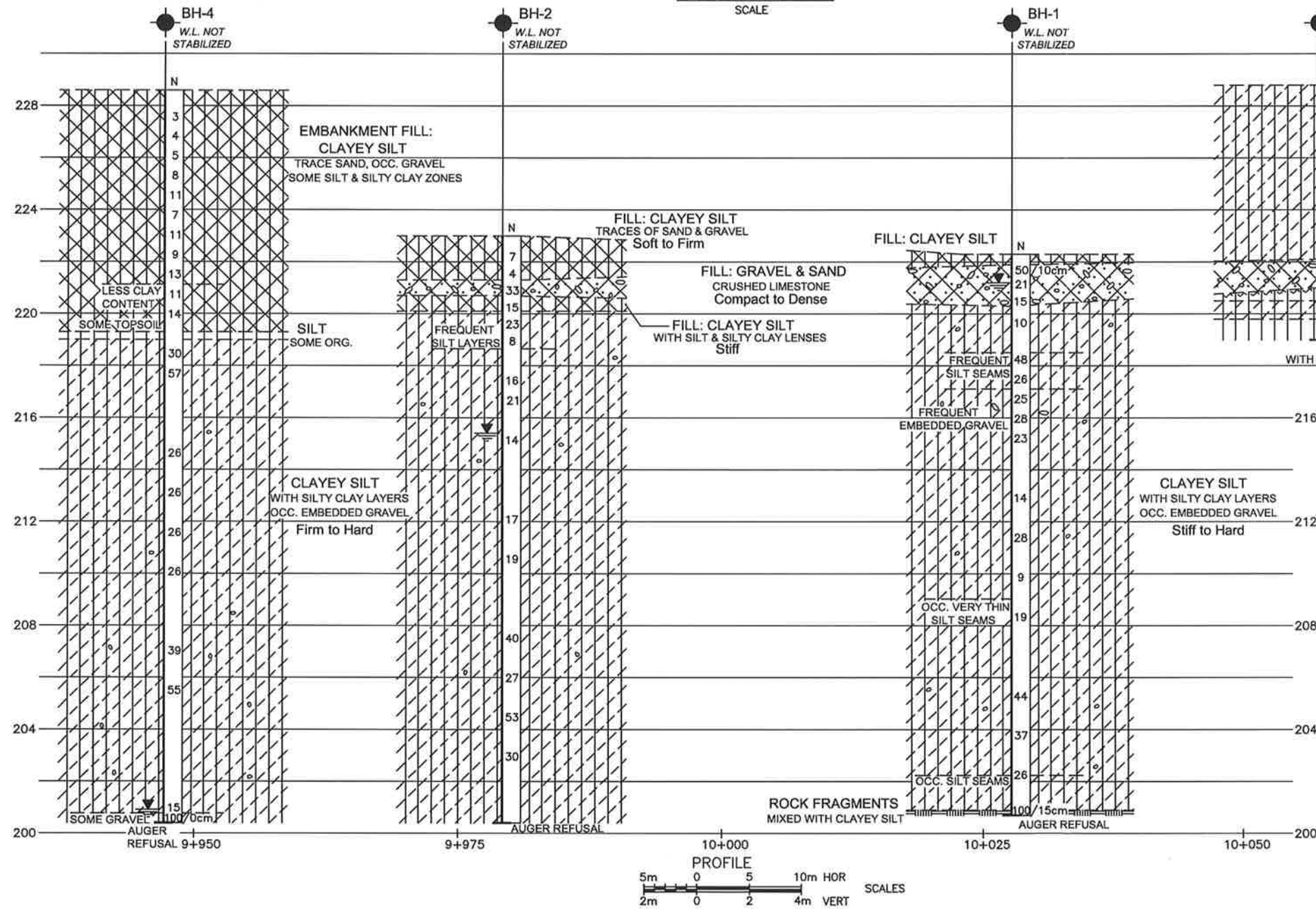
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION

Geocres No.

HWY No. 6	DIST CENTRAL		
SUBM'D ZO	CHECKED FZ	DATE Apr, 2004	SITE 36-496
DRAWN JZ	CHECKED	APPROVED	DWG 1



# Appendix A

## Records of Boreholes

SPT 1120

## 1 OF 2

METRIC

WP	605-00-01	LOCATION	Butter Road, Hamilton; Sta:10+027.7, 2.4 m Lt; - Coords: N 4 780 848.2; E 267 407.0	ORIGINATED BY	Y.L.
DIST	Central	HWY	6	BOREHOLE TYPE	Hollow Stem Augers
DATUM	Geodetic	DATE	1/27/2004	COMPILED BY	J.Z.
				CHECKED BY	Z.O.

[illegible]

Continued Next Page

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




SPT 1120

# RECORD OF BOREHOLE No 1

2 OF 2

METRIC

WP 605-00-01 LOCATION Butler Road, Hamilton; Sta:10+027.7, 2.4 m Lt; - Coords: N 4 780 848.2; E 267 407.0 ORIGINATED BY Y.L.  
DIST Central HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY J.Z.  
DATUM Geodetic DATE 1/27/2004 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL		
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE								WATER CONTENT (%)	
207.3 15.0	CLAYEY SILT with silty clay layers occ. embedded gravel very stiff to hard		17	TW	PH		207							21.5	consolidation test		
					18		SS	44									21.1
			19	SS	37		204							20.5			
							203										
			20	SS	26		202										
200.9 21.4	ROCK FRAGMENTS MIXED WITH CLAYEY SILT		21	SS	100/15		201										
200.7 21.6	End of Borehole.  Rods bouncing. Auger refusal at 21.6 m probable on bedrock.  Borehole open to 1.5 m on completion.  *Water level on: Jan.27, 2004 - 1.1 m (El. 221.2 m)-Completion (not stabilized)																

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

20  
15  
10  
(%) STRAIN AT FAILURE

SPT 1120

# RECORD OF BOREHOLE No 2

1 OF 2

METRIC

WP 605-00-01 LOCATION Butter Road, Hamilton; Sta: 9+979.2, 4.4 m Rt; - Coords: N 4 780 831.2; E 267 361.1 ORIGINATED BY Y.L.

DIST Central HWY 6 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.

DATUM Geodetic DATE 1/28/2004 to 1/29/2004 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa								WATER CONTENT (%)
							○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
223.0	Ground Surface		1	AS	-										
0.0	FILL: CLAYEY SILT traces of sand and gravel brown, soft to firm, wet		2	SS	7										
			3	SS	4										
221.3			4	SS	33										
1.7	FILL: GRAVEL & SAND (crushed limestone) grey, dense													sampler wet	
220.7	FILL: CLAYEY SILT with silt and silty clay lenses brown, stiff		5	SS	15										
2.3			6	SS	23										
220.1	CLAYEY SILT with silty clay layers occasional embedded gravel firm to very stiff		7	SS	8									1 3 87 9	
2.9			8	TW	PH									0 1 53 46	
			9	SS	16									0 2 55 43	
			10	SS	21									consolidation test	
			11	TW	PH									switched from	
			12	SS	14									regular MTO vane	
														to small vane.	
														Jan. 28	
														Jan. 29	
														consolidation test	

Continued Next Page

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



SPT 1120

# RECORD OF BOREHOLE No 2

2 OF 2

METRIC

WP 605-00-01 LOCATION Butter Road, Hamilton; Sta:9+979.2, 4.4 m Rt; - Coords: N 4 780 831.2; E 267 361.1 ORIGINATED BY Y.L.  
 DIST Central HWY 6 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.  
 DATUM Geodetic DATE 1/28/2004 to 1/29/2004 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL							× LAB VANE
208.0 15.0	CLAYEY SILT with silty clay layers occasional silt seams/layers occasional embedded gravel grey, very stiff to hard		17	SS	40	208								21.3	0 2 55 43		
							207										
			18	SS	27		206									20.2	
							205										
			19	SS	53		204										
							203									21.1	
			20	SS	30		202										
							201									20.1	
			21	TW	PH												
200.4 22.6	End of Borehole.  **Auger refusal at 22.6 m. Rods bouncing, no penetration, probably on bedrock.  Borehole open to the full depth on completion.  *Water level on: Jan. 29, 2004 -22.6 m(EI.200.4 m)-Completion Jan. 30, 2004 - 7.6 m(EI.215.4 m) (not stabilized)		22	SS	-**												

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

SPT 1120

# RECORD OF BOREHOLE No 3

1 OF 1

METRIC

WP 605-00-01 LOCATION Butter Road, Hamilton; Sta:10+057.2, 0.3 m Rt; - Coords: N 4 780 851.9; E 267 436.4 ORIGINATED BY Y.L.

DIST Central HWY 6 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.

DATUM Geodetic DATE 1/30/2004 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								○ UNCONFINED    + FIELD VANE				
								● QUICK TRIAXIAL    × LAB VANE				
						WATER CONTENT (%)						
						PLASTIC LIMIT    NATURAL MOISTURE CONTENT    LIQUID LIMIT						
						w <sub>p</sub> w                      w <sub>L</sub>						
228.8 0.0	Ground Surface											
			1	AS	-							
			2	SS	7		228				20.1	
			3	SS	4		227				19.7	
			4	SS	6						19.0	
			5	SS	12		226				19.5	
			6	SS	13						20.5	
			7	SS	12		224				20.2	
			8	SS	10						20.3	
			9	SS	15						20.1	
222.1 6.7			10	SS	50/10		222				22.2	
221.0 7.8			11	SS	32						20.5	
220.5 8.3			12	SS	75/28		220				19.5	
219.8 9.0			13	SS	72						20.4	
219.0 9.8							219					
	End of Borehole.  Borehole dry and open to the full depth on completion (not stabilized).  *Bulk sample from 3 m Liquid Limit: 25% Plastic Limit: 17% Plasticity Index: 8% Natural Moisture: 18.2%											

EMBANKMENT FILL:  
CLAYEY SILT  
trace sand, occasional gravel inclusions  
some silt and silty clay layers/pockets  
brown

FILL: GRAVEL & SAND  
(crushed limestone)  
grey, very dense

0.1 m SILTY TOPSOIL  
SILT  
brown, dense, damp

CLAYEY SILT  
with silty clay layers  
brown, hard

SILT  
with clayey silt and silty clay seams  
brown, hard

End of Borehole.  
Borehole dry and open to the full depth on  
completion (not stabilized).

\*Bulk sample from 3 m  
Liquid Limit: 25%  
Plastic Limit: 17%  
Plasticity Index: 8%  
Natural Moisture: 18.2%

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



SPT 1120

# RECORD OF BOREHOLE No 4

2 OF 2

METRIC

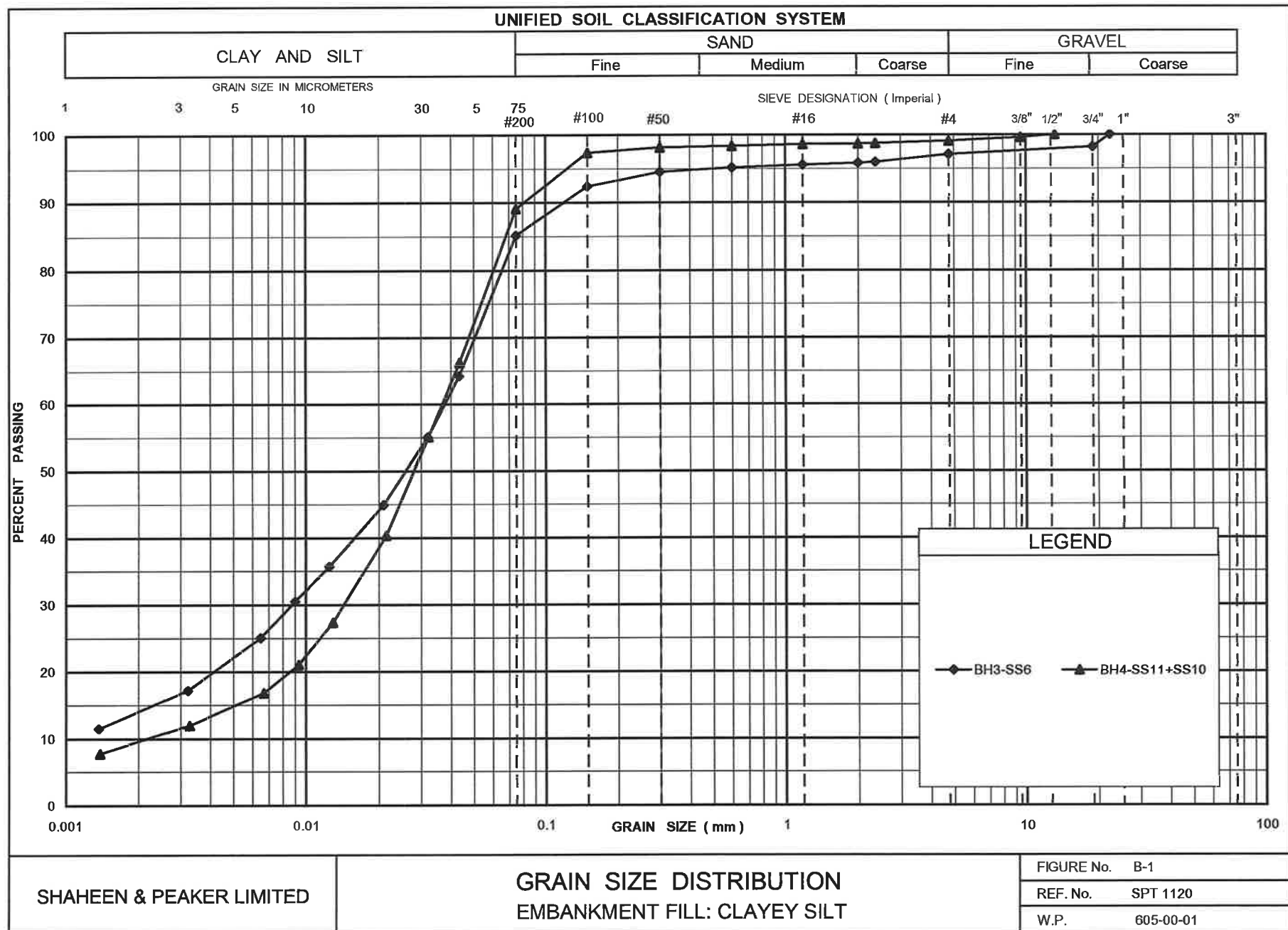
GWP 605-00-01 LOCATION Butter Road, Hamilton, Sta: 9+947.1, 5.1 m RL; - Coords: N 4 780 823.6, E 267 329.9 ORIGINATED BY Y.L.  
DIST Central HWY 6 BOREHOLE TYPE Hollow Stem Augers & Solid Stem Augers COMPILED BY J.Z.  
DATUM Geodetic DATE 2/2/2004 to 2/3/2004 CHECKED BY Z.O.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W			LIQUID LIMIT W <sub>L</sub>
213.6 15.0	CLAYEY SILT trace sand occasional gravel some silty clay zones brown, very stiff to hard		18	SS	26		213				21.2	Feb. 2 ----- Feb. 3	
							212						20.7
			19	SS	26		211						
							210						
			20	SS	26		209						
							208						
			21	TW	PH		207						
							206						
			22	SS	39		205						
							204						
			23	SS	55		203						
							202						
			24	TW	PH		201						
200.4	some gravel		25	SS	15								
28.2			26	SS	100/0								
End of Borehole.													
Field vane bent and damaged. Auger refusal at 28.2 m, probably on bedrock.													
Borehole open to the full depth on completion.													
*Water level on: Feb. 3, 2004 - 27.7 m (El.200.9m)-Completion (not stabilized)													

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

# Appendix B

## Laboratory Test Results



SHAHEEN & PEAKER LIMITED

**GRAIN SIZE DISTRIBUTION**

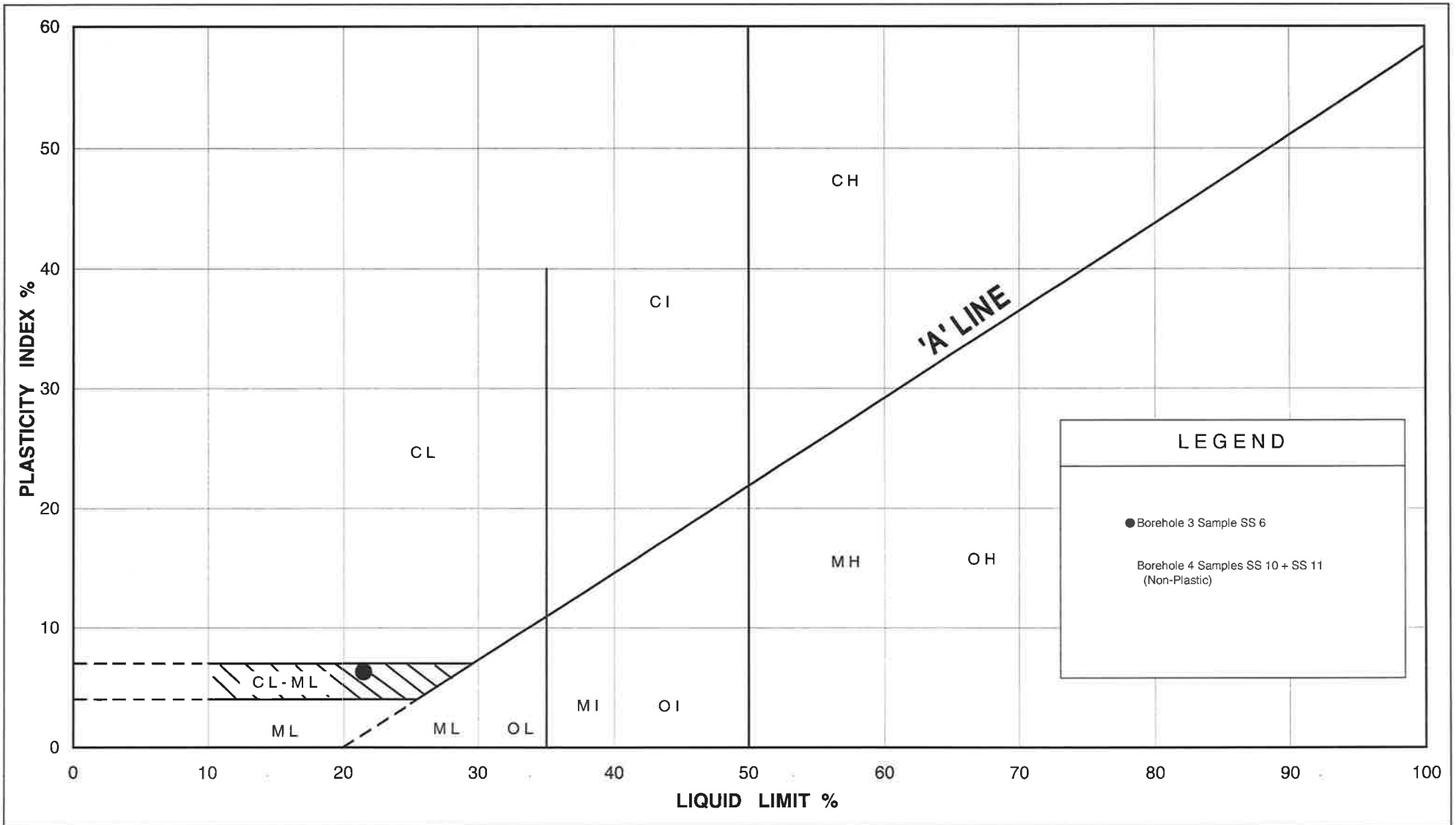
EMBANKMENT FILL: CLAYEY SILT

FIGURE No. B-1

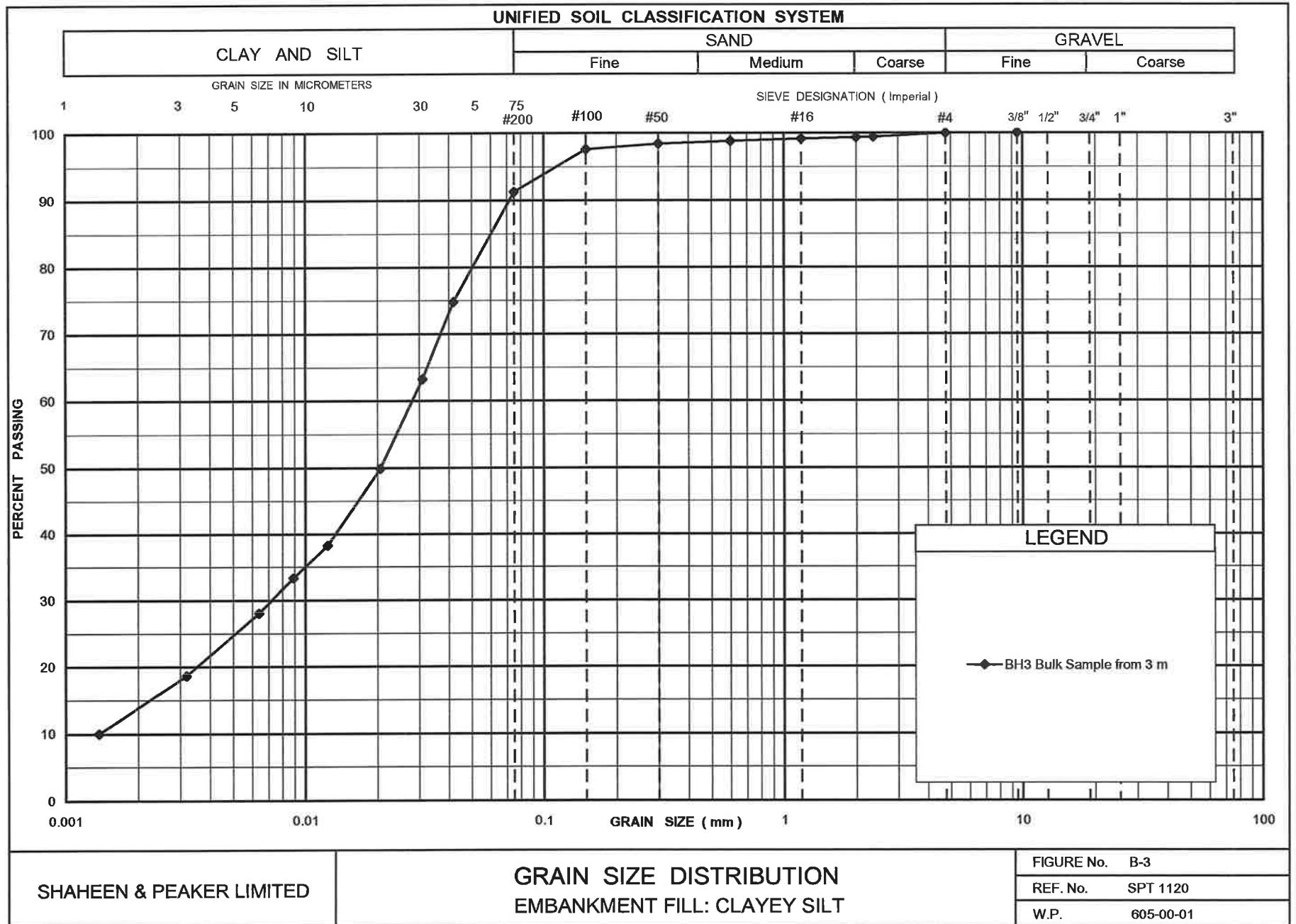
REF. No. SPT 1120

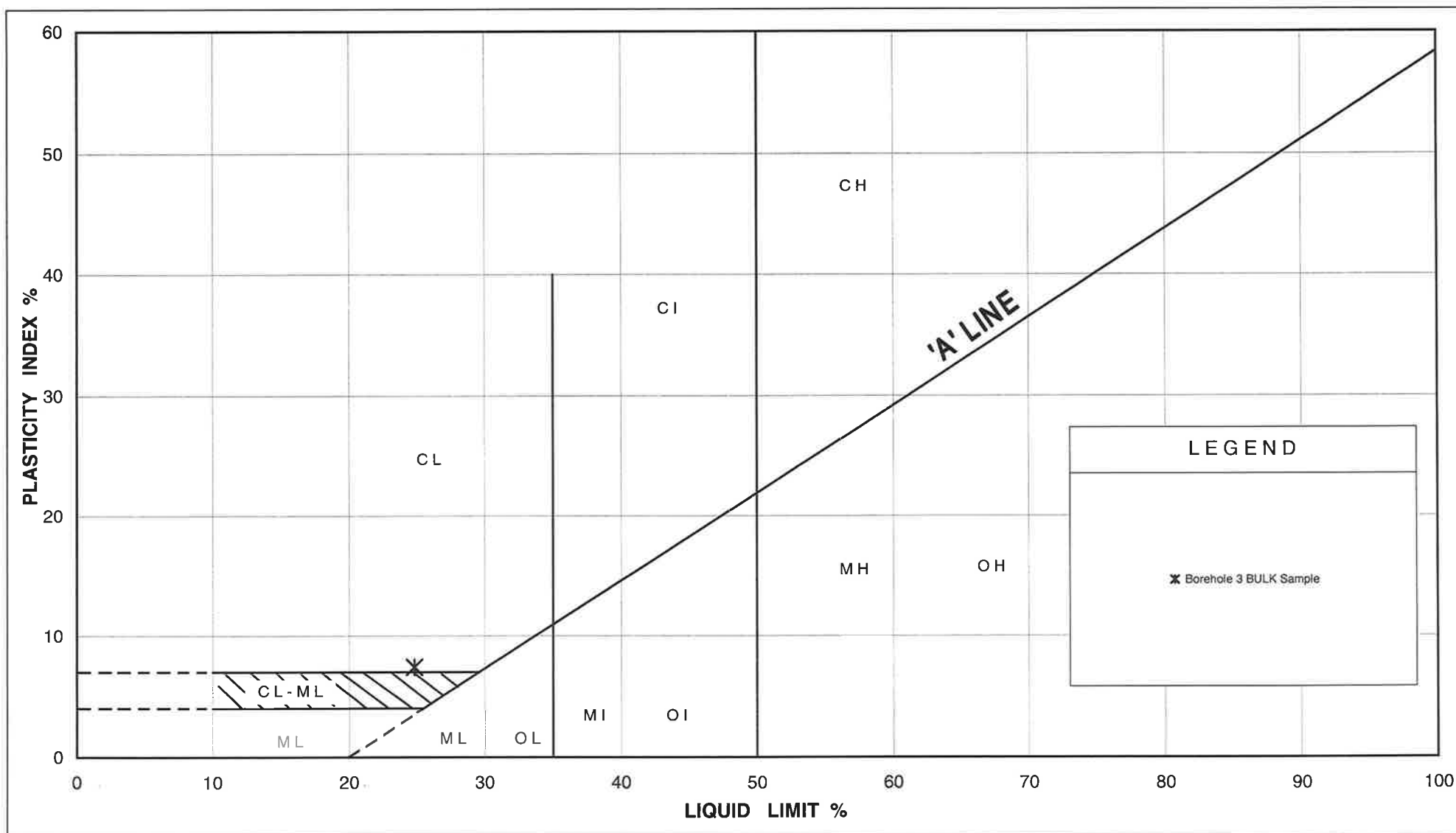
W.P. 605-00-01





SHAHEEN & PEAKER LIMITED	PLASTICITY CHART EMBANKMENT FILL: Clayey Silt	FIGURE No. B-2
		REF. No. SPT 1120
		W.P. 605-00-01





SHAHEEN & PEAKER LIMITED

PLASTICITY CHART  
EMBANKMENT FILL: Clayey Silt

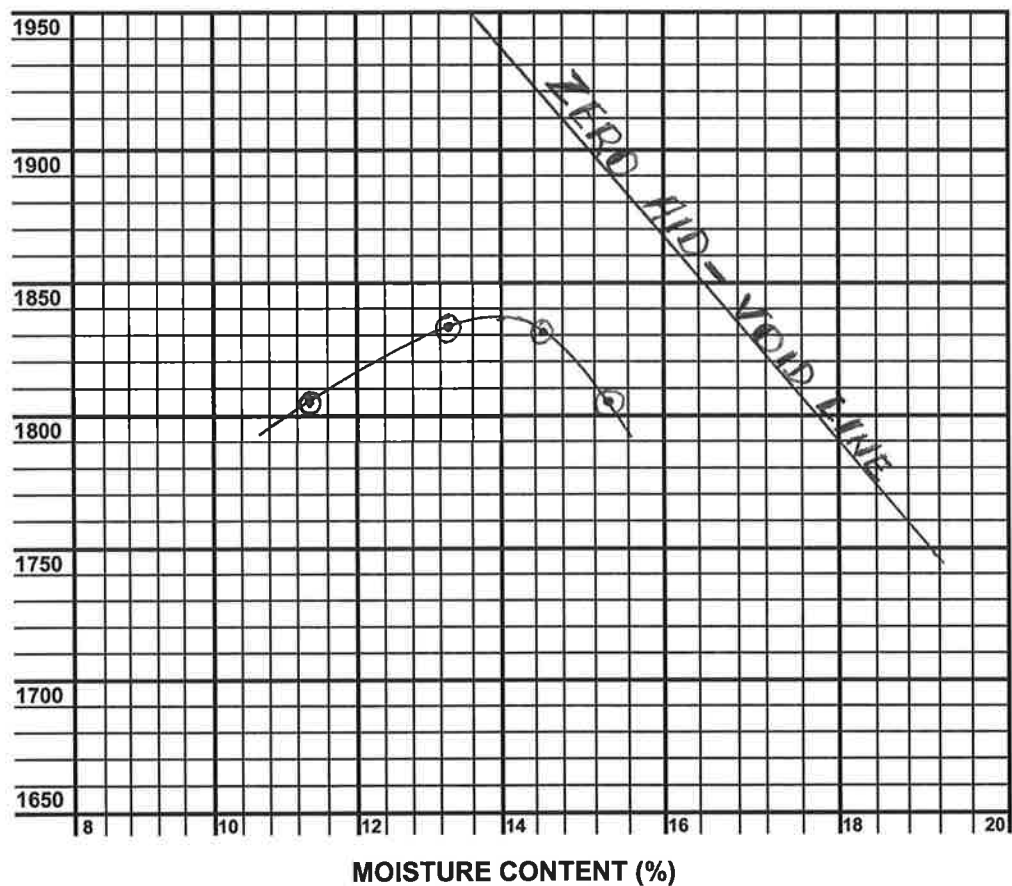
FIGURE No. B-4

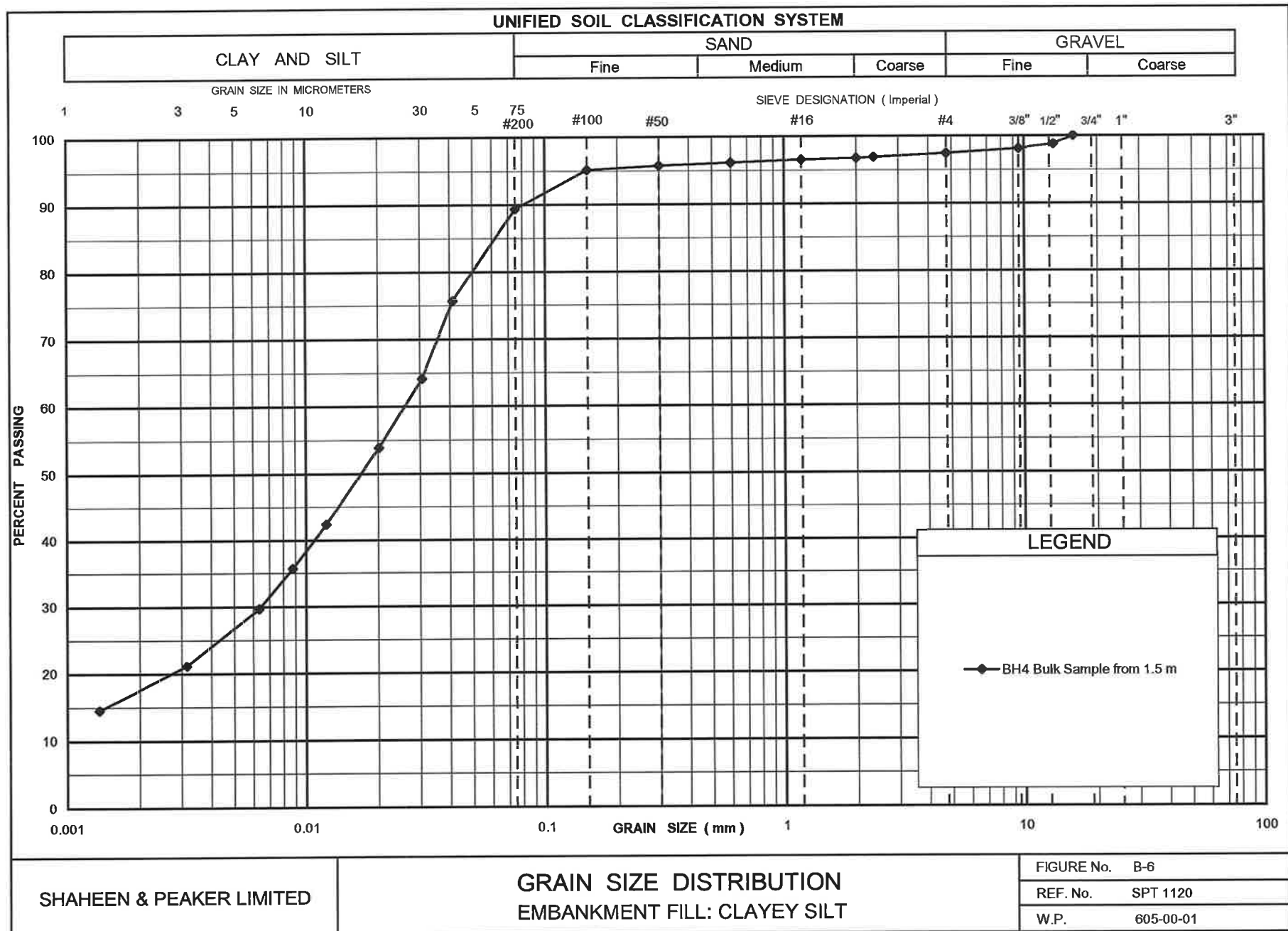
REF. No. SPT 1120

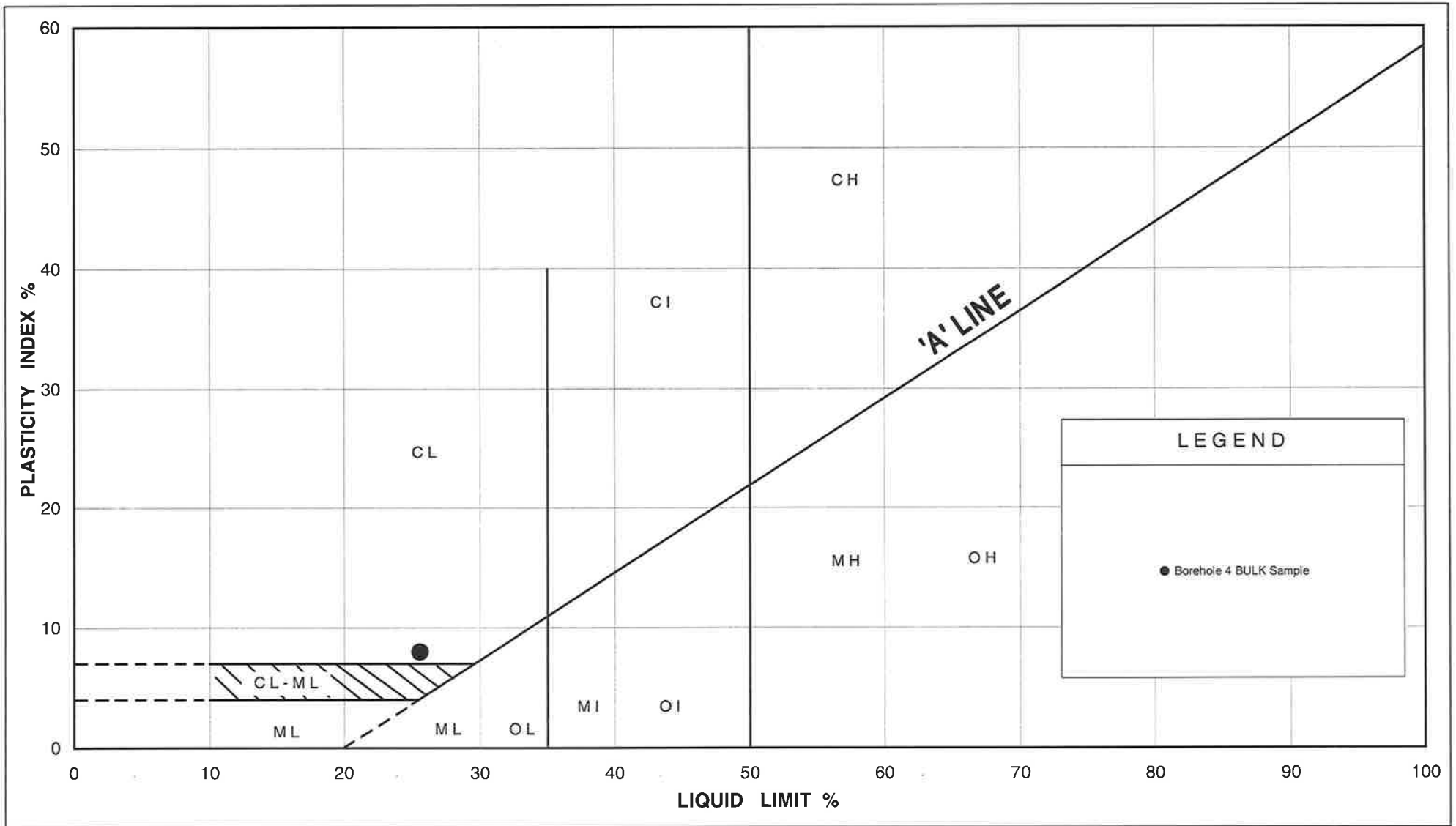
W.P. 605-00-01

**SHAHEEN & PEAKER LIMITED****Consulting Geo-Environmental and Construction Materials Engineers****PROCTOR TEST RESULT**

Project Name:	Highway 6 (New) and Butter Road	
Project No. :	SPT1120	
Material Supplier:	N / A	
Sample Location:	BH – 3 (10 Feet)	
Sampled By :	Wolfe Lang	
Date Sampled :	February 20, 2004	
Laboratory No:	3903	
Proctor Method:	Standard	Method A
Sample Description:	Silt some clay, trace sand	
Maximum Dry Density :	1837 (kg/m <sup>3</sup> )	
Optimum Moisture Content:	13.9 %	
Natural Moisture Content :	18.6 %	

DENSITY (kg/m<sup>3</sup>)





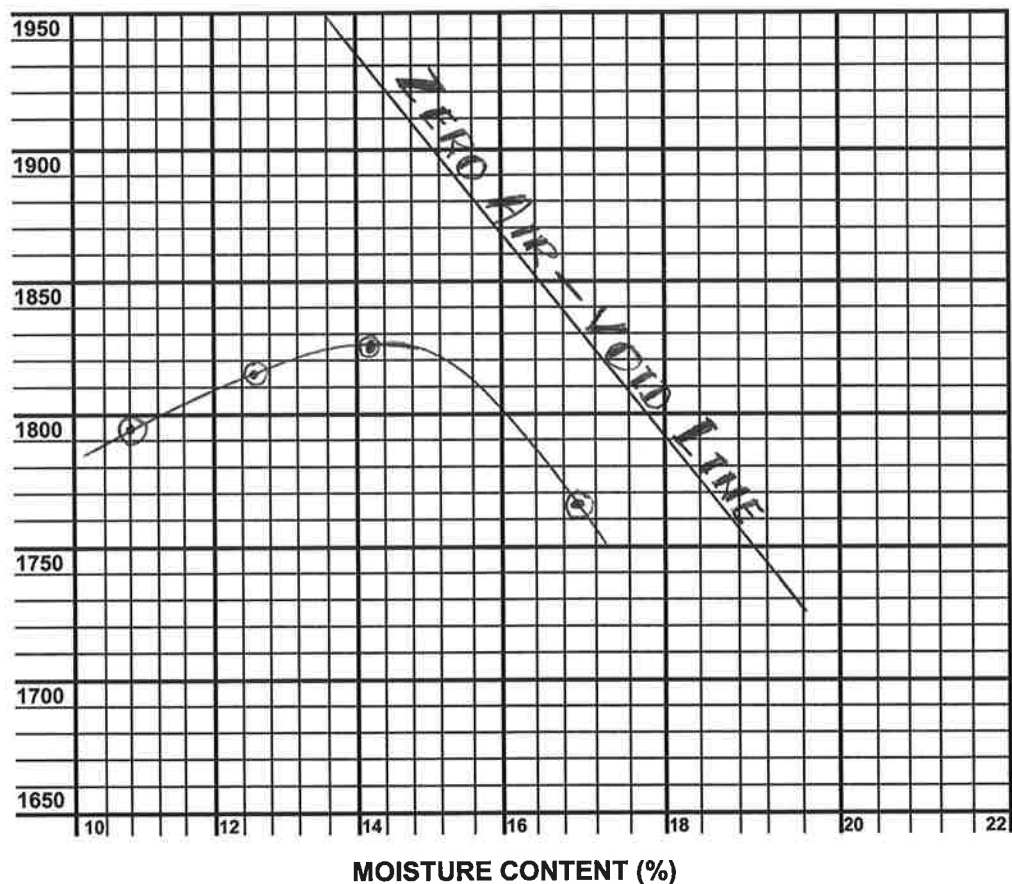
SHAHEEN & PEAKER LIMITED

PLASTICITY CHART  
EMBANKMENT FILL: Clayey Silt

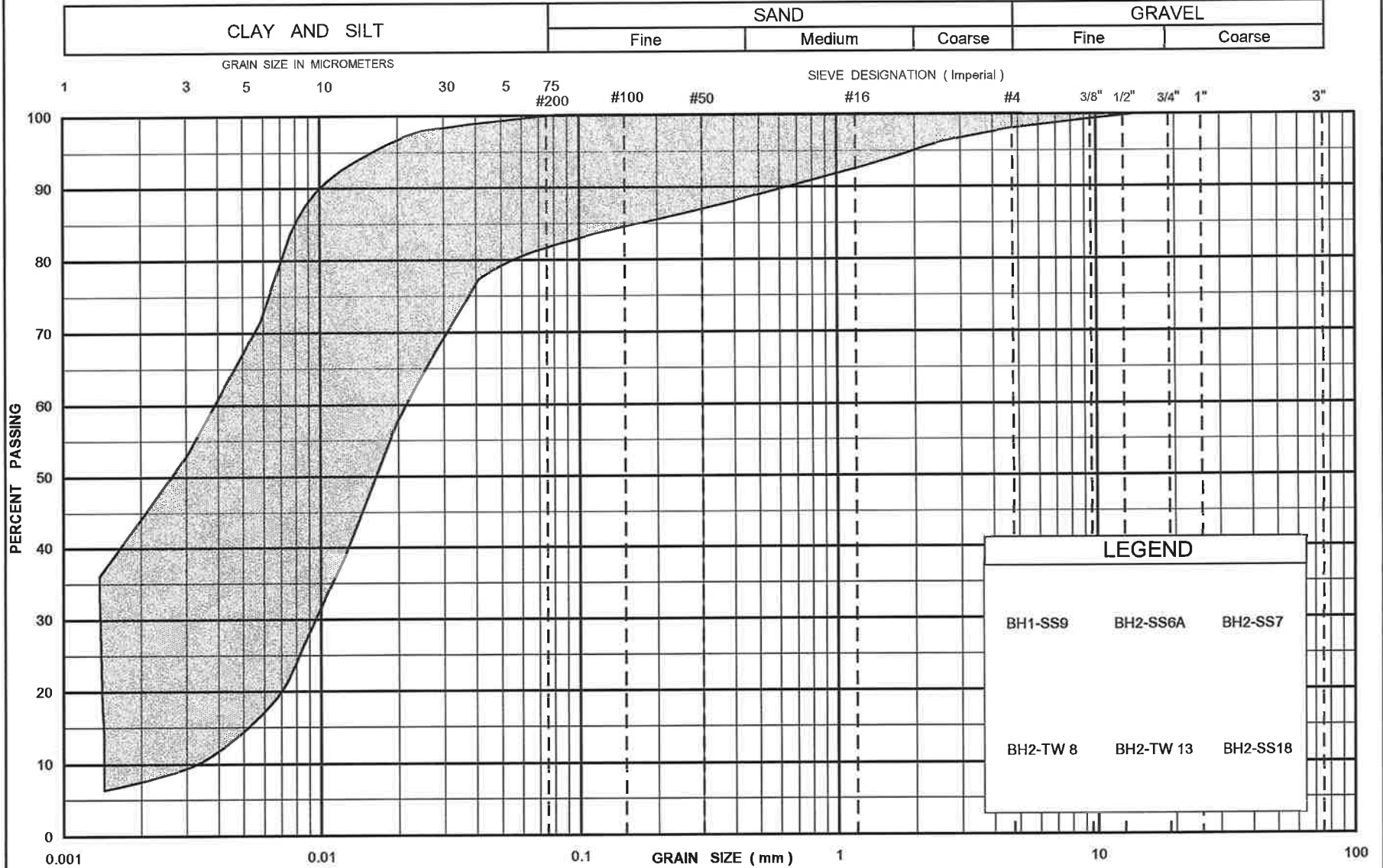
FIGURE No. B-7  
REF. No. SPT 1120  
W.P. 605-00-01

**SHAHEEN & PEAKER LIMITED****Consulting Geo-Environmental and Construction Materials Engineers****PROCTOR TEST RESULT**

Project Name:	Highway 6 (New) and Butter Road	
Project No. :	SPT1120	
Material Supplier:	N / A	
Sample Location:	BH – 4 ( 5 – Feet )	
Sampled By :	Wolfe Lang	
Date Sampled :	February 20, 2004	
Laboratory No:	3904	
Proctor Method:	Standard	Method A
Sample Description:	Clayey silt to silty clay	
Maximum Dry Density :	1825 (kg/m <sup>3</sup> )	
Optimum Moisture Content:	14.8 %	
Natural Moisture Content :	18.1 %	

DENSITY (kg/m<sup>3</sup>)

# UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER LIMITED

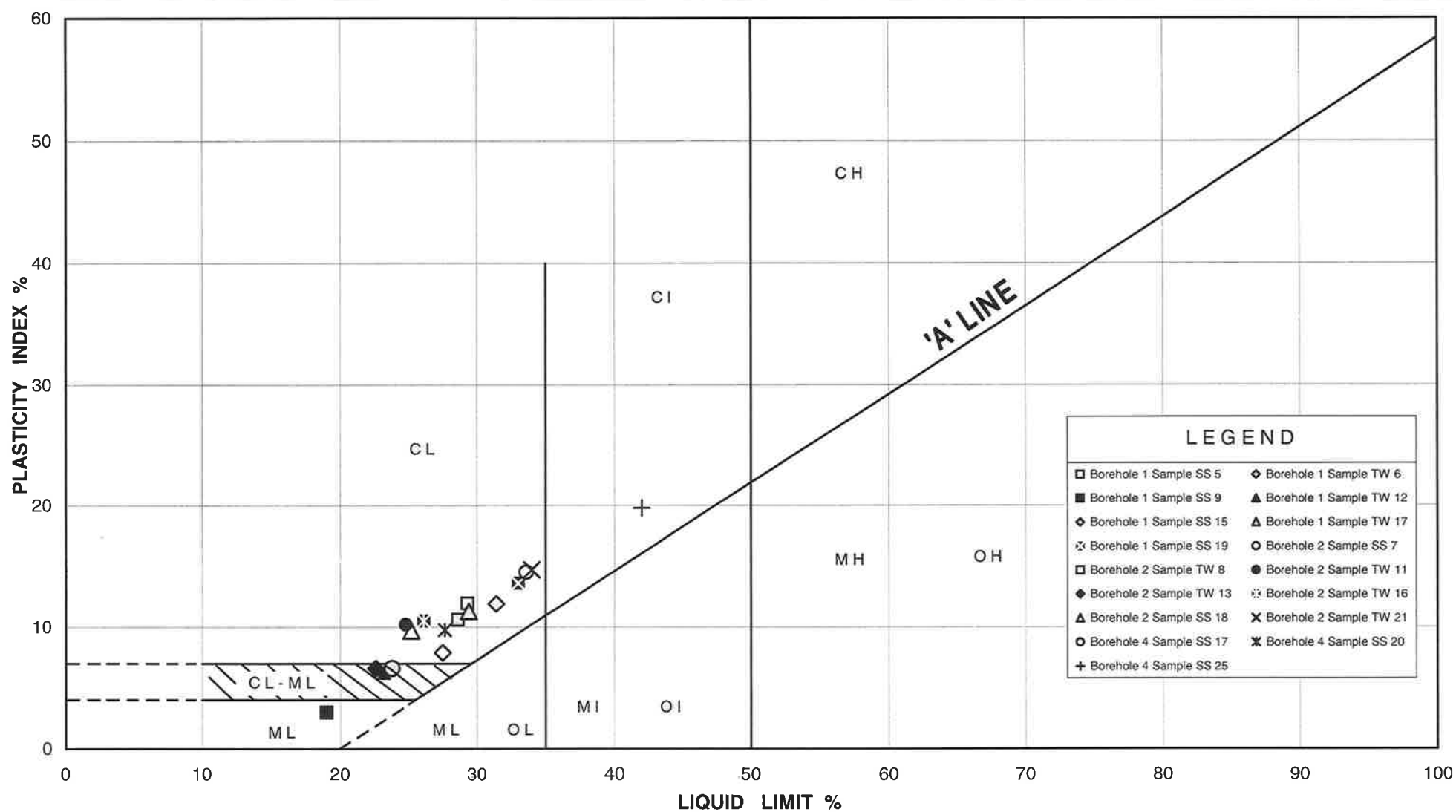
**GRAIN SIZE DISTRIBUTION**  
CLAYEY SILT

FIGURE No. B-9

REF. No. SPT 1120

W.P. 605-00-01





SHAHEEN & PEAKER LIMITED

PLASTICITY CHART  
CLAYEY SILT

FIGURE No. B-10

REF. No. SPT 1120

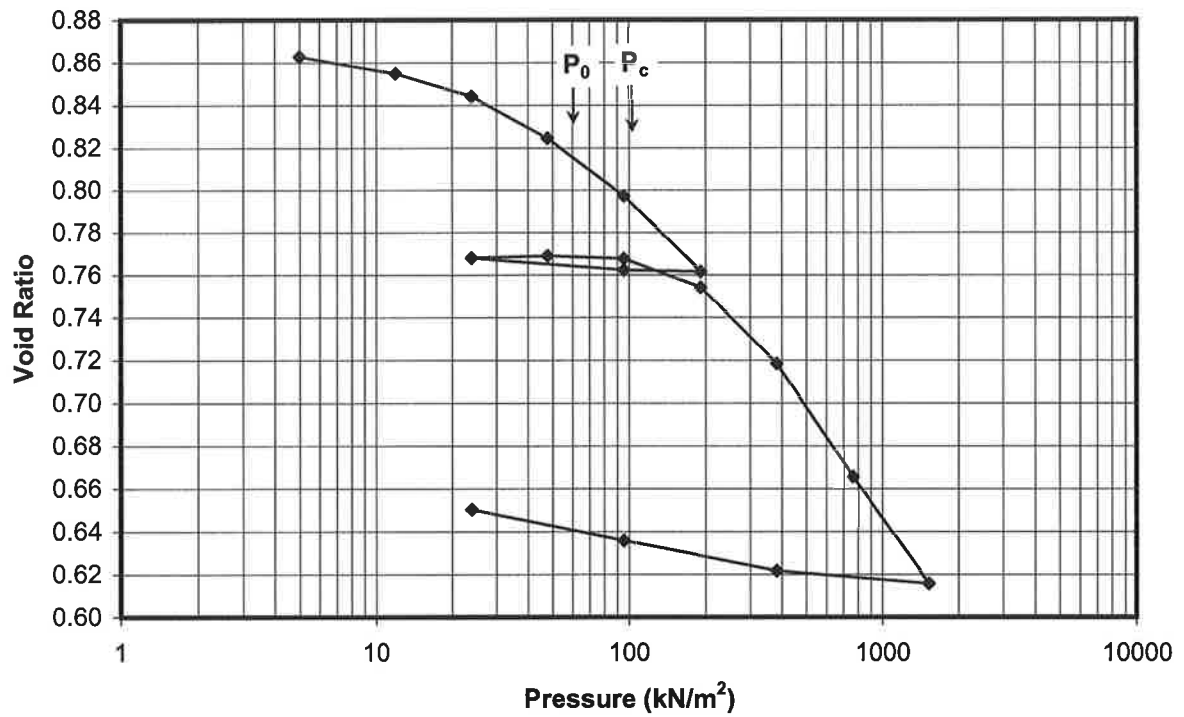
W.P. 605-00-01

Borehole BH1

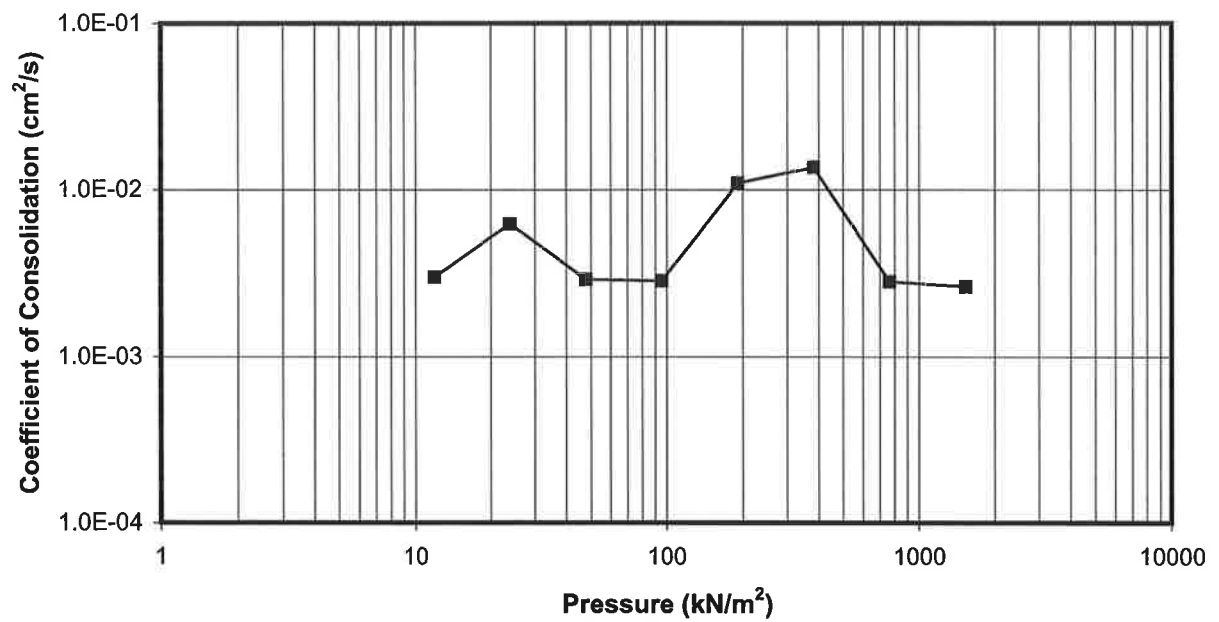
TW 6 Depth 3.2 m

FIGURE B-11

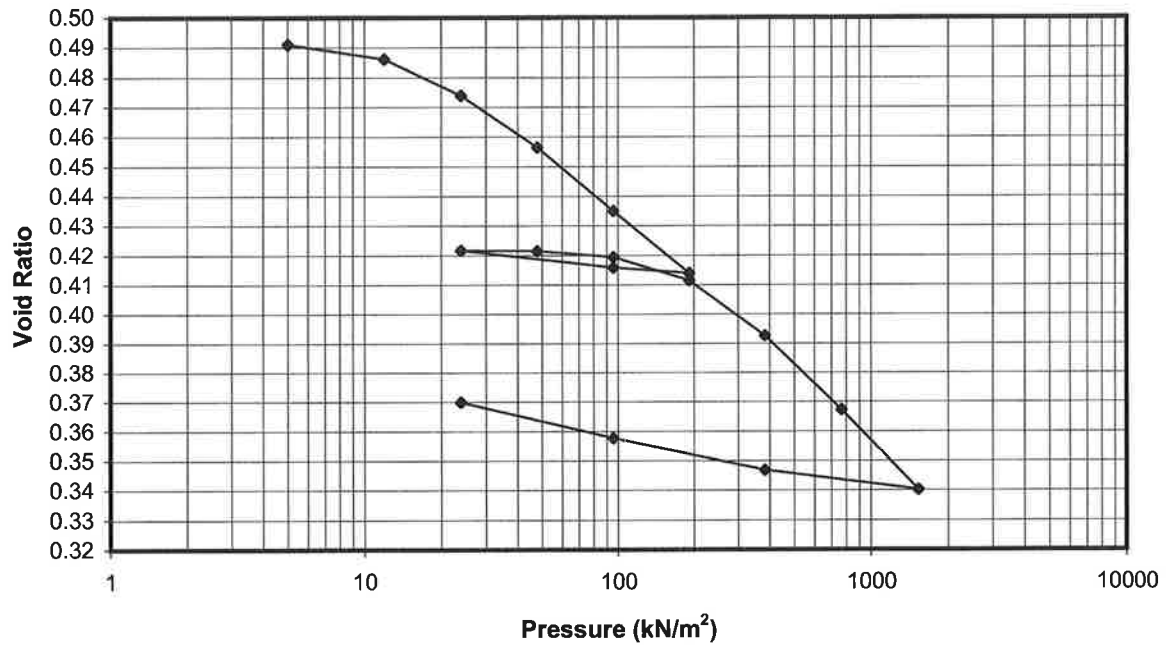
### Void Ratio versus Pressure



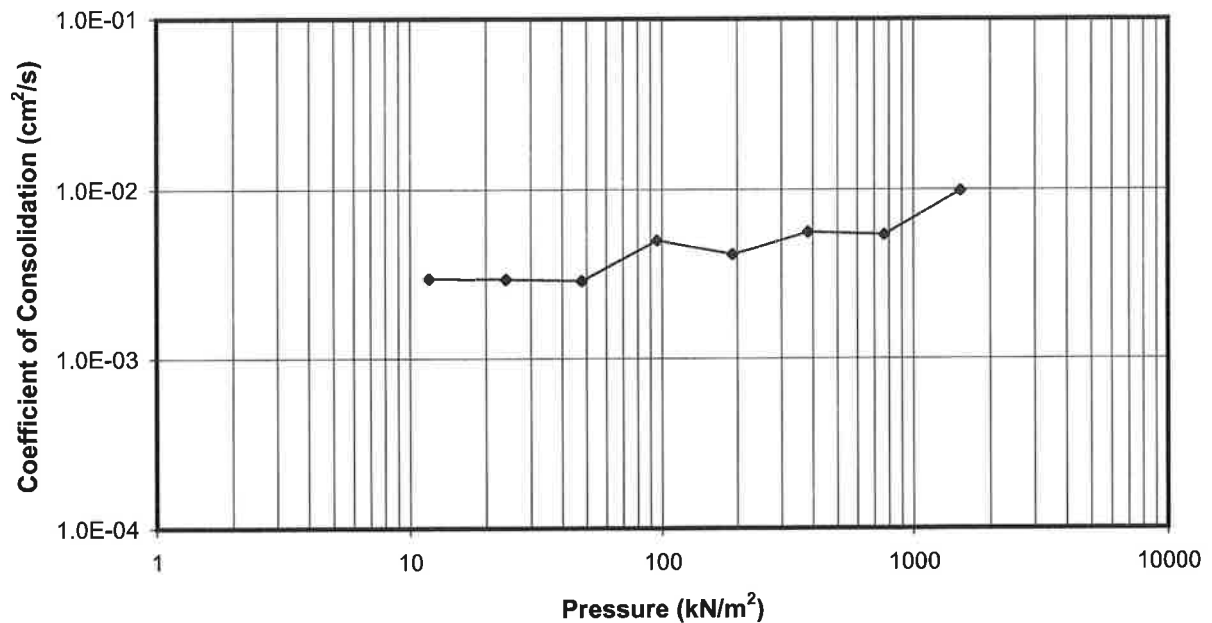
### Coefficient of Consolidation vs. Pressure



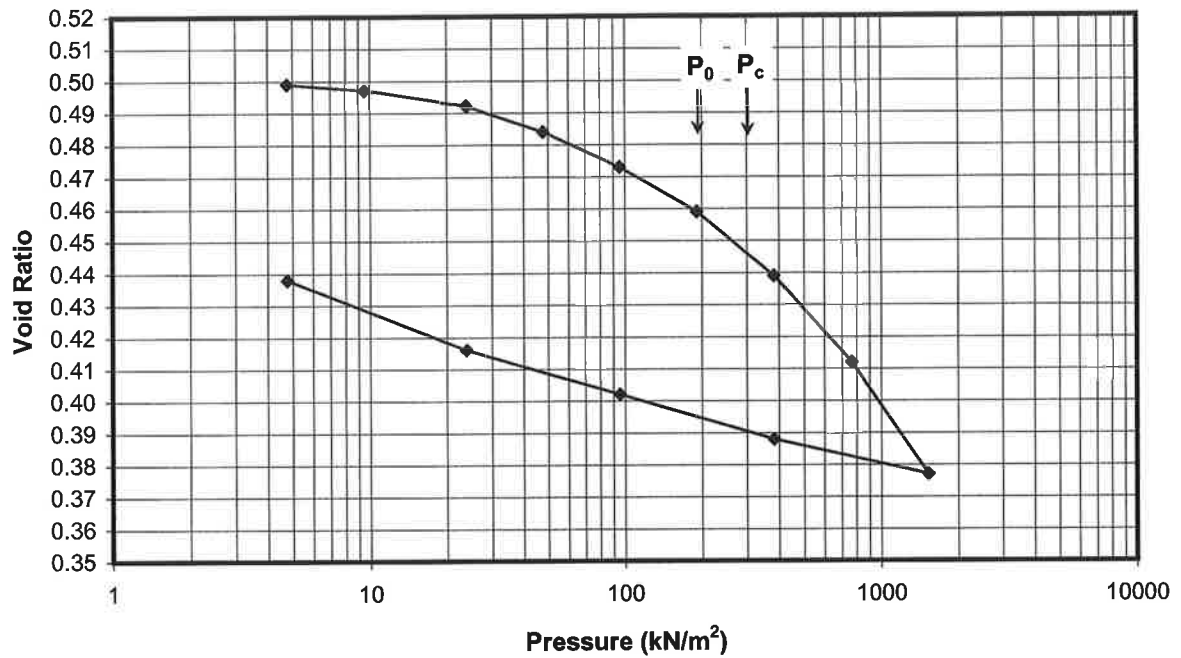
**Void Ratio versus Pressure**



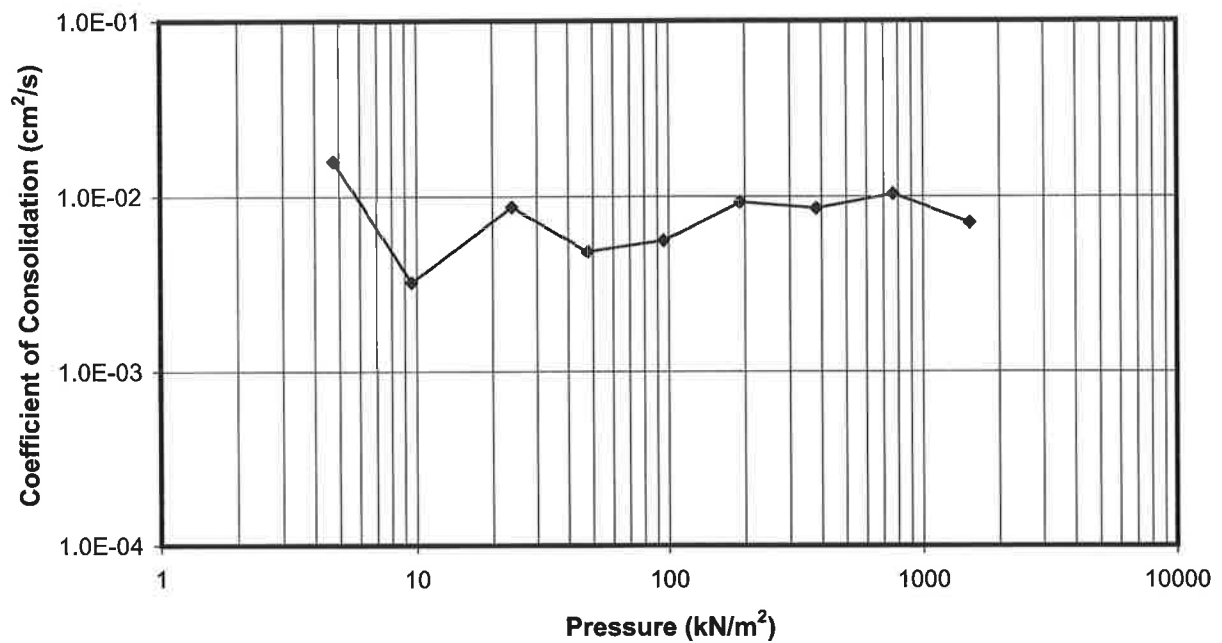
**Coefficient of Consolidation v.s. Pressure**



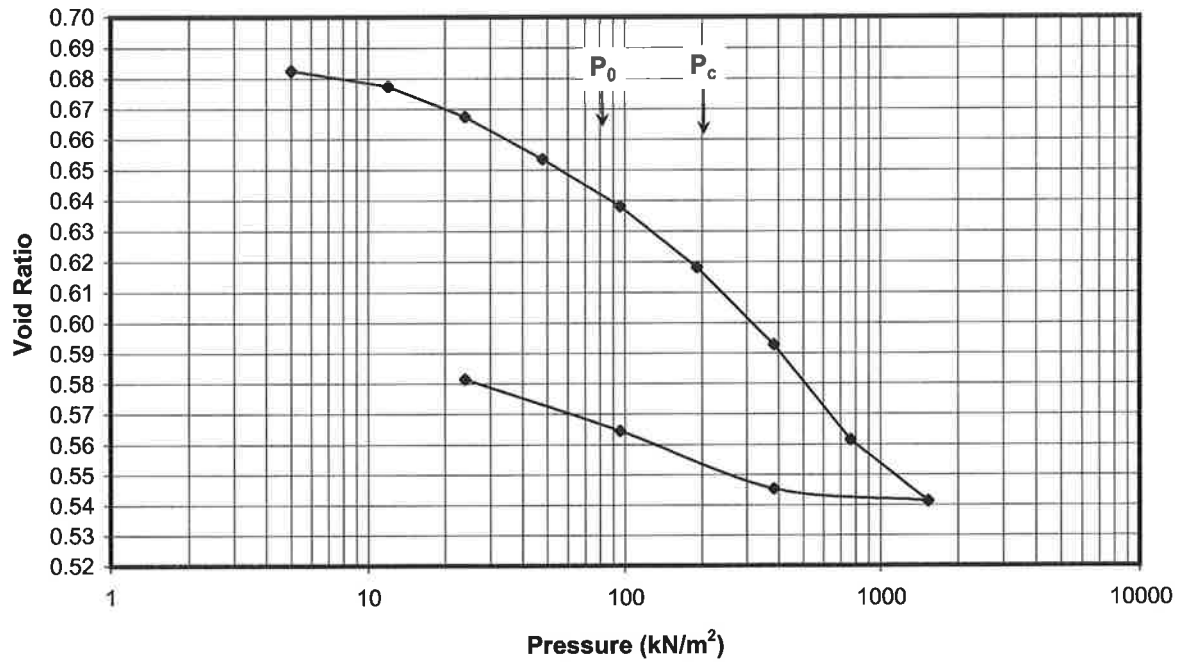
### Void Ratio versus Pressure



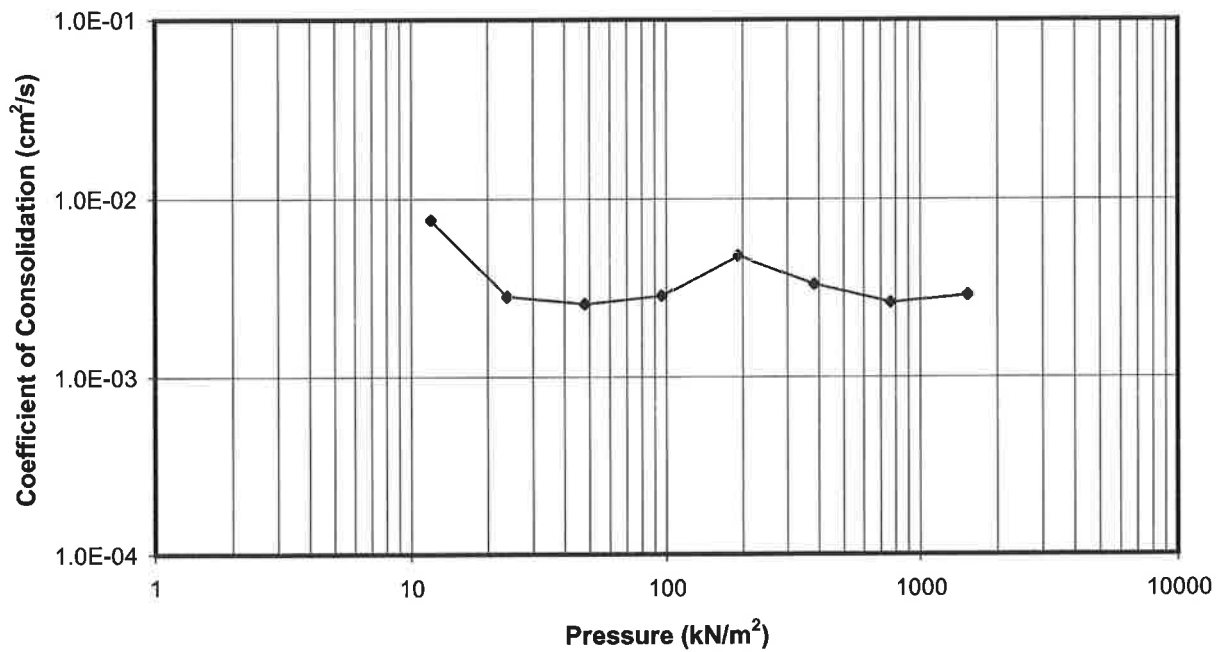
### Coefficient of Consolidation vs. Pressure



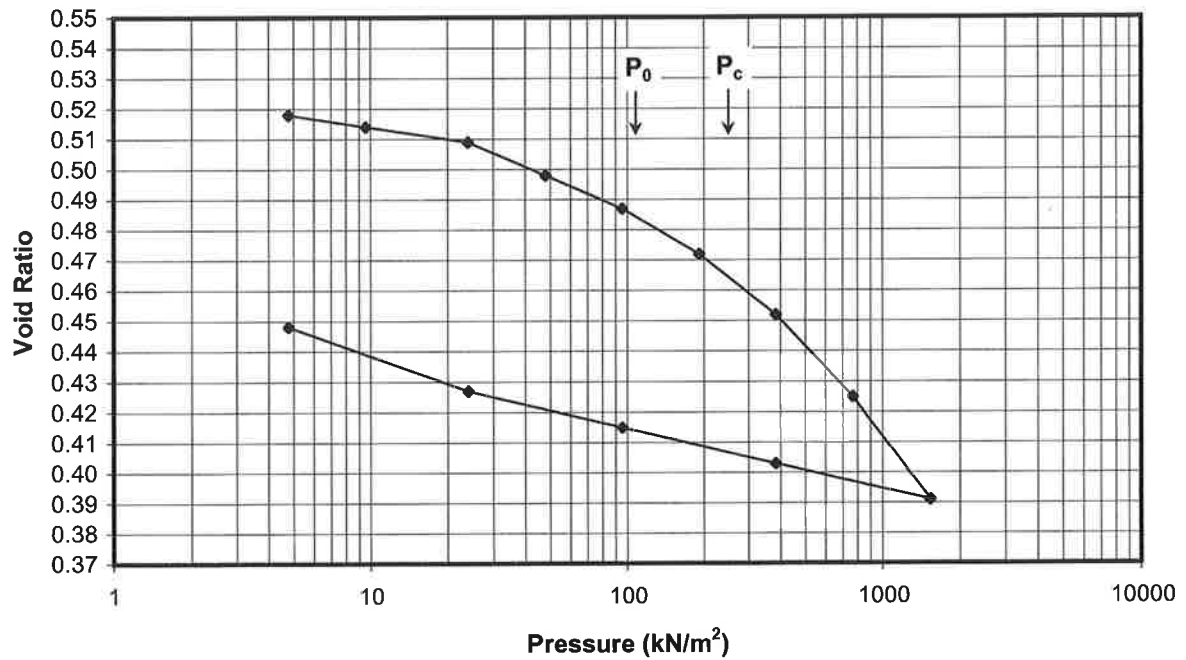
**Void Ratio versus Pressure**



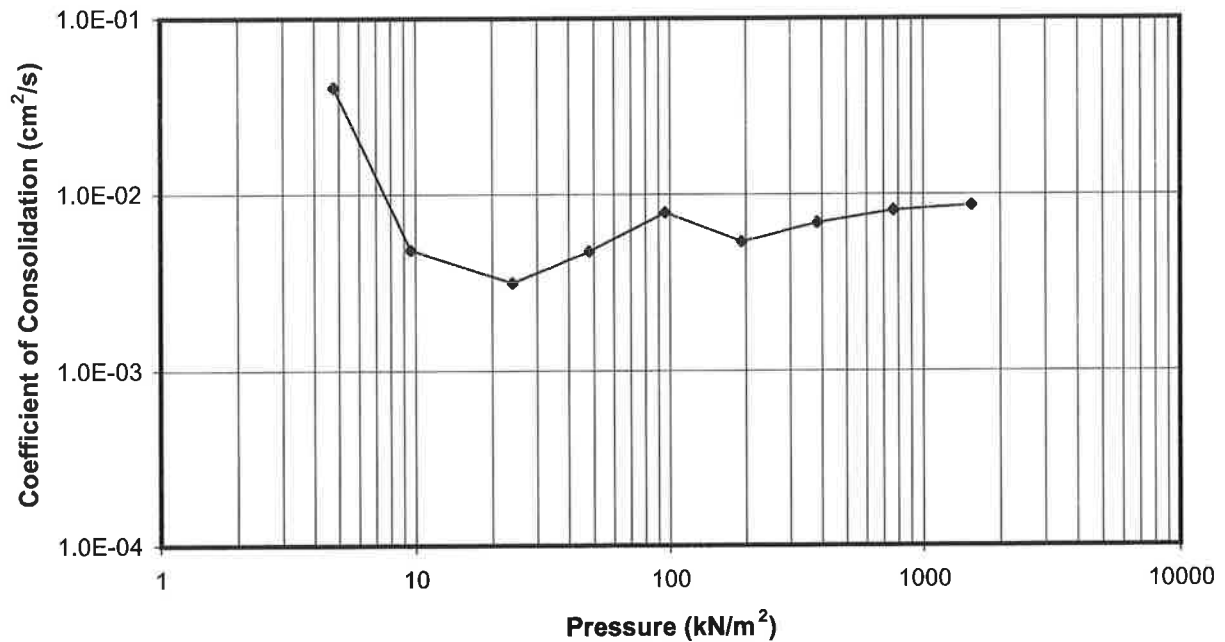
**Coefficient of Consolidation v.s. Pressure**



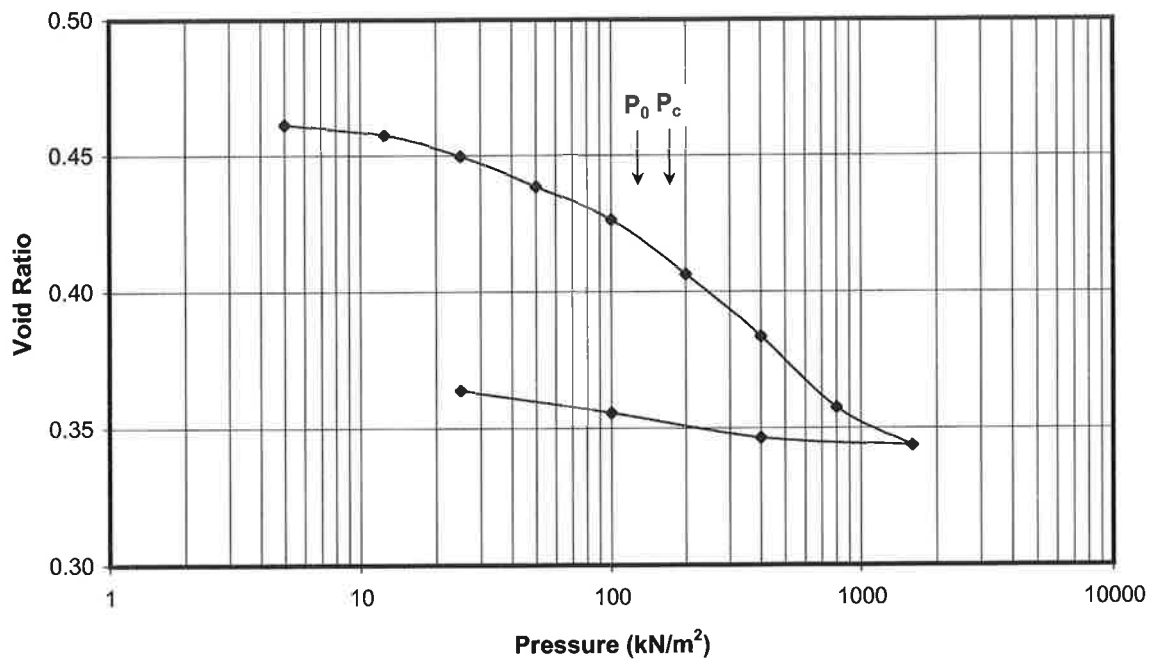
### Void Ratio versus Pressure



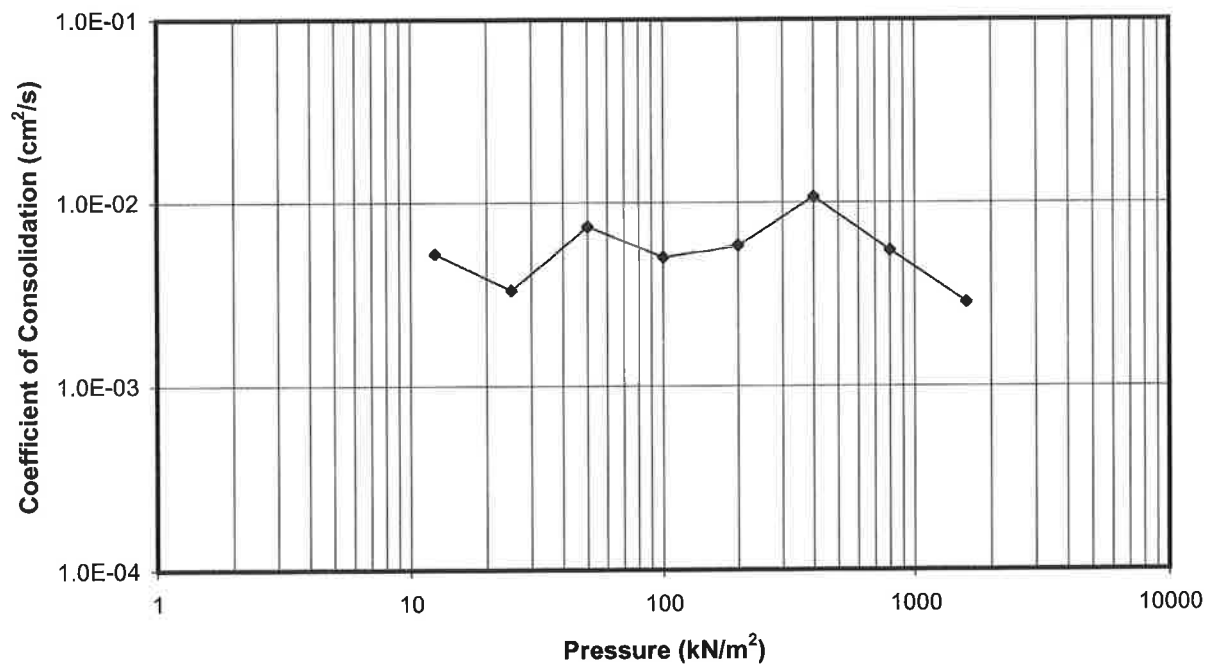
### Coefficient of Consolidation v.s. Pressure



**Void Ratio versus Pressure**



**Coefficient of Consolidation vs. Pressure**



## Appendix C

### Measured Natural Moisture Content and Standard Penetration Test Results of Advance Fill

### Measured Undrained Shear Strength Results of Clayey Silt Overburden



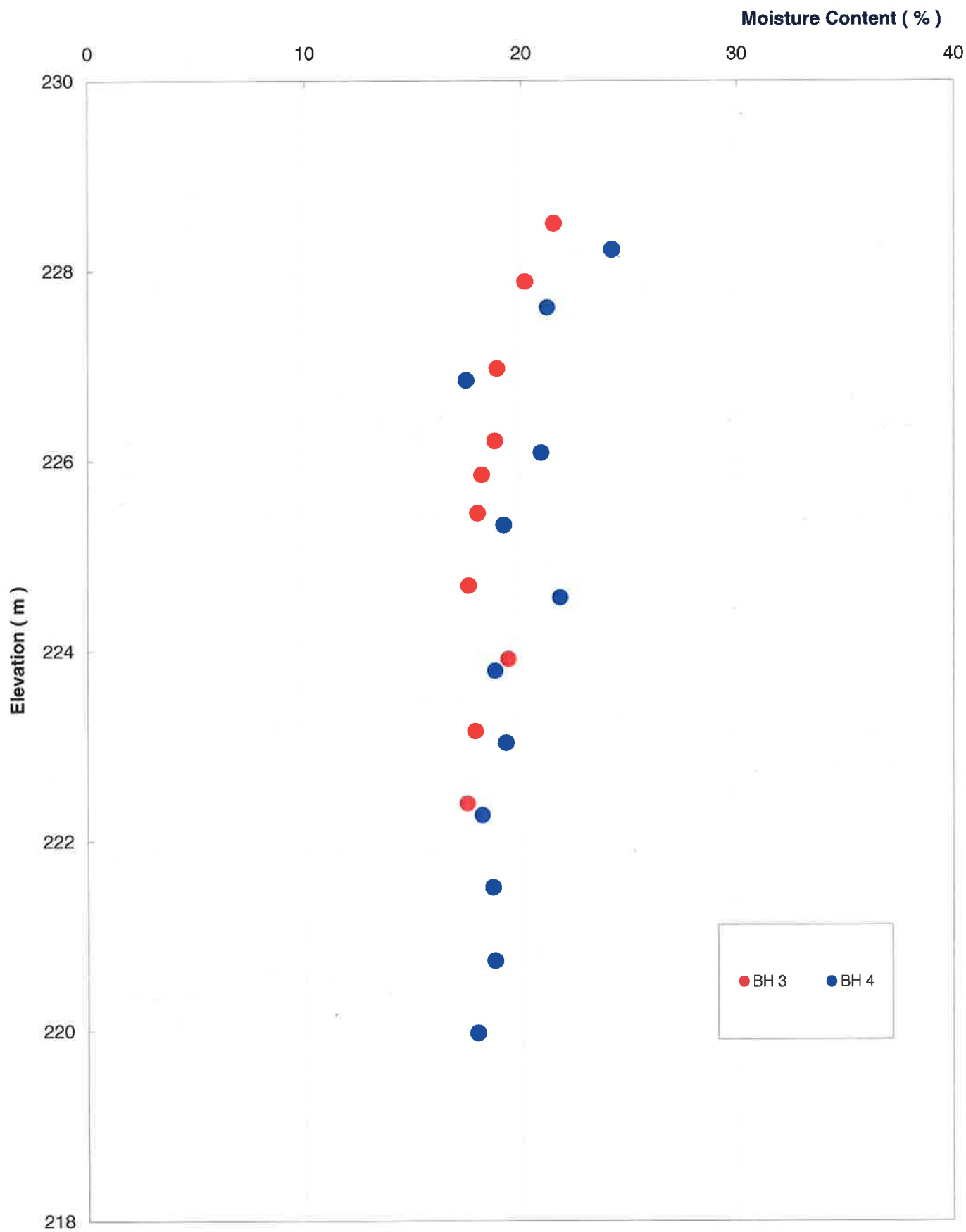


FIGURE C-1: Measured Moisture Contents of Advance Clayey Silt Fill versus Elevation

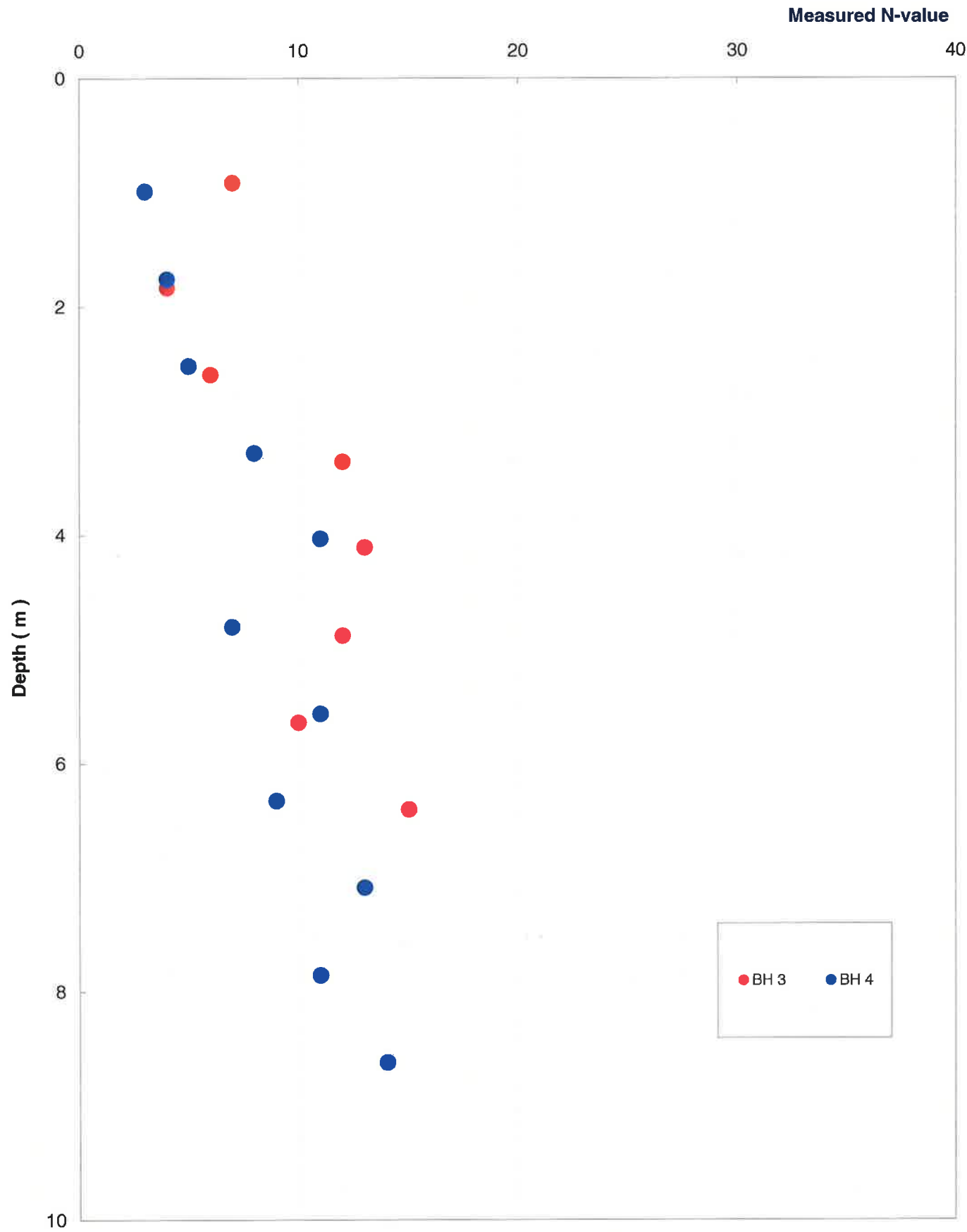


FIGURE C-2: Variation of Measured N-values with Depth in the Advance Embankment Fill

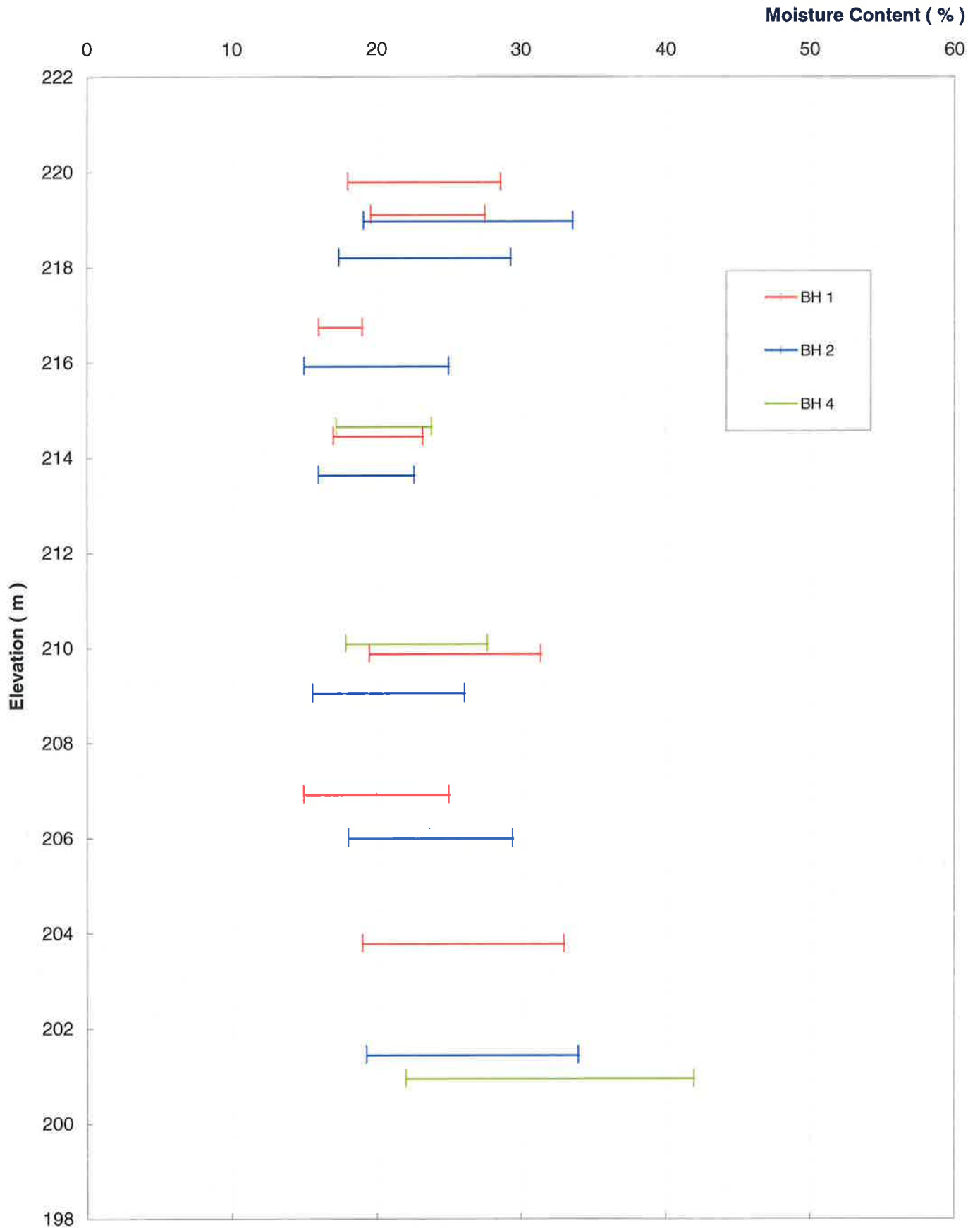


FIGURE C-3: Variation of Atterberg Limits with Elevation in the Clayey Silt Deposit

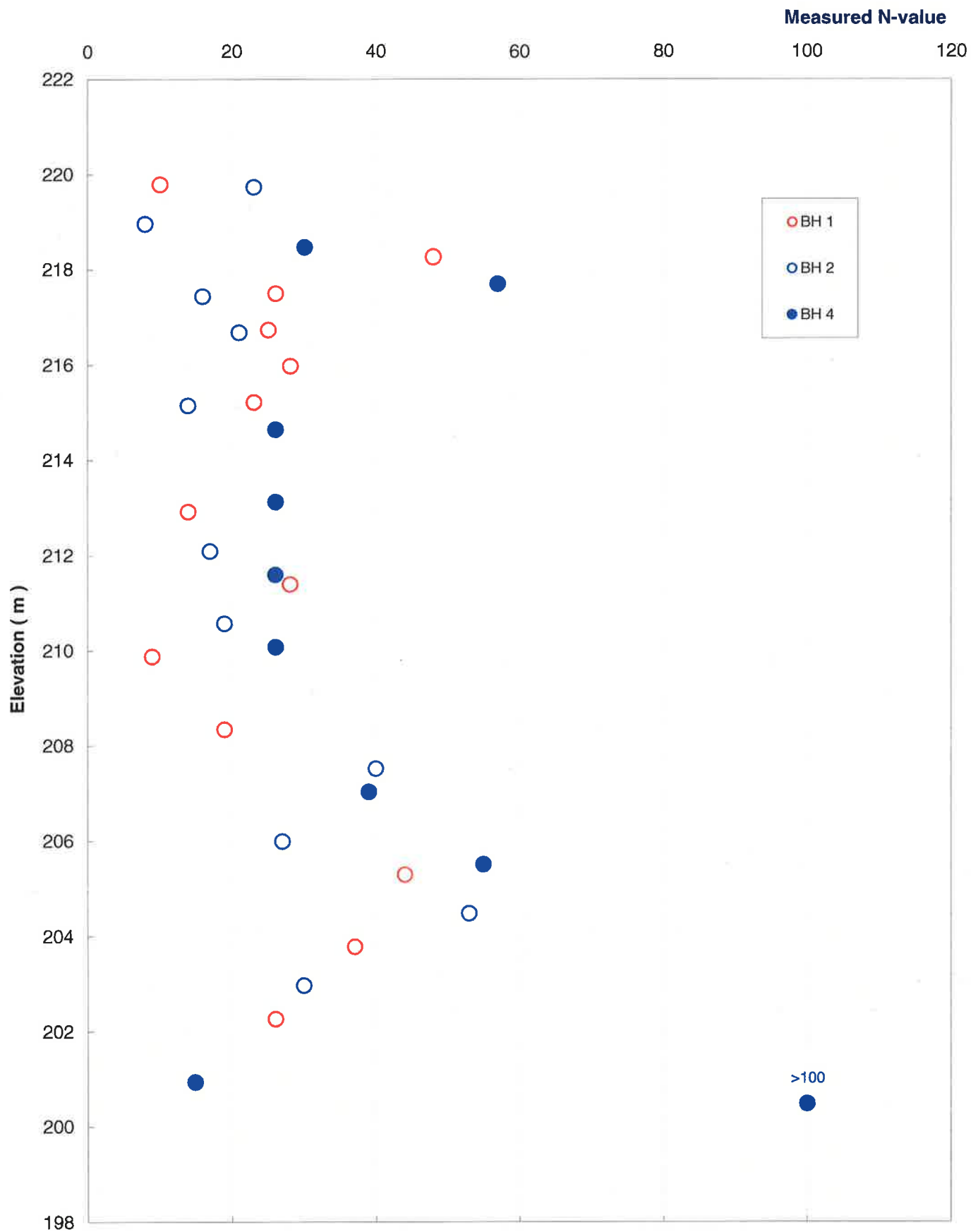


FIGURE C-4: Variation of Measured N-values with Elevation in the Clayey Silt Deposit

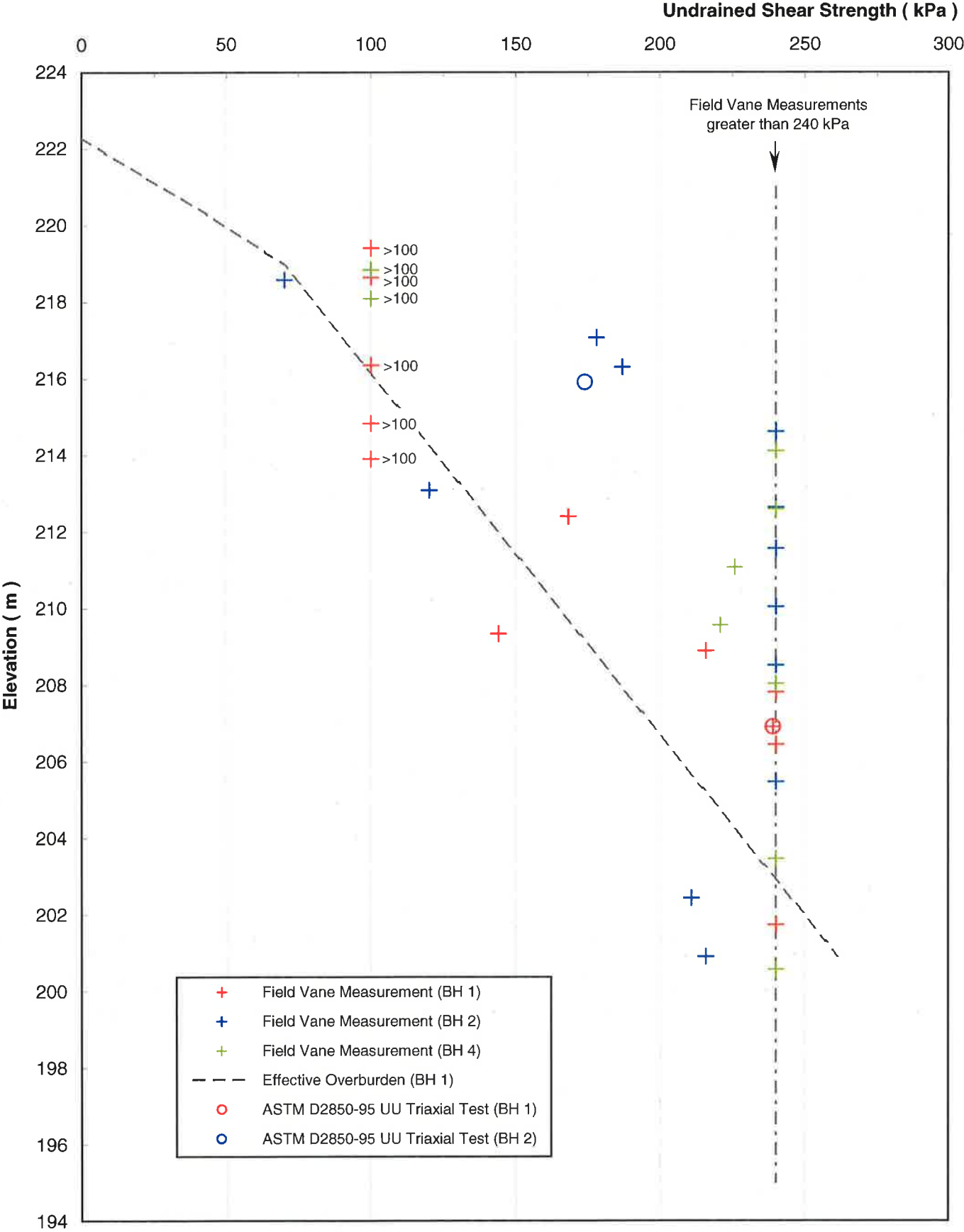


FIGURE C-5: Variation of Measured Undrained Shear Strength with Elevation in the Clayey Silt Deposit

# Appendix D

## Site Photographs



Figure D-1 Overall Site View





Figure D-2 View of West Abutment and Central Pier Locations



Figure D-3 View of East Abutment Location



# Appendix E

## Records of Boreholes (by others)

# RECORD OF BOREHOLE No 1

N 4 780 825  
E 267 333

W.P. 605-00-01 LOCATION BUTTER ROAD UNDERPASS AT HWY 6 (NEW) ORIGINATED BY M.R.  
DIST. HWY. 6 BORING DATE October 23, 2000 COMPILED BY M.R.A.  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Solid Stem Augers CHECKED BY D.W.K.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION * RESISTANCE PLOT STANDARD PENETRATION TEST *				LIQUID LIMIT — W <sub>L</sub> PLASTIC LIMIT — W <sub>p</sub> WATER CONTENT — W			UNIT WEIGHT Y	REMARKS % GR. SA. SI. CL.			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N - VALUES		20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>				
221.24	Ground Level					221.0												
0.00	FILL: crushed limestone Compact Grey		1	SS	23	220.0												
219.84																		
1.5	Fill, Clay, silty, trace of sand; decayed woody and peaty organics		2	SS	4	219.0												
219.09	Soft to firm Brown		3	SS	8	218.0												
2.15																		
3.0	Clay, silty, trace of sand; occ. layers of wet silt and sand		4	SS	20	217.0												
3.17.14	Stiff																	
4.05	Very stiff Brown		5	SS	16	216.0												
4.5																		
	Clay till, silty, trace of sand					215.0												
6.0	Very stiff Grey		6	SS	28	214.0												
214.69																		
6.55	End of Borehole																	
7.5																		
9.0																		
10.5																		
12.0																		
13.5																		
15																		
16.5																		

Date Depth to  
Water (m)  
Nov. 3 1.80  
Nov. 14 1.05

N 4 780 830  
E 267 352

# RECORD OF BOREHOLE No 2

W.P. 605-00-01 LOCATION BUTTER ROAD UNDERPASS AT HWY 6 (NEW)  
DIST. HWY. 6 BORING DATE October 20 & 23, 2000  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring

ORIGINATED BY M.R.  
COMPILED BY M.R.A.  
CHECKED BY D.W.K.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION * RESISTANCE PLOT STANDARD PENETRATION TEST *				LIQUID LIMIT — W <sub>L</sub> PLASTIC LIMIT — W <sub>P</sub> WATER CONTENT — W			UNIT WEIGHT Y KN/m <sup>3</sup>	REMARKS % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N - VALUES		20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>	
0	221.48 0.0														
	Fill, crushed limestone Compact Grey		1	SS	29	221.0									
1.5	220.08 1.40		2	SS	9	220.0									
	Clay, silty, trace of sand; occ. thin partings of silt Stiff Brown		3	SS	25	219.0									
2.50	Silt, trace of clay and sand		4	SS	9	218.0									
3.0	217.58 3.90														
	Clay till, silty, trace of sand Stiff Grey		5	SS	9	217.0									
4.5			6	SS	11	216.0									
6.0			7	SS	15	215.0									
			8	SS	11	214.0									
7.5			9	SS	10	213.0									
9.0			10	SS	8	212.0									
10.5	211.48 10.00														
	Silt, trace of sand, with layers of silty clay Compact Grey		11	SS	29	211.0									
12.0	209.94 11.50														
	Clay, silty, trace of sand Stiff to very stiff Grey														
13.5															
15															
16.5															
	Continued on Page 2 of 2														

Hole was  
extended to a  
depth of  
15.7m and  
completed 3  
days later.  
Water at  
14.30m.

W.P. 605-00-01 LOCATION BUTTER ROAD UNDERPASS AT HWY 6 (NEW)  
DIST. HWY. 6 BORING DATE October 20 & 23, 2000  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring

ORIGINATED BY M.R.  
COMPILED BY M.R.A.  
CHECKED BY D.W.K.

[illegible]

W.P. 605-00-01 LOCATION BUTTER ROAD UNDERPASS AT HWY 6 (NEW) ORIGINATED BY M.R.  
DIST. HWY. 6 BORING DATE October 19, 2000 COMPILED BY M.R.A.  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring CHECKED BY D.W.K.

[illegible]

N 4 780 844  
E 267 379

# RECORD OF BOREHOLE No 3 Cont.

W.P. 605-00-01 LOCATION BUTTER ROAD UNDERPASS AT HWY 6 (NEW)  
DIST. HWY. 6 BORING DATE October 19, 2000  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring

ORIGINATED BY M.R.  
COMPILED BY M.R.A.  
CHECKED BY D.W.K.

SOIL PROFILE		SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT STANDARD PENETRATION TEST *		LIQUID LIMIT — W <sub>L</sub> PLASTIC LIMIT — W <sub>P</sub> WATER CONTENT — W		UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS % GR. SA. SI. CL.			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		N — VALUES	20	40	60			80	100	W <sub>P</sub>
16.5	Ground Level													
	Continued from Page 1 of 2													
	Clay, silty, trace of sand, till-like													
18.0	Very stiff Grey		12	SS	25									
19.5														
200.88														
21.0	Dolostone Bedrock		13	SS	50	25mm								
22.5														
24.0														
24.10	End of Borehole													
25.5														
27.0														
28.5														
30.0														
32.5														
34.0														

Upon completion of augering, no water, no cave.

W.P. 605-00-01 LOCATION BUTTER ROAD UNDERPASS AT HWY 6 (NEW)  
DIST. HWY. 6 BORING DATE October 24 & 25, 2000  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring

ORIGINATED BY M.R.  
COMPILED BY M.R.A.  
CHECKED BY D.W.K.

[illegible]

W.P. 605-00-01 LOCATION BUTTER ROAD UNDERPASS AT HWY 6 (NEW)  
DIST. HWY. 6 BORING DATE October 24 & 25, 2000  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring

ORIGINATED BY M.R.  
COMPILED BY M.R.A.  
CHECKED BY D.W.K.

[illegible]



# RECORD OF BOREHOLE No 5

N 4 780 855  
E 267 428

W.P. 605-00-01 LOCATION BUTTER ROAD UNDERPASS AT HWY 6 (NEW)  
DIST. HWY. 6 BORING DATE October 25, 2000  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Solid Stem Augers

ORIGINATED BY M.R.  
COMPILED BY M.R.A.  
CHECKED BY D.W.K.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION * RESISTANCE PLOT STANDARD PENETRATION TEST *					LIQUID LIMIT — W <sub>L</sub> PLASTIC LIMIT — W <sub>P</sub> WATER CONTENT — W			UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS % GR. SA. SI. CL.			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N - VALUES		20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>					
222.31 0.00	Ground Level					222.0													
221.41 0.90	Pavement, 150mm asphalt over 750mm crushed limestone		1	SS	12	221.0													
220.91 1.40	Silt, trace of clay and fine sand		2	SS	9	220.0													
220.21 2.10	Loose Brown		3	SS	23	220.0													
3.0	Clay, silty, trace of sand		4	SS	44	219.0													
	Stiff Brown																		
4.5	Silt, trace of sand and clay					218.0													
	Compact to dense Brown		5	SS	30	217.0													
216.70 5.60	Clay till, silty, trace of sand		6	SS	13	216.0													
215.63 6.70	Stiff Brown																		
7.5	End of Borehole																		
9.0																			
10.5																			
12.0																			
13.5																			
15																			
16.5																			

Upon  
completion of  
augering,  
water at  
5.80m, no  
cave.

## 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 64.5kg, 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 N.

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

**JOINT AND BEDDING:**

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

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

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

### STRESS AND STRAIN

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

### MECHANICAL PROPERTIES OF SOIL

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

### PHYSICAL PROPERTIES OF SOIL

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

**FOUNDATION DESIGN REPORT  
HIGHWAY 6 (NEW) AND BUTTER ROAD  
HAMILTON, ONTARIO  
W.P. 605-00-01**

**Prepared For:**

**MINISTRY OF TRANSPORTATION – CENTRAL REGION**

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**FOUNDATION DESIGN REPORT  
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**5. DISCUSSION & RECOMMENDATIONS**

The proposed Highway 6 underpass structure at Butter Road in Hamilton will be a two-span bridge, each span measuring approximately 31 m in length. The bridge will be approximately 12 m wide.

The proposed elevation for the top of the bridge is approximately 231.1 m. The original grade at the site was approximately 221 m and therefore, the height of the approach fills will be about 10 m in relation to the original grades. The ground surface elevations of boreholes drilled by others in October 2000 ranged from 221.2 to 222.3 m, since it appears that most of these boreholes were drilled from the previous Butter Road embankment. During our investigation in early 2004, Butter Road had already been re-aligned (i.e. temporary detour) for the construction of the proposed new bridge and advance fills had been placed for pre-loading the site. We understand that the advance fills were placed in or about November 2003 to about El. 228 (i.e. about 3 m below the final grades). The advance fills were placed somewhat short of the abutment locations (see Photographs presented in Appendix D) to facilitate the driving of the piles at the proposed abutment locations, without having to remove these fills. Therefore, at the abutment support locations, little or no preloading was effected.

During our investigation, the ground elevation at borehole locations were 222.3 m and 223.0 m of the location of Boreholes 1 and 2 (i.e. about 1.3 and 2.0 m above the original ground of 221.0± m), which were put down at or near east and west abutment locations, respectively, while at Boreholes 3 and 4, drilled from the top of the advance fills about 25 m beyond the abutment locations, it was about 228.7 m.

The bridge will be supported on steel H-piles (HP310 x 110), which were driven shortly after our investigation in February 2004.

The boreholes drilled for this investigation have shown, below some fill and surficial silt deposits, the presence of an extensive clayey silt deposit starting below about El. 220-219 m and extending to the surface of bedrock at about 200.5 m. The clayey silt has some silty clay zones and thin silt interbeds. Owing to the presence of occasional embedded fine gravel and coarse sand particles (grits), at most locations, the deposit resembles a glacial till. Standard Penetration tests performed in the deposit gave a wide range of values, ranging from 8 to 53 blows for 0.3 m penetration and field vane tests yielded undrained in-

situ shear strengths ranging from 70 to in excess of 240 kPa. Based on these results, the consistency of the deposit is described as firm to hard but generally very stiff.

The clayey silt till is underlain by a dolostone bedrock which was encountered at El. 200.7 to 200.4 m. The groundwater table at the site was inferred to be at El. 220 m, but could be subject to seasonal fluctuations and in response to major weather events.

## 5.1 SETTLEMENTS

Due to the stresses induced on the foundation soils, settlements can be anticipated primarily due to the consolidation of the extensive clayey silt deposit underlying the site, as well as due to the settlement of the embankment fills under their own weight.

To reduce the anticipated foundation settlements due to the weight of the approach fills, MTO has decided to use light-weight fill (i.e. pelletized slag) immediately adjacent to the bridge abutment. The proposed configuration by MTO for this is shown in Drawing No. 1 in Appendix G, except that the use of ultra-light weight fill as shown on the drawing has been changed to a light-weight fill with a compacted unit weight of 14.2 kN/m<sup>3</sup>.

For our settlement analyses, we have chosen three locations. Point No. 1 is located about 30 m away from the east and west ends of the bridge, where only ordinary fill will be used to build the embankments (i.e. no light-weight fill). Point No. 2 is at the edge of the approach slabs. Here the fill consists of ordinary earth fill near the bottom but light-weight fill will be used in the upper zones, as shown in Drawing No. 1 in Appendix G. At abutment support locations (Point No. 3), where pile foundations have already been installed, the fill will consist of light-weight fill in order to minimize the impending settlements and to possibly reduce the magnitude of down-drag loads on the piles.

Based on these above conditions, the following paragraphs summarize our settlement analysis.

### 5.1.1 FOUNDATION SETTLEMENTS

(a) Point No. 1 – Predicted stress distribution due to 10 m high embankment fills at a distance of 30 m (and beyond) from the bridge abutments is shown in Figure H-1 in Appendix H.

Our settlement analysis based on measured engineering characteristics of foundation soils as well as a visual and tactile examination of the soil samples recovered from foundation soils indicate that a total settlement of about 110 mm can be expected under the weight of the embankment fills. As the site has already been preloaded with an about 7 m high advance fill placement, which has been in place for about seven months, it is believed that about 30 mm of this settlement has already taken place and assuming that the embankment

is constructed to its full height shortly, about 80 mm of further settlements can be expected. About one-third of this 80 mm settlement can be expected to take place within the first six months of the placement at the embankment to its full height. About 90% of the remainder of the settlements (i.e. about 50 mm) can be expected to take place within the following five years with the remaining 5 mm thereafter.

(b) Point No. 2 – The anticipated foundation settlements at the edge of the approach slabs, (as shown in Drawing No. 1 in Appendix G) are of the order of 90 mm, minus the settlements that have taken to date, which are calculated to be of the order of 30 mm. Therefore a residual settlement of 60 mm can be expected at this location. About one-third of the settlement (i.e. about 20 mm) can be expected to take place within six months following the construction of the road embankment to its full height and about 90% of the remainder within the next five years.

(c) Point No. 3 – The predicted stress distribution at the foundation location due to embankment fill is shown in Figure H-1, in Appendix H. At this location, the preloading due to advance fills was minimal. At the east abutment location, there was about 1 m of fill due to previous Butter Road embankment and at the west abutment location between about 1 and 2 m of fill had been placed on top of this bringing the total height of fill over and above O.G. (original ground) to 2 to 3 m. Here light-weight fill will be used and based on these, the calculated settlements range from 45 to 50 mm at the west abutment location to 50 to 60 mm at the east abutment location. Again, about one-third of this (i.e. 15 to 20 mm) can be expected to take place within six months of the placement of the fill and about 90% of the remainder of the settlement within the following five years.

#### 5.1.2 SETTLEMENT OF EMBANKMENT FILLS UNDER THEIR OWN WEIGHT

The approach fills can be expected to settle under their own weight.

(a) Advance Fills – As was discussed before in or about November 3, 2003, advance fills were placed to about Elev. 228 m or about 7 m above O.G..

The results of field and laboratory testing carried out on advance fills are given on the Record of Boreholes 3 and 4 in Appendix A and in Figures B-1 through B-8 in Appendix B.

Two Standard Proctor compaction tests carried out in the laboratory on samples from the fill (see Figures B-5 and B-8, Appendix B) gave a Standard Proctor Maximum Dry Density (SPMDD) of about  $18.0 \text{ kN/m}^3$  with an optimum moisture content of about 14 to 15%. The measured natural moisture contents of samples recovered from the fill are generally of the order of 18% near the bottom and about 21% near the top, as shown in Figure C-1 in Appendix C. These indicate that the moisture content of the material is considerably wet of the optimum.



Standard Penetration Resistances (N-values) of the fill obtained by Standard Penetration tests range from 3 to 15 blows/0.3 m. A plot of the measured N-values versus depth is given in Figure C-2 in Appendix C. These indicate that within the upper 3 m of the advance fill, the measured N-values range from 3 to 7 blows for 0.3 m penetration, with values ranging from 7 to 15 blows for 0.3 m penetration below this depth. It is of interest to note that the measured natural moisture contents are relatively higher within the upper 2 to 3 m of the fill where lower N-values were recorded.

These results lead us to believe that the fill has not received adequate compaction, particularly in the upper zones. This is believed to be primarily due to the following reasons:

- \* The fill was placed and compacted at moisture contents wet of optimum.
- \* The fill is not a good material for constructing the embankment. This type of soil behaves like a silt and is a dilatant material. Such soils generate high pore water pressures during compaction and unless sufficient time is allowed for the dissipation of pore water pressures between lifts, the ensuing pumping action or bulking will prevent achieving good compaction. What is worse, the underlying previously compacted layer is frequently disturbed and dilates while the upper layer is compacted, especially if vibration is introduced. One redeeming aspect is that soils of the lowest layers appear to be placed on or close to underlying granular road fill from the previous Butter Road and wherever this happened, a better degree of compaction probably took place due to firm bottom and downward drainage for the dissipation of pore water pressures.

It is, therefore, doubtful that a reasonable degree of compaction was achieved when placing the fill, especially in the upper zones. In addition, the degree of compaction was probably reduced when placing each subsequent lift due to pumping action and the generated pore water pressures.

For these reasons, ideally the top 6 m or so of this fill should be removed and replaced but since this is probably very costly, as a minimum the upper 2.5 to 3.0 m of the existing fill should be removed. The exposed subgrade should be inspected and where necessary very wet or otherwise unsuitable soils, if observed, should also be removed or otherwise stabilized. It should then be compacted from the surface to at least 95% of the SPMDD of the material. At this stage, the placement of a granular fill layer can be considered to provide a suitable base for the placement of compaction of the subsequent layers, if conditions dictate this operation.

As mentioned before, this fill is not a good material for embankment construction (and also for the surficial stability of the embankment, as will be discussed in the next section of this report), but if it has to be re-used, it should be placed at moisture contents at or within  $\pm 2\%$

of the optimum and should be uniformly compacted with a suitable compactor to not less than 95% SPMD. Sufficient time should be provided between the lifts to allow for the dissipation of excess pore pressures. If at all possible, the construction should be performed during the relatively dry summer months. Although this is not included in MTO practice (OPSS 501), it is always a good practice to increase the degree of compaction to 98% within the upper 0.5 m of the final road subgrade level.

Assuming these are carried out or a better quality of fill is used, with proper compaction, as per MTO procedures, the settlements of the embankment under its own weight can be expected to range from 50 to 65 mm. About 40% of these settlements (i.e. about 20-25 mm) can be expected to take place within three months after the construction of the embankment reaches its final grade (i.e. top of asphalt) with the remainder within the next 12 to 15 months, assuming that same or similar materials as the advance fills are used.

These settlements will be in addition to the predicted settlements due to the consolidation of the clayey silt deposit underlying the site. However, if the subgrade is properly prepared (i.e. properly compacted, properly cambered to promote rapid drainage to the sides, the subgrade is not disturbed during construction by heavy vehicle traffic, etc), such settlements should not present major problems for the performance of the flexible pavement structure, since the settlements can be expected to be rather uniform. Allowance can be made, if necessary, for a slightly wider road platform to allow for such settlements (i.e. foundation settlements plus settlements due to settlement of the embankment under its own weight).

## 5.2 EMBANKMENT STABILITY

Global stability of the proposed embankments, as well as the possibility of slope failures within the embankment fill, were analyzed by means of limit equilibrium methods, utilizing the computer program Slope/W. In our analyses, Bishop's Simplified method was used, for both short-term (undrained) and long-term (drained) analyses calculations.

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

Table 5.2(a)  
Soil Parameters Utilized for Slope Stability Analyses

Soil Type	Short-Term Analyses			Long-Term Analyses		
	$\phi$ (degrees)	c (kPa)	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (degrees)	c' (kPa)	$\gamma'$ (kN/m <sup>3</sup> )
Embankment fill (clayey silt)	0	35	20	26	3	20
Clayey silt (Natural soil)	0	70-200	19.5-21	26-28	3-4	19.5-21

### 5.2.1 GLOBAL STABILITY

Based on the selected parameters, our analyses indicate that there is adequate factor of safety against a deep seated (i.e. foundation) failure both in the short and long terms. Calculation results obtained for short-term conditions by forcing the slip circles below the embankment itself indicate safety factors in excess of 1.8 which are considerably greater than normally acceptable values of between 1.3 and 1.4. Figure I-1 in Appendix I presents typical results.

Figure I-2 in Appendix I shows typical results for long-term analysis, which indicate a minimum safety factor of the order of 1.5. This too is in excess of normally used minimum safety factors quoted before, and is, therefore, considered acceptable.

In conclusion, we do not perceive a problem with global stability of the proposed up to 10 m high embankments

### 5.2.2 EMBANKMENT FILL STABILITY

The parameters to be used for embankment fills depend on the type of soil as well as other factors such as degree of compaction received during construction. Boreholes 3 and 4 show that clayey silt soils were used for the construction of the advance fills. These soils, which are readily available in the general area, will probably be continued to be used to build the remainder of the embankments. Normally, most types of soils which meet MTO criteria for embankment fill construction usage would be stable at 2H:1V slopes and routinely a mid-height 2 m wide berm is introduced for improved surficial erosion management. As was mentioned before, the clayey silt soils available in the area which were used for advance fills, behave like a somewhat cohesive silt rather than a truly clayey soil. For our short-term analysis, we assumed a low undrained shear strength (c-value) of 35 kPa in view of the fact that the soils appeared to have low strengths as measured by natural moisture contents of the material were considerably in excess (i.e. wet) of the optimum. In this case, the obtained safety factor, as determined by limit equilibrium methods and slip circle analysis, is of the order of 1.5 and is therefore acceptable (see Figure I-3, Appendix I). If a c-value of 50 kPa is assumed, which probably is more realistic, the minimum factor safety increases to in excess of 2 (see Figure I-4, Appendix I).

For long-term analysis, using effective soil parameters shown in Table 5.2(a), presented earlier, the minimum safety factor is of the order of 1.4 which is also adequate (Figure I-5, Appendix I).

These calculations show that there is no theoretical slope failure problem for the proposed 10 m high embankments with a 2 m wide mid-height berm.

From a practical point of view, however, surficial slope instability problems in abutments built from the indigenous clayey silts obtained from this general area are not uncommon. These can manifest themselves as lateral deformations of the embankment which can cause spreading and surface cracking. At the expense of being repetitious, the following are our comments for this type of soil.

This type of soil behaves like a silt, in spite of its relatively high clay size particle content. Unfortunately, such soils are not easily identified on the basis of routine laboratory testing (and consequently would pass MTO's acceptability criteria for use in embankment construction), except by visual observations by personnel experienced in field compaction process.

Based on our experience, such soils which show dilatancy, generate high pore pressures during compaction especially when placed at or particularly in excess of optimum moisture content. They require sufficient time to elapse between lifts to sufficiently reduce the induced pore pressures in the previously compacted lower lifts. As they are susceptible to pumping action, pore pressures are generated in the lower lifts, which can cause disturbance and loss of compaction of the previously placed lifts, especially if vibration is introduced during compaction. They are also easily disturbed by the construction traffic.

In our opinion if such materials are used for road embankment construction, slightly flatter than 2H:1V side slopes should be used (i.e.  $2\frac{1}{4} \sim 2\frac{1}{2}$  :1V), depending on the height of the embankment along with the mid-height berm. In addition, the placement of the top asphalt layer of the road pavement immediately adjacent to the bridge abutments can be delayed for a period of several weeks, before opening the road to truck traffic.

### 5.3 PILE FOUNDATIONS

As mentioned before, advance embankment fills of about 7 m height have already been placed at the site. Steel H-piles (HP-310x110) which will support the abutment foundations have also been installed. The profile of the existing embankment fill as provided by MTO, is shown in a simplified form in Figures J-1, Appendix J. It is noted that the advance fill placed at the east abutment support location is somewhat lower than that at the west abutment support location. Therefore the analyses for downdrag and lateral bending of the piles are based on the existing embankment fill profile of the east approach.

In Figure J-1, Materials 3 and 5 represent the advance embankment fills, which are presently in place. However, Material 3 will be excavated and replaced by light-weight fill. Additional fill in this area immediately adjacent to the abutments will consist of light-weight fill, as represented by Material 2 shown in Figure J-1. Material 4 represents the embankment fills which will be placed on top of the advance fills to the pavement subgrade level of Butter Road, while Material 1 represents the pavement structural materials. Material 6 is native soil, which is underlain by bedrock.

It is understood that the existing embankment fill was placed in November 2003. Our analyses indicate that approximately one-third of the settlement of the native soil under the advance fill has occurred to date.

Additional light-weight fill and regular fill materials as represented by Materials 1 to 4 will be placed for the construction of the approach embankments. The embankment fill will cause the soil at the pile locations to move vertically and horizontally away from the approach embankments. The displacements of the soil will cause lateral load and downdrag load (i.e. negative skin friction) on the piles.

A finite element model, utilizing the SIGMA/W computer software, was used to estimate the horizontal and vertical displacements of the (native) soil below El. 221 m at the pile locations due to the placement of the approach embankment fill. In the analysis, a 2-dimensional finite element model established along the centre line of Butter Road was used. The bedrock was assumed at El. 200 m while the depth of the native soil was taken to be 21 m (from El. 221 to 200 m). The height of the embankment ranged from about 10 m adjacent to the abutment wall to about 8.5 m at a location 140 m away from the centre line of Highway 6 (New), as shown in Figure J-1 in Appendix J. The general patterns of the vertical and horizontal displacements as predicted by the finite element analysis are given in Figures J-2 and J-3, Appendix J.

The calculated settlement of the ground surface (El. 221 m) along Butter Road due to the stress imposed by embankment fill by finite element analysis is shown in Figure J-4, Appendix J. Based on this type of analysis, a maximum settlement of about 87 mm occurs at a location of about 55 m from the centre line of Highway 6 (New), or about 25 m from the abutment wall. As mentioned before, this analysis takes into consideration the settlement that has already taken place due to the advance fills and the predicted settlements are over and above these settlements.

Based on the finite element analysis, the displacements of the native soil (below El. 221 m) at the pile location were obtained, as shown in Figure J-5 (Appendix J). The estimated settlement of soil ranged from 50 mm at original ground surface (El. 221 m) to zero at the bedrock surface (El. 200 m). Our analysis shows that maximum horizontal displacements occur about 3 m and 10 m below the original ground surface or between El. 218 and 211 m, respectively, with a maximum horizontal displacement of about 21 mm at about El. 214 m.

The effects of the soil displacements on the existing piles are discussed in the following sections. In the analysis, the piles were assumed to be straight and vertical, although two of the piles (i.e. outer edge piles) were battered. The piles were assumed to extend to the surface of the bedrock. Therefore, the tip of the piles is assumed not to move vertically and horizontally and the top of the abutment wall is assumed to be fixed horizontally.

### 5.3.1 AXIAL LOADING ON THE PILES

As indicated in Figure J-5, the soil around the existing piles would settle due to the embankment fill and cause downdrag load on the piles, in addition to the loads from the structure at the top of the pile. It is understood that the axial resistance of the pile at ULS is 2000 kN. The actual load on a single pile is assumed to be 1600 kN after allowing a load factor of 1.25 (i.e.  $2000/1.25 = 1600$  kN).

Based on the settlement of the soil at the pile location as shown in Figure J-5 and an estimation of the adhesion between the pile and soil, the downdrag load on the pile can be evaluated. A critical parameter in the evaluation of the downdrag load is the adhesion ( $C_a$ ), between the pile and the soil, which is related to the shear strength of the soil. The value of the adhesion  $C_a$  is very empirical. Based on our knowledge and literature search in this regard, a maximum adhesion value of 47 kPa was used in the downdrag analysis according to the past experience (Tomlinson 1957, 1963; Prakash and Sharma, 1990; Poulos 2003\*). Our analysis was based on the premise that the maximum adhesion value of 47 kPa is mobilized at a relative settlement of 15 mm between the pile and the soil.

Based on the above mentioned assumptions, the estimated accumulated downdrag load on the pile ranges from zero at ground surface (El. 220 m) to about 1006 kN at the tip of the pile (El. 200 m), as shown in Figure J-6, Appendix J. Under the downdrag load, the top of the pile would settle for about 4 mm due to the compression of the steel pile.

In summary, the calculated vertical loads on the existing piles are as follows:

Working load:	1600 kN/pile
Maximum downdrag load:	1006 kN/pile (at El. 200 m)

### 5.3.2 LATERAL BENDING

As shown in Figure J-5, the soil at the pile location moves horizontally due to the placement of the embankment fill. This will cause a lateral displacement of the pile. Our analysis indicates that the pile is flexible laterally and will move horizontally with the soil. The lateral displacement of the pile will result in bending stress in the pile due to lateral soil load.

---

#### \* References:

- Poulos, M.G. (2003). "Analysis and Design of Pile Foundations" Seminar organized by Canadian Geotechnical Society, Toronto, October 2003 (handout material).
- Prakash, S. and Sharma, H.D. (1990). "Pile Foundations in Engineering Practice." John Wiley & Sons Inc., New York.
- Tomlinson, M.J. (1957). "The Adhesion of Piles Driven into Clay Soils." Proc. 4<sup>th</sup> Int. Conference on Soil Mechanics and Foundation Eng., London.
- Tomlinson, M. J. (1963). "Foundations Design and Construction." Wiley, New York.

Using a finite element model in which the pile is simulated by beam elements, Figure J-7, Appendix J presents a general approach model used to analyse this problem. With this model, bending stress along the depth of the pile can be estimated (see Figure J-8, Appendix J). The maximum bending stress of 8 MPa occurs at a depth of about 7 m (El. 214 m). Based on the analysis, we do not anticipate problems due to lateral bending (i.e. lateral yield) of the piles.

It should be noted that lateral design loads (i.e. lateral load due to traffic and earth pressure on abutment walls) were not included in the above-mentioned analysis. This aspect should be discussed with the structural engineer.

### 5.3.3 PILE STRESS

The analyses presented above indicate that the pile stresses will be combination of axial working load (1600 kN), accumulated downdrag load and bending due to lateral displacement. Based on the analysis results, the maximum stress of the pile due to a combination of axial working load, downdrag load and bending will occur at the pile tip (El. 200 m), as presented below:

Stress due to working load (1600 kN):	113 MPa
Stress due to downdrag (1006 kN):	71 MPa (at El. 200 m)
Bending stress due to lateral displacement:	0 MPa (at El. 200 m)
Maximum total stress:	184 MPa (at El. 200 m)

This is because, in spite of the fact that the bending stresses due to lateral displacement are zero (since the pile is driven into bedrock the tip is assumed to be fixed horizontally and vertically but free to rotate), the accumulated downdrag is quite high. At a depth of 10 m below O.G. (i.e. at El. 211 m), the calculated stresses due to bending is about 8 MPa, but the accumulated total stress is 149 MPa (i.e. less than 184 MPa).

As mentioned, the maximum total pile stress is estimated at 184 MPa. It is understood that the yield stress of the HP-310x110 pile is 350 MPa. This indicates that the factor of safety of pile is 1.90 in terms of yield stress (i.e.  $350 \text{ MPa} \div 184 \text{ MPa} = 1.90$ ).

If the elastic yield range of the stress in the steel pile is exceeded, the pile would not be expected to totally collapse but would yield in the plastic range at an increased strain rate (while still withstanding additional stresses) until an ultimate stress is reached. Assuming that the ultimate failure stress for the steel used for the piles is 500 MPa, the factor of safety against a total steel failure would be in excess of 2. In addition as the pile compresses in the plastic range, the downdrag loads would be slightly reduced. The slight reduction may, however, reverse itself if the soil surrounding the pile continues to settle under the embankment loads.

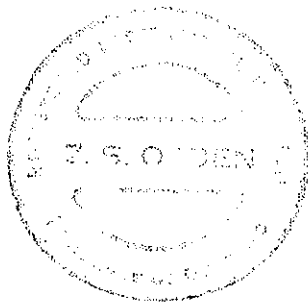
It should also be pointed out that the maximum stress of 184 MPa was obtained from combined working loads and downdrag loads. The working load normally includes live and dead loads. Since the transient live loads need not necessarily be included in the calculation of ultimate stresses when combined with stresses due to downdrag loads, the actual stresses may be somewhat smaller than 184 MPa. This aspect may be discussed with the Structural Engineer and a discussion of this aspect is available in Section 20.2.5.1 of the Canadian Foundation Engineering Manual, 3rd Edition, 1992.

It must be pointed out that the downdrag load is estimated based on an adhesion of 47 kPa between the pile and the soil. As discussed previously, the adhesion parameter is very empirical and the value of 47 kPa for the analysis was selected based on our best knowledge. The actual value of adhesion for downdrag may be higher than 47 kPa.

In summary, our analysis indicates that the maximum total stress of the pile resulting from a combination of axial working load, downdrag load and lateral bending is 184 MPa, which is smaller than the yield stress of 350 MPa. While the possibility of the pile stress being higher than the yield stress cannot be excluded due to possible higher downdrag load, our analyses lead us to believe that there is no conceivable danger of the collapse of the structure due to the yield of the piles.

## 6.0 CLOSURE

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



### SHAHEEN & PEAKER LIMITED

Fanyu Zhu, Ph.D., P.Eng. \*

Z.S. Ozden, P.Eng.

ZO/tr:drive

K. R. Peaker, Ph.D., P.Eng. \*\*

\*The original draft report was signed and sealed by Dr. Fanyu Zhu who is no longer with the company.

\*\* The original draft report was signed and sealed by Dr. Ken Peaker who is no longer with the company.

The report was finalized on December 14, 2009 without making any changes to the draft report.



## Appendix G

# Proposed Configuration of Bridge and Light-Weight Fill by MTO

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

DIST 4  
CONT No 2003-2011  
WP No 605-00-01



HIGHWAY 6 NEW UNDERPASS  
AT BUTTER ROAD  
GENERAL ARRANGEMENT

SHEET  
403A

#### NOTE

W.P. DENOTES WORKING POINT  
T/P DENOTES TOP OF PAVEMENT  
T/FTG. DENOTES TOP OF FOOTING

#### GENERAL NOTES

##### CLASS OF CONCRETE

PRECAST GIRDERS..... 50 MPa  
REMAINDER..... 30 MPa

##### CLEAR COVER TO REINFORCING STEEL

FOOTINGS..... 100±25  
DECK : TOP..... 70±20  
BOTTOM..... 40±10  
REMAINDER..... 70±20 UNLESS  
OTHERWISE NOTED

##### REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BARS MARKED WITH PREFIX 'C' DENOTE COATED BARS. BARS MARKED WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS. STAINLESS REINFORCING STEEL SHALL BE TYPE 316 LN OR DUPLEX 2205 AND HAVE A YIELD STRENGTH OF 420 MPa. UNLESS SHOWN OTHERWISE, LAP LENGTHS NOT INDICATED ON THE CONTRACT DRAWINGS SHALL BE CLASS 'B' AS PER OHBDC. BAR HOOKS, WHERE REQUIRED, SHALL BE MINIMUM LENGTH AND STIRRUPS SHALL HAVE MINIMUM HOOKS (SEE DWG. 15), UNLESS INDICATED OTHERWISE.

##### RETAINED SOIL SYSTEM (RSS)

- APPLICATION: WALL/SLOPE  
- PERFORMANCE RATING: HIGH  
- APPEARANCE RATING: HIGH

THE SYSTEM SHALL BE DESIGNED USING THE ULTRA LIGHT WEIGHT FILL BEHIND THE ABUTMENTS AS AN ENGINEERED FILL.

##### APPROACH EMBANKMENTS AND ABUTMENTS PILES

THE PLAN AREA OF THE PROPOSED APPROACH EMBANKMENTS SHALL BE PROOF ROLLED AND ANY SOFT GROUND SHOULD BE REMOVED AND REPLACED WITH GRANULAR MATERIAL. PLACEMENT OF THE ULTRA LIGHTWEIGHT FILL SHALL BE CARRIED OUT AS SPECIFIED ELSEWHERE IN THE CONTRACT.

#### CONSTRUCTION NOTES

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

NO BACKFILL SHALL BE PLACED UNTIL DECK CONCRETE HAS REACHED 75% OF ITS SPECIFIED STRENGTH.

BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROX. THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATIONS BE GREATER THAN 500mm.

#### LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS & SOIL STRATA
3. PILE LAYOUT AND FOOTING DETAILS
4. ABUTMENT DETAILS
5. PIER DETAILS
6. PRESTRESSED GIRDERS AND BEARINGS
7. DECK DETAILS
8. DECK REINFORCEMENT I
9. DECK REINFORCEMENT II
10. 6000mm APPROACH SLAB
11. BARRIER WALL WITH SIDEWALK AND RAILING
12. RAILING FOR BARRIER WALL
13. DETAILS OF CONCRETE SLOPE PAVING
14. STANDARD DETAILS
15. ELECTRICAL EMBEDDED WORK
16. QUANTITIES - SHEET II

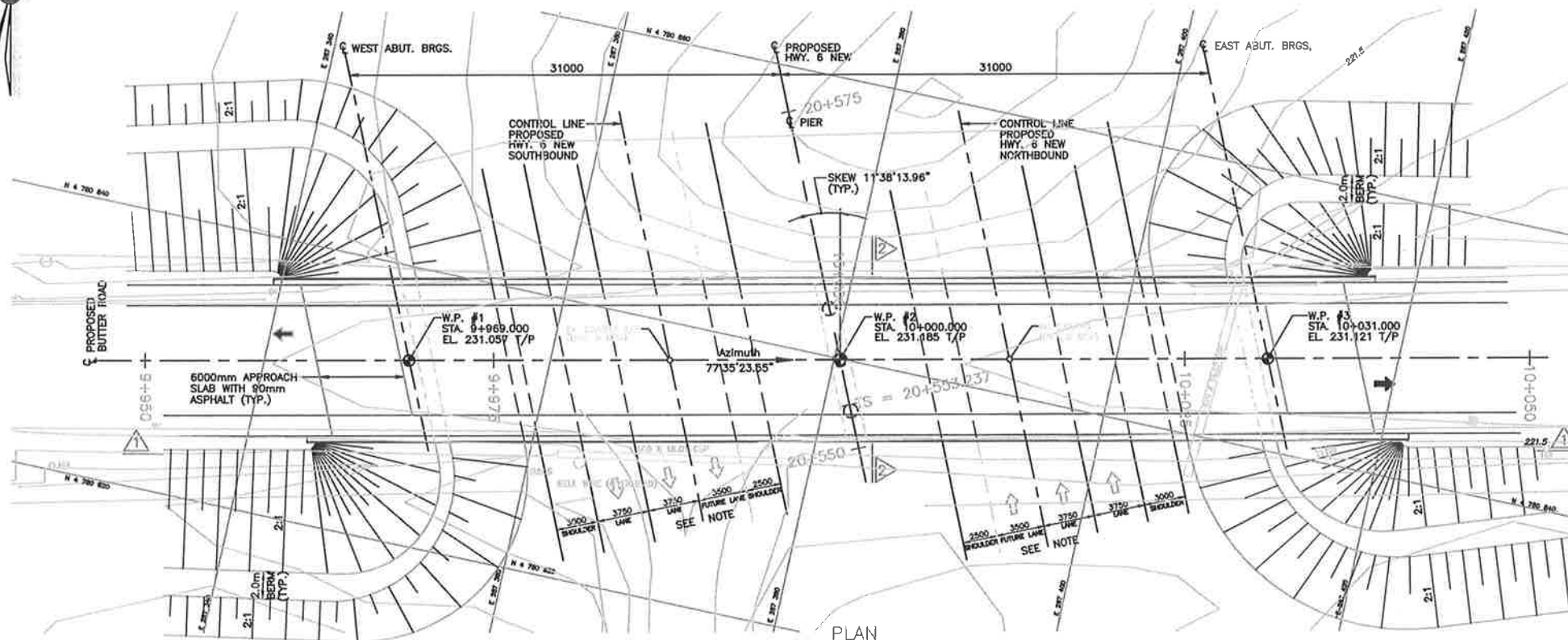
#### APPLICABLE STANDARD DRAWINGS

OPSD-918.01 TRANSITION TO STRUCTURES  
OPSD-3906.02 BRIDGE DECK WATERPROOFING  
OPSD-4010.00 GUIDE RAIL AND CHANNEL ANCHORAGE  
OPSD-4601.000 LOCATION OF SITE NUMBER AND DATE FIGURES  
OPSD-4670.000 TYPICAL JOINT DETAILS

REVISIONS	DATE	DESCRIPTION
JAN. 04	NEW FOUNDATION REPORT	
NOV. 03	BERMS AROUND BRIDGE APPROACHES SHOWN	
DESIGN	M.M. CHK E.G. CODE OHBDC-91	LOAD CL-A DATE JUNE 2002
DRAWN	A.P. CHK M.M. SITE 36-496	DWG 1

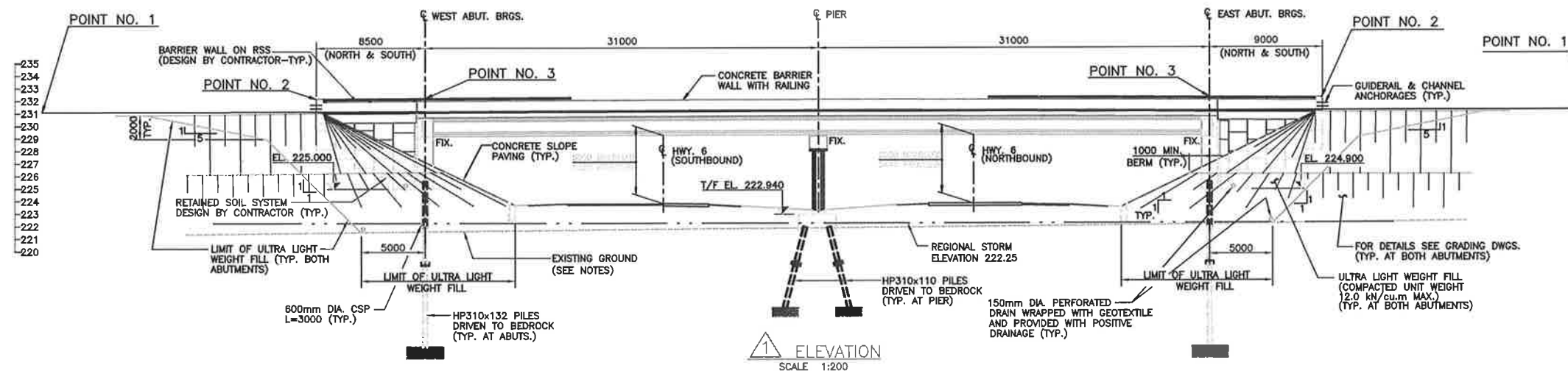


MINISTRY OF TRANSPORTATION, ONTARIO

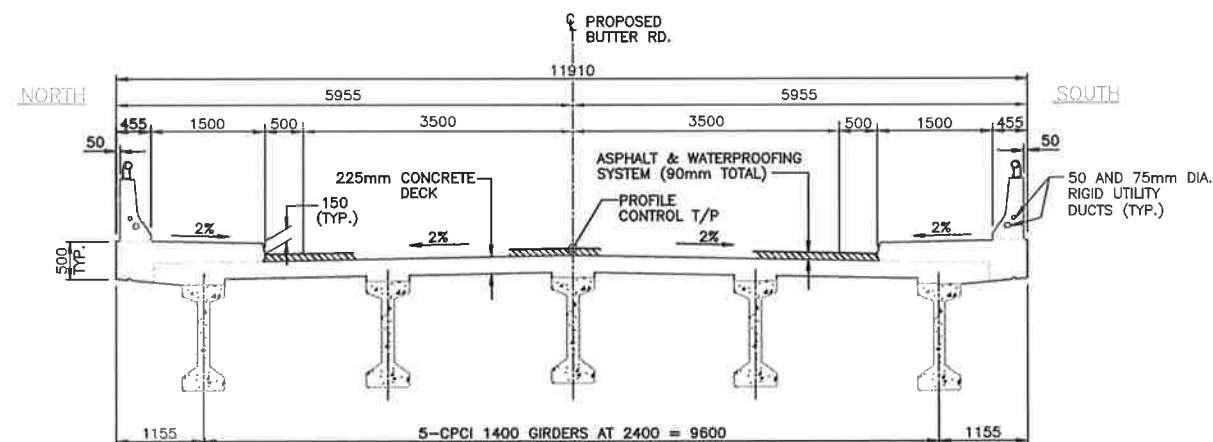


NOTE: ULTIMATE SITUATION SHOWN. FOR PRESENT SITUATION SEE GRADING DRAWINGS.

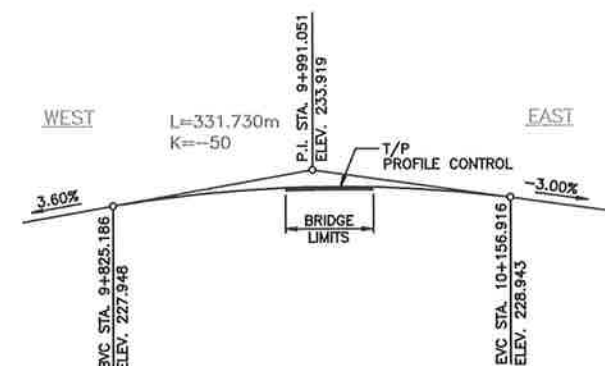
PLAN  
SCALE 1:200



ELEVATION  
SCALE 1:200



TYPICAL DECK SECTION  
SCALE 1:50



PROFILE OF BUTTER ROAD  
N.T.S.



DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

B.M. 220.692  
TOP OF S-E CORNER  
CONCRETE CULVERT  
8.558 RT. STA. 9+940.123

# Appendix H

## Stress Distribution Under Embankment Fills

SPT1120 - Highway 6 (New) at Butter Road, Hamilton, ON  
Vertical Stresses due to Embankment Fill

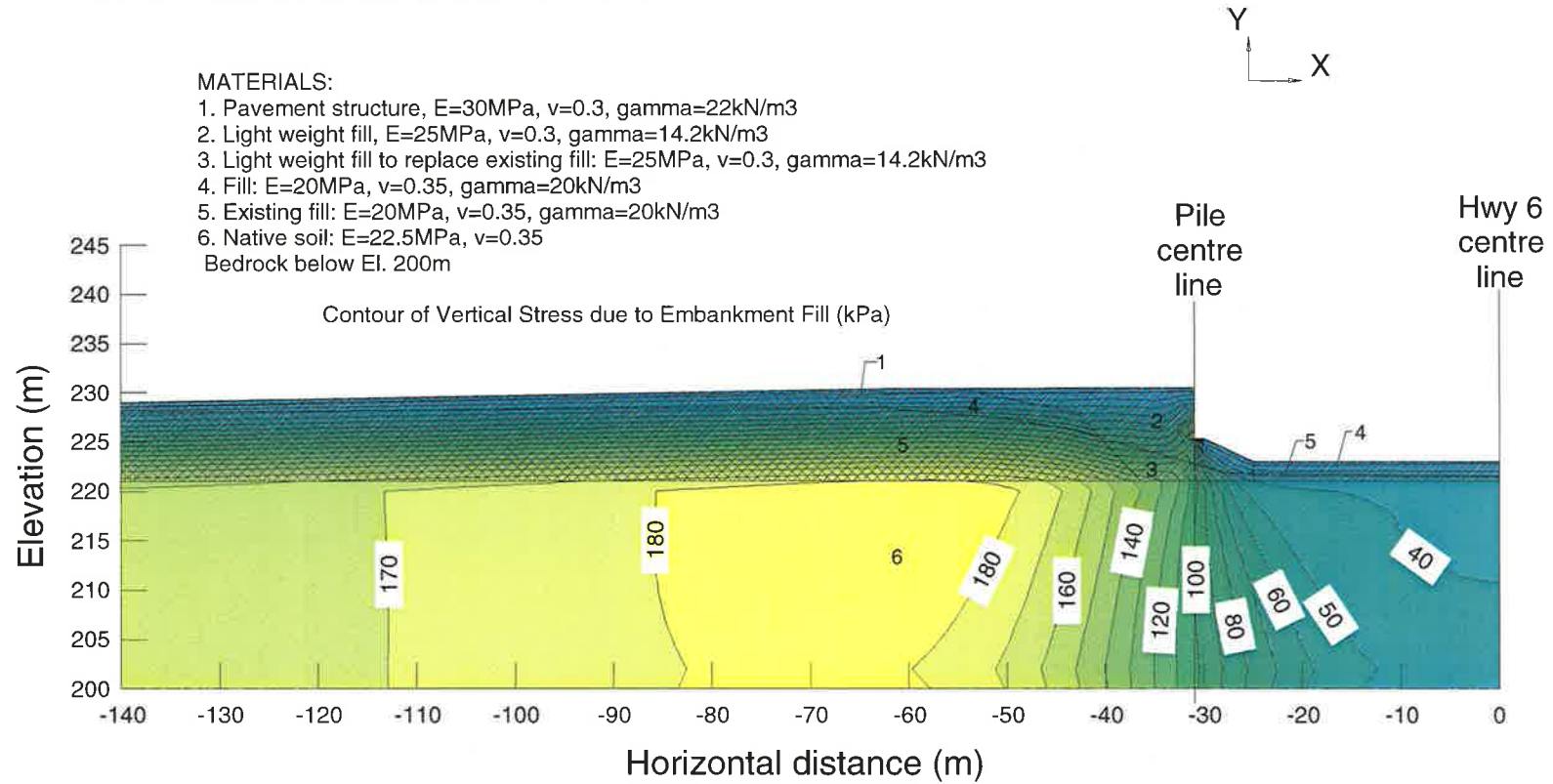


Figure H-1a

SPT1120 - Butter Road at Hwy 6 (New), Hamilton, ON

Vertical Stress due to Embankment Fill - Cross Section

Fill:  $E=20\text{MPa}$ ,  $\nu=0.35$ ,  $\gamma=20\text{kN/m}^3$

Native:  $E=22.5\text{MPa}$ ,  $\nu=0.35$

Embankment: Height = 10m, Top width = 15m; Berm width = 2m, Slope 2H:1V

Contour of Vertical Stress due to Embankment Fill (kPa)

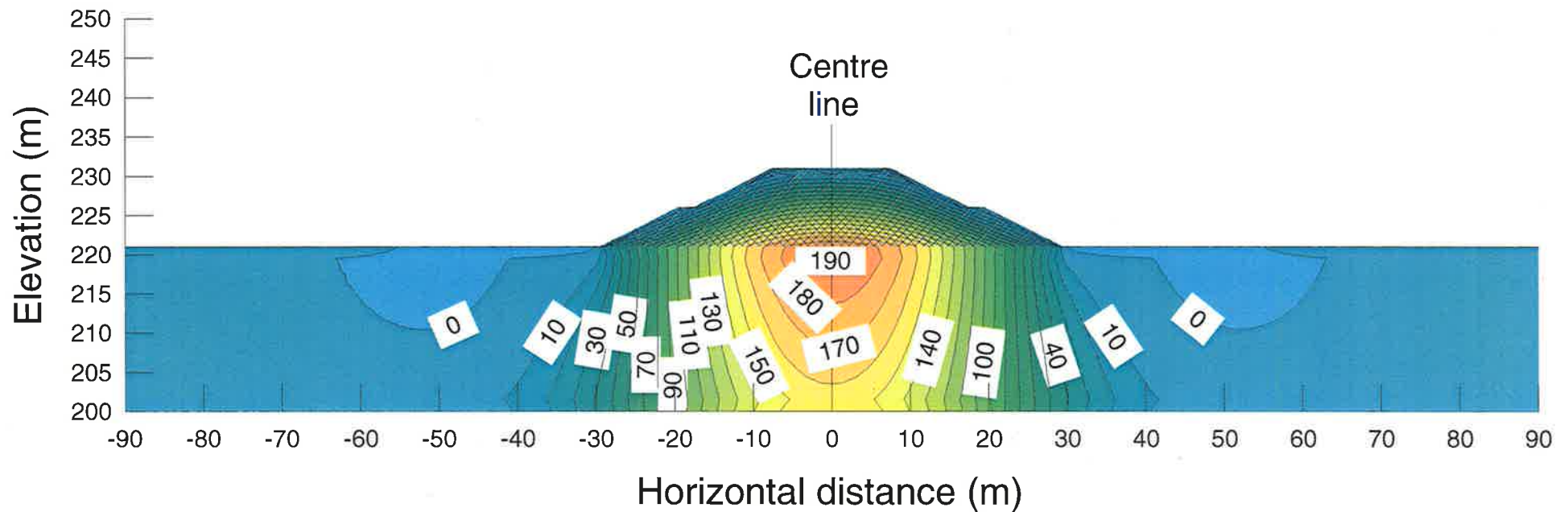


Figure H-1b

# Appendix I

## Typical Slope Stability Results

Highway 6 (New) / Butter Road  
 10 m High, 15 m Wide Embankment  
 2 m Mid-height Berm  
 (Undrained Case)

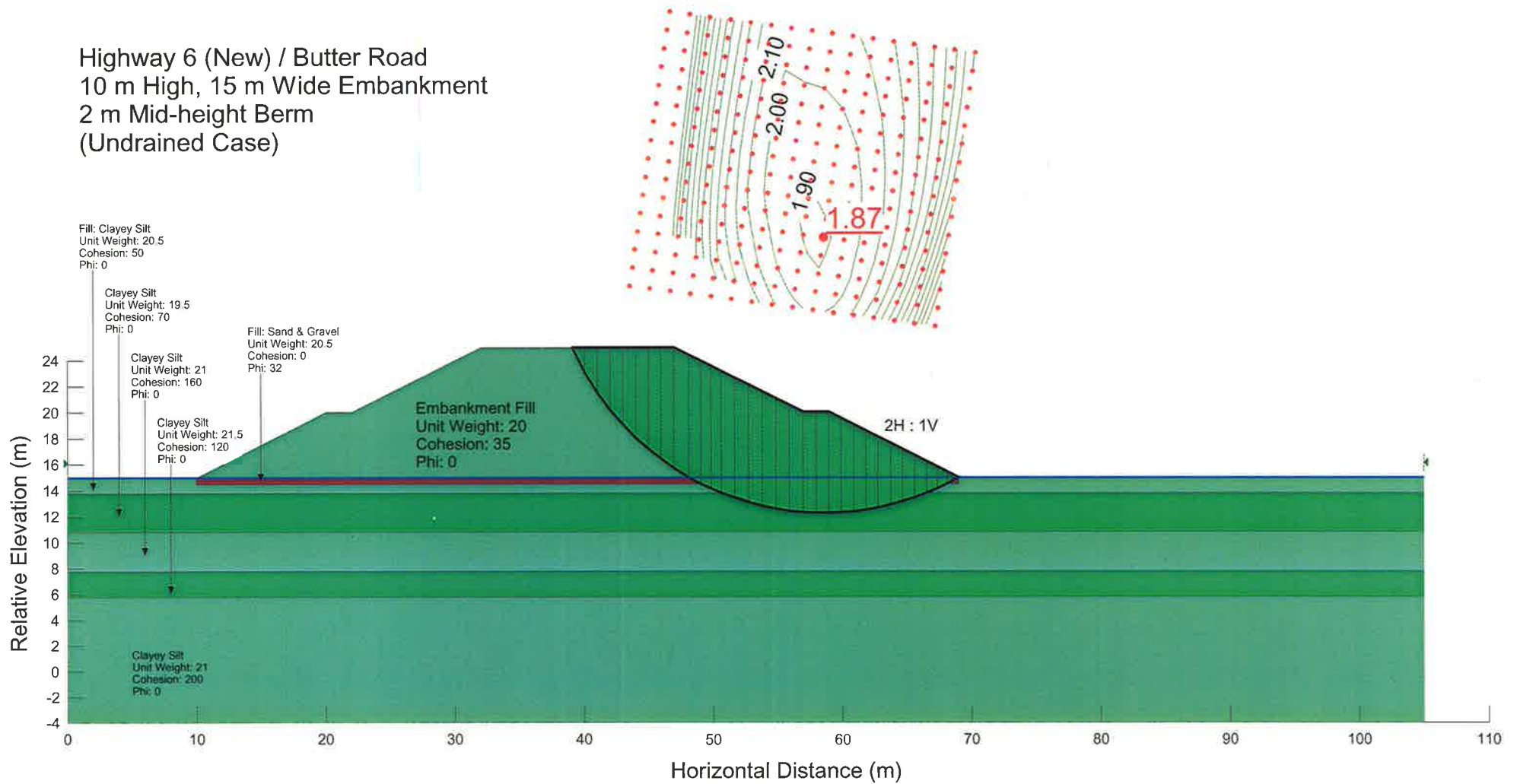


FIGURE I-1



Highway 6 (New) / Butter Road  
 10 m High, 15 m Wide Embankment  
 2 m Mid-height Berm  
 (Drained Case)

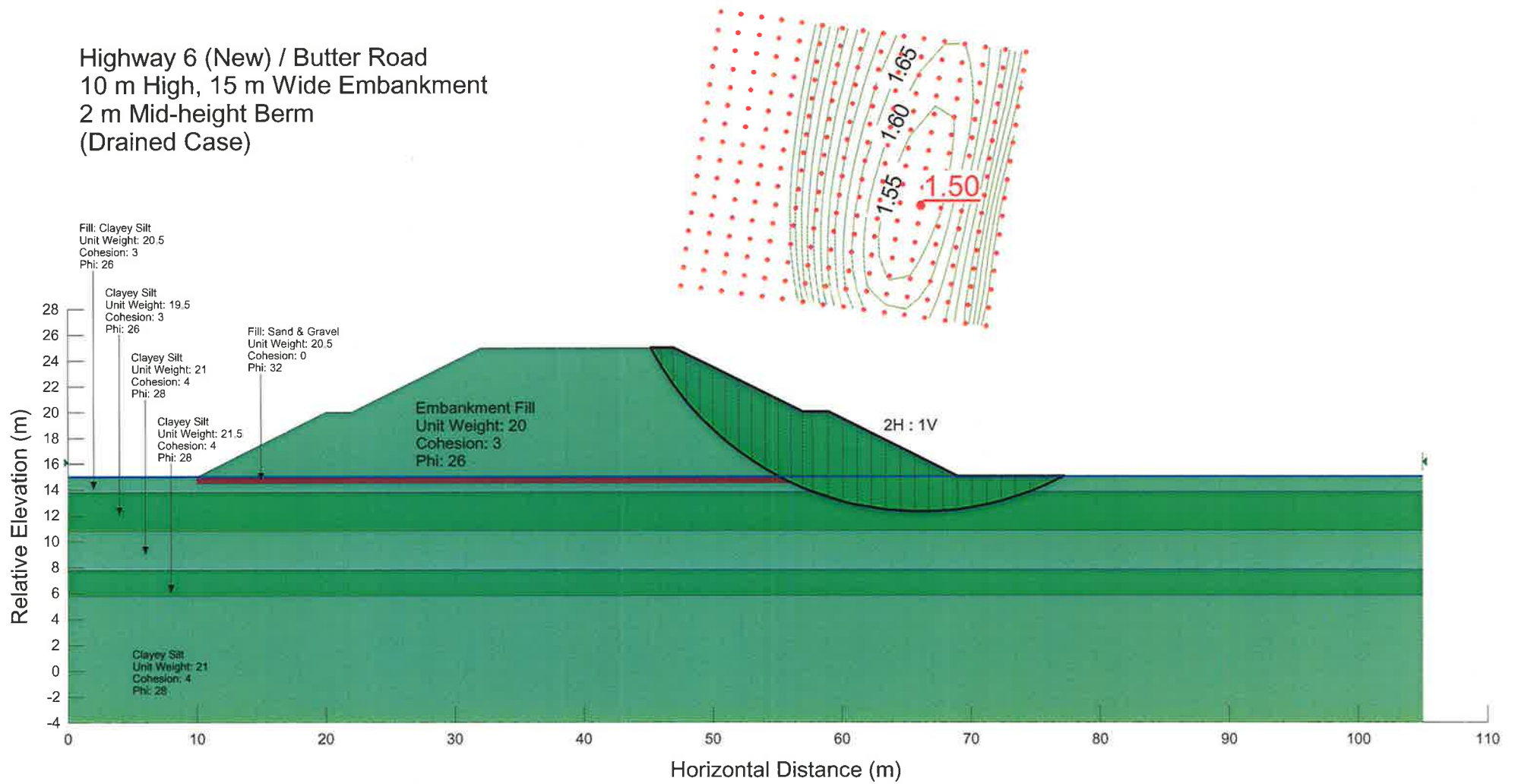


FIGURE I-2



Highway 6 (New) / Butter Road  
 10 m High, 15 m Wide Embankment  
 2 m Mid-height Berm  
 (Undrained Case)

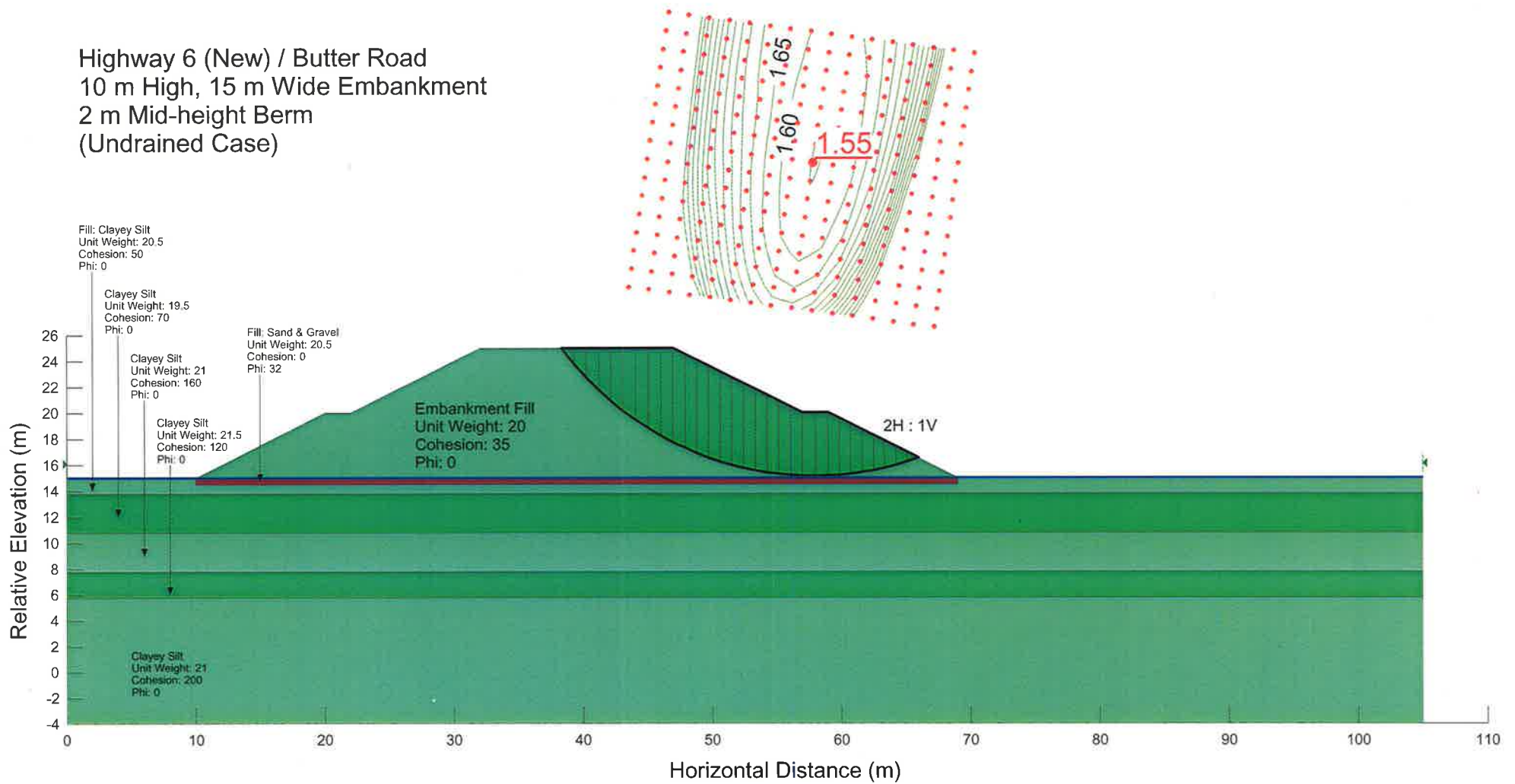


FIGURE I-3

Highway 6 (New) / Butter Road  
 10 m High, 15 m Wide Embankment  
 2 m Mid-height Berm  
 (Undrained Case)

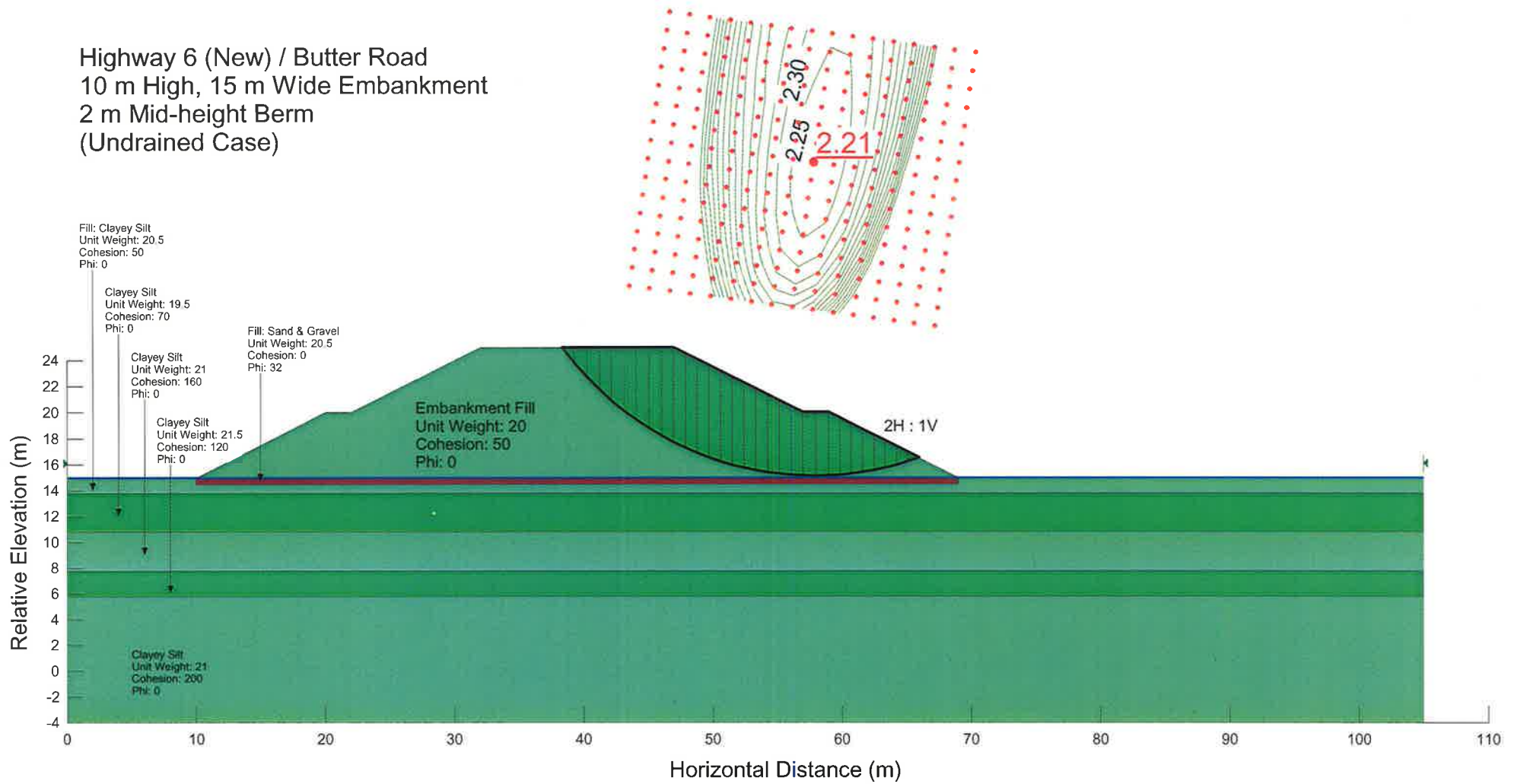


FIGURE I-4

Highway 6 (New) / Butter Road  
 10 m High, 15 m Wide Embankment  
 2 m Mid-height Berm  
 (Drained Case)

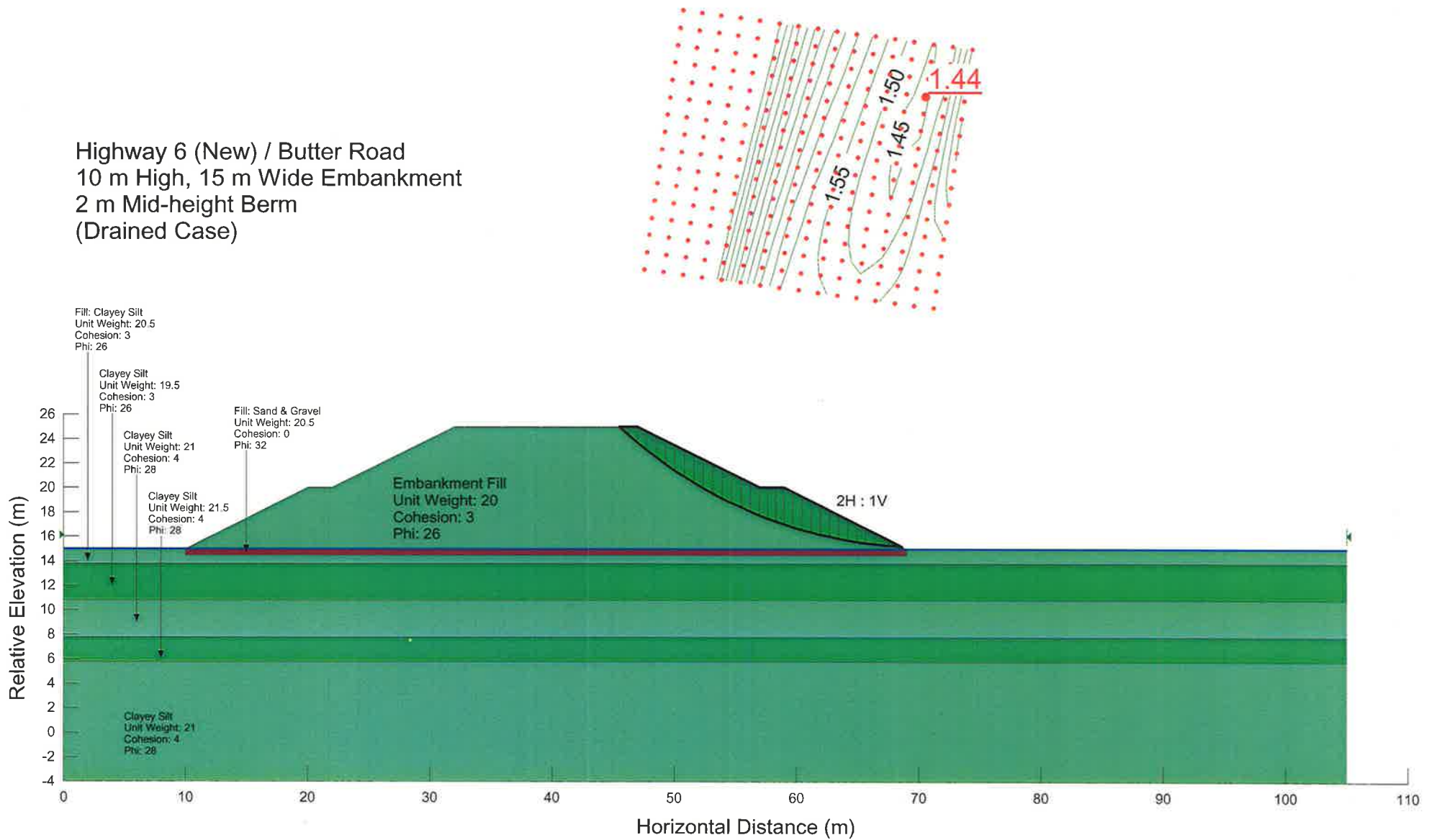


FIGURE I-5

# Appendix J

## Effects on Piles

## SPT1120 - Highway 6 (New) at Butter Road, Hamilton, ON

### Soil Parameters Effective to Settlement Analysis

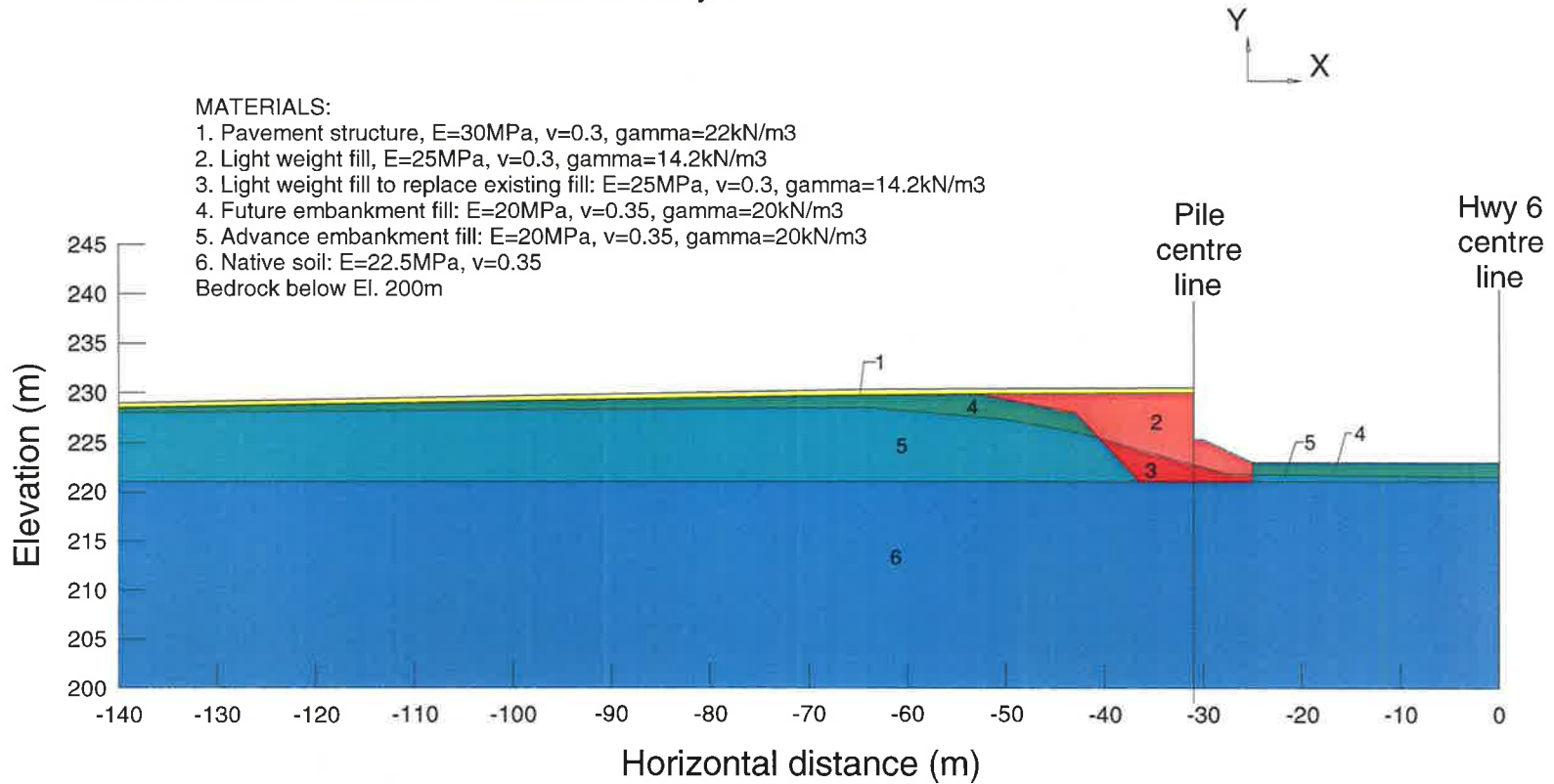


Figure J-1

SPT1120 - Highway 6 (New) at Butter Road, Hamilton, ON  
Settlement Analysis

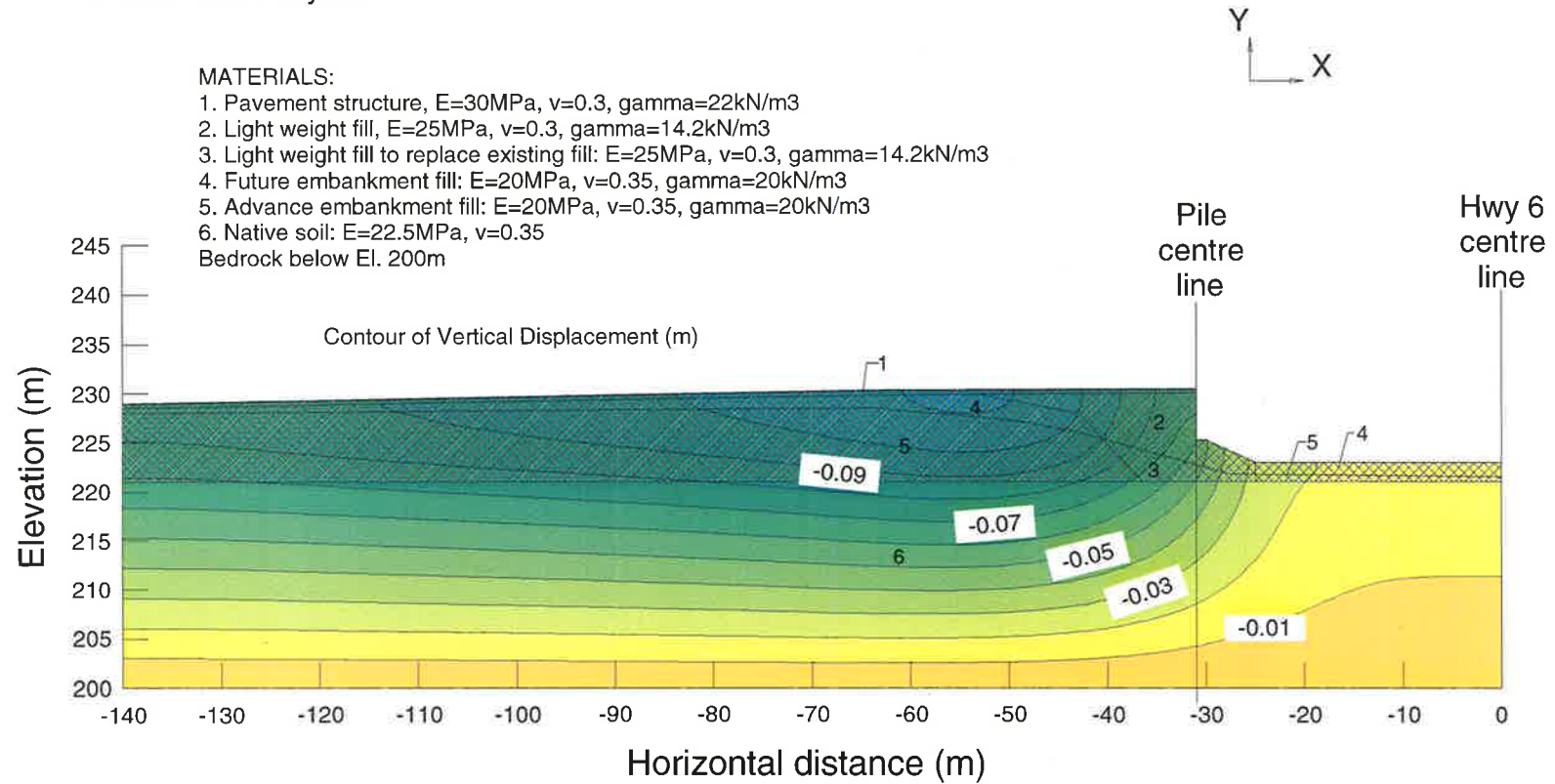


Figure J-2

SPT1120 - Highway 6 (New) at Butter Road, Hamilton, ON  
Settlement Analysis

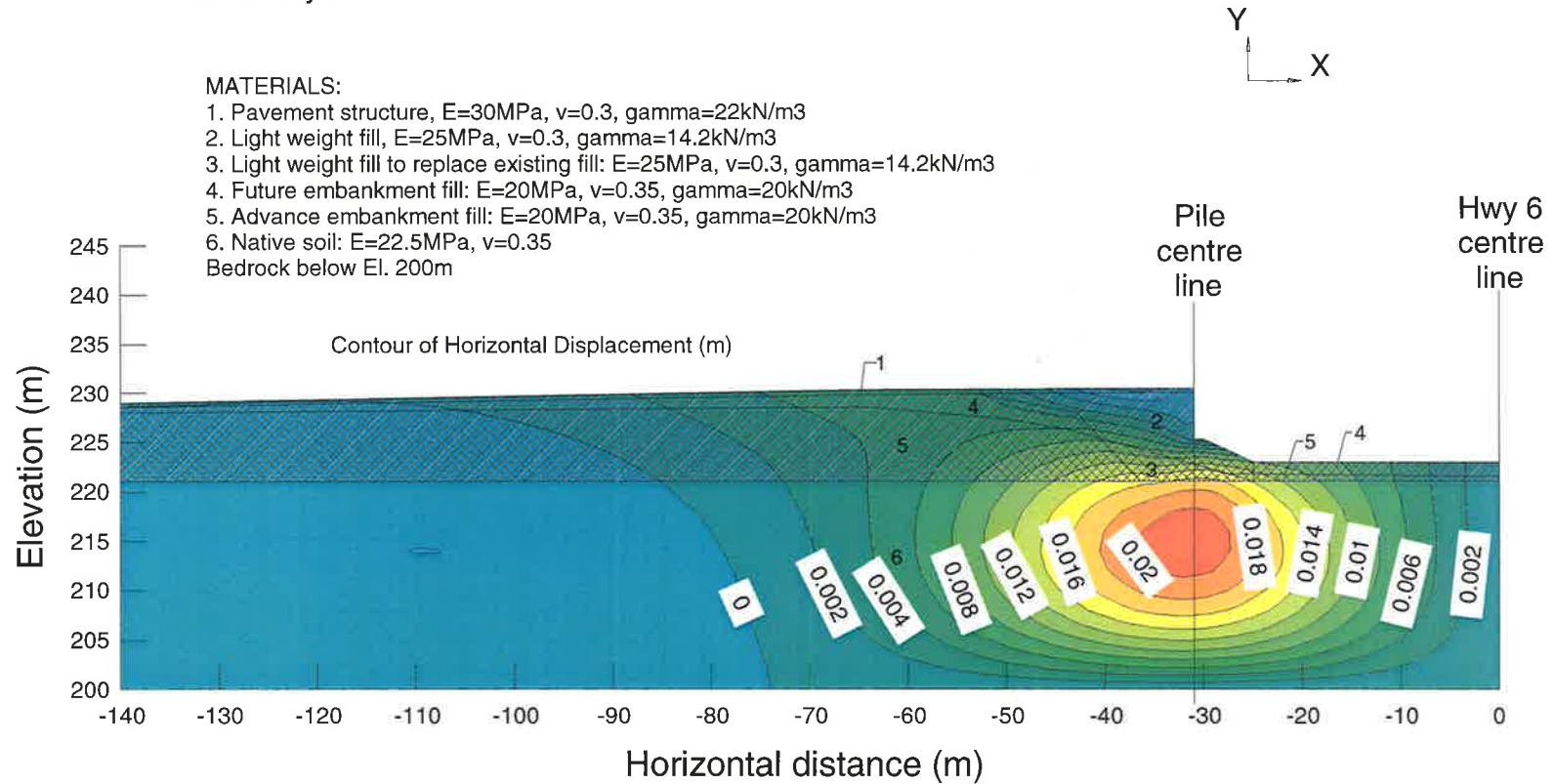


Figure J-3

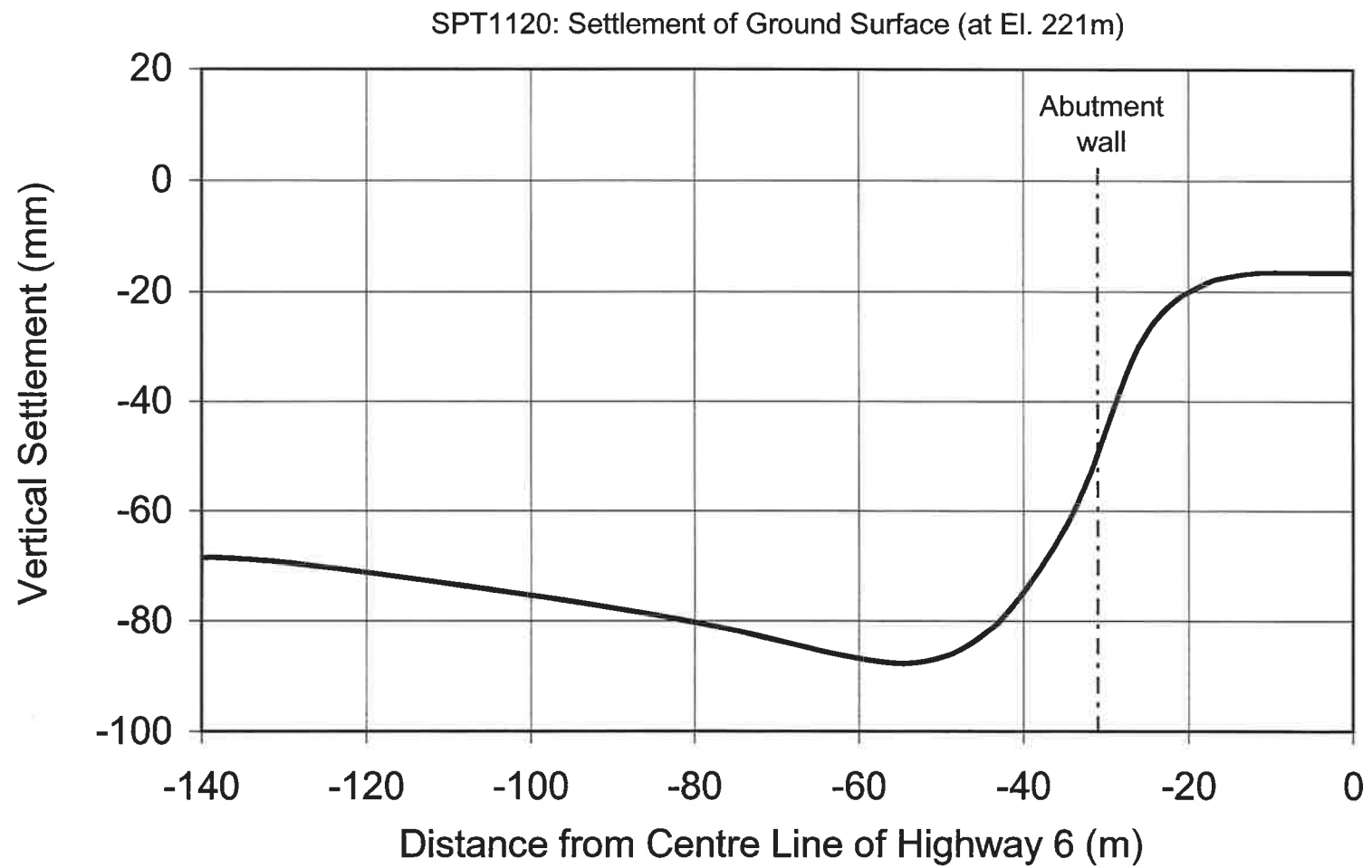


Figure J-4



SPT1120: Soil Displacements at Pile Location

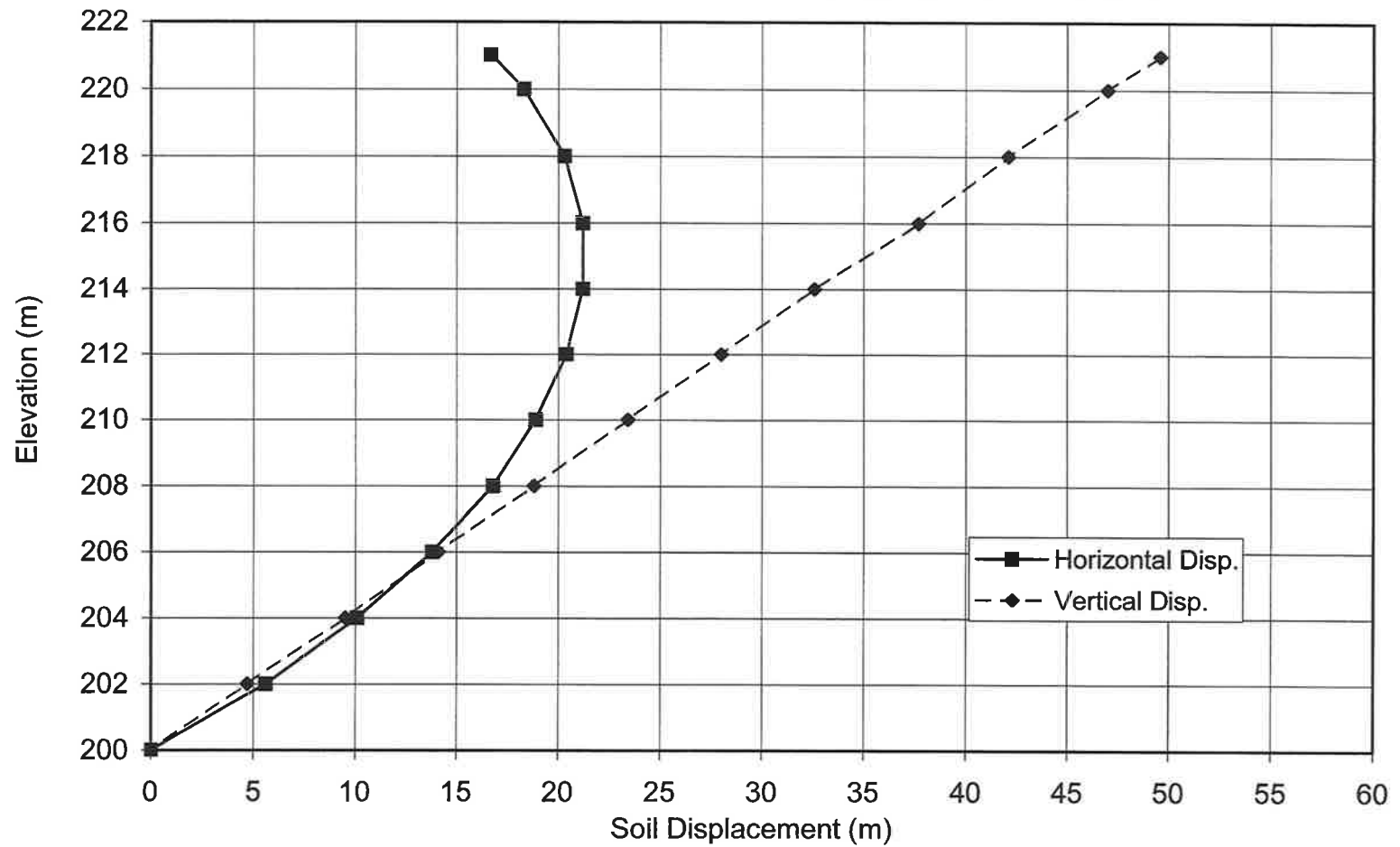


Figure J-5

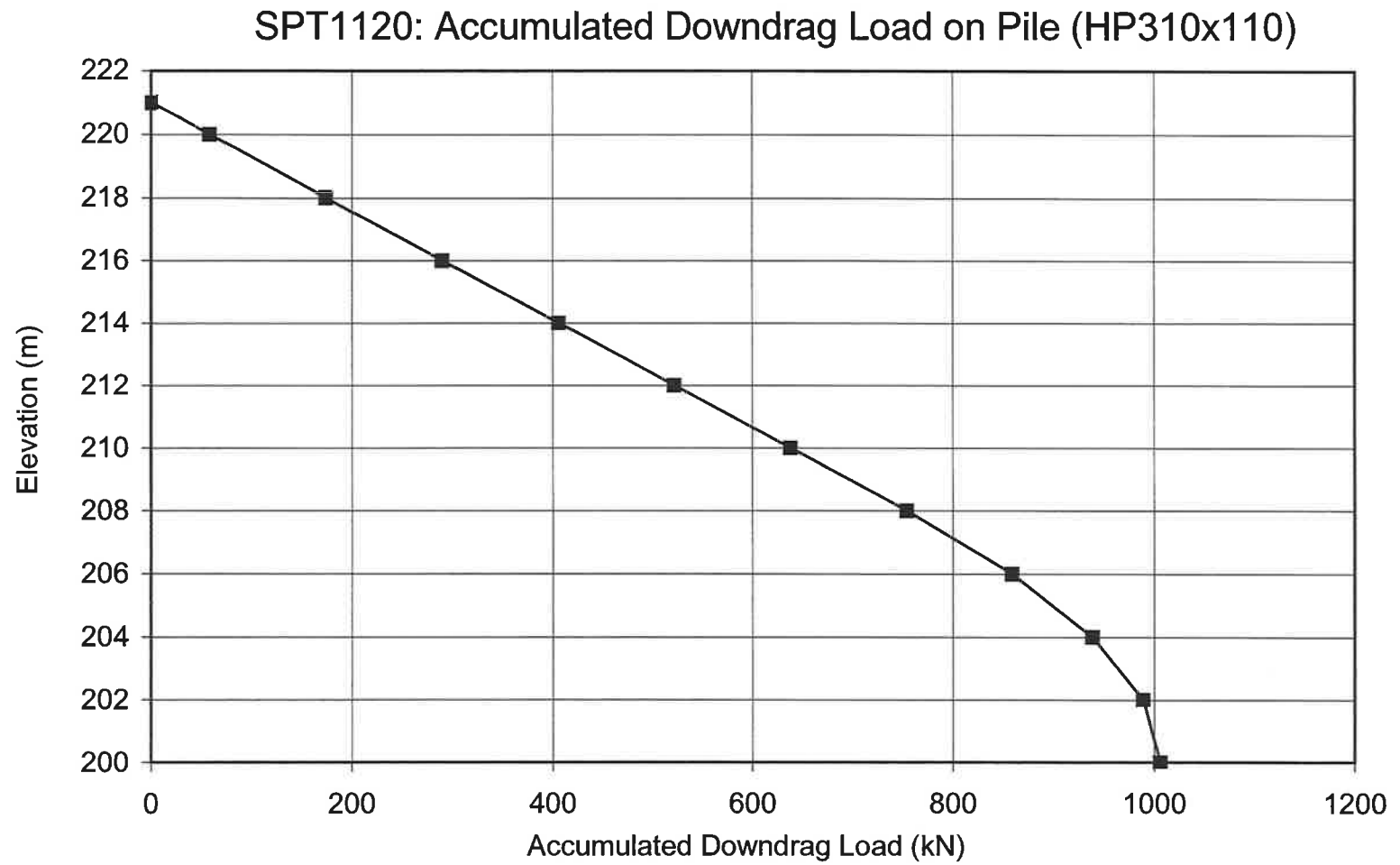


Figure J-6

SPT1120 - Highway 6 (New) at Butter Road, Hamilton, ON  
Pile Displacement Analysis due to Lateral Soil Load

- HP-310x110 Pile tip in Rock at Depth 21 m (El. 200 m).
- Pile in CSP at Depth 0 to -3 m
- Rigid Abutment Wall at Depth -3 to -7.0 m

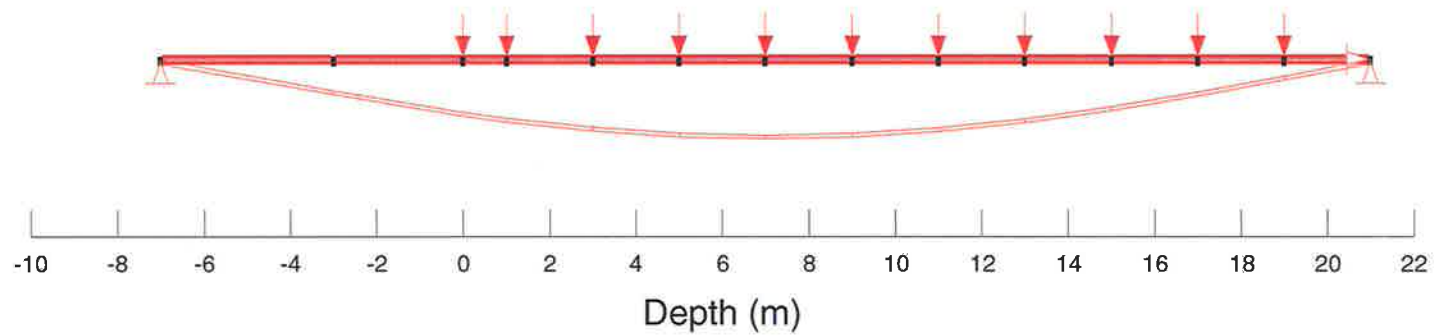


Figure J-7

SPT1120: Bending Stress due to Lateral Displacement of Pile

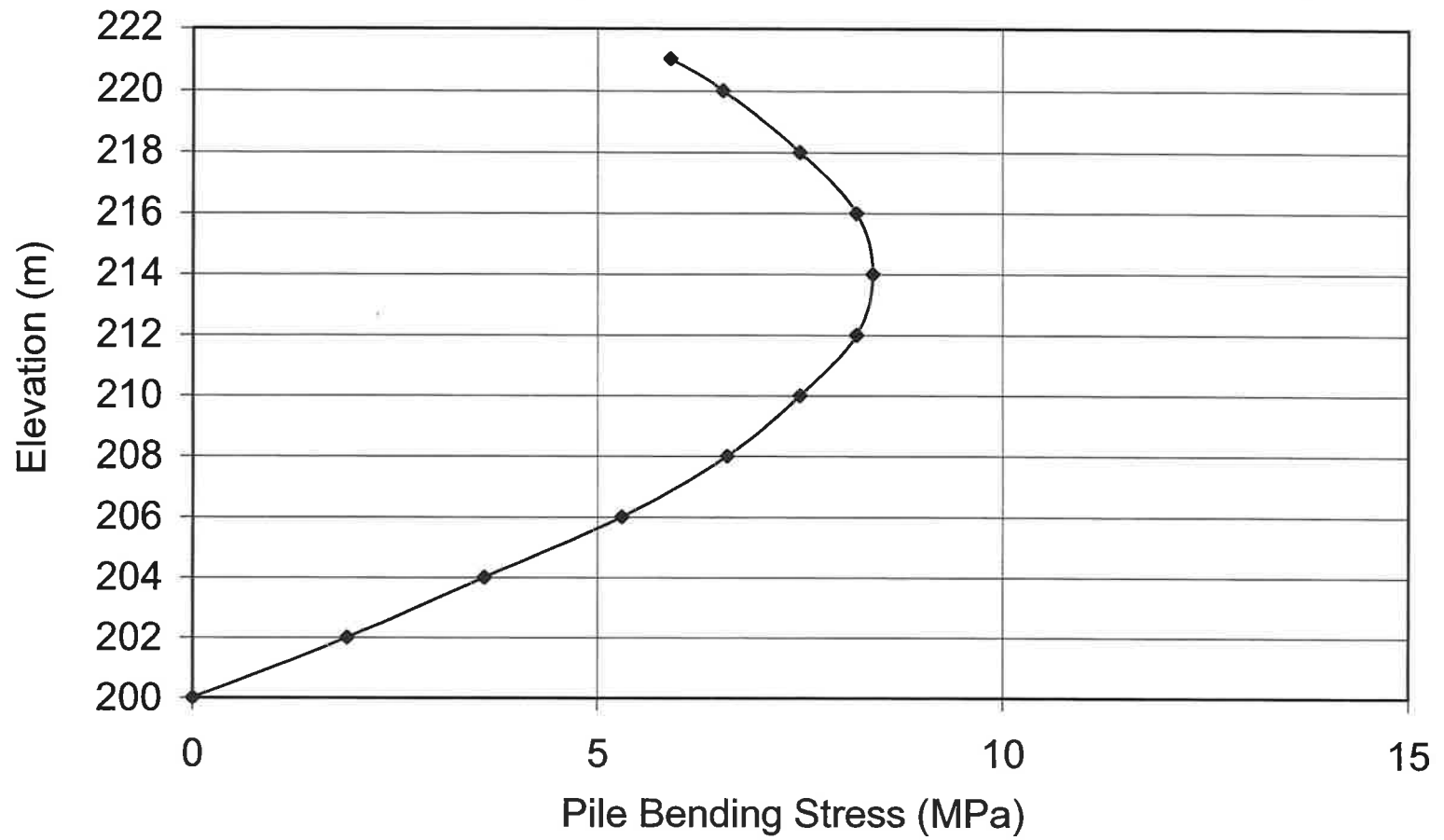


Figure J-8

# 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 Shaheen & Peaker Limited at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker Limited, 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. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time. Any user of this report specifically denies any right to claims against the Consultant, Sub-Consultants, their officers, agents and employees in excess of the fee paid for professional services.