FOUNDATION INVESTIGATION REPORT
HIGHWAY 6 (NEW) AND GLANCASTER ROAD
HAMILTON, ONTARIO
W.P. 604-00-01

Prepared For:

MINISTRY OF TRANSPORTATION – CENTRAL REGION

Prepared by:

SHAHEEN & PEAKER LIMITED

Project: SPT1105
December 3, 2003

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Table of Contents

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1. INTRODUCTION

Shaheen & Peaker Limited (S&P) was retained by the Ministry of Transportation Ontario (MTO) to carry out a foundation investigation at the site of the existing Highway 6 (New) underpass at Glancaster Road in Hamilton, Ontario.

The site is located at Glancaster Road about 0.6 m south of Airport Road and several kilometers 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 vane and laboratory testing, as detailed in MTO's RFQ documents, dated July 23, 2003.

The findings of the investigation are presented in this report.

2. SITE DESCRIPTION AND GEOLOGY

The site is situated at Glancaster Road approximately 600 m south of the intersection of Airport Road and Glancaster Road in Hamilton. The lands next to this site are mainly agricultural and the topography is gently rolling.

The existing structure carries traffic on Glancaster Road over Highway 6 (New) which is presently under construction.

South of the existing bridge (i.e. south of the south abutment) the profile grade of Glancaster Road falls by about 11 m over a horizontal distance of approximately 440 m between Sta. 9+980 and Sta. 9+540. At the north side, the grade falls from about El.228 m (near the north abutment of the structure) to El. 223 m some 280 m north of the structure.

The site is located south of the Niagara Escarpment in the physiographic region known as the Haldimand City Plain. This is a broad undulating plain of glaciolascutrine surface sediments that stretches north to south from the edge of the Niagara Escarpment to the Onondaga Escarpment in the south. This plain was all submerged under Lake Warren.

The underlying rocks consist of a succession of Paleozoic beds dipping slightly southward and under Lake Erie. This bedrock consists of dolostone of the Guelph formation, belonging to the Middle Silurian Period of the Paleozoic Era and are approximately 425 million years old.

3. METHOD OF INVESTIGATION

The fieldwork for this project was performed during the period from October 5, 2003 to October 31, 2003 and consisted of drilling and sampling six boreholes, as well as performing dynamic cone penetration test (DCPT). The plan location of the boreholes is shown on Drawing No. 1.

The boreholes were advanced to depths ranging between 11.9 and 32.5 m, using a truck-mounted drilling rig, under the full-time supervision of Geotechnical Engineers from S&P.

Boreholes were advanced using continuous-flight hollow-stem auger (except for Borehole 101 which was drilled using both hollow and solid stem augers). Sampling in the boreholes was effected at frequent intervals of depth by means of continuous Standard Penetration testing and 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.

At Boreholes 101, 101A, 102, 104 and 105, dynamic cone penetration tests (DCPT) were performed. In these tests, a 51 mm diameter, 60-degree apex cone point, screw-attached to the tip of A-size rods, is driven into the ground using the same driving energy as in the SPT method. By recording the number of blows to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of relative density/consistency is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT method and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic effects which in many cases affect the SPT results, especially in the fine-grained granular soils. The DCPT commenced from the bottom of the boreholes and generally terminated when the number of blows to drive the cone/rod assembly 0.3 m exceeded 100.

Piezometers were installed in the Boreholes 101, 102, 104 and 105 to enable us to monitor the groundwater level over a prolonged period of time without interference from surface water. Three impervious seals were placed at each piezometer installation. Groundwater conditions in the boreholes were observed during drilling in the open boreholes and subsequently in the piezometers. The recorded water levels are presented on the appropriate Record of Borehole Sheets.

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 Bennett and Young Surveyors.

4. SUBSURFACE CONDITIONS

In general, beneath the existing embankment fill and fill that was placed during the stripping of the site, the boreholes show the presence of a 0.9 to 2.7 m thick surficial silt deposit. Underlying this is an extensive clayey silt deposit with some silty clay layers. This deposit is 22 to 23 m thick and extends to the surface of the underlying dolostone bedrock at about El. 195 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 Records of Boreholes, drilled at the site by others in 2000, prior to the construction of the bridge, are presented in Appendix D. The individual soil strata encountered in the boreholes drilled for the present investigation are described briefly in the following paragraphs.

4.1 FILL

4.1.1 PAVEMENT FILL

Boreholes 103 through 106, which were drilled from the top of the road embankment (shoulder) contacted a 0.9 to 1.0 m thick granular layer consisting of 19 mm crusher run limestone.

Standard Penetration tests performed in the granular fill in Boreholes 104 and 105, yielded N-values of 8 and 3 blows/0.3 m, respectively, which indicate a very loose to loose condition while in Boreholes 103 and 106, the recorded values are 20 and 41 blows/0.3 m, respectively, indicating a compact to dense condition.

4.1.2 EMBANKMENT FILL

Underlying the granular shoulder fill, a clayey silt embankment fill with some silt zones was contacted. The presence of occasional gravel and topsoil inclusions was noted in the embankment fill. The fill was found to extend to the following depths/elevations.

Borehole 103 – 7.2 m (El.220.5 m)
Borehole 104 – 8.4 m (El.219.7 m)
Borehole 105 – 9.0 m (El.219.0 m)
Borehole 106 – 8.6 m (El.219.0 m)

The colour of the soil is generally yellowish brown with occasional grey silty clay inclusions, as well as occasional dark grey (slightly organic) or black (topsoil) inclusions.

The grain size distribution of selected samples from the fill is presented in an envelope form in Figure B-1 in Appendix B and in Figure B-2. Figure B-1 indicates the following grain-size distribution.

Gravel:	0 – 3 %
Sand:	6 – 14%
Silt:	62-70%
Clay:	21-27%

One sample shows a local condition of 20% gravel, 4% sand, 54% silt and 22% clay size particles.

Atterberg Limits test results performed in the laboratory on selected samples from Boreholes 103, 104 and 106 are given in Figure B-2 in Appendix B, which indicate the following index values:

Liquid Limit:	26-40%
Plastic Limit:	18-20%
Plasticity Index:	7-22%

These results are characteristic of clayey soils of generally low plasticity (i.e. clayey silt to silty clay). Visual examination of the soil samples indicated a cohesive silt like behaviour (i.e. somewhat dilatant) in spite of the fact that relatively high percentage of clay size particles (i.e. 21 to 27%). In other words, the usually observed behaviour of the soil samples resembles that of a cohesive silt rather than a clayey silt to silty clay material.

The measured moisture contents of the samples generally range from 13 to 26%, majority of the values being in the 15 to 20% range.

A Standard Proctor compaction test performed on a combined split-spoon samples plus one Shelby tube sample gave the following results (Figure B-3, Appendix B).

Standard Proctor Maximum Dry Density = 17.2 kN/m^3
Optimum Moisture Content = 18.4%

Bulk unit weights of selected relatively undisturbed samples obtained from split-spoon samples and one thin-walled Shelby tube sample were measured from 20.1 to 21.2 kN/m^3.

Standard Penetration resistance of the fill was measured by means of continuous Standard Penetration testing. Recorded values range from 2 to 33 blows/0.3 m. Plots of the measured values versus depth are given in Figure C-1 and C-1a, Appendix C. The results indicate that in Borehole 104 the recorded values are generally 2 to 9 blows/0.3 m, with three values of between 12 and 20 blows/0.3 m. In Borehole 105, a similar trend is noted to a depth of 5 m, with values increasing to generally in between 14 and 19 blows/0.3 m below 5 m depth, with one higher value of 28.

In Borehole 103, values between 6 and 11 blows/0.3 m were recorded to an approximate depth of 3 m, with values of 22 to 33 below this depth.

In Borehole 106, a value of 15 blows/0.3 m was recorded at surface, all values being between 18 and 33 below this depth.

4.1.3 LOWER FILLS

Underlying the clayey silt embankment fill, the boreholes drilled from the top of the embankment encountered a 0.2 to 0.8 m thick granular fill consisting of basically 19 mm crusher run limestone at depths ranging between 7.2 m (El.220.5 m) and 9.0 m (El.219.0 m).

This material, which was in some cases mixed with some silt, silty clay, clayey silt and topsoil, was probably placed after stripping the site and/or to provide access for construction equipment. In Borehole 104, the granular fill is underlain by a 0.8 m thick layer of silt fill which extends to El. 218.6 m.

In Borehole 101, drilled from near the toe of the existing road embankment, a 0.9 m thick fill layer was contacted. The fill consisted of silt to clayey silt and extended to El. 219.2 m.

4.2 TOPSOIL

In Borehole 102, drilled from near the toe of the embankment, a 0.35 m thick topsoil layer was encountered at surface (El. 219.5 m).

At Boreholes 104 and 105, topsoil and/or organic silty clay/clayey silt were contacted, underlying the crusher run granular fill beneath the embankment. In Borehole 104, the organic soil was found to be 0.2 m thick (extended to El.218.4 m) while at Borehole 105 the thickness of topsoil and organic clayey silt was found to be 0.9 m (extended to El. 217.6 m).

At Boreholes 103 and 106, the topsoil and other organic soils appear to be fully stripped, as basically inorganic natural soils were encountered below the crusher run granular fill placed beneath the embankment.

4.3 SILT

Surficial silt deposits ranging from silt to clayey silt were contacted below fill and/or topsoil in all the boreholes. In Borehole 105, this surficial silt deposit was found to be organic. The thickness of these surficial silt deposits at the borehole locations ranges from 0.9 to 2.7 m, extending to El.218.2 to 216.6 m.

Grain-size distribution of two samples from this unit is given in Figure B-4 in Appendix B.

The recorded N-values in these silt to clayey silt deposits ranged from 6 to in excess of 87 blows/0.3 m which indicate loose/firm to very dense/stiff soils.

4.4 CLAYEY SILT

The predominant overburden underlying the site is cohesive clayey silt with some silty clay layers. In many cases, owing to the presence of occasional embedded coarse sand (grits) and gravel, the material resembles a glacial till. The presence of very occasional thin silt seams/lenses was also noted in some of the boreholes.

In the deep boreholes, the deposit generally extends down to bedrock except in Borehole 102, where a 0.6 m thick granular silty sand till was contacted at 23.5 m depth

(El. 196.0 m) immediately overlying inferred bedrock at 24.1 m or El. 195.4 m. In the remaining boreholes, the clayey deposit extends to inferred bedrock between El. 195.4 and 194.9 m.

Boreholes 103 and 106 were terminated in the clayey silt deposit.

The results of particle size distribution analyses on samples from the deposit are given in an envelope form in Figure B-5 in Appendix B. The following particle size distribution is indicated.

Gravel:	1 – 8%
Sand:	3 – 5%
Silt:	60-63%
Clay:	29-35%

Atterberg Limits tests performed in the laboratory on selected samples gave the following index values (Figure B-6, Appendix B).

Liquid Limit:	26-39%
Plastic Limit:	17-21%
Plasticity Index:	8-19%

The measured natural moisture contents generally range from 16 to 23% (with some higher values but primarily near the bottom of the deposit where the soil is somewhat more clayey and is of relatively higher plasticity).

The measured index values are characteristic of clayey soils of low plasticity. In general, the measured natural moisture contents are closer to the measured plastic limits rather than the liquid limits and this indicates some degree of pre-consolidation. As mentioned before, higher Plasticity Indices (PI) of 17 and 19 were obtained from samples located near the bottom of the stratum mantling the bedrock, while values of 13 and 14 were obtained from near the surface. The remaining values of PI (between 8 and 12) were generally recorded on samples from the main body of the deposit in the middle (i.e. below the upper 2± m and above the bottom 4± m) of this 22 to 23 m thick deposit.

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 reported above. Another unusual feature was that the samples of the material obtained from the boreholes showed a 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 7 to 58 blows/0.3 m. These results indicate firm to hard consistency, but generally very stiff. A plot of N-values recorded in Boreholes 101, 102, 104 and 105 versus elevation is given in Figure C-2, Appendix C. The plot shows a trend for an increase in N-values between about El. 205 and 200 m. For example, between El. 214 and 205 m the recorded N-values are generally between 10 and 20 but below this elevation and between El. 205 and 200 m, they are generally between 12 and 45 with highest values recorded in all four boreholes at about El. 202 m (N-values between 20 and 45). Below about El. 199 m, the N-values gradually drop. N-values of 7 were recorded in Boreholes 104 and 105 at about El. 199 m with N-values of between 11 and 16 at about El. 196 m. This drop in N-values towards the bottom of the stratum seems to coincide with increased plasticity index values, as mentioned before.

In general, somewhat higher N-values were recorded in Boreholes 101 and 102 in comparison with Boreholes 104 and 105 which were drilled from the top of Glancaster Road embankment.

Field vane tests gave undrained in-situ shear strengths ranging from 96 kPa to in excess of 240 kPa. Three quick triaxial compression tests performed in the laboratory yielded undrained shear strength values of 84, 113 and 151 kPa (Figures B-7, B-8 and B-9, Appendix B). A plot of the measured undrained shear strengths (c) vs. elevation is given in Figure C-3 of Appendix C. In general, the measured field vane test results are in excess of 150 kPa.

Seven one-dimensional consolidation (oedometer) tests were performed on selected thin-walled open drive Shelby tube (TW) samples. The results are given in Figure B-10 through B-16 in Appendix B). These results show probable pre-consolidation pressures of the order of 100 kPa in excess of the existing overburden pressures.

The measured bulk unit weights on relatively undisturbed samples range from 18.8 to 21.7 kN/m^3 with an average value of 20.7 kN/m^3.

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

4.5 INFERRED BEDROCK

From refusal to augering and/or dynamic cone penetration tests (DCPT) in Boreholes 101A, 102, 104 and 105, the surface of the bedrock was inferred at Elevations ranging from 195.4 to 194.9 m.

4.6 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed while drilling and at the completion of each borehole. In addition, piezometers were installed in four of the boreholes to enable us to monitor prolonged groundwater level measurements without interference from surface water. Three impervious seals were placed at each piezometer installation. The observations and recorded values are shown on the individual Record of Borehole Sheets. The recorded water levels in the piezometer installations are summarized in the following table, which also includes the lowest impervious seal elevations.

Table 4.8.1
Measured Groundwater Levels In Piezometers

Borehole No.	Piezometer Tip Elevation (m)	Bottom of Borehole Elevation (m)	Lowest Impervious Seal Elevation (m)	Measured Water Level Elevation (m)	Elapsed Time After Installation
101	207.3	203.9	209.0	216.7	12 days
102	195.4*	195.4*	197.5	216.0	11 days
104	205.3	195.6	209.8	218.2	22 days
105	217.3	196.0	221.8	218.1	4 days
6**	213.2	212.8	215.5	218.3	14 days
9**	195.2***	192.4	211.0	215.0	14 days

* In the silty sand till immediately above inferred bedrock surface
**Piezometers installed by another firm in October, 2000.
***Bedrock surface

The results indicate that stabilized water levels are probably between about El. 218 and 217 m. These coincide with the observed change in the colour of the soil from brown to grey (observed at between El. 218.2 and 217.5 m in Boreholes 101, 102, 103 and 104 and at about El. 215 m in Borehole 105).

The recorded values indicate that there is an upward gradient, and that interface of the bedrock and the clayey silt to silty clay deposit mantling the bedrock, as well as the clayey silt deposit are under excess hydrostatic pressure. For example, the piezometer in Borehole 102 was installed within the overburden at the surface of the inferred bedrock at El. 195.4 m with an impervious seal at about 2 m above that elevation. After eleven days, the water level was recorded at El. 216 m (i.e. about 19 m above the seal elevation).

It is also of interest that water levels in piezometers installed in Boreholes 104 and 105 drilled within the footprint of the existing road embankment show higher water level elevations in comparison with Boreholes 101 and 102 drilled beyond the footprint of the embankment.

It should be pointed out that the groundwater levels can be subject to seasonal fluctuations and fluctuations in response to major weather events.

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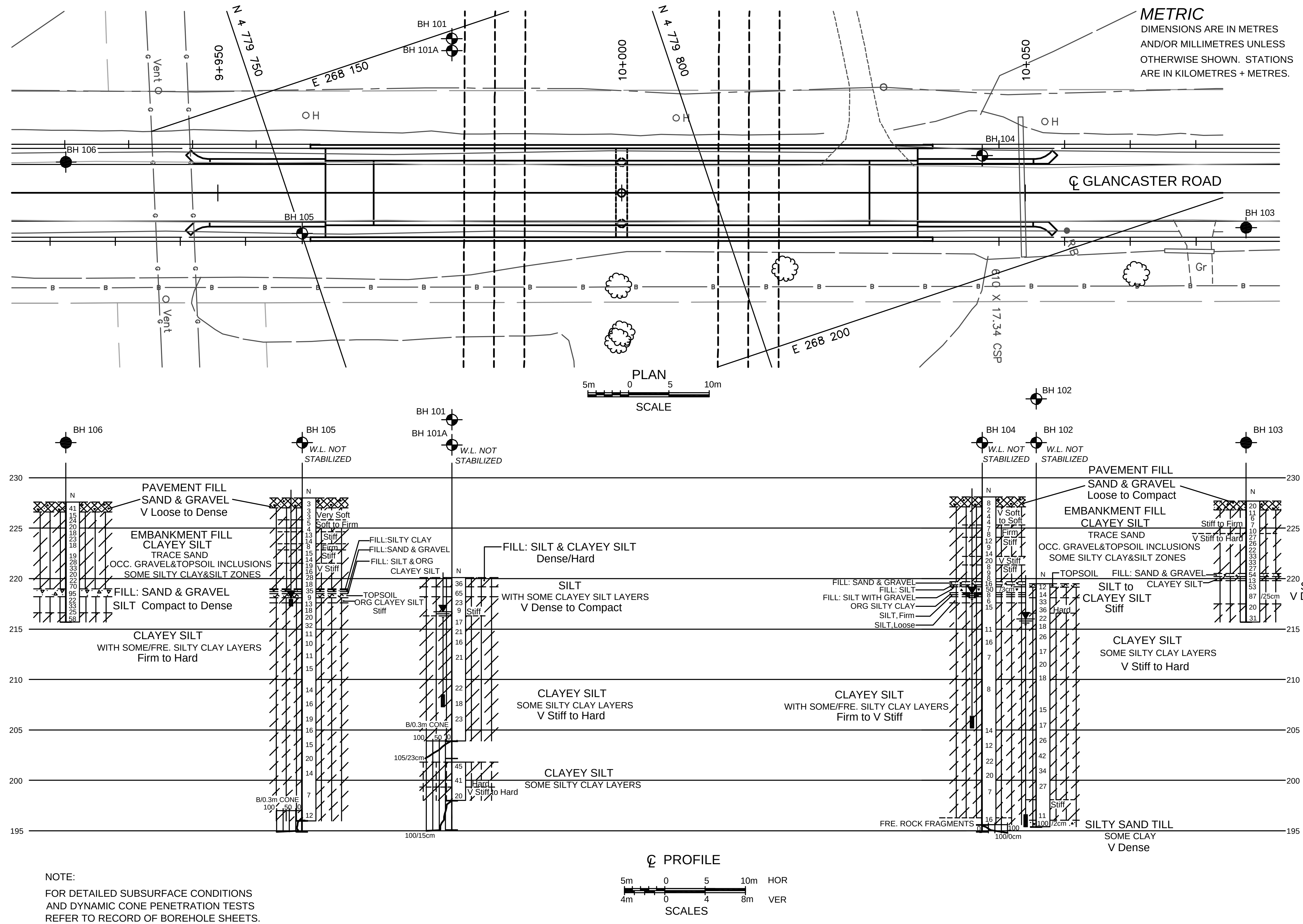
Drawing



NOTE:

FOR DETAILED SUBSURFACE CONDITIONS AND DYNAMIC CONE PENETRATION TESTS REFER TO RECORD OF BOREHOLE SHEETS.

METRIC

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

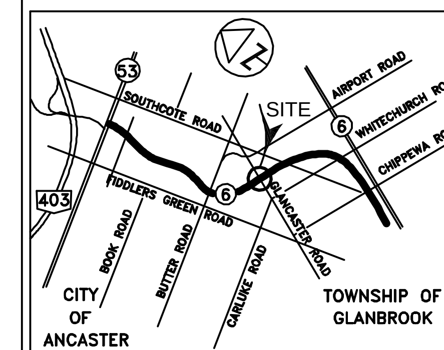
CONT No.

WP: 604-00-01

HWY 6
GLANCASTER ROAD UNDERPASS
BOREHOLE LOCATION & SOIL STRATA

SHEET

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S.

LEGEND

- Bore Hole
- Bore Hole & Cone
- N — Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation Oct. , 2003
- Water Level in Piezometer
- Piezometer

No.	ELEV.	CO-ORDINATES NORTH	EAST
BH 101	220.1	4 779 775.4	268 151.0
BH 101A	220.1	4 779 774.9	268 152.4
BH 102	219.5	4 779 825.7	268 221.9
BH 103	227.7	4 779 861.2	268 204.5
BH 104	228.1	4 779 833.0	268 185.6
BH 105	228.0	4 779 750.1	268 167.9
BH 106	227.6	4 779 725.2	268 150.2

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.

			DIST CENTRAL
SUBM'D ZO	CHECKED RM	DATE Nov,2003	SITE 36-495
DRAWN JZ	CHECKED	APPROVED	DWG 1

Appendix A

Record of Borehole Sheets

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 101

METRIC

WP: 604-00-01	LOCATION: HWY 6 GLANCASTER ROAD- Coords: N 4 779 775.4 ; E 268 151.0		ORIGINATED BY: R.A.
DIST: Central	HWY: 6	BOREHOLE TYPE: Hollow Stem Augers to 12.5 m, Solid Stem Augers below & D.C.P.T.	COMPILED BY: J.Z.
DATUM: Geodetic	DATE: 10/19/2003		CHECKED BY: Z.O.

SOIL PROFILE / SAMPLES / GROUND WATER CONDITIONS / ELEVATION SCALE / DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 40, 80, 120, 160, 200) / PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (10, 20, 30) / UNIT WEIGHT γ kN/m³ / REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
220.1	Ground Surface						
0.0	**FILL: SILT AND CLAYEY SILT** brown with organic stained dark brown layers dense/hard	1	SS	36		20.8	
219.2 / 0.9	**SILT** with some clayey silt layers very dense to compact brown, damp	2	SS	65		21.5	
		3	SS	23			
218.2 / 1.9	**CLAYEY SILT** some silty clay layers grey, very stiff to hard	4	SS	9	+2	19.5	
	stiff	5	TW	PH	>100	21.4	3 4 62 31 Consolidation Test
		6	SS	17	>240	21.3	
		7	SS	21			
		8	TW	PH		20.8	8 3 60 29
		9	SS	16	>240		
		10	SS	21	>240	21.2	
		11	TW	PH	>240	20.2	
	occ. thin wet silt seams	12	SS	22			
		13	SS	18			Plug lost in the hole. Hole abandoned. Hole moved 1.5m and redrilled.
		14	SS	23	>240	20.6	

Elevation scale: 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208, 207, 206

Continued Next Page

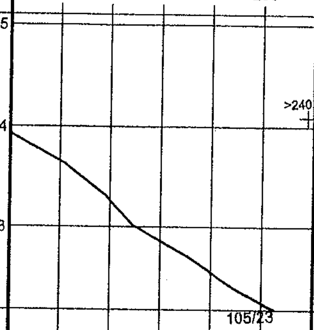
$+^3, \times^3$: Numbers refer to Sensitivity   15 ⊕ 5 (20, 10) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 101

METRIC

WP 604-00-01	LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 775.4 ; E 268 151.0		ORIGINATED BY R.A.
DIST Central	HWY 6	BOREHOLE TYPE Hollow Stem Augers to 12.5 m, Solid Stem Augers below & D.C.P.T.	COMPILED BY J.Z.
DATUM Geodetic	DATE 10/19/2003		CHECKED BY Z.O.

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 40 80 120 160 200)	WATER CONTENT (%) (PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; 10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
205.1 / 15.0	**CLAYEY SILT** frequent silty clay layers grey, very stiff to hard		15	TW	PH		205	>240		21.0	2 5 63 30
203.9 / 16.2	End of Borehole. Augers snapped; borehole abandoned and redrilled 1.8 m away. See log of Borehole 101A below 18 m. Dynamic Cone Penetration Test (D.C.P.T.) performed from 16.2 m to 17.9 m.						204, 203				
202.2 / 17.9	End of Dynamic Cone Penetration Test. Piezometer installed to 12.8 m. Water level on Oct. 19, 2003 - Dry; Oct. 21, 2003 - 12.3 m (El. 207.8 m); Oct. 27, 2003 - 4.3 m (El. 215.8 m); Oct. 28, 2003 - 4.1 m (El. 216.0 m); Oct. 31, 2003 - 3.4 m (El. 216.7 m)							105/23			



$+^3, \times^3$: Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 101 A

METRIC

WP 604-00-01	LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 774.9 ; E 268 152.4	ORIGINATED BY Y.L.
DIST Central HWY 6	BOREHOLE TYPE Hollow Stem Augers & D.C.P.T.	COMPILED BY J.Z.
DATUM Geodetic	DATE 10/21/2003	CHECKED BY Z.O.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (40 80 120 160 200)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
220.1 0.0	Ground Surface						220				
							219				
							218				
							217				
							216				
	Augered to 18.3 m without sampling. Refer to Borehole 101 for subsurface information to 18.3 m depth.						215				
							214				
							213				
							212				
							211				
							210				
							209				
							208				
							207				
							206				

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 101 A

METRIC

WP 604-00-01
LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 774.9 ; E 268 152.4
ORIGINATED BY Y.L.

DIST Central HWY 6
BOREHOLE TYPE Hollow Stem Augers & D.C.P.T.
COMPILED BY J.Z.

DATUM Geodetic
DATE 10/21/2003
CHECKED BY Z.O.

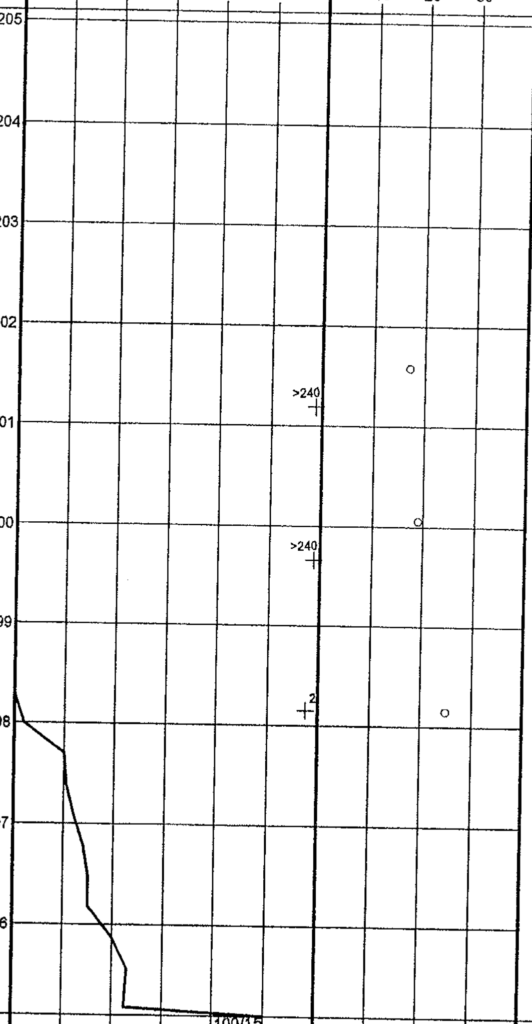
ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
205.1 / 15.0	Augered to 18.3 m without sampling. Refer to Borehole 101 for subsurface information to 18.3 m depth.						205, 204, 203, 202		
201.8 / 18.3	**CLAYEY SILT** some silty clay layers grey		1	SS	45		201		
	hard		2	SS	41		200		
	very stiff to hard		3	SS	20		199		
198.0 / 22.1	End of Borehole. Dynamic Cone Penetration Test (D.C.P.T.) performed from 22.1 m to 25.0 m.						198, 197, 196		
195.1 / 25.0	End of Dynamic Cone Penetration Test.								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 40 80 120 160 200

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30



+ 3, × 3: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 102

METRIC

WP 604-00-01
LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 825.7 ; E 268 221.9
ORIGINATED BY Y.L.

DIST Central HWY 6
BOREHOLE TYPE Hollow Stem Augers & D.C.P.T.
COMPILED BY J.Z.

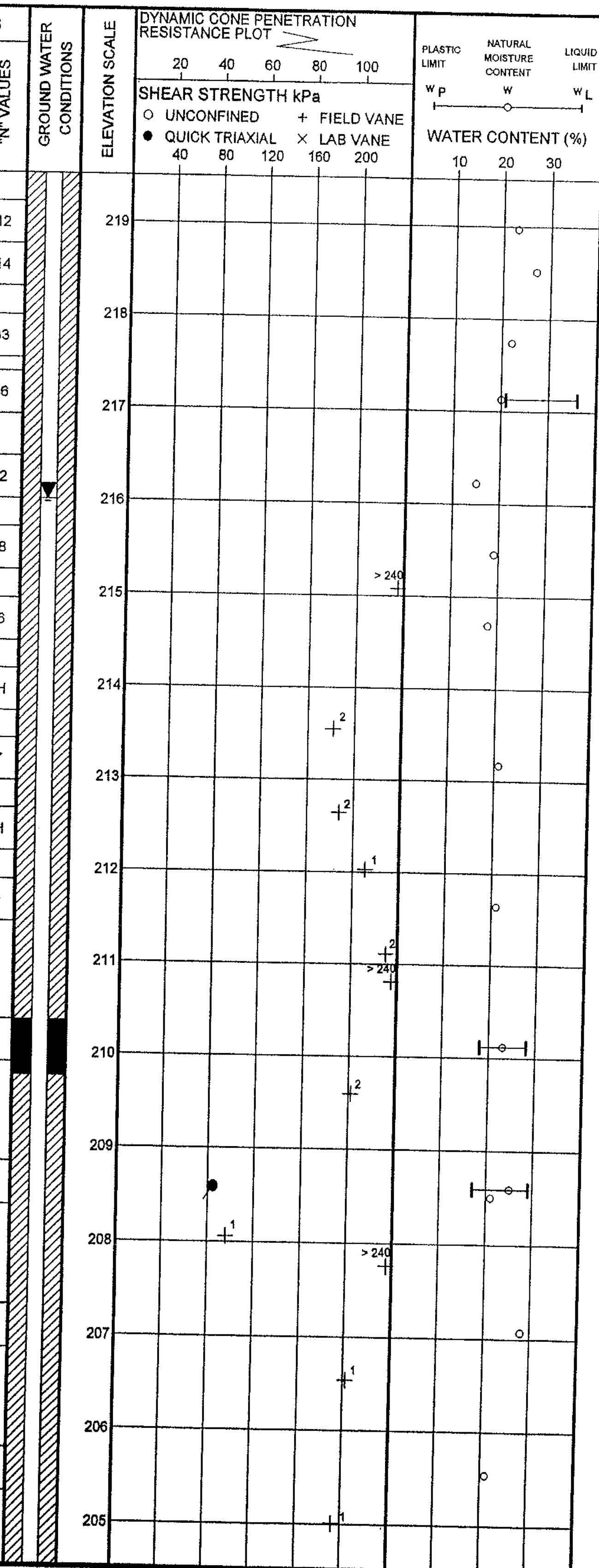
DATUM Geodetic
DATE 10/20/2003
CHECKED BY Z.O.

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
219.5	Ground Surface
0.0	0.35 m **TOPSOIL**
	SILT to CLAYEY SILT traces of organics to 0.6 m brown, stiff
218.1 1.4	**CLAYEY SILT** some silty clay layers grey, very stiff (brown / hard)

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	12
2	SS	14
3	SS	33
4	SS	36
5	SS	22
6	SS	18
7	SS	26
8	TW	PH
9	SS	17
10	TW	PH
11	SS	20
12	SS	18
13	TW	PH
14	SS	15
15	SS	17



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
19.9 20.4	Consolidation Test

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 102

METRIC

WP 604-00-01
LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 825.7 ; E 268 221.9
ORIGINATED BY Y.L.
DIST Central HWY 6
BOREHOLE TYPE Hollow Stem Augers & D.C.P.T.
COMPILED BY J.Z.
DATUM Geodetic
DATE 10/20/2003
CHECKED BY Z.O.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS
204.5 / 15.0	CLAYEY SILT some silty clay layers very stiff to hard	16	SS	26	204	
		17	SS	42	203, 202	
		18	SS	34	201, 200	
		19	SS	27	199	
		20	TW	PH	198, 197	
	stiff	21	SS	11		
196.0 / 23.5	SILTY SAND TILL some clay grey, moist, very dense				196	Unable to push vane to 23.5 m
195.4 / 24.1	End of Borehole.	22	SS	100/2		

Shear strength (field vane): > 240 (at approx. El. 203.5, 202.0, 200.5, 199.0); 2 (sensitivity, at approx. El. 197.5).

Dynamic Cone Penetration Test (D.C.P.T.) performed from bottom of the hole. Rod bouncing, no further progress.

Piezometer installed to 24.1 m.
Water level on
Oct. 21, 2003 - 19.2 m (El. 200.3 m)
Oct. 27, 2003 - 3.9 m (El. 215.6 m)
Oct. 31, 2003 - 3.5 m (El. 216.0 m)

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (40, 80, 120, 160, 200): O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 103

METRIC

WP 604-00-01 | LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 861.2 ; E 268 204.5 | ORIGINATED BY R.A.

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

DATUM Geodetic | DATE 10/5/2003 | CHECKED BY Z.O.

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
227.7	Ground Surface
0.0	**PAVEMENT FILL: SAND AND GRAVEL** grey, compact
226.8 / 0.9	**EMBANKMENT FILL: CLAYEY SILT** trace sand, occasional gravel and topsoil inclusions, some silty clay and silt zones, brown; stiff to firm / very stiff to hard
220.5 / 7.2	**FILL: SAND & GRAVEL** brown, wet, very dense
220.2 / 7.5	**CLAYEY SILT** traces of organics, brown and dark brown, stiff
219.8 / 7.9	**SILT** brown, very dense, wet
217.5 / 10.2	**CLAYEY SILT** some silty clay layers, grey, very stiff
215.7 / 12.0	End of Borehole. Borehole dry (not stabilized) and open to the full depth on completion.

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
1	AS						
2	SS	20					
3	SS	11					
4	SS	6					
5	SS	7	20.1	1	7	67	25
6	SS	10					
7	SS	27					
8	SS	26					
9	SS	22		0	8	68	24
10	SS	33	20.6				
11	SS	33					
12	SS	27	20.3				
13	SS	54	20.4				
14	SS	13	19.5				
15	SS	53		0	8	82	10
16	SS	87/25					
17	SS	20	19.2				
18	TW	PH					
19	SS	31	20.7				

GROUND WATER CONDITIONS

ELEVATION SCALE: 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 40, 80, 120, 160, 200

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10, 20, 30

REMARKS & GRAIN SIZE DISTRIBUTION (%)

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 104

METRIC

WP	604-00-01	LOCATION	HWY 6 GLANCASTER ROAD- Coords: N 4 779 833.0 ; E 268 185.6
DIST	Central HWY 6	BOREHOLE TYPE	Hollow Stem Augers & D.C.P.T.
DATUM	Geodetic	DATE	10/9/2003

ORIGINATED BY R.A.
COMPILED BY J.Z.
CHECKED BY Z.O.

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
228.1	Ground Surface
0.0	**PAVEMENT FILL: SAND AND GRAVEL** grey, loose, damp
227.1 / 1.0	**EMBANKMENT FILL: CLAYEY SILT** trace sand, occasional gravel and topsoil inclusions, some silt and silty clay zones, brown; very soft to soft / firm / stiff / very stiff / stiff
219.7 / 8.4	**FILL: SAND & GRAVEL** some clay&organics, grey
8.7	**FILL: SILT** brown, wet
8.9	**FILL: SILT with gravel** some organics, brown & dark brown, wet
9.6 / 9.7	**ORGANIC SILTY CLAY** dark grey*
218.2 / 9.9	**SILT** trace clay, some organic content, grey to dark grey, firm; **SILT** brown, loose, wet
217.6 / 10.5	**CLAYEY SILT** with some silty clay layers, grey, stiff to very stiff; firm; 0.15 m thick wet silt seam @ 11.0 m

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	AS			
2	SS	8		
3	SS	2		
4	SS	4		
5	SS	4		2 6 70 22
6	SS	7		* unable to push sample tube
7	TW*	PH		
8	SS	8		
9	SS	12		2 14 62 22
10	SS	9		
11	SS	14		
12	SS	20		
13	SS	8		
14	SS	9	20.4	
15	SS	8	20.4	3 10 66 21
16	SS	16		
17	SS	50/3		
18	SS	8		* probably original topsoil
19	SS	6		
20	SS	15		
21	TW	PH	21.1	1 4 60 35 Consolidation Test
22	SS	11		
23	SS	16		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 40 80 120 160 200
Field vane: > 240, +1, > 240, +2, +1
WATER CONTENT (%): PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; 10 20 30
ELEVATION SCALE: 228 227 226 225 224 223 222 221 220 219 218 217 216 215 214

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity; 15-○-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 104

METRIC

WP 604-00-01 | LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 833.0 ; E 268 185.6 | ORIGINATED BY R.A.

DIST Central HWY 6 | BOREHOLE TYPE Hollow Stem Augers & D.C.P.T. | COMPILED BY J.Z.

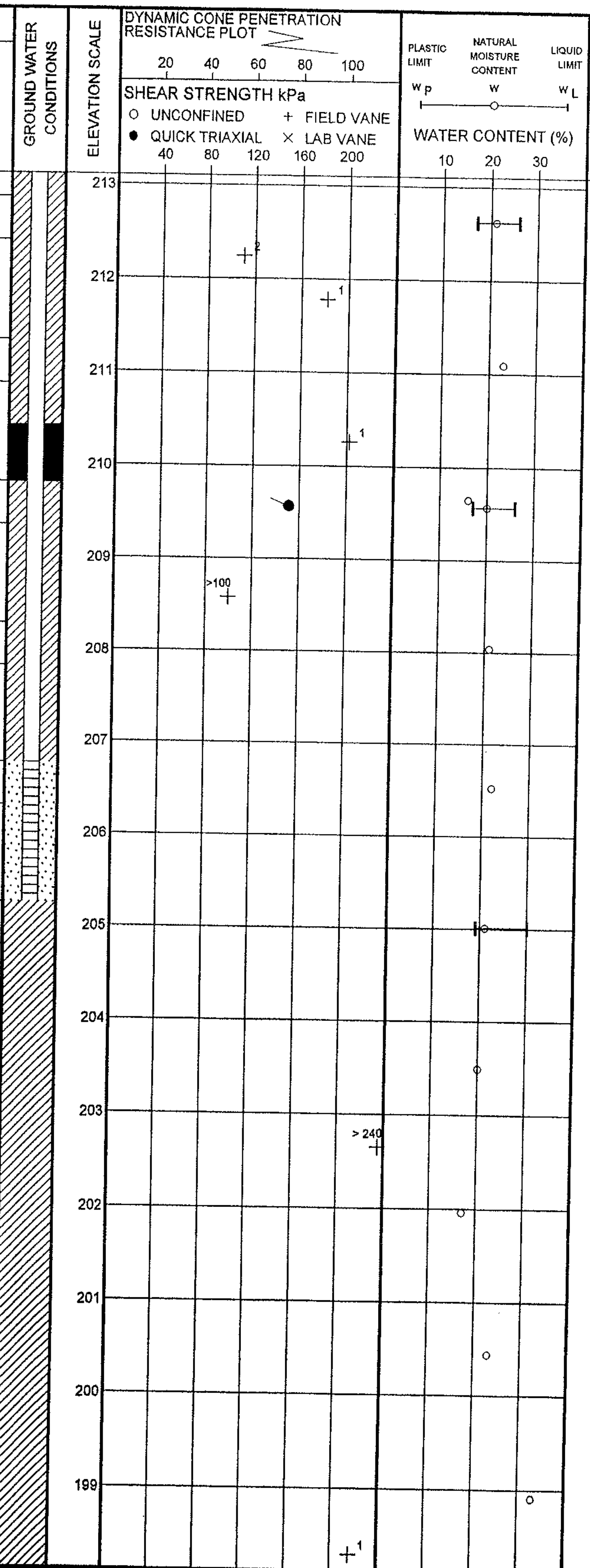
DATUM Geodetic | DATE 10/9/2003 | CHECKED BY Z.O.

SOIL PROFILE: ELEV DEPTH 213.1 / 15.0

CLAYEY SILT
with some silty clay layers
grey, firm to very stiff

0.1 m thick silt seam/lense @ 21.0 m

NUMBER	TYPE	"N" VALUES	REMARKS
24	SS	7	
25	TW*	PH	* no recovery, split spoon sample taken
26	TW	PH	Consolidation Test (Unit weight 21.7)
27	SS	*	* No N-value recorded.
28	SS	8	
29	SS	14	
30	SS	12	
31	SS	22	
32	SS	20	
33	SS	7	



UNIT WEIGHT γ kN/m³ | GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No 104

METRIC

WP 604-00-01 — LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 833.0 ; E 268 185.6 — ORIGINATED BY R.A.

DIST Central HWY 6 — BOREHOLE TYPE Hollow Stem Augers & D.C.P.T. — COMPILED BY J.Z.

DATUM Geodetic — DATE 10/9/2003 — CHECKED BY Z.O.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
198.1 30.0	**CLAYEY SILT** with frequent silty clay layers grey, very stiff	34	TW	PH	198 / 197	19.8 / 18.8	Consolidation Test
	frequent rock fragments	35	SS	16	196		
195.6 32.5	End of Borehole. Dynamic Cone Penetration Test (D.C.P.T.) performed from 32.5 m to 33.2 m.						
194.9 33.2	End of Dynamic Cone Penetration Test.				195		

DCPT: 100/0

Shear strength (quick triaxial): 16

Rods bouncing at 33.2 m, probably on bedrock.

Piezometer installed to 22.9 m.

Water level on:
Oct, 27, 2003 - 9.4 m (El. 218.7 m)
Oct, 28, 2003 - 9.7 m (El. 218.4 m)
Oct, 31, 2003 - 9.9 m (El. 218.2 m)

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 40 80 120 160 200

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30

GR SA SI CL

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

RECORD OF BOREHOLE No 105

METRIC

WP 604-00-01 | LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 750.1 ; E 268 167.9 | ORIGINATED BY Y.L.

DIST Central HWY 6 | BOREHOLE TYPE Hollow Stem Augers & D.C.P.T. | COMPILED BY J.Z.

DATUM Geodetic | DATE 10/27/2003 | CHECKED BY Z.O.

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 40 80 120 160 200 | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
228.0	Ground Surface			
0.0	**PAVEMENT FILL: SAND AND GRAVEL** grey, very loose, wet	1	SS	3
227.0 / 1.0	**EMBANKMENT FILL: CLAYEY SILT** trace sand, occasional gravel and topsoil inclusions, some silty clay and silt zones, very soft to very stiff, brown	2	SS	3
	very soft	3	SS	3
	soft to firm	4	SS	5
		5	SS	4
	stiff	6	SS	13
		7	SS	14
	firm	8	SS	8
	stiff	9	SS	15
		10	SS	14
	very stiff	11	SS	19
		12	SS	16
		13	SS	28
		14	SS	18
219.0 / 9.0	**FILL: SILTY CLAY**, dark grey			
9.1	**FILL: SAND & GRAVEL** grey, dense, wet	15	SS	35
9.3	**FILL: SILT & ORGANIC CLAYEY SILT**			
9.5	**TOPSOIL**	16	SS	9
218.1 / 9.9	**ORGANIC CLAYEY SILT** black to dark grey, stiff	17	SS	13
217.6 / 10.4	**CLAYEY SILT** with some silty clay layers	18	SS	18
	brown, very stiff	19	SS	20
	occasional silt seams@12.3 m	20	SS	32
	grey, stiff	21	SS	11
		22	TW	PH
		23	SS	10

Elevation scale: 228, 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213

Shear strength: > 240 (at approx. elev. 216.8); > 240 (at approx. elev. 216.0); $+^2$ (at approx. elev. 214.5); $+^1$ (at approx. elev. 213.7)

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity   15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 105

METRIC

WP 604-00-01 LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 750.1 ; E 268 167.9 ORIGINATED BY Y.L.

DIST Central HWY 6 BOREHOLE TYPE Hollow Stem Augers & D.C.P.T. COMPILED BY J.Z.

DATUM Geodetic DATE 10/27/2003 CHECKED BY Z.O.

SOIL PROFILE: ELEV / DEPTH; DESCRIPTION; STRAT PLOT

SAMPLES: NUMBER; TYPE; "N" VALUES

GROUND WATER CONDITIONS

ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE; 40 80 120 160 200

PLASTIC LIMIT w_P; NATURAL MOISTURE CONTENT w; LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

ELEV DEPTH	DESCRIPTION
213.0 / 15.0	**CLAYEY SILT** some silty clay layers grey, stiff to very stiff
	0.05 m thick silt seam @27.5 m
	firm to stiff

NUMBER	TYPE	"N" VALUES
24	SS	11
25	SS	15
26	SS	14
27	SS	16
28	SS	19
29	SS	16
30	SS	15
31	SS	20
32	SS	14
33	SS	7

Elevation scale: 213, 212, 211, 210, 209, 208, 207, 206, 205, 204, 203, 202, 201, 200, 199, 198

Field vane readings: $+^1$, $+^1$, > 240, $+^1$, $+^2$, $+^1$, $+^1$, > 240, > 240, $+^2$, $+^1$

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No 105

METRIC

WP 604-00-01
LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 750.1 ; E 268 167.9
ORIGINATED BY Y.L.

DIST Central HWY 6
BOREHOLE TYPE Hollow Stem Augers & D.C.P.T.
COMPILED BY J.Z.

DATUM Geodetic
DATE 10/27/2003
CHECKED BY Z.O.

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 40 80 120 160 200)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
198.0 / 30.0	**CLAYEY SILT** with frequent silty clay layers grey, stiff to very stiff		34	TW	PH		198				
							197	$+^1$			
			35	SS	12				○		
196.0 / 32.0	End of Borehole. Dynamic Cone Penetration Test (D.C.P.T.) performed from 32.0 m to 33.1 m.						196				
194.9 / 33.1	End of Dynamic Cone Penetration Test. Dynamic Cone Penetration Test (D.C.P.T.) performed from bottom of the hole. Rod bouncing at 33.1 m, no further penetration. Borehole dry (not stabilized) and open to the full depth on completion. Piezometer installed to 10.7 m. Water level on: Oct, 31, 2003 - 9.9 m (El. 218.1 m)						195				

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 106

METRIC

WP 604-00-01 LOCATION HWY 6 GLANCASTER ROAD- Coords: N 4 779 725.2 ; E 268 150.2 ORIGINATED BY R.A.
DIST Central HWY 6 BOREHOLE TYPE Hollow Stem Augers COMPILED BY J.Z.
DATUM Geodetic DATE 10/5/2003 CHECKED BY Z.O.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
227.6	Ground Surface								
0.0	**PAVEMENT FILL: SAND AND GRAVEL** grey, dense	1	AS						
		2	SS	41					
226.6 / 1.0	**EMBANKMENT FILL: CLAYEY SILT** trace sand, occasional gravel and topsoil inclusions, some silty clay and silt zones, brown, very stiff	3	SS	15					
	sand and gravel layer or pocket @ 1.8 m	4	SS	24					
		5	SS	20					
		6	SS	18					
		7	SS	23					
		8	SS	18		1	10	62	27
		9	TW	PH	20.4				
		10	SS	19	20.8				
		11	SS	28	20.2				
		12	SS	33	21.2				
		13	SS	20	21.0	20	4	54	22
		14	SS	22	20.1				
219.0 / 8.6	**FILL: SAND & GRAVEL** some silt, traces of clayey silt pockets, trace asphalt pieces, grey, damp to 9 m, moist below	15	SS	70	20.8				
		16	* SS	95					
218.2 / 9.4	**SILT** traces of clay, compact to dense, brown, wet, dilatant	17	SS	22	19.7				
		18	SS	33	20.4	0	2	87	11
216.6 / 11.0	**CLAYEY SILT** with some silty clay layers, brown, hard	19	SS	25	19.3				
		20	SS	58	21.0				
215.7 / 11.9	End of Borehole. Borehole dry (not stabilized) and open to the full depth on completion.								

REMARKS: * faint gasoline odour

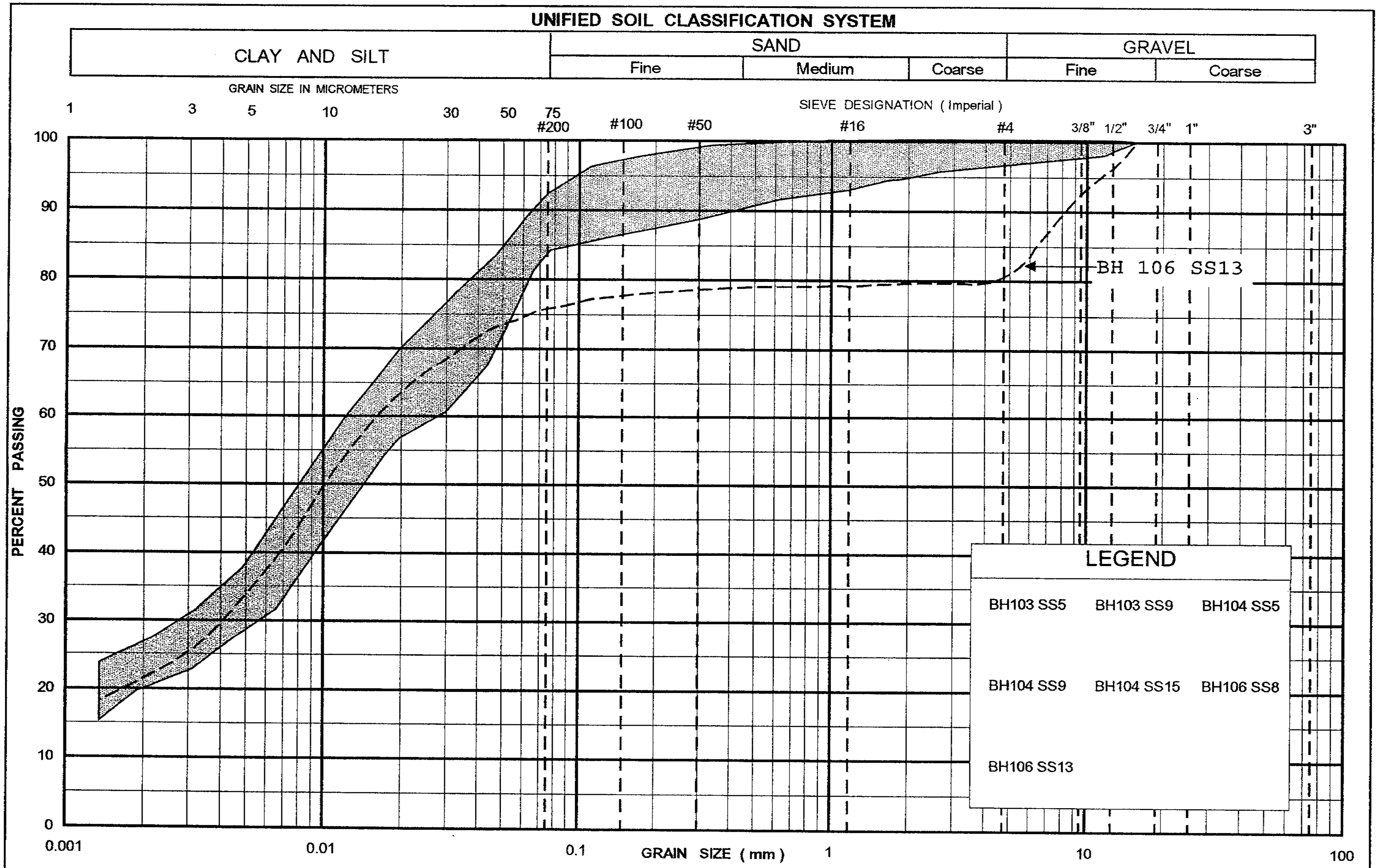
Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 40 80 120 160 200; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; REMARKS & GRAIN SIZE DISTRIBUTION (%)

+³, ×³: Numbers refer to Sensitivity

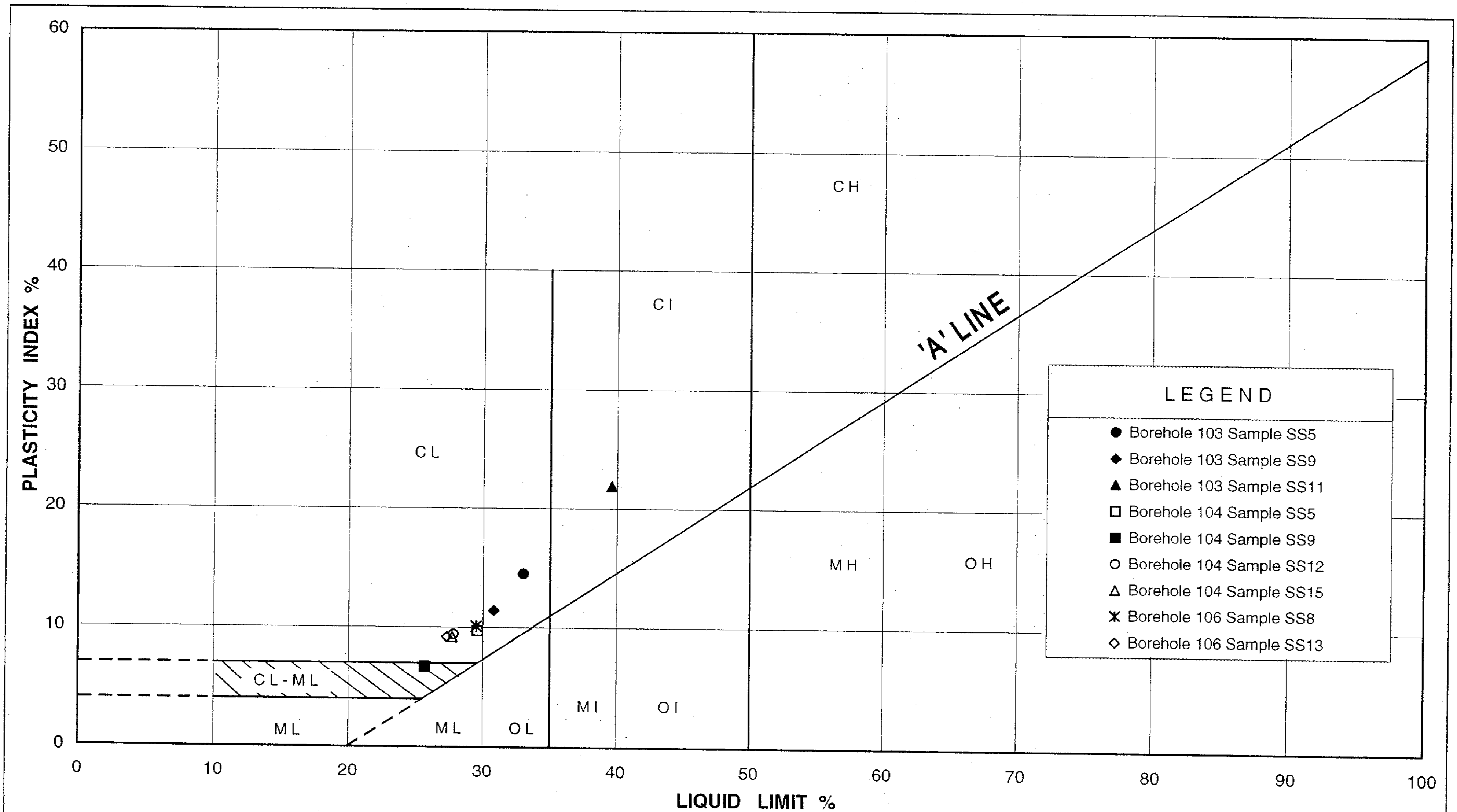
20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results



SHAHEEN & PEAKER LIMITED	**GRAIN SIZE DISTRIBUTION** EMBANKMENT FILL: Clayey Silt	FIGURE No. B-1 REF. No. SPT 1105 W.P. 604-00-01



SHAHEEN & PEAKER LIMITED	PLASTICITY CHART EMBANKMENT FILL: Clayey Silt	FIG No B-2 W.P. 604-00-01 SPT 1105

SHAHEEN & PEAKER LIMITED
Consulting Geo-Environmental and Construction Materials Engineers

PROCTOR TEST RESULT

Project Name:	Highway 6 Underpass	
Project No. :	SPT1105	
Material Supplier:	N / A	
Sample Location:	Composite Sample (BH104 & BH105)	
Sampled By :	Wolfe	
Date Sampled :	October 28, 2003	
Laboratory No:	3756	
Proctor Method:	Standard	Method C
Sample Description:	Clayey Silt	
Maximum Dry Density :	1754 (kg/m^3)	
Optimum Moisture Content:	18.4 %	

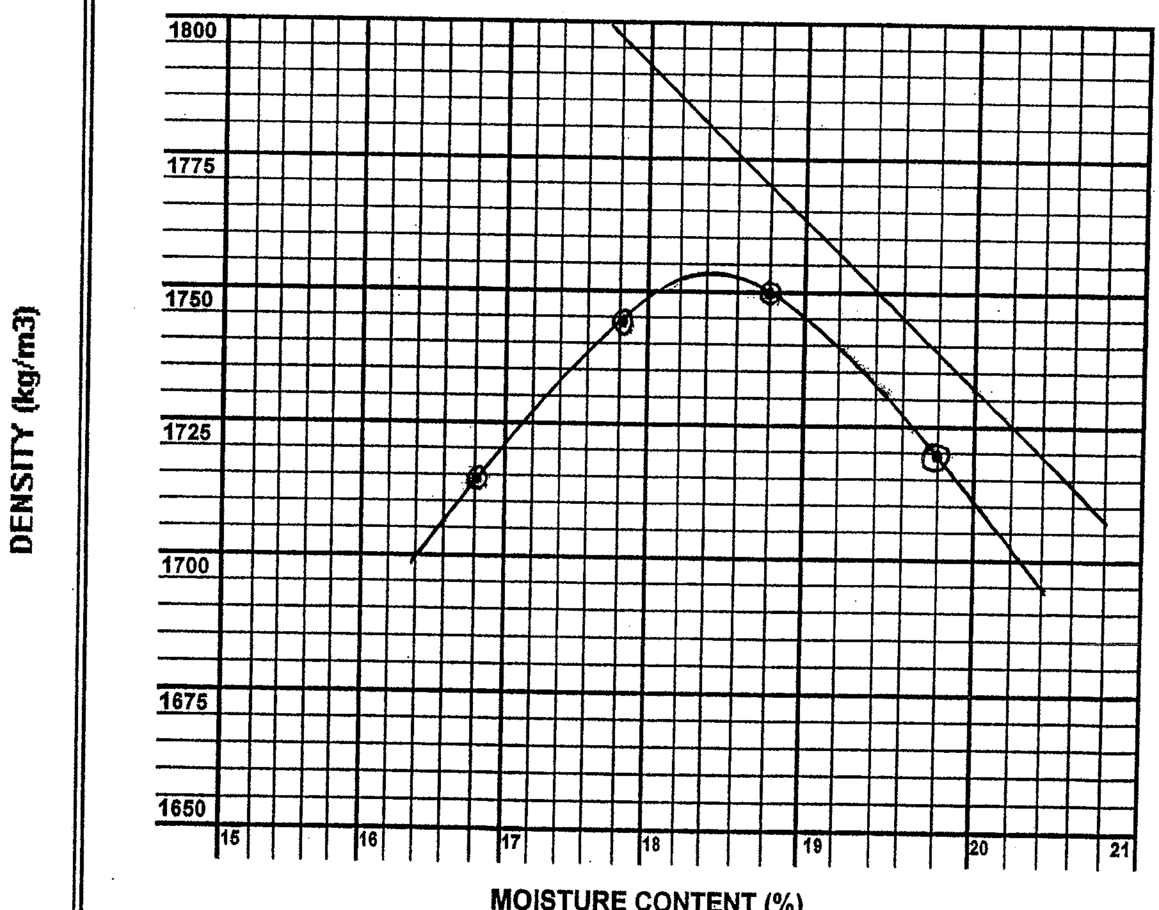
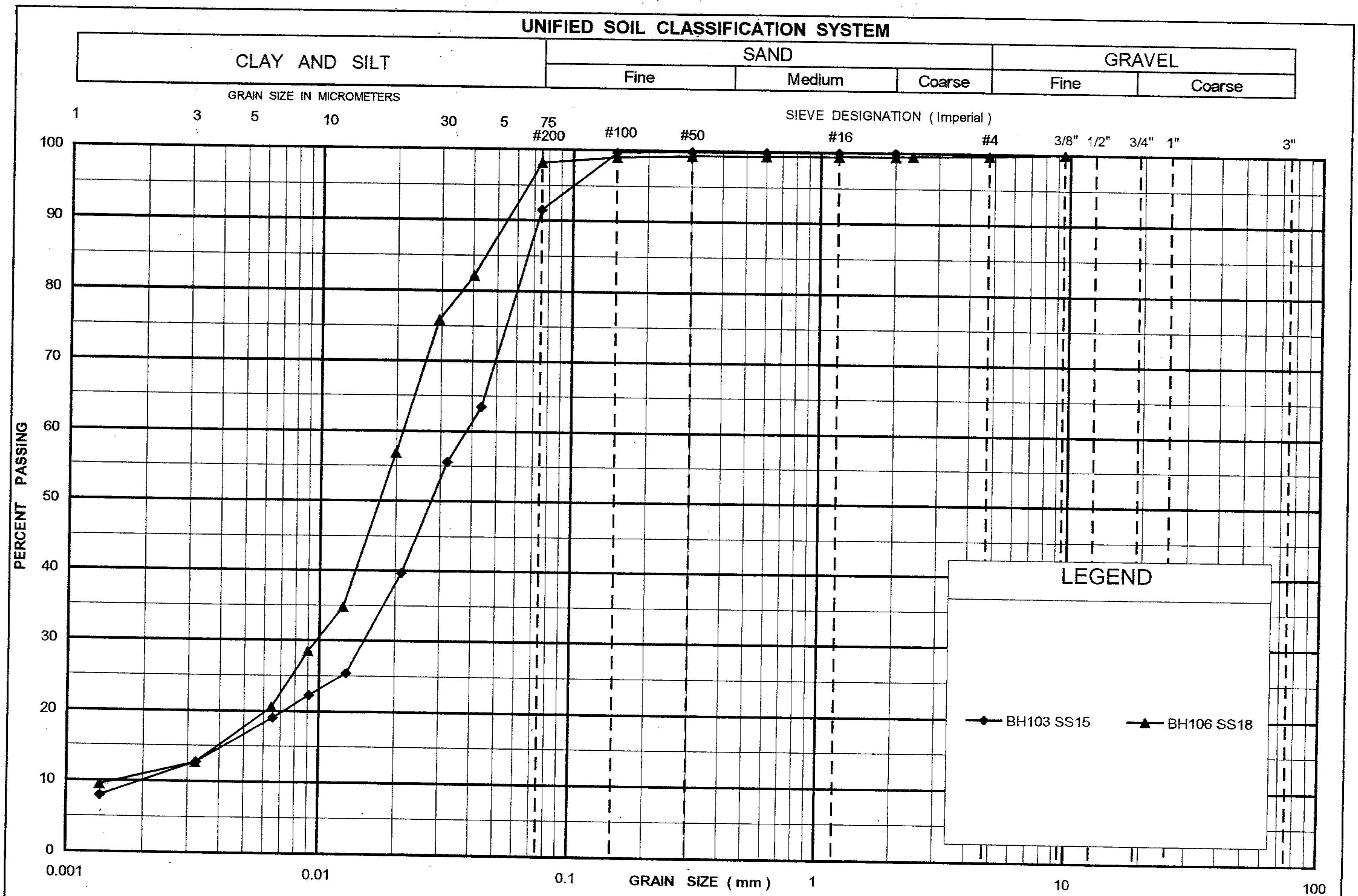
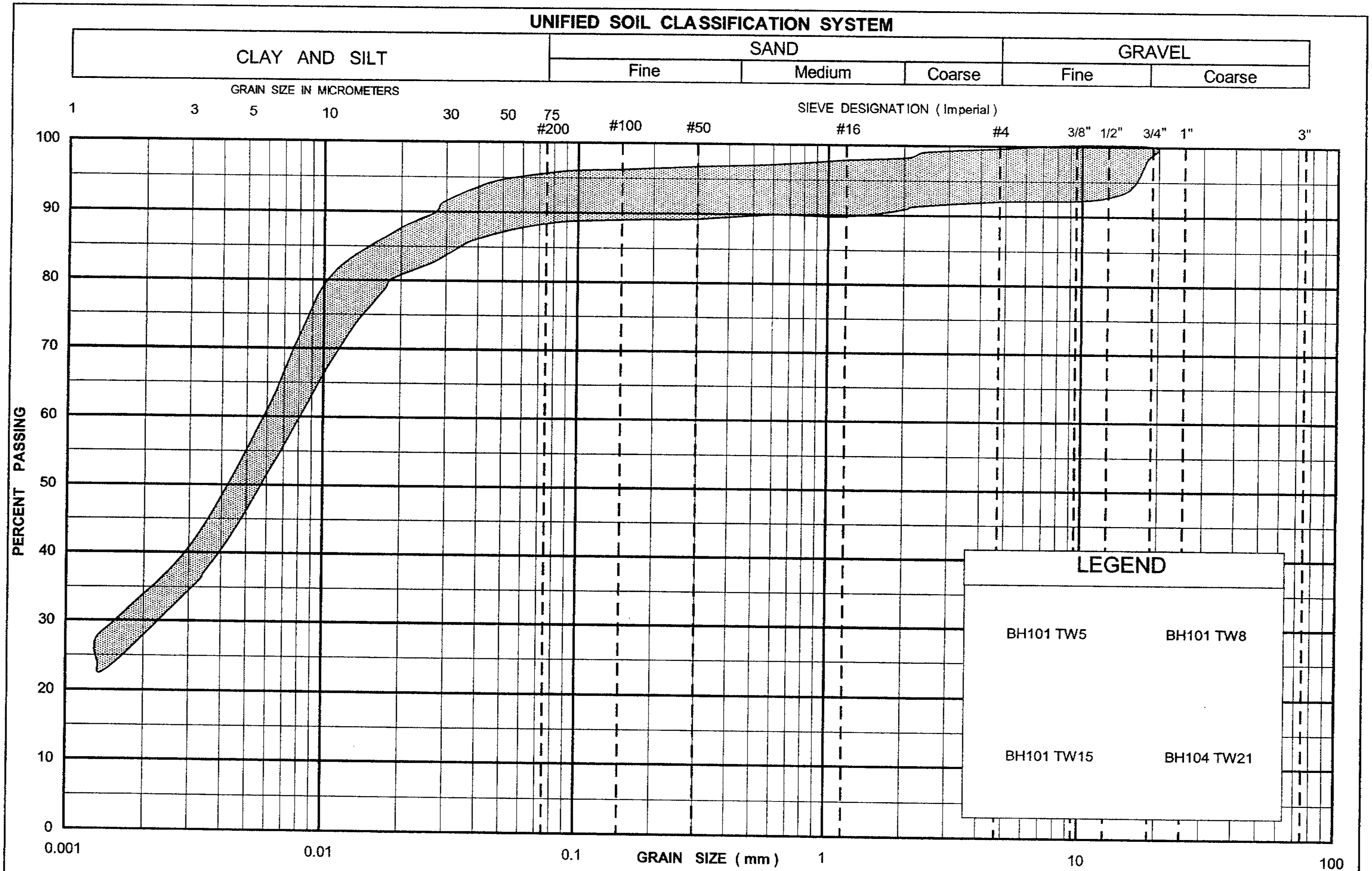


FIGURE B-3



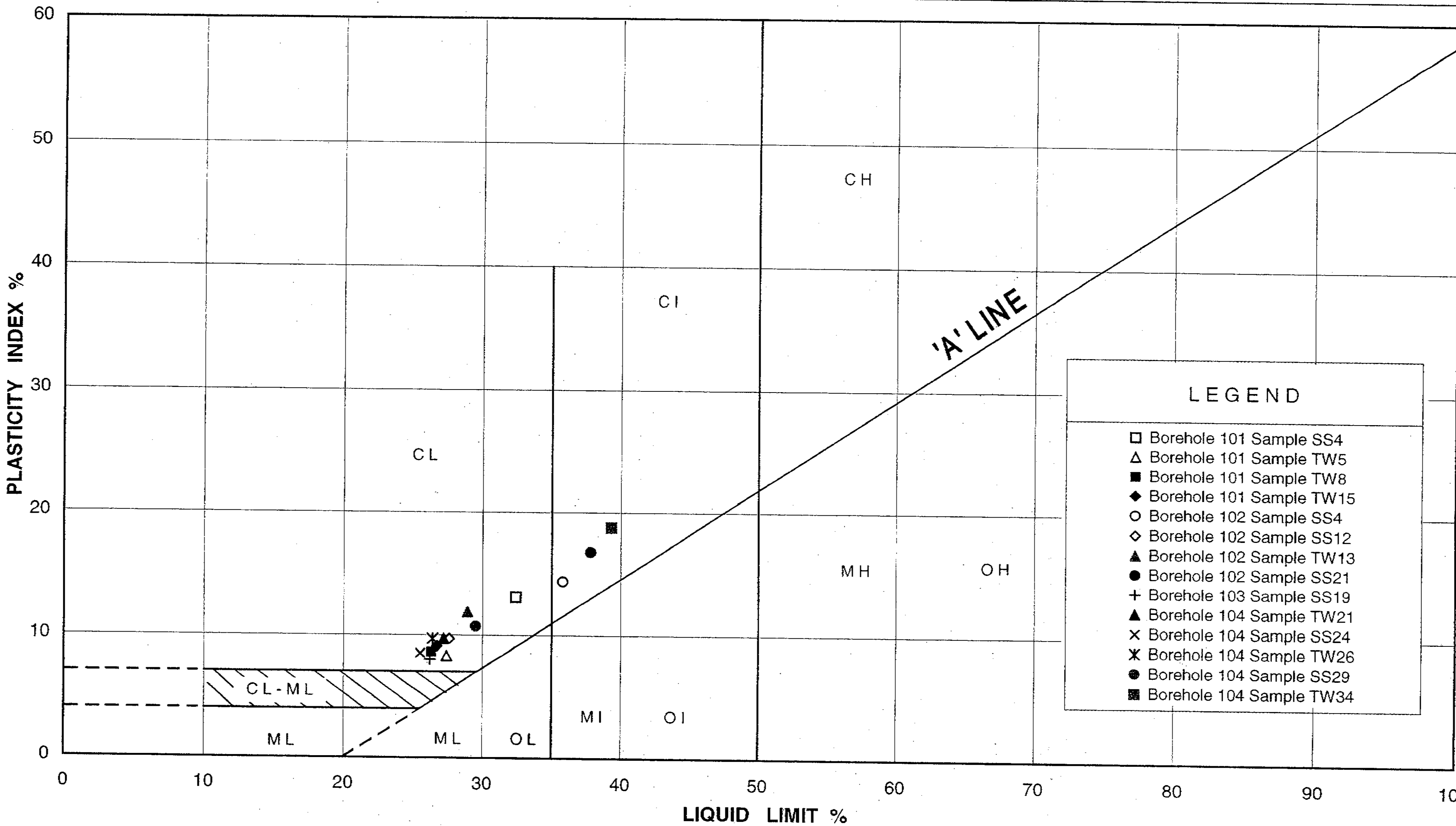
SHAHEEN & PEAKER LIMITED	GRAIN SIZE DISTRIBUTION SILT	FIGURE No. B-4 REF. No. SPT 1105 W.P. 604-00-01



SHAHEEN & PEAKER LIMITED

GRAIN SIZE DISTRIBUTION
CLAYEY SILT

FIGURE No.	B-5
REF. No.	SPT 1105
W.P.	604-00-01



SHAHEEN & PEAKER LIMITED	PLASTICITY CHART CLAYEY SILT with some silty clay layers	FIG No B-6 W.P. 604-00-01 SPT 1105

SPT 1105
W.P. 604-00-01

BH 102 - TW 13
Depth 10.7-11.2 m

Highway 6 (New) and Glancaster Road
Hamilton, Ontario

Figure B-7

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST (UU)

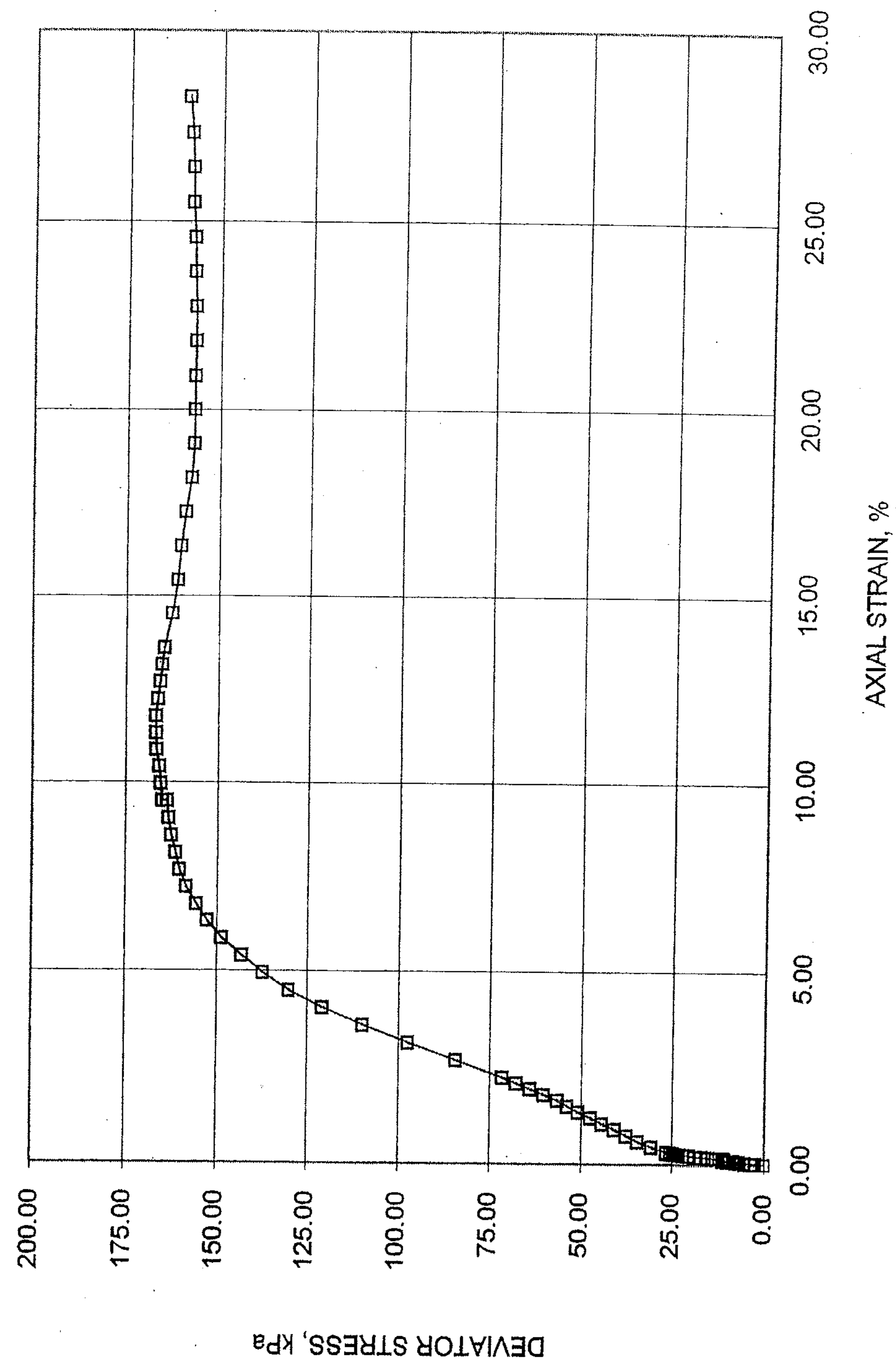
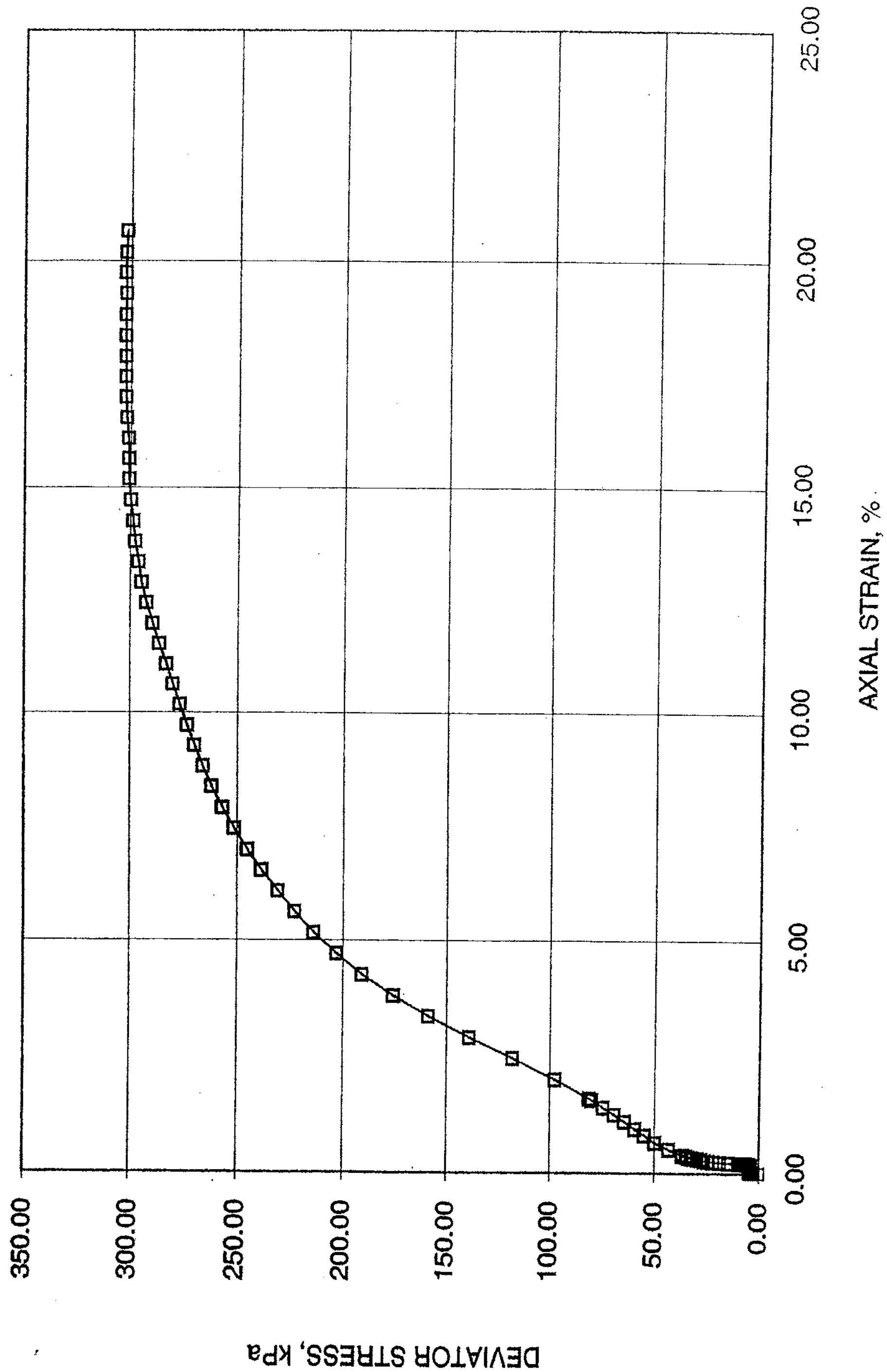


Figure B-8

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST (UU)



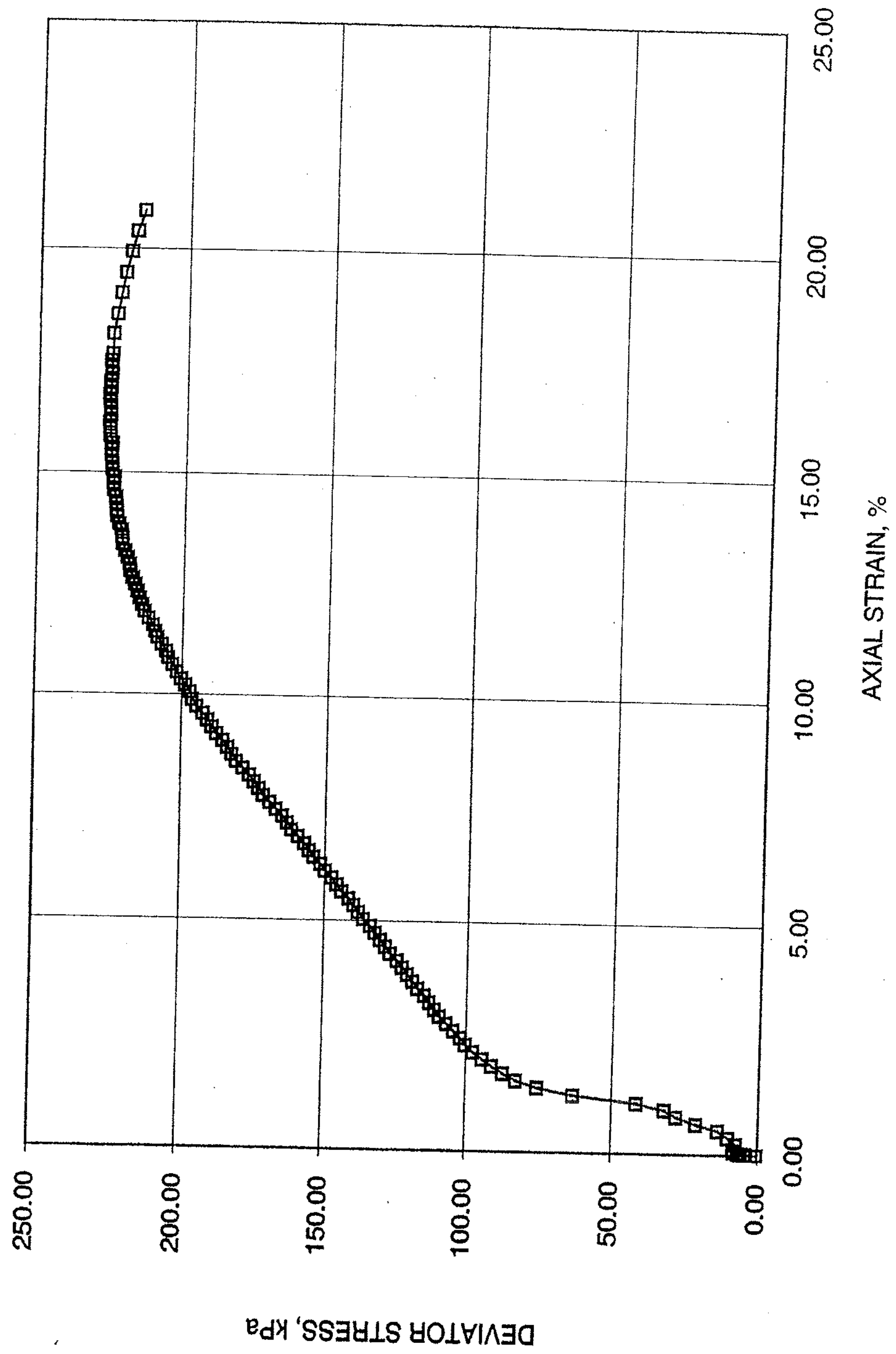
SPT 1105
W.P. 604-00-01

BH 104 - TW 34
Depth 30.5-31.0 m

Highway 6 (New) and Glancaster Road
Hamilton, Ontario

Figure B-9

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST (UU)

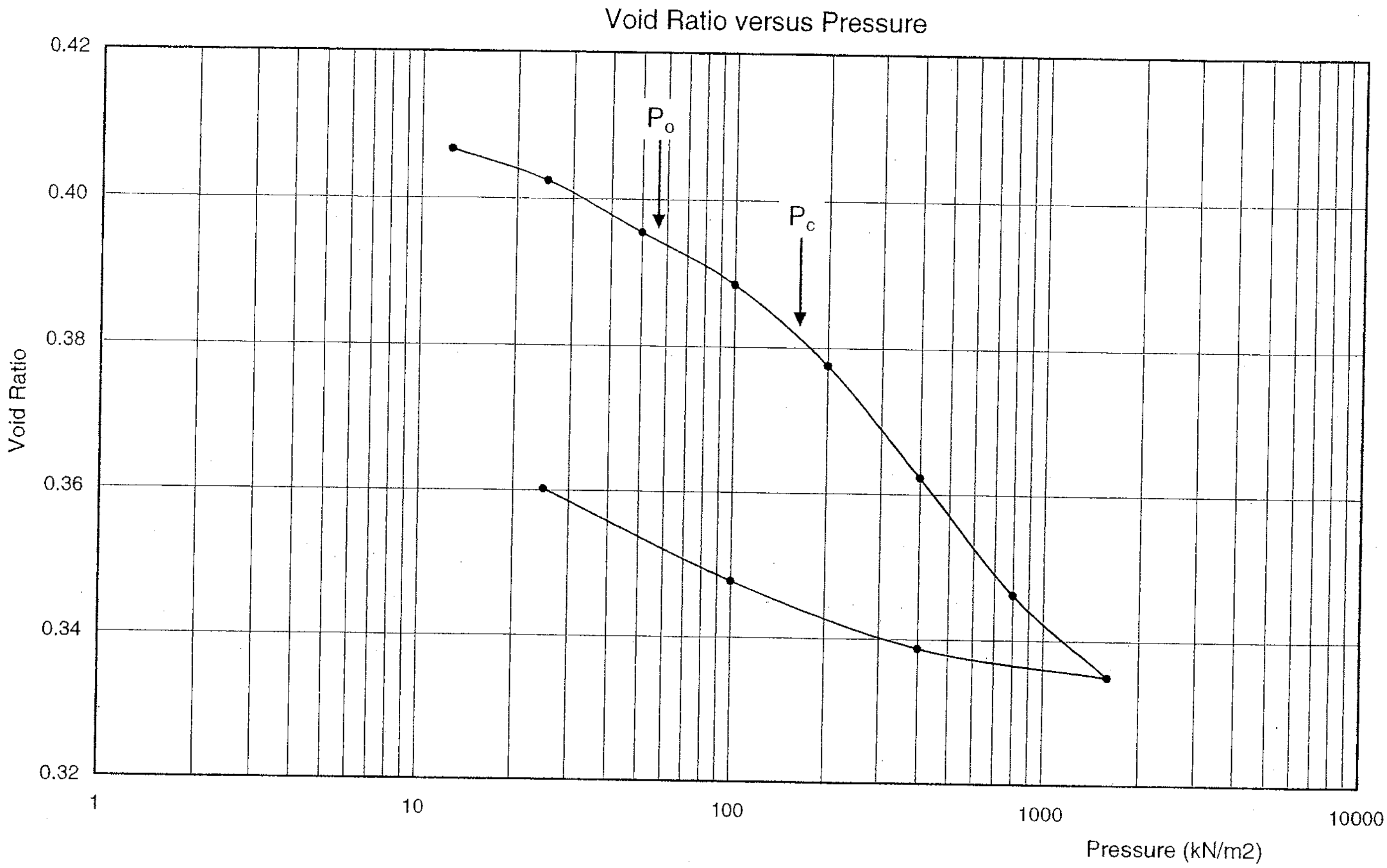


SPT 1105
W.P. 604-00-01

BH 101 - TW 5
Depth 3.0-3.5 m

Highway 6 (New) and Glancaster Road
Hamilton, Ontario

Figure B-10



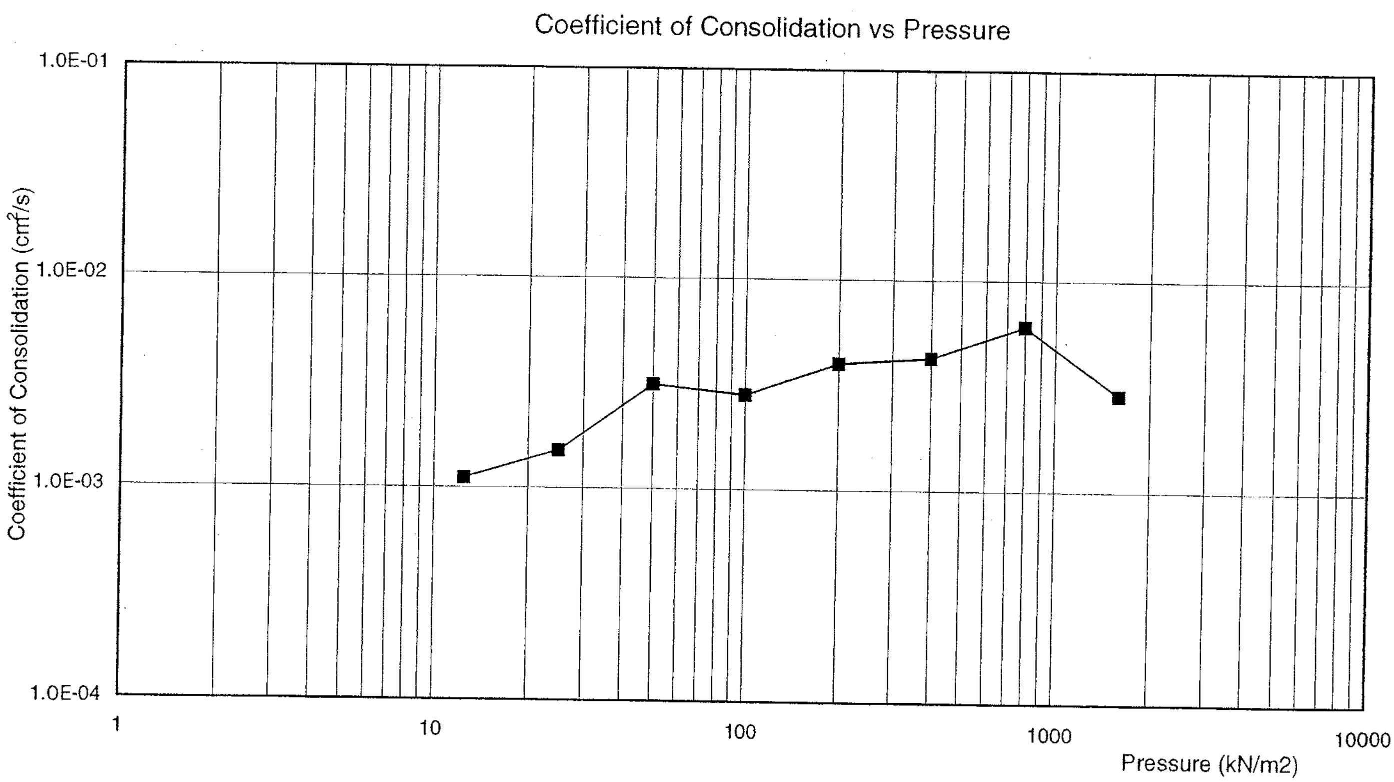
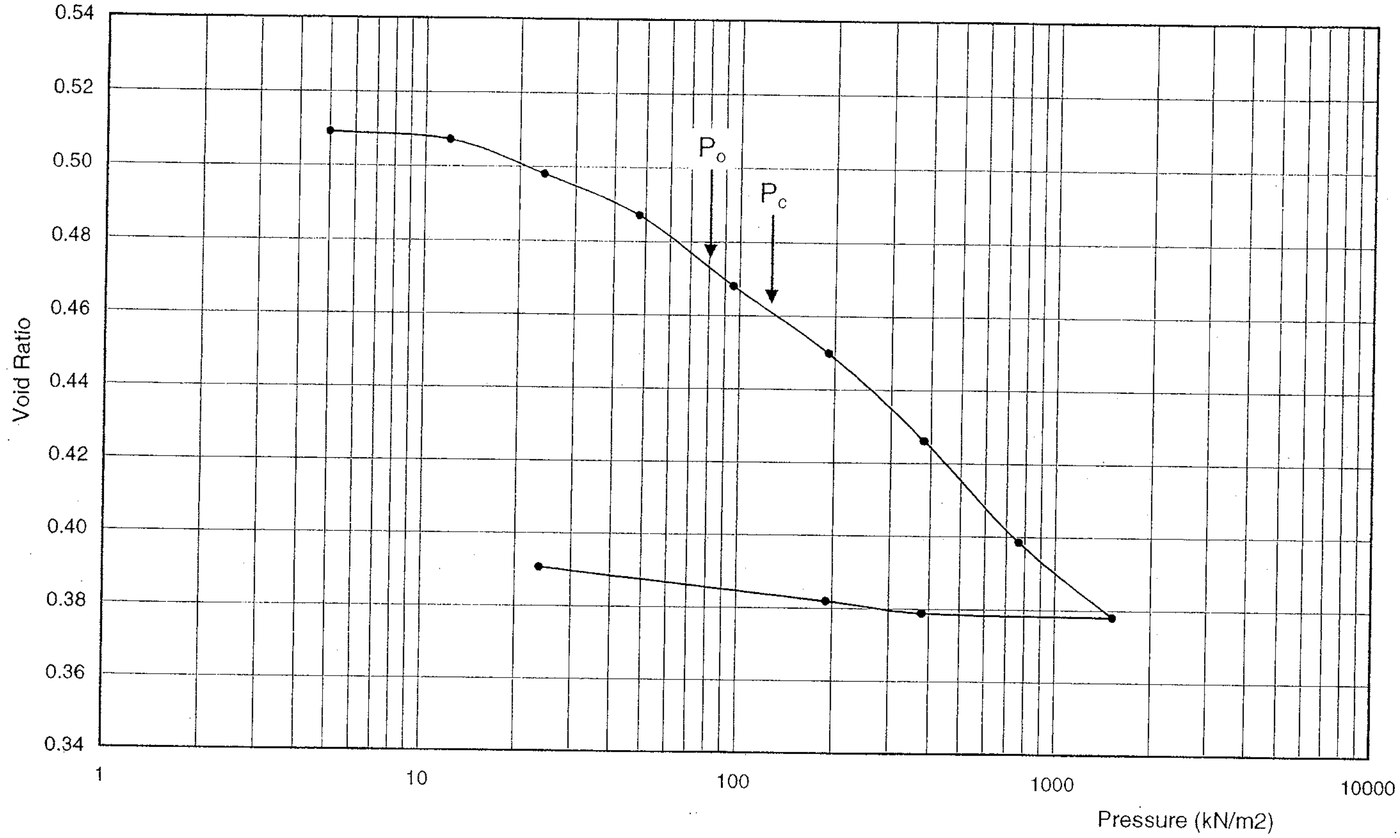
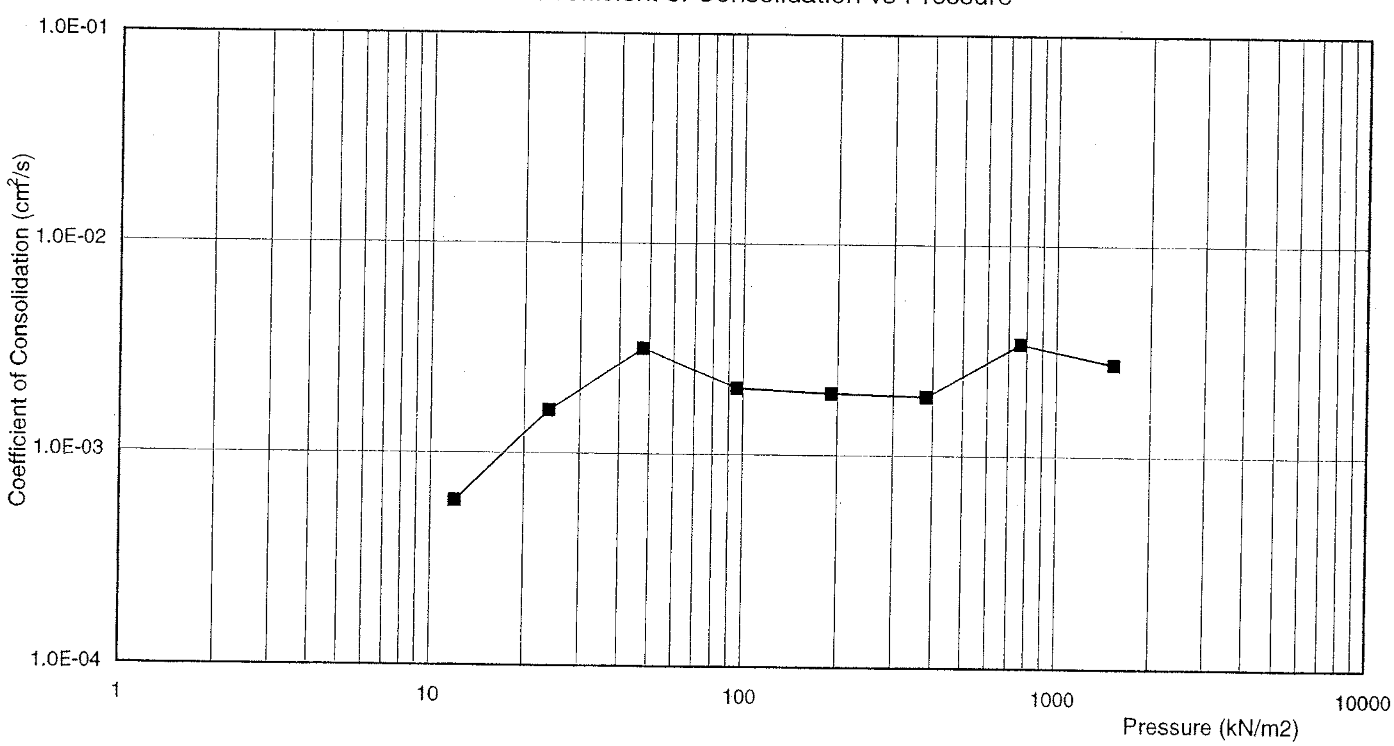


Figure B-11

Void Ratio versus Pressure



Coefficient of Consolidation vs Pressure



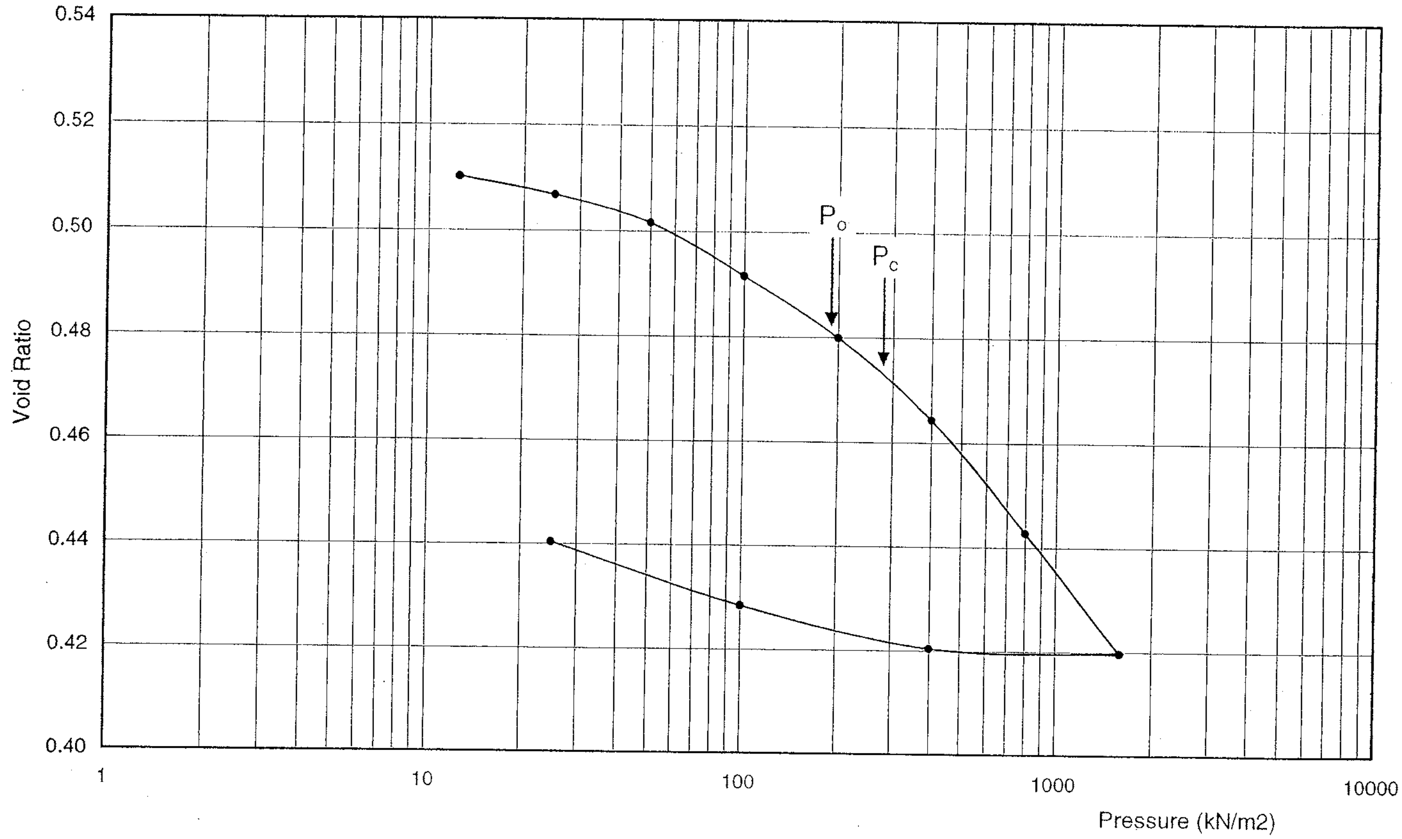
SPT 1105
W.P. 604-00-01

BH 101 - TW 15
Depth 15.2-15.7 m

Highway 6 (New) and Glancaster Road
Hamilton, Ontario

Figure B-12

Void Ratio versus Pressure



Coefficient of Consolidation vs Pressure

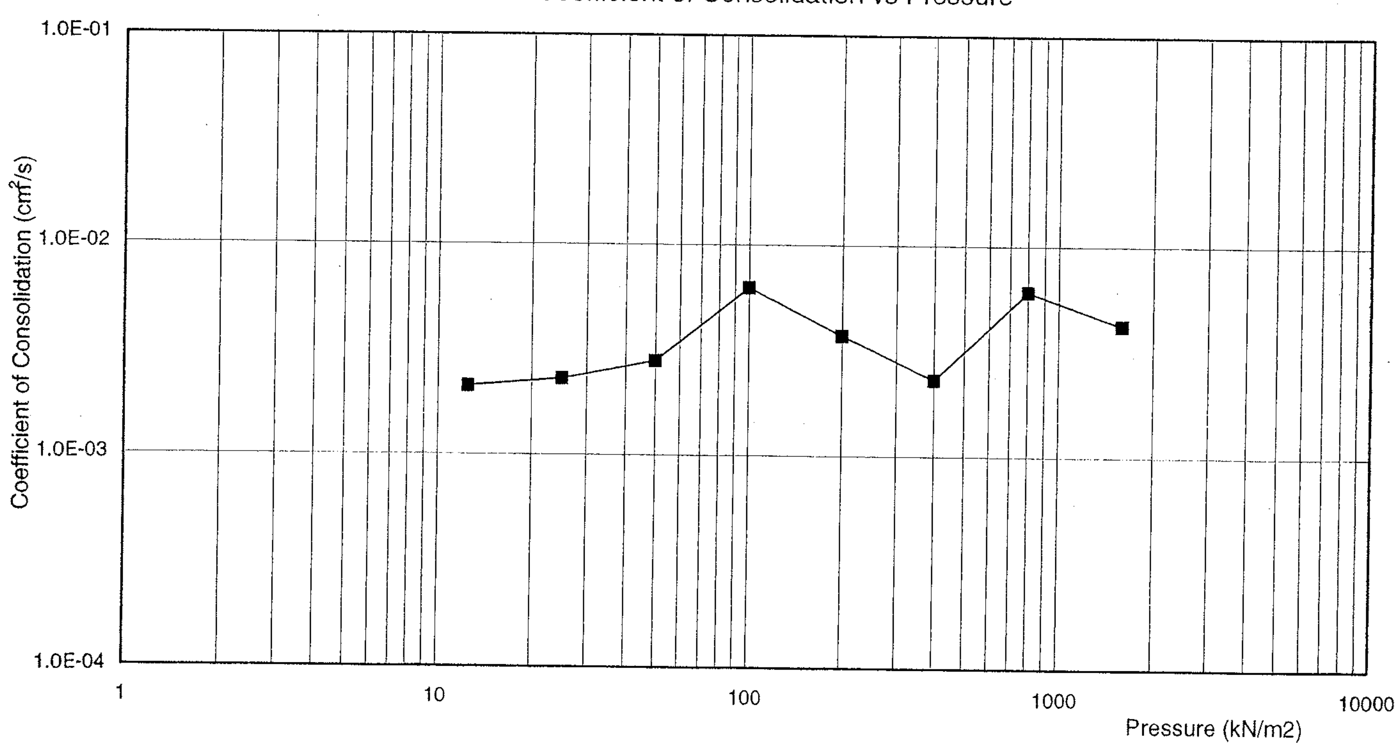
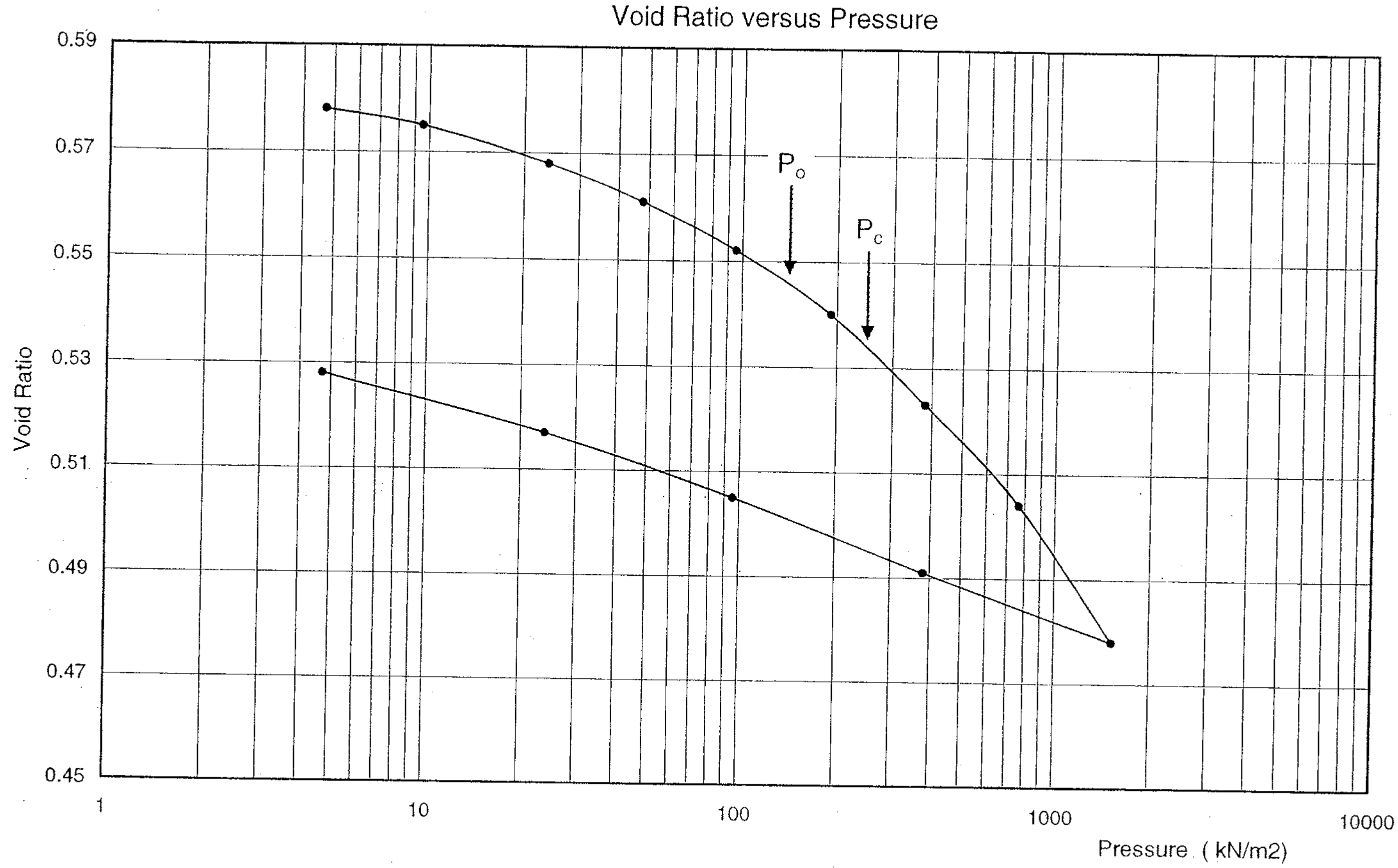
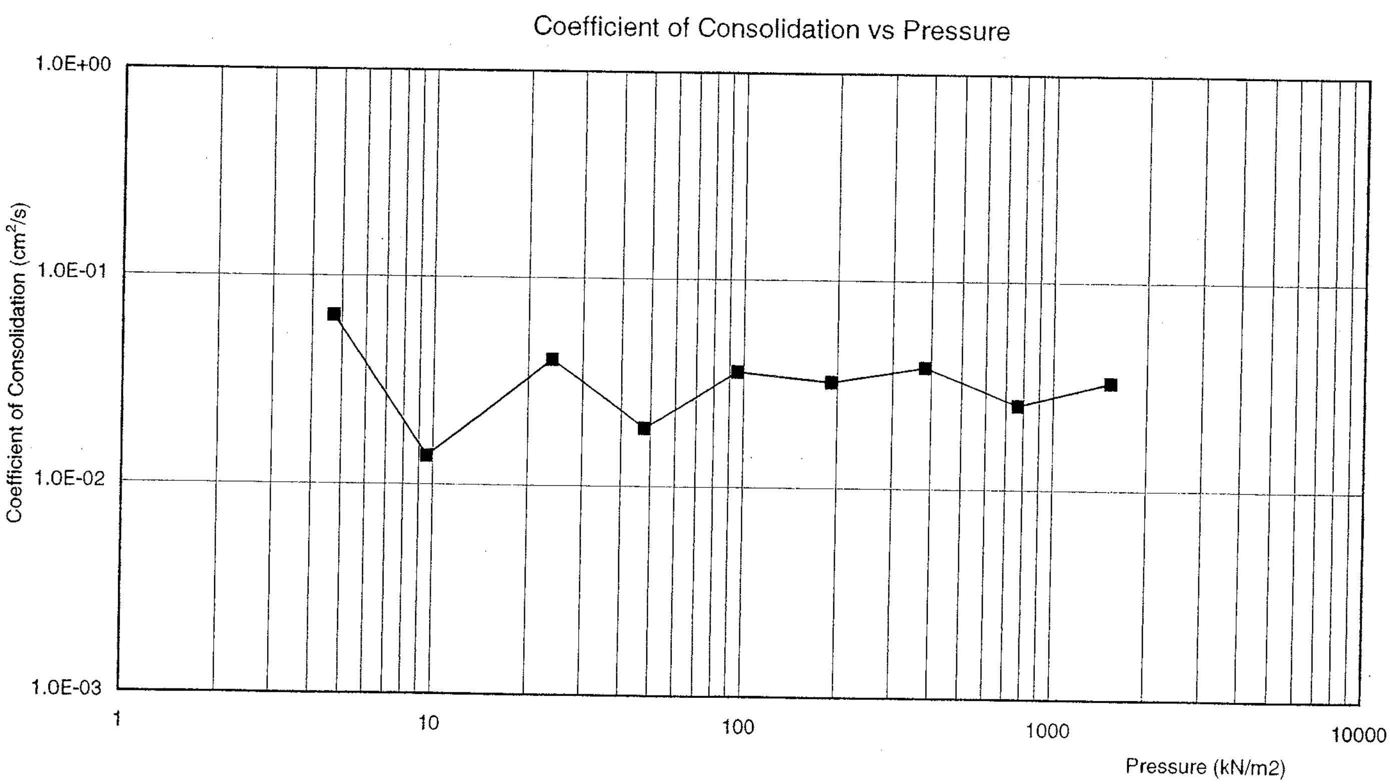


Figure B-13


Void Ratio versus Pressure
0.59
0.57
0.55
0.53
0.51
0.49
0.47
0.45
Void Ratio
P_o
P_c
1
10
100
1000
10000
Pressure (kN/m2)

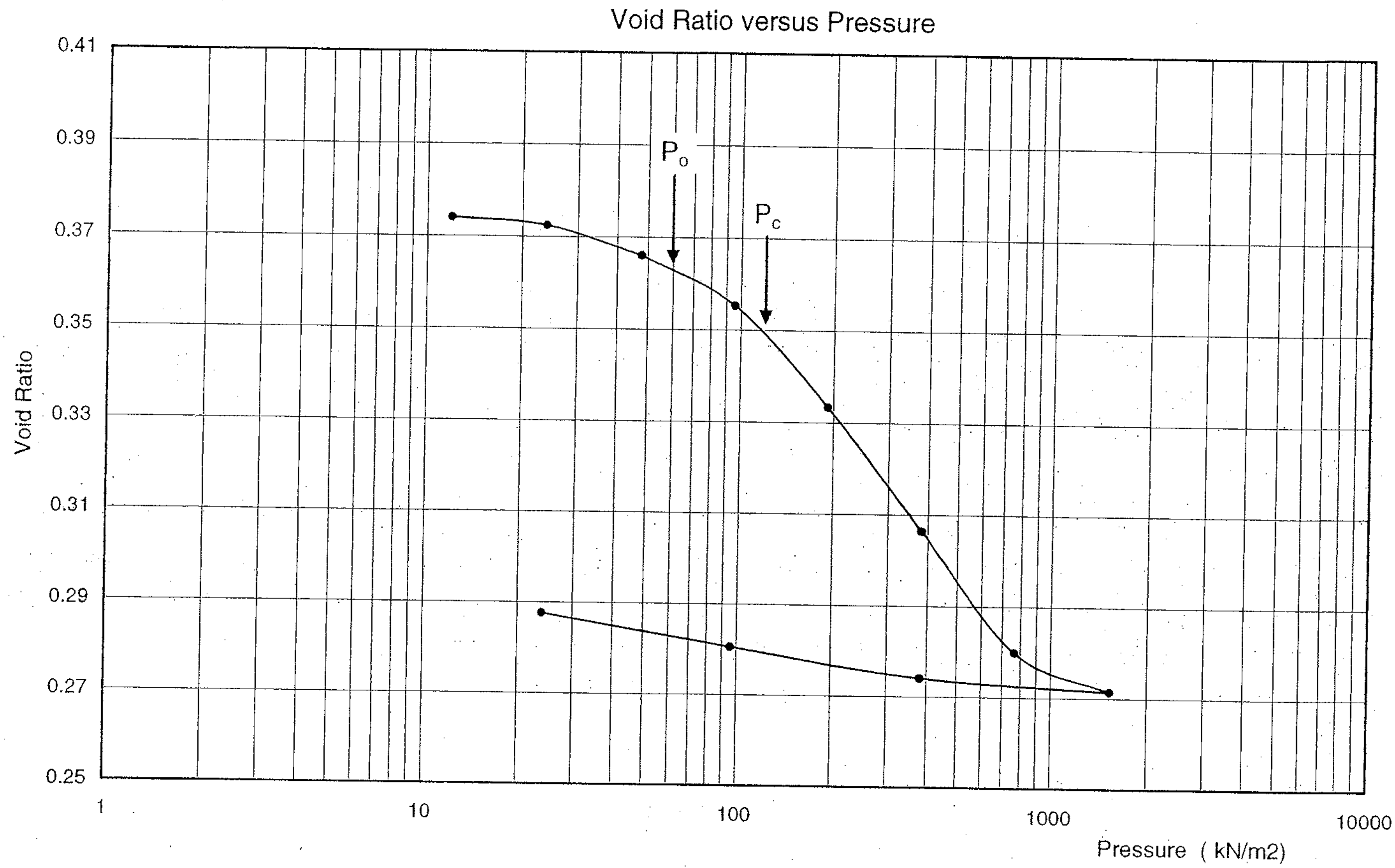

Coefficient of Consolidation vs Pressure
1.0E+00
1.0E-01
1.0E-02
1.0E-03
Coefficient of Consolidation (cm^2/s)
1
10
100
1000
10000
Pressure (kN/m2)

SPT 1105
W.P. 604-00-01

BH 104 - TW 21
Depth 12.2-12.7 m

Highway 6 (New) and Glancaster Road
Hamilton, Ontario

Figure B-14



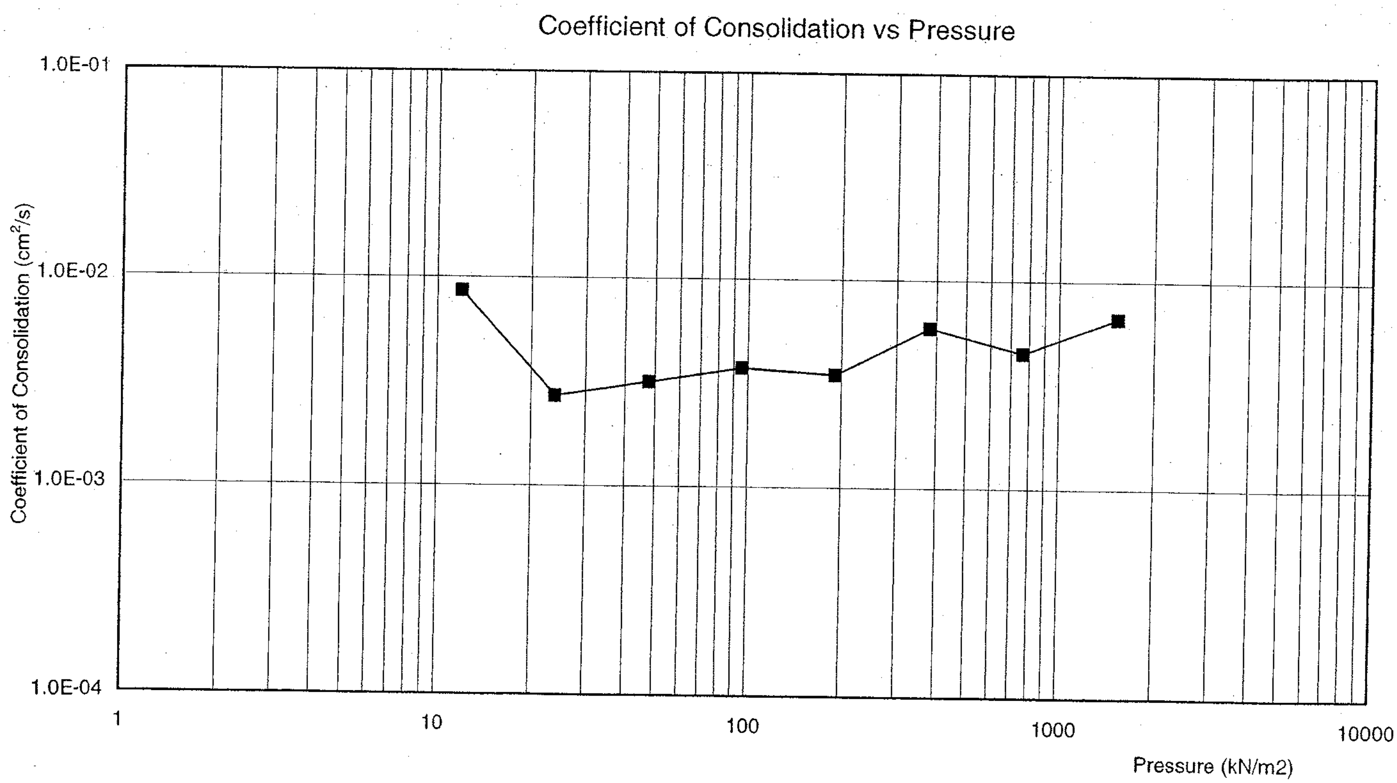
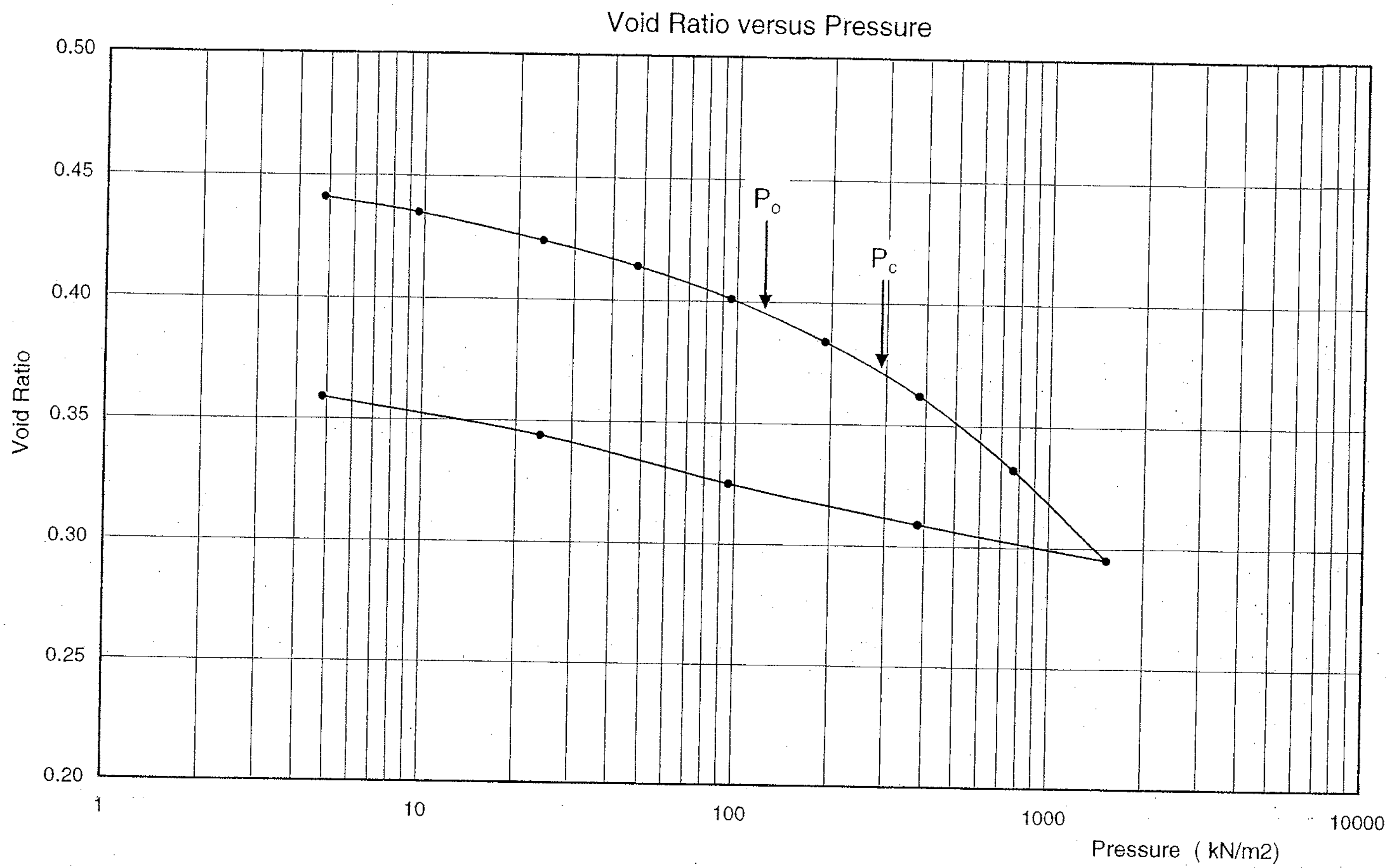


Figure B-15


Void Ratio versus Pressure
0.50
0.45
0.40
0.35
0.30
0.25
0.20
Void Ratio
P_o
P_c
1
10
100
1000
10000
Pressure (kN/m2)

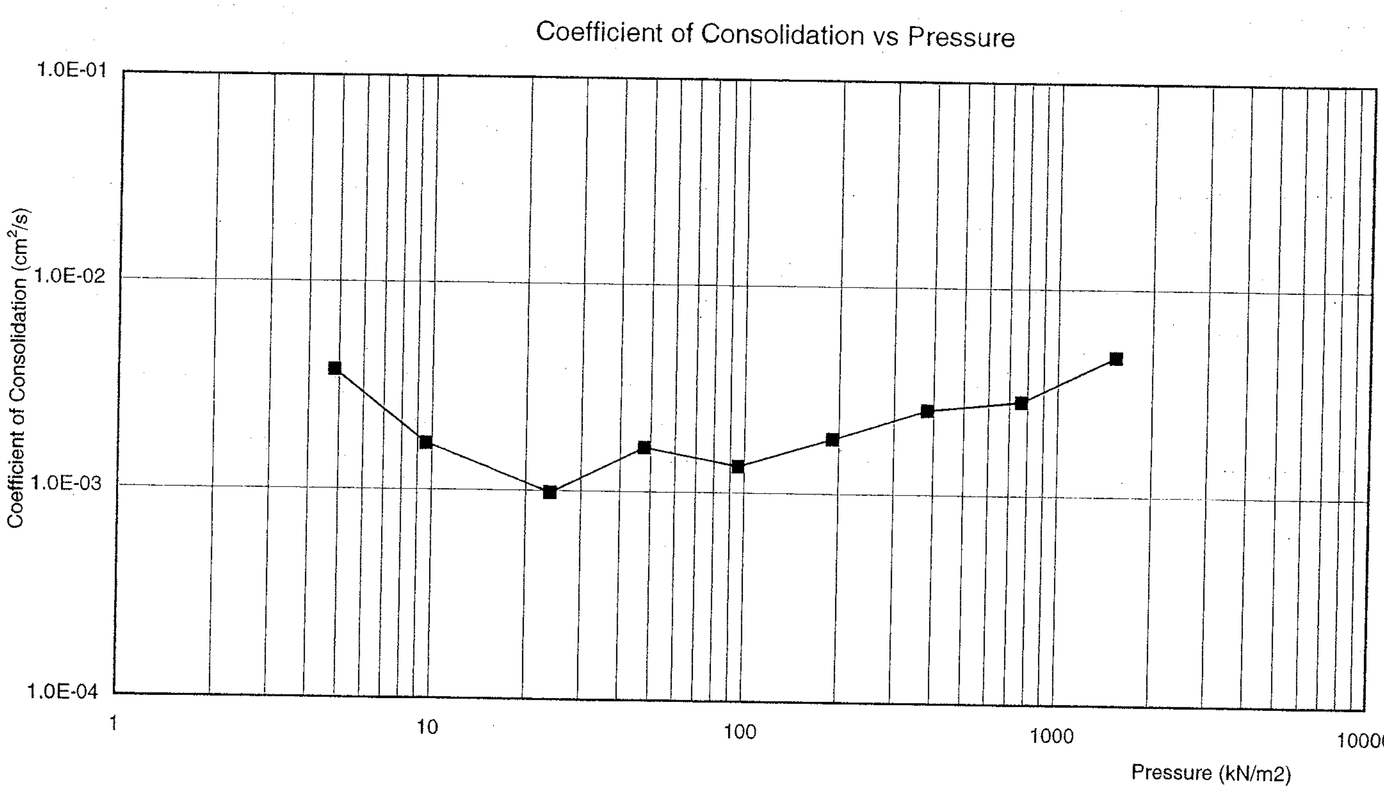
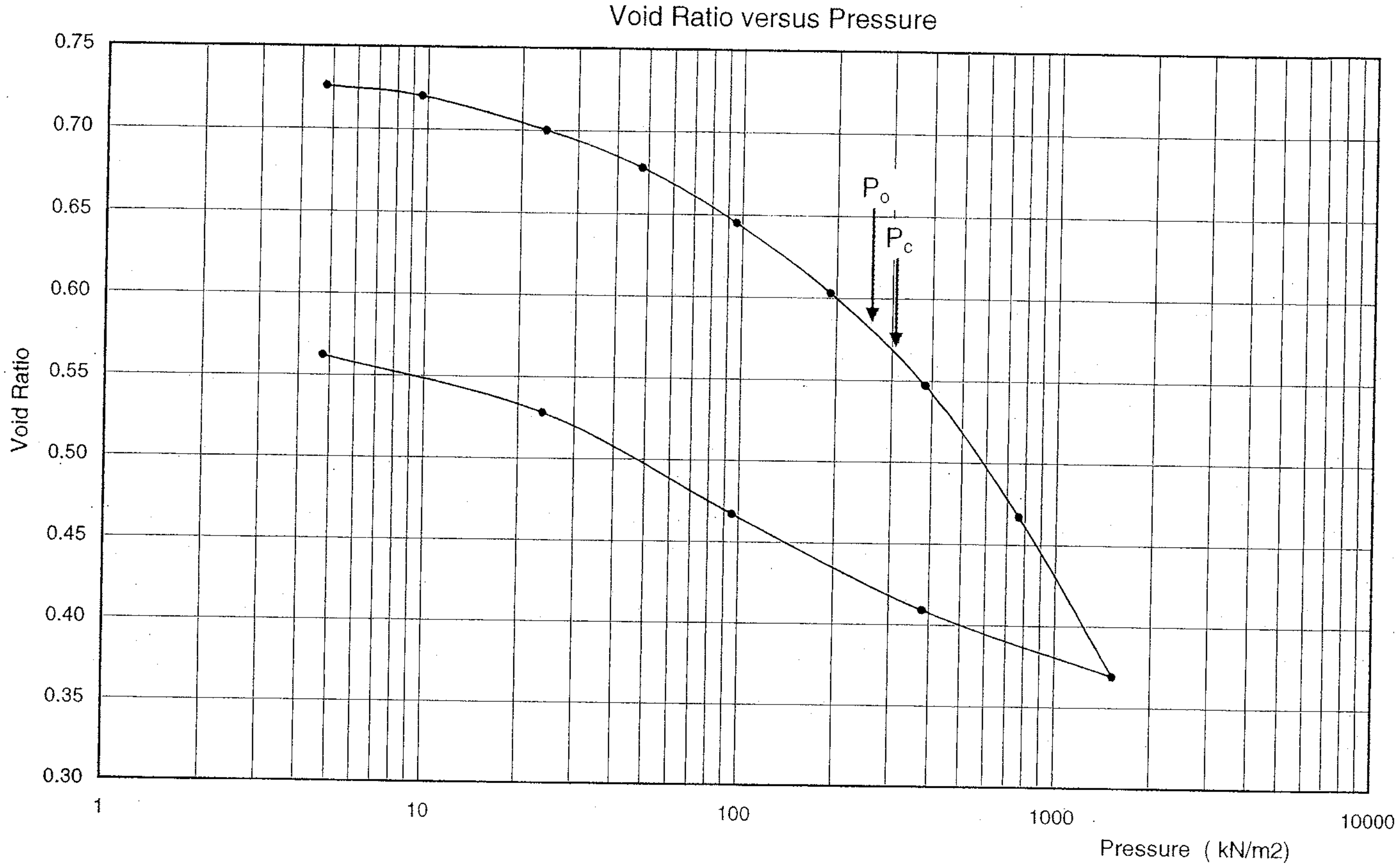
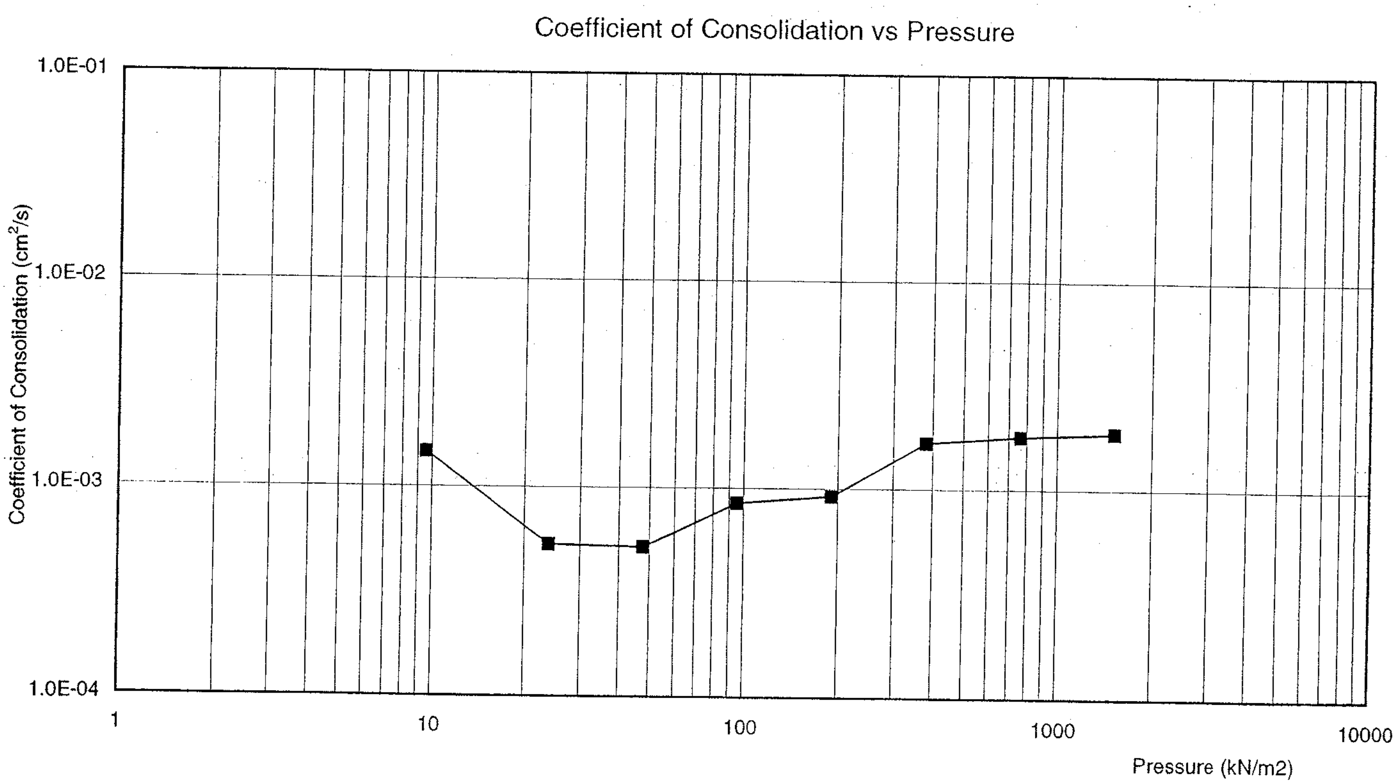

Coefficient of Consolidation vs Pressure
1.0E-01
1.0E-02
1.0E-03
1.0E-04
Coefficient of Consolidation (cm^2/s)
1
10
100
1000
10000
Pressure (kN/m2)

Figure B-16


Void Ratio versus Pressure
0.75
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
Void Ratio
P_o
P_c
1
10
100
1000
10000
Pressure (kN/m2)


Coefficient of Consolidation vs Pressure
1.0E-01
1.0E-02
1.0E-03
1.0E-04
Coefficient of Consolidation (cm^2/s)
1
10
100
1000
10000
Pressure (kN/m2)

Appendix C

Standard Penetration Test Results and Measured Undrained Shear Strength Results

SPT 1105
W.P. 604-00-01

Highway 6 (New) and Glancaster Road
Hamilton , Ontario

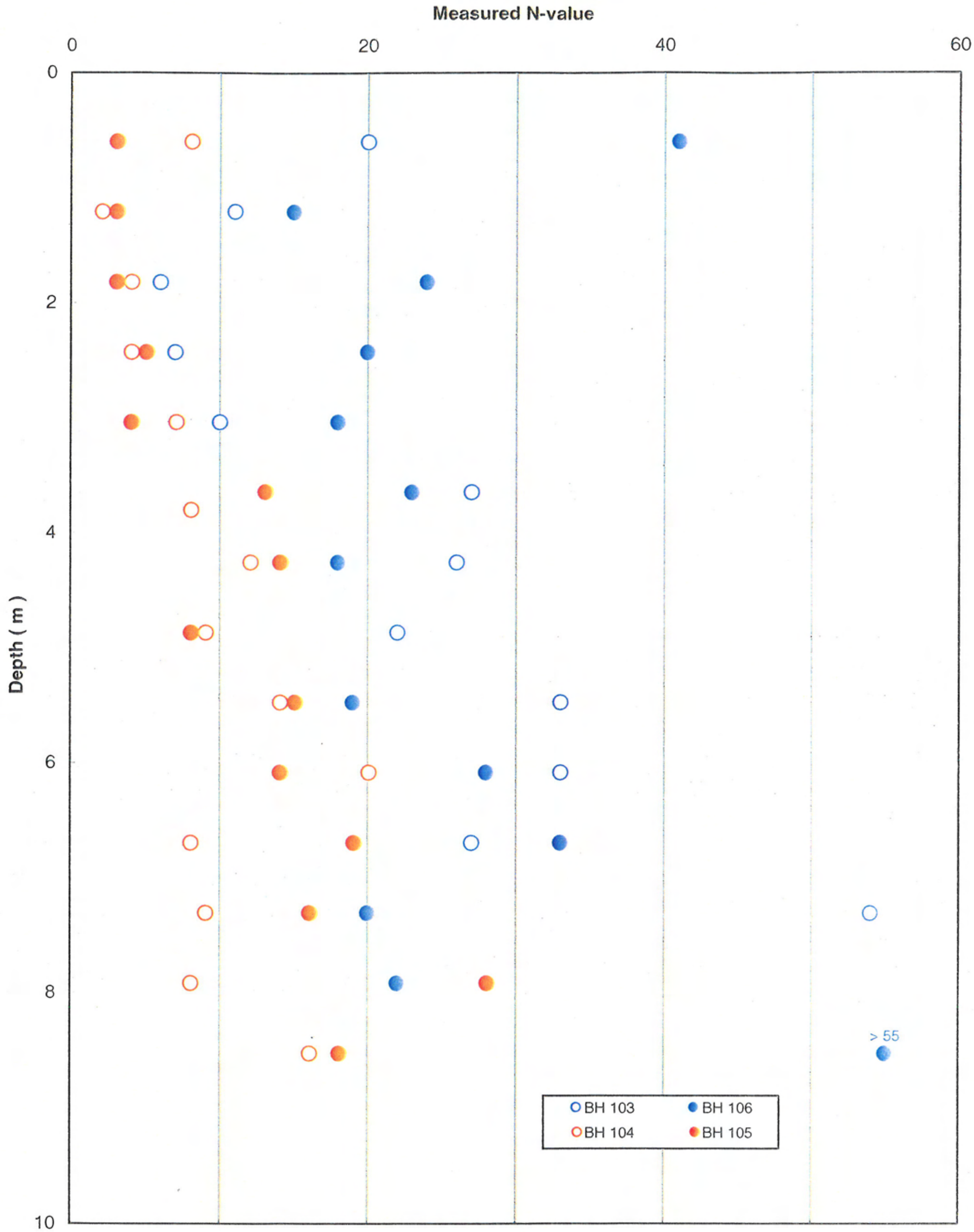


FIGURE C-1: Variation of Measured N-values with Depth in the Embankment Fill

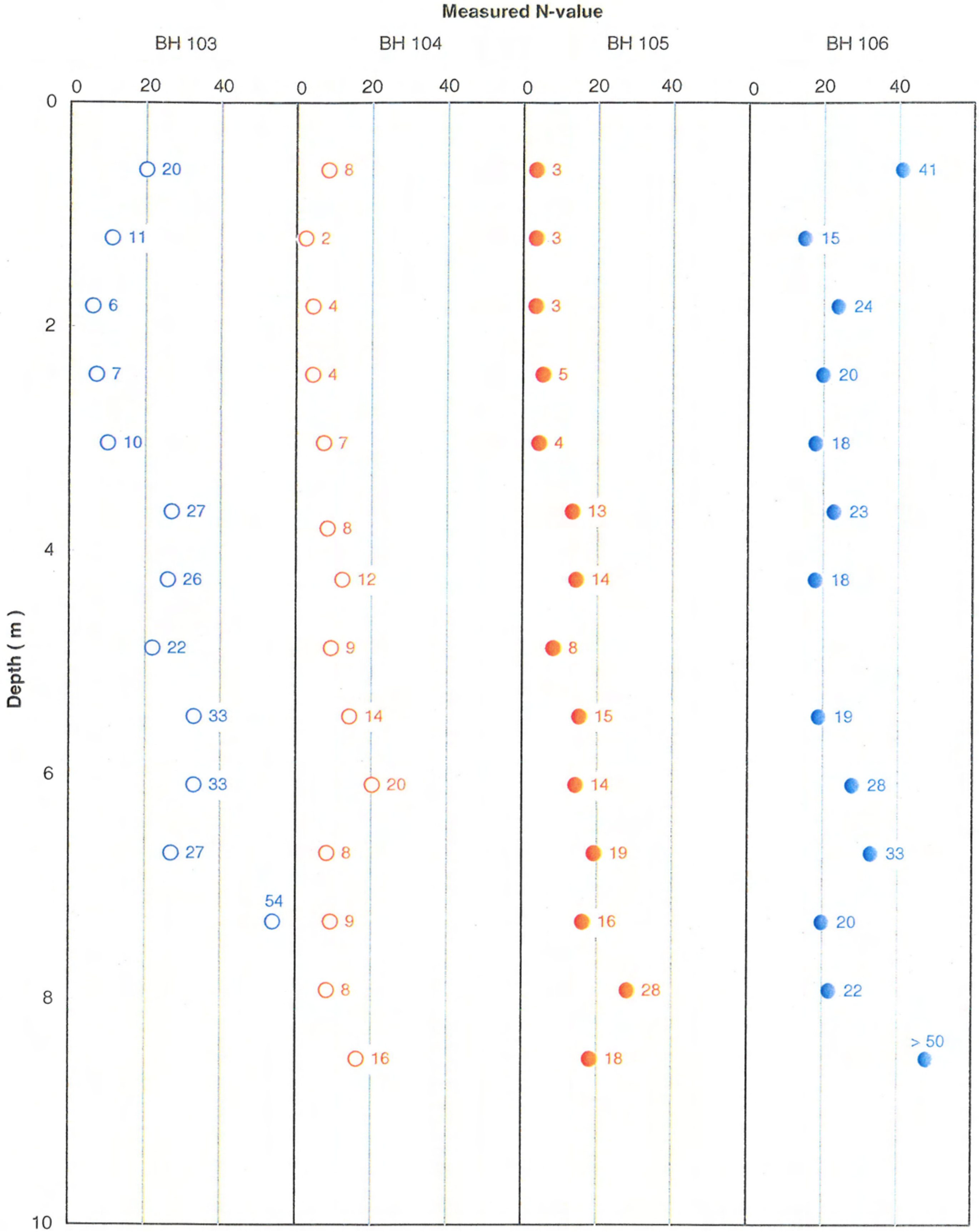


FIGURE C-1a: Measured N-values with Depth in the Embankment Fill

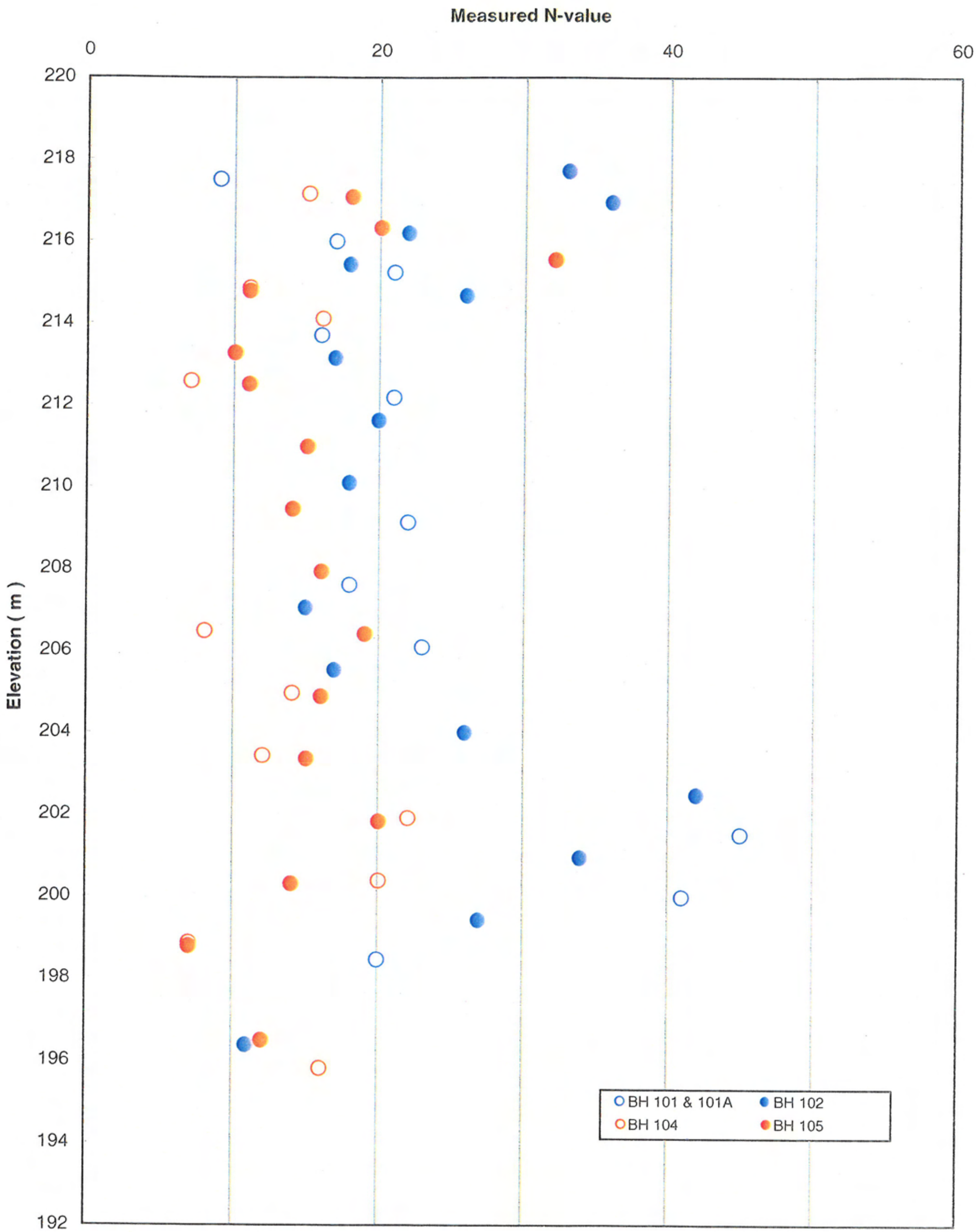


FIGURE C-2: Variation of Measured N-values with Elevation in the Clayey Silt Deposit

SPT 1105
W.P. 604-00-01

Highway 6 (New) and Glancaster Road
Hamilton , Ontario

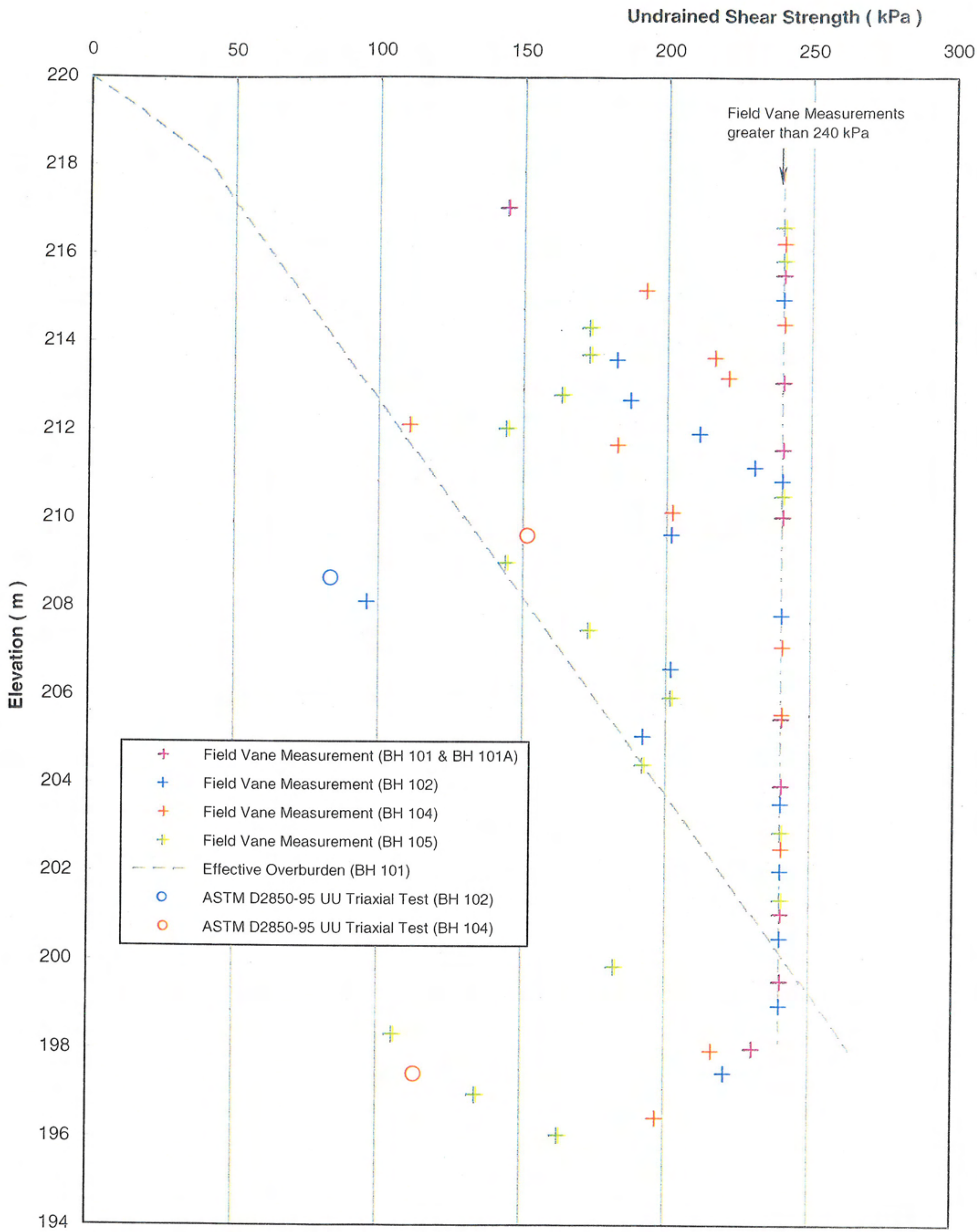


FIGURE C-3: Variation of Measured Undrained Shear Strength with Elevation in the Clayey Silt Deposit

Appendix D

Record of Borehole Sheets (by others)

PetoMacCallum Ltd.
CONSULTING ENGINEERS

RECORD OF BOREHOLE No 6

N 4 779 746
E 268 166

W.P. 604-00-01 LOCATION GLANCASTER ROAD UNDERPASS AT HWY 6 (NEW) ORIGINATED BY M.R.
DIST. ____ HWY. 6 BORING DATE October 31, 2000 COMPILED BY M.R.A.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Solid Stem Augers CHECKED BY D.W.K.

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	N - VALUES
219.36	Ground Level			
0.00	Fill, clayey silt and crushed limestone, Dark brown			
218.76 / 0.60	Topsoil, clayey silt, Dark brown	1	SS	2
217.36 / 1.40 (1.70)	Alluvium, clayey silt, trace of sand, Firm, Dark brown	2	SS	6
	Clay, silty, trace of sand; thin partings of silt, Very stiff, Brown	3	SS	18
		4	SS	18
215.36 / 4.00	Clay till, silty, trace of sand and gravel, Very stiff, Grey	5	SS	17
212.81 / 6.55	End of Borehole	6	SS	17

Depth scale: 0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15, 16.5

GROUND WATER ELEV.: 219.0, 218.0, 217.0, 216.0, 215.0, 214.0, 213.0, 212.0

DYNAMIC CONE PENETRATION RESISTANCE PLOT ×
STANDARD PENETRATION TEST •
20 40 60 80 100

SHEAR STRENGTH (kPa)
O UNCONFINED + FIELD VANE
■ PENETROMETER × LAB VANE
50 100 150 200 250

LIQUID LIMIT W_L
PLASTIC LIMIT W_P
WATER CONTENT W
W_P W W_L
WATER CONTENT %
20 40 60

UNIT WEIGHT γ kN/m³

REMARKS
×
GR. SA. SI. CL.

Native Backfill
19mm Ø PVC Pipe
Bentonite Seal
Filter Gravel

Upon completion of augering, no water, no cave.

Date	Depth to Water (m)
Nov. 3	4.45
Nov. 14	1.05

PetoMacCallum Ltd.
CONSULTING ENGINEERS

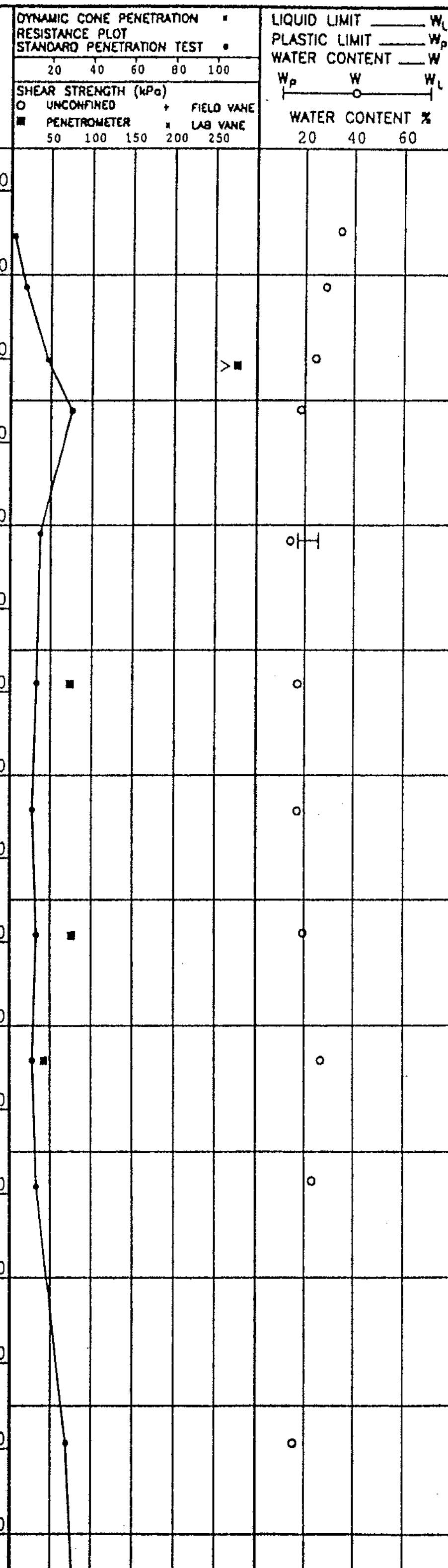
N 4 779 759
E 268 171

RECORD OF BOREHOLE No 7

W.P. 604-00-01 LOCATION GLANCASTER ROAD UNDERPASS AT HWY 6 (NEW) ORIGINATED BY M.R.
DIST. ____ HWY. 6 BORING DATE October 31, 2000 COMPILED BY M.R.A.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring CHECKED BY D.W.K.

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	N – VALUES	GROUND WATER ELEV.	GR.	SA.	SI.	CL.
219.47	Ground Level								
0.0	Fill, silt, trace of clay and sand				219.0				
218.42 / 1.05	Topsoil, silt, Dark brown to black	1	SS	3					
1.40		2	SS	8	218.0				
217.47 / 2.00	Alluvium, clayey silt, trace of sand, Stiff, Brown								
	Clay, silty, trace of sand, Very stiff, Brown	3	SS	19	217.0				
		4	SS	30	216.0				
215.47 / 4.00	Clay till, silty, trace of sand and gravel, Stiff, Grey				215.0				
		5	SS	15	214.0	1	4	66	29
		6	SS	13	213.0				
		7	SS	11	212.0 / 211.0				
		8	SS	13	210.0				
		9	SS	12	209.0				
208.47 / 11.00	Clay, silty, with thin lenses and layers of silt, Stiff, Grey				208.0				
		10	SS	13	207.0 / 206.0				
205.47 / 14.00	Clay till, silty, trace of sand and gravel, Very stiff, Grey				205.0				
		11	SS	27	204.0 / 203.0				

Depth scale: 0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15, 16.5



UNIT WEIGHT γ kN/m³

REMARKS %

Continued on Page 2 of 2

PetoMacCallum Ltd.
CONSULTING ENGINEERS

RECORD OF BOREHOLE No 7 Cont.

N 4 779 759
E 268 171

W.P. 604-00-01 LOCATION GLANCASTER ROAD UNDERPASS HWY 6 (NEW) ORIGINATED BY M.R.
DIST. ____ HWY. 6 BORING DATE October 31, 2000 COMPILED BY M.R.A.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring CHECKED BY D.W.K.

SOIL PROFILE: ELEV./DEPTH; DESCRIPTION; STRAT. PLOT
SAMPLES: NUMBER; TYPE; N - VALUES
GROUND WATER; ELEV.
DYNAMIC CONE PENETRATION RESISTANCE PLOT ×; STANDARD PENETRATION TEST • (20, 40, 60, 80, 100)
SHEAR STRENGTH (kPa): O UNCONFINED; + FIELD VANE; ■ PENETROMETER; × LAB VANE (50, 100, 150, 200, 250)
LIQUID LIMIT ____ W_L; PLASTIC LIMIT ____ W_P; WATER CONTENT ____ W; W_P — W — W_L; WATER CONTENT % (20, 40, 60)
UNIT WEIGHT γ kN/m³
REMARKS % GR. SA. SI. CL.

Depth scale: 16.5, 18.0, 19.5, 21.0, 22.5, 24.0, 25.5, 27.0, 28.5, 30.0, 32.5, 34.0

ELEV. / DEPTH	DESCRIPTION	NUMBER	TYPE	N - VALUES	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)	REMARKS
	Ground Level								
	Continued from Page 1 of 2								
202.47 / 17.00	Silt, trace of clay; thin lenses of brown and grey clay. Dense, Grey	12	SS	30					
199.47 / 20.00	Clay, silty; occassional thin partings and lenses of silt. Stiff, Layered Brown and grey	13	SS	12					
195.02 / 24.45	Dolostone Bedrock	14	RC		1475	100	98	100	
		15	RC		1525	100	93	100	
		16	RC		305	100	100	100	
191.72 / 27.75	End of Borehole								Upon completion of augering, no water, no cave.

Ground water elevations: 202.0, 201.0, 200.0, 199.0, 198.0, 197.0, 196.0, 195.0, 194.0, 193.0, 192.0, 191.0

PetoMacCallum Ltd.
CONSULTING ENGINEERS

N 4 779 791
E 268 172

RECORD OF BOREHOLE No 8

W.P. 604-00-01 LOCATION GLANCASTER ROAD UNDERPASS AT HWY 6 (NEW) ORIGINATED BY M.R.
DIST. ___ HWY. 6 BORING DATE October 30 & 31, 2000 COMPILED BY M.R.A.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring CHECKED BY D.W.K.

Column headings: SOIL PROFILE (ELEV./DEPTH, DESCRIPTION, STRAT. PLOT); SAMPLES (NUMBER, TYPE, N - VALUES); GROUND WATER ELEV.; DYNAMIC CONE PENETRATION RESISTANCE PLOT × / STANDARD PENETRATION TEST • (20 40 60 80 100); SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ■ PENETROMETER ▪ LAB VANE (50 100 150 200 250); LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W, WATER CONTENT % (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS % GR. SA. SI. CL.

ELEV. / DEPTH	DESCRIPTION
220.03 / 0.0	Ground Level
	Fill, crushed limestone
219.43 / 0.60	Silt, trace of clay and fine sand; Loose, Grey; Compact, Dense, Brown; Compact, Brownish Grey
216.03 / 4.00	Clay till, silty, trace of sand and gravel; Stiff, Grey
208.33 / 11.70	Silt, trace of sand and clay; Dense, Grey
206.03 / 14.00	Clay till, silty, trace of sand and gravel; Very stiff, Grey

NUMBER	TYPE	N - VALUES	GR.	SA.	SI.	CL.
1	SS	20				
2	SS	35				
3	SS	13	2	3	81	14
4	SS	15				
5	SS	10				
6	SS	13				
7	SS	13				
8	SS	15				
9	SS	10				
10	SS	38				
11	SS	16				

Depth scale: 0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15, 16.5

Elevations: 219.0, 218.0, 217.0, 216.0, 215.0, 214.0, 213.0, 212.0, 211.0, 210.0, 209.0, 208.0, 207.0, 206.0, 205.0, 204.0, 203.0

Continued on Page 2 of 2

PetoMacCallum Ltd.
CONSULTING ENGINEERS

N 4 779 791
E 268 172

RECORD OF BOREHOLE No 8 Cont.

W.P. 604-00-01 LOCATION GLANCASTER ROAD UNDERPASS AT HWY 6 (NEW) ORIGINATED BY M.R.
DIST. ___ HWY. 6 BORING DATE October 31, 2000 COMPILED BY M.R.A.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring CHECKED BY D.W.K.

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	N-VALUES	ELEV.	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)	REMARKS GR.	SA.	SI.	CL.
16.5	Ground Level												
	Continued from Page 1 of 2				203.0								
	Clay till, silty, trace of sand and gravel												
18.0	Very stiff Grey				202.0								
		12	SS	20	201.0								
19.5													
200.0?/20.00	Clay, silty, faintly layered; occ. thin partings of silt				200.0								
21.0					199.0								
	Stiff Grey	13	SS	13	198.0								
22.5					197.0								
24.0					196.0								
		14	SS	14						3	4	47	46
195.0?/25.00	Dolostone Bedrock				195.0								
25.5		15	RC		194.0	1525	97	82	100				
27.0		16	RC		193.0	1525	100	97	100				
191.9?/28.05	End of Borehole				192.0								
28.5					191.0								
30.0													
32.5													
34.0													

Remarks: Upon completion of augering, water at 23.70m, no cave.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT ×; STANDARD PENETRATION TEST ● (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE, ■ PENETROMETER, × LAB VANE (50 100 150 200 250); LIQUID LIMIT W_L, PLASTIC LIMIT W_P, WATER CONTENT W; WATER CONTENT % (20 40 60); UNIT WEIGHT γ kN/m³; GROUND WATER; STRAT. PLOT

Peto MacCallum Ltd.
CONSULTING ENGINEERS

N 4 779 816
E 268 190

RECORD OF BOREHOLE No 9

W.P. 604-00-01 LOCATION GLANCASTER ROAD UNDERPASS AT HWY 6 (NEW) ORIGINATED BY M.R.
DIST. HWY. 6 BORING DATE October 26 & 27, 2000 COMPILED BY M.R.A.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring CHECKED BY D.W.K.

ELEV. / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	N-VALUES	GROUND WATER ELEV.	REMARKS
220.20 / 0.0	Ground Level					
219.60 / 0.60	Fill, clayey silt and gravel					
218.80 / 1.40	Topsoil, silty clay, Dark brown	1	SS	2	219.0	
1.85 / 2.10	Alluvium, clayey silt, trace of sand, Soft, Dark brown	2	SS	10	218.0	19mm Ø PVC Pipe; Water observed after augering to 2.30m
217.30 / 2.90	Silt, trace of clay and sand, Compact, Brown	3	SS	17	217.0	
216.20 / 4.00	Clay, silty, trace of sand, thin lenses of silt, Very stiff, Brown	4	SS	19	216.0	Native Backfill
	Silt, trace of clay and fine sand, Compact, Brownish grey	5	SS	10	215.0	
	Clay till, silty, trace of sand, Stiff, Grey	6	SS	13	214.0	
	Very stiff	7	SS	15	213.0 / 212.0	Bentonite Seal
		8	SS	21	211.0 / 210.0	
		9	SS	18	209.0	GR. SA. SI. CL.: 0 1 65 34
208.50 / 11.70	Clay, silty, with thin layers of silt, Stiff, Grey	10	SS	10	208.0 / 207.0	Filter Gravel
206.20 / 14.00	Clay till, silty, trace of sand, Very stiff, Grey	11	SS	20	206.0 / 205.0 / 204.0	After sample 11, water at 12.70m

Depth scale: 0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15, 16.5

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT ×; STANDARD PENETRATION TEST ●; 20 40 60 80 100; SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE, ■ PENETROMETER, × LAB VANE; 50 100 150 200 250; LIQUID LIMIT W_L; PLASTIC LIMIT W_P; WATER CONTENT W; WATER CONTENT %: 20 40 60; UNIT WEIGHT γ kN/m³

Continued on Page 2 of 2

PetoMacCallum Ltd.
CONSULTING ENGINEERS

N 4 779 816
E 268 190

RECORD OF BOREHOLE №9 Cont.

W.P. 604-00-01 LOCATION GLANCASTER ROAD UNDERPASS AT HWY 6 (NEW) ORIGINATED BY M.R.
DIST. ___ HWY. 6 BORING DATE October 31, 2000 COMPILED BY M.R.A.
DATUM Geodetic BOREHOLE TYPE Continuous Flight Hollow Stem Augers, NQ Rock Coring CHECKED BY D.W.K.

Ground Level

Continued from Page 1 of 2

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	N - VALUES
	Clay till, silty, trace of sand. Very stiff, Grey	12	SS	24
200.20 / 20.00	Clay, silty, occ. thin partings of silt. Firm, Brown	13	SS	7
196.90 / 23.30	Clay till, silty, sandy. Firm, Brown	14	SS	3/150mm
195.20 / 25.00	Dolostone Bedrock	15	RC	
		16	RC	
192.35 / 27.85	End of Borehole			

Depth scale: 16.5, 18.0, 19.5, 21.0, 22.5, 24.0, 25.5, 27.0, 28.5, 30.0, 32.5, 34.0
Ground water elev.: 203.0, 202.0, 201.0, 200.0, 199.0, 198.0, 197.0, 196.0, 195.0, 194.0, 193.0, 192.0, 191.0

DYNAMIC CONE PENETRATION RESISTANCE PLOT ×
STANDARD PENETRATION TEST •
20 40 60 80 100
SHEAR STRENGTH (kPa)
O UNCONFINED + FIELD VANE
■ PENETROMETER × LAB VANE
50 100 150 200 250

LIQUID LIMIT W_L
PLASTIC LIMIT W_P
WATER CONTENT W
WATER CONTENT % 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS % GR. SA. SI. CL.

Sample	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)
15 RC	1320	100	75	100
16 RC	1525	100	93	100

Remarks: 19mm Ø PVC Pipe; Filter Gravel; Screen

Date	Depth to water (m)
Oct. 30	5.30
Nov. 3	5.35
Nov. 14	5.20

RECORD OF BOREHOLE №10

N 4 779 838
E 268 188

W.P. 604-00-01 LOCATION GLANCASTER ROAD UNDERPASS AT HWY 6 (NEW) ORIGINATED BY M.R.

DIST. ___ HWY. 6 BORING DATE October 31, 2000 COMPILED BY M.R.A.

DATUM Geodetic BOREHOLE TYPE Continuous Flight Solid Stem Augers CHECKED BY D.W.K.

SOIL PROFILE

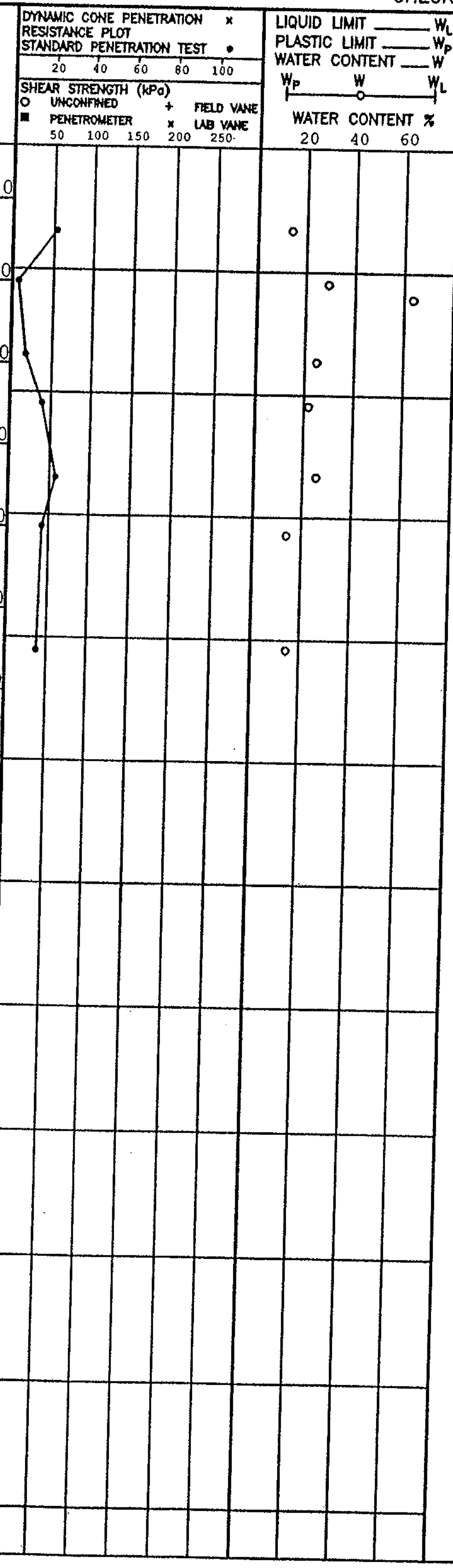
ELEV. / DEPTH	DESCRIPTION
220.65 / 0.00	Ground Level
	Pavement, 40mm asphalt over 860mm crushed limestone
219.75 / 0.90	Fill, silt, trace of clay and fine sand; compact — very loose; Dark brown to brown
1.85	
2.00	
	Topsoil, clayey silt; Black
217.05 / 3.60	Silt, trace of clay and sand, occ. layers of silty clay; Loose to compact; Khaki brown
215.10 / 5.55	Clay, silty, trace of sand; occ. thin partings of silt; Very stiff; Brown
213.95 / 6.70	Clay till, silty, trace of sand; Very stiff; Grey
	End of Borehole

SAMPLES

NUMBER	TYPE	N - VALUES
1	SS	21
2	SS	4
3	SS	8
4	SS	15
5	SS	23
6	SS	18
7	SS	16

GROUND WATER ELEV.: 220.0, 219.0, 218.0, 217.0, 216.0, 215.0, 214.0, 213.0

Depth scale: 0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15, 16.5



UNIT WEIGHT γ kN/m³

REMARKS: GR. SA. SI. CL.

Upon completion of augering, water at 5.70m, no cave.

METRIC

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

CONT No.

WP No. 604-00-01

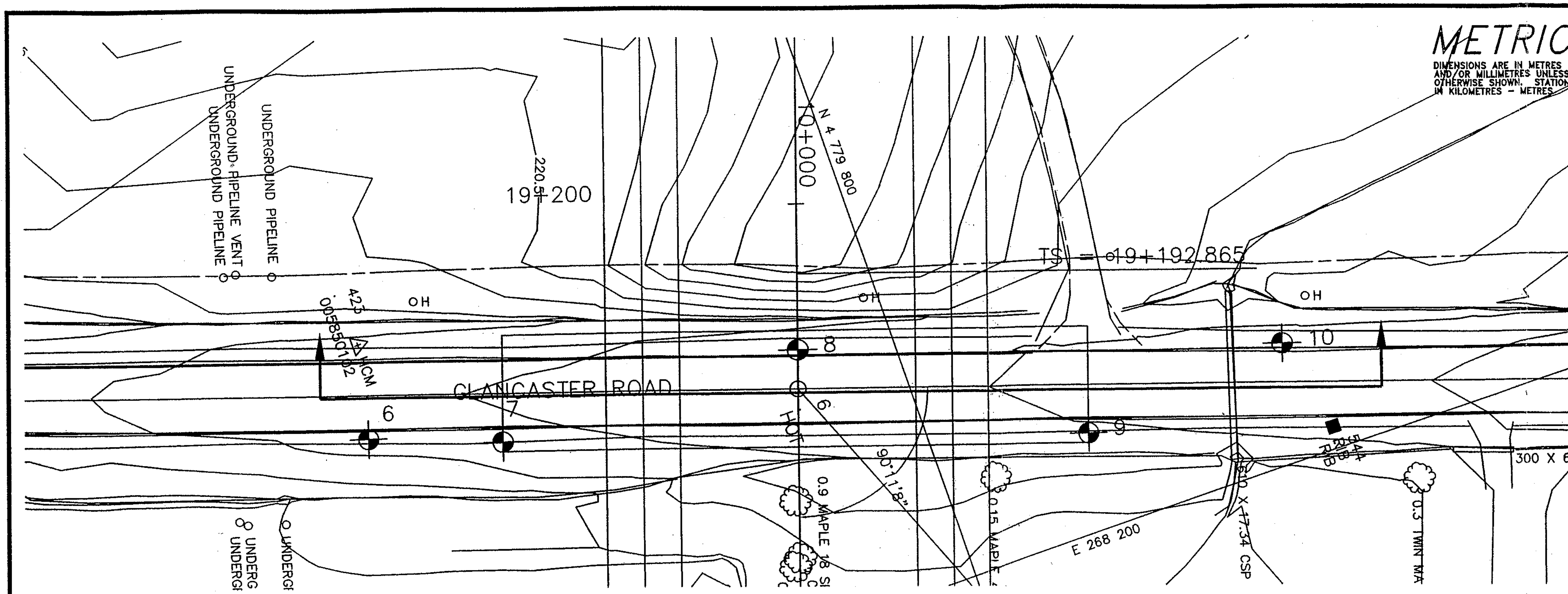
HWY 6 (NEW) FROM HWY 403 TO EXISTING HWY 6

GLANCASTER ROAD UNDERPASS

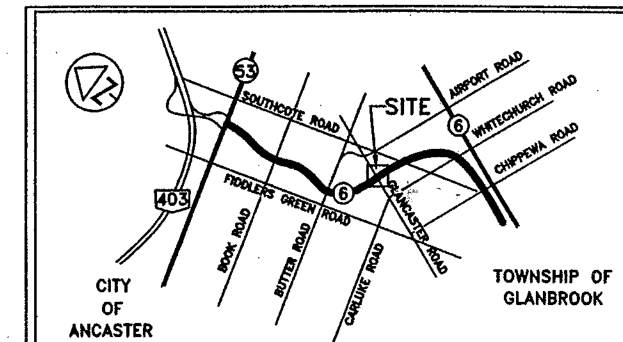
BOREHOLE LOCATION & SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN
SCALE
5 0 5 10m
HORIZONTAL



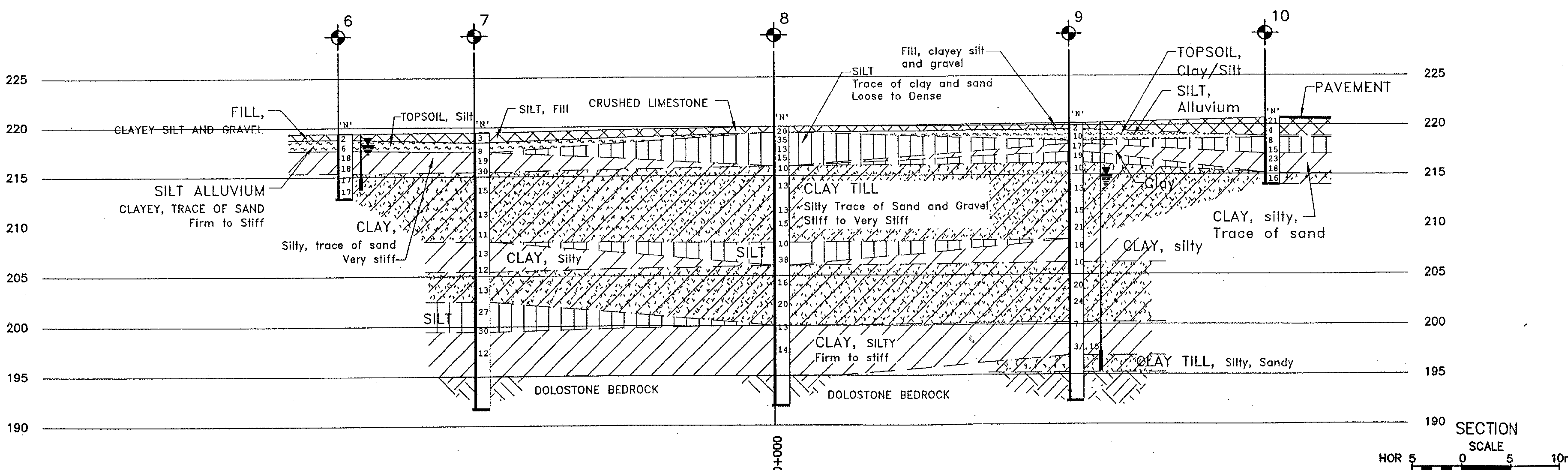
KEY PLAN
N.T.S.

LEGEND

- Borehole
- N — Blows/0.3m (Std. Pen Test, 475 J / blow)
- W L at time of investigation or in piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST
6	219.36	4 779 746	268 166
7	219.47	4 779 759	268 171
8	220.03	4 779 791	268 172
9	220.20	4 779 816	268 190
10	220.65	4 779 838	268 188

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



℄ PROFILE LINE

SECTION
SCALE
HOR 5 0 5 10m
VER 5 0 5 10m

REV.	DATE	BY	

Geocres No.

HWY No. 6			DIST
SUBM'D M.R.A.	CHECKED M.R.A.	DATE 2000 11 14	SITE
DRAWN M.M.	CHECKED D.W.K.	APPROVED D.W.K.	DWG 1

Appendix E

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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 COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

FOUNDATION DESIGN REPORT
HIGHWAY 6 (NEW) AND GLANCASTER ROAD
HAMILTON, ONTARIO
W.P. 604-00-01

Prepared For:

MINISTRY OF TRANSPORTATION – CENTRAL REGION

Prepared by:

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Project: SPT1105
December 3, 2003

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Table of Contents

FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 6 (NEW) AND GLANCASTER ROAD
HAMILTON, ONTARIO
W.P. 604-00-01

5. DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

Post-construction settlements of the newly constructed approach embankments of Highway 6 (New) and Glancaster Road underpass have resulted in deformation of the embankments and damage to the bridge structure. This foundation investigation was carried out to determine possible causes of the settlement.

The new bridge is a two-span concrete structure, each span measuring 30.1 m in length. The width of the bridge is approximately 12 m and the width of the embankment beyond the abutments is approximately 13 m at top. The height of the approach embankments above the original grades is about 8.2 m.

The bridge was designed and constructed as an integral abutment structure, supported on HP 310 x 110 Steel H-piles, with Retained Soil System (RSS) false abutments. After driving all the piles to refusal on bedrock, granular fill (maximum particle size 75 mm) was placed around the pile group at each abutment foundation element to the underside of the concrete foundation wall at El. 222.16 m (north abutment) and El. 222.325 m (south abutment). The piles incorporate a flex zone consisting of 600 mm diameter CSP's extending 3.0 m below these elevations for flex capability of H-piles supporting integral abutments. Each abutment foundation element is supported on six piles (four vertical, two battered at 1:4 on the outside). The central pier is supported on a group of two rows of piles, each row having six piles (i.e. total of 12 piles), battered at 1:4.

In summary, prior to the construction activities, the site was generally underlain by 0.6 to 1.9 m of fill, which was in turn underlain by 0.15 to 0.8 m of clayey topsoil (earth moving and stripping have since changed this picture). Beneath the topsoil, the site is generally underlain by a surficial layer of silt to clayey silt, followed by a major deposit of clayey silt with silty clay layers and very occasional silt seams/lenses. This deposit is generally 22 to 23 m thick and extends to the surface of bedrock at about 24 to 25 m below the ground surface or at about El. 195 m. The bedrock consists of dolostone of good to excellent quality.

The groundwater table appears to be at about El. 218 to 217 m but would be subject to variations.

From an examination of some of the construction records made available to us and our conversations with MTO personnel, the following is a sequence of events during and subsequent to the construction of the bridge structure.

The construction started with pile driving on April 24, 2002, prior to any stripping or fill placement. Pile driving was completed on May 9, 2002 when the contractor moved off the site. It is our understanding that there were some problems during the driving of the piles and an additional pile was driven under each abutment to replace suspect piles.

North abutment area was sub-excavated on May 21, 2002. It is not known whether the granular pads surrounding the piles under the embankments were placed with or without stripping.

In early June, the abutments were formed and concrete poured. Sub-excavation under the approach fills started in late June 2002. The bridge deck was formed in early July and the placement of deck concrete started on July 19.

The construction of the approach embankments was substantially completed towards the end of August and the RSS walls were constructed during the first week of August 2002.

Granular pavement fill placement was completed in mid-September and paving was carried out on September 18, 2002.

The settlement of the pavement adjacent to the approach slabs at the abutment locations was first noticed in the Spring of 2003, after the Spring thaw. Shortly thereafter, longitudinal cracks were noted between the asphalt and concrete curb immediately north and south of the existing bridge structure. During intense rains, water was noted to drain into these cracks and disappear very rapidly. The magnitude of settlements and the thickness of padding are not known. Subsequently, the pavement was padded (see Photograph J-3, Appendix J) and these cracks were covered with asphalt (Photo J-4). But other cracks are still evident (Photographs J-5 and J-6). Surficial instability ranging from erosion to sloughing and slumping is evident along the embankment slopes, particularly on the southwest side (see Photo J-7). Settlement has caused distress to the bridge as evidenced by separation of the approach fill from the bridge structure and surficial cracking of the concrete at the seat (Photographs J-8 and J-9).

5.2 FINDINGS OF THIS INVESTIGATION

5.2.1 REMOVAL OF UNSUITABLE SOILS FROM BENEATH EMBANKMENT FILLS

Boreholes 103, 104, 105 and 106 were drilled from the top of the embankment. The findings of these boreholes are as follows:

Borehole 103 – No topsoil or other organic soils were found, except for minor amounts of organics in the clayey silt immediately beneath the fill.

Borehole 104 – The presence of an about 0.2 m thick layer of organic silty clay (probably original topsoil) was found underlying the fill. In addition, some organic soil was found to be mixed to the underlying 0.2 m thick silt layer.

Borehole 105 – The fill is underlain by a 0.9 m thick layer of topsoil and organic clayey silt layer.

Borehole 106 – No organic soil was encountered.

In summary, the organic soils at the location of Boreholes 103 and 106, which are located further away from the abutment locations, had been stripped while some organic soils were left in place at Boreholes 104 (about 0.4 m thick) and Borehole 105 (about 0.9 m thick). These latter boreholes are located closer to the abutment locations.

5.2.2 COMPACTION OF EMBANKMENT FILLS

We reviewed the results of available laboratory tests carried out by Peto MacCallum Limited and Dufferin Construction Company on samples obtained from test pits for the construction of the embankment fills at this site. Tests conducted between April 4 and August 22, 2002 shows the following results:

Grain-Size Distribution (8 tests)

Gravel:	0-3%
Sand:	5-18%
Silt:	50-78%
Clay:	17-42%

Plasticity Index:	5-16%
Field Moisture Content	12-21%
Standard Proctor Maximum Dry Density (SPMDD):	1602-1835 kg/m^3
Optimum Moisture Content:	15-22%

These values are similar to results obtained on samples (from the embankment fill) in the boreholes drilled for this investigation, including the results of a Standard Proctor Compaction test (see Section 4.1.2 of this report).

We also reviewed the results of available field compaction tests that were carried out during the construction of the embankments. A total of 20 test results is available from tests carried out between Sta. 9+900 and 10+100 (i.e. within one hundred metres of the centre line of the bridge – Sta.10+000). These were carried out generally between August 21 and

August 30, 2002, but other details such as lift thickness and levels (i.e. depth of tests in relation to final or original grade) are not available. The results indicate a degree of compaction generally in excess of 95% of the Standard Proctor Maximum Dry Density (SPMDD) which is acceptable for normal embankment construction (except for three tests which were marginally lower i.e. 93.8-94.9%).

Assessment of the present state of compaction of the embankment fills was made by conducting continuous Standard Penetration tests in four of the boreholes drilled from the top of the embankment (i.e. Boreholes 103, 104, 105 and 106). The results, presented on the individual Record of Borehole Sheets, are also summarized in Figures C-1 and C-1a, Appendix C.

In Boreholes 103 and 106, which were drilled about 40 m away from the abutments, recorded N-values in the clayey silt fill range from 6 to 33 blows/0.3 m but are generally between 18 and 27 blows/0.3 m. In our opinion, the state of compaction (and consistency) of the embankment at these two borehole locations are adequate, except for an approximately 1.5 m thick zone in Borehole 103 at about 1.5 m below the ground surface, where N-values of 6 and 7 were recorded.

In Boreholes 104 and 105, drilled closer to the abutment locations, the recorded values are generally between 3 and 28 blows/0.3 m. In Borehole 104, the majority of the test results are less than 10 blows/0.3 m throughout the depth of embankment with the exception of some zones below 4 m where the values range from 12 to 20 blows/0.3 m. In Borehole 105, the recorded values within the upper 3.5 ±m range from 3 to 5 blows/0.3 m. These values are generally indicative soft consistency. Below about 3.5 m the recorded values range from 8 to 28 blows/0.3 m but are generally in the 13 to 18 blows/0.3 m range. These results indicate that present state of compaction of the embankment fills at these two borehole locations is inadequate, especially within the upper 4 to 5 m. This inadequacy may have been partly caused by such occurrences as disturbance due to construction traffic on the compacted, dilatant clayey silt soil after the placement of each lift, insufficient elapsed time between lifts (i.e. inadequate pore pressure dissipation between lifts), type of compactor used, lateral movement of soil towards the abutment walls after construction, as well as lateral yield of the embankment, particularly in areas closer to the shoulders.

5.2.3 SETTLEMENT OF APPROACH FILLS

The approach fills (i.e. approach embankments) are expected to settle within their own mass, and also due to the settlement of the foundation soils (including the secondary consolidation of the organic soils immediately beneath the embankment fills).

The presence of the bedrock (i.e. rigid base) underlying the overburden soils was taken into consideration for stress distribution to be used in the settlement analysis. A general picture of the vertical stresses is given in Figure F-1, Appendix F.

5.2.3.1 SETTLEMENT OF APPROACH FILLS UNDER THEIR OWN WEIGHT

Approach fills can be expected to undergo settlements due to self weight. These settlements can be expected to vary with such factors as adequacy of compaction when first placed, any disturbance that may occur immediately after the compaction of individual lifts, type of materials used for embankment construction, moisture content of fill when placed, prevailing weather conditions, height of embankments, etc. The time rate of settlement is also dependent on the type of material used (i.e. in general the settlement of granular soils would be largely completed with several weeks of the embankment while with clayey soils this process would take longer).

With the type of materials used for this project, the anticipated settlement of adequately compacted materials (e.g. Borehole 106 location – as evidenced by the results of continuous penetration tests) would in our opinion be of the order of 0.5% of the total embankment height. This would translate into about 40 mm settlement for an embankment height of 8.2 m. This settlement can be expected to be substantially completed within a period of 12 to 16 months after the paving of the road. Softer or less competent zones, as evidenced in Boreholes 103 (1.5 m thick zone), Borehole 104 (approximately 5 m thick zone) and in Borehole 105 (approximately 4 m zone) can be expected to increase the magnitude of settlements by about 5 mm at Borehole 103 (to about 45 mm), by about 15 mm at Borehole 105 (to about 55 mm) and by about 20 mm at Borehole 104 (to about 60 mm).

5.2.3.2 SETTLEMENT OF APPROACH FILLS DUE TO ORGANIC SOILS LEFT IN PLACE AT THE ORIGINAL GROUND SURFACE

In Boreholes 103 and 106, all the organic soils appear to have been stripped while at Boreholes 104 and 105, the presence of approximately 0.4 and 0.9 m thick organic soil was noted, respectively beneath the embankment fills. The settlements due to the compression of these soils under the embankment fill are estimated to be about 10 mm at Borehole 104 and 25 mm at Borehole 105, prior to paving, with several millimeters since paving to date. Secondary consolidation settlements of about 2 to 4 mm can be expected to occur within the next ten years.

5.2.3.3 SETTLEMENT OF FOUNDATIOIN SOILS

The settlement of the surficial silt deposits is expected to be of the order of 8 to 10 mm at Boreholes 103, 104 and 106.

The consolidation settlement of the underlying overburden (i.e. 22 to 23 m thick clayey silt deposit) due to the stresses imposed by the embankment loads was analyzed. To this end, seven one-dimensional consolidation tests were performed on relatively undisturbed samples obtained from various boreholes. Our theoretical calculations indicate a total settlement of about 115 mm can be expected. Based on our calculations, about 40 mm of this consolidation settlement has already occurred to date with an equal amount to come within the next 5 years. The remainder (i.e. 35 mm) can be expected to occur thereafter.

5.2.3.4 DISCUSSION OF SETTLEMENTS

Settlement estimates, as summarized in Table 5.2.3.4.1, indicate that the total settlements since the completion of the embankments to date can be expected to be about 80 to 85 mm at Boreholes 106 and 103, respectively and 101 and 98 mm at Boreholes 104 and 105, respectively.

Of these settlements, about 40 mm would be attributable to the settlement of embankment under its own weight* when the embankment is in an adequately compacted state (such as in Borehole 106) but some can be attributed to lack of compaction or due to disturbance in between lifts, etc. Estimated settlements due to inadequate state of compaction as revealed by the boreholes are as follows:

Borehole 103 – 5 mm
Borehole 104 – 15 mm
Borehole 105 – 20 mm
Borehole 106 – 0 mm (i.e. is in an adequately compacted state)

In our opinion, the presence of organic soils has contributed to a settlement of only several millimeters.

The consolidation settlement of foundation soils since the completion of the embankments to date was calculated to be 40 mm. These consolidation settlements can be expected to continue (as detailed in Section 5.2.3.3 and in Table 5.2.3.4.1).

* In many situations, delaying of the paving of the road immediately adjacent to the bridge structure helps by causing some of these settlements to be effected prior to paving, especially when the embankment fill is a dilatant material and can be expected to generate high pore pressures. Sufficient time lag between lifts when the actual compaction is taking place, as well as the use of a non-vibratory equipment also help to alleviate problems due to future settlements.

Table 5.2.3.4.1
SETTLEMENT ESTIMATES OF APPROACH FILLS
(in mm)

	Borehole 103	**Borehole 104**	**Borehole 105**	**Borehole 106**
Settlement Due to Organic Soils left in place (immediate)*	-	10	25	-
Settlement of Surficial Silt (immediate)	10	8	-	10
Settlement Due to Organic Soil (completion of embankment to date)	-	1	3	-
Settlement of Embankment Fill Under Own Weight (completion of embankment to date)	45	60	55	40
Consolidation Settlement of Clayey Silt Foundation soils (completion of embankment to date)	40	40	40	40
Anticipated Further Consolidation Settlement of Clayey Silt Foundation Soils next 5 years	40	40	40	40
Anticipated Further Consolidation Settlement of Clayey Silt Foundation Soils thereafter (5-20 years)	35	35	35	35
Anticipated Further Secondary Consolidation Settlement of organic soils	-	2	4	-
Total Settlement From Start of Construction to Completion of Embankment	10	18	25	10
Total Settlement From Completion of Embankment to Date	85	101	98	80
Total Settlement From Completion of Embankment to 20 years	160	178	177	155

*immediate: can be expected to be substantially completed during construction.

5.2.4 EMBANKMENT STABILITY

Global stability of embankments as well as possibility of slope failures within the embankment fill were analyzed by means of limit equilibrium method, utilizing the computer program Slope/W. In most cases, Bishop's Simplified method was used, which is known to be slightly conservative (in comparison with more rigorous methods), as in this method, side forces on the individual slices are ignored. Stability was investigated both in short-term (undrained) and long-term (drained) analyses.

The following soil parameters were used in the slope stability analyses for main soil types.

Table 5.2.4.1
Soil Parameters Used in Slope Stability Analyses

Soil Type	**Short-Term Analyses**			**Long-Term Analyses**		
	ϕ **(degrees)**	**c (kPa)**	γ **(kN/m^3)**	ϕ' **(degrees)**	**c' (kPa)**	γ' **(kN/m^3)**
Granular Pavement (Embankment) Fill (crusher run limestone)	39	0	22	39	0	22
Embankment Fill (clayey silt)	0	35-50	20.5	28	4	20.5
Surficial Silt (natural soil)	28	0	19	28	0	19
Clayey silt	0	120-150	21	28	2	21

5.2.4.1 GLOBAL STABILITY

Based on the selected soil parameters, the analyses results indicate no danger of global instability (i.e. deep-seated slope failures) with undrained (short-term) analysis. Results show that when the slip circles are forced to penetrate the foundation soils the safety factors are considerably greater than the normally acceptable value of 1.3. Figure G-1 in Appendix G presents the results of a typical calculation. Since there has been no evidence of a deep-seated slope failure, these results are not unexpected.

With the assumed soil parameters, the long-term global stability was analyzed by forcing potential slip circles to below the embankment depths. The computer analyses indicated a safety factor in excess of 1.5. Therefore, no problems are anticipated concerning global stability in the long-term. A typical computer print out is presented in Figure G-2, Appendix G.

5.2.4.2 EMBANKMENT FILL STABILITY

The boreholes show that clayey silt soils were used to construct the approach embankments. Based on visual observation, the fill appeared to behave like a somewhat cohesive silt, rather than a truly cohesive (i.e. clayey soil). For short-term analysis, an assumed undrained shear strength (c) of 50 kPa was used. When this value is used, the short-term stability presents no problems (i.e. factor of safety is in excess of 2, as shown by typical results presented in Appendix G (Figure G-3). For sensitivity analyses, the undrained shear strength was reduced to 35 kPa and even in this instance, there is no problem with short-term stability within the embankment fills (see Figure G-4 – Appendix G).

Long-term slope stability analysis, using effective soil parameters shown in Table 5.2.4.1, indicates a safety factor of the order of 1.6, which is acceptable. Typical results are shown in Appendix G (Figure G-5).

Figure G-6 through G-10 (Appendix G) present typical results assuming that unsuitable soils were not fully stripped (e.g. Borehole 105) and the slip circles are forced to maximize their path through these layers. In these cases, the calculated safety factors are adequate.

In conclusion, our analyses show that there is no theoretical slope failure problems associated with this site.

From a practical point of view, however, the surficial slope instability, which was noted (e.g. slumping along the west face of the south approach embankment) is attributed to surficial erosion of the approach embankment slopes, as well as the lateral deformation of the embankments. Such lateral deformations can cause spreading and surface cracking. From a visual examination of the soil samples obtained from the boreholes, this is partially attributed to the local clayey silt materials used for the construction of the embankments. This indigenous material is in our opinion not a particularly good material for embankment construction. As was discussed before, it behaves like a silt, in spite of its high clay size particle content.

Unfortunately, such soils are not easily identified on the basis of laboratory tests, except for visual observation 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 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 construction traffic.

In our opinion, if such materials are to be utilized for road embankment construction slightly flatter slopes should be used (e.g. 2 ½H:1V) depending on the height of the embankment or a mid-height berm should be introduced for embankment heights in excess of 5 to 6 m. In addition, the paving of the road immediately adjacent to the bridge abutments should be delayed for a period of at least six weeks.

5.2.5 EXISTING PILES

It is understood that the piles (HP-310x110) supporting the abutment walls were installed prior to the placement of the approach embankment fill. The placement of the embankment fill would cause the soil at the pile locations to move vertically and horizontally away from the approach embankments. The displacements of the soil would 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. 220 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 Glancaster Road was used. The bedrock was assumed at El. 195 m and the depth of the native soil to be 25 m (from El. 195 to 220 m). The height of the embankment ranged from 8.2 to 8.4 m adjacent to the abutment wall to 6.0 m at a location 150 m away from the centre line of Highway 6 (New). The general patterns of the vertical and horizontal displacements as predicted by the finite element analysis are given in Figures H-1 and H-2, Appendix H.

Based on the finite element analysis, the displacements of the native soil (below El. 220 m) at the pile location were obtained, as shown in Figure H-3 (Appendix H). The estimated settlement of soil ranged from 86 mm at ground surface (El. 220 m) to zero at the bedrock surface (El. 195 m). Our analysis shows that maximum horizontal displacements occur about 5 m and 15 m below the ground surface or between El. 215 and 205 m, respectively, with a maximum horizontal displacement of about 37 mm at about El. 210 m. Based on our estimation, about half of the calculated horizontal displacement would have taken place to date. 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 prior to the placement of the embankment fill, 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.

5.2.5.1 AXIAL LOADING ON THE PILES

As indicated in Figure H-3, 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 H-3 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 (Ca), between the pile and the soil, which is related to the shear strength of the soil. The value of the adhesion Ca 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 1210 kN at the tip of the pile (El. 195 m), as shown in Figure H-4, Appendix H. Under the downdrag load, the top of the pile would settle for about 6 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:	1210 kN/pile (at El. 195 m)

5.2.5.2 LATERAL BENDING

As shown in Figure H-3, the soil at the pile location moves horizontally due to the placement of the embankment fill. This will cause the lateral displacement of the pile. Our analysis indicates that the pile is very 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.

Using a finite element model in which the pile is simulated by beam elements (see Figure H-5, Appendix H), bending stress along the depth of the pile is estimated (see Figure H-6, Appendix H). The maximum bending stress of 12 MPa occurs at a depth of about 10 m (El. 210 m). Based on the analysis, we do not anticipate problems due to lateral bending (i.e. lateral yield) of the piles.

---

* 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. 4th Int. Conference on Soil Mechanics and Foundation Eng., London.
- Tomlinson, M. J. (1963). “Foundations Design and Construction.” Wiley, New York.

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.

5.2.5.3 PILE STRESS

The analyses presented above indicate that the pile stresses will result from working load (1600 kN), accumulated downdrag load and bending due to lateral displacement. At a depth of 10 m (El. 210 m), the estimated pile stress is 162 MPa. Based on the analysis results, the maximum stress of the pile due to working load, downdrag load and bending will occur at the pile tip (El. 195 m), as presented below:

Stress due to working load (1600 kN):	113 MPa
Stress due to downdrag (1210 kN) :	86 MPa (at El. 195 m)
Bending stress due to lateral displacement:	0 MPa (at El. 195 m)
Maximum total stress:	199 MPa (at El. 195 m)

The maximum total pile stress is estimated at 199 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 1.76 in terms of yield stress (i.e. 350 MPa ÷ 199 MPa = 1.76).

If the elastic yield range of the stress in the steel pile is exceeded, the pile would not be excepted 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 consolidate under the embankment loads.

It should also be pointed out that the maximum stress of 199 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 199 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 working load, downdrag load and lateral bending is 229 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 significant danger on the structure due to the yield of the piles.

5.3 POSSIBLE REMEDIAL MEASURES

In view of the anticipated continuing foundation settlements, it is our opinion that measures need to be taken to minimize the continued maintenance at the bridge abutment and approach fill interface, as well as to minimize the risk of excessive damage to the abutments.

In our opinion, for this purpose the most expedient method is to reduce the stresses on the foundation soils due to the weight of the approach fills adjacent to the abutments. This will substantially eliminate or reduce any further settlements of the foundation soils under the approach fills adjacent to the abutments, as well as reduce forces on the existing abutments. Reduction of abutment loads will also likely help reduce risks due to bending and over stressing of the piles supporting the abutments.

This can be achieved by replacing the existing embankment fills to a sufficient depth by lightweight fill.

For this purpose, a polystyrene lightweight fill, which is virtually weightless, is considered most effective. A second type of material, which is more economical and practical to implement but much less effective in reducing the stresses, is the use of ultra-light weight granular blast furnace slag. These options are briefly discussed in the following sub-sections of this report.

5.3.1 POLYSTYRENE TYPE OF BACKFILL

Polystrene type of backfill (e.g. Styrofoam H.I.) is a virtually weightless material with a unit weight of generally between 0.2 and 1.2 kN/m^3. This type of light-weight fill will need to be of high density type to sustain traffic loads through the pavement structure (i.e. asphalt and granular base and sub-base materials to be placed over the light-weight fill). In addition, a layer of reinforced concrete is needed to be placed to protect the polystyrene from possible environmental degradation.

Assuming that a 0.9 m thick reinforced concrete, granular pavement fill, asphaltic concrete (combined thickness), excavation to a minimum depth of 4.4 m below the present road surface grade is considered desirable. This will provide an approximately 3.5 m thick light-weight fill zone. If the thickness of pavement required to maintain the road over the light-weight fill needs to be increased (e.g. due to dynamic forces generated by trucks, differential de-icing considerations) then the depth of excavation may need be increased to maintain a minimum selected light-weight fill thickness.

The polystrene fill will need to extend from the interface of the abutments (both north and south abutments) to a horizontal distance of about 14 m and then gradually taper to zero thickness at a horizontal distance of 9 metres beyond this (i.e. total distance of about 23 m). A schematic representation of this approach is given in Figure I-1, Appendix I. Tapering is necessary to reduce the detrimental effects of differential settlements (i.e. embankment beyond the light-weight fill will continue to settle).

The anticipated stress distribution with this approach is given in Figure I-2, Appendix I.

A possible problem with this approach is the presence of RSS walls which extend laterally 8 m beyond each abutment. The feasibility of the approach described here should be discussed with the supplier of the RSS walls for this project (i.e. Durisol).

The requirement of a granular layer beneath the light-weight fill should be checked with the supplier of the light-weight fill. In addition, compaction of the exposed subgrade with a static roller may be necessary before placing the granular fill underlying the light-weight fill, depending on the site conditions. Provision may need to be made to improve the subgrade drainage at the interface where the light-weight fill starts to be tapered off, to prevent accumulation of surface water. If this is considered to be a problem, tapering procedure can be revised. This and other details should be discussed with the supplier as well as drainage experts.

This can be implemented immediately but in order to avoid winter work and to reduce residual settlements after the placement of the light-weight fill, it is considered that it would be possible to wait until May 2004. With this approach, the anticipated further settlements would be greatly reduced. For example, if a 3.5 m thick polystyrene light-weight fill is used, further settlements would be limited to about 10 mm, about 4 mm of which would be expected to occur within the next five years after implementation (i.e. after May 2004), with the remainder thereafter.

If additional foundation support to this bridge abutments is required, methods such as mini-piles could be available and we will be pleased to look into this aspect, if required.

During this period (i.e. until the placement of light-weight fill), however, we recommend that settlement of the bridge as well as the approach fills be monitored monthly and if the settlement of the bridge exceeds a cumulative value of 5 mm (i.e. during the next five to seven months) then immediate action should be taken. This could consist of relieving stresses immediately adjacent to the abutments, as discussed above. The monitoring can be done by reliable surveying methods, ensuring that the benchmark is absolutely fixed and reliable (i.e. not settling or heaving). We recommend that the monitoring be continued at bi-monthly (i.e. 6 times a year) after the placement of the light-weight fill for a period of at least five years. In addition, the yield and rotation of the abutments should also be monitored.

5.3.2 PELLETIZED SLAG BACKFILL

If the use of polystyrene is considered too costly or impractical to implement then ultra-light weight pellitized slag backfill can also be considered. The compacted bulk unit weight of this material is 11.5 kN/m^3 which is less than earth fill. It is, however, heavier than polystyrene type material and is, therefore, much less effective in reducing stresses. If a soil replacement of 3.6 m is effected using this material (i.e., 3.6 m x (20.5-11.5)kN/m^3). It is anticipated that this will reduce further settlements to about 25 to 30 mm.

As was discussed in the preceding section, monitoring would be required.

5.3.3 'DO NOTHING' OPTION

If some risk is acceptable, 'do-nothing' option would be available, with proper monitoring and maintenance.

All these options and in particular the do-nothing option should be discussed with the Structural Engineer for the bridge structure.

6.0 CLOSURE

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

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Appendix F

Stress Distribution

SPT1105 - Glancaster Road at Highway 6 (New), Hamilton, ON
Stress Analysis - Cross Section
Fill: E=20MPa, v=0.35, gamma=20.5kN/m3
Native: E=20MPa, v=0.35
Embankment: Height = 8.2m, Top width = 13 m; Slope 2H:1V

Vertical Stress Contour due to Embankment Fill (kPa)

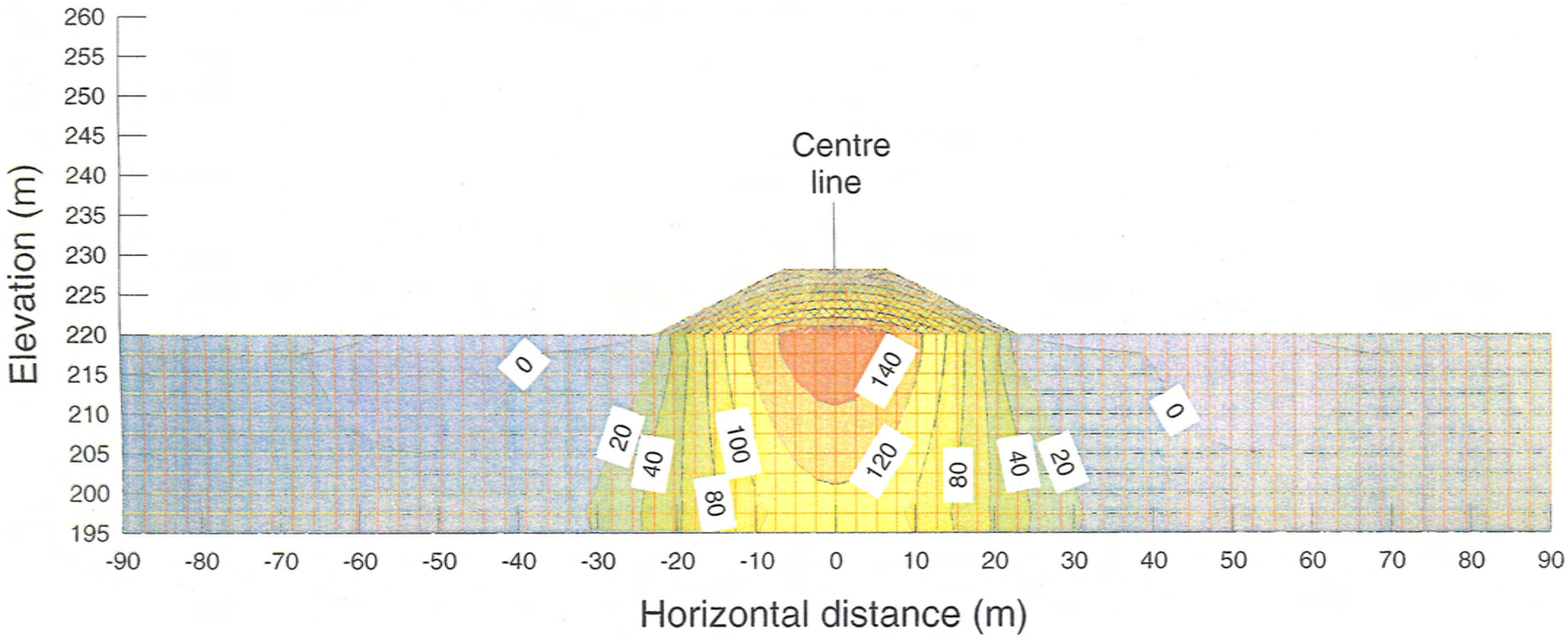


Figure F-1

Appendix G

Slope Stability Analysis Results

SPT 1105 - Highway 6 (New) and Glancaster Road
Hamilton, Ontraio

Undrained Case: Total Stress Analysis

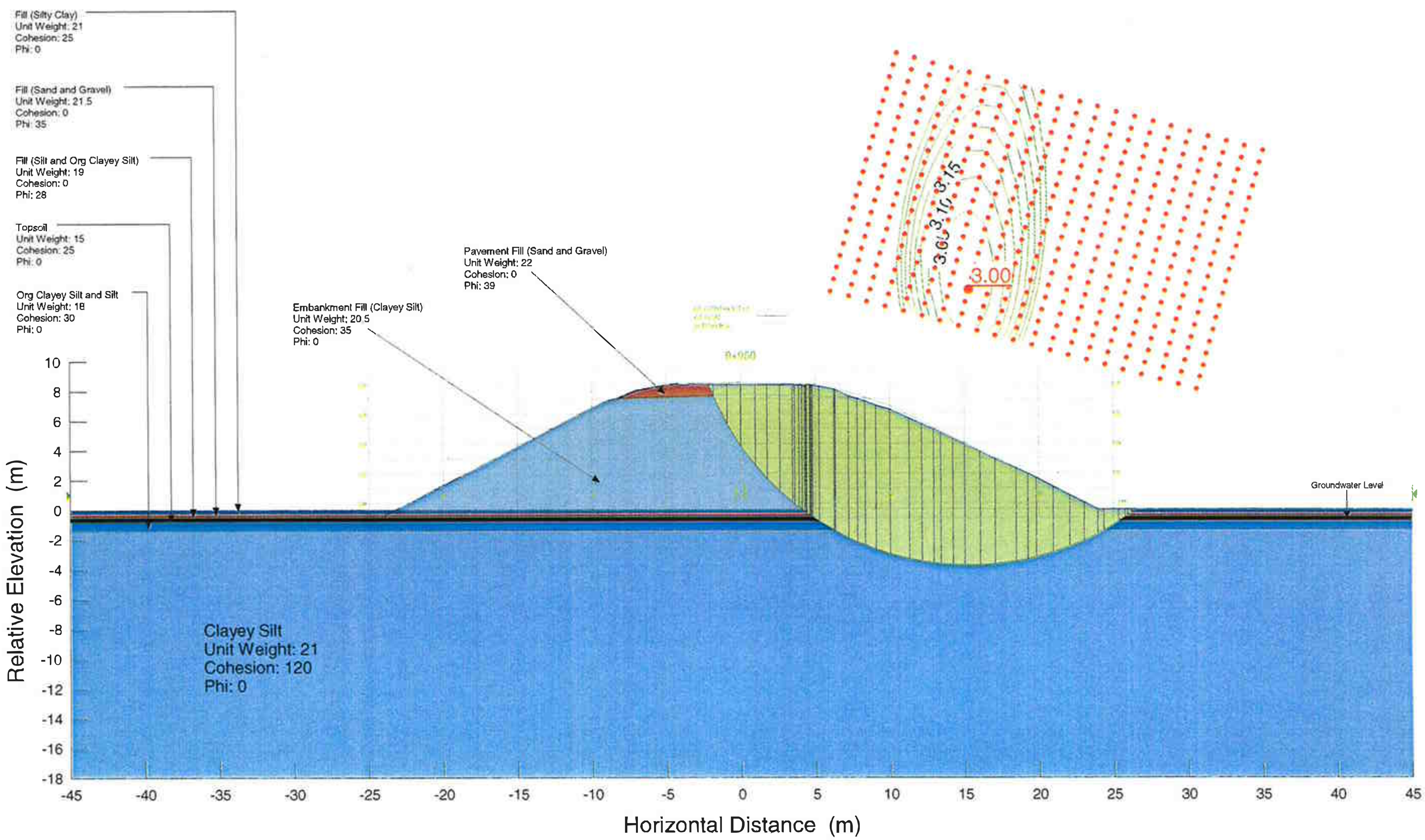


Figure G-1

SPT 1105 - Highway 6 (New) and Glancaster Road
Hamilton, Ontraio

Drained Case: Effective Stress Analysis

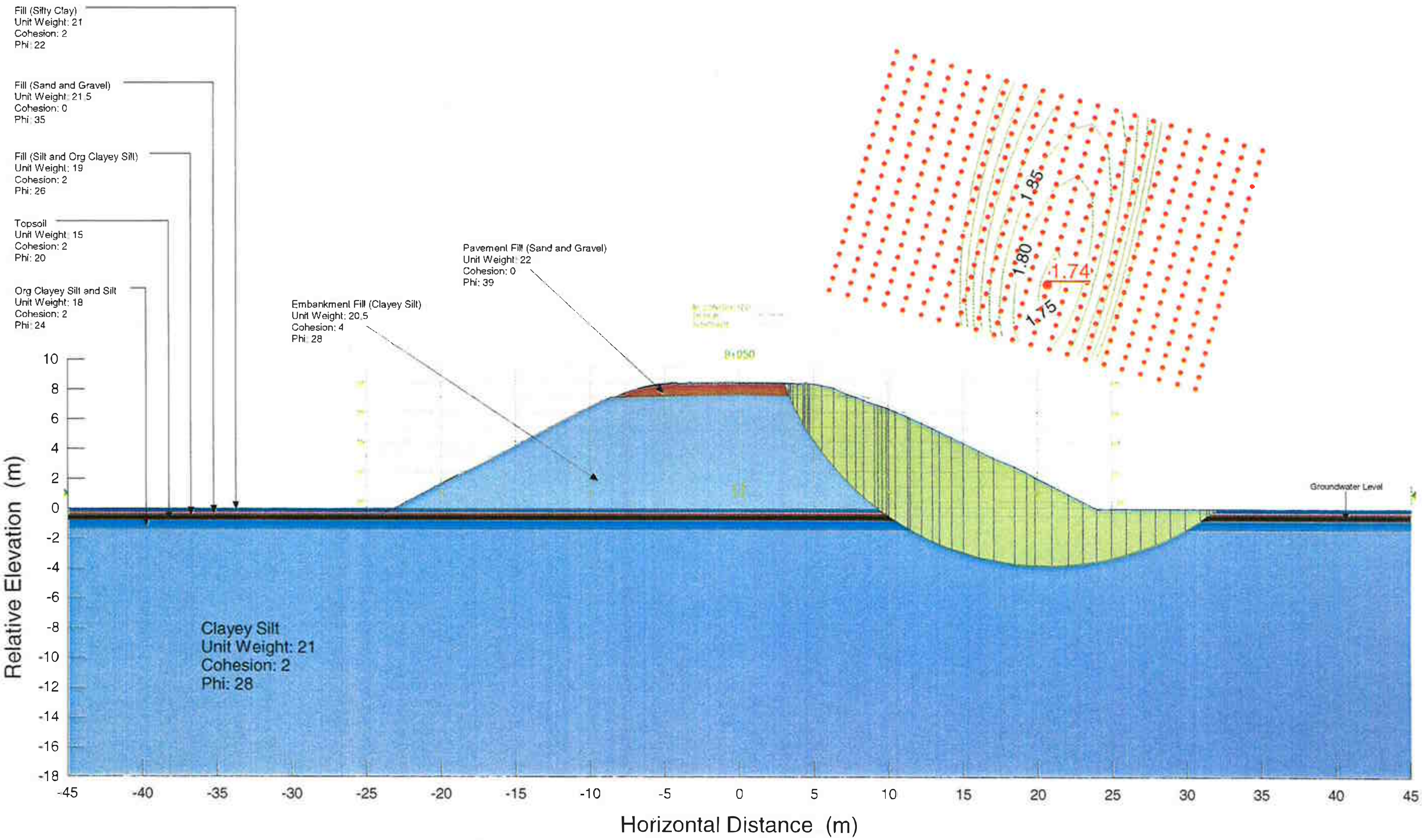


Figure G-2

SPT 1105 - Highway 6 (New) and Glancaster Road
Hamilton, Ontraio

Undrained Case: Total Stress Analysis

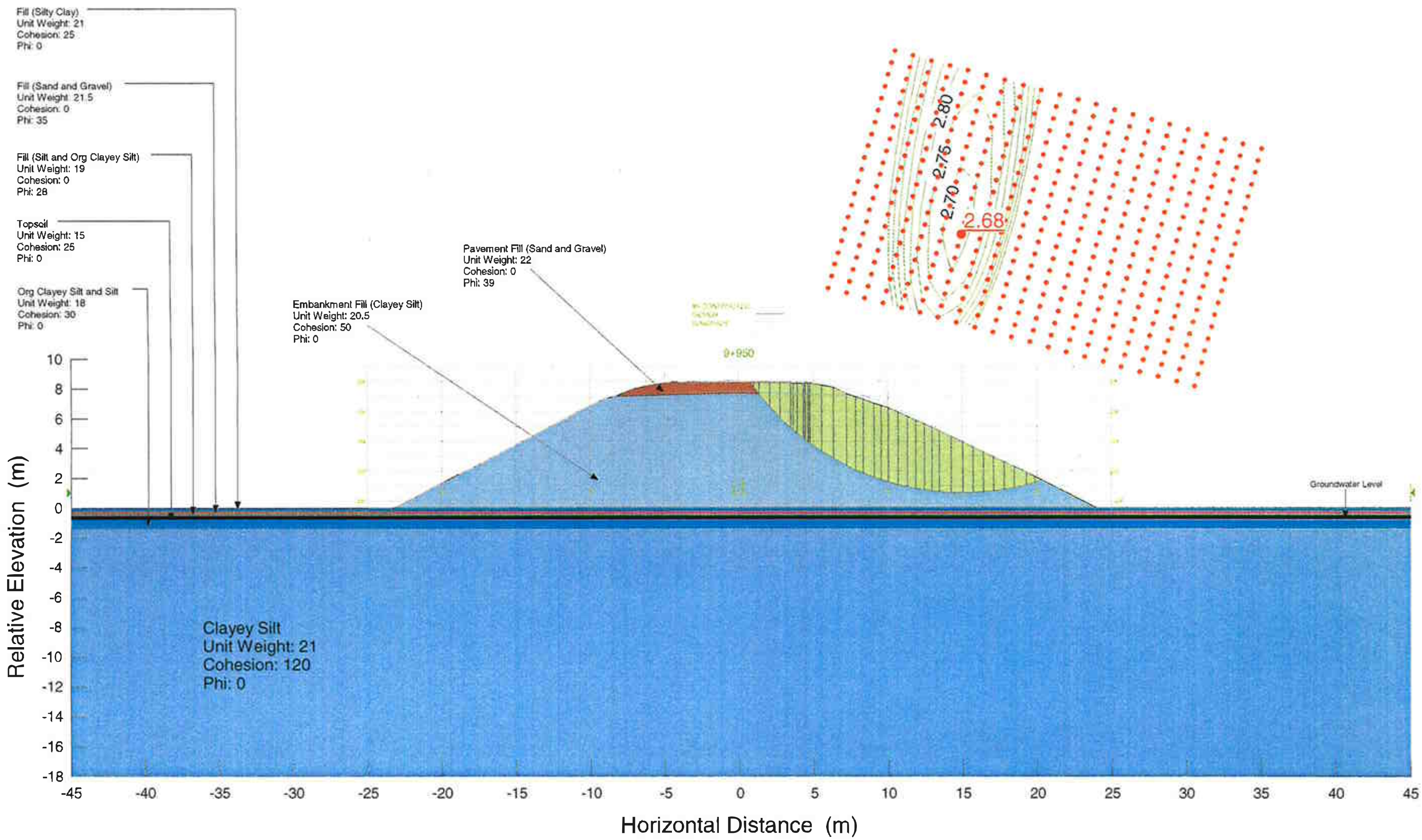


Figure G-3

SPT 1105 - Highway 6 (New) and Glancaster Road
Hamilton, Ontraio

Undrained Case: Total Stress Analysis

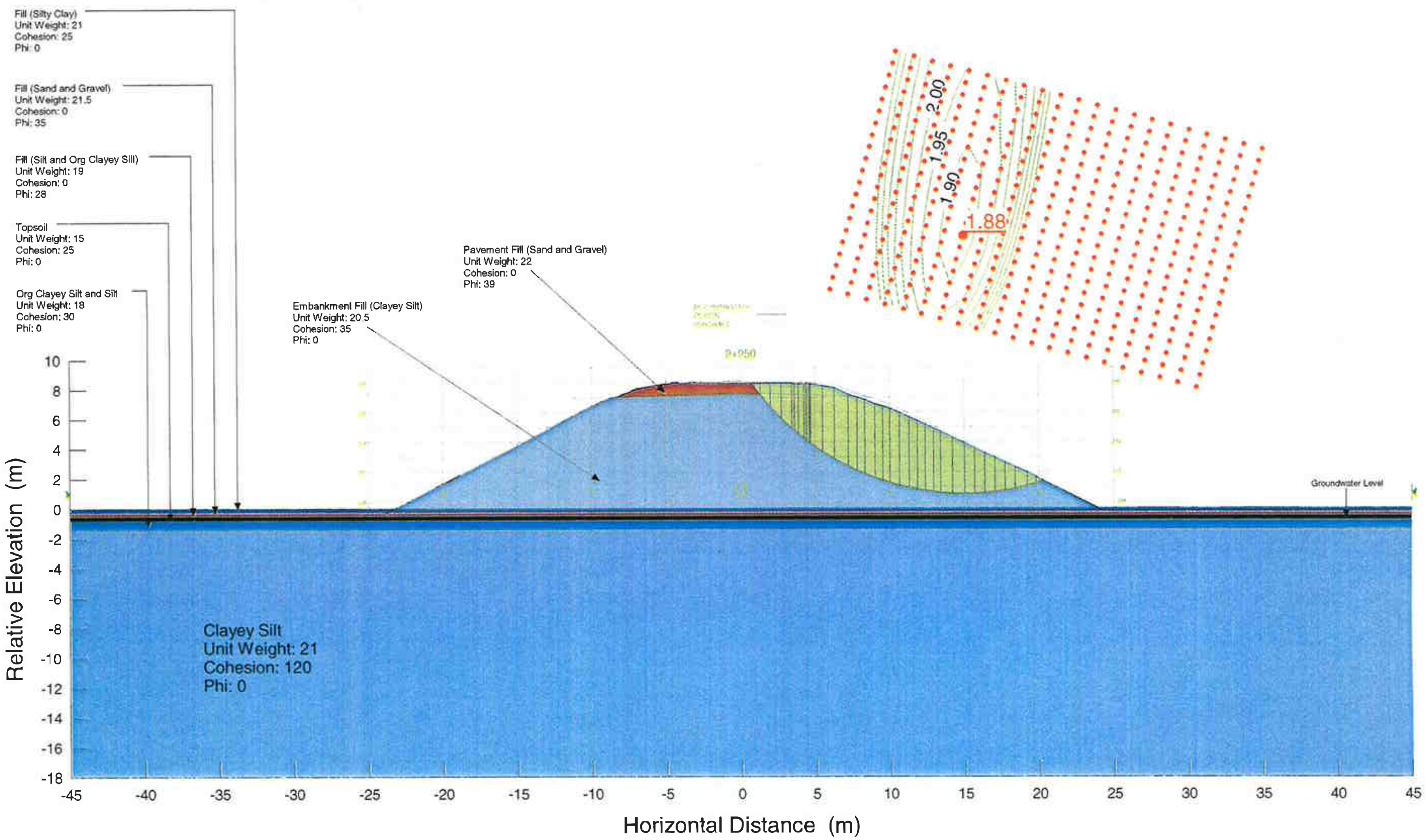


Figure G-4

SPT 1105 - Highway 6 (New) and Glancaster Road
Hamilton, Ontraio

Drained Case: Effective Stress Analysis

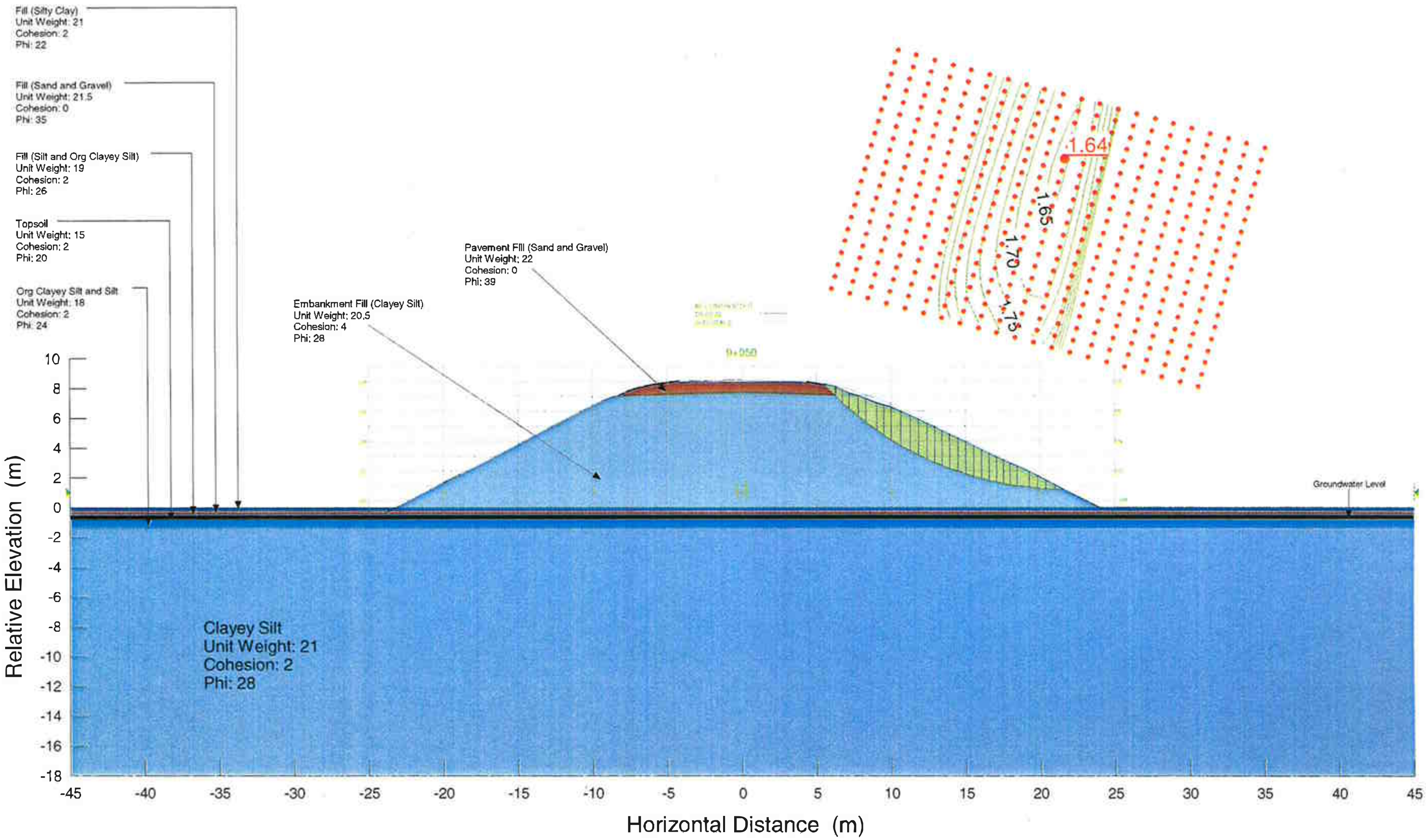


Figure G-5

SPT 1105 - Highway 6 (New) and Glancaster Road
Hamilton, Ontraio

Undrained Case: Total Stress Analysis

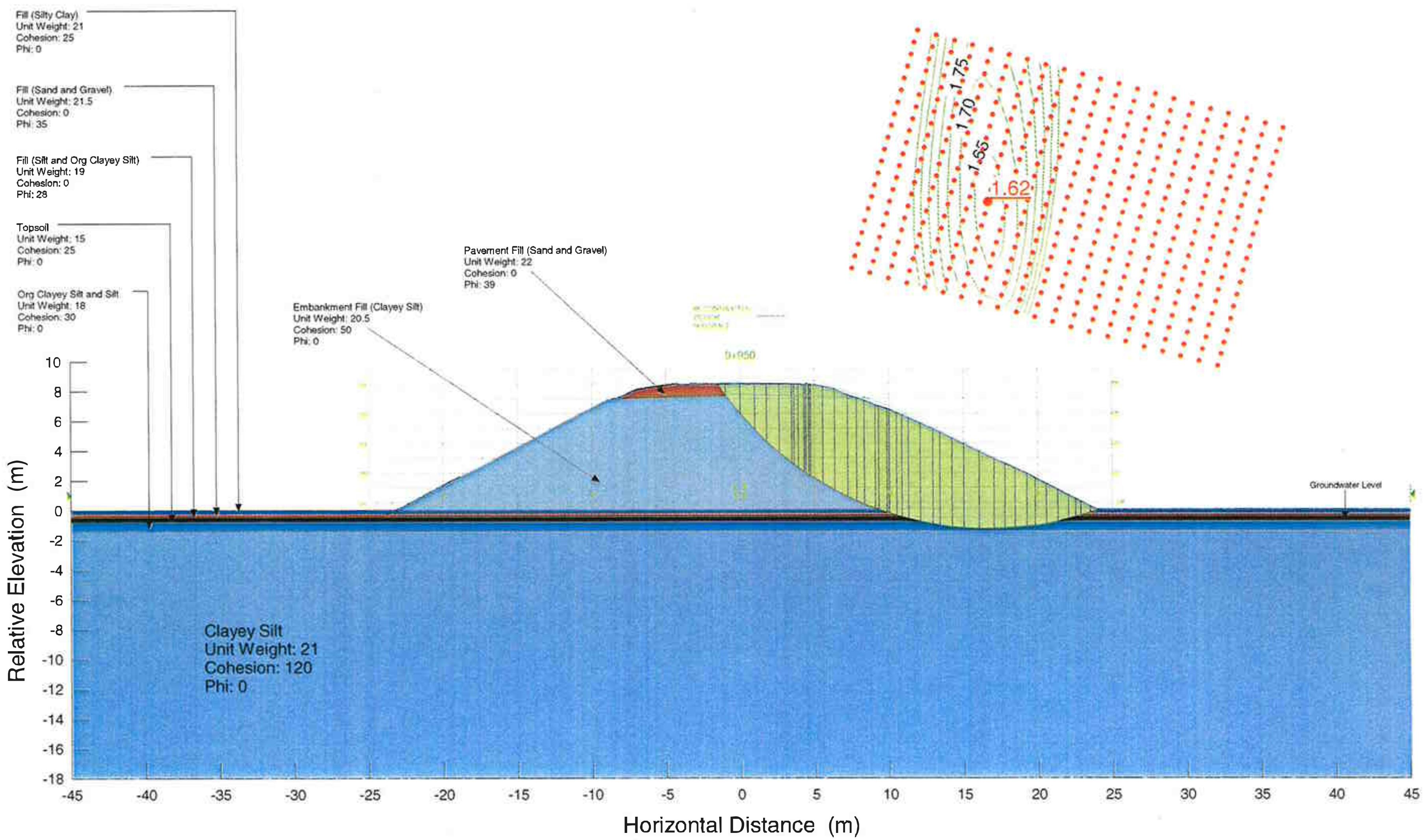


Figure G-6

SPT 1105 - Highway 6 (New) and Glancaster Road
Hamilton, Ontraio

Undrained Case: Total Stress Analysis

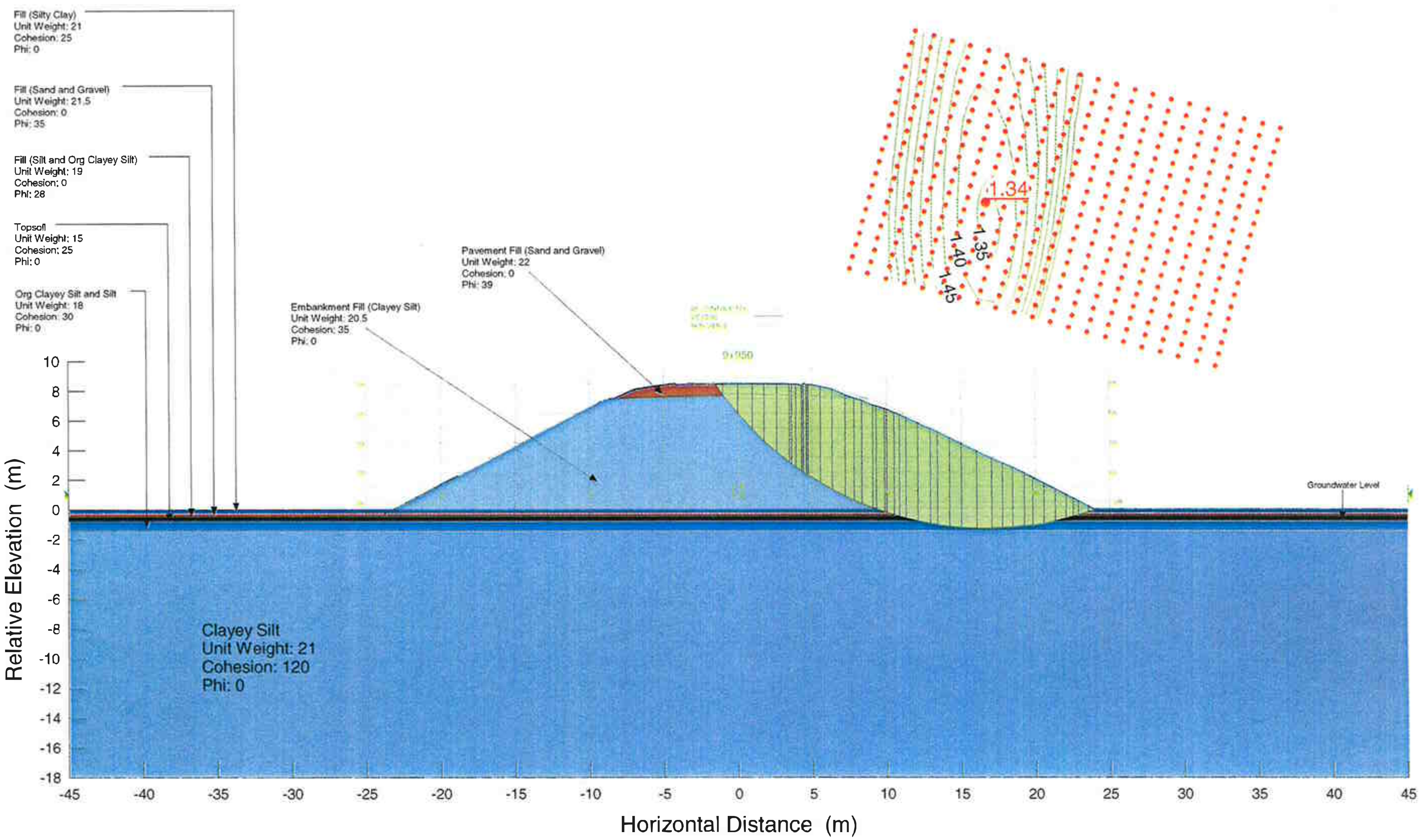


Figure G-7

SPT 1105 - Highway 6 (New) and Glancaster Road
Hamilton, Ontraio

Drained Case: Effective Stress Analysis

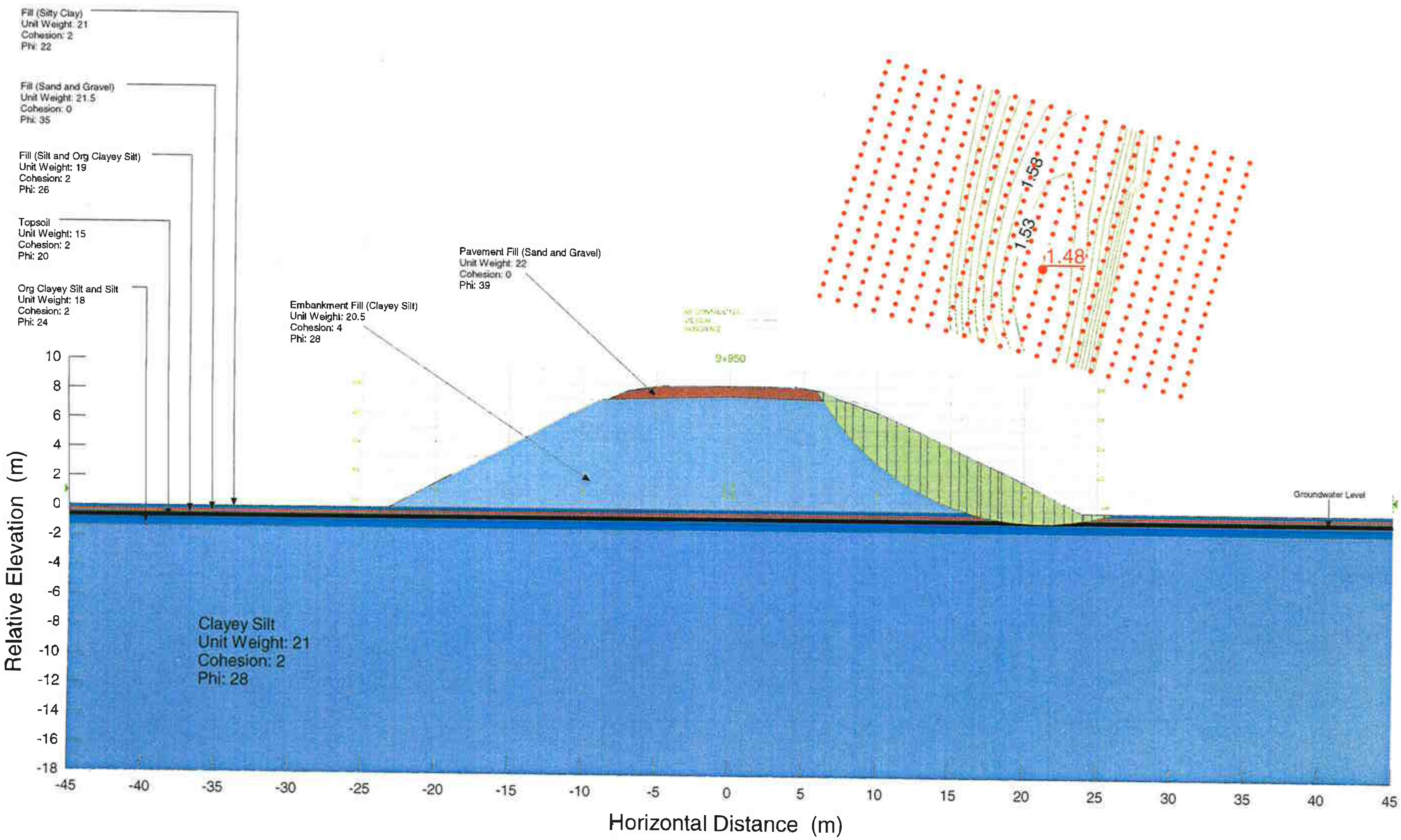


Figure G-8

SPT 1105 - Highway 6 (New) and Glancaster Road
Hamilton, Ontraio

Undrained Case: Total Stress Analysis

Analysis Method: Janbu

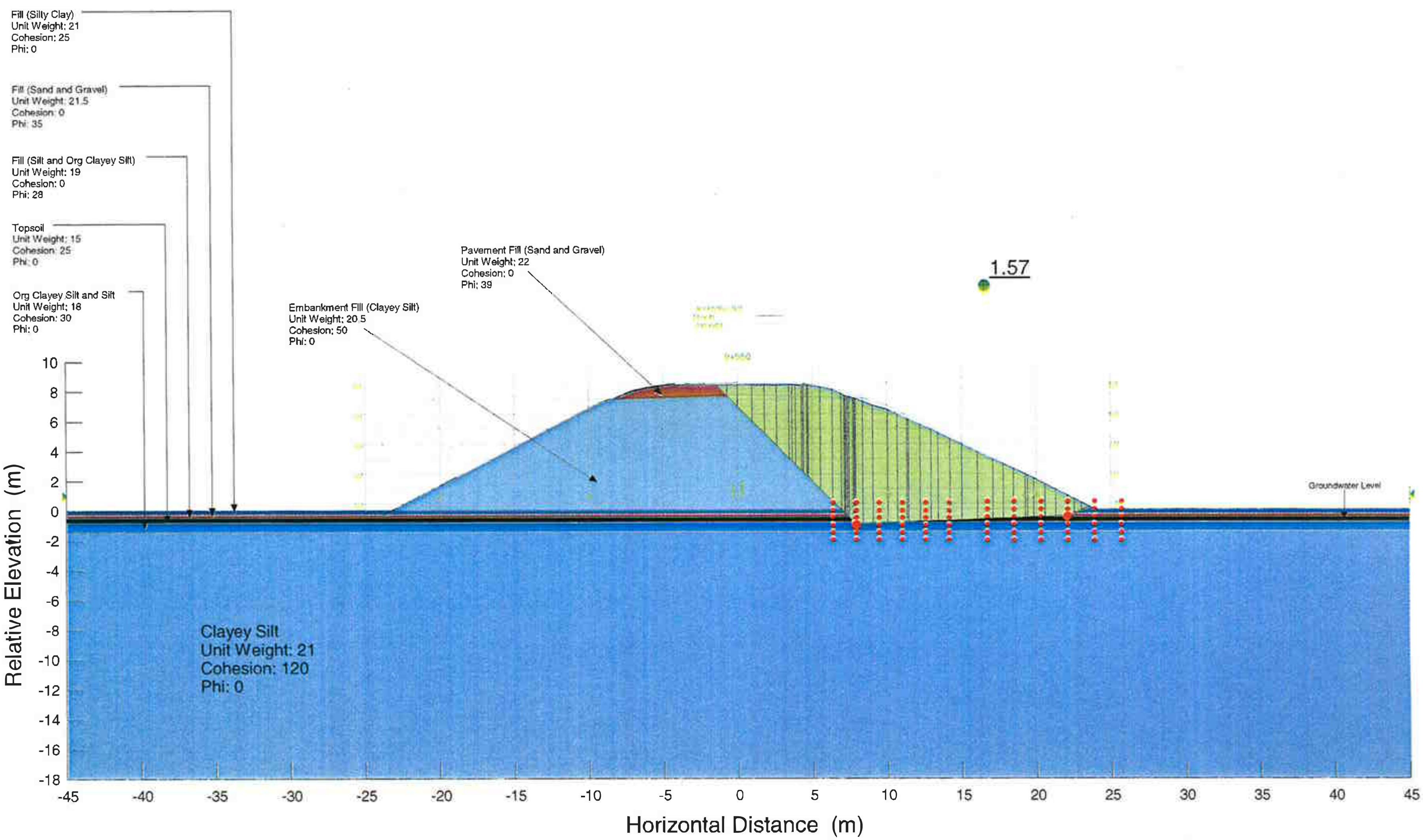


Figure G-9

SPT 1105 - Highway 6 (New) and Glancaster Road
Hamilton, Ontraio

Undrained Case: Total Stress Analysis

Analysis Method: Janbu

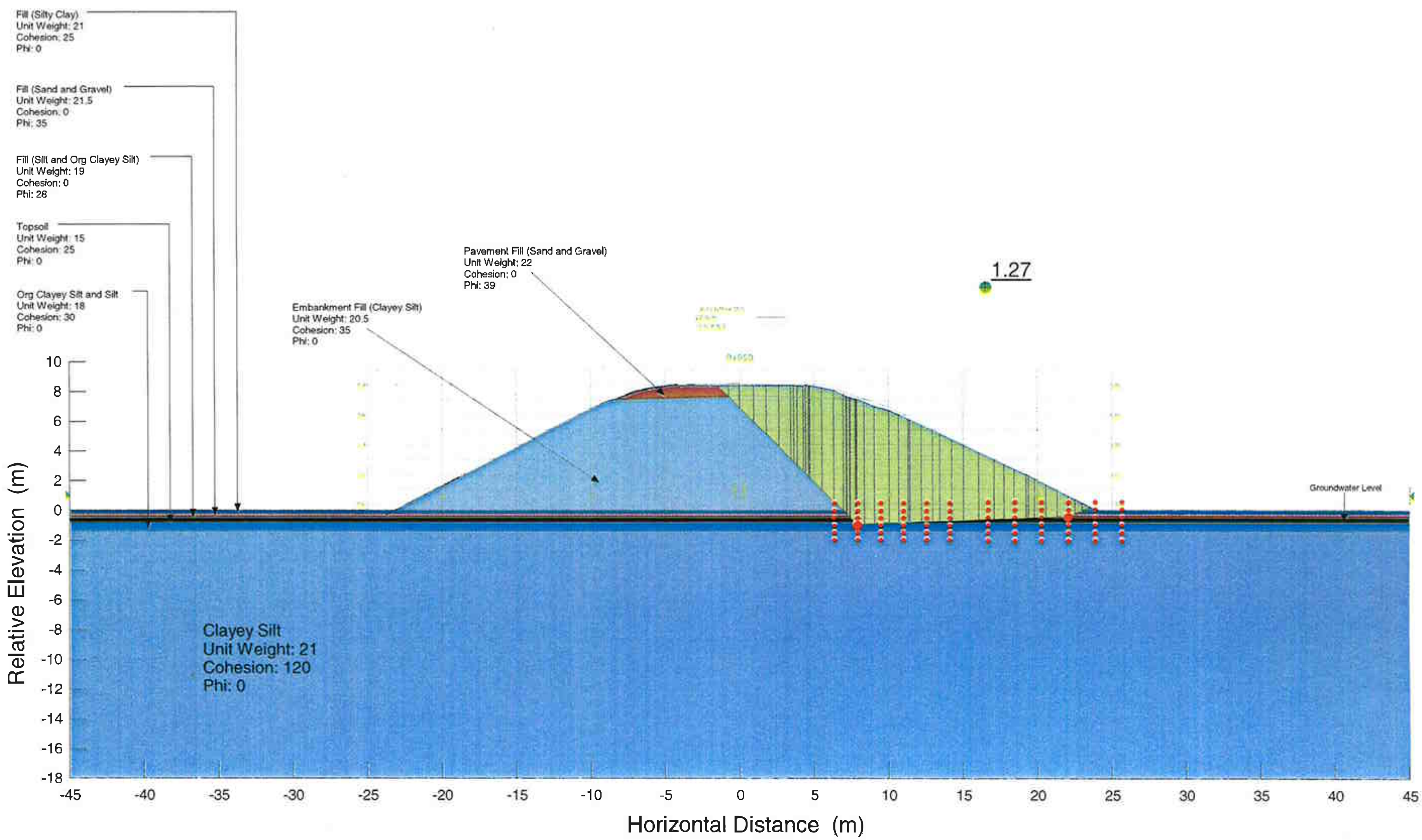


Figure G-10

Appendix H

Effects on Piles

SPT1105 - Glancaster Road at Highway 6 (New), Hamilton, ON
Settlement Analysis
Fill: E=20MPa, v=0.35, gamma=20.5kN/m3
Native: E=20MPa, v=0.35

Contour of Vertical Displacement (m)

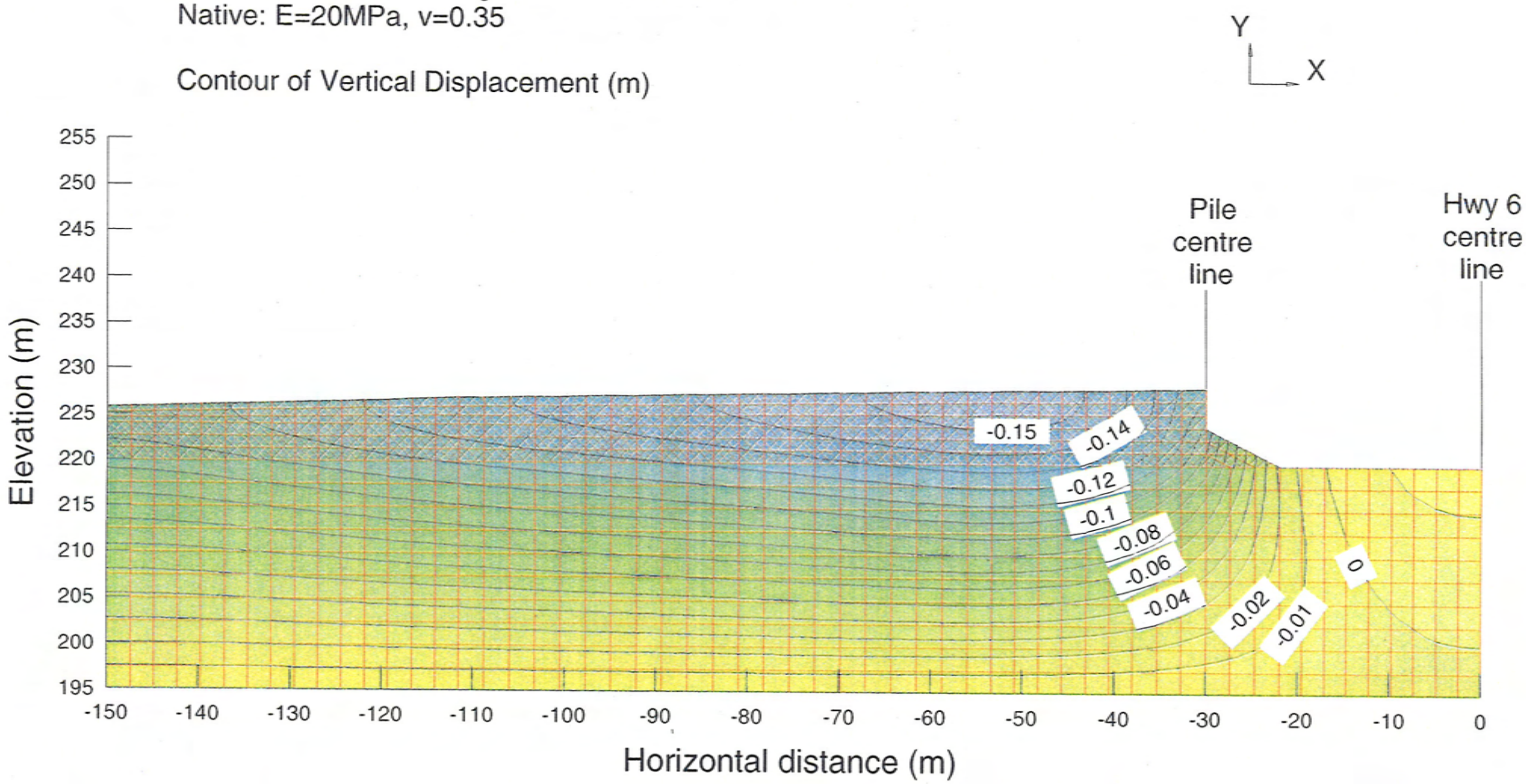


Figure H-1

SPT1105 - Glancaster Road at Highway 6 (New), Hamilton, ON
Settlement Analysis
Fill: E=20MPa, v=0.35, gamma=20.5kN/m3
Native: E=20MPa, v=0.35

Contour of Horizontal Displacement (m)

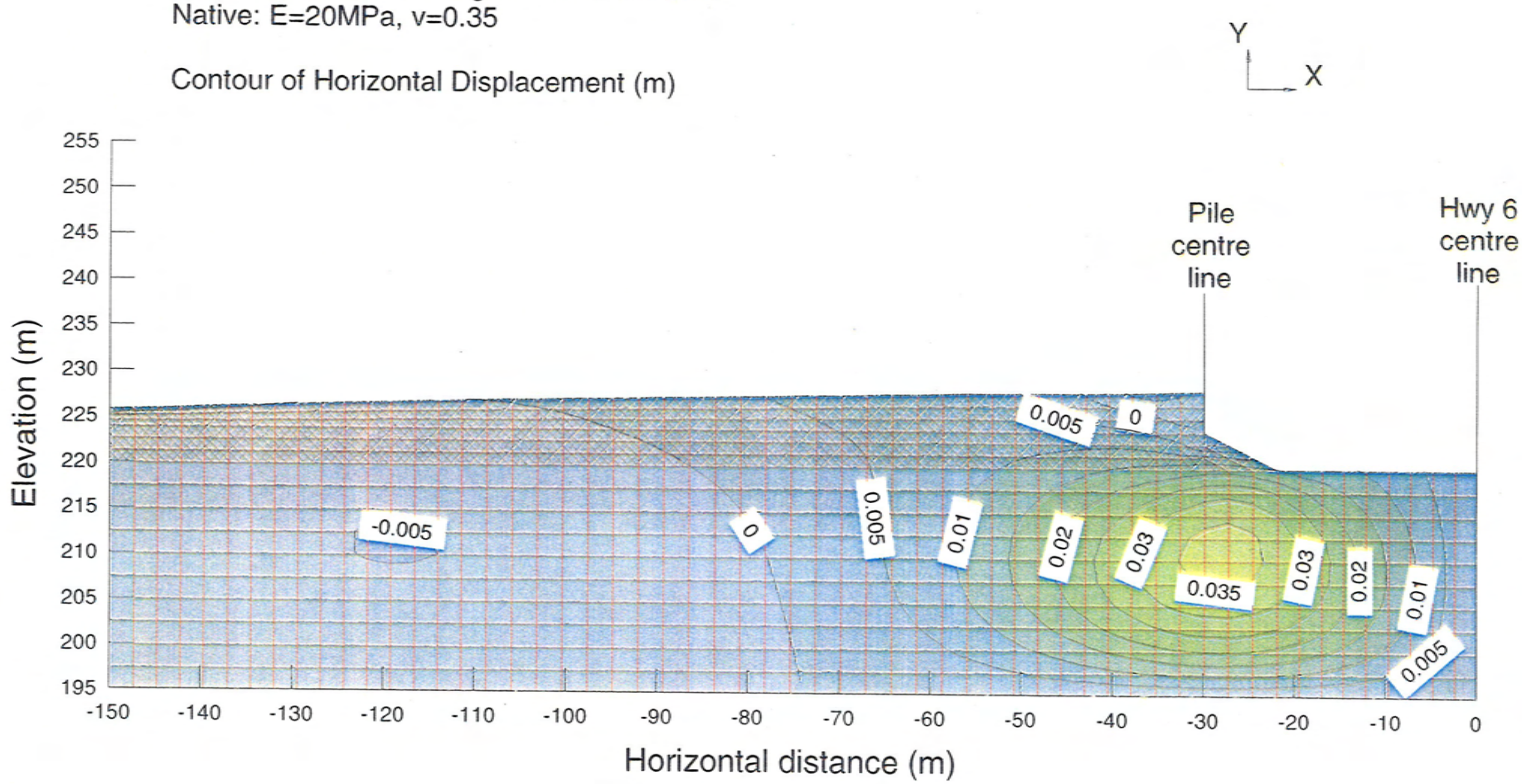


Figure H-2

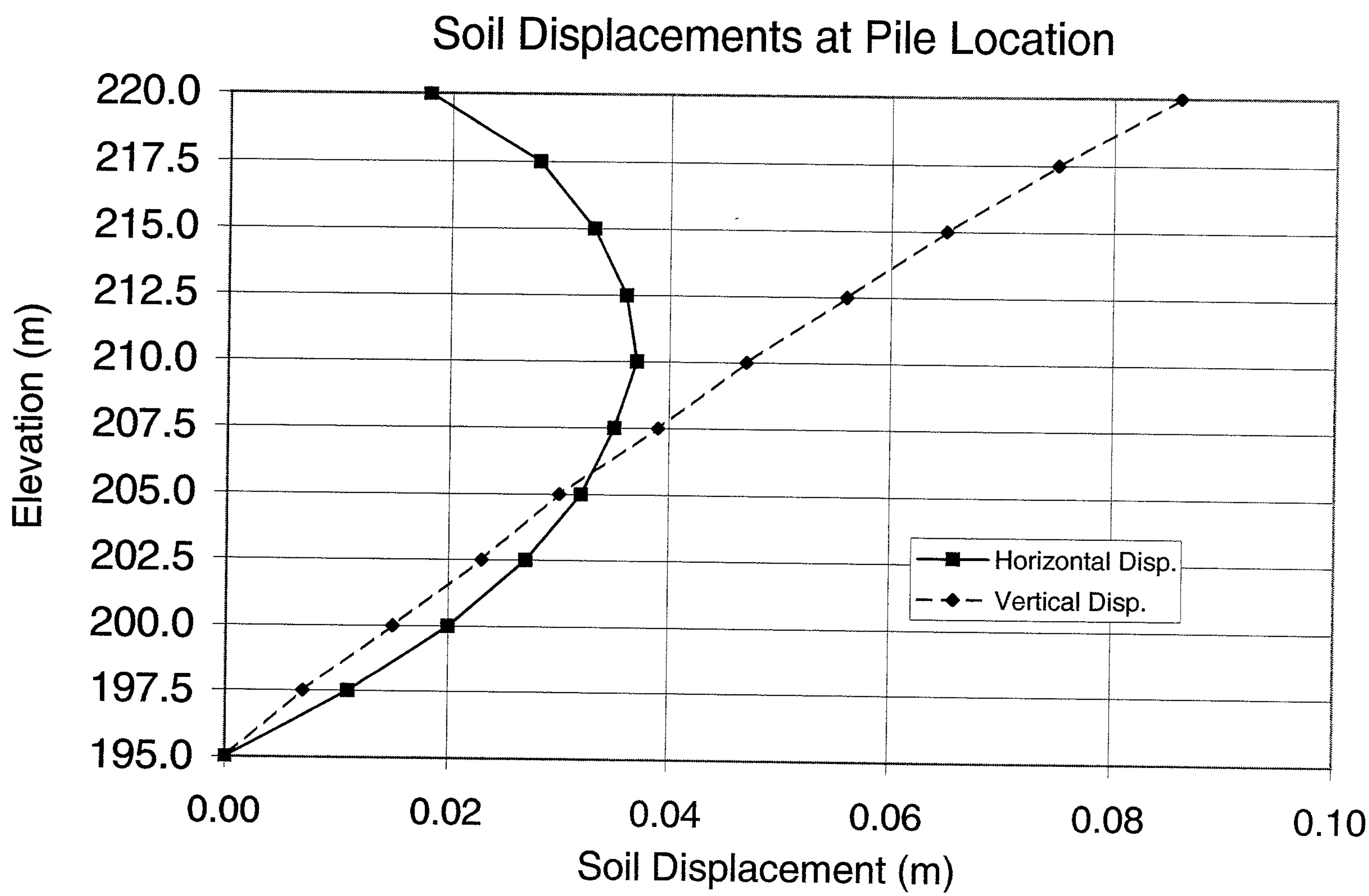
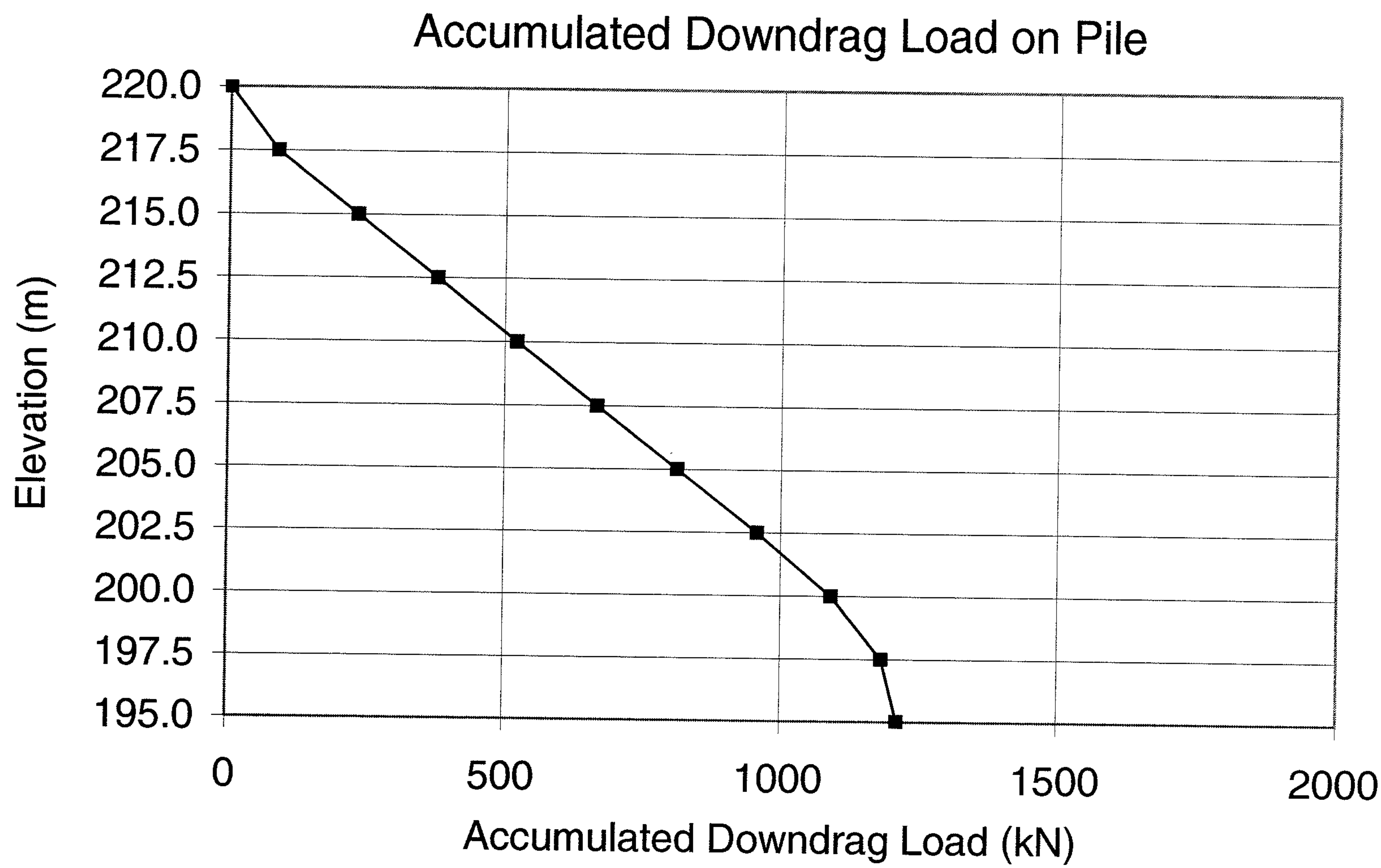

Soil Displacements at Pile Location
220.0
217.5
215.0
212.5
210.0
207.5
205.0
202.5
200.0
197.5
195.0
Elevation (m)
0.00
0.02
0.04
0.06
0.08
0.10
Soil Displacement (m)
Horizontal Disp.
Vertical Disp.

Figure H-3



Accumulated Downdrag Load on Pile
Elevation (m)
220.0
217.5
215.0
212.5
210.0
207.5
205.0
202.5
200.0
197.5
195.0
0
500
1000
1500
2000
Accumulated Downdrag Load (kN)

Figure H-4

SPT1105 - Glancaster Road and Highway 6 (New), Hamilton, ON
Pile Displacement Analysis due to Lateral Soil Load

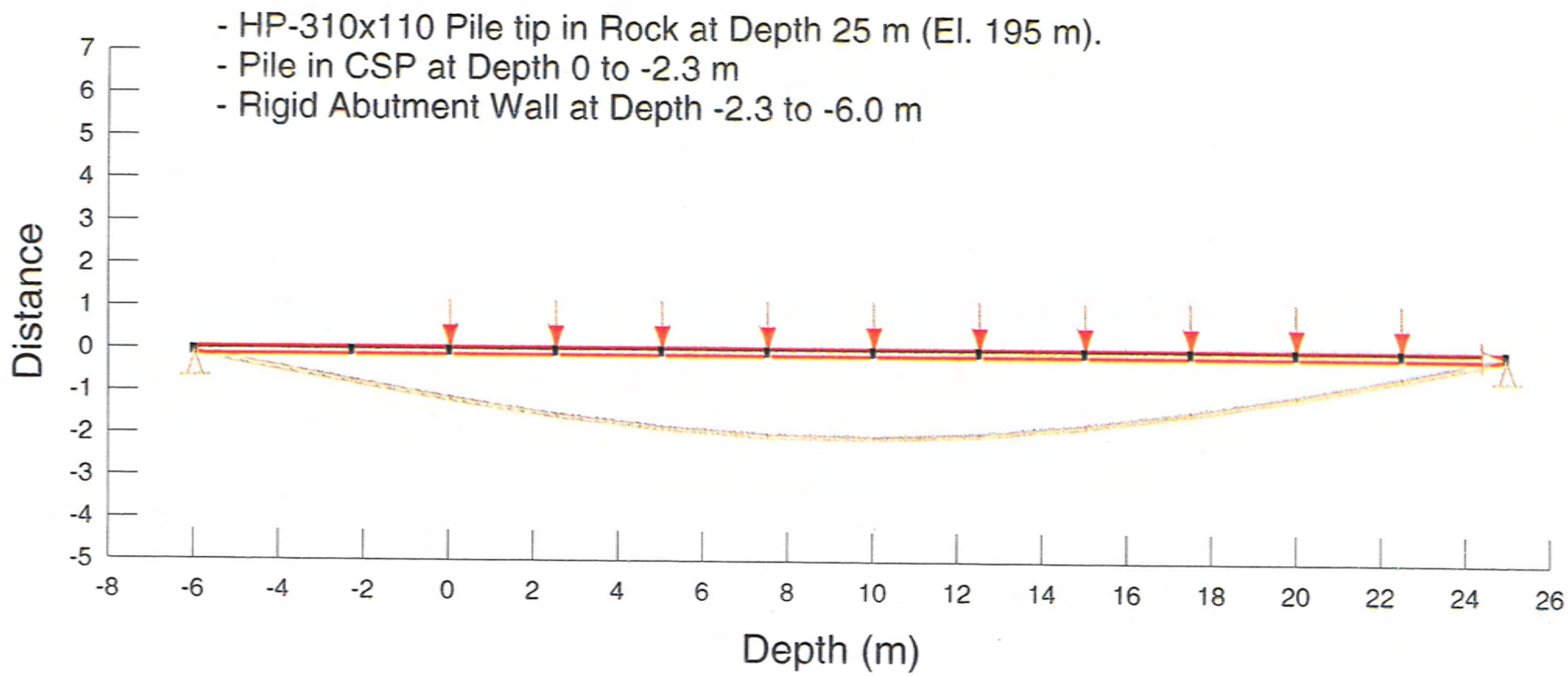


Figure H-5

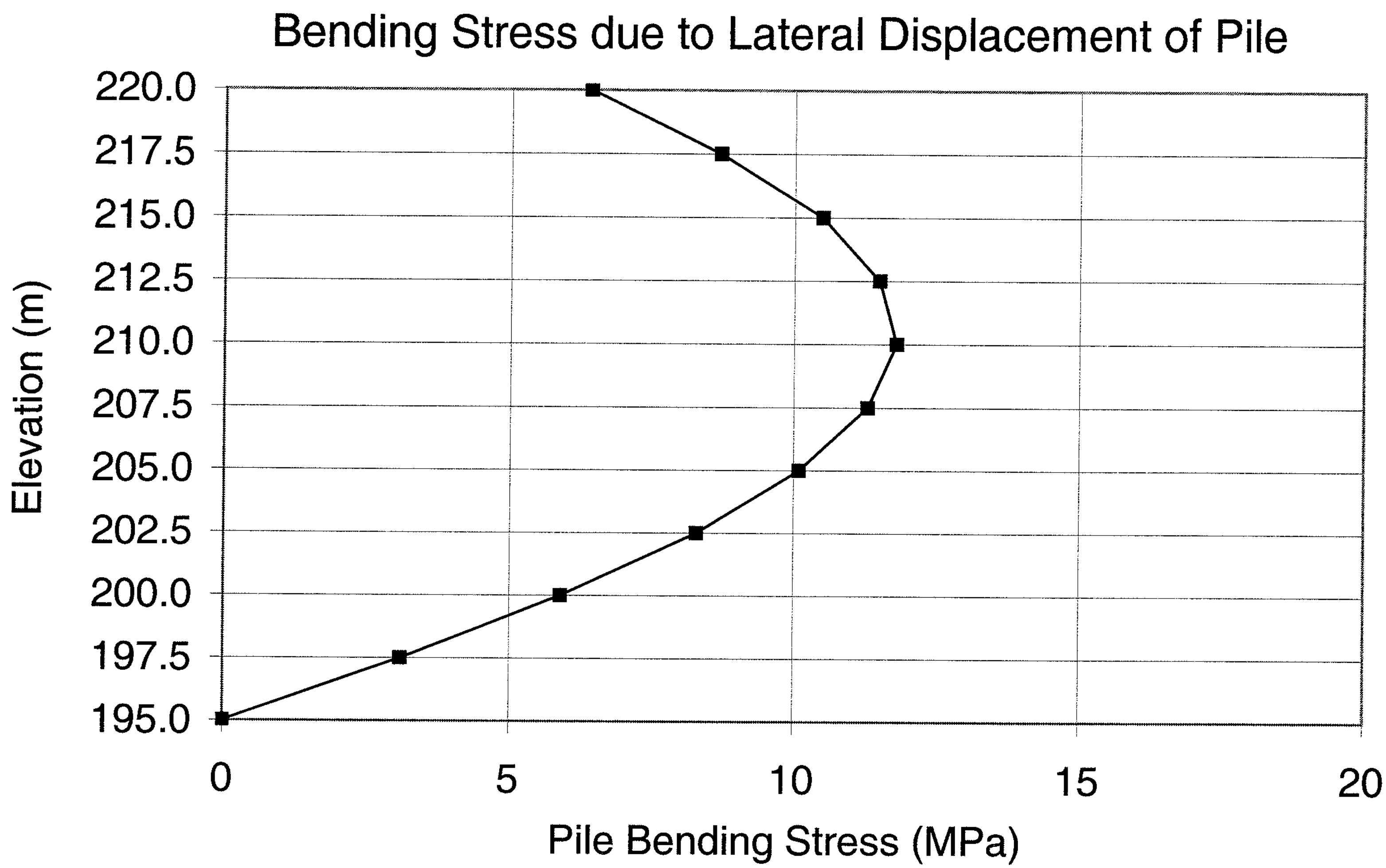
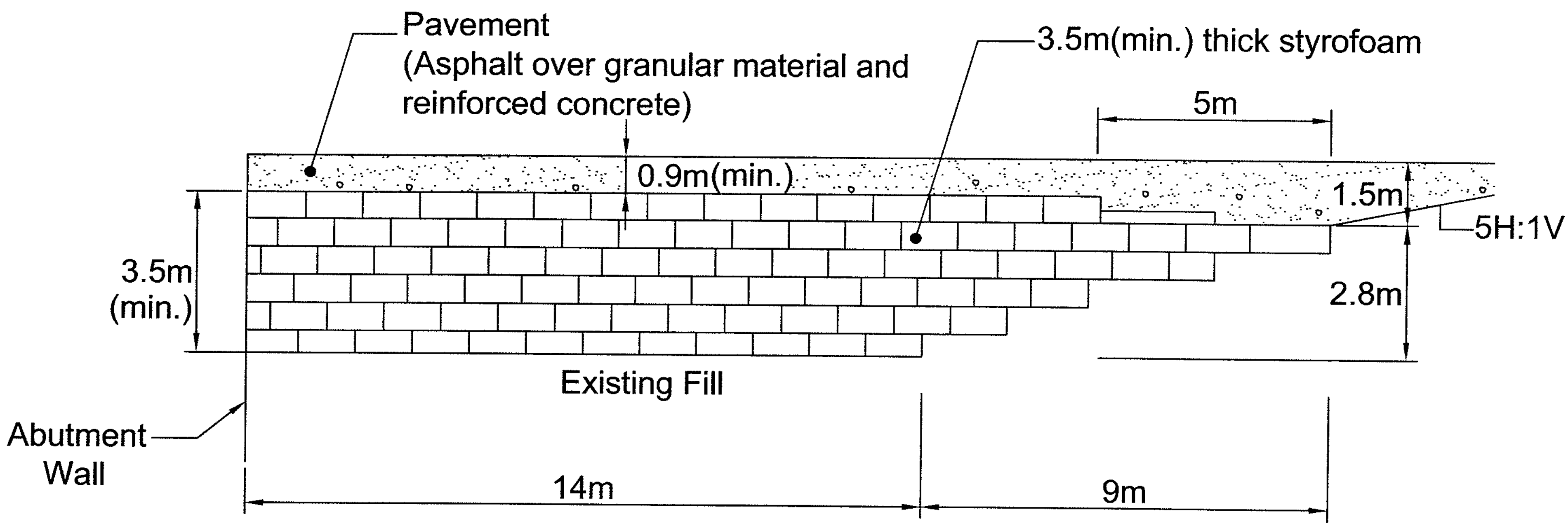

Bending Stress due to Lateral Displacement of Pile
Elevation (m)
220.0
217.5
215.0
212.5
210.0
207.5
205.0
202.5
200.0
197.5
195.0
0
5
10
15
20
Pile Bending Stress (MPa)

Figure H-6

Appendix I

Remedial Measures

SPT 1105



Conceptual Diagram for Styrofoam Fill
Placement Adjacent to the Abutment Walls
(N.T.S)

Figure I-1

SPT1105 - Glancaster Road at Highway 6 (New), Hamilton, ON
Stress Analysis - Cross Section
Surface Granular: E=20, v=0.35, gamma=22kN/m3
Light weight material: E=20, v=0.2, gamma=0.3kN/m3
Fill: E=20MPa, v=0.35, gamma=20.5kN/m3
Native: E=20MPa, v=0.35
Embankment: Height = 8.2m, Top width = 13 m; Slope 2H:1V

Vertical Stress Contour (kPa) due to Embankment Fill with 3.5m Light Weight

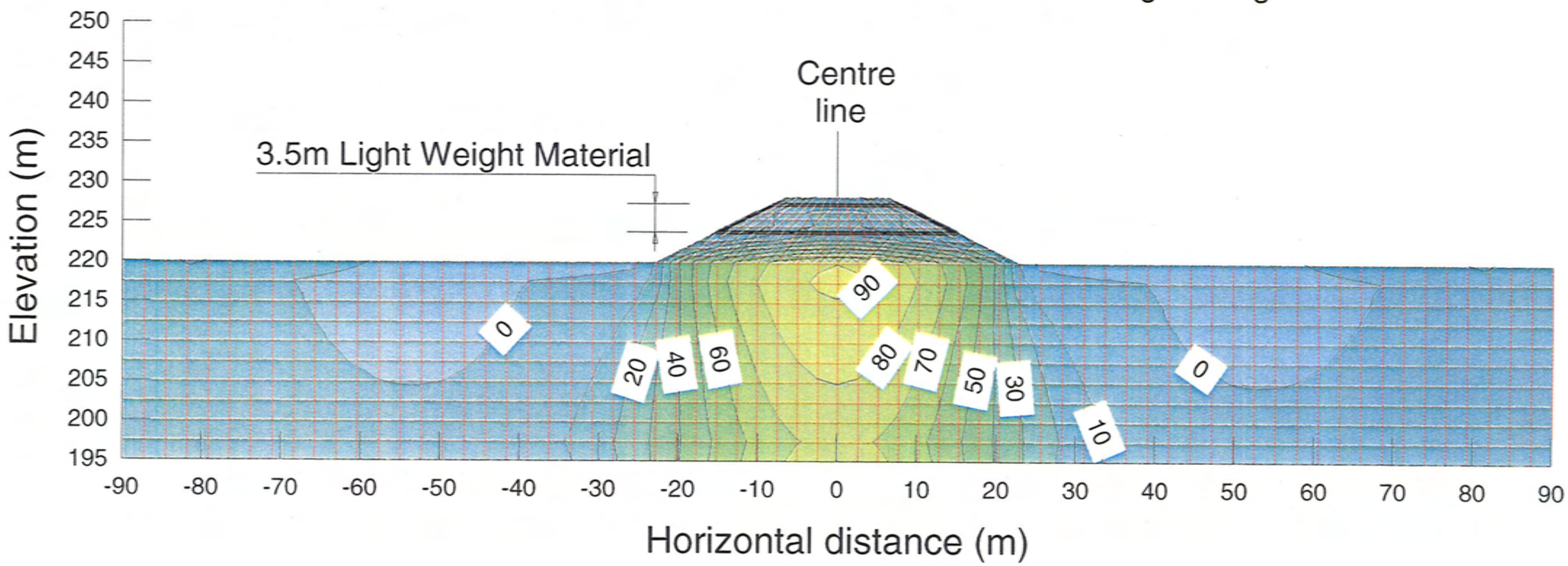


Figure I-2

Appendix J

Field Photographs (August-October 2003)

Figure J-1 Bridge View, Looking West
(Photograph Date : October 18, 2003)

Figure J-2 North Abutment, Looking West
(Photograph Date : October 18, 2003)

Figure J-3 Asphalt Patching Adjacent to North Abutment
(Photograph Date : August 01, 2003)

Figure J-4 Sealing of Cracks Adjacent to Asphalt Patching
Near North Abutment
(Photograph Date : August 01, 2003)

Figure J-5 Crack Along North-West Shoulder
(Photograph Date : August 01, 2003)

Figure J-6 South of South Abutment, West Shoulder Area,
Various Cracks
(Photograph Date : October 18, 2003)

Figure J-7 Slumping and Erosion Along
West Embankment Slope, South of South Abutment
(Photograph Date : August 01, 2003)

Figure J-8
North-West Abutment Area
(Photograph Date : August 01, 2003)

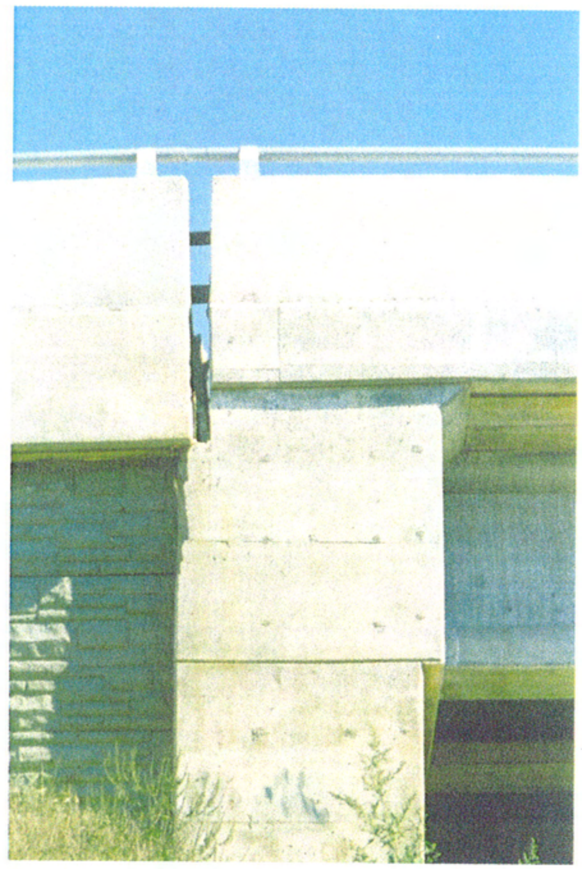


Figure J-9
North Abutment –
Cracking of Concrete at the Seat
(Photograph Date : September 17, 2003)

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.