DOCUMENT MICROFILMING IDENTIFICATION

G.1-30 SEPT. 1975

GEOCRES No. 40∅1-9

DIST. 1 REGION southwestern

W.P. No.

CONT. No.

W. O. No.

STR. SITE No.

HWY. No.

LOCATION CO. RD. 2 & ABERARDER CREEK, LAMBTON CO.

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS: DOCUMENTS TO BE UNFOLDED BEFORE MICROFILM

Plot on 40 Φ map

E. M. PETO ASSOCIATES LIMITED

Our Job Number 63180

1287 Caledonia Road,
Toronto 19, Ontario,
789-1126.

30th October, 1963.

The County of Lambton,
County Building,
700 Christina Street, North,
Sarnia, Ontario.

40Φ1-9
GEOCRES No.

Attention: Mr. O. van Deurs

Gentlemen:

Re: Soil Investigation,
County Bridge C 12-6,
Aberarder Creek.

We have pleasure in forwarding to you, six copies of our soil investigation report.

In the following report we have described the soil conditions encountered and have given our observations and conclusions regarding the proposed extension to the existing crossing.

You will find, that due to the lack of information about the existing depth of foundation we have given two foundation elevations. If these foundation depth, as given in the report, do not coincide with the existing, we will be pleased to review our observations for the existing foundation depth if you so desire. However, for any intermediate depth (i. e. between elevation 90 and 85) you may obtain the bearing value by a simple linear interpolation.

STRUCTURE SITE No. 15-294

The observation regarding the settlement, stability and the constructional problems such as the depth of unsupported cut, bottom heave, etc. will only be slightly affected by changes in the foundation depth within the range of depth investigated.

We trust that you will find the report to be complete and will permit you to proceed with the final design. Should you however, wish to discuss any points arising from the report, we shall be pleased to have you call us.

Yours very truly,

F. M. PETO ASSOCIATES LTD.,

F. M. Peto, P. Eng.

BL/vm

COUNTY OF LAMBTON

SOIL INVESTIGATION REPORT

OF

COUNTY BRIDGE C-12-6

ABERARDER CREEK

E. M. PETO ASSOCIATES LIMITED
1287 Caledonia Road,
Toronto 19, Ontario.

TABLE OF CONTENTS

A. INTRODUCTION

We were requested by Mr. O. van Deurs, County Engineer, to carry out a subsoil investigation at the proposed extension to the existing structure over Aberarder Creek.

It is proposed to extend the existing R. C. culvert by some 85 feet in the upstream direction. The span of the existing culvert is 35 feet, with a radius of 20 feet and a length of 125 feet. The existing footings of the culvert are 8 feet wide. Fill cover over the crown of the structure is some 18 feet and the maximum height of the embankment at the culvert is about 30 feet.

B GENERAL INFORMATION

The location of the testholes in relation to the existing culvert together with the assumed soil profile is shown on the attached drawing.

A detailed description of soils encountered and its stratifications is given on the attached borehole logs.

Elevations as given in this report, drawings and graphs are in reference to elevation 100.0 as denoted for the TBM location of which is shown on the attached site plan.

B. GENERAL INFORMATION - Cont'd.

Laboratory test results are given in Appendix A.

Various graphs such as geotechnical soil properties, results of stability analyses, lateral pressures on vertical bracing and time-settlement relationship for the embankment are given on Figures 1 to 5 in Appendix B.

C. SITE AND GEOLOGY

Aberarder Creek at the site follows a south-north course. At the time of this investigation only the part of the creek adjacent to the existing culvert had ponded water. The creek south of the present road and the abondoned road was dry. The existing gravel road crosses Aberarder Creek by means of a R. C. culvert of 35 feet span. The approach embankment is about 30 feet high. West and east of the creek crossing the existing grade rises some 40 to 60 feet above the creek level.

Generally the creek valley (flood plain) is some 50 to 60 feet wide, with the grade rising on both sides. On the east side the banks are some 50 to 60 feet high and much steeper than on the westerly side, where the grade slopes in the form of wide terraces down to the creek bed. Some 30 to 40 feet south from the toe of the existing embankment there are some old bridge abutments. These concrete abutments are very badly

C. SITE AND GEOLOGY - Cont'd.

broken up. In both cases the abutments are separated from the wing walls. Boulders were strewn along and beside the creek. A particularly large number of boulders is located near the east end of the abandoned bridge abutment.

According to the soil investigation between 30 and 50 feet of glacial drift covers the bedrock on this site. The bedrock is a black, fissile shale, part of the late Devonian or early Mississippian Kettle Point formation. During the Pleistocene period, thick layers of clay and clayey till were laid down. The lowest layer just above the bedrock consists of clayey till and boulders. This is possibly Illinoian or early Wisconsin till. The overlying sandy silt till was probably laid down during the first half of the Wisconsin glaciation.

Overlying the sandy silt till there is a layer 2 feet thick of stratified clay and silt with organic matter. Above the organic layer there is 10 to 12 feet of silty clay which together with the underlying silts and clays was probably deposited during the warmer Cary Port Huron interval when glaciers retreated to the Georgian Bay area and glacial Lake Arkona covered the Lake Huron and Erie Basins. The ice sheet advanced once more over this area and left another silty clay till layer. After the ice

C. SITE AND GEOLOGY - Cont'd.

retreated, lakes covered the area from time to time, but due to the variation in water level only a few feet of sediment were deposited. The covering sand was probably deposited in the post glacial Lake Nipissing.

D. SOIL CONDITIONS

The following strata were encountered in the order indicated:

1. Brown silty fine sand and gravel.
2. Grey-brown very silty clay with grits and pebbles.
3. Grey-brown to grey silty clay, odd grit.
4. Grey interbedded silt, silty clay and organic clayey silt.
5. Grey sandy silt till.
6. Dark grey boulder clay.
7. Dark grey shale.

Some of the geotechnical properties of these strata consisting of Water content, Liquid and Plastic Limits, N values and Undrained shear strength are given given on Figure 1 in Appendix B. The following is

D. SOIL CONDITIONS - Cont'd.

a brief description of each stratum:

1. Brown silty fine sand and gravel

This uppermost layer in combination with the topsoil varied in thickness between 1 ft. 4 in. and 4 ft. 6 in. It contained numerous boulders and the density of this layer could be described only as compact.

2. Grey-brown very silty clay with grits and pebbles

The average thickness of this deposit is about 10 feet. Its average lower limit can be assumed to be at elevation 90. It was of soft to firm consistency with the N-values varying between 4 and 13. The natural water content varied between 11 and 22 % and was only slightly higher than the plastic limit of the deposit. According to a consolidation test, this deposit is pre-consolidated. The Atterberg limits placed the soil in the CL-range of the Casagrande's Classification system.

D. SOIL CONDITIONS - Cont'd.

The average values for this deposit are:

Liquid Limit - 29 %

Plastic Limit - 13 %

Plasticity Index - 16 %

Liquidity Index - 0.26 to 0.53

Wet density γ = 130 lb./cu.ft.

Dry density γ_d = 112 lb./cu.ft.

Void ratio e - 0.5

Coefficient of Volume Change m_v = 0.006 sq.ft./ton

Coefficient of Consolidation c_v = 0.011 sq.in./min.

Coefficient of permeability k = 1.8×10^{-7} in/min.

Overconsolidation ratio - 2.4

Assumed average undrained shear strength, based on N-values and natural water content, is 1200 lb./sq.ft.

3. Grey-brown to grey silty clay, odd grit

This deposit contained less grits then the overlying soil and was much more uniform. The average thickness was about 12 feet with its average lower limit at about elevation 78.

The undrained shear strength, at the upper portion, was lower than for the overlying deposit, and increased practically linearly with

D. SOIL CONDITIONS - Cont'd.

depth. The consolidation characteristics of this deposit were only slightly different from those of the overlying stratum although the Atterberg limits indicated that the deposit is a CI clay.

Generally the N-values varied between 4 and 11. The natural water content were higher than for the grey-brown very silty clay but still nearer to the plastic than the liquid limit.

The average values are:

Liquid Limit	-	44 %
Plastic Limit	-	19 %
Plasticity Index	-	25 %
Liquidity Index	-	0.06 to 0.37
Wet density γ	-	125 lb/cu. ft.
Dry density γ_d	-	97 lb/cu. ft.
Void ratio	-	$e = 0.72$
Coefficient of Volume Change		$m_v = 0.008$ sq. ft/ton
Coefficient of Consolidation		$c_v = 0.014$ sq. in/min.
Coefficient of Permeability		$k = 3.2 \times 10^{-7}$ in/min.
Overconsolidation ratio		2.0
Undrained shear strength	-	minimum 850 lb/sq. ft.
	-	average 1,000 lb/sq. ft.

D. SOIL CONDITIONS - Cont'd.

4. Grey interbedded silt, silty clay and organic clayey silt

This deposit was encountered at testholes 1 and 3 only in the area of the flood plain. Its thickness varied between 2 and 4 feet. The N-values varied between 5 and 10 and the natural water content between 15 and 23 %. Organic matter was present in very thin laminations and should not have any adverse effect on the structure.

5. Grey sandy silt till

Underlying the interbedded silt stratum there is a sandy silt till, which contained seams and pockets of sand and clay. The average natural water content was about 15 %.

6. Dark grey boulder clay

Immediately overlying the bedrock there was a 2 to 4 feet thick deposit of silty sand, intermixed with clay and containing grits, pebbles, stones and boulders. It is to be classified as boulder till. The N-values were about 50, suggesting that the deposit is dense.

D. SOIL CONDITIONS - Cont'd.

7. Dark grey shale

At depths varying between 34 and 37 feet below grade bedrock was encountered. The bedrock was a dark grey, with light grey bands, fissile shale. The recovery of the upper 5 feet of the bedrock was just above 50 % which indicates that the upper 5 feet of the bedrock may be fissured and unsound. At this depth a source of artesian water was found with an equivalent head of about 30 feet. With depth the recovery increased (see borehole log for testhole 3) to 97 %.

E. OBSERVATIONS AND CONCLUSIONS

1. Foundation elevations and allowable bearing values

We do not have the information available about the existing foundation depth and thus two foundation elevation were choosen: elevation 90 i.e. about 10 feet below the average existing grade and elevation 85 i.e. about 15 feet below the average existing grade. The bottom of the creek, at the time of this investigation, was at about elevation 95, thus foundation elevation 90 will give 5 feet cover and elevation 85 about 10 feet of cover. Scour conditions were not known, however, it is assumed that 5 feet of foundation depth (i.e. elevation 90)

E. OBSERVATIONS AND CONCLUSIONS - Cont'd.

will give sufficient protection against scour. If it is not sufficient a shallow cut-off may be installed.

The allowable bearing values were calculated for strip footings 6 to 16 feet wide and it was found that only a slight variation exists in bearing values for different footing sizes. This is due to the variation of the undrained shear strength with depth. Thus in order to simplify, one average bearing value is given for each foundation depth, regardless of footing width, but not exceeding 15 feet in width.

1. for foundation elevation 90

   total allowable bearing value: 2.6 kip/sq.ft.

2. for foundation elevation 85

   total allowable bearing value: 3.2 kip/sq.ft.

These values were calculated on the basis of the undrained shear strength only (factor of safety of 3 against shear failure) and do not take into account any possible settlements. They also include 5 feet and 10 feet respectively, of effective overburden.

F. OBSERVATIONS AND CONCLUSIONS - Cont'd.

2. Maximum permissible height of fill

Based on the average undrained shear strength of the cohesive deposits (C_u = 1,000 lb/sq.ft.) and an assumed average density of fill (125 lb/cu.ft.) the maximum permissible height of fill, at the site investigated, is 44 feet. This height applies for a factor of safety of 1. With factor of safety of 1.5 the permissible height of fill will be 29.5 feet.

3. Settlement

An estimate of settlement due to 30 feet of fill (estimated height of present fill) was made. The total consolidation settlem nt was about 3 inches under the centre of the embankment. Th time-settlement relationship is shown on Figure 5, Appendix B. Thus according to this calculation a settlement of 1 inch will occur in a time p riod of less than 1 year, 2 inches in about 2.5 years and 3 inches well over 10 years' time.

Elastic settlement (i.e. immediate settlement) was obtained estimating the E-value, or Modulus of Elasticity of the soil, from the consolidation test. Assuming E = 170 kip/sq.ft. an elastic settlement approaching the total consolidation settlement, i.e. of over 3 inches was obtained.

E. OBSERVATIONS AND CONCLUSIONS - Cont'd.

4. Stability Analyses

Stability analyses were carried out for the case when the toe of the existing embankment will be undercut in order to place the proposed culvert close to the existing one. It was assumed that the excavation for the new foundations will start from elevation 100.0.

The results of the analyses for the case of deep seated failures and shallow failures is shown on Figure 2 and 3 in Appendix B.

For the deep seated failures, depending on the location of the trial circle the factor of safety varied between 1.4 and 1.6, which was considered to be adequate.

Slightly lower values for the factor of safety exist for the case of shallow failures. The critical failure surface, neglecting any possible seepage in the embankment was parallel to the slope of the embankment. Assuming further a tension crack, the lowest factor of safety obtained was 1.3 which is also considered to be adequate.

Thus it appears there is little danger of a slip failure during the construction phase.

F. OBSERVATIONS AND CONCLUSIONS - Cont'd.

5. Constructional Problems

1. Height of unsupported cut

Based on the minimum available shear strength of 850 lb/sq.ft. and the density of 130 lb/cu.ft. the maximum unsupported height is about 13 feet. Thus excavations down to elevation 90 may be made theoretically in an unsupported vertical cut, provided there is no additional surcharge.

For the cuts down to elevation 85 the averag undrained shear strength of 1,000 lb/sq.ft. may be assumed; with this value the unsupported vertical cut may be made to about 15.5 feet.

At th side of the embankment, due to existing surcharge, bracing of cuts will be required. The estimated distribution of lateral pressures on bracing (based on th undrained shear strength, i.e. construction period) is shown on Figure 4 in Appendix B. In arriving at these lateral pressures, a surcharg equal to a 15 feet of vertical fill was assumed (i.e. half of the embankment height).

2. Factor of safety against bottom heave

For the excavations down to elevation 90, or about 10 feet below existing grade, th factor of safety against bottom heave was estimated to be 3.9 where there is no surcharge and 1.6 at the embankment side.

F. OBSERVATIONS AND CONCLUSIONS - Cont'd.

For excavations 15 feet deep (i.e. foundation elevation 85) the factor of safety against bottom heave was 3.7 for the sides where there is no surcharge, and 1.9 at the embankment side. The higher factor of safety for the foundation elevation 85 is due to higher undrained shear strength existing at this elevation.

3. Water control

It was established during this investigation that artesian water is present in the upper layers of the bedrock. The general water table was found to be at about elevation 90 to 91, or at about the upper proposed foundation elevation; but during the drilling operations it was noted that even without casing no water was present at the testholes until they reached the sandy till stratum. Based on this observation, it may be stated that the excavations down to elevation 85 may be made without the requirement of elaborate water control. Any water coming into the excavations may be dealt with by pumping from within a sump installed slightly lower than the excavated level.

E. OBSERVATIONS AND CONCLUSIONS - Cont'd.

6. General Observations

Due to probable differential settlement between the existing and the new culvert it would be advisable to construct a flexible connection between the existing and the proposed structure.

In order further, to reduce the possible differential settlement it would be of advantage to place part of the fill before the foundations to the structure. Thus after some settlement has taken place the new culvert could be constructed.

An alternative foundation scheme may be adopted, if the allowable bearing values as given in this report for shallow foundations, are inadequate. The use of piles may be employed, which will penetrate into either the sandy silt till stratum and be terminated in this layer, or will go all the way to the bedrock. Based on the minimum assumed undrained shear strength of the sandy silt till of 6,000 lb/sq. ft. the allowable bearing value for the pile foundations based on this stratum will be about 9 to 10 ton/sq. ft.

If the piles penetrate to the shale bedrock and rest on its upper surface an allowable bearing value of 15 ton/sq. ft. may be used. However, for the piles reaching the shale bedrock the effect of artesian water present in thos shale should be taken into account in the pile design.

F., OBSERVATIONS AND CONCLUSIONS - Cont'd.

The quite variable nature of the sandy silt till and the presence of boulders in the dark grey boulder clay may require the use of the steel piles.

The effect of negative friction on the piles, due to settlement of the fill, should be considered if the piles are driven before placement of the fill; to reduce this effect, part of the fill could be placed and allowed to stay for a definite period (say about 1 year) and then the piles driven.

Yours very truly

E. M. PETO ASSOCIATES LTD.,

C. F. Freeman

C. F. Freeman, P. Eng.
Chief Engineer.

Report Prepared by:

B. Lewicki

B. Lewicki, P. Eng.

BL/vm

Our Job Number 63180

October, 1963.

A P P E N D I X "A"

LABORATORY TEST RESULTS

ATTERBERG LIMITS

Job No. 63180

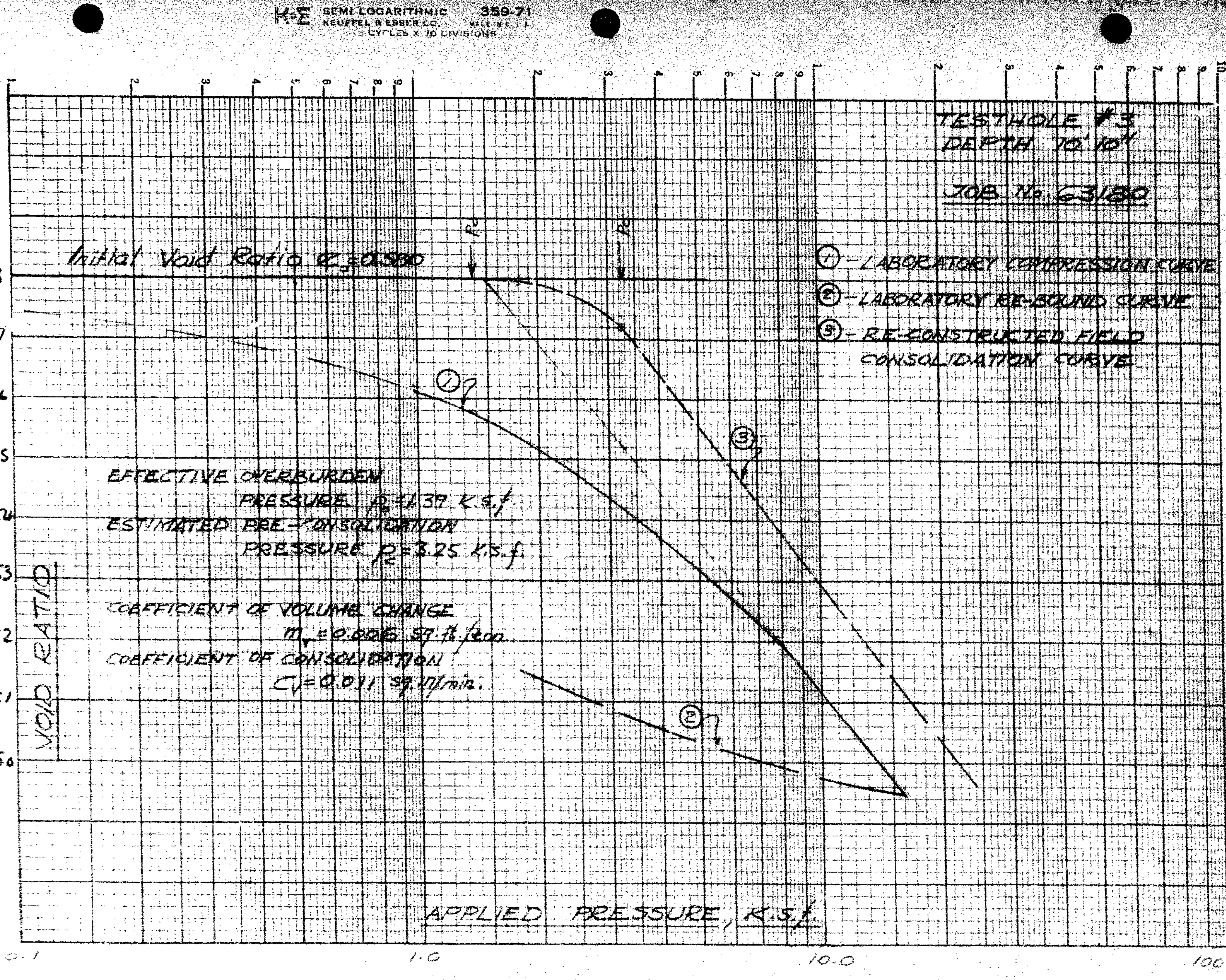
Testhole	Depth	Elev.	L.L. in	P.L. per	P.I. cent	M/C	L.I.
1	15'-16'	82.8	42.4	19.[illegible]	22.6	25.[illegible]	27
2	7'-8'	97.0	2[illegible].3	13.[illegible]	15.5	17.8	.26
2	21'6"-23'	[illegible]2.2	42.6	17.6	25.0	26.[illegible]	.[illegible]7
3	5'-6'6"	9[illegible].0	2[illegible]	13.2	16.2	21.7	.53
3	11'6"-13'	[illegible]8.4	[illegible]6.5	1[illegible].7	26.8	21.4	06
[illegible]	22'-23'	7[illegible].2	31.4	16.2	15.2	22.[illegible]	.43

Job No. 6110
15th October 1961
P. M. A.

E. M. PETO ASSOCIATES LIMITED

UNCONFINED COMPRESSION TEST DATA SHEET

Borehole Number	Sample Number	Depth feet	Elevation Feet	Nat. M. C.	Wet Density p.c.f.	Dry Density p.c.f.	Degree of Saturation %	Void Ratio, e	% Strain at Failure	u/c Shear Strength p.s.f.
1	5	7'6" 8'0"	90.5	18.6	129.0	109.0	92.5	0.54	20.0	810
1	6	8'0"-8'6"	90.0	15.1	133.0	115.0	88.0	0.46	20.0	900
1	8	12'6"-13'0"	85.5	26.7	124.0	98.0	100.0	0.72	17.0	2710
1	9	13'0"-13'6"	85.0	31.7	123.0	93.5	100.	0.86	10.0	1620
1	12	17'6" 18'0"	80.5	24.3	122.5	98.0	91.5	0.72	7.0	1300
1	13	18'0"-18'6"	80.0	29.0	116.0	90.0	90.0	0.87	11.0	1300
1	17	23'-23'6"	75.0	14.1	137.0	120.	94.0	0.41	20.0	7800
2	7	15'-15'4"	89.3	20.6	135.2	112.2	100.0	0.50	20.0	690
2	8	15'4"-15'8"	89.0	26.2	125.5	99.5	100.0	0.69	15.0	1470
2	12	20'-20'4"	84.3	25.3	127.2	101.4	100.0	0.66	20.0	1540
2	13	20'4" 20'8"	84.0	23.7	129.7	104.8	100.0	0.61	8.3	2345
3	11	15'-15'4"	85.5	26.5	128.8	101.6	100.0	0.66	20.0	1038
3	12	15'4"-15'8"	85.2	26.5	127.2	100.7	100.0	0.66	15.0	1100



APPENDIX 'A'
FIG. 1

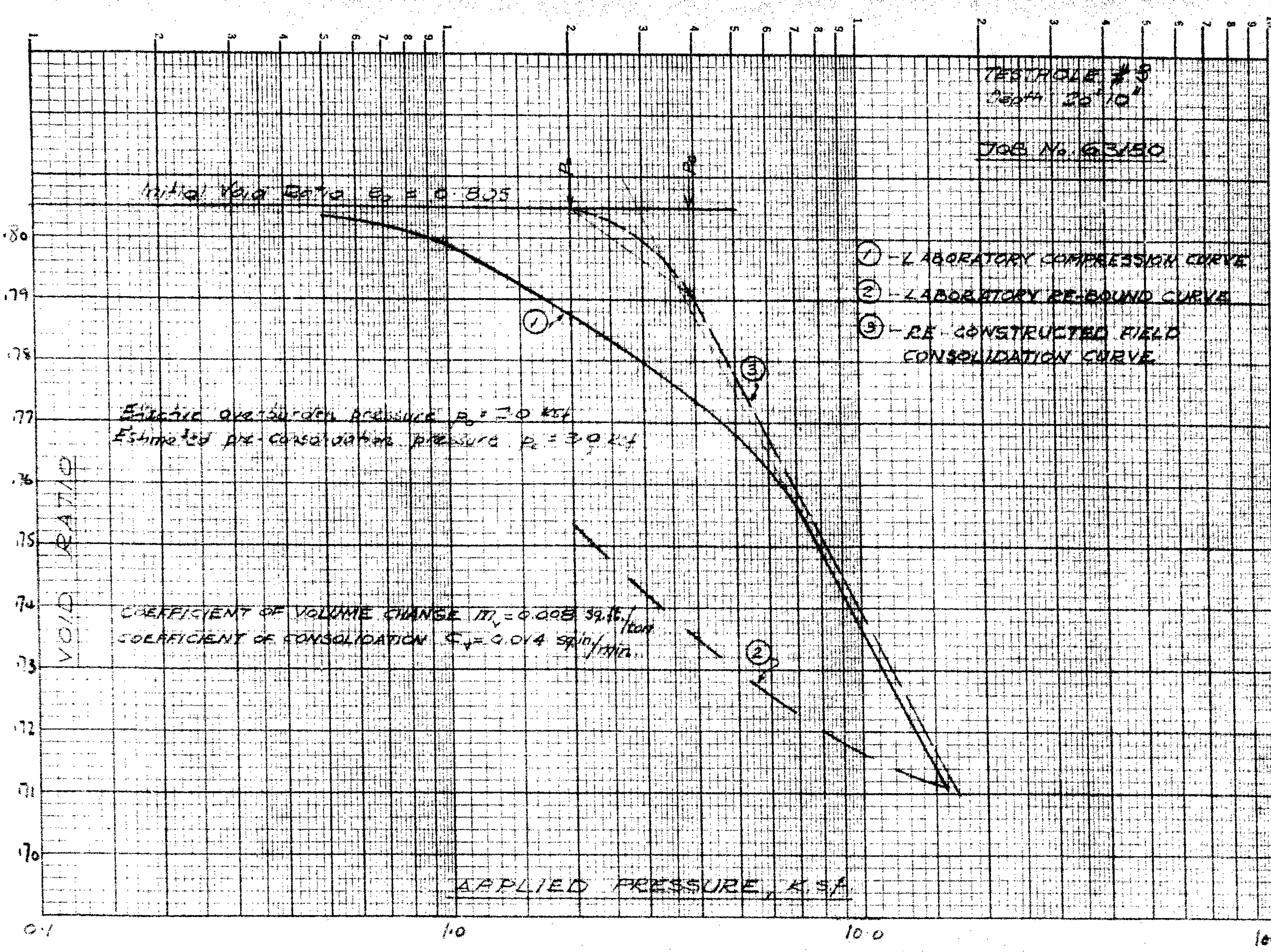

TESTHOLE #3
Depth 20'10"
JOB No. 63/50
Initial Void Ratio e_o = 0.805
p_o
p_c
① - LABORATORY COMPRESSION CURVE
② - LABORATORY RE-BOUND CURVE
③ - RE-CONSTRUCTED FIELD CONSOLIDATION CURVE
Effective over-burden pressure p_o = 2.0 ksf
Estimated pre-consolidation pressure p_c = 3.9 ksf
COEFFICIENT OF VOLUME CHANGE m_v = 0.008 sq.ft./ton
COEFFICIENT OF CONSOLIDATION c_v = 0.014 sq.in./min.
VOID RATIO
.80
.79
.78
.77
.76
.75
.74
.73
.72
.71
.70
APPLIED PRESSURE, K.S.F.
0.1
1.0
10.0
100.0

FIG. 2

A P P E N D I X " B "

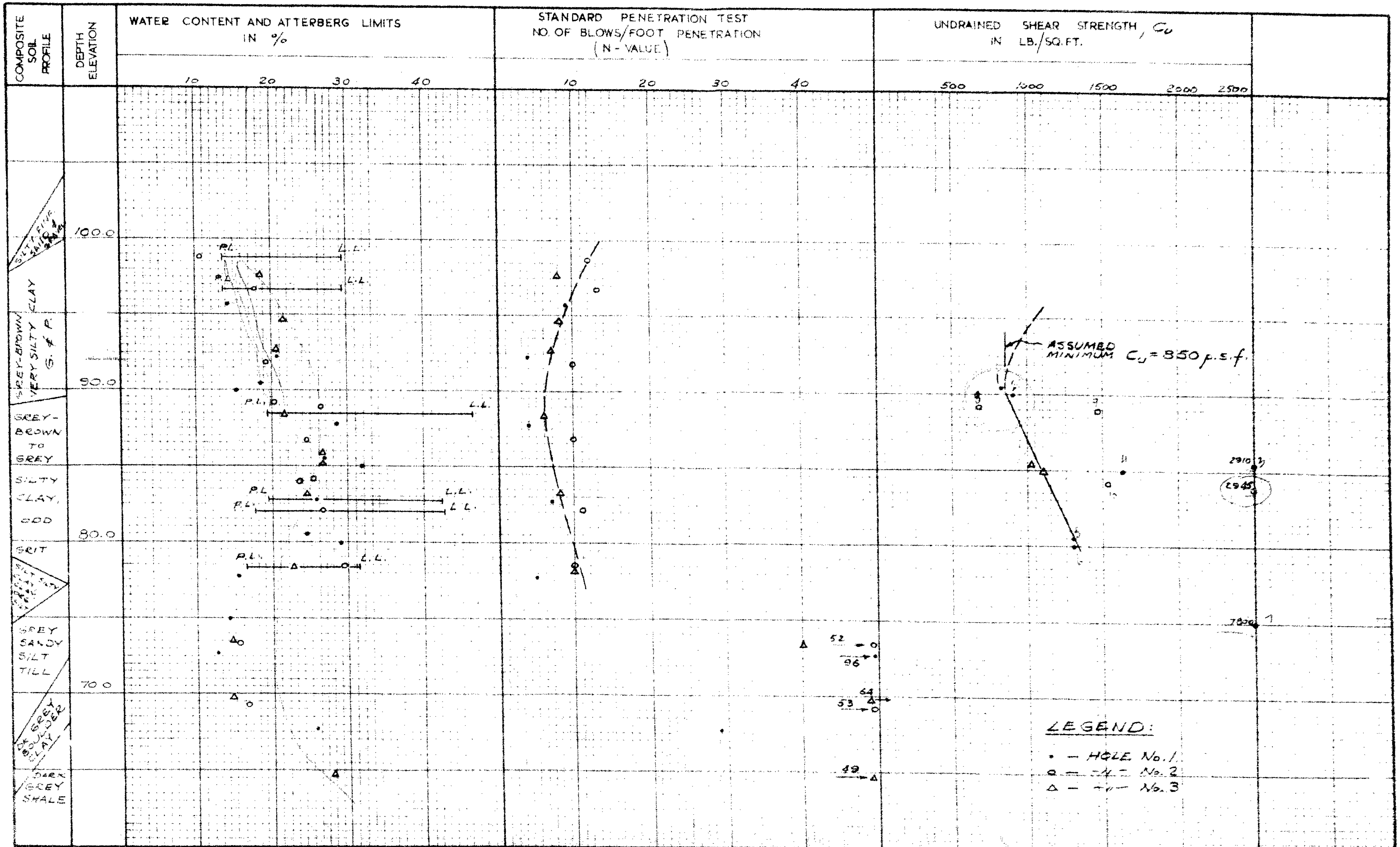
VARIOUS GRAPHS

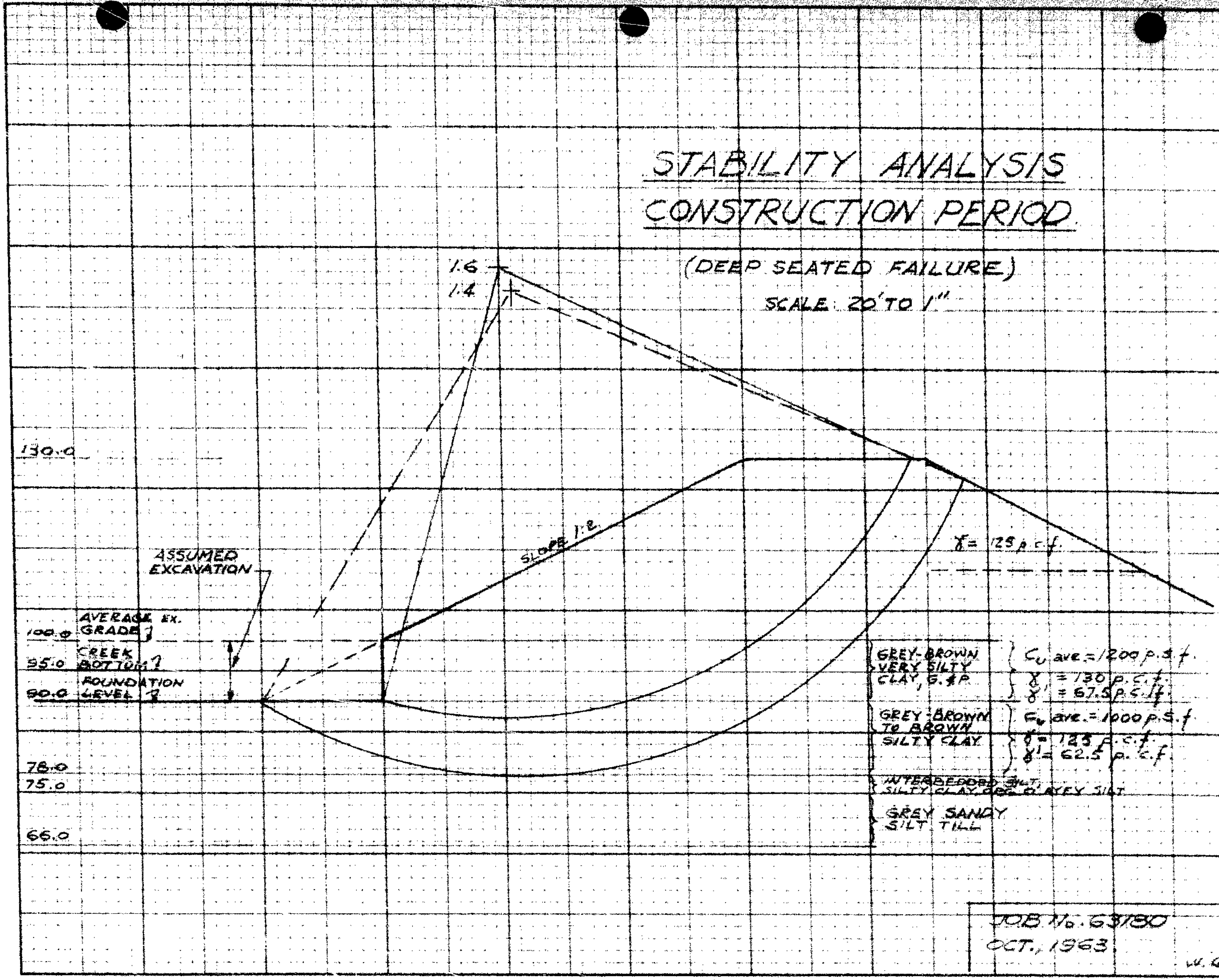
e. m. peto associates ltd.

GEOTECHNICAL SOIL PROPERTIES

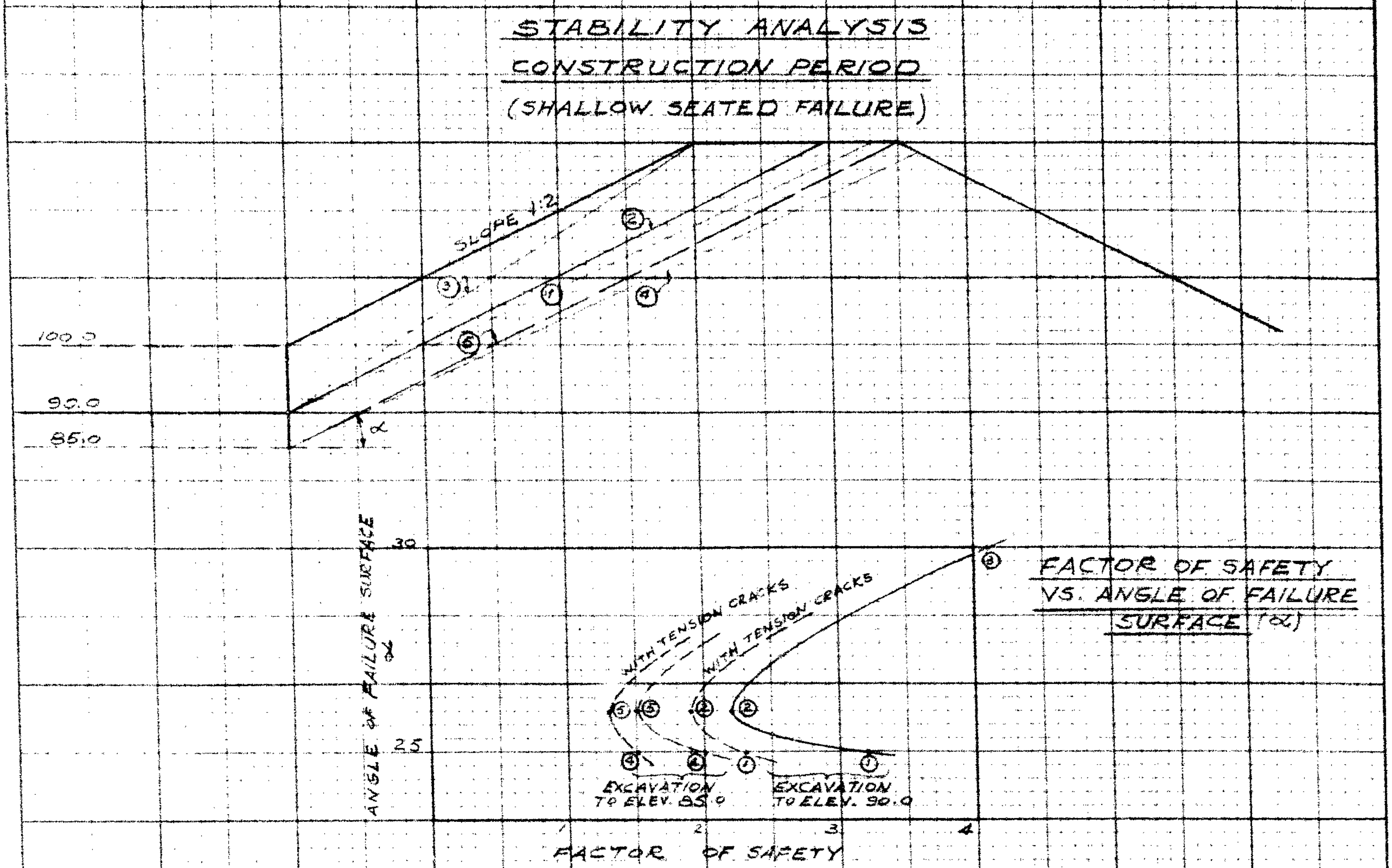
40Ø1-9
GEOCRES No.

JOB NO. 63/80
APPENDIX B
FIG. 1



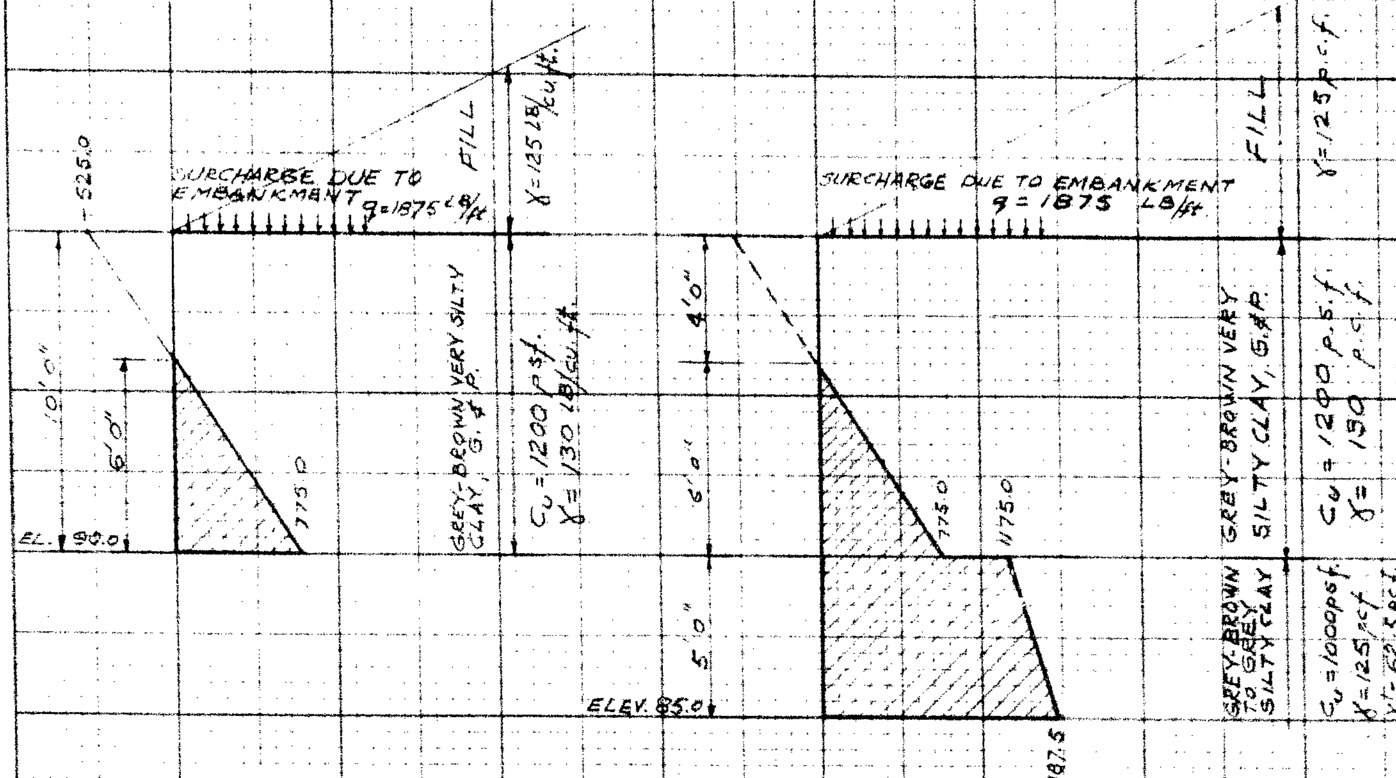


APPENDIX "B"
FIG. 2



JOB No. 63180
OCT., 1963
W. G.

APPENDIX "B"
FIG. 3



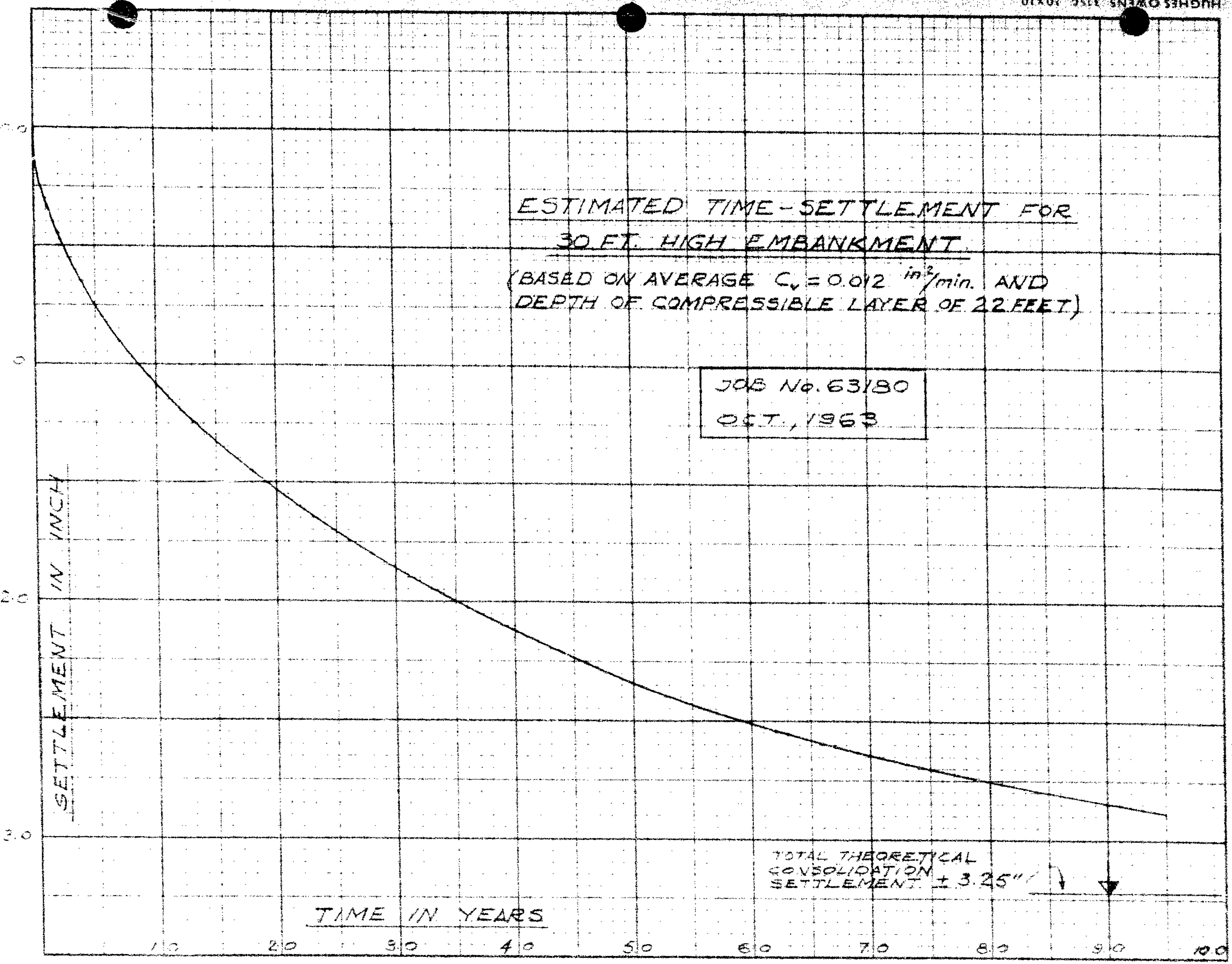
ESTIMATED DISTRIBUTION OF LATERAL PRESSURES ON BRACING OF VERTICAL CUTS

(BASED ON EQUATION $P_a = \gamma H - 2C_u$)

JOB No. 63180
OCT., 1963

W. G.

APPENDIX "B"
FIG. 4



APPENDIX "B"
FIG 5

63180 #1

this margin reserved for binding

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name County Bridge C.12.6 — Job No. 63180 — Borehole No. 1

Client The County of Lambton — Casing 4" ≠ BX — Boring Date Oct. 4-5/63

Elevation assumed — Compiled By BL — Checked By W.G.

4061-9 GEOCRES No.

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- A.S. AUGER SAMPLE
- C.S. CASING SAMPLE
- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- M. MOIST
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL
- W.T.P.L. WETTER THAN PLASTIC LIMIT
- D.T.P.L. DRIER THAN PLASTIC LIMIT
- A.P.L. ABOUT PLASTIC LIMIT

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No and Condition	Sample Type	No. of Blows per Ft.	Natural Moisture Content	WATER LEVELS & REMARKS
		98.3	0'0"						
Silty fine sand & gravel	Grey-brown				1	c.s.			Almost dry.
			1'6"		2	c.s.			
Very silty clay, grits pebbles, odd stones(till soil)	Grey-brown	firm to stiff			3	s.s.	9	14.2	at P L
			5'0"						
As above	As above	Soft to firm			4	s.s.	4	20.8	W T P L
					5 6	2"sl			
			10'0"						
Silty clay, odd grit and ston s	As above slightly more grey	As above			7	s.s.	4	28.3	W T P L
					8 9	2"sl			Gradual transition into grey
			15'0"						
As above	grey	Firm			10	s.s.	7	25.9	W T P L
					11 to 13	2"sl			
			20'0"						
Interbedded with silty clay, clay and organic clayey silt	Grey	Firm			14	s.s.	5	15.2	W T P L
			22'0"		15 to 17	2"sl			Started using washwater from 22' Seam of silty fine sand 23'-23'6"
			25'0"						
Sandy silt till	Grey	Extremely dense.			18	s.s.	96	12.5	Moist
			30'0"						
As above with pockets of clay, seam of coarse sand	As above	Compact to dense			19	s.s.	29	25.3	Wet
			32'5"						
Weathered Shale (or boulder)	Dk. grey								
			34'2"		20	w.s.			
						r.c.			Recovery: 51%
									Artesian Water at about 39 ft. depth.
			39'3"						

Testhole terminated at 39'3"

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name: County Bridge C 12. 6 Job No.: 63180 Borehole No.: 2

Client: The County of Lambton Casing: 4" BX Boring Date: Oct. 7-8, 1963

Elevation: Assumed Compiled By: BL Checked By: W. G.

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- A.S. AUGER SAMPLE
- C.S. CASING SAMPLE
- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- M. MOIST
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL
- W.T.P.L. WETTER THAN PLASTIC LIMIT
- D.T.P.L. DRIER THAN PLASTIC LIMIT
- A.P.L. ABOUT PLASTIC LIMIT

GEOCRES No. 40P1-9

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	Moisture Content	WATER LEVELS & REMARKS
		104.5	0'0"						
Top Soil	Black		0'3"		1	c. s.			Almost dry
Silty fine sand, some pebbles & stones	Brown				2	c. s.			
silty fine sand & gravel	as above	compact			3	s. s.	18/6"		Boulder at 2'6"
			4'6"						
Very silty clay, grits pebbles, odd stone (till texture)	Brown	Stiff			4	s. s.	12	10.6	D T P L
As above	Grey-brown	as above			5	s. s.	13	17.8	at about P. L.
			10'0"						
						3"SL.			
As above, less grits & pebbles	As above	As above			6	s. s.	10	19.2	W T P L
			15'0"		7 to 10	3"SL			
Silty clay, odd grits & pebbles	Grey	As above			11	s. s.	10	24.5	W T P L
			20'0"						
					12 to 15	3"SL			
As above	As above	As above			16	s. s.	11	26.8	W T P L
			25'0"						
As above	As above	As above			17	s. s.	10	29.3	W T P L
			27'6"		18 19	2"SL			
									Stiffens at 27.5 ft.
			30'0"						
Clayey silt, grits and pebbles	Grey	Very Hard			20	s. s.	5	15.5	
			32'0"						
Silty sand, some clay, grits, pebbles & stones	Dk. grey	Very dense	36'0"		21	s. s.	53	16.4	Fragments of shale
			36'5"		21a	s. s.	57/5"		Refusal at 36'5"
		Testhole terminated at 36 ft. 5 in.							

this margin reserved for binding

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name: County Bridge C.12.6 | Job No.: 63180 | Borehole No.: 3

Client: The County of Lambton | Casing: 4" | Boring Date: Oct. 9th, 1963

Elevation: assumed | Compiled By: B. L. | Checked By: W. G.

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

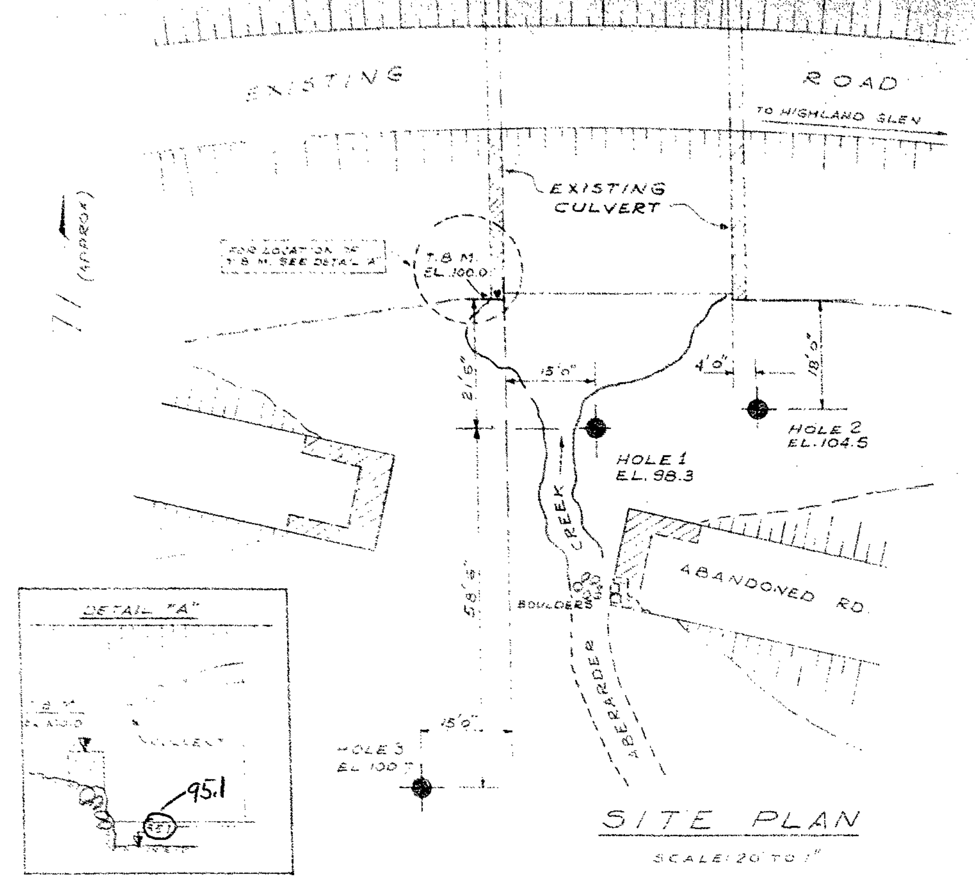
- A.S. AUGER SAMPLE
- C.S. CASING SAMPLE
- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- M. MOIST
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL
- W.T.P.L. WETTER THAN PLASTIC LIMIT
- D.T.P.L. DRIER THAN PLASTIC LIMIT
- A.P.L. ABOUT PLASTIC LIMIT

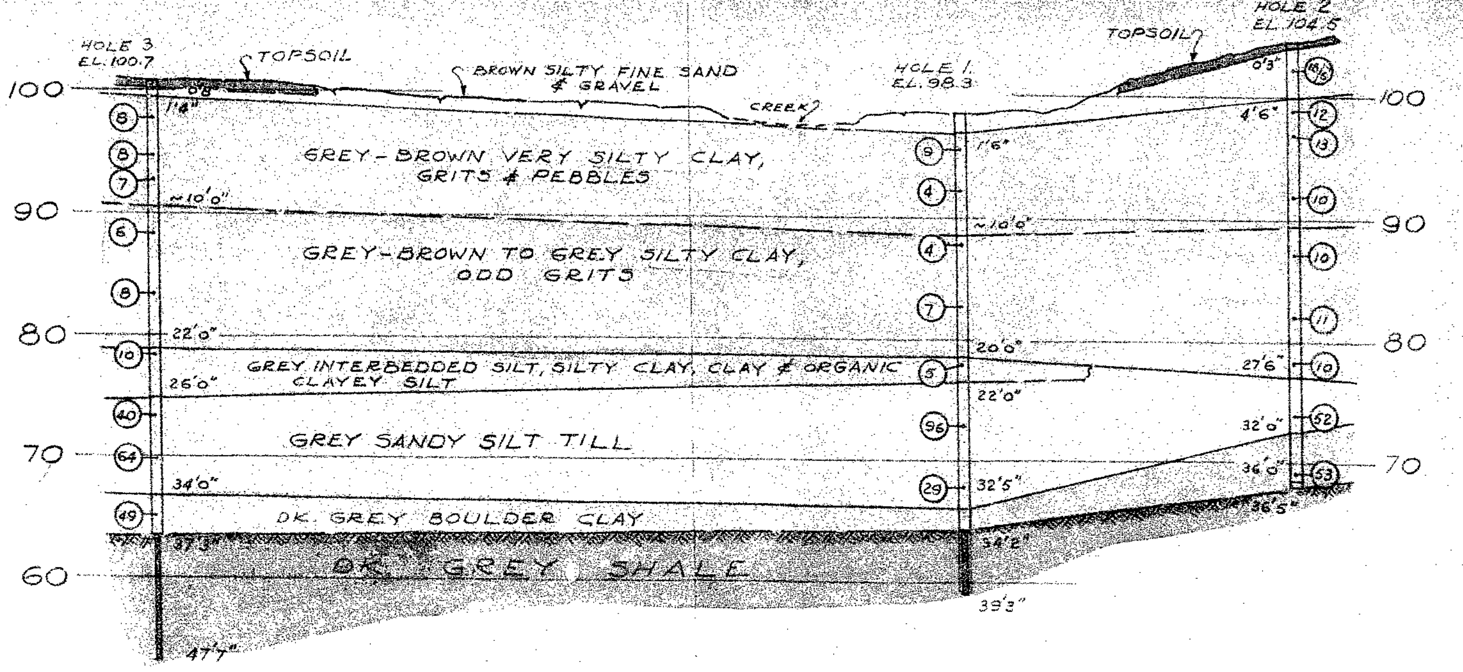
SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Natural Moisture Content	WATER LEVELS & REMARKS
		100.7	0'0"						
Top Soil			0'6"		1	c.s.			Dry
Fine sand & Gravel	Brown		1'4"		2	c.s.			
Very Silty clay, grits, odd pebbles & stones (till texture)	Grey-brown	Firm to stiff			3	s.s.	8	18.6	W T P L
Silty clay, grits, odd pebbles	As above, slightly more grey	As above			4	s.s.	8	21.7	W T P L
As above	As above	Firm			5	s.s.	7	20.7	W T P L
			10'0"		6 to 9	3" sl			
Silty clay, odd grits	Grey-brown	As above			10	s.s.	6	21.4	W T P L
					11 to 14	3" sl			
As above	Changing to Grey	Firm to Stiff			15	s.s.	8	24.6	W T P L
			20'0"		16 to 19	3" sl			
			22'0"						
Interbedded silt, silty clay and organic matter	Mixed Grey	Stiff			20	s.s.	10	22.8	W T P L stiffens at 22.5 ft.
			26'0"		21 to 24	3" sl			
Sandy silt till	Grey	Dense			25	s.s.	40	14.2	Moist
			30'0"						
Clayey silt, grits, odd pebbles	Grey	Very Dense			26	s.s.	64	14.4	O. Moist
			34'0"						
Boulder Till	DK. grey	Hard			27	s.s.	49	27.8	Wet & W T P L
			37'1"						
Shale	Dk. grey with lt. grey bands		37'3"			r.c.			Started diamond drilling at 37'3" Recovery 52%
			42'4"			r.c.			Recovery : 97%
			47'7"						

Testhole terminated at 47'7"



SITE PLAN

SCALE: 20' TO 1"



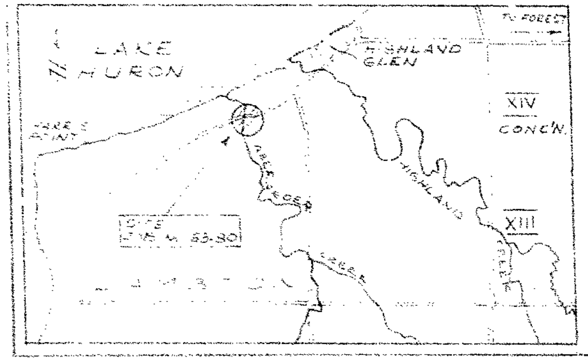
SECTION ON HOLES 3, 1 & 2

SCALES: HOR.: 10' TO 1"
VERT.: 10' TO 1"

4001-9
GEOCRES No.

REGISTERED PROFESSIONAL ENGINEER
E. M. PETO
PROVINCE OF ONTARIO

KEY PLAN



LEGEND

● - BOREHOLE

(5) - BLOWS/FOOT (S.P.T.)

NOTE:

SEE BOREHOLE LOGS FOR COMPLETE SOIL DETAILS.

NOTE: The actual soil stratification has been verified from data obtained at the borehole locations only. The inferred contacts shown are based on geological evidence and these may vary from those shown between borings.

THE COUNTY OF LAMBTON
COUNTY BUILDING

COUNTY BRIDGE C.12.6 (#3)

PREPARED BY:
e. m. peto associates ltd

JOB # 63180	OCT. 1963	DWN BY: W.G.	CHECKED: E...