FOUNDATION INVESTIGATION REPORT
PROPOSED HIGHWAY 17 (NEW) TWIN CULVERT STRUCTURE
AT STATION 13+072 MEDIAN CENTRELINE
HIGHWAY 17 (NEW) FROM ECHO RIVER TO BAR RIVER ROAD
DISTRICT 62, SAULT STE. MARIE, ONTARIO
G.W.P. 354 AND 352-94-00
SITE:

GEOCRES No. 41K00-058

Prepared For:

MARSHALL MACKLIN MONAGHAN LTD.

Prepared by:

SHAHEEN & PEAKER LIMITED

Project: SPT1055
August 27, 2003

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1. INTRODUCTION

Realignment of Highway 17 to the east of the existing highway from the Lower Echo River to Bar River Road includes new twin culverts at Station 13+072 Median Centreline of Highway 17 New, south of existing Highway 638 in Sault Ste. Marie, Ontario. The two twin culverts, which are 38 m in length under the proposed Hwy 17 westbound lanes (WBL) and 34 m in length under the proposed eastbound lanes (EBL), will be 2.2 m high and 5.7 m wide (2 x 2.84 m).

Shaheen & Peaker Limited (S&P) was retained by Marshall Macklin Monaghan Ltd. to carry out a foundation investigation for the proposed new twin culverts.

The findings of investigation are presented in this report.

2. SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located in a swampy area, about 250 m south of the existing Highway 638 and about 150 m west of Pioneer Road (see Drawing No. 1A).

Typically in the general area, the low-lying areas are characterized by surficial peat and topsoil, covering glaciolacustrine deposits. The glaciolacustrine deposits typically consist of clay and silt, with minor sand deposited in basin and quiet water environments. The depth of clay in these areas can exceed 40m. In the higher lying areas, bedrock of undifferentiated igneous and metamorphic classifications (Southern Province) is exposed at surface forming shallow hills. These rocks are generally Pre-cambrian formations while some Cambrian unconformities are also noted. The bedrock at the site consists of Cambrian sandstone of Jacobsville Formation at the interface with Pre-cambrian Lorrain Formation which consists of quartzite, siltstone, greywacke and conglomerate.

The grade in the general area is flat with the existing ground elevation at the borehole locations varying from about Elevation 178.2 m to about Elevation 178.5 m. An existing creek, which flows from southeast to northwest, meanders about 15m north of the proposed culverts location. The creek will be diverted to flow through the proposed culverts. Further east of the site, at a distance of about 250 m, the grade starts to rise towards a high ground area towards a rock outcrop.

3. INVESTIGATION PROCEDURES

For the proposed twin culverts, boreholes were drilled at the proposed median centerline of Highway 17 (New), at the east end of the WBL culvert and at the west end of the EBL culvert (i.e., a total of three boreholes). The boreholes were numbered C2, C2-B and C2-A, respectively, as shown on the Borehole Location Plan, Drawing No. 1. Dynamic Cone Penetration tests (DCPT) were also performed adjacent to two of the boreholes (to about 15 m depth) and from the bottom of the centre borehole (Borehole C2) to a depth of 51.8 m. The boreholes were drilled on March 26, 2002 and on December 14 and 15, 2002.

The boreholes were advanced using hollow stem continuous flight augers with track-mounted vehicles owned and operated by Colbar Resources of Sudbury, Ontario, under the supervision and direction of Geotechnical Engineers from our office. The depths of the boreholes range from 21.0 to 31.0 m. Sampling in the overburden was effected starting at the ground surface at 0.76 m intervals of depth by the Standard Penetration Test (SPT) method, as specified in ASTM D1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter O.D. split-spoon (split barrel) sampler into the relatively undisturbed ground. The number of blows required to drive the sampler into the ground by a vertical distance of 0.30 m is recorded as the Standard Penetration or the N-value of the soil and this gives an indication of the consistency or the compactness condition of the soil deposit.

As mentioned before, Dynamic Cone Penetration tests (DCPT) were performed at all borehole locations. In this test, a 51 mm diameter, 60-degree apex cone, screw attached to the tip of an A-size rod, is driven into the ground, using the same driving energy as the SPT method. By recording the number of blows of the hammer to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of soil compactness condition is obtained, while the test also provides some limited information on the consistency of cohesive (clay) soils.

The subsurface stratigraphy encountered in the boreholes, type of samples and sampling depths, N-values and DCPT results are presented on the Record of Borehole Sheets, in Appendix A of this report, and in Drawing No. 1.

Upon their completion, the boreholes were backfilled to about 6 m below the ground surface with soils brought up by augering (i.e auger cuttings). The upper 6 m of the open boreholes were then grouted using a cement/bentonite mixture.

The borehole locations were established in the field by S & P personnel and the surveyors from Marshall Macklin Monaghan Ltd. provided us with ground surface elevations at the borehole locations along with borehole co-ordinates.

A laboratory testing programme, consisting of natural moisture content measurements, grain-size analyses, bulk unit weight determination, Atterberg limits tests and consolidation tests, was performed on selected soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets and also in Appendix B.

4. SUBSURFACE CONDITIONS

The boreholes show, below a 0.2 to 0.5 m thick peat or peaty topsoil layer and about 0.9 m thick layer of surficial silty sand, the presence of an upper clay deposit extending to depths of 4.4 to 7.3 m, which is underlain by about 7 to 10 m thick silt to sandy silt layer. These silt to sandy silt layers are further underlain by a lower clay deposit extending to at least 30 m below existing grade. At the time of the investigation, the groundwater level at the site was found to be close to the existing ground surface.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets, Appendix A. The individual strata are briefly described in the following subsections of this report.

4.1 PEAT/TOPSOIL

Boreholes C2 and C2-B contacted a 0.2 to 0.3 m thick layer of peat, while Borehole C2-A encountered 0.5 m thick layer of peaty topsoil. It should be pointed out that at the time of our investigation the ground was mostly frozen and therefore, the description of the soil within the upper several decimeters may not be accurate. In addition, the thickness of organic deposits may vary between and beyond borehole locations, especially in depressions and near water courses.

4.2 SILTY SAND

Underlying the peat, an about 0.9 m thick surficial silty sand layer was contacted, extending to depths ranging from 1.1 to 1.4 m below the existing ground surface.

Standard Penetration tests performed in this silty sand layer yielded N-values ranging from zero (hammer and drill rods sank under own weight) to 3 blows/0.3 m, indicating very loose relative density.

The grain size distribution of two samples from boreholes immediately to the north and south of the present boreholes are presented in Figure B9 in Appendix B.

4.3 CLAY

Below the peat/peaty topsoil and surficial silty sand, all the boreholes encountered an upper clay deposit extending to depths ranging between 4.4 m at Borehole C2-B and 7.3 m at Borehole C2-A, below existing grade. At a depth of about 14.8 m, a lower unit of the clay deposit was also encountered in the boreholes and this extends to the remaining depths of the boreholes (21 and 31 m).

The clay is a layered material with highly plastic (fat) to medium plasticity clay with some low plasticity (lean) silty clay seams/lenses. It is generally irregularly layered with occasional thin clayey silt to sandy silt seams, some zones being more layered than others. In general, the soil has a grey to reddish grey colour.

The results of grain-size analyses carried out on two samples from the deposit are presented in Figure B1 of Appendix B. These indicate 0-1% sand, 18-51% silt and 48-82% clay-size particles. It should be pointed out that the samples tested were composed of various interbeds, which may range from silt to fat (highly plastic) clays. From the above results, the deposit can be expected to be highly impermeable and because of the interbedded structure, its mass permeability can be expected to be variable, especially in the horizontal direction.

Atterberg limits tests carried out in the laboratory on samples from the clay deposit gave the following index values:

Liquid Limit:	46 to 92%
Plastic Limit:	22 to 31%
Plasticity Index:	23 to 62%

As presented in Figures B2 and B3 in Appendix B, these values are characteristics of clay soils of medium to high plasticity. The measured natural moisture contents generally range from 40 to 90%, that is, generally at or in excess of the liquid limit values. The Liquidity Index values range between 0.8 and 1.9, but generally 1.0 to 1.4. These results are generally indicative of weak and compressible (generally normally consolidated) clays.

Two consolidation tests were performed on samples from this deposit (from Boreholes C2-A and C2-B) and the results are shown in Figures B4 and B5 in Appendix B. These tests indicate probable pre-consolidation pressures (Pc) of about 50 kPa within the upper clay unit (3 to 3.5 m depth) and 210 kPa in the lower clay deposit (about 15.5 m depth), which are about 30 and 65 kPa in excess of the existing overburden pressure (Po), respectively. The test results also indicate relatively high C_c-values (e.g., in the range of 1.2) indicating an extremely compressible clay structure, especially beyond the pre-consolidation pressure range.

The measured bulk unit weights of samples from the clay range from 14.7 to 15.6 kN/m^3 .

Standard Penetration tests performed in this deposit gave N-values varying between 0 and 4 blows/0.3 m. Field vane tests yielded undrained in-situ shear strength values ranging from 36 to 40 kPa within the top 0.5 ± m of the deposit, and ranging from 12 to 28 kPa below this depth, within the upper clay deposit. Undrained shear strength values ranging between 20 and 84 kPa were recorded within the lower clay deposit. These values indicate that the consistency of the material can be described as soft to firm in the upper clay deposit and soft to stiff in the lower clay deposit.

The variation of measured undrained shear strengths with elevation from the boreholes is presented in Figure C1 in Appendix C. Figure C2 shows typical plot of undrained shear strength versus elevation at the location of Borehole C2-A. In general, the undrained shear strength increases with depth which indicates normally consolidated clays.

Dynamic Cone Penetration tests (DCPT) were conducted in Borehole C2 from a depth of 31 m to 51.8 m below existing grade, and in Boreholes C2-A and C2-B from the ground surface to a depth of about 15 m. The results of the tests are presented on the Record of Borehole sheets. In Borehole C2 (which was terminated in the clay at a depth of 31 m), the presence of a somewhat 'stiffer' or relatively more competent stratum was inferred from the DCPT at a depth of about 38 (Elevation 140 ± m) becoming somewhat more competent below 46 m (Elevation 132 ± m). The test was terminated at a depth of 52 ± m (Elevation 126.5 m).

4.4 SILT TO SANDY SILT

Below the upper clay deposit, at depths of 4.4 to 7.3 m below existing ground surface, all the boreholes contacted an approximately 7 to 10 m thick layer of silt to sandy silt extending to a depth of about 14.8 m (about Elevation 163.6 m). This deposit contains traces to some clay and occasional silty sand seams or layers.

The results of the grain-size distribution analyses performed on samples from this deposit are presented in Figures B6, B7 and B8 of Appendix B. They indicate the following particle size distribution:

Gravel	=	0 %
Sand	=	3 to 32%
Silt	=	51 to 85%
Clay	=	1 to 17 %

Based on N-values which range from 1 to 28 blows/0.3 m, this basically cohesionless material is described as very loose to compact. In Borehole C2-A, the silt layer, between 7.3 and 10.3 m depth, contains some amount of clay with traces of sand and is considered a cohesive material with soft consistency.

4.5 GROUNDWATER CONDITIONS

Water level observations in the boreholes were made during drilling and at completion of each borehole. The recorded water levels at completion ranges between 3.1 m and 7.6 m below existing grades at Boreholes C2-A and C2-B or at between Elevation 175.1 m to 170.9 m, respectively, but these are unlikely to represent the stabilized water levels. At Borehole C2, a wet cave at a depth of 0.3 m was recorded, at completion of the borehole.

From visual and tactile examinations and the measured moisture contents of the soil samples, and considering that this site is within a swampy area, the groundwater table at the site is believed to be at or close to the existing ground surface. It should, however, be pointed out that the groundwater table can be expected to fluctuate seasonally and in response to major weather events.

Yours very truly,

SHAHEEN & PEAKER LIMITED

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

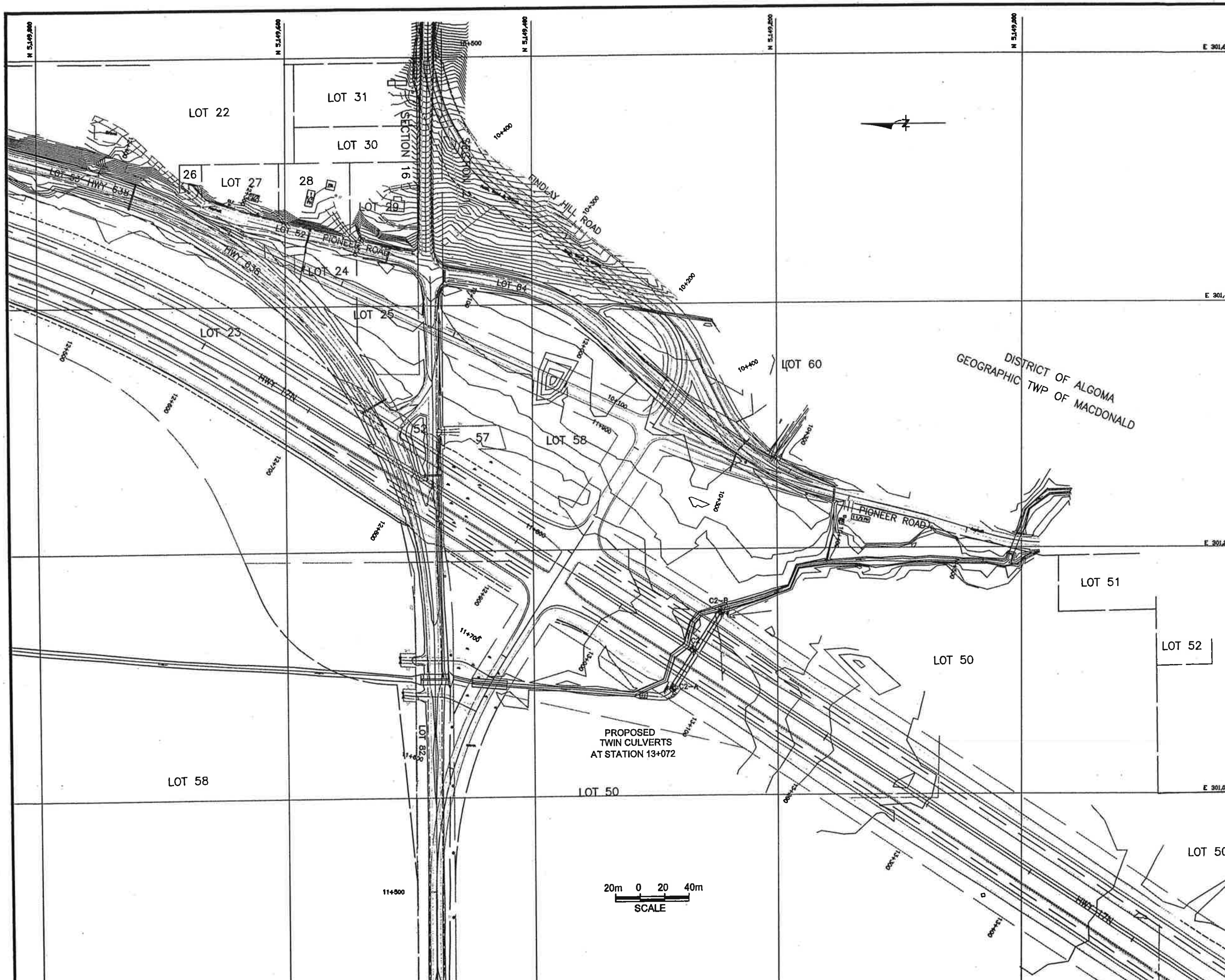
R. Miranda, P.Eng.

Z. S. Ozden, P.Eng.

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

RM:rm

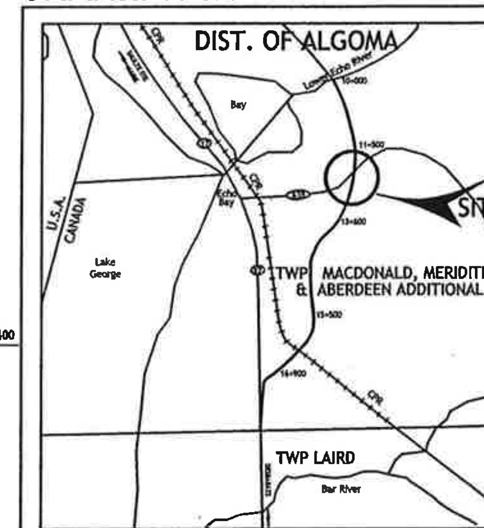
Drawings



CONT No.
GWP: 354-94-00

HIGHWAY 17 (NEW)
ECHO RIVER TO BAR RIVER ROAD
TWIN CULVERTS AT 13+072
SITE PLAN

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

Bore Hole & Cone

No.	ELEV.	CO-ORDINATES NORTH	CO-ORDINATES EAST
C2	178.3	5 149 270.6	301 118.2
C2–A	178.2	5 149 287.3	301 086.1
C2–B	178.5	5 149 247.3	301 148.3

=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. 41K00-058

HWY No. 17 (New)			DIST 62
SUBM'D ZO	CHECKED RM	DATE Jun, 2003	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 1A

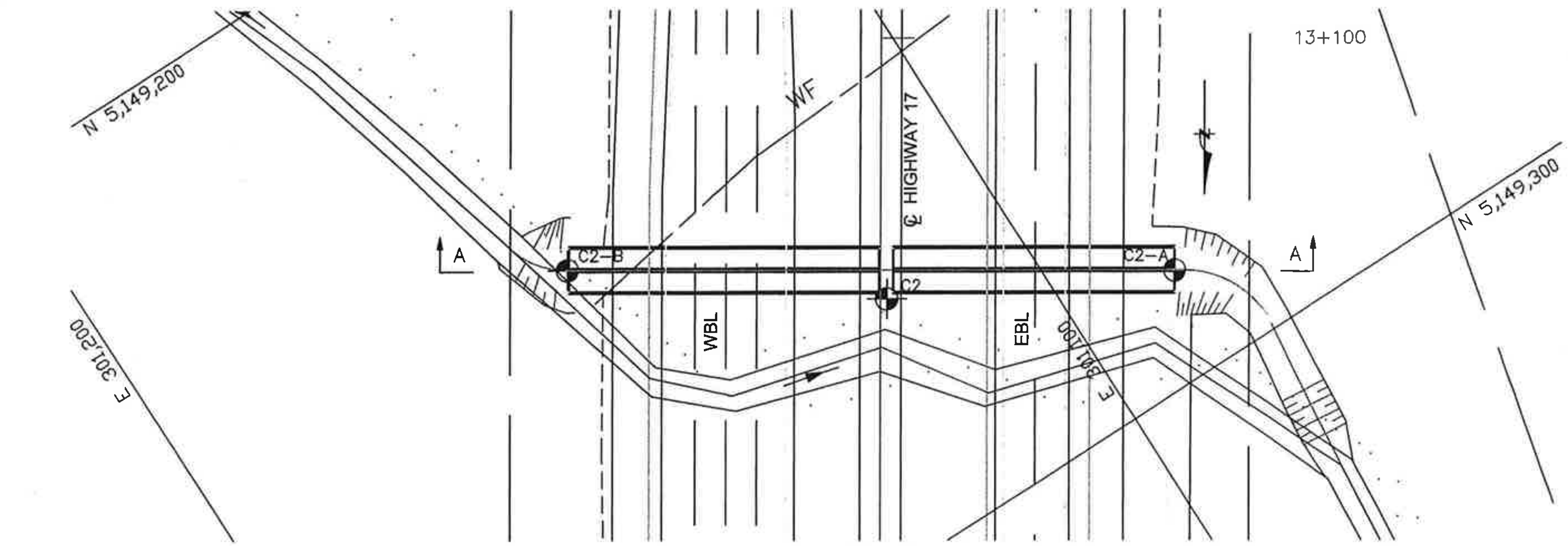
METRIC

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

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

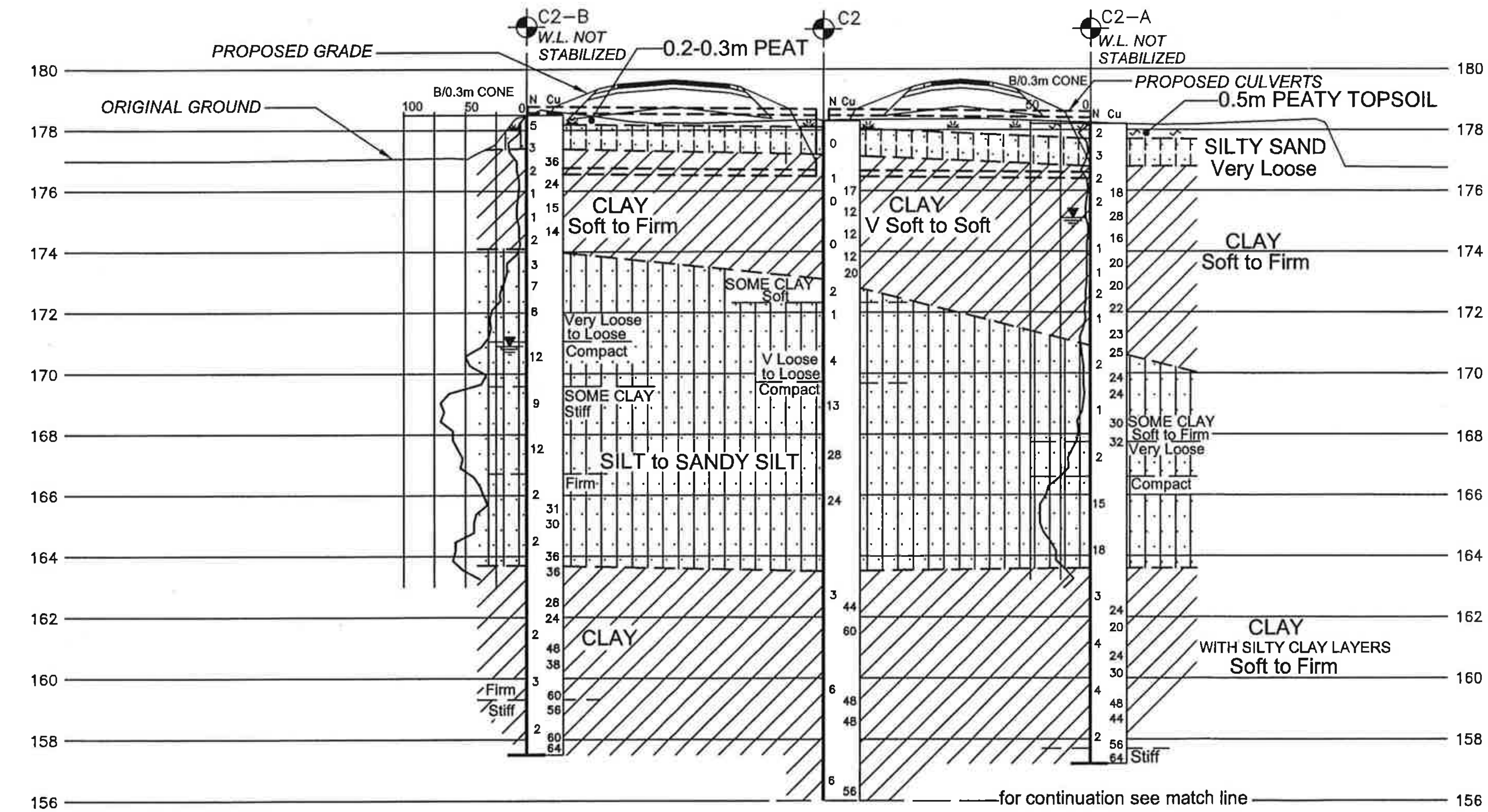
REGISTERED PROFESSIONAL ENGINEER Z. S. OZDEN 28/08/03 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER RAMON MIRANDA 28/08/03 PROVINCE OF ONTARIO



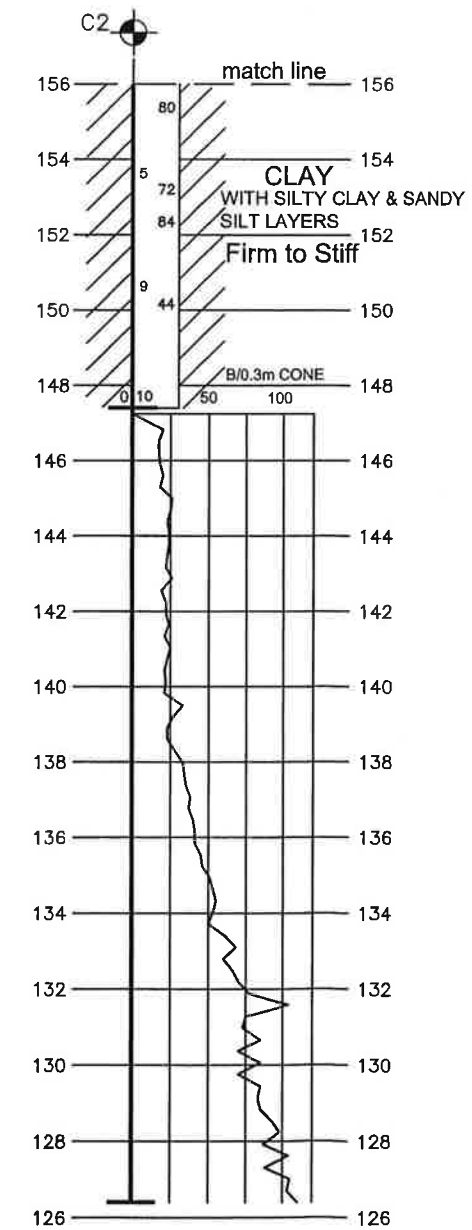
PLAN

SCALE



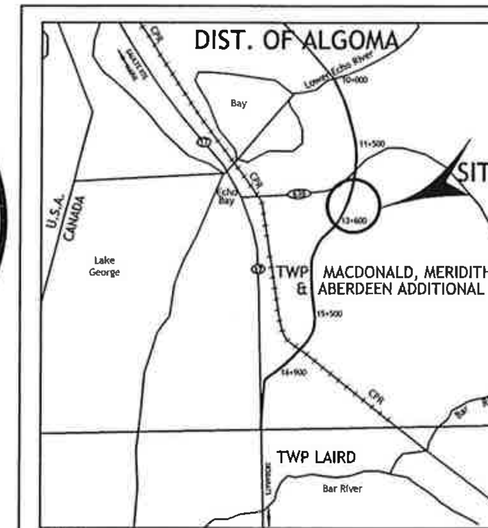
SECTION A-A

HOR
VERT
SCALES



CONT No.
GWP: 354-94-00
HIGHWAY 17 (NEW) ECHO RIVER TO BAR RIVER ROAD CULVERTS AT STATION 13+072 BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

Symbol	Description
⌖	Bore Hole & Cone
N	Blows/0.3m (Std. Pen. Test, 475 J/blow)
Cu	Undrained Shear Strength measured by Field Vane Test
CONE	Blows/0.3m (60° Cone, 475 J/blow)
▽	Water Level at Time of Investigation Dec. ,2002

No.	ELEV.	CO-ORDINATES NORTH	EAST
C2	178.3	5 149 270.6	301 118.2
C2–A	178.2	5 149 287.3	301 086.1
C2–B	178.5	5 149 247.3	301 148.3

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

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REV.	DATE	BY	DESCRIPTION

Geocres No. 41K00-058

HWY No. 17 (New)			DIST 62
SUBM'D ZO	CHECKED RM	DATE May, 2003	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 1

Appendix A

Record of Borehole Sheets

Ministry of Transportation
Ontario

Foundation Design

SPT1055

RECORD OF BOREHOLE No C2

METRIC

GWP 354-94-00 | LOCATION Station: 13+069 CL - Coords: N ; 5 149 270.6; E 301 118.2 | ORIGINATED BY S.O.

DIST 62 HWY 17 (New) | BOREHOLE TYPE Hollow Stem Augers and D.C.P.T | COMPILED BY Y.L.

DATUM Geodetic | DATE 3/26/2002 | CHECKED BY Z.O.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	"N" VALUES
178.3	Ground Surface			
178.0 / 0.2	0.2 m **Peat** / **SILTY SAND** trace organics brown, wet, very loose	1	AS	-
		2	SS	0
177.2 / 1.1	**CLAY** grey, wet very soft to soft	3	SS	1
		4	SS	0
		5	TW	PH
		6	SS	0
		7	TW	PH
173.1 / 5.2	**SILT to SANDY SILT** grey, wet; some clay soft	8	SS	2
		9	SS	1
	very loose to loose	10	SS	4
	compact	11	SS	13
		12	SS	28
		13	SS	24
163.5 / 14.8				

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (178 to 164); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No C2

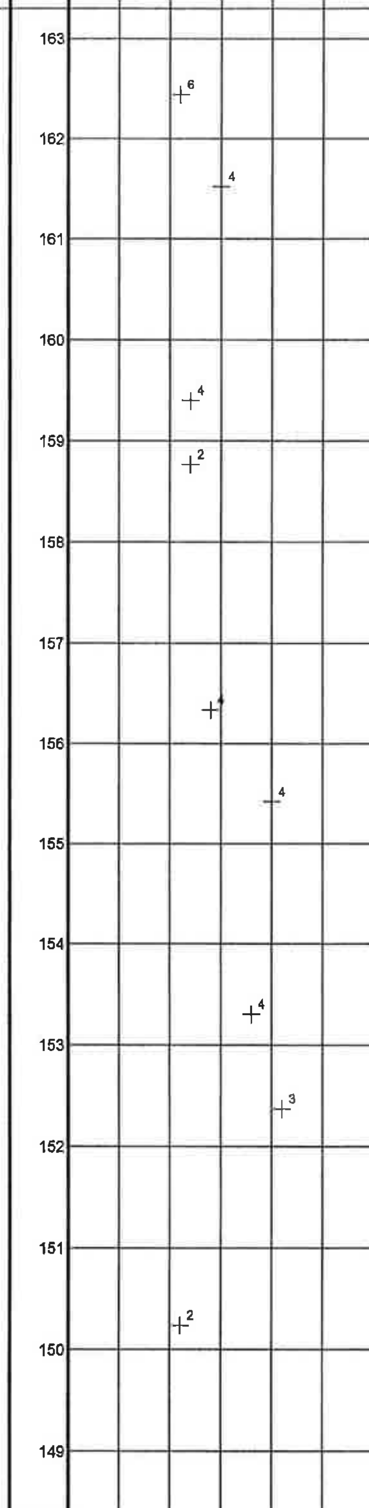
METRIC

GWP	354-94-00	LOCATION	Station: 13+069 CL - Coords: N ; 5 149 270.6; E 301 118.2	ORIGINATED BY	S.O.
DIST	62 HWY 17 (New)	BOREHOLE TYPE	Hollow Stem Augers and D.C.P.T	COMPILED BY	Y.L.
DATUM	Geodetic	DATE	3/26/2002	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: O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE (20 40 60 80 100)
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

CLAY
with silty clay and sandy silt seams
reddish grey, wet
firm to stiff

NUMBER	TYPE	"N" VALUES
14	SS	3
15	SS	6
16	SS	6
17	SS	5
18	SS	9



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+ ³, × ³: Numbers refer to Sensitivity

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

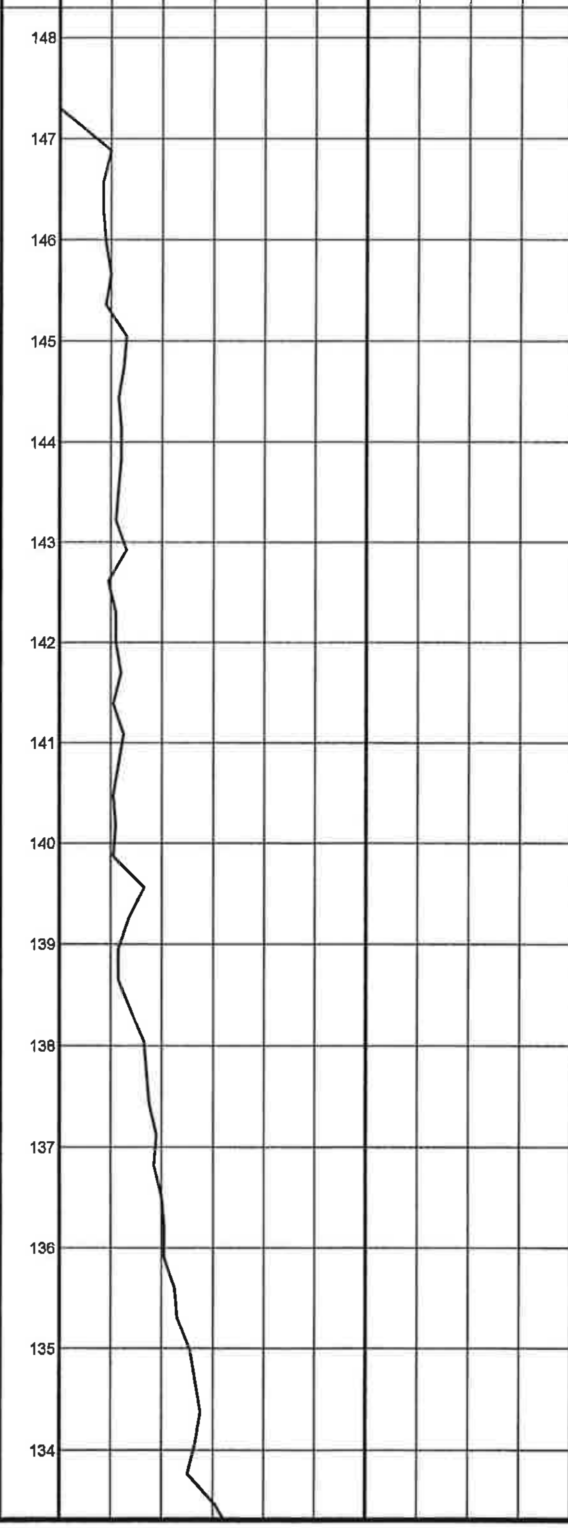
RECORD OF BOREHOLE No C2

METRIC

GWP 354-94-00	LOCATION Station: 13+069 CL - Coords: N ; 5 149 270.6; E 301 118.2	ORIGINATED BY S.O.	
DIST 62	HWY 17 (New)	BOREHOLE TYPE Hollow Stem Augers and D.C.P.T	COMPILED BY Y.L.
DATUM Geodetic	DATE 3/26/2002	CHECKED BY Z.O.	

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAY with silty clay and sandy silt seams reddish grey, wet firm to stiff		19	SS	10		148	March 26
147.3 / 31.0	End of Bore Hole						147	March 27

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³



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+ ³, × ³: Numbers refer to Sensitivity

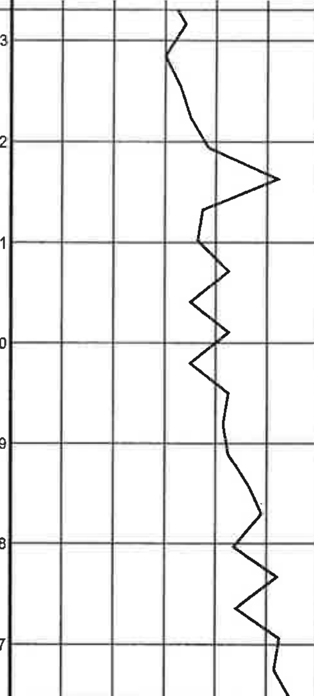
20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No C2

METRIC

GWP 354-94-00	LOCATION Station: 13+069 CL - Coords: N : 5 149 270.6; E 301 118.2		ORIGINATED BY S.O.
DIST 62	HWY 17 (New)	BOREHOLE TYPE Hollow Stem Augers and D.C.P.T	COMPILED BY Y.L.
DATUM Geodetic	DATE 3/26/2002		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: O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
							133				
							132				
							131				
							130				
							129				
							128				
							127				
126.4 51.8	End of Dynamic Cone Penetration Test. Dynamic Cone Penetration Test performed from 31 m to 51.8 m. Move to N: 5 149 212.3; E: 301 116.9 and performed another D.C.P.T. Refer to 13+068; 2 m Rt for results * Wet cave at 0.3 m below ground surface.										



+³, ×³: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No C2-A

METRIC

GWP 354-94-00	LOCATION Station: 13+072; 36 m Rt. - Coords: N 5 149 287.3; E 301 086.1	ORIGINATED BY G.I.
DIST 62 HWY 17 (New)	BOREHOLE TYPE Hollow Stem Augers and D.C.P.T	COMPILED BY Y.L.
DATUM Geodetic	DATE 12/14/2002	CHECKED BY Z.O.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	NATURAL MOISTURE CONTENT / LIMITS (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
178.2	Ground Surface										
0.0	0.5 m **Topsoil**	1	SS	2							
177.7 / 0.5	**SILTY SAND** brown, wet, very loose	2	SS	3							
176.8 / 1.4	**CLAY** occasional clayey silt seams grey, wet soft to firm	3	SS	2							
		4	SS	2		84					
		5	TW	PH	* **	92	14.7	0	1	51	48
		6	SS	1							
		7	SS	1		86					
		8	SS	2							
		9	SS	1							
170.9 / 7.3	**SILT to SANDY SILT** grey wet	10	SS	2							
		11	SS	1				0	3	80	17
	some clay soft / very loose	12	SS	2				0	13	85	2
	compact	13	SS	15							
		14	SS	18				0	29	70	1
163.6 / 14.6											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

ELEVATION SCALE: 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164

Continued Next Page

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

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

RECORD OF BOREHOLE No C2-A

METRIC

GWP 354-94-00 LOCATION Station: 13+072; 36 m Rt. - Coords: N 5 149 287.3; E 301 086.1 ORIGINATED BY G.I.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers and D.C.P.T COMPILED BY Y.L.

DATUM Geodetic DATE 12/14/2002 CHECKED BY Z.O.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	CLAY with silty clay layers reddish grey, wet soft to firm	15	SS	3	163
		16	SS	4	162 / 161
		17	SS	4	160 / 159
		18	SS	2	158
	stiff				
157.2 / 21.0	End of borehole.				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Field vane sensitivities plotted: 4, 3, 3, 3, 4, 4, 4, 3

* Water level at 3.1 m (not stabilized) and hole open to 5.5 m on completion.

** Consolidation Test performed using sample TW 5

+ 3, × 3: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

SPT1055

RECORD OF BOREHOLE No C2-B

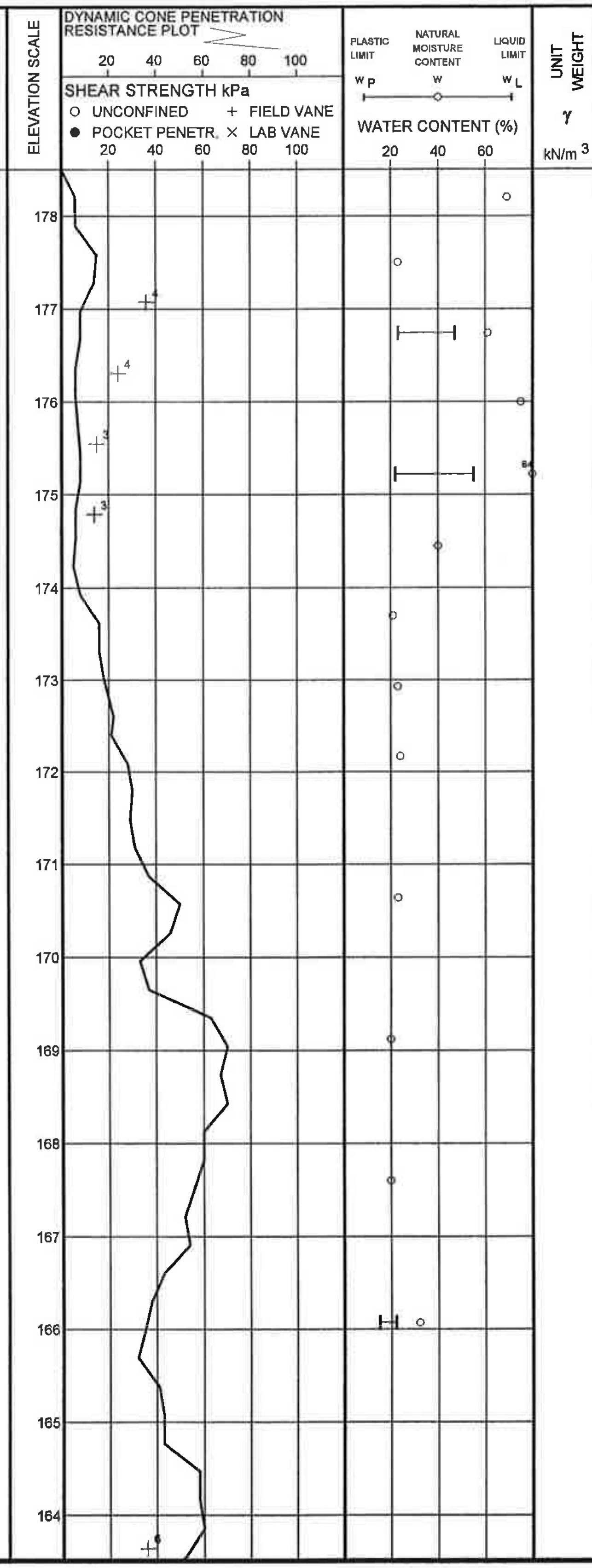
METRIC

GWP 354-94-00 LOCATION Station: 13+072; 38 m Lt. - Coords: N 5 149 247.3; E 301 148.3 ORIGINATED BY G.I.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers and D.C.P.T COMPILED BY Y.L.

DATUM Geodetic DATE 12/15/2002 CHECKED BY Z.O.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
178.5	Ground Surface					
178.2 / 0.0	0.3 m **Peat**	1	SS	5		
0.3	**SILTY SAND** brown, wet, very loose	2	SS	3		
177.4 / 1.1	**CLAY** occasional clayey silt seams grey, moist to wet soft to firm	3	SS	2		
		4	SS	1		
		5	SS	1		
		6	SS	2		
174.1 / 4.4	**SILT to SANDY SILT** trace clay occasional silty sand layers reddish grey, wet	7	SS	3		0 30 68 2
		8	SS	7		
	very loose to loose	9	SS	8		0 28 66 6
	compact				▼*	
		10	SS	12		
	some clay stiff	11	SS	9		0 32 51 17
		12	SS	12		
	firm	13	SS	2		
		14	SS	2		
163.7 / 14.8						



Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No C2-B

METRIC

GWP 354-94-00 LOCATION Station: 13+072; 38 m Lt. - Coords: N 5 149 247.3; E 301 148.3 ORIGINATED BY G.I.

DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers and D.C.P.T COMPILED BY Y.L.

DATUM Geodetic DATE 12/15/2002 CHECKED BY Z.O.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	CLAY reddish grey, wet	15	TW	PH	**	163	15.6	0	0	18	82
						162					
		16	SS	2		161					
	firm / stiff	17	SS	3		160					
						159					
		18	SS	2		158					
157.4 21.0	End of borehole.										

* Water level at 7.6 m (not stabilized) and hole open to 8.5 m on completion.

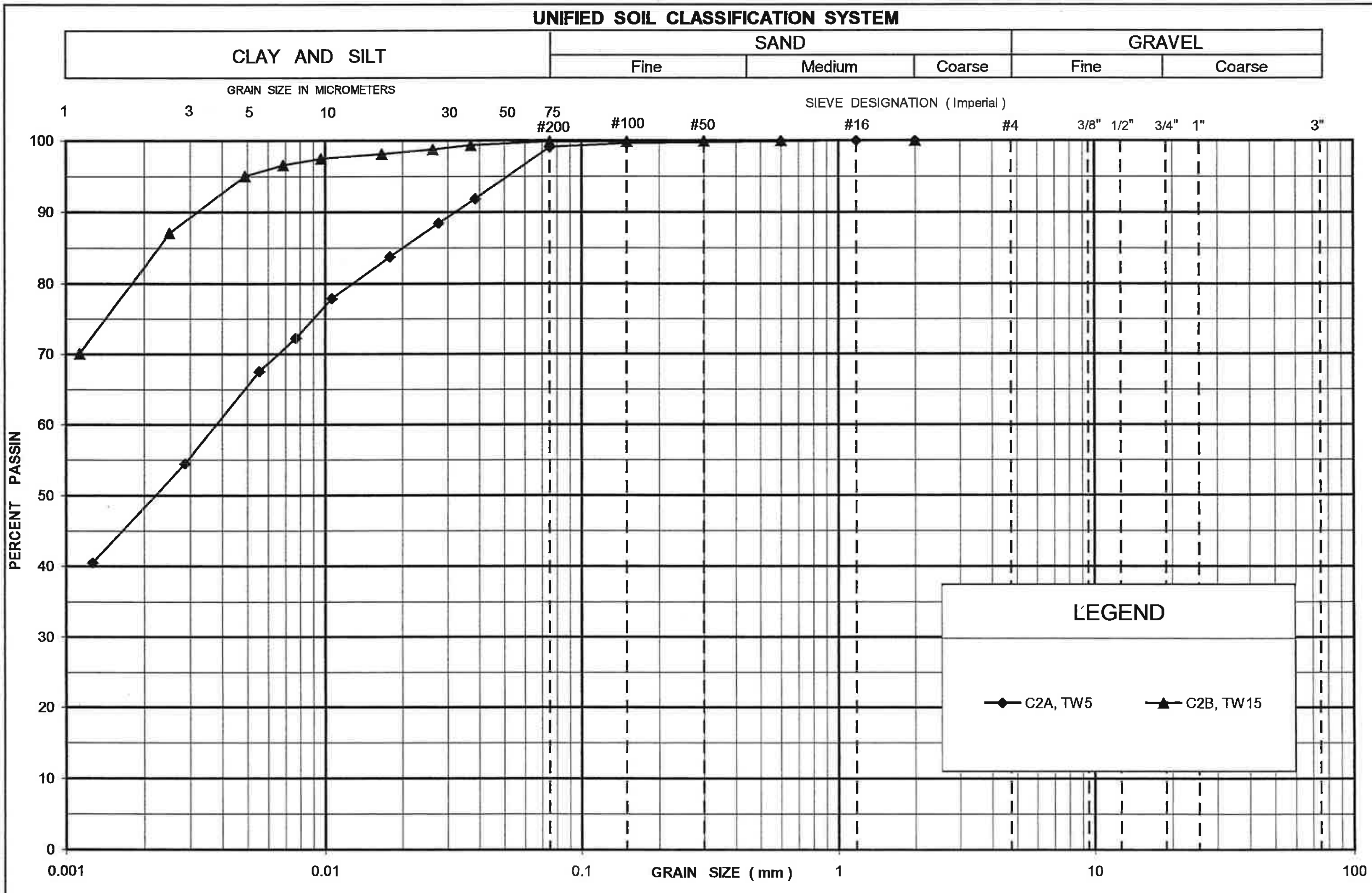
** Consolidation Test performed using sample TW 15

+ ³, × ³: Numbers refer to Sensitivity

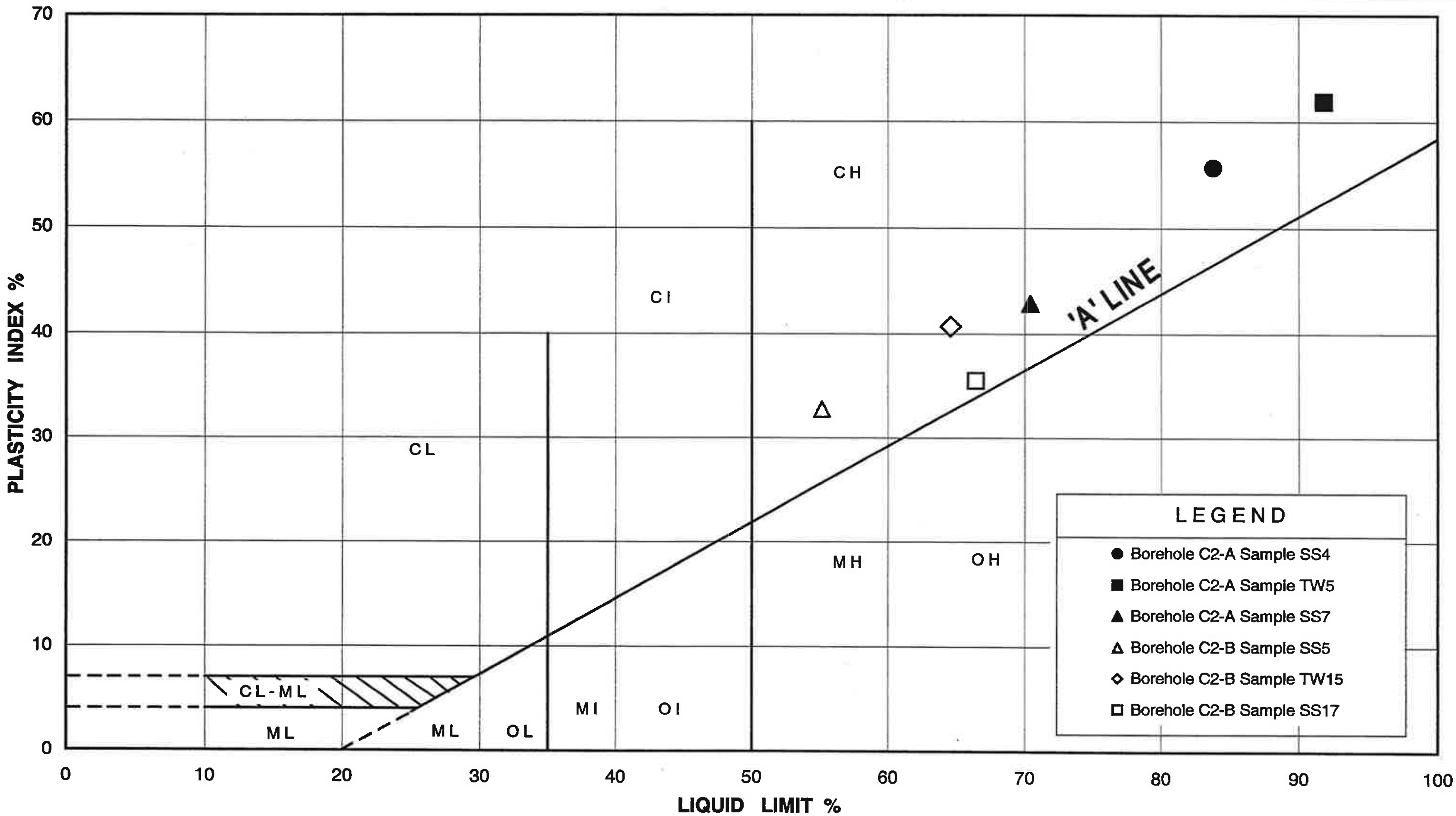
15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Appendix B

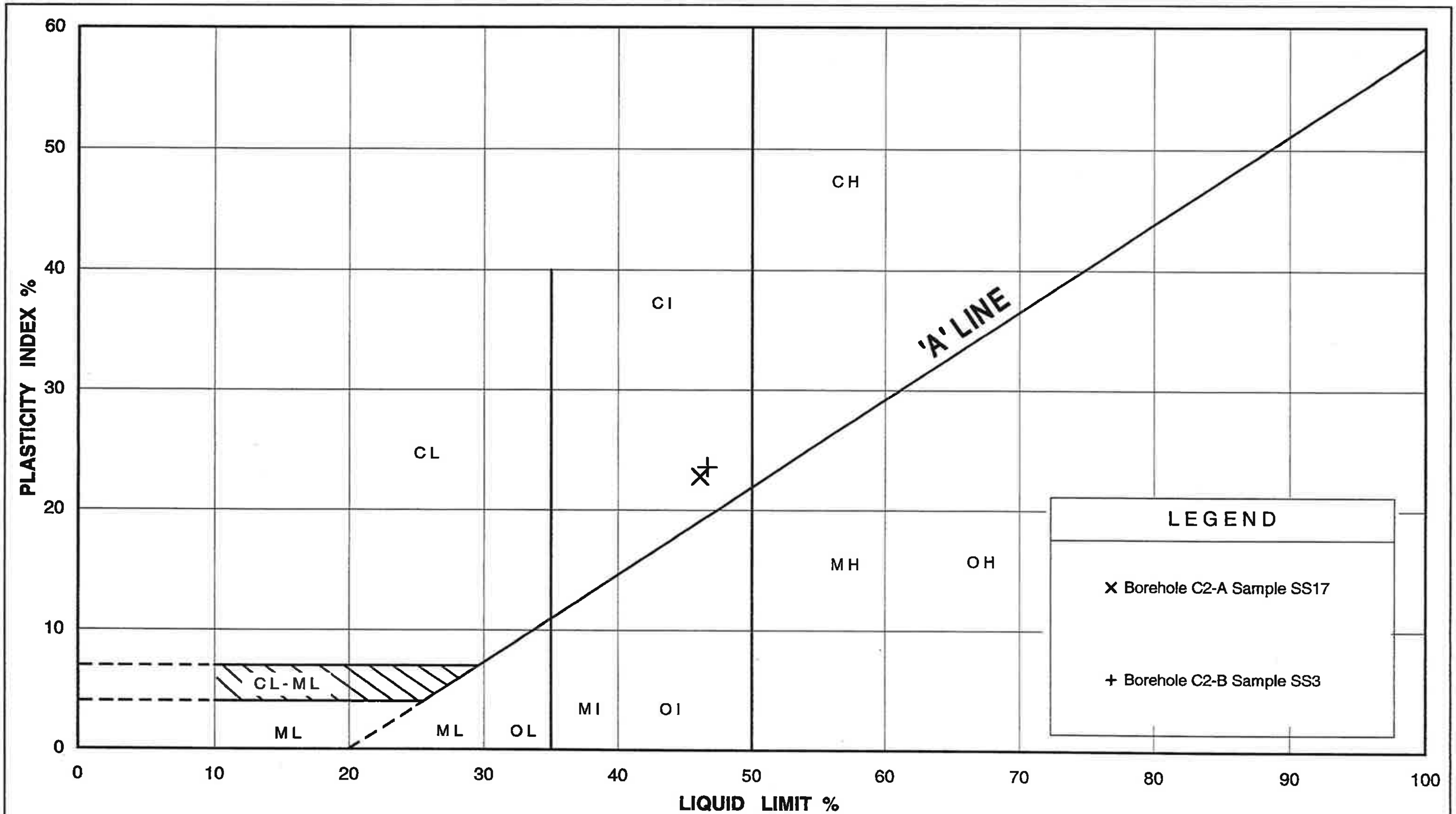
Laboratory Test Results



SHAHEEN & PEAKER LIMITED	GRAIN SIZE DISTRIBUTION **CLAY**	FIGURE No.	B1
		REF. No.	SPT 1055
		G.W.P	354-94-00



SHAHEEN & PEAKER LIMITED	PLASTICITY CHART CLAY	FIG No B-2 G.W.P. 354-94-00 SPT 1055

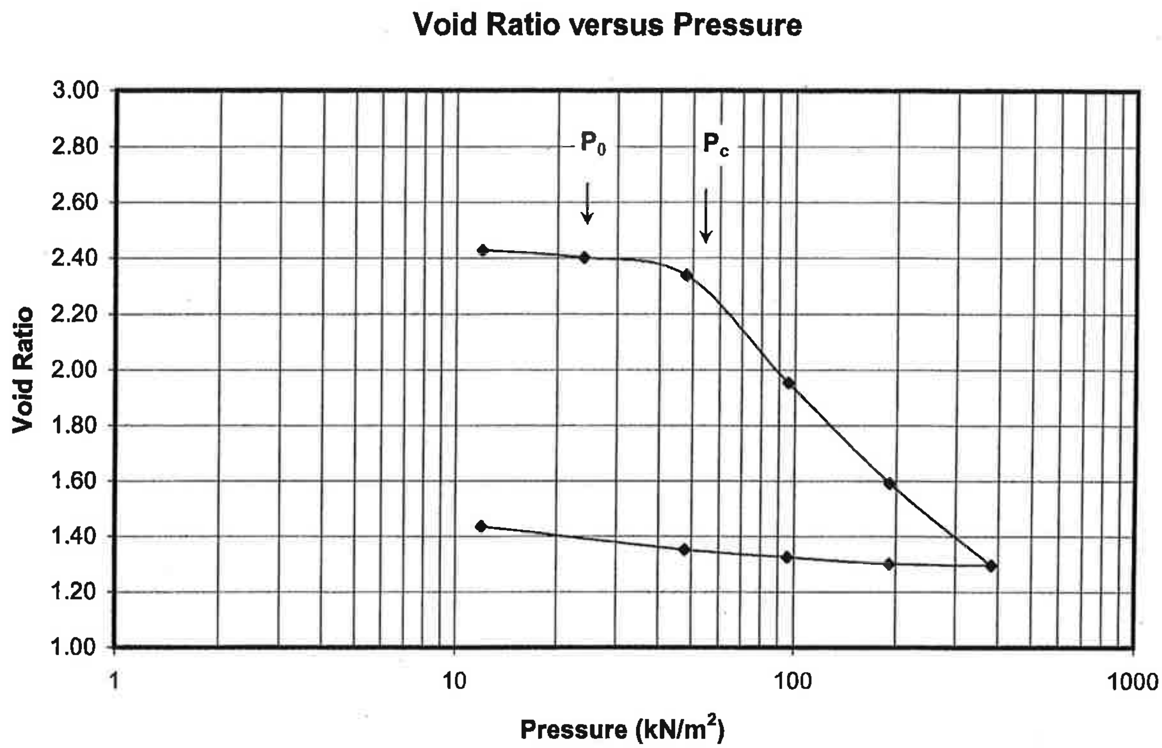


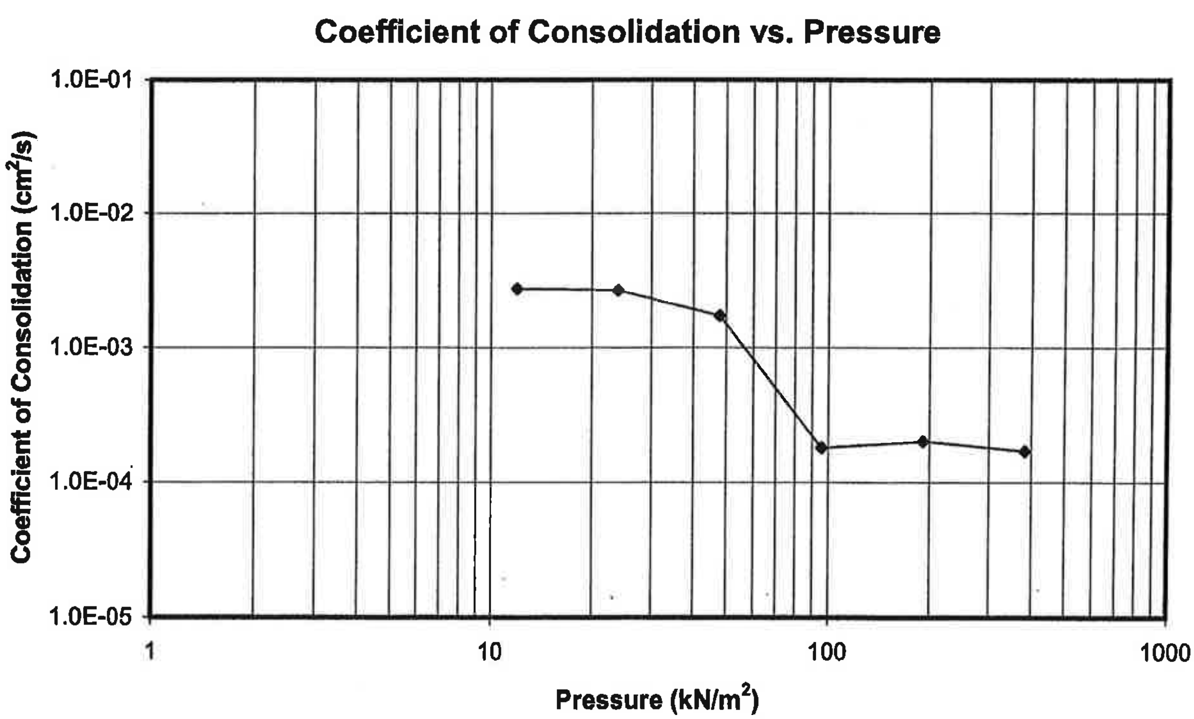
SHAHEEN & PEAKER LIMITED	PLASTICITY CHART SILTY CLAY	FIG No B-3 G.W.P. 354-94-00 SPT 1055

C2A

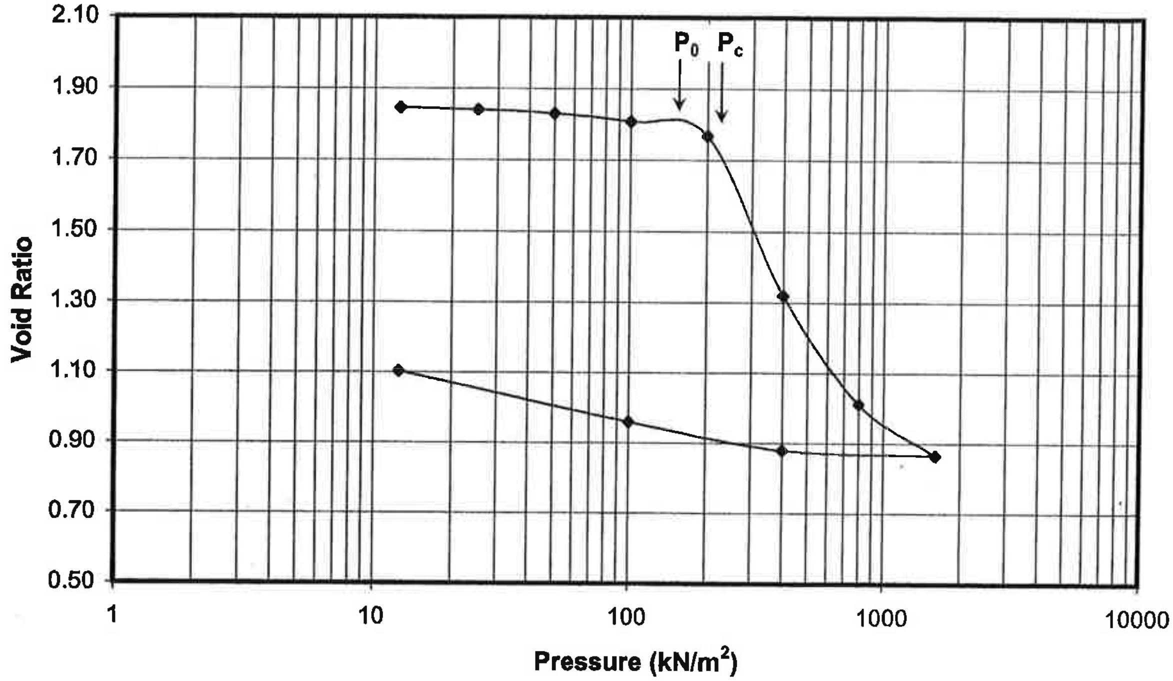
TW5 Depth 3.0-3.5 m

Fig. B4

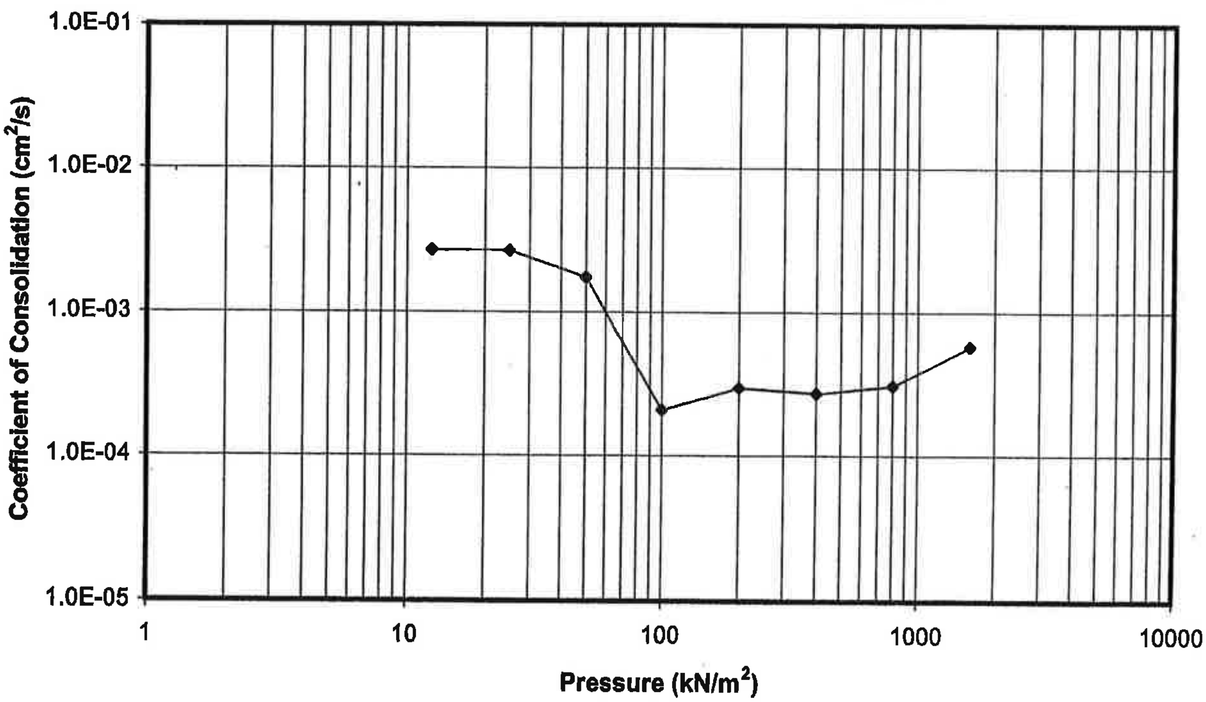


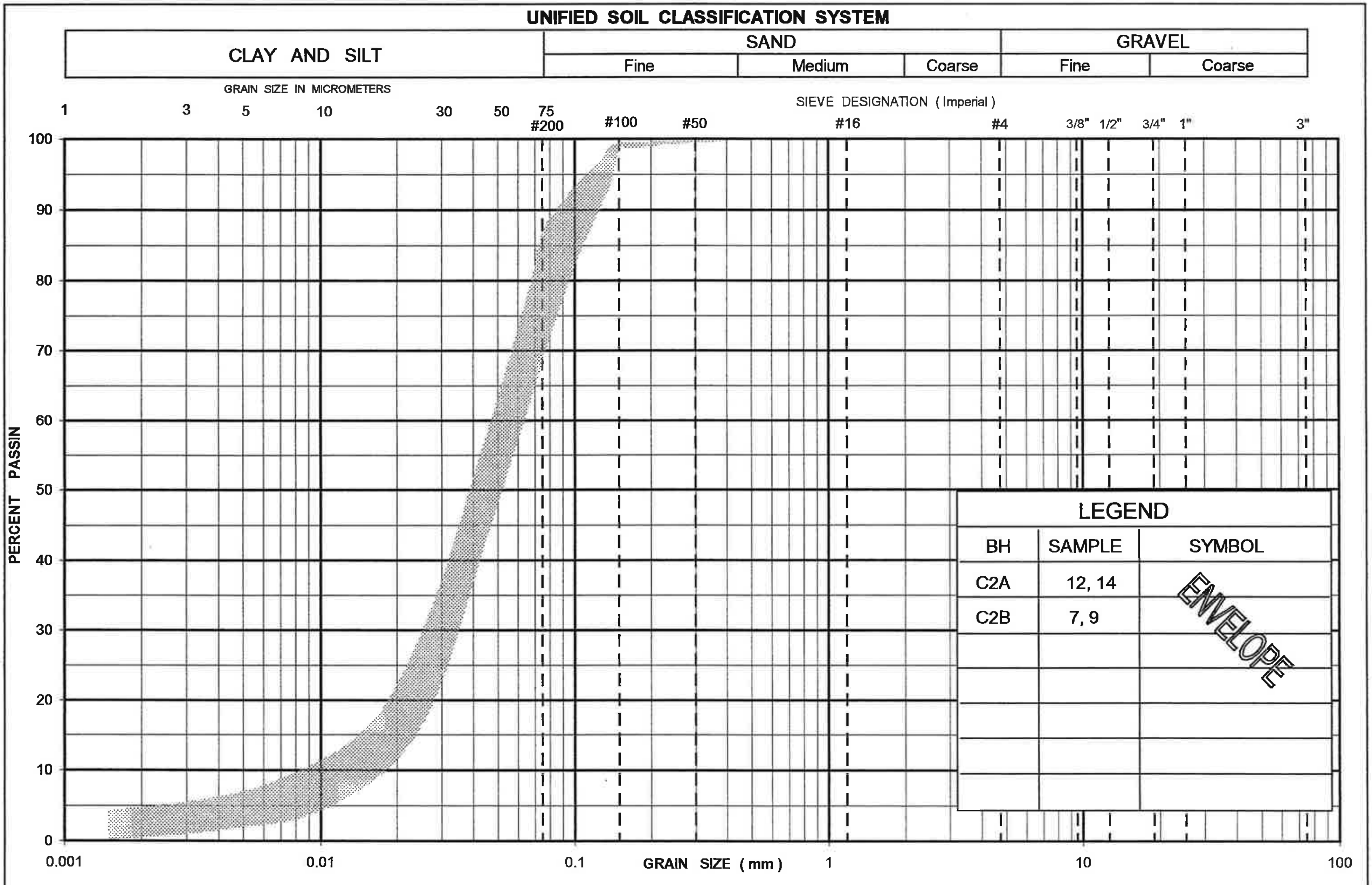


Void Ratio versus Pressure

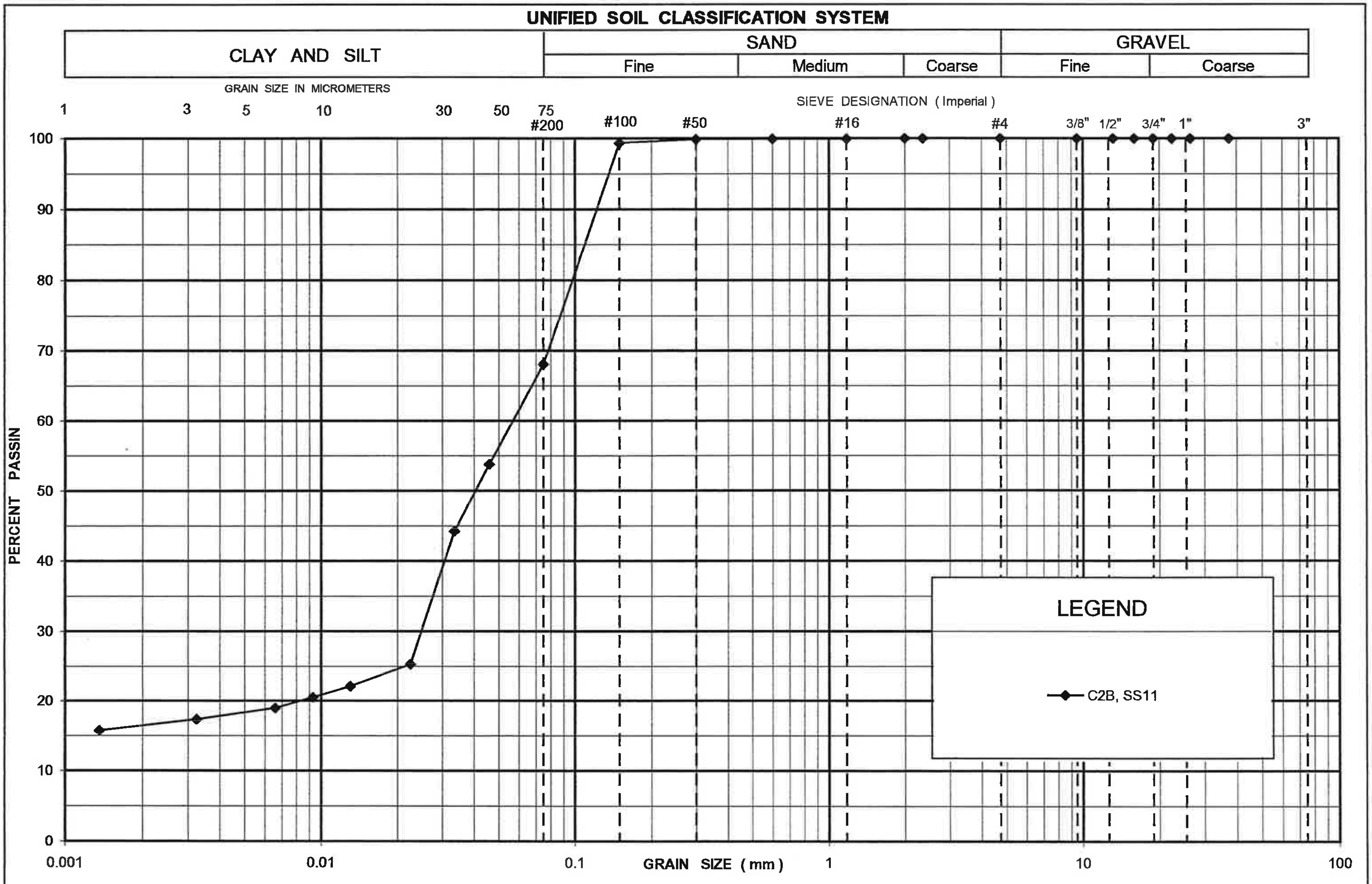


Coefficient of Consolidation vs. Pressure

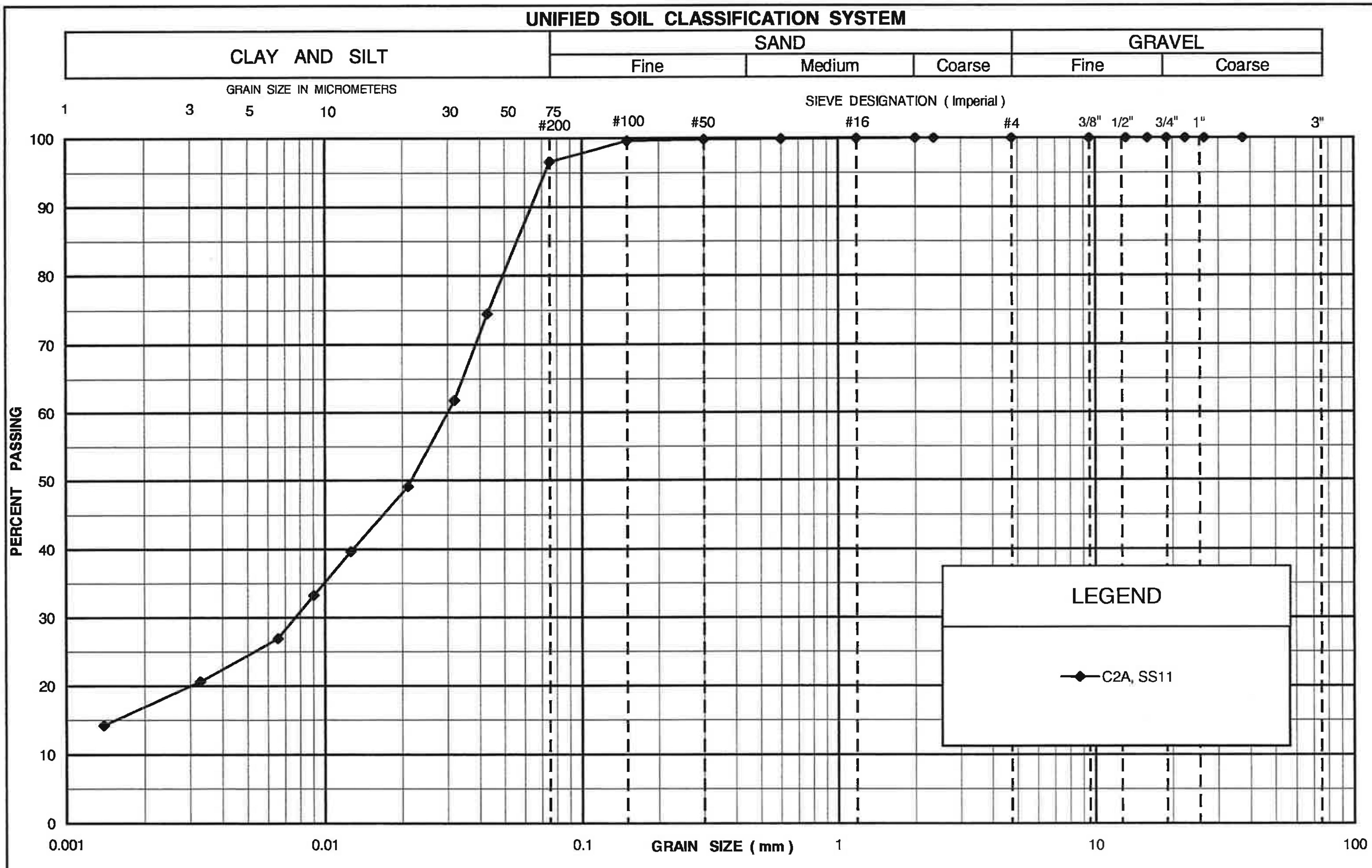




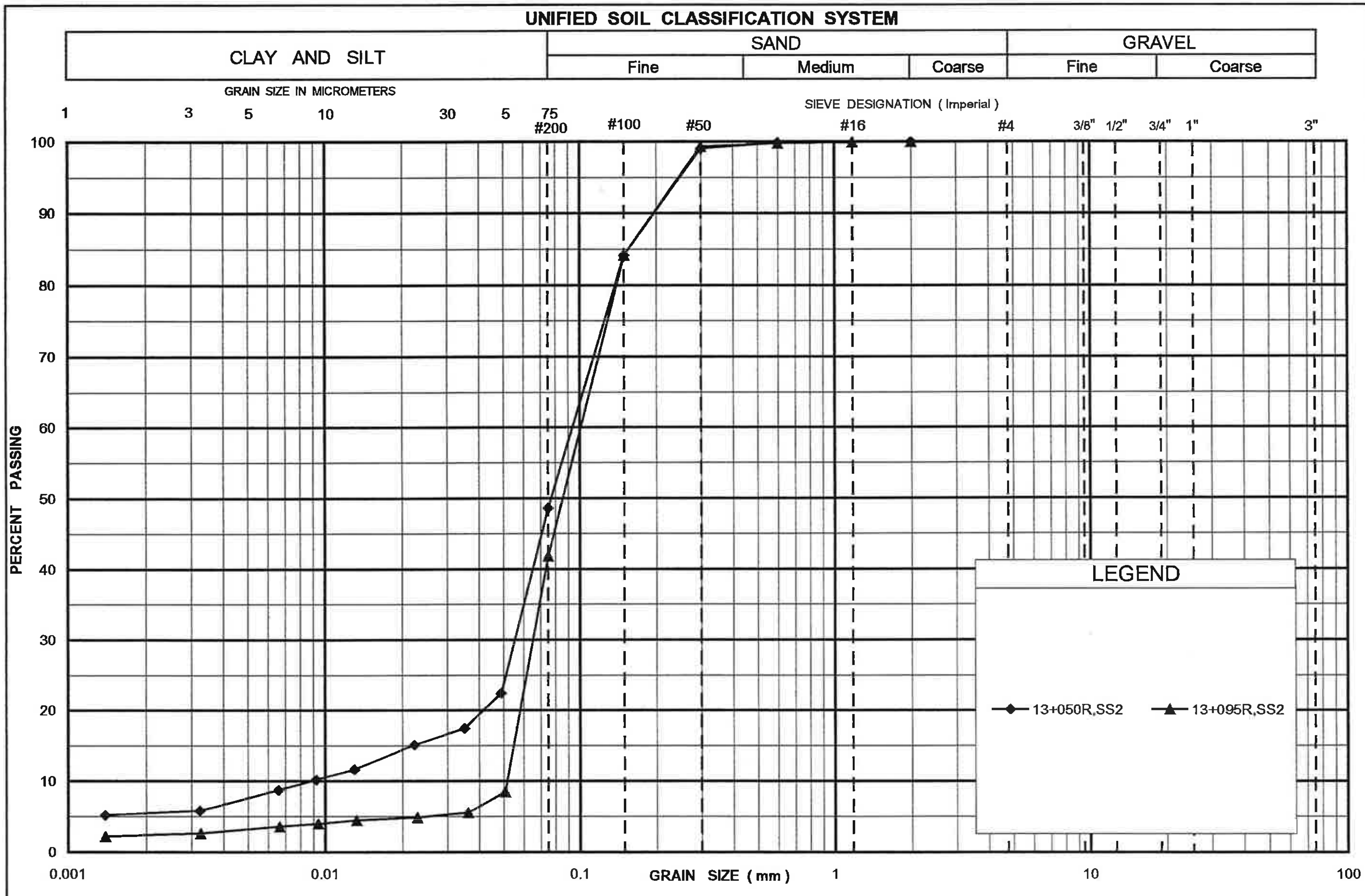
SHAHEEN & PEAKER LIMITED	GRAIN SIZE DISTRIBUTION **SANDY SILT, trace clay**	FIGURE No. B6 REF. No. SPT 1055 G.W.P 354-94-00



SHAHEEN & PEAKER LIMITED	GRAIN SIZE DISTRIBUTION **SANDY SILT, some clay**	FIGURE No. B7 REF. No. SPT 1055 G.W.P 354-94-00



SHAHEEN & PEAKER LIMITED	GRAIN SIZE DISTRIBUTION **SILT, some clay**	FIGURE No.	B8
		REF. No.	SPT 1055
		G.W.P	354-94-00



SHAHEEN & PEAKER LIMITED	GRAIN SIZE DISTRIBUTION SILTY SAND	FIGURE No.	B9
		REF. No.	SPT 1055
		G.W.P.	354-94-00

Appendix C

Measured Undrained Shear Strength Results

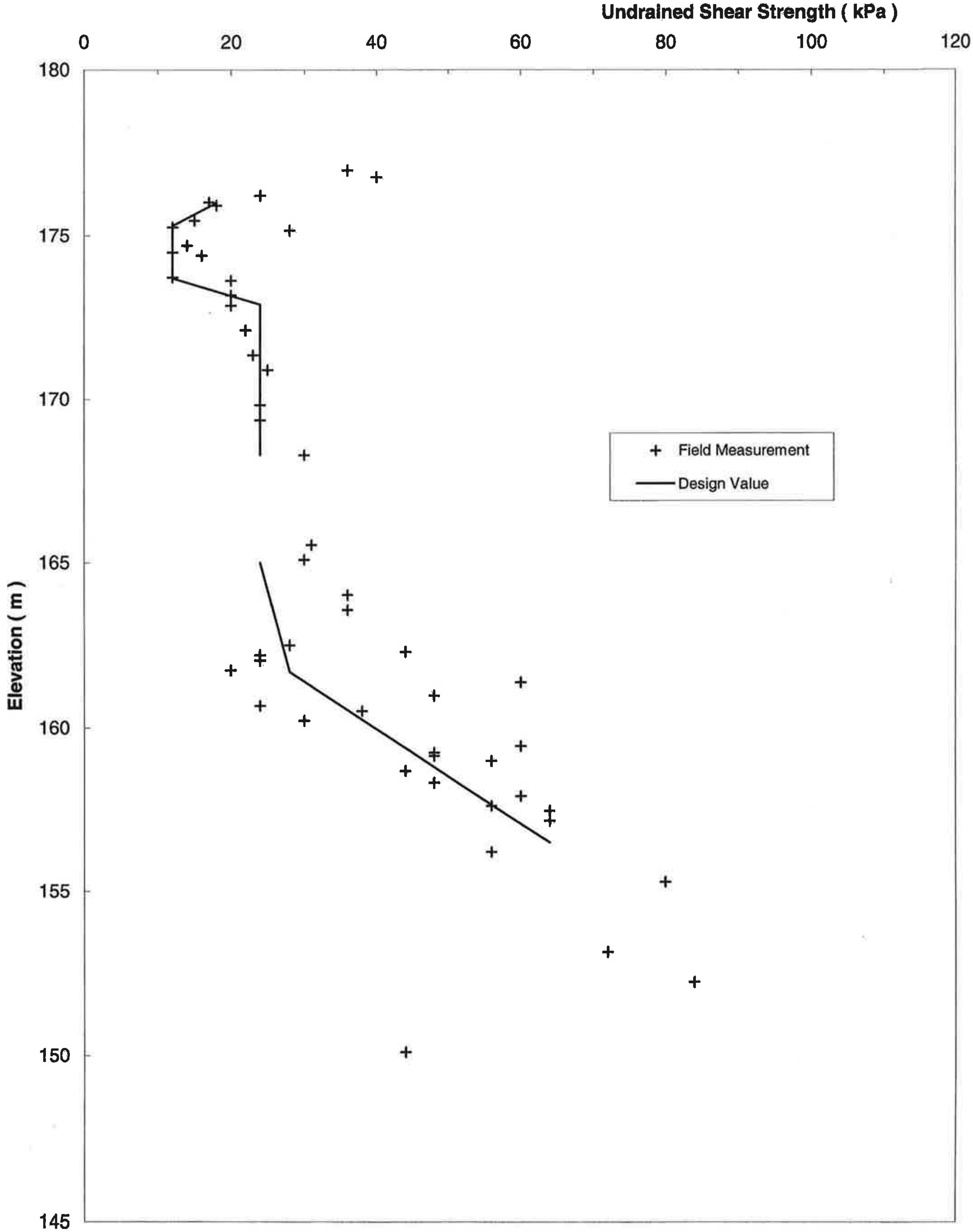


Fig. C1: Variation of Undrained Shear Strength (as measured by field vane tests) with Elevation

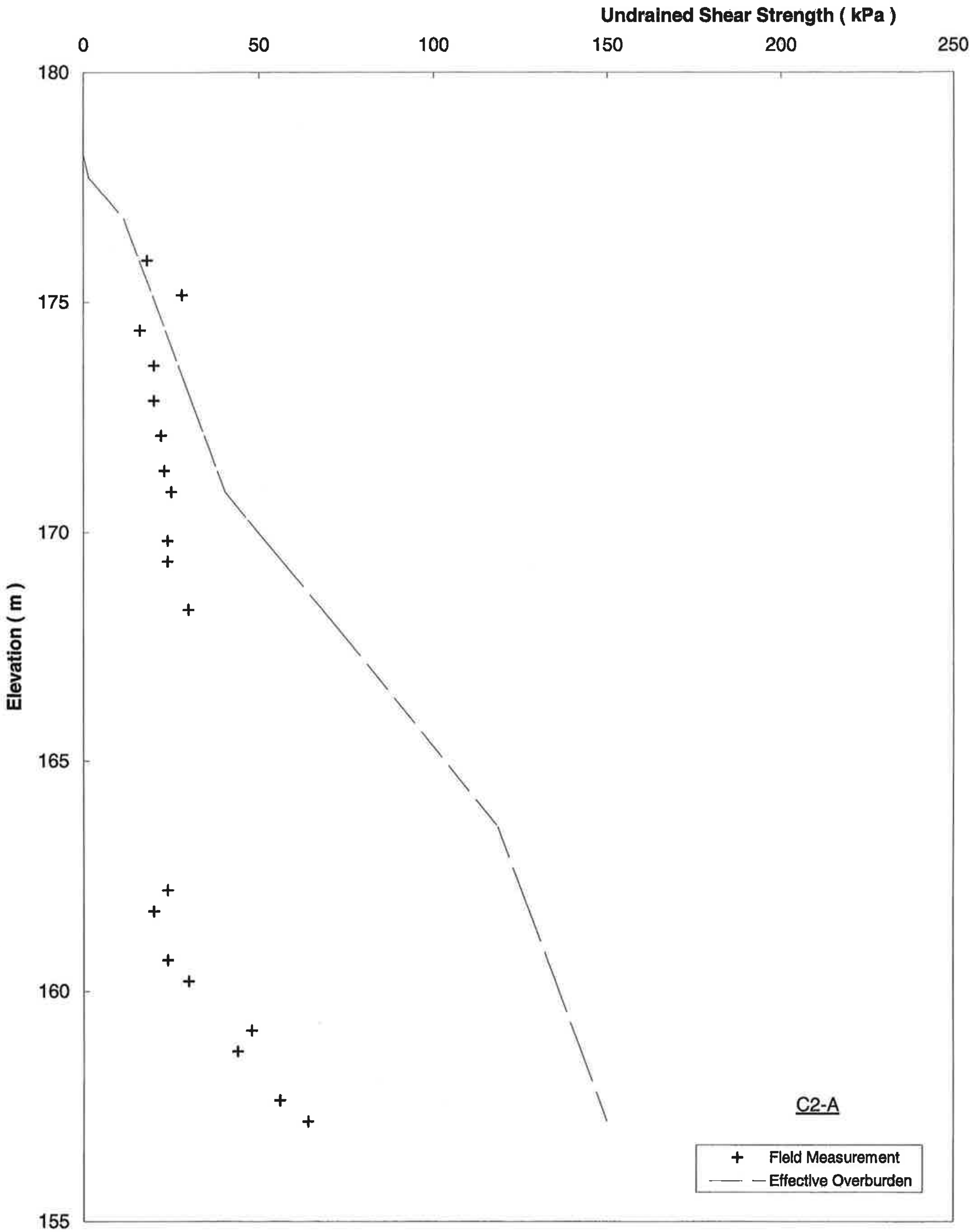


Fig. C2: Variation of Undrained Shear Strength (as measured by field vane tests) with Elevation
(Borehole C2-A)

Appendix D

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = $(w_L - w_P)$
I_L	1	LIQUIDITY INDEX = $(w - w_P)/I_P$
I_C	1	CONSISTENCY INDEX = $(w_L - w)/I_P$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 17 (NEW) TWIN CULVERT STRUCTURE
AT STATION 13+072 MEDIAN CENTRELINE
HIGHWAY 17 (NEW) FROM ECHO RIVER TO BAR RIVER ROAD
DISTRICT 62, SAULT STE. MARIE, ONTARIO
G.W.P. 354 AND 352-94-00
SITE:

GEOCRES No. 41K00-058

Prepared For:

MARSHALL MACKLIN MONAGHAN LTD.

Prepared by:

SHAHEEN & PEAKER LIMITED

Project: SPT1055
August 27, 2003

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SAULT STE. MARIE, ONTARIO
G.W.P. 354 AND 352-94-00
SITE:

5. DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

As part of the realignment of Highway 17 to the east of the existing highway between the Lower Echo River to Bar River Road, twin culverts (one under EBL and another under WBL) will be constructed at Station 13+072 under the proposed four-lane Highway 17. The twin culverts will be located generally about 15 m south of the existing creek (i.e. the creek will be relocated).

Based on the drawings provided by Marshall Macklin Monaghan Limited, the proposed box culverts will consist of 5.7 m wide by 2.2 m high rigid frame twin concrete culverts of 38 m and 34 m in lengths under the proposed WBL and EBL, respectively. The proposed bottom elevation of the culverts along the WBL and EBL centrelines are 176.5 m and 176.4m, respectively, or about 1.7 to 2.0 m below existing ground surface. We also understand that the culverts will be separated into two structural units in the vicinity of the median centreline with a horizontal distance of about 2 m in between.

As shown in Drawing No. 1, the three boreholes (Boreholes C2-B, C2 and C2-A) drilled and sampled during this investigation provide subsurface information along the proposed culvert alignment. In general, below the peat/topsoil and very loose silty sand layers extending to depths of 1.1 to 1.4 m, the site is underlain by a very soft to firm clay deposit to depths of 4.4 to 7.3 m, which is further underlain by very loose to compact and/or soft to stiff silt to sandy silt. This layer is about 7 to 10 m thick and is in turn underlain by a lower soft to stiff clay deposit which extends to at least 30 m below existing grade. DCPT performed below the bottom of the central borehole (Borehole C2) leads us to believe that this weak and compressible clay deposit probably extends without interruption to a depth of about 38 m (Elevation 140 $\pm$ m). Below this elevation the soil appears to be relatively more competent. The groundwater level along the culvert alignment is believed to be at about the ground surface, but may fluctuate throughout the year.

The existing ground surface along the alignment of the proposed culvert is flat at about Elevation 178.3 m. The bottom of the existing creek, which is about 15 m north of the proposed culverts, is approximately at Elevation 176.5 m, which is also about the proposed bottom of the culverts. The anticipated height of the proposed embankment above the original grades, at the culvert location, is approximately 1.5 m or Elevation 179.8 $\pm$ m. From the construction point of view, the area of the new alignment has the advantage of constructing the culverts away from open water of the existing creek, which would facilitate construction. From our knowledge of the overall project, it is anticipated that at this location the Highway 17 embankment will be constructed and surcharged by 1.5 m of additional fill for about two years, prior to paving of the roadway.

We understand that, based on environmental considerations, an open footing type culvert would normally be the preferred option since this culvert type will have less impact to the fish habitats and environment of the creek. However, the prevailing very soft to firm clay below the anticipated structure bottom elevation of 176.5 m is considered unsuitable to support normal spread footing foundations.

In addition, regardless of the foundation type selected, excessive settlements can be expected which will require preloading. Furthermore, an uplift condition due to hydrostatic pressures is possible during construction in areas where the relatively more pervious silt to sandy silt, underlying the practically impervious weak clay deposit, is close to the bottom of the excavation (i.e., Borehole C2 and especially C2-B locations).

Normally, with weak clay subgrade conditions, steel pipe culverts on a granular engineered fill or if the use of steel pipe culvert is undesirable, then closed bottom (box) concrete culverts (prefabricated sections) also supported on engineered fill provide better solutions. In this case, twin concrete culverts are required and therefore the latter will have to be resorted to.

Alternatively, the twin culverts can be supported on deep foundations but the lack of a well defined bearing stratum and the presence of deep weak and compressible clay, together with anticipated difficulties and cost of supporting a pile driver at the site render the use of deep foundations rather uneconomical and impractical.

With this preamble, the following is a discussion of available foundation options.

5.2 FOUNDATION OPTIONS

The boreholes show the presence of compressible layers of soft to firm clay deposit and very loose to loose silt to sandy silt with lenses of compact zones of variable thickness at the site. The thickness and degree of compactness of these silty deposits are inconsistent through the three boreholes drilled along the culvert alignment. Therefore, the presence of the compact or slightly competent material is not considered reliable beneath the full length of the proposed culverts. Due to these conditions, in order to install the piles or any other foundation types, the weak and compressible layers need to be preloaded/surcharged to effect the settlements.

Among the deep foundation alternatives, drilled caissons are not suitable due to a lack of a suitable end bearing stratum. Driven piles can be considered but again lack of a suitable end bearing stratum means that friction piles will need to be utilized. The possible presence of excessive hydrostatic pressure within the water-bearing silt to sandy silt deposit extending to about Elevation 164 $\pm$ m, indicates that timber piles are better suited because of their tapered shape, in comparison with steel piles. They can also be driven with lighter, less sophisticated equipment. This is advantageous since the prevailing soils will not support heavy equipment to drive the piles.

The following table provides a summary of foundation alternatives.

TABLE 5.2.1 Foundation Type Summary

Foundation Type	Comments	Recommendations
Normal Spread Footings	Not feasible due to very low shear strength and highly compressible clays	Not recommended based on reliability
Normal Spread Footings on Compacted Granular 'A' pad	Not feasible due to very low shear strength and highly compressible clays	Not recommended based on reliability
Flexible Steel Pipe Culvert on Compacted Granular 'A' pad after strengthening the subgrade by pushing rockfill plus a sufficient surcharging period prior to construction	Considered feasible	Represent the best alternative with the prevailing subsurface conditions. However, this type of structure may not be acceptable to MTO
Closed Bottom Precast Concrete Box Culvert on Compacted Granular 'A' pad after strengthening the subgrade by pushing rockfill plus a sufficient surcharging period prior to construction	Considered feasible	Represents the best alternative to steel pipe culverts with the prevailing subsurface conditions
Drilled Caissons	Not feasible due to prevailing soil conditions	Not recommended based on cost and reliability
Steel H – Piles	Not feasible due to a lack of suitable bearing stratum	Not recommended based on reliability and economics
Steel Tube Piles	Not a good choice due to a lack of suitable bearing stratum	Not recommended based on economics and reliability but considered a better alternative, if necessary, than H-Piles
Timber Piles	Will require Granular 'A' pad and surcharging	Can be considered as an alternative to supporting the structures on strengthened subgrade + granular pad, but more costly

The recommended option (i.e., box culvert on granular pad with improved subgrade and surcharging) as well as the use of timber piles are discussed in the following sections.

5.2.1 CULVERT FOUNDATIONS ON GRANULAR PAD

As mentioned above, since pipe culverts are not favored by MTO, box culverts are preferred at this site, rather than open bottom concrete culverts. These culverts can usually tolerate more differential settlements with proper design of the box sections in comparison with cast-in-place concrete culverts. Considering the presence of soft to firm clays and very loose to loose or soft to firm silt to sandy silt below the proposed structure, excessive settlements of the foundation soils under the weight of the embankment are anticipated and therefore, preloading/surcharging will be required prior to the construction of the concrete box culverts. The separation between the culverts under WBL and EBL should also help to alleviate some of the adverse effects of any differential settlements.

The new culverts under the WBL are expected to be founded at Elevations 176.57 m and 176.48 m at the east and west limits, respectively, while the culverts under the EBL will be founded at Elevations 176.48 m and 176.40 m at the east and west limits, respectively. Boreholes C2-B and C2, located under the proposed WBL, encountered peat extending to depths between 0.2 m and 0.3 m below existing grade. Along the EBL, the boreholes (Boreholes C2-A and C2) encountered peat/topsoil to a depth of 0.2 to 0.5 m below existing grade. These are underlain by a surficial silty sand deposit to a depth of 1.1 to 1.4 m (Elevation 177.4 to 176.6 m) which is in turn underlain by clay. The invert elevations can therefore be expected to be in the clay, which is very weak and compressible.

In order to avoid excessive settlements, to provide a uniform founding subgrade condition and to improve the load carrying capacity of the upper zones of the founding soils, we recommend that the excavation be carried out to about 0.5 m below the proposed founding elevation within an area extending at least 3 m beyond the perimeter of the structures. The excavation should be carried out in conjunction with the stripping and/or sub-excavation requirements for the Highway 17 embankment. From the shear strengths of the soil and the presence of water-bearing silt to sandy silt under excess hydrostatic pressure, basal heave may occur. Based on this, it is recommended that the excavation be carried out in short sections (e.g., in about 4 to 5 m wide strips) and should be backfilled immediately with Granular 'B' Type II material to about 0.3 m above the proposed founding elevation and then by suitable granular materials to ground surface as soon as possible, followed by surcharging. The surcharging, which would entail the raising of the grade to 1.8 m above the final finished grade, should be carried out at the same time under the WBL and EBL to minimize differential settlements. In addition, to effect the settlements at the west end of the of the EBL culvert and at the east end of the WBL culvert, we recommend the placement of 2 m high and 4 m wide (from the toe) temporary berms during surcharging. As well, the surcharging should extend along the entire width of the highway, including the median section.

The following are the details of the proposed procedures:

- Dewater the site within the peat and in the underlying silty sand. Some of the possible methods to accomplish this include gravity drainage and pumping from perimeter trenches extending to about 0.25 m into the clay deposit underlying the surficial silty sand, including pumping from strategically placed, filtered sumps; pumping from within tight interlocking sheet pile enclosures around the perimeter of the site; reducing the flow from the adjacent creek by means of sand bags or a similar cofferdam along with gravity drainage and pumping; or a combination of some of these methods, etc.

- After dewatering, the site would be excavated to 0.5 m below the founding level of the structures within the footprint and extending at least 3 m beyond the perimeter. Excavation should be carried out in about 4 to 5 m wide strips. Immediately upon excavation rockfill should be pushed into the subgrade to a depth of at least 0.6 m below the bottom of the excavation (i.e., to a depth of at least 1.1 m below the founding elevation). The maximum size of rockfill should be 300 mm with an average size between 150 and 200 mm with no size less than about 70 mm. The purpose of this exercise is to strengthen the very weak subgrade.

- Upon pushing in the rock to strengthen the subgrade, each strip should be immediately backfilled with Granular 'B' Type II or Granular 'A' material to at least 0.5 m above the founding elevation (i.e., to about Elevation 177.0 m). The next 4 m wide strip can be constructed in the same manner while effecting, if necessary, additional dewatering from filtered sumps within the granular fill. No heavy construction equipment or vehicle should be operated on the prepared granular fill surface until the grade was raised to at least 0.7 m above the founding level (i.e., the grade should be at Elevation 177.2 m or higher).

- Above Elevation 177.0 m, another type of granular fill such as Granular 'B' Type I material may, if desired, be used for backfill to raise the grade to the existing ground level. These operations should preferably be performed simultaneously or prior to placing the embankment fills in the vicinity of the culvert structures. If the culvert sites are prepared after the preparation of the embankment fills and surcharging, the existing embankments will need to be removed to about the existing grade level within 8 m of the culvert excavations in order to prevent a failure of the weak clay during the excavation process due to the weight of the adjacent embankment fills. The side slopes for the excavations in the natural soils and in the existing embankment should be cut at no steeper than 4H:1V side slopes (see Figure E-1 and E-2 in Appendix E).

- After the site is prepared in this manner, some light compaction of the granular material can be effected (e.g., operating track-mounted construction equipment). The grade can then be raised to the existing ground surface level and beyond by placing the backfill in lifts not exceeding 0.2 m thickness and compacting each lift in this manner.

- The surcharge fills will need to be extended to 1.8 m above the final highway grade (i.e., top of proposed asphalt elevation) as shown in Figure E-3 in Appendix E. The surcharge materials can consist of clean inorganic earth fills (preferably granular fill materials) which should also be lightly compacted, as discussed. The 1.8 m high surcharge should extend at least 4 m beyond the footprint of the culverts. Beyond this zone the anticipated height of the surcharge is 1.6 m.

- After a minimum surcharge period of about 1.5 years, the site would be subexcavated to the proposed founding level. The excavation should extend at least about 2.5 m beyond the perimeter of the culvert. The excavation of the existing embankment fills should be carried out at least 6 m beyond the perimeter of the excavation in order to prevent a failure of the weak clay at the base of the excavation, underlying the granular pad, as shown in Appendix E. In addition, the temporary side slopes of the embankment fill should be no steeper than 4H:1V. Dewatering should be carried out to intercept and remove any surface water above the clay deposit (similar to original excavation dewatering) as well, if necessary, from within the granular subgrade material in order to provide a reasonably dry working condition to carry out the construction and to prevent a possible bottom heave condition. If sufficiently dry conditions can not be maintained and/or a bottom heave condition seems to occur, dewatering will be required to lower the hydrostatic pressures within the silt to sandy silt deposit underlying the clay. The possibility of this occurrence is considered small, especially within the west half of the site where the upper clay deposit appears to be thicker.

- If conditions permit, the granular pad materials (i.e. Granular 'B' Type II) should be lightly compacted from the surface, before receiving the prefabricated concrete units. The sections should be placed as expeditiously as possible and backfilled with granular materials. All construction should be carried out without delay.

- The construction should be performed under the supervision of the Quality Verification Engineer (QVE) who should also inspect and approve all bearing surfaces.

A Factored Geotechnical Resistance at U.L.S. of 120 kPa and a Geotechnical Resistance at S.L.S. equal to 75 kPa can be assigned to the founding granular subgrade (top of granular pad Elevation 176.48 to 176.57 m) prepared and surcharged in this manner.

If this procedure is unacceptable because it involves double handling (i.e. preparing site prior to preloading as well as dewatering the site twice) then, if some moderate risk is acceptable, the procedure can be modified as follows.

After normal stripping and preparation of the embankment, the site should be surcharged by 1.8 m of additional fill (i.e. over and above the finished highway grade). After a surcharging period of not less than 1.5 years, the surcharging and the embankment fills would be removed to a depth of about 0.3 m of the original ground level (i.e. to about El. 178.5 m). The removal of the embankments would be carried out to at least 10 m beyond perimeter of the culvert foundation excavation (see Figure E-5 in Appendix E). The site would then be properly dewatered, as discussed before, and sub-excavated, also as discussed before, within an area 3 m beyond the perimeter of the proposed culvert footprints, to a depth of at least 0.6 m below the founding level of the structures (i.e. generally to El.175.9 m). The excavation should be carried out in narrow strips, not exceeding 4 m in width. Rockfill would be immediately pushed into the subgrade, as was discussed before, in order to strengthen the weak clay. This operation must be performed without any delay. Upon pushing the rock into the clay subgrade the excavation would be immediately backfilled to the proposed founding level (i.e. El. 176.5 ± m) with Granular 'B' Type II material, before proceeding with the excavation of the next strip. After the site is prepared in this manner, compaction can be applied to the granular pad. This must, however, be done with extreme caution in order not to disturb the underlying weak clay subgrade.

As mentioned before, this process will be less costly than previously discussed foundation pad preparation, since it involves only a single operation of site excavation and dewatering. However, it also involves some risks which include a less reliable granular pad which may not be properly compacted, possible uplift during foundation pad preparation, deeper sub-excavation as well as wider site preparation. It is, therefore, considered a significantly less desirable method, based on reliability.

After the surcharging of the foundation soils for the culverts above the final grade levels with 1.8 m of suitable soils for at least 1.5 years, the anticipated post construction settlements are of the order of 25 mm. The total settlement under the proposed embankment at the culverts location is estimated to be 300 mm. About 200 mm of this figure is estimated to take place as consolidation settlements (about 120 mm in the upper clay deposit and 80 mm in the lower clay deposit) and the other 100 mm as elastic (including settlement of the silt layer). The 1.8 m surcharge is expected to generate 175 mm consolidation settlement in 1.5 years and the post construction settlement is expected to be about 25 mm (i.e., total anticipated settlement of 300 mm minus 275 mm of settlement during construction and surcharging). Most of this residual settlement can be expected to take place within 5 to 8 years after the completion of construction of the roadway. These total settlements can translate into differential settlements between the centre and the ends of the culverts, and therefore we recommend that construction joints be introduced in order to mitigate any adverse effects of differential settlements. Consideration should also be given to providing water tight joint treatment. Provided that flexible, water-tight joints are utilized, camber is not required for this project.

5.2.2 TIMBER PILES

Based on the findings of Boreholes C2, C2-A and C2-B, the use of timber piles is also considered a feasible foundation alternative. The piles should be driven to Elevation 165 m where they will derive their resistance from both end bearing and friction/adhesion.

Timber piles with a butt head diameter of 350 mm and driven to Elevation 165 m can be expected to provide a Factored Axial Resistance at U.L.S. of about 140 kN and an Axial Resistance at

S.L.S. of about 90 kN per pile. As the existing grade is about 178 m, the anticipated length of timber piles below the existing ground surface is about 13 m.

To confirm the resistance values presented above, it is recommended that pile load testing be carried out.

Timber piles should be of sound structural quality and should conform to the current CSA Standards and the requirements of the applicable building code.

The elevation of the tops of the driven piles should be measured immediately after driving. If uplift occurs in any piles during subsequent driving of adjacent piles, the displaced piles should be re-driven to their previous elevation.

The site should be prepared and surcharged in the manner described in the previous section of this report except rock fill should not be pushed into the soil, as this may impede the driving of the piles. The piles should be driven after a minimum surcharging period of 1.5 years.

As mentioned before, supporting the structures on engineered granular pad after strengthening the subgrade is the preferred option at this site, rather than piles.

5.3 DESIGN FEATURES

The culverts should be designed to resist frost forces, weight of embankment fill, hydraulic and earth pressures and traffic loadings.

Design frost penetration for the general area is 1.8 m. For frost protection, the footings should have a permanent earth cover of at least 1.8 m or in case of a box culvert, the structure may be designed to resist frost forces. The depth of the foundations, including cut-off walls and retaining walls, should be determined on the basis of frost and scour depths, whichever is greater. In computing frost protection, only one half of the thickness of rip-rap should be considered.

The unfactored horizontal resistance against sliding between concrete and Granular 'B' Type II or Granular 'A' type material can be calculated using a friction angle of 35 degrees, although sliding is unlikely to a problem for the culverts.

A design feature regarding closed bottom box type culverts is potential uplift pressures that could be generated by the high water level prevailing at the site. These uplift pressures will be resisted by the weight of the structure and the permanent cover above the top of the culvert structure. The side friction will also have some contribution but to a much smaller degree. If the weight is insufficient to resist the uplift forces (assuming the groundwater level at about the ground surface level), consideration could be given to extending the bedding material along the sides of the culvert while ensuring proper geotextile protection against the infiltration of the silty sand. A 100 mm diameter weeper pipe (properly filtered to prevent soil fines from the surrounding bedding material entering the pipe) could then be placed about 0.9 m above the invert of the culvert, subject to approval by the Structural Engineer. Water collected in the backfill and the weeper pipe could be suitably discharged in a frost free manner. This aspect is also discussed in Section 5.5 of this report.

5.4 BACKFILLING

Backfill arrangements around the culvert should be carried out as per OPSD 803.02. Backfill to the culvert should consist of free-draining, non-frost susceptible granular materials in accordance with OPSS 1010. The excavated material is not suitable for backfilling purposes due to its high frost susceptibility and high (wetter than optimum) natural moisture contents. All granular fill should be placed in loose lifts not exceeding 200 mm thick and be compacted to at least 95% of its SPMDD.

Heavy compaction equipment should not be used adjacent to the walls and roof of the culvert. The height of the backfill to the culvert walls should be maintained equal on both sides of the structure during all stages of backfill placement.

Since the cover above the proposed culvert is only about 1.0 to 1.4 m and considering that embankment material in this area could probably consists of earthfill, the embankment above the proposed culvert should be constructed using granular materials (e.g., Granular 'B' or Granular 'A').

5.5 LATERAL EARTH PRESSURES

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B') and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures acting against rigid culvert walls should be in accordance with C.H.B.D.C. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A' or Granular 'B' Type II
Unit Weight = 22 kN/m^3
Coefficient of Lateral Earth Pressures:
Ka = 0.27
Ko = 0.43

Compacted Granular 'B'
Unit Weight = 21 kN/m^3
Coefficient of Lateral Earth Pressures:
Ka = 0.33
Ko = 0.50

These values are based on the assumption that the backfill behind the rigid culvert walls and any retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind any retaining structure is level.

The earth pressure coefficient adopted will depend on whether the culvert walls or any retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the culvert wall is restrained and does not allow lateral yielding, then at rest pressures should be used as per Clause 6.9.2 of CAN/CSA S6-00 C.H.B.D.C. current edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6.9.2 of CAN/CSA S6-00 C.H.B.D.C..

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

The foundations for any conventional wingwalls can be supported on engineered fill and/or timber piles as discussed in previous sections of this report. Surcharging should be applied, as also detailed in the previous sections. For foundations not more than 1.2 m wide, a Factored Bearing Resistance at U.L.S. of 100 kPa can be utilized, while for Bearing Resistance at S.L.S., a value equal to 60 kPa can be assumed when supported on granular pad constructed (including rock fill push) and surcharged as described before. In case this arrangement is insufficient to provide adequate resistance, consideration can be given to the use of light weight fill behind the retaining structure in order to reduce the loads. Consideration can also be given to a flexible structure such as gabion walls, in view of the prevailing poor soil conditions. Foundations for the gabion type walls will need to be prepared and surcharged in the manner detailed earlier.

As an alternative to conventional retaining walls, if any, MTO's Retained Soil System may be used. The following should be included in the Contract Documents:

- identify longitudinal extent in plan of the Retained Soil System
- identify in plan transverse space constraints (top of wall and bottom of wall)
- identify elevation of top of wall and bottom of wall
- include NSSP for Retained Soil Systems in Contract Documents

The Retained Soil System (RSS) should be of high performance and high appearance.

The design of the RSS, including the foundation for the facing wall of the RSS, is the responsibility of the RSS provider. It should, however, be pointed out that support for the foundation of the wall and the facing of the RSS may also consist of the Granular 'B' Type II or Granular 'A' pad, which would include proper surcharging.

In using RSS in the vicinity of the water course, consideration should be given to possible problems due to erosion.

5.6 EMBANKMENT STABILITY

As discussed in Section 5.1 of this report, the proposed highway grade at the culvert area will be approximately 1.5 m (to Elevation 179.8 m) above the existing grade under the proposed WBL and EBL. In addition, a 1.8 m high surcharge is also proposed and the embankment surcharge material in this section of the swamp area will consist of earthfill.

Slope stability analysis was conducted on the 1.5 m high embankment plus 1.8 m of surcharge (total 3.3 m), assuming that the recommendations provided in this report will be implemented. For the undrained (short-term) stability analysis, undrained shear strengths (c-values) were utilized based on the field vane test results at individual borehole locations. The angle of internal friction was assumed to be zero, as is normally done in undrained stability analysis. The c-values used in our analysis ranged from 12 to 84 kPa. No correction factor (such as Bjerrum or Aas correction) was applied to the field vane test results. A minimum factor of safety of 1.40 was deemed necessary, because of the generally low shear strengths (c-values) that were measured.

Long-term or drained analysis was also carried out at selected borehole locations and, as can be expected, these were found to be less critical than the short-term (undrained) analysis.
The soil parameters used in the slope stability analyses are presented in Table 5.6.1.

Table 5.6.1: Soil Parameters Used in Slope Stability Analyses

Soil Type	Short-Term Analysis			Long-Term Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m^3)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Embankment Fill (select subgrade material)	30	0	21.5	30	0	21.5
Sand Backfill (used to replace existing peat and other surficial unsuitable soils)	30	0	18-20	30	0	18-20
Rock Backfill (used to replace existing peat and other surficial unsuitable soils)	36	0	18-20	36	0	18-20
Surficial Silty Sand	30	0	20.0	30	0	20.0
Clay	0	4-84	15.0	24	2	15.0
Silt to Sandy Silt	26	0	20.5	26	0	20.5

Typical embankment slope stability sections are presented in Appendix E.

Based on the findings of the boreholes, our analysis showed that embankments constructed of earthfill with 4H:1V slopes, as per MTO standard procedures and after the sub-excavation and replacement of organics, is considered stable, as presented in Appendix E.

5.7 CONSTRUCTION COMMENTS

Excavations should be carried out in accordance with the Safety Regulations of the Province (i.e., Occupational Health and Safety Act O.Reg. 213/91), as well as the following specifications:

SP 539S01	-	Protection Schemes
SP 902S01	-	Excavation and Backfilling to Structures

The boreholes show that the excavations can be expected to extend through peat/topsoil, and loose to very loose silt to sandy silt below the groundwater table. Provided that the groundwater is properly controlled, open cut excavations can be expected to stand temporarily at 2H:1V slopes in the silty fine sand and at 4H:1V side slopes in the underlying very weak clay. As was discussed before, if the founding soils for the culverts are prepared after the placement of the road embankment and surcharge, the existing embankment will need to be removed to a considerable distance beyond the perimeter of the culverts' excavation in order not to induce a foundation failure in the clay. This concern also applies to excavation procedures after the surcharging period, as was discussed earlier in this report.

Water can be removed by pumping and the effluent from dewatering operations should be filtered or passed through sediment traps to prevent turbidity.

During the construction, temporary runoff controls such as sediment trap, interceptor drain, dike and/or silt fence should be provided and installed to prevent uncontrolled water flow and sediment down towards the creek.

We recommend that any surface water be diverted away from the culvert excavation, in addition to the chosen groundwater control scheme, to enable the culvert construction and fill placement to be carried out in the dry. Major problems due to groundwater seepage are not anticipated, provided groundwater control is carried out properly.

In order to avoid unbalanced loading on the culvert, the height of the backfill around the culvert should be maintained equal on both sides throughout construction as much as practically possible.

The placement and compaction of fills should be carried out under the supervision of the Quality Verification Engineer (QVE).

5.8 EROSION PROTECTION

Erosion protection should be provided at the culvert inlet (including the slopes and sides) and outlet. We recommend the use of cutoff walls at the inlet and outlet to prevent erosion of the founding granular soil. Headwalls and/or wingwalls should also be provided to ensure that the granular backfill against the sides of the culvert are protected from seepage forces.

Alternatively, clay seal can be provided at the inlet. The purpose of the clay seal is to ensure that water flow is channeled through the culvert and does not seep through the backfill around the structure and underneath the structure. The clay seal should therefore be continuous and at least 0.6 m thick. It should comply with the material specifications given in OPSS 1205. The existing clay is considered suitable for this purpose, subject to verification by the QVE. It should be extended around the culvert from at least 0.3 m above the high water level down to cover the silty sand to the channel bed. It should be ensured that it extends to cover all the granular backfill materials to prevent any seepage through them. The clay seal should be protected by laying a 0.6 m thick rock protection over it. The boreholes show the presence of clay below Elevations ranging between 177.4 m (Borehole C2-B) and 176.8 m (Borehole C2-A), which is considered suitable as a clay seal (i.e., need not be replaced), subject to verification by QVE. It should however be protected by placing suitable rock fill (i.e., at least 0.6 m thick after sinking into subgrade).

At the outlet, a 0.6 m thick layer of rock protection consisting of 300 mm size rock should be used, overlaying a 300 mm thick layer of filter material. The filter material should consist of a granular material such as Granular 'A' or equivalent. Alternatively a suitable geotextile could be used in lieu of the granular filter (i.e. Granular 'A'). This filtered rock protection should extend at least 10 m along the channel from the outlet. It should also extend at least 10 m adjacent to the culvert outlet to the high water level and should protect the granular fill around and beneath the culvert. A headwall can also be used to protect the granular fill around the culvert outlet against erosion. In this case, however, filtered erosion protection such as rip-rap should be provided as per OPSD 810.010 Type B, along the channel and the sides beyond the concrete cut-off, headwalls at the outlet.

The above recommendations are suggestions only. We recommend that a qualified Hydraulics Engineer should be consulted to design the specifics of the channel, and culvert outlet and inlet (i.e. thickness and extent of protection).

5.9 FROST PROTECTION

Design frost penetration for the general area is 1.8 m. Frost protection is not required for the box culvert provided that it is designed against frost forces. But for wingwalls or any other retaining walls, a permanent soil cover of not less than 1.8 m or its thermal equivalent is required.

6. CLOSURE

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

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

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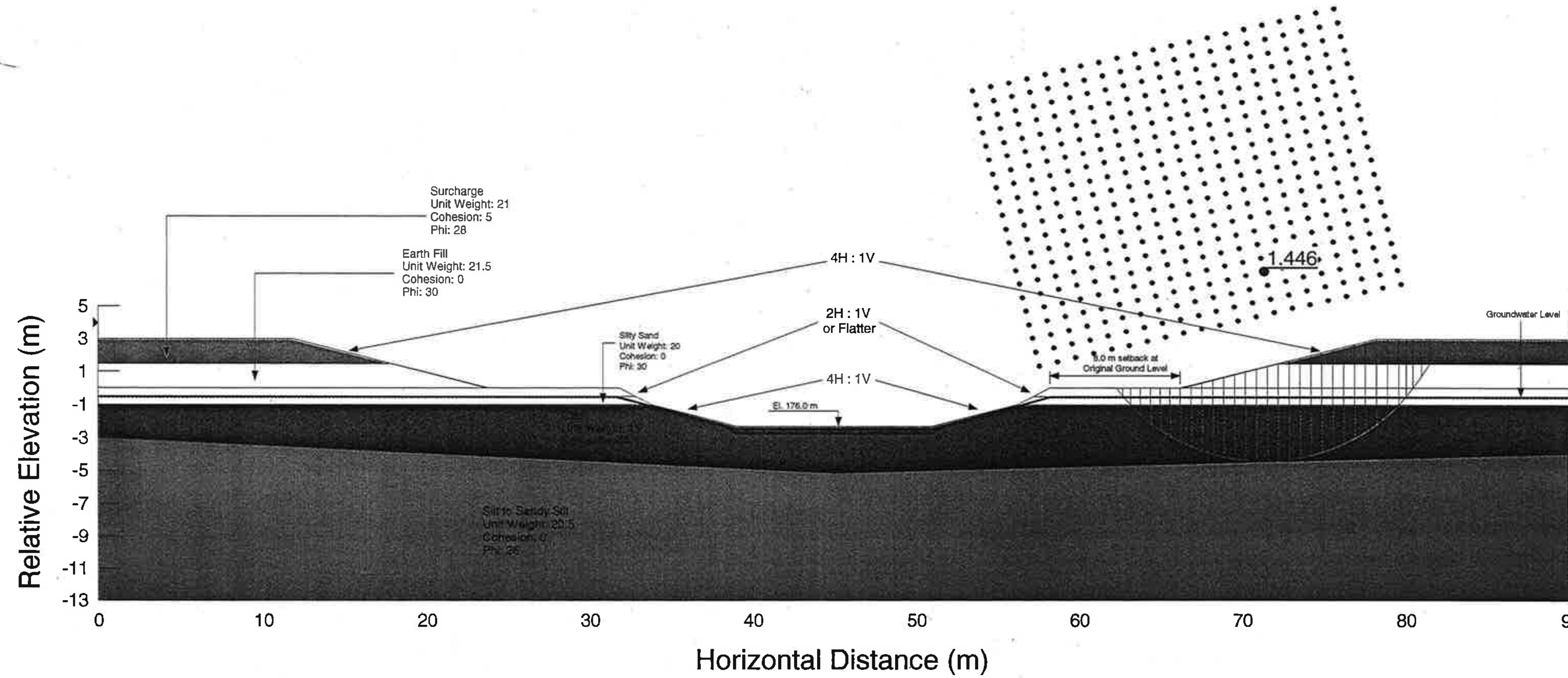
Appendix E

Slope Stability Analysis Results

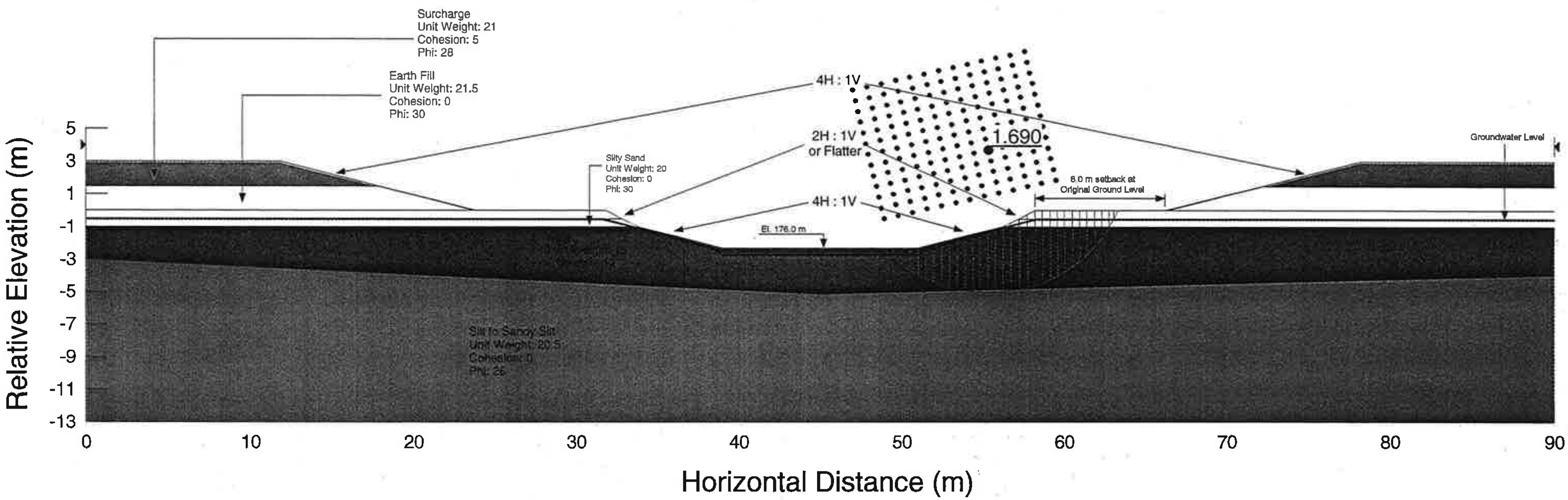
SPT 1055, Highway 17 (New), Sault Ste.Marie
Proposed Culvert Construction, Station 13+072
1.5m High, Earth Fill Embankment (Plus 1.5 m Surcharge)
Undrained Case (Total Stress Analysis)

Figure E-1



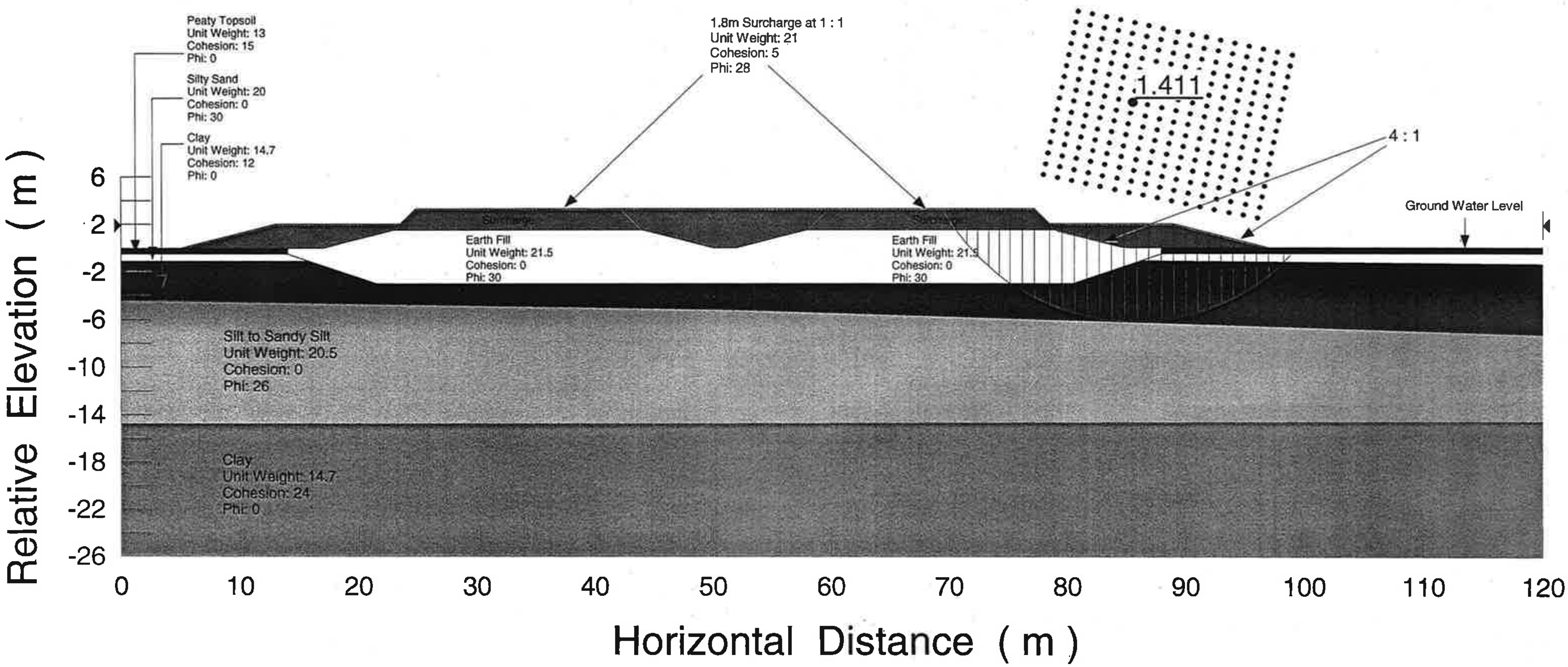
SPT 1055, Highway 17 (New), Sault Ste.Marie
Proposed Culvert Construction, Station 13+072
1.5m High, Earth Fill Embankment (Plus 1.5 m Surcharge)
Undrained Case (Total Stress Analysis)

Figure E-2



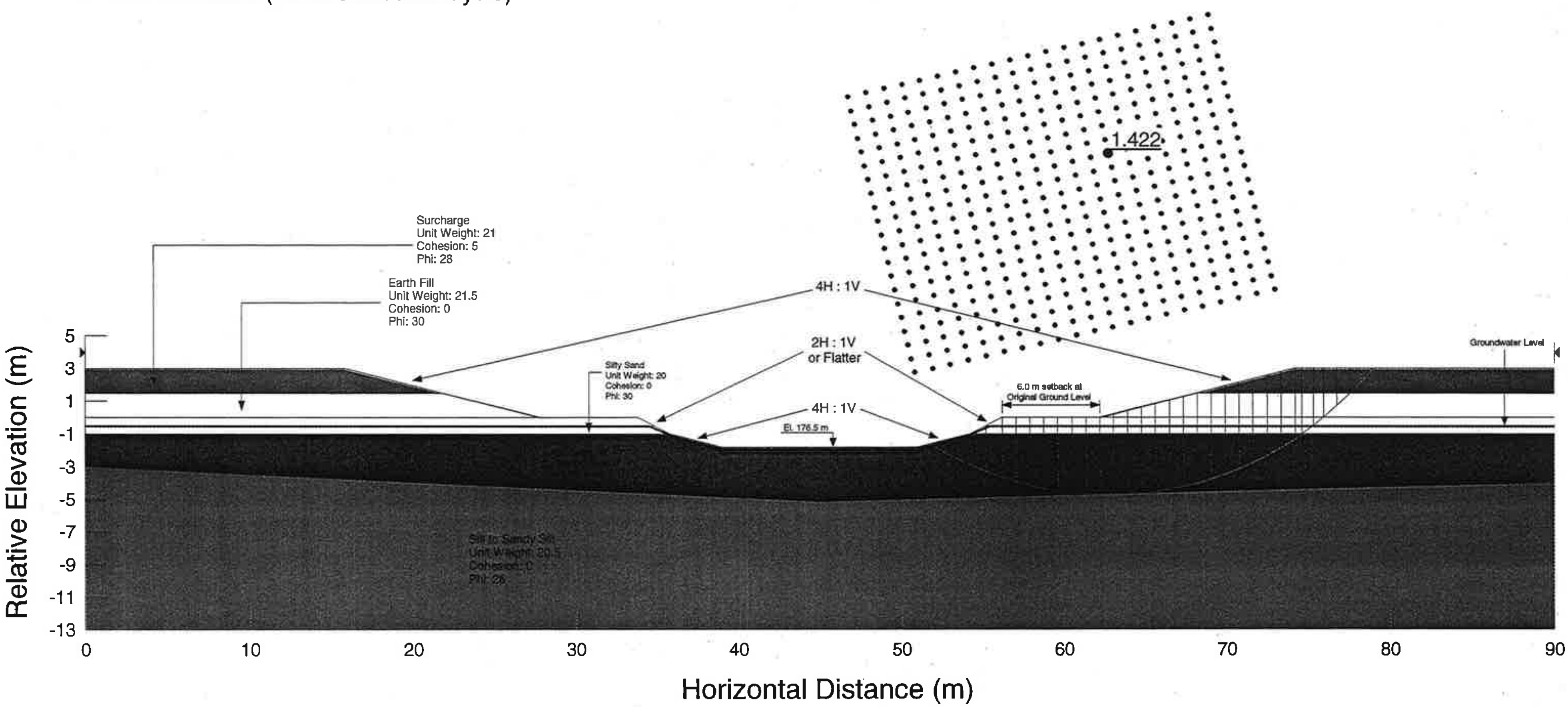
SPT 1055, Highway 17 (New), Sault Ste.Marie
Proposed Culvert Construction, Station 13+072
1.5m High, Earth Fill Embankment (Plus 1.8m Surcharge)
Undrained Case (Total Stress Analysis)

Figure E-3



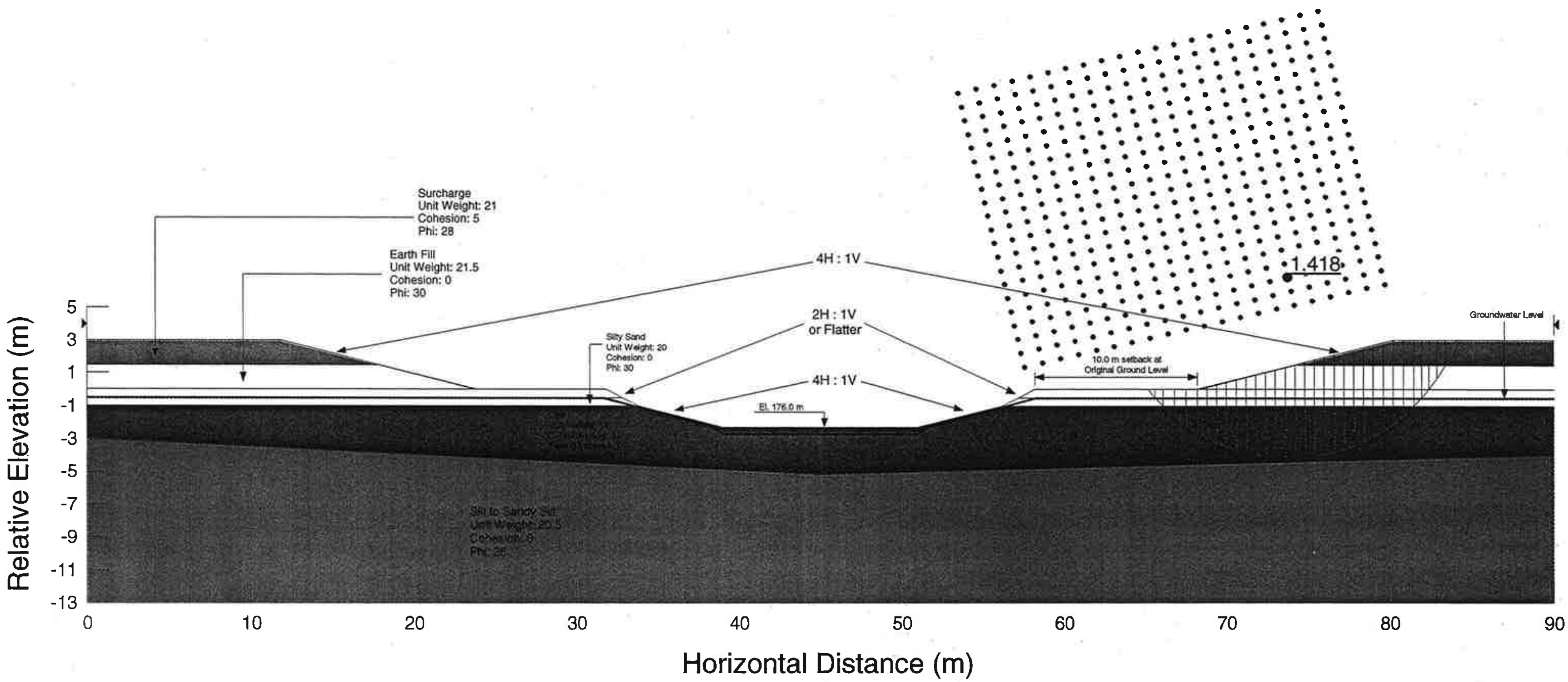
SPT 1055, Highway 17 (New), Sault Ste.Marie
Proposed Culvert Construction, Station 13+072
1.5m High, Earth Fill Embankment (Plus 1.5 m Surcharge)
Undrained Case (Total Stress Analysis)

Figure E-4



SPT 1055, Highway 17 (New), Sault Ste.Marie
Proposed Culvert Construction, Station 13+072
1.5m High, Earth Fill Embankment (Plus 1.5 m Surcharge)
Undrained Case (Total Stress Analysis)

Figure E-5



Appendix F

Limitations of Report

LIMITATIONS OF REPORT

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.