

**FOUNDATION INVESTIGATION REPORT
PROPOSED HIGHWAY 17 (NEW) SNOWMOBILE CULVERT STRUCTURE
AT STATION 10+875 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. 41K-064

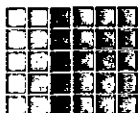
Prepared For:

MARSHALL MACKLIN MONAGHAN LTD.

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1055
October 8, 2003**



**20 Meteor Drive
Toronto, Ontario
M9W 1A4**

**Tel: (416) 213-1255
Fax: (416) 213-1260**

WEB SITE: WWW.SHAHEENPEAKER.CA

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G.W.P. 354 AND 352-94-00
SITE:**

1. INTRODUCTION

Realignment of Highway 17 to the east of the existing highway will include the construction of a culvert for a snowmobile crossing under the proposed highway at Station 10+875, about 1 km south of the Lower Echo River. The new culvert, which is about 110 m in length under the proposed Highway 17(New), will have a clear opening of about 4.0 m high and 4.0 m wide to accommodate snowmobile trail grooming equipment, in addition to normal snowmobile traffic.

Shaheen & Peaker Limited (S&P) was retained by Marshall Macklin Monaghan Ltd. to prepare a foundation investigation report for the proposed culvert, based on existing borehole information at the site (i.e., Site No. 3 of the previous investigation by S & P *).

The findings of four boreholes (Boreholes 10+860 CL, 10+885 Lt, 10+900 CL and 10+923 Rt) from the previous investigation, which are closest to the proposed culvert alignment, are presented in this report.

2. SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located in a low-lying area, about 1 km south of Lower Echo River, on the west side of the existing Echo Lake, as shown in Drawing 1.

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 40 m. 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

* "Foundation Investigation Report, Proposed Highway 17(New)
From Echo River to Bar River Road, District 62, Sault Ste. Marie, Ontario
G.W.P. 354 and 352-94-00", prepared by S & P dated August 2003.

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 at the culvert site slightly slopes down towards the northeast with the existing ground elevation at the borehole locations varying from Elevation 189.6 m at Borehole 10+923 Rt to Elevation 188.1 m at Borehole 10+885 Lt. Further north of the site at a distance of about 100 m, the existing grade rises to a high ground area with elevation of 198 m at about Station 10+770.

3. INVESTIGATION PROCEDURES

The four boreholes (Boreholes 10+860 CL, 10+885 Lt, 10+900 CL and 10+923 Rt), which are located closest to the proposed snowmobile culvert location, were drilled on April 3 and 4, 2002 for the proposed Highway 17 (New) embankment. The borehole locations are shown on the Borehole Location Plan, Drawing No. 1 and on the Borehole Locations & Soil Strata, Drawing No. 2.

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 10.2 to 13.3 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 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.

A Dynamic Cone Penetration test (DCPT) was also performed from the bottom of Borehole 10+900 CL, from a depth of 13.3 m to 29.0 m below existing ground surface. 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. 2.

Upon their completion, the boreholes were backfilled to about 6 m below the ground surface with clay 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. Where piezometer was installed, the bottom section of the open hole was backfilled with clayey soil to about 0.3 m below the bottom of the piezometer. The slotted section of the piezometer was backfilled with clean sand extending up to about 0.3 m above the slotted section. Above this depth, the borehole was then grouted up to the ground surface with 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 drilled in this area show, below about 0.15 to 0.3 m of topsoil, the presence of an extensive clay deposit. The boreholes were terminated within this clay deposit at depths ranging from 10.2 to 13.3 m. At the location of Borehole 10+900 CL, as mentioned before, a Dynamic Cone Penetration test (DCPT) was extended from the bottom of the borehole at 13.3 m to a refusal depth of about 29 m (Elevation 159.7 m). The groundwater level at the site was found to be close to the existing ground surface, but can be expected to fluctuate seasonally and in response to major weather events.

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

4.1 TOPSOIL

The boreholes encountered 0.15 to 0.3 m of topsoil.

4.2 SURFICIAL SAND

In Borehole 10+923 Rt, a 0.2 m thick silty sand layer was contacted below the topsoil and this extends to a depth of 0.4 m below existing grade. Measured N-value in this layer was 4 blows/0.3 m indicating very loose relative density.

4.3 CLAY

Below the topsoil and surficial silty sand, all the boreholes encountered a deep clay deposit extending to a depth of at least 10 m below existing grade. Boreholes 10+860 CL, 10+885 Lt and 10+923 Rt were terminated in this deposit at a depth of 10.2 m below existing grade. In Borehole 10+900 CL, which was terminated in the clay at a depth of 13.3 m, the presence of a somewhat 'stiffer' and a competent strata were inferred from a DCPT at depths of about 24 and 28 m, respectively.

The clay in this area has a reddish grey colour and consists of high plasticity material with some intermediate and occasional low plasticity zones / layers.

The results of the grain-size distribution analyses performed on three of the selected clay samples are presented in Figure B-1, Appendix B. They indicate the following particle size distribution:

Gravel	=	0%
Sand	=	0%
Silt	=	13 to 15%
Clay	=	85 to 87%

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

Liquid Limit :	61 to 85%
Plastic Limit :	24 to 27%
Plasticity Index :	34 to 59%

As presented in Figure B-2 in Appendix B, these values are characteristic of clay soils of high plasticity. The measured natural moisture contents generally range from 36 to 88%, that is, generally at or in excess of the liquid limit values (except in the upper desiccated zone near the ground surface). The Liquidity Index values range between 0.2 and 1.4, but generally 1.0 to 1.4. The results which are obtained from below the upper 1 to 2 m thick

desiccated zone generally indicate weak and compressible (generally normally consolidated) clays.

Two consolidation tests were performed on samples from this deposit (from Boreholes 10+860 CL and 10+900 CL) and the results are shown in Figures B-3 and B-4 in Appendix B. These tests indicate a probable preconsolidation pressure (P_c) of about 80 to 100 kPa, which is about 65 to 80 kPa in excess of the existing overburden pressure (P_o). The test results also show high C_c -values (e.g., in the range of 1.8) indicating very compressible clay structure, especially beyond the preconsolidation pressure range. Specific gravity of these two samples was also measured and this ranged from 2.72 to 2.75.

The measured bulk unit weights range from 16.4 to 17.6 kN/m³ within the upper desiccated zone, and are between 14.3 and 15.0 kN/m³ within the weaker clay below.

Standard Penetration tests performed in this deposit gave N-values varying generally between 1 and 2 blows/0.3 m, except in the upper 1.5 m of the deposit where N-values of 3 to 6 blows/0.3 m were recorded. Field vane tests yielded undrained in-situ shear strength values ranging from 58 to in excess of 100 kPa within the top 1.5 ± m of the deposit, and ranging from 18 to 49 kPa below this depth. These values indicate that the consistency of the material can be described as very stiff to stiff in the upper 1.5 m of the clay deposit and soft to firm below.

The variation of measured undrained shear strengths with elevation as measured by field vane tests in the boreholes is presented in Figure C-1 in Appendix C. Figure C-2 in the same Appendix shows typical plot of undrained shear strength versus elevation at the location of Borehole 10+900 CL.

4.4 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 6.1 and 8.2 m below existing grades, but these are unlikely to represent the stabilized water levels.

To enable us to monitor the groundwater level over a prolonged period of time without interference from surface water, a piezometer was installed in Borehole 10+860 CL. Water level in the piezometer was measured on October 19, 2002 or about six months after completion of the borehole, and the recorded water level was at a depth of 0.6 m below existing ground surface or at Elevation 188.0 m. Based on the above observations and the colour of the soil, the groundwater level is expected to be near the existing ground surface.

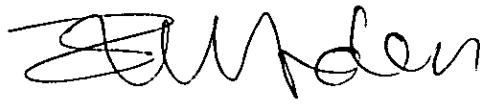
The groundwater table can be expected to fluctuate seasonally and in response to major weather events.

Yours very truly,

SHAHEEN & PEAKER LIMITED


R. Miranda, P.Eng.



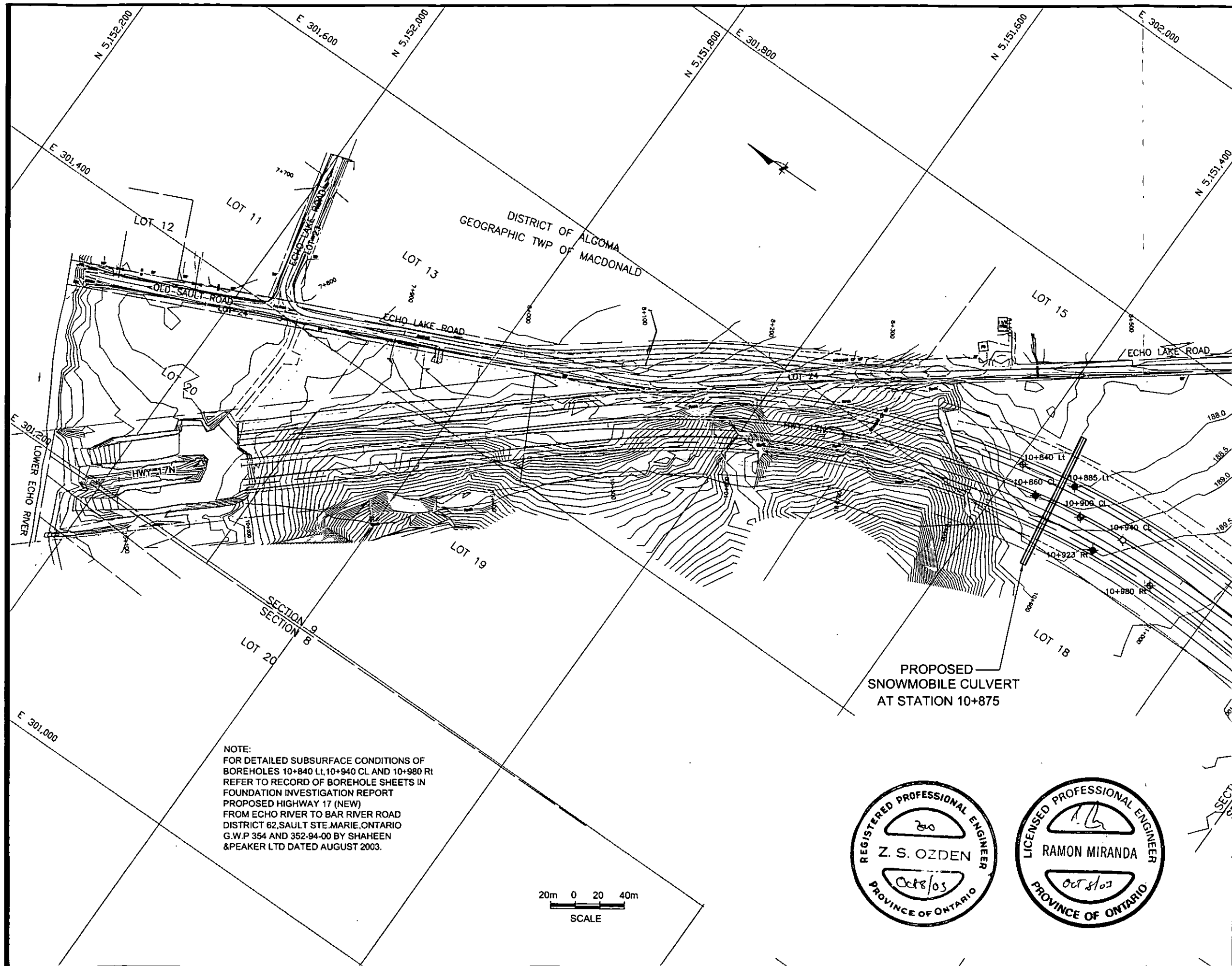

Z. S. Ozden, P.Eng.



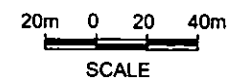
*Project: SPT1055
Marshall Macklin Monaghan Ltd.*

*Foundation Investigation Report
Proposed Highway 17 (New) Snowmobile Culvert Structure
At Station 10+875 Median Centreline
Highway 17 (New) from Echo River to Bar River Road
District 62, Sault Ste. Marie, Ontario*

Drawings

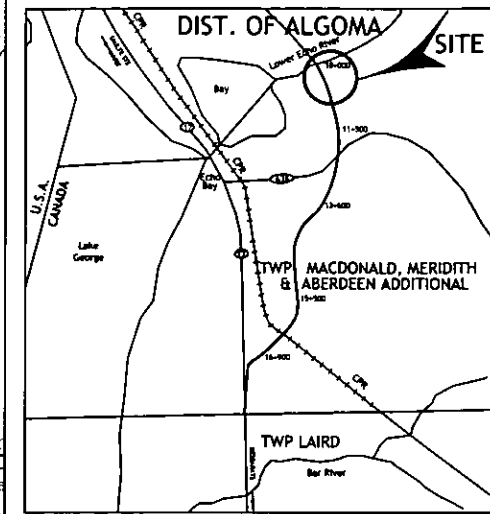


NOTE:
FOR DETAILED SUBSURFACE CONDITIONS OF
BOREHOLES 10+840 LT, 10+940 CL AND 10+980 RT
REFER TO RECORD OF BOREHOLE SHEETS IN
FOUNDATION INVESTIGATION REPORT
PROPOSED HIGHWAY 17 (NEW)
FROM ECHO RIVER TO BAR RIVER ROAD
DISTRICT 62, SAULT STE. MARIE, ONTARIO
G.W.P 354 AND 352-94-00 BY SHAHEEN
& PEAKER LTD DATED AUGUST 2003.



CONT No.
GWP: 354-94-00
HIGHWAY 17 (NEW)
ECHO RIVER TO BAR RIVER ROAD
SNOWMOBILE CULVERT AT STATION 10+875
BOREHOLE LOCATION PLAN

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S.

- LEGEND
- ◆ Bore Hole
 - ◆ Bore Hole & Cone
 - ◆ Bore Hole In Site No.3
 - ◆ Bore Hole & Cone In Site No.3

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
10+860 CL	188.6	5 151 372.0	301 633.3
10+885 Lt	188.1	5 151 350.0	301 657.7
10+900 CL	188.7	5 151 331.8	301 639.8
10+923 Rt	189.6	5 151 307.6	301 623.5

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. 41K-064			
HWY No. 17 (New)			DIST 62
SUBM'D ZO	CHECKED RM	DATE Sep. 2003	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 1

METRIC

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

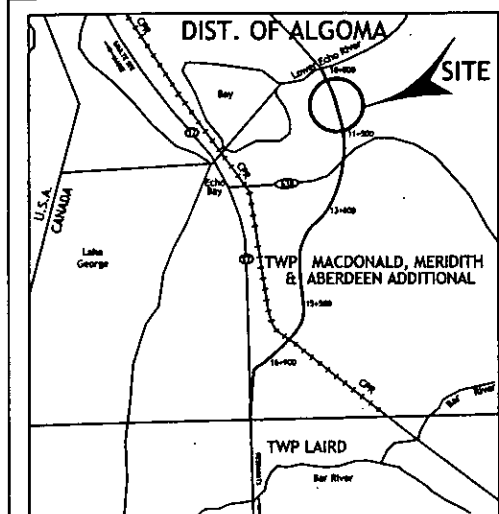
NOTE:
FOR DETAILED SUBSURFACE CONDITIONS OF ALL
BOREHOLES REFER TO RECORD OF BOREHOLE
SHEETS.

CONT No.
GWP: 354-94-00

HIGHWAY 17 (NEW)
ECHO RIVER TO BAR RIVER ROAD
CULVERT AT STATION 10+875
BORE HOLE LOCATIONS & SOIL STRATA



SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Bore Hole
- 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 Apr., 2002
- Water Level at Time of Investigation
- Piezometer

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
10+860 CL	188.6	5 151 372.0	301 633.3
10+885 Lt	188.1	5 151 350.0	301 657.7
10+900 CL	188.7	5 151 331.8	301 639.8
10+923 Rt	189.6	5 151 307.6	301 623.5

NOTE

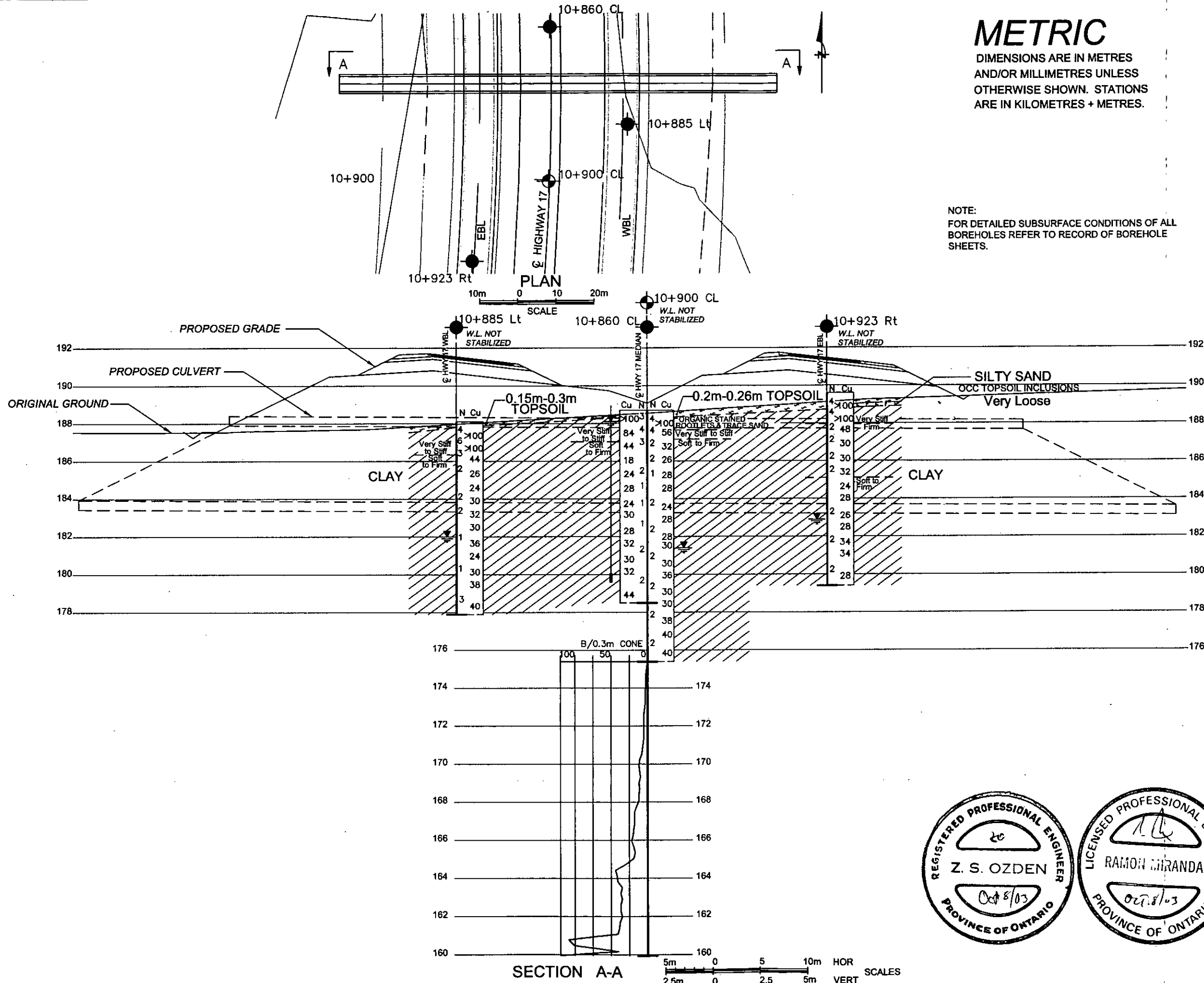
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REV.	DATE	BY	DESCRIPTION

Geocres No. 41K-064

HWY No. 17 (New)			DIST 62
SUBM'D ZO	CHECKED RM	DATE AUG, 2003	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 2



Appendix A

Record of Borehole Sheets

SPT 1055

RECORD OF BOREHOLE No 10+860 CL

1 OF 1

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 151 372.0; E 301 633.3 ORIGINATED BY G.I.
DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers COMPILED BY M.L.
DATUM Geodetic DATE 4/4/2002 CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● POCKET PENETR.	+ FIELD VANE × LAB VANE						
188.6 0.0	Ground Surface 0.15 m Topsoil		1	SS	3										
			2	SS	4										
			3	SS	3										
			4	TW	PH										
			5	SS	2										
			6	SS	1										
			7	SS	1										
			8	SS	1										
			9	SS	2										
			10	SS	2										
178.4 10.2	End of borehole														
	<p>* Water level at 6.7 m (not stabilized) and hole open to 7.6 m on completion.</p> <p>** Piezometer installed to 9.1 m. Water level on: Oct. 19, 2002 - 0.8 m (El. 188.0 m)</p> <p>Borehole advanced 0.3 m right of median centre line.</p> <p>Borehole was backfilled with clay soil to about 0.3 m below the bottom of the piezometer. Slotted section of the piezometer was backfilled with clean sand up to about 0.3 m above the slotted section. Above this depth, the borehole was grouted with cement/bentonite mixture up to the ground surface.</p>														

+ 3, x 3; Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT 1055

RECORD OF BOREHOLE No 10+885; 20 m Lt 1 OF 1

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 151 350.0; E 301 657.7 ORIGINATED BY G.I.
DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers COMPILED BY G.T.
DATUM Geodetic DATE 4/3/2002 CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)		
								20	40	60			80	100	20
188.1	Ground Surface														
0.0															
187.8	TOPSOIL		1	SS	4		188								
0.3			2	SS	6		187					16.4			
			3	SS	3		186					16.9			
			4	SS	2		185								
			5	TW	PH		184								
			6	SS	2		183								
			7	SS	2		182								
			8	SS	1		181								
			9	SS	1		180								
			10	SS	3		179								
177.9							178								
10.2	End of borehole														
	* Water level at 6.1 m (not stabilized) and hole open to 7.6 m on completion. Borehole was backfilled to about 6 m below the ground surface with clay soil brought up by augering (i.e. auger cuttings). The upper 6 m of the open borehole was then grouted using a cement/bentonite mixture.														

+³, x³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT 1055

RECORD OF BOREHOLE No 10+900 CL

1 OF 2

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 151 331.8; E 301 839.8 ORIGINATED BY G.I.
DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers & DCPT COMPILED BY M.L.
DATUM Geodetic DATE 4/3/2002 CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
188.7	Ground Surface												
0.0	0.26 m Topsoil organic stained, rootlets and trace sand		1	SS	4								
			2	SS	4								
			3	SS	2								
			4	SS	2								
			5	SS	1								
			6	TW	PH								
			7	SS	2								
			8	SS	2								
			9	SS	2								
			10	SS	2								
			11	SS	2								
			12	SS	2								
175.4	End of borehole												
13.3													
	* Water level at 7.3 m (not stabilized) and hole open to 9.1 m on completion. Borehole advanced 0.3 m right of median centre line. Borehole was backfilled to about 6 m below												

Continued Next Page

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

SPT 1055

RECORD OF BOREHOLE No 10+900 CL

2 OF 2

METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 151 331.8; E 301 639.8 ORIGINATED BY G.I.
DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers & DCPT COMPILED BY M.L.
DATUM Geodetic DATE 4/3/2002 CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
FLEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES								
	the ground surface with clay soil brought up by augering (i.e. auger cuttings). The upper 6 m of the open borehole was then grouted using a cement/bentonite mixture.						<p> SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. x LAB VANE </p>						
159.7							173						
							172						
							171						
							170						
							169						
							168						
							167						
							166						
							165						
							164						
							163						
							162						
							161						
29.0	End of Dynamic Cone Penetration Test						160						
	Dynamic Cone Penetration Test performed from 13.3 m to 29.0 m.												

SPT 1055

RECORD OF BOREHOLE No 10+923; 19 m Rt 1 OF 1 METRIC

GWP 354-94-00 LOCATION Echo River to Bar River Road, Sault Ste. Marie - Coords: N 5 151 307.6; E 301 623.5 ORIGINATED BY G.I.
DIST 62 HWY 17 (New) BOREHOLE TYPE Hollow Stem Augers COMPILED BY M.L.
DATUM Geodetic DATE 4/3/2002 CHECKED BY R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
189.6	Ground Surface							20 40 60 80 100						
0.0	0.2 m Topsoil							20 40 60 80 100						
189.2	SILTY SAND occasional topsoil inclusions grey, wet, very loose		1	SS	4									
0.4														
	very stiff		2	SS	4									
	firm		3	SS	2									
			4	SS	2									
			5	SS	2									
	CLAY		6	SS	2									
	soft to firm		7	TW	PH									
			8	SS	2									
			9	SS	2									
			10	SS	2									
179.4	End of borehole													
10.2	* Water level at 6.7 m (not stabilized) and hole open to 7.6 m on completion. Borehole was backfilled to about 6 m below the ground surface with clay soil brought up by augering (i.e. auger cuttings). The upper 6 m of the open borehole was then grouted using a cement/bentonite mixture.													

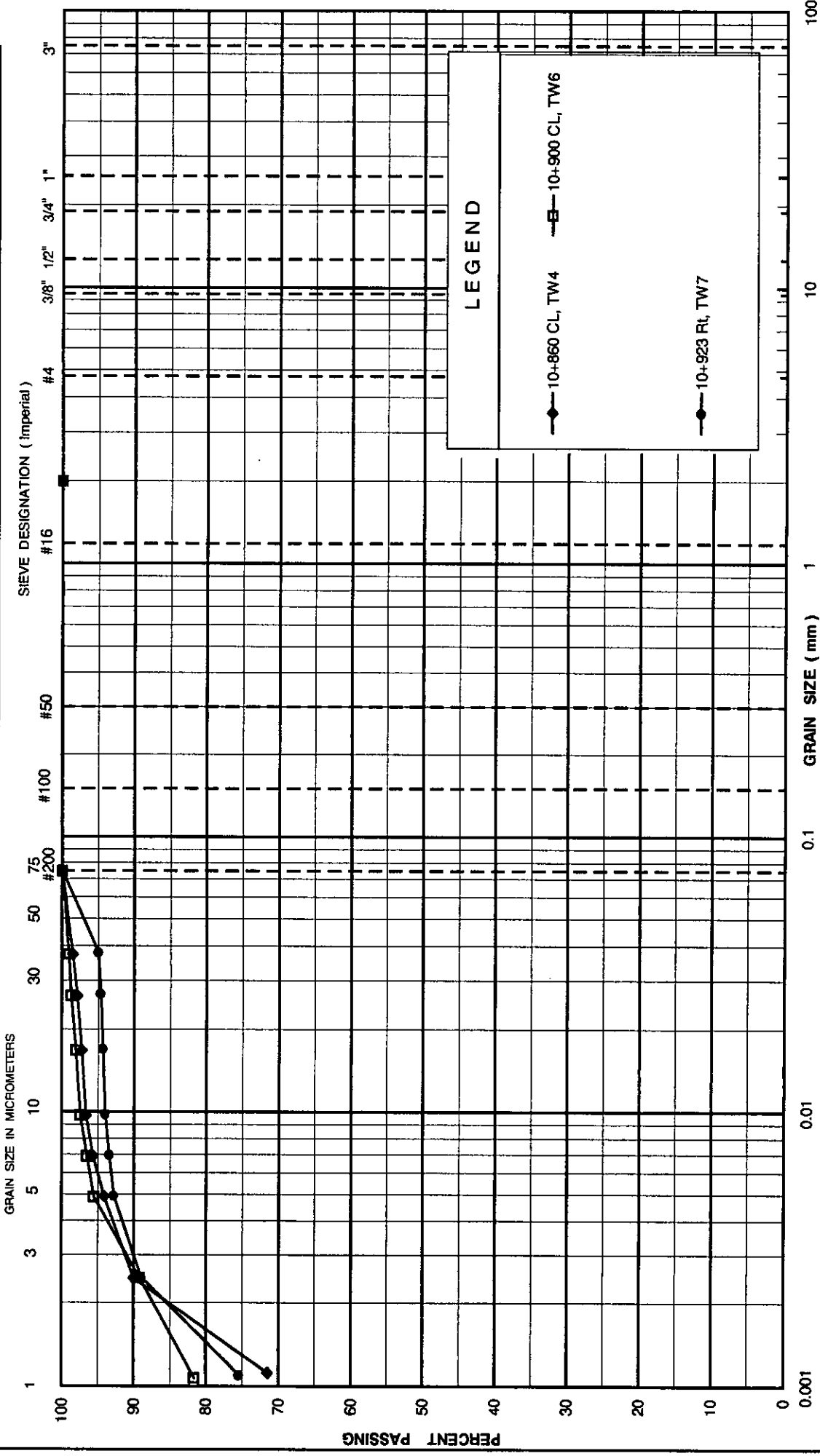
* Water level at 6.7 m (not stabilized) and hole open to 7.6 m on completion.
Borehole was backfilled to about 6 m below the ground surface with clay soil brought up by augering (i.e. auger cuttings). The upper 6 m of the open borehole was then grouted using a cement/bentonite mixture.

Appendix B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS			Fine	Medium	Coarse	Fine	Coarse	Coarse

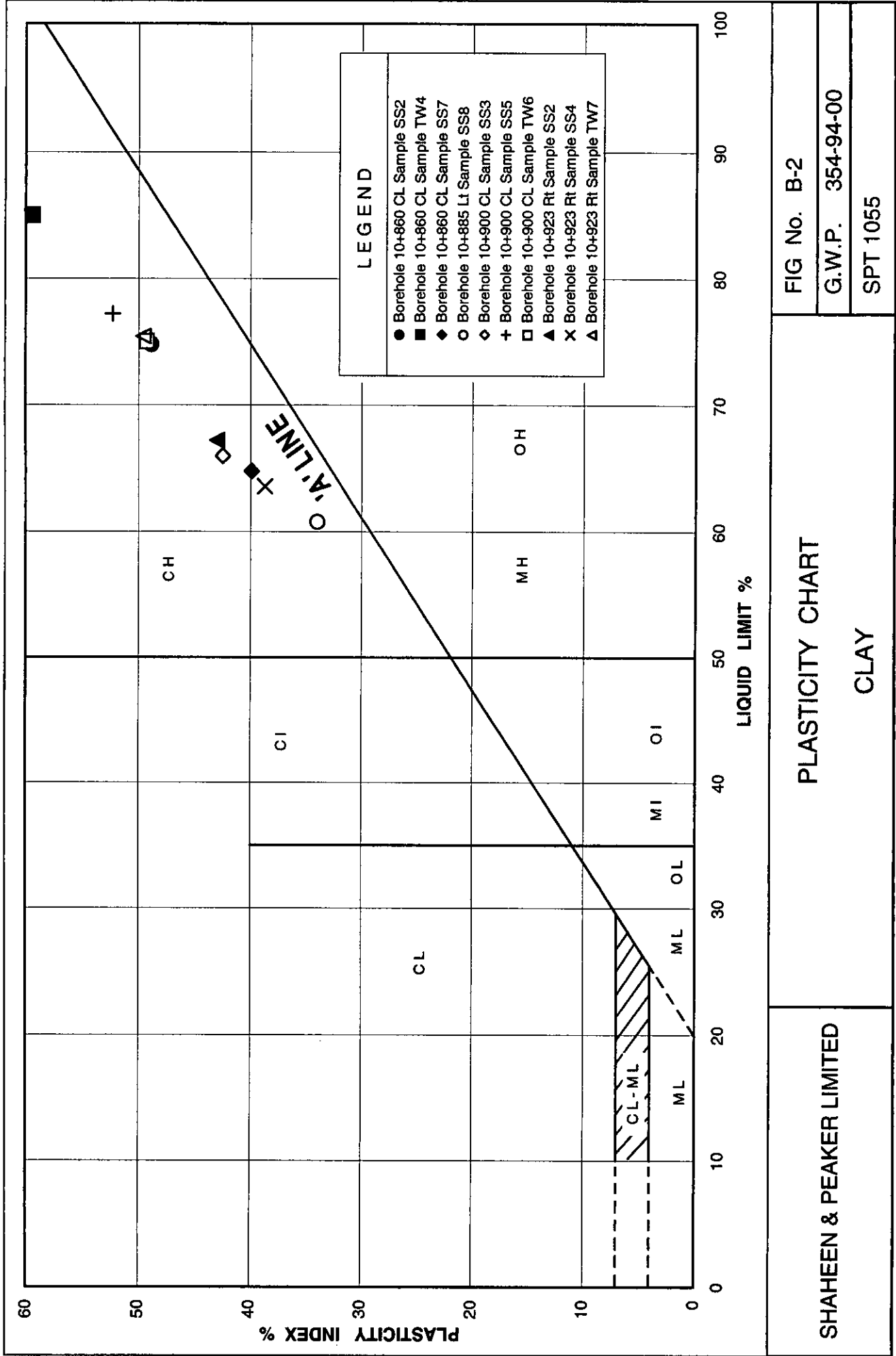


GRAIN SIZE DISTRIBUTION

CLAY

FIG. No. B-1
REF. No. SPT 1055
G.W.P. 354-94-00

SHAHEEN & PEAKER LIMITED

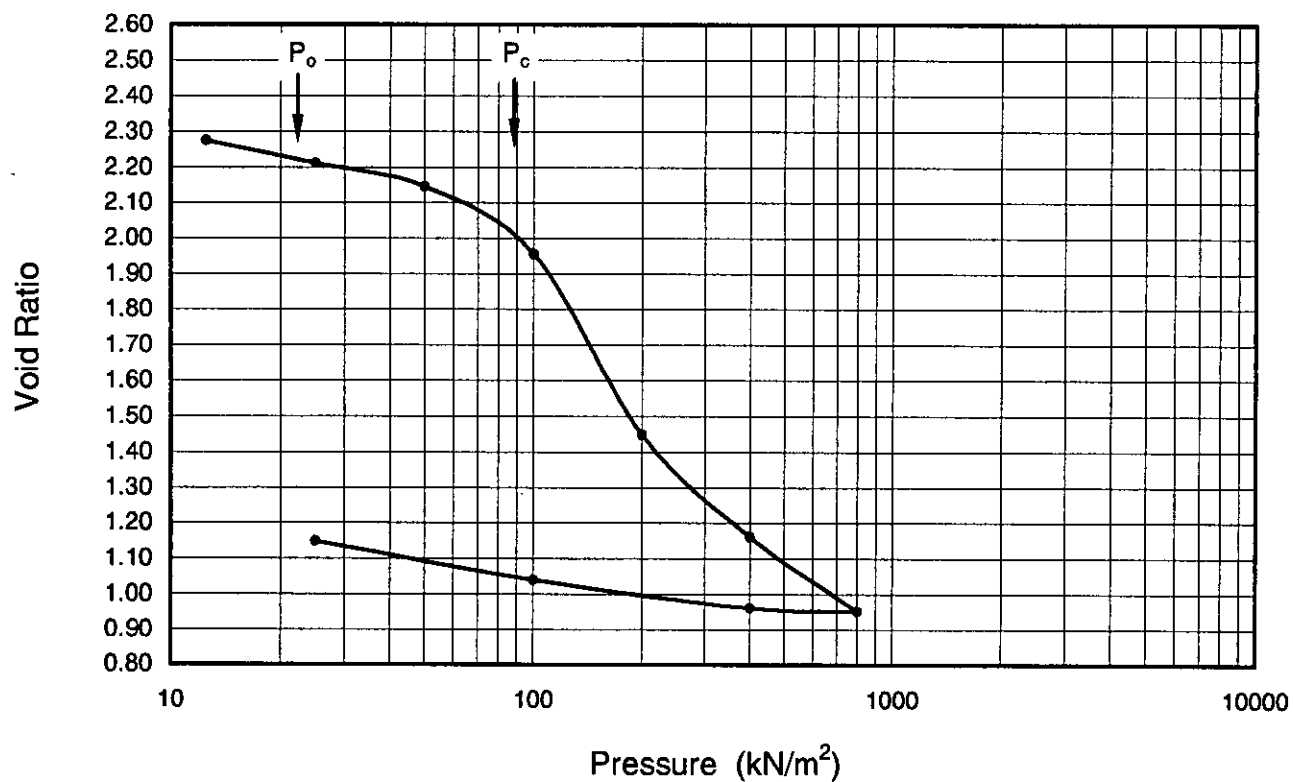


Borehole 10+860 CL

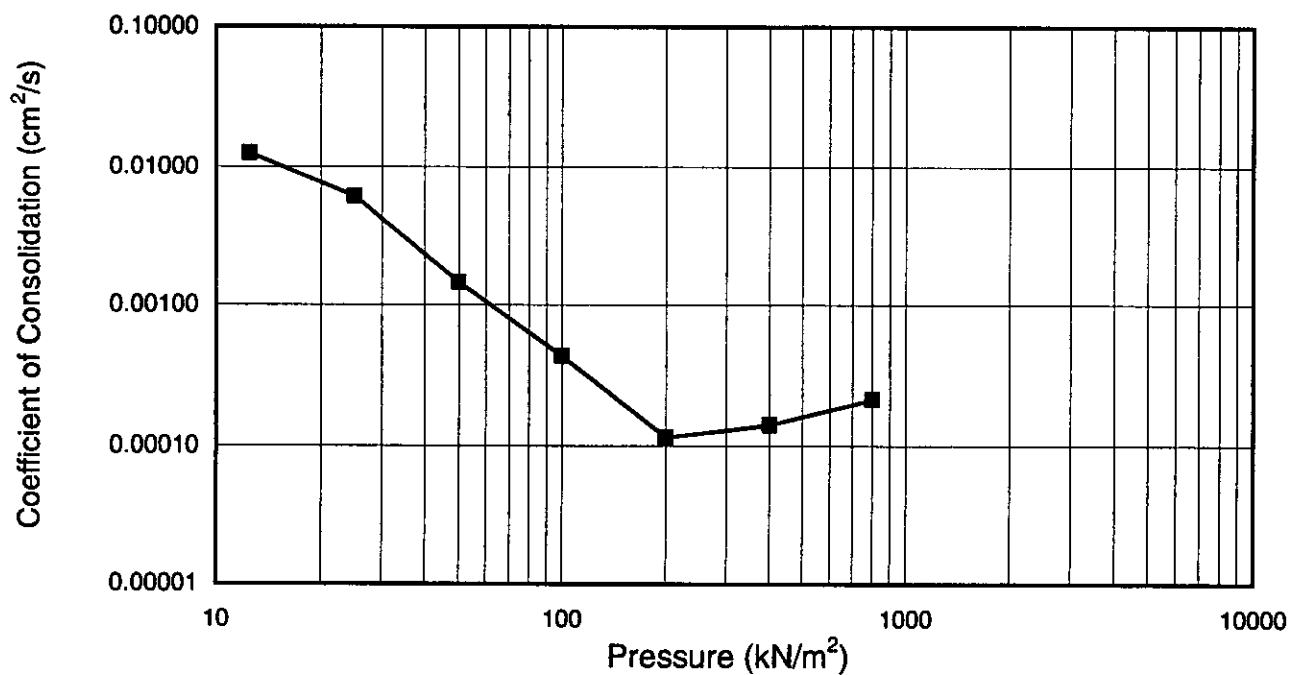
TW 4 Depth 2.50 m

Fig. B-3

Void Ratio versus Pressure



Coefficient of Consolidation vs Pressure

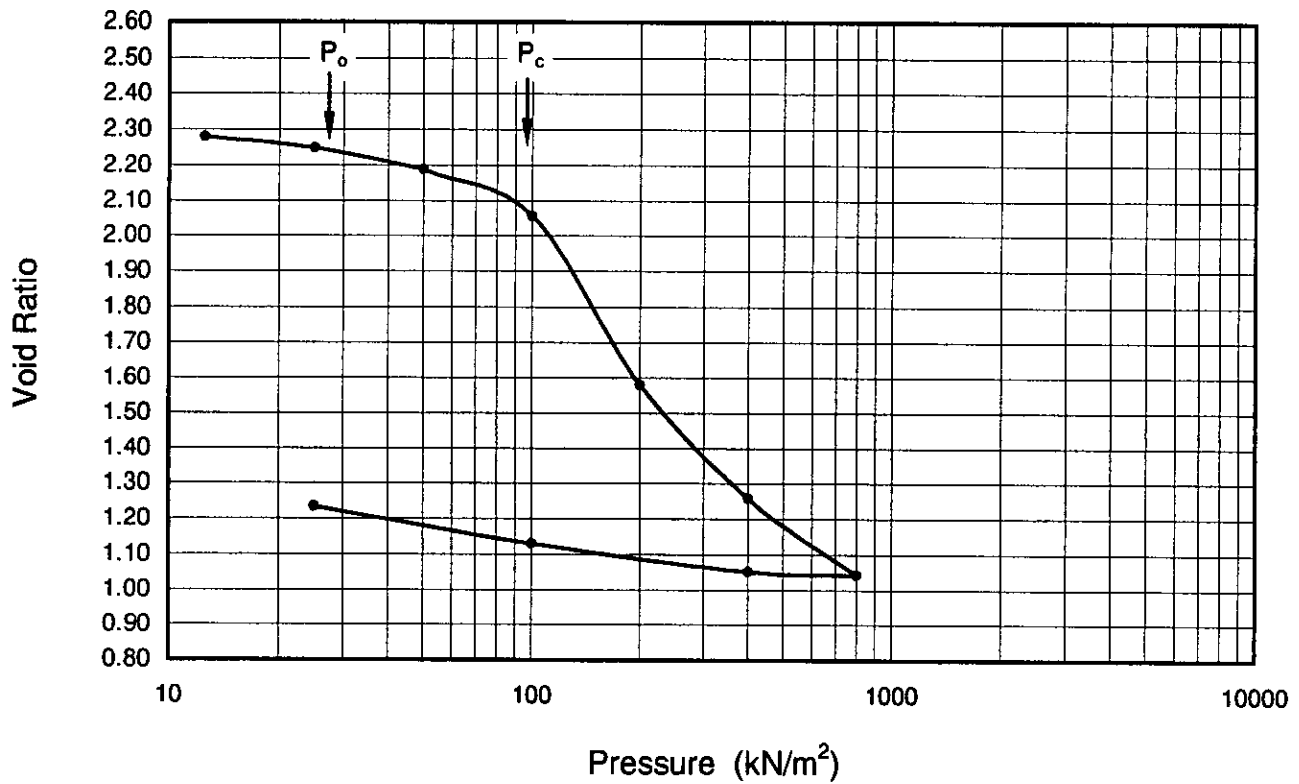


Borehole 10+900 CL

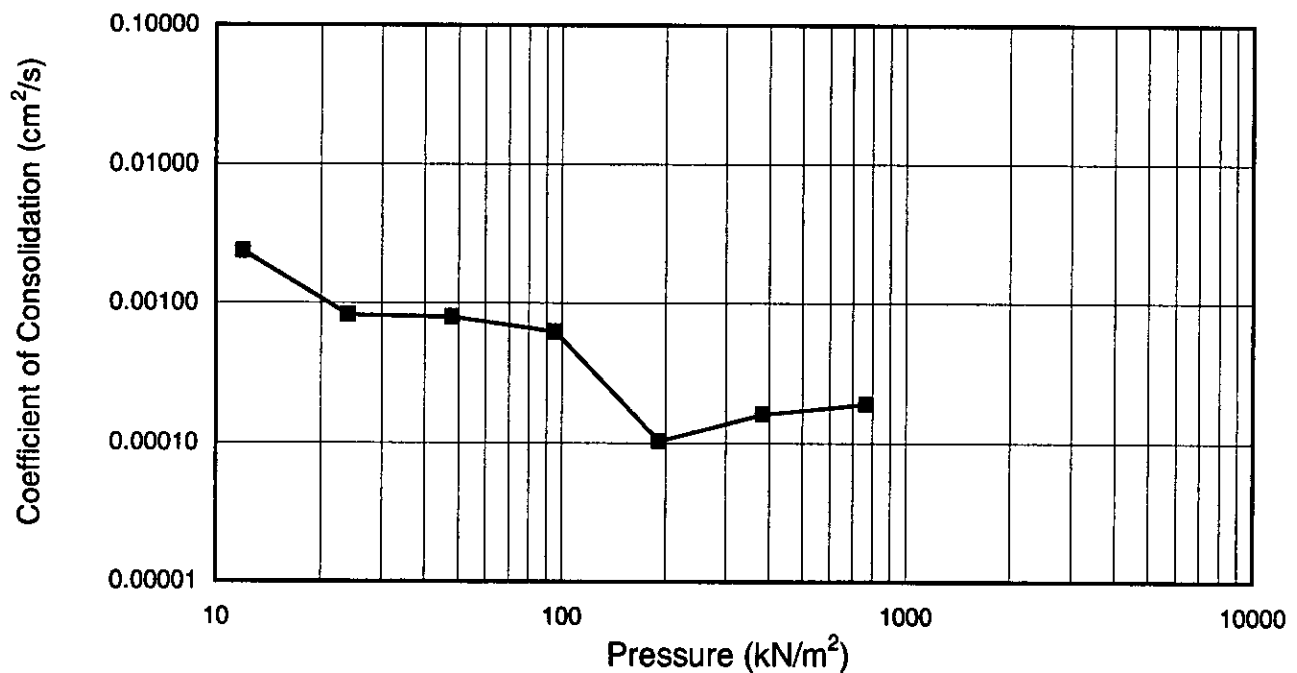
TW 6 Depth 4.00 m

Fig. B-4

Void Ratio versus Pressure



Coefficient of Consolidation vs Pressure



Appendix C

Measured Undrained Shear Strength Results

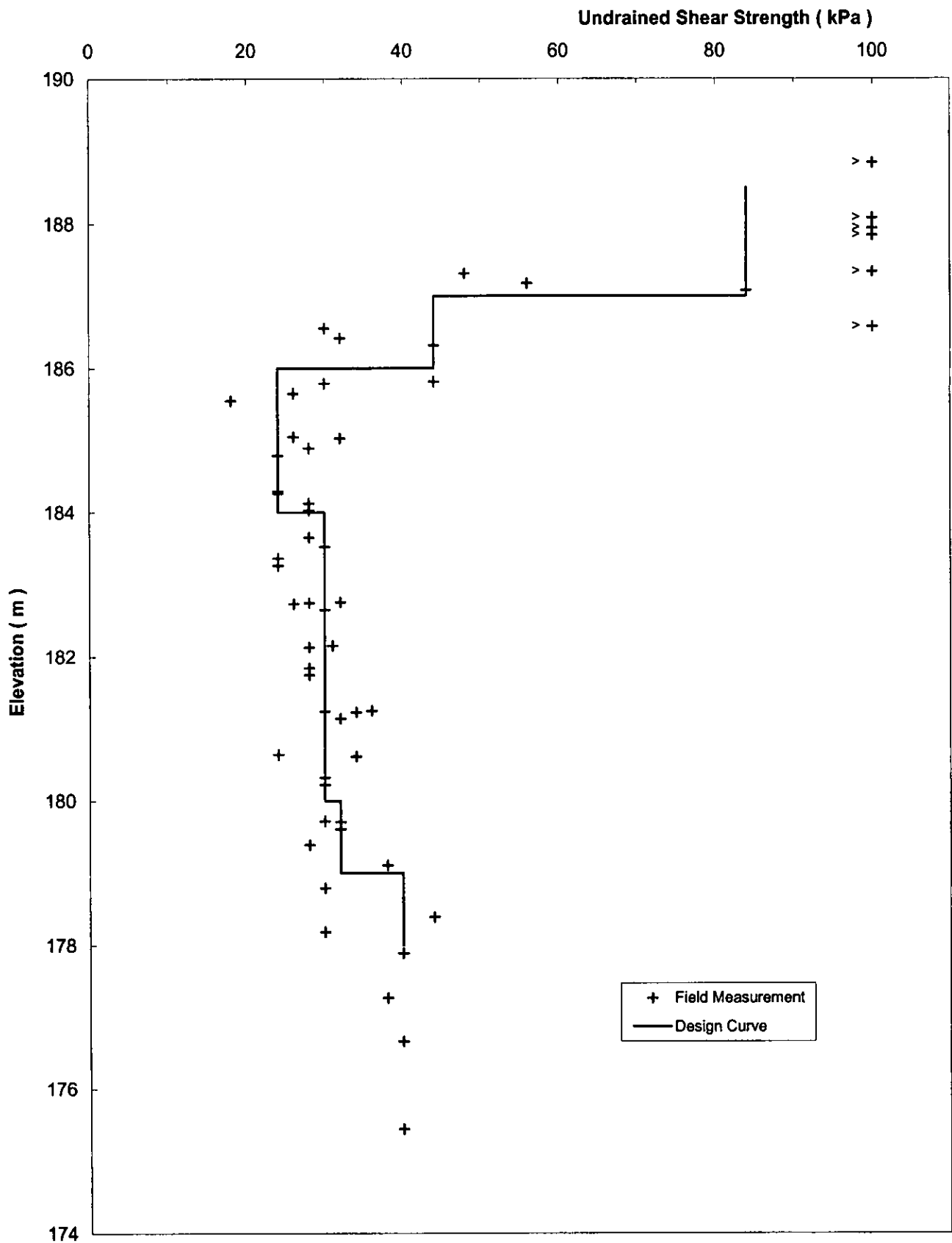


Fig. C-1: Variation of Undrained Shear Strength (as measured by field vane tests) with Elevation
(Boreholes 10+860 CL, 10+885 Lt, 10+900 CL and 10+923 Rt)

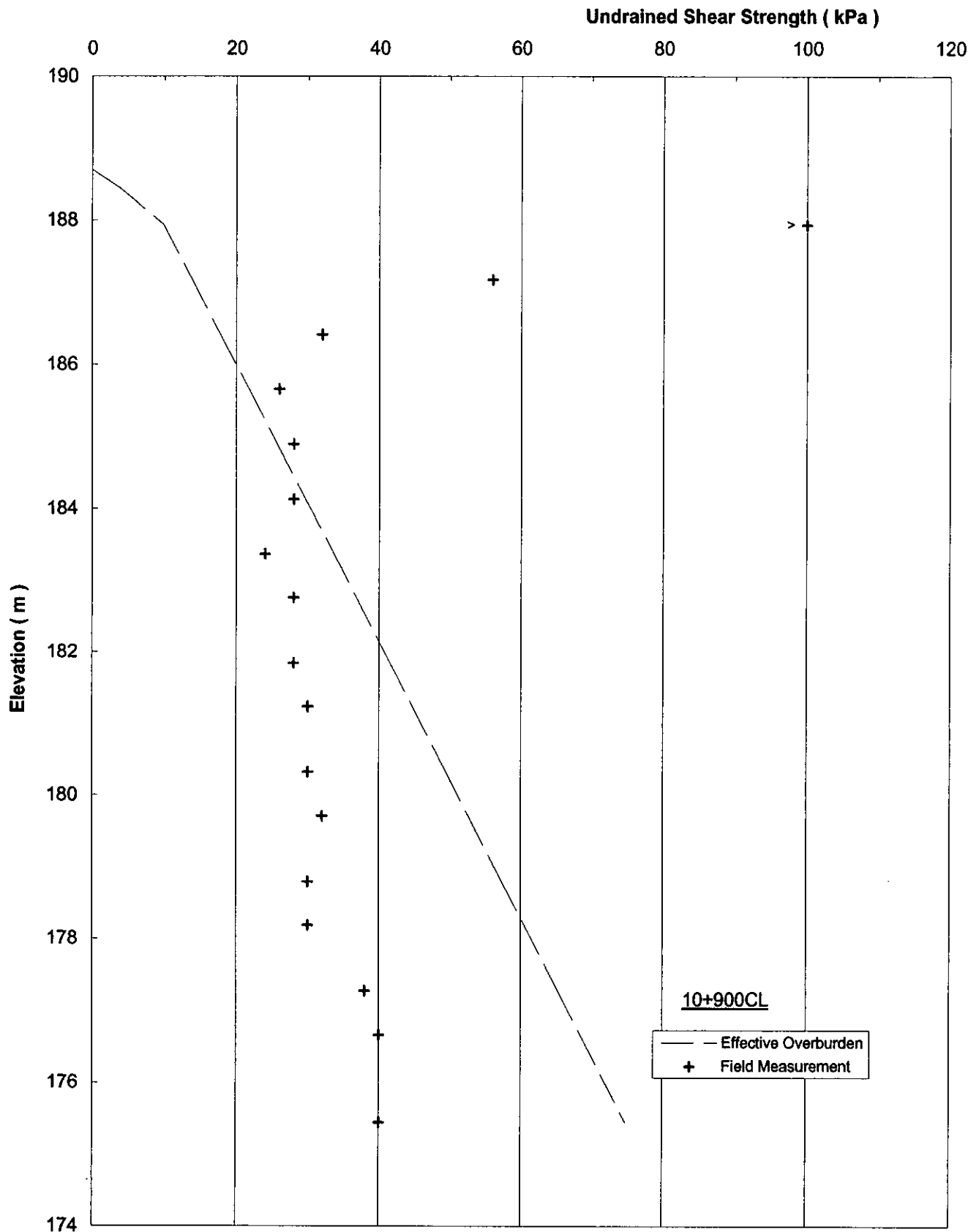


Fig. C-2: Variation of Undrained Shear Strength (as measured by field vane tests) with Elevation
(Borehole 10+900 CL)

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
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_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_r	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	l_p	%	PLASTICITY INDEX = $(w_L - w_p) / l_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	l_L	1	LIQUIDITY INDEX = $(w - w_p) / l_p$	l	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	l_C	1	CONSISTENCY INDEX = $(w_L - w) / l_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 17 (NEW) SNOWMOBILE CULVERT STRUCTURE
AT STATION 10+875 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. 41K-064

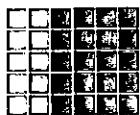
Prepared For:

MARSHALL MACKLIN MONAGHAN LTD.

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1055
October 8, 2003**



**20 Meteor Drive
Toronto, Ontario
M9W 1A4**

**Tel: (416) 213-1255
Fax: (416) 213-1260**

WEB SITE: WWW.SHAHEENPEAKER.CA

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APPENDICES

APPENDIX E: SLOPE STABILITY ANALYSIS RESULTS

APPENDIX F: LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT
PROPOSED HIGHWAY 17 (NEW) SNOWMOBILE CULVERT
AT STATION 10+875 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:**

5. DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

As part of the realignment of Highway 17 from Echo River to Bar River Road, a snowmobile crossing culvert will be constructed at Station 10+875 Median Centreline of Highway 17 (New).

Based on the drawings provided by Marshall Macklin Monaghan Limited, it is anticipated that the proposed box culvert will consist of a square concrete structure, about 110 m in length, with an opening of 4.0 m x 4.0 m, under the proposed Highway 17 (New). The proposed bottom elevation of the culvert along the eastbound lanes (EBL) and westbound lanes (WBL) centerlines are 183.2 m and 183.3 m, or about 5.9 m and 4.8 m, respectively, below existing ground surface. We were asked to provide a report giving recommendations for the geotechnical foundation design of the proposed culvert based on available information at the site.

As shown in Drawing No. 2, four previously drilled boreholes (Boreholes 10+860 CL, 10+885 Lt, 10+900 CL and 10+923 Rt) provide subsurface information along and in the vicinity of the proposed culvert alignment which this report is based on. In general, below the topsoil and very loose silty sand layer in Borehole 10+923 Rt, extending to a depth of 0.4 m, the site is underlain by an extensive clay deposit, which has a very stiff to stiff crust to a depth of about 1.5 m, becoming firm to soft (generally firm) below this upper zone. The clay extends to at least 13 m below existing grade. DCPT performed below the bottom of Borehole 10+900 CL leads us to believe that this soft to firm clay deposit probably extends without interruption to a depth of about 24 m (Elevation 165 ± m). Below this elevation, the presence of a somewhat 'stiffer' layer was inferred from the DCPT and a more competent stratum at a depth of about 28 m (Elevation 161 m). DCPT refusal was encountered at a depth of about 29 m. The groundwater level along the culvert alignment is believed to be close to the existing ground surface, but may fluctuate throughout the year.

In general, the existing ground surface at the proposed culvert location slopes very mildly down towards the northeast from about Elevation 189.8 m to about Elevation 187.6 m. The anticipated heights of the proposed highway embankment above the original grades are approximately 2.3 m

and 3.5 m along the EBL and WBL centrelines, respectively. From our knowledge of the overall project, it is anticipated that at this location, the Highway 17 embankment will be constructed and surcharged by 1.7 m of additional fill for about two years, prior to paving of the roadway.

The prevailing soft to firm clay below the anticipated structure bottom elevation of about 183.2 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/surcharging.

Normally, with weak clay subgrade conditions, flexible steel pipe culvert on a granular engineered fill is preferred because these are able to withstand greater than normal differential settlements. However, in this case because of the fact that opportunity to surcharge the site is available, closed bottom (box) concrete culverts (prefabricated sections), supported on engineered fill, also provide adequate solution.

Alternatively, the culvert can be supported on deep foundations but the lack of a well defined bearing stratum with the available subsurface information, together with anticipated higher foundation costs at the site render the use of deep foundations rather uneconomical.

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

5.2 FOUNDATION OPTIONS

The boreholes show the presence of compressible soft to firm clay deposit, probably extending to in excess of 20 m. The consistency of the deposits is relatively consistent within the four boreholes drilled in the vicinity of the culvert. Due to the depth of the compressible clay deposit, surcharging is needed to effect majority of the settlements prior to the construction of the culvert. This is true for both engineered fill and deep foundation alternatives.

Among the deep foundation alternatives, drilled caissons are not suitable due to a lack of a suitable end bearing stratum. Driven steel piles can be considered but additional boreholes would be needed to verify the presence of a suitable end bearing stratum below about 29 m. Friction piles can be considered but in this case timber piles will likely be more economical than steel piles. They can be driven with lighter, less expensive and less sophisticated equipment.

Another important issue for the design of the culvert is frost protection.

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 low shear strength and highly compressible clays	Not recommended based on reliability
Normal Spread Footings on Compacted Granular pad	Not feasible due to low shear strength and highly compressible clays	Not recommended based on reliability
Flexible Steel Pipe Culvert on Compacted Granular pad 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 pad plus a sufficient surcharging period prior to construction	Considered feasible	Represents the best alternative along with 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 with the presently available information	Not recommended based on reliability and economics
Steel Tube Piles	Not a good choice due to a lack of suitable bearing stratum with the presently available information	Not recommended based on economics and reliability but considered a better alternative, if necessary, than H-Piles
Timber Piles	Represents a better alternative in comparison with other driven pile types. Will require Granular 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, pre-cast 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 clay 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 new culvert under the proposed highway is expected to be founded at Elevations 183.3 m and 183.2 m under the WBL and EBL centrelines, respectively. From the findings of boreholes, these bottom elevations of the culverts are expected to be in the clay deposit, which is weak and compressible.

Since the interior of the proposed snowmobile crossing will be kept dry and therefore will be exposed to freezing temperatures, the underlying clay deposit and any water within granular backfill beneath and adjacent to the culvert, within the frost depth, should be kept from freezing. This can be achieved by the provision of artificial insulation by the use of rigid artificial insulation as well as by removing the water within the granular backfill with the installation of filtered subdrains on both sides of the culvert at sufficient depth below the culvert invert and also within the backfill.

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, and to provide frost protection, we recommend the following procedures.

- After the preparation of the embankment, the site should be surcharged with 2.2 m of additional fill (i.e. over and above the finished highway grade) for a period of not less than 1½ years. The excavation should be carried out in conjunction with the stripping and/or sub-excavation requirements for the Highway 17 embankment. In addition, to effect the settlements at the outside ends of the culvert, we recommend the placement of surcharge at a slope of 3.5H:1V on the side slopes of the embankment, extending 4 m from the toe of the embankment at original grade level (see Figure E-1 in Appendix E). The surcharge materials can consist of clean inorganic earth fills (preferably granular fill materials) which should also be lightly compacted.
- After a surcharging period of not less than 1.5 years, the surcharge and the embankment fills would be removed at the proposed culvert location to the original ground level (i.e., to El. 188 to 189 m). The removal of the surcharge and the embankment fill would be carried out to at least 12 m beyond the perimeter of the culvert foundation excavation (Figure E-2, Appendix E). The side slopes for excavations in the existing embankment and surcharge should be cut at no steeper than 4H:1V. As also shown in Figure E-2, the surcharge portion of the embankment fill would be removed to about 8 m beyond the top of the embankment.
- If necessary, after stripping, dewater the site near the surface and also in the underlying silty sand in Borehole 10+923 Rt. Some of the possible methods to accomplish this

include gravity drainage and pumping from perimeter trenches extending to about 0.3 m into the clay deposit, including pumping from strategically placed, filtered sumps; or a combination of these methods, etc.

- The site would then be sub-excavated within an area 2 m beyond the perimeter of the proposed culvert footprint, to a depth of 0.8 m below the founding level of the culvert (i.e. generally to a depth of about 5.4 to 6.5 m below original grade or to about El. 182.4 m). The side slopes for the excavations in the clay should be cut at no steeper than 2H:1V. But the bottom 0.8 m (i.e. below invert level) can be cut at near vertical slopes, if desired. Heavy construction equipment should not be permitted within 3 m of the crest of the slope. The lower portion of the excavation should be carried out in short sections, not exceeding 10 m in width. The bottom of the excavation should be immediately backfilled to about 0.2 m below the proposed founding level (i.e., El. 183.0 \pm m) with Granular 'B' Type II or Granular 'A' (with the maximum percentage of fines passing the No. 200 sieve (75 μ m) limited to 5%) material, before proceeding with the excavation and backfilling of the next section. The surface of the clay subgrade should be adequately sloped towards a ditch or outlet to facilitate subsurface drainage. Some light compaction can be applied on the granular material by a suitably light compactor. This must, however, be done with extreme caution in order not to disturb the underlying weak clay subgrade. The operation of heavy equipment should be allowed neither at the bottom of the excavation nor on the top of granular backfill.
- The next section can be constructed in the same manner while effecting, if necessary, additional dewatering from filtered sumps within the granular fill. After the area is backfilled with granular material to an elevation of about 0.2 m below the bottom of the culvert, filtered subdrains should be installed on both sides of the culvert as shown in Drawing No. 3, Alternative 1. If this is considered impractical, Alternative 2 in Drawing No. 4 can also be used, but the preferred scheme is Alternative 1. The subdrains should be positioned as low as possible (minimum of 0.6 m below the bottom of the culvert) to ensure that water is drained away from the structure. Also as shown in Drawing Nos. 3 and 4, a second set of drains should also be installed at about the invert level to ensure drainage above the rigid insulation. The drains should be discharged to an appropriate outlet in a frost free manner. To facilitate drainage and to minimize depth of excavation, consideration should be given to raising the culvert as high as possible. We also recommend an adequate fall in grade (e.g., 1%) to ensure good drainage and to prevent collection of water at the bottom of the granular backfill. The excavation should be executed in such a manner that the bottom of the excavation (i.e., top of clay subgrade) is smooth to effect drainage. Depressions or undulations which could promote water ponding or prevent rapid drainage should not be allowed. Water accumulation will lead to subgrade softening and frost penetration.

- At least two overlapping layers (100 mm or 4-inches in total thickness) of rigid insulation (i.e., STYROFOAM HI – 100 or approved equivalent) should be placed over the granular material (i.e., extend at least 2.0 m beyond the perimeter of the culvert) to provide frost protection over the clay subgrade*. The insulation boards should then be topped with polyethylene sheeting (6 mil) and about 100 mm of fine to medium sand bedding. No heavy construction equipment or vehicle should be operated directly on top of the insulation and sand bedding. Insulation boards could also be placed (optional) on vertical surfaces around the culvert exterior to help prevent adfreeze. This could consist of a single layer of 38 mm (1 ½ ") thick STYROFOAM SM insulation material, or approved equivalent. This can be omitted if considered too costly especially if intermediate drains are placed within the granular backfill, as discussed below.
- The granular backfill around the culvert (i.e., above the horizontal insulation boards and 100 mm sand topping) should consist of free draining granular materials conforming to OPSS Form 1010 for either Granular 'A' or 'B' Type I. To maintain free draining characteristics in these granular fill materials, the maximum percentage passing the No. 200 sieve (75 µm) should be limited to 5%. Depending on the design, the placement of additional drains (i.e., in addition to those shown in Drawing Nos. 3 and 4) at intermediate depth(s) along the vertical face of the culvert structure would be desirable.
- The prefabricated culvert sections should be placed as expeditiously as possible and backfilled with granular materials. All construction should be carried out without delay. Our stability calculations assume that the unprotected slopes of the excavation will be substantially backfilled within about three to four weeks of the excavation. During this period the side slopes of the temporary excavation should be monitored daily (by means of visual inspection) for any signs of instability. They should also be protected against erosion (e.g., by placing tarpaulin where necessary, etc.)
- The construction should be performed under the supervision of the Quality Verification Engineer (QVE) who should also inspect and approve all bearing surfaces, inspect and verify stability of temporary slopes, adherence to equipment movement, compaction equipment and methods, etc.

* Reference: E. I. Robinsky and K. E. Besflug: "Design of Insulated Foundations", Journal of the Soil Mechanics and Foundations Division, A.S.C.E. Volume 99, SM9, September 1973.

** Place all insulation in at least two overlapping layers (i.e., vertical joints must be staggered). Insulation must be glued to all vertical surfaces, either pegged together or spot glued together on near horizontal surfaces. Protect insulation from ultraviolet light and any other harmful agents. The subgrade must be frost free when construction starts and frost must not be allowed to penetrate subgrade during construction.

A Factored Geotechnical Resistance at U.L.S. of 200 kPa and a Geotechnical Resistance at S.L.S. equal to 120 kPa can be assigned to the founding granular subgrade (at El. 183.2 m) prepared and surcharged in this manner.

Provided that the clay subgrade is undisturbed during construction, the maximum total settlement under the proposed embankment at the culvert location is estimated to be about 600 mm. From this, approximately 500 mm is expected to take place as consolidation settlement in the clay deposit, and the other 100 mm as elastic settlement (it is anticipated that this will occur immediately, after the placement of embankment fill). The surcharge programme with 2.2 m of additional fill over the embankment height, as mentioned above, is designed to accelerate the settlement and to minimize the post-construction settlements. After a surcharging period of 1.5 years, the post-construction settlement is expected to be about 30 mm. This is expected to take place within the next 4 to 6 years of the completion of construction of the roadway. Subsequent settlements are likely to be less than 10 mm. These settlements can translate into differential settlements between the centres and the EBL and WBL, and therefore we recommend that construction joints be introduced for about every 6 to 8 m section 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. However, as mentioned before, adequate slope should be provided to ensure adequate drainage and to prevent collection of water. For this reason and for ease of construction and slope stability, we recommend that the invert level of the culvert be raised as much as possible.

If a steel pipe culvert is to be considered, the site would be prepared as discussed above for a concrete box culvert, including surcharging. Vertical insulation boards would not be needed but the compaction of the granular fills around the pipe should be carried out with utmost care since the structural integrity of the pipe depends on the side support provided (i.e. lateral yield should be prevented).

5.2.2 TIMBER PILES

Based on the findings of the boreholes, 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 both from adhesion and tip.

An 18 m long timber pile 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 270 kN and an Axial Resistance at S.L.S. of about 180 kN per pile.

To confirm the resistance values presented above, it is recommended that pile load testing be carried out. We recommend that a period of about 30 days be allowed between the driving of the test piles and test loading of the pile. Depending on the outcome of the test(s), possibly

combined with drilling borehole(s), the recommended resistances may need to be adjusted (possibly upwards).

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 site should be prepared and surcharged in the manner described in the previous section of this report. The pile tips should be protected from damage while driving through the granular fill. This may involve locally sub-excavating the granular backfill in order to reduce its thickness at each pile location. The piles should be driven after a minimum surcharging period of 1.5 years.

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

5.3 DESIGN FEATURES

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

Design frost penetration for the general area is 1.8 m. For frost protection, the foundations 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 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. It should be pointed out that where the granular pad is overlain by insulation pads and especially the polyethylene sheeting, the friction will be negligible (i.e., will not provide sliding resistance).

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. As was mentioned before, the maximum percentage of fines passing the No.200 sieve (75µm) should be limited to 5% in order to provide free drainage of the backfill. 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 about 3 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 (i.e. wingwalls), 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'

Unit Weight = 22 kN/m³

Coefficient of Lateral Earth Pressures:

Ka = 0.27

Ko = 0.43

Compacted Granular 'B'

Unit Weight = 21 kN/m³

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. Since the culvert wall is restrained and does not allow lateral yielding, the 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. Where wingwalls are to be used, a minimum surcharge height of 1.8 m should be maintained for at least 1.5 years. For foundations not more than 1.5 m wide, a Factored Bearing Resistance at U.L.S. of 110 kPa can be utilized and a Bearing Resistance at S.L.S. equal to 70 kPa can be assumed when supported on granular pad constructed and surcharged as described before, provided that the clay subgrade is undisturbed during construction. 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. The maximum total settlements can be expected to be of the order of 30 mm within the first three years of construction, with a further 15 mm thereafter. Consideration can also be given to a flexible structure such as gabion walls, in view of the prevailing poor soil conditions. This is because gabion walls are flexible and can withstand relatively large deformations without exhibiting signs of excessive damage. If however they are undesirable for this project, for reasons other than the above, they should not be used. Foundations for the gabion type walls will need to be prepared and surcharged in the manner detailed earlier. The foundation configuration and design values should be reviewed when details are available.

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.

5.6 EMBANKMENT STABILITY

As discussed in Section 5.1 of this report, the proposed highway grade at the culvert area will be approximately 2.3 to 3.5 m (to Elevation 191.5 m) above the existing grade. The embankment and surcharge material in this section will consist of earthfill.

Slope stability analysis was conducted on the 3.5 m high embankment with 2.2 m high surcharge, 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 24 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.

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	21.0	30	0	21.0
Embankment Fill (Granular)	32	0	21.5	32	0	21.5
Granular 'A'	35	0	21.5	35	0	21.5
Clay	0	24-84	15.0-16.0	22-24	2-4	15.0-16.0

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.

We also conducted stability analyses for the subsequent excavation (i.e. after surcharging) for the construction of the culvert. We assumed that the construction would be done relatively rapidly (i.e. the excavation would be substantially backfilled within a period of several weeks). For short-term (undrained analysis), a factor of safety of 1.3 was considered sufficient. Typical results are given in Figure E-2. We also carried out a drained (effective stress) analysis, assuming the phreatic surface as shown in Figure E-3. In this case, a safety factor of 1.23 was considered adequate, subject to monitoring during the short period the excavation would be left open (i.e. before it is substantially backfilled).

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 some silty sand (encountered in one borehole only) into stiff to very stiff clay to 1.5 ±m depth changing to soft to firm but generally firm clay. The groundwater table is expected to be about 1 m below the ground surface but a perched water condition could also occur due to accumulation of surface water on the practically impervious clay. Provided that the groundwater is properly controlled, open cut excavations can be expected to stand temporarily at 2H:1V side slopes as was discussed earlier. However, limitations are imposed due to the presence of the road embankment fill and surcharge. As was discussed before, the existing embankment will need to be removed to a distance of 12 m beyond the perimeter of the culvert excavation in order not to induce a foundation failure and side slope failure in the clay. As was discussed before, operating heavy equipment should be avoided or minimized in order not to disturb the clay subgrade underlying the granular fill prepared to support the culvert.

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

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 protect the founding granular soil from protruding into the weak clay. 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 and erosion.

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.

SHAHEEN & PEAKER LIMITED


R. Miranda, P.Eng.



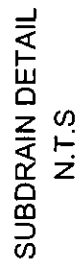

Z. S. Ozden, P.Eng.



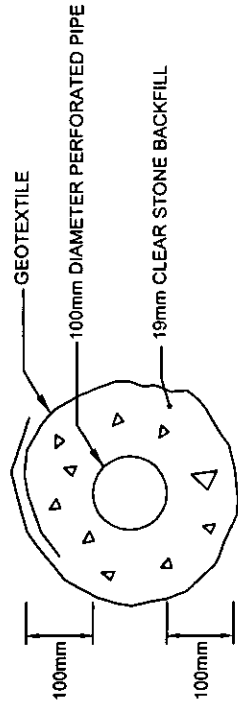
Project: SPT1055
Marshall Macklin Monaghan Ltd.

Foundation Design Report
Proposed Highway 17 (New) Snowmobile Culvert Structure
At Station 10+875 Median Centreline
Highway 17(New) from Echo River to Bar River Road
District 62, Sault Ste. Marie, Ontario

Drawings

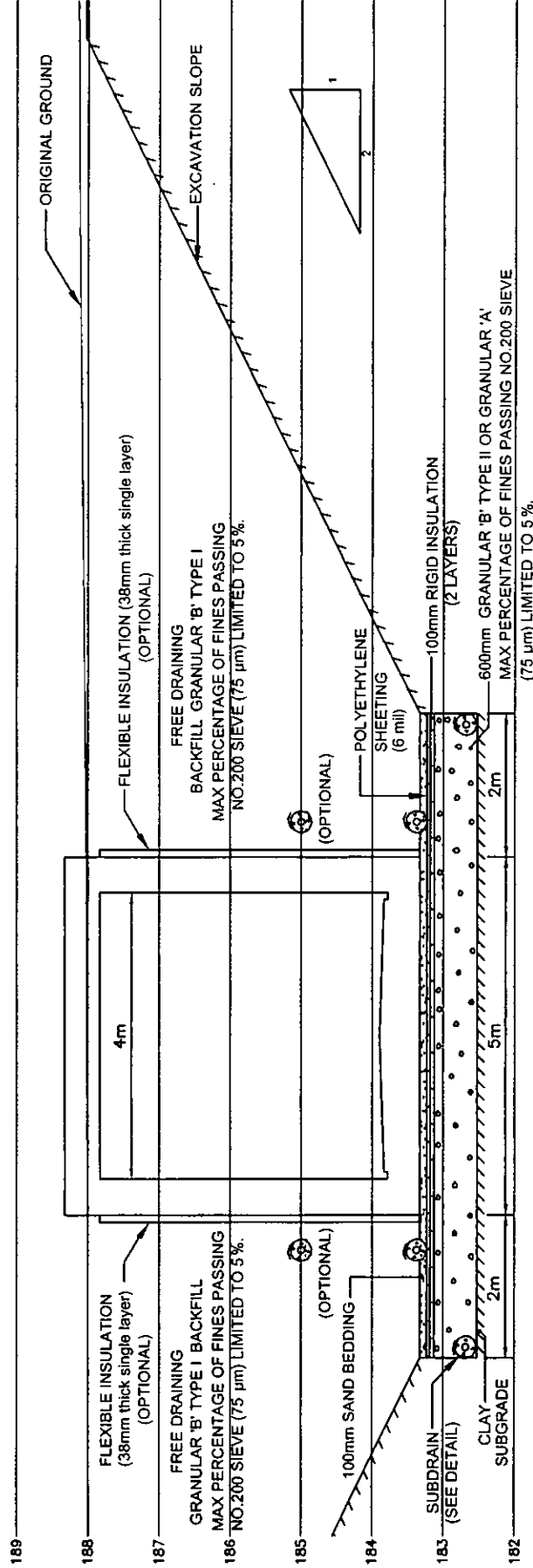


DWG NO.3



SUBDRAIN DETAIL
N.T.S

EMBANKMENT FILL



TYPICAL SECTION OF CULVERT CONSTRUCTION

@ STA 10+875 HWY 17 WBL

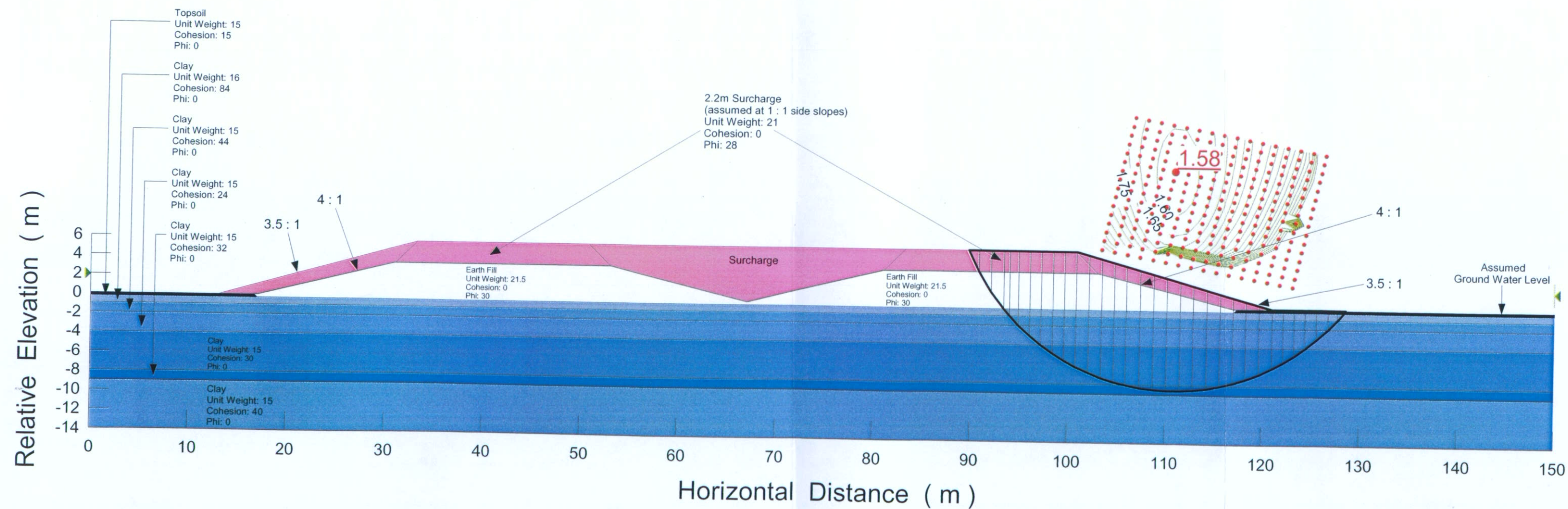
ALTERNATIVE 2

Appendix E

Slope Stability Analysis Results

SPT 1055, Highway 17 (New), Sault Ste.Marie
Proposed Culvert Construction, Station 10+875
3.5m High, Earth Fill Embankment (Plus 2.2m Surcharge)
Undrained Case (Total Stress Analysis)

Figure E-1



SPT 1055, Highway 17 (New), Sault Ste.Marie
Proposed Culvert Construction, Station 10+875
3.5m High, Earth Fill Embankment (Plus 1.7 m Surcharge)
Undrained Case (Total Stress Analysis)

Figure E-2

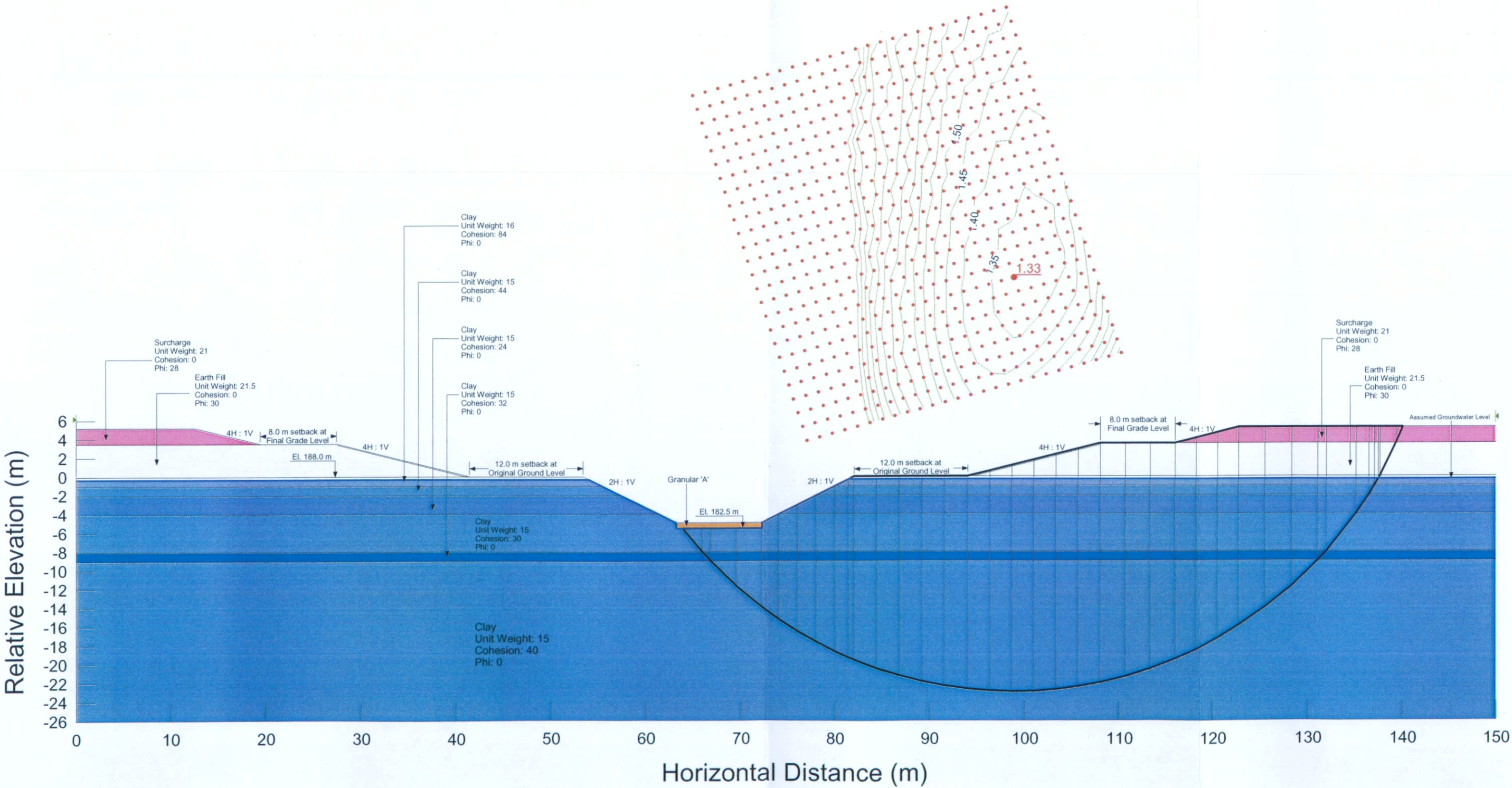
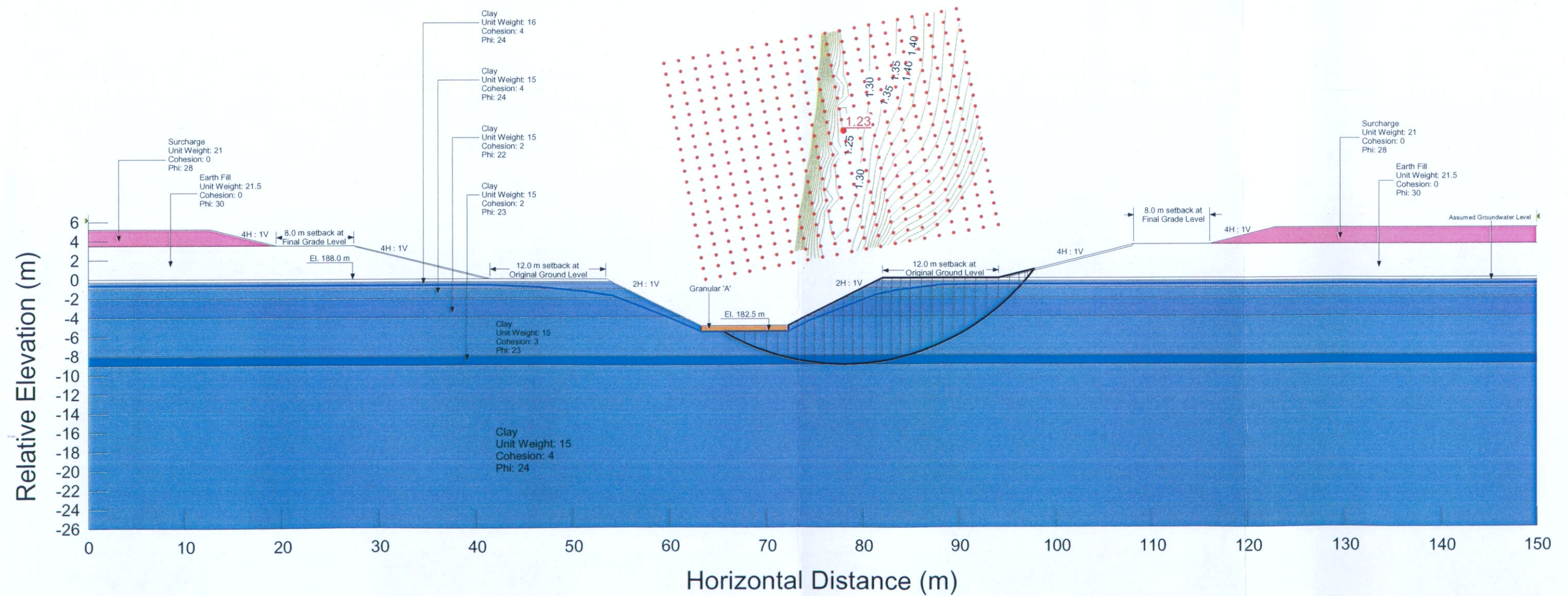


Figure E-3



Appendix F

Limitations of Report

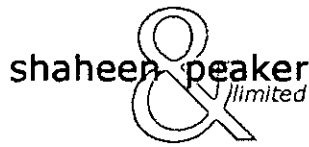
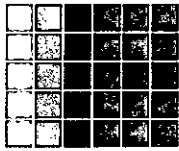
LIMITATIONS OF REPORT

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.



shaheen & peaker *limited*
consulting engineers

20 Meteor Drive
Toronto, Ontario, M9W 1A4
T: 416.213.1255
F: 416.213.1260
INFO@SHAHEENPEAKER.CA

Our Reference No. SPT 1055

October 10, 2003

Ministry of Transportation, Ontario
Pavements and Foundations Section
Room 232, Building C
Downsview, Ontario

Attention: Mr. Tae Kim, P. Eng.
Senior Foundation Engineer

Re: Draft Foundation Investigation and Design Reports
Proposed Highway 17 Snowmobile Culvert Structure at Station 10+875
GWP 352-94-00, Sault Ste. Marie

Dear Sirs:

Further to your memorandum dated September 29, 2003, regarding your review of the above captioned report, the following is our response.

1. The Geocres Number you have provided will be shown on the cover page of the final Foundation Investigation and Design Reports and on the Drawings.
2. Drawings 1 and 2 will be stamped and sealed by two professional engineers.
3. Borehole abandonment procedure is described in Section 3 and the borehole logs.
4. Detailed procedures for piezometer installation and their backfill is described in the text of the report and the borehole logs.
5. Our analyses show that the embankment plus 2.2 m surcharge can be built at once and stage construction is not required.
6. Our analyses indicate that 12 m set back is required. Figures E-2 and E-3 of our Foundation Design Report depict this scheme.
7. Insulation boards along the vertical exterior surface of the culvert is optional and can be omitted if intermediate drains are placed.
8. "Monitoring" was meant to be by visual inspection only and no instrumentation is required.
9. Pile tips were initially recommended in case of hard driving through the granular backfill. However, this can be achieved by subexcavation or loosening of the granular backfill prior to driving of the timber piles and reinforcing of the pile tips is not required. This statement will be removed from the report.

10. Redriving was recommended only in case when uplift is noticed due to the driving of the adjacent piles as this is routine and good engineering practice. However, we removed this from our report as it seems to be objectionable to MTO.
11. Gabion walls were recommended only because they are flexible and can withstand relatively large deformations without without excessive damage. If however they are undesirable for this project, for reasons other than the above, they should not be used.
12. We feel that factor of safety of 1.4 is appropriate for the site and this is consistent with our previous reports at the site which were reviewed and approved by MTO.

We thank you for your prompt and fair review of our draft report.

Yours very truly,

Shaheen & Peaker Limited

A handwritten signature in black ink, appearing to read 'Z. S. Ozden', with a stylized flourish at the end.

Z. S. Ozden, P. Eng.