

#63 - F - 236 M

HARDY CREEK BRIDGE

BROOKE TWP.

BA 1752
SITE K-2PP

E. M. PETO ASSOCIATES LIMITED

Our Job Number 63184

1287 Caledonia Road,
Toronto 19, Ontario,
RU9-1126.

December 5th, 1963.

The Township of Brooke,
c/o J. A. Monteith Associates Ltd.,
Consulting Engineers,
4236 Petrolia Street,
Petrolia, Ontario.

Attention: Mr. G. Ingram

Gentlemen:

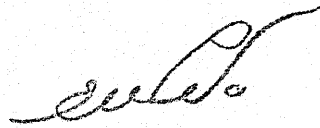
Re: Soil Investigation
Hardy Creek Bridge

We have pleasure in forwarding four copies of our soil investigation report. Two copies of this report have been sent directly to Mr. O. van Deurs, County Engineer.

We trust that the following report covers all the points in which you were interested. Should you however, have some questions arising from this report, do not hesitate to call on us.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

BL/vm

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TABLE I - Atterberg Limits

TABLE II - Unconfined Compressive Tests and Volumetric Analyses

FIGURE 1 to 4 incl. - Mechanical Analyses. Grading curves

FIGURE 5. - Consolidation Test Result (e-log p curve)

FIGURE 6. - Geotechnical Soil Properties

FIGURE 7. - Settlement versus net applied pressure for various footing
and foundation elevations

FIGURE 8. - Results of total and effective stress stability
for various heights of embankment.

FIGURE 9. Consolidation settlement for various heights

BOREHOLE LOG

SITE PLAN & PROFILE

I. INTRODUCTION

We were retained by the Consulting Engineers on behalf of the Township of Brooke to conduct a soil investigation at the location proposed for the Hardy Creek Bridge.

We understand that it is proposed to divert the present course of Hardy Creek so that three existing bridges will be eliminated. These three bridges will then be removed and the creek channel filled in. The location of the proposed diversion with the site of the new bridge is shown on the attached site plan.

The purpose of this investigation was to establish the soil conditions at the site for the proposed bridge.

The investigation consisted of drilling two testholes at the locations shown on the attached site plan. A number of laboratory tests were necessary to establish the soil properties.

II. GENERAL INFORMATION

1. The inferred soil profile is shown on the drawing whilst a more detailed description of soils encountered, with their stratification is given on the borehole logs.
2. The elevations given in the report, drawings and figures are based on elevations supplied by the consulting engineers.
3. The laboratory test results are given on Tables I and II, and Figures 1 to 5 inclusive.
4. Geotechnical soil properties plotted versus elevation are given on Figure 6.
5. Graphs representing the relation between applied net pressures and consolidation settlement are given on Figure 7 for bridge foundations, and Figure 8 for embankments.
6. Results of stability analyses for embankments of various heights is given on Figure 9.

III. SITE AND GEOLOGY

The site proposed for the new bridge is located some 250 feet north of the present intersection between roads dividing Lots 24 and 25 and Concessions X and XI. At present there are three bridges immediately adjacent to this intersection. Further west, in the valley of Hardy Creek, another bridge is situated. All existing bridges are steel girders with concrete deck construction.

Hardy Creek at the site is located in a fairly wide valley, whose sides follow the course of the Creek. The south slopes of the valley rise some 6 to 12 feet above the valley floor and have moderately sloping sides; the northern slopes are steeper and higher.

Physiographically this area lies in the north western part of the Ekfrid clay-plain. The soil investigation showed that between 40 and 45 feet of drift covers the black, fissile shale bedrock. This bedrock belongs to the Devonian Kettle Point formation. The shale is covered by a stratum of silty clay / till which was probably laid down by the Wisconsin glacier. After the glacier retreated this area was covered by glacial lakes from time to time. The existence of these lakes resulted in thick stratified sediments consisting of mainly silty clay, with some silt seams. Recent fluvial sands and silts with some organic matter cover the soil strata.

IV. SOIL CONDITIONS

According to the geological deposition the strata encountered may be subdivided into three main groups:

- a) Recent, fluvial deposits. To this group belongs the uppermost layer of yellow-brown to olive-brown silty fine sand to sandy silt.
- b) Lacustrine deposits, such as:
 - i) Mottled gray-brown very silty clay with grits and pebbles.
 - ii) Grey silty clay with silt seams, and
 - iii) Grey clayey silt.
- c) Glacial deposits to which belong the following strata:
 - i) Dark grey sandy silt with some clay content, grits, pebbles and shale fragments, and
 - ii) Dark grey sand and gravel.

The following is a brief description of the main characteristic of each stratum.

- i) Yellow-brown to olive-brown silty fine sand to sandy silt with clay and organic matter content

This stratum extended to 3.5 feet at testhole 1 and 4.0 feet at testhole 2. It was of loose to compact density with quite high water content.

IV. SOIL CONDITIONS - Cont'd.

(ii) Mottled grey-brown very silty clay with grits and pebbles

This was a lacustrine deposit, which was desiccated and fissured with N-values (i.e. number of blows per foot penetration as obtained during the standard penetration tests) varying between 20 and 25. Thus this layer was of stiff to very stiff density. It was drier or just at the plastic limit of the soil. Due to this condition the mottled grey-brown very silty clay deposit will exhibit characteristics of a pre-consolidated deposit. The lower limit of the layer was 8.5 feet below grade at testhole 1 and 7 ft. 9 in. below grade at testhole 2.

(iii) Brown fine sand

At testhole 1, underlying the desiccated grey-brown silty clay there was a layer of brown fine sand with some clay content. This seam was water bearing.

(iv) Grey silty clay with silt seams

This stratum is the principal layer in the area investigated. The terminal depth of this layer was established at 31 ft. 6 in. at testhole 1 and 30 ft. at testhole 2. The average N-curve for this

IV. SOIL CONDITIONS - Cont'd.

deposit is shown on Figure 6, together with the values of natural water contents and the undrained shear strength. Generally the water content increased with depth, and values as high as 44% were recorded. Due to the presence of the silt seams however, the lower limit of the water contents was about 20%. These silt seams were considered to be the cause of the variation in the Atterberg Limits. The Liquid Limit varied between 13 and 44%, and the Plastic Limit between 20 and 26 %. Therefore, generally this deposit was wetter than its plastic limit, but only in isolated cases did the water content approach the value of the Liquid limits.

The N values decreased at first from an average value of 16 at the upper portion of the deposit to about 9 at about elevation 82, from here a slight increase in N-values was noted.

The undrained shear strength of this deposit increased linearly from a value of 750 lb./sq.ft. at about elevation 100 to 1620 lb./sq. ft. at about elevation 82, from here on the shear strength is virtually constant.

IV. SOIL CONDITIONS - Cont'd.

The grading curve of this deposit is given on Figure 1. The sample was taken from the upper portion of the deposit and the influence of the tilt zone is quite evident.

A consolidation test carried out on a sample which was composed mainly of silty clay is shown as a $\log p$ curve on Figure 5. It is evident from this test, that the deposit of grey silty clay is pre-consolidated.

The average value of the properties of this stratum are:

Wet density 125.6 lb./cu. ft.

Dry density 98.5 lb./cu. ft.

Void ratio $e = 0.71$

Coefficient of volume change $m_v = 0.012$ sq. ft. /ton.

The distribution of undrained shear strength with depth is shown on Figure 6.

V. Grey sandy silt

There was a compact layer of sandy silt in the matrix of grey silty clay, which was located between 16 ft. 6 in. and 19 ft. 3 in. at testhole 2. The sandy silt was saturated and water bearing. A grading curve is given on Figure 1.

IV. SOIL CONDITIONS - Cont'd.

vi) Gray clayey silt

Probably the last layer belonging to the lacustrine deposits is the stratum of gray clayey silt with seams of silty clay. This stratum was met at both testholes. It was of stiff consistency and wetter than its plastic limit. The results of mechanical analyses are shown on Figure 2.

vii) Dark-gray sandy silt with clay content, grits, pebbles and shale fragments

This is the first of the glacial deposits on the site investigated. It was located between depths of 34.5 and 38.0 feet at testhole 1, and 33.0 and 40.0 feet at testhole 2 and was compact. This stratum in the area of testhole 1 was a source of water under pressure.

viii) Dark-gray sand and gravel

Underlying the sandy silt till stratum at testhole 2 there was a 3 feet thick stratum of sand and gravel. It was compact and saturated. Similarly to the sandy silt till at testhole 1, this stratum contained water under pressure.

ix) Dark-gray weathered shale

A layer of weathered shale, 2 feet thick was found overlying the intact shale. The shale itself was in a sound condition and capable of supporting large loads.

V. WATER CONDITIONS

As mentioned in the description of the soil conditions, there are water bearing seams in the stratum of grey silty clay; in addition, water under pressure was found to exist on top of the shale.

The water level readings carried out during the field work were taken over a period of insufficient duration to establish a definite position of the ground water table; however, water level was established at a depth of 3 ft. 6 in. below grade at testhole 1, and 9 ft. 4 in. below grade at testhole 2. Thus the water level was at about elevation 100.5 in both testholes at the time of this investigation.

VI. OBSERVATIONS AND CONCLUSIONS

1. No information is available about the proposed depth of the channel. This depth will determine the minimum foundation depth.

Assuming that the depth of channel will be 4 feet, the minimum foundation depth will then be at elevation 96, if the existing grade at testhole 1 is used as the datum.

2. At elevation 96 a shear strength of 1,000 lb/sq. ft. is available which increases to a value of 1,750 at elevation 82. Based

VI. OBSERVATIONS AND CONCLUSIONS

on this shear strength distribution, the allowable bearing values were calculated using Skempton's formula. It was found that due to the increase of the shear strength with depth the net allowable bearing pressures increased only slightly with the footing sizes. Therefore, for simplification a single value of the net permissible bearing pressure is given at each indicated elevation regardless of footing width. This net allowable bearing pressure is deduced, of course, from shear strength consideration only and does not take into account consolidation settlement, which is dealt with in a subsequent section.

3. The net permissible bearing values, therefore, which may be applied in excess of the present effective overburden pressures are:

<u>Foundation elevation</u>	<u>Allowable net bearing value in kip/sq. ft.</u>
96	
96	2.25
94	2.50

The values as given above incorporate a factor of safety of 3 against shear failure.

VI. OBSERVATIONS AND CONCLUSIONS

4. Settlement analyses carried out on the basis of a consolidation test result given on Figure 5 showed that for increasing footing sizes the settlements will increase. The results of these analyses are presented in graphical form on Figure 7 as settlement versus net applied pressures for various footing sizes and foundation elevations. In fact two foundation elevations were considered: elevation 96 and 94. The footing width was varied from 5 to 30 feet. On this figure the limiting values of the net permissible bearing pressures are given for factors of safety 3 and 2. We recommend the adoption of a factor of safety of 3. If this factor of safety is then adopted, then for a 5 feet wide foundation, the settlement resulting from the application of a net surcharge of 2.25 kip/sq. ft. will be in the order of 0.5 inches. On the other hand for a 30 feet wide footing subjected to the same loading the resulting settlement will amount to about 1.7 inches. The graph given on Figure 7 may be used to arrive at the net permissible bearing value for a given amount of theoretical consolidation settlement.

5. If the net allowable bearing values as given in the proceeding section are insufficient to carry the calculated loads, a pile foundation may be employed. The pile foundation will be of end-bearing type with

VI. OBSERVATIONS AND CONCLUSIONS

piles resting on or in the shale. As the shale was found to be in a sound condition an allowable bearing value for the shale of 20 ton/sq. ft. may be adopted. However, attention will have to be paid to the presence of water under pressure just above the shale surface.

6. Stability analyses were carried out for various heights of embankment placed at the site, assuming that the embankment slopes are 1 vertical to 2 horizontal. The results of these analyses, in terms of total and effective stresses, are given on Figure 8. The effective stress analysis was carried out assuming that the silty clay soil has the following shear parameters:

Cohesive Intercept $c' = 100$ lb/sq. ft. and

Angle of Internal Friction $\phi' = 28^\circ$

These values were selected on the basis of the data published by L. G. Soderman, T. C. Kenney and A. K. Loh in a paper entitled "Geotechnical Properties of glacial clays in Lake St. Clair Region of Ontario".

For the total stress analysis a minimum undrained shear strength of 800 lb/sq. ft. was assumed.

VI. OBSERVATIONS AND CONCLUSIONS

Thus, selecting a factor of safety of 1.5 for the construction period (total stress analysis) the height of embankment which may be placed is 26.5 feet. For long term condition (effective stress analysis), assuming further that the pore pressure ratio is 0.2, the permissible height of embankment (factor of safety 1.5) is only 18 feet. Therefore, the governing factor for the selection of the height of embankment on this site is the long-term condition.

7. The results of settlement analysis for various heights of embankment are given on Figure 3. The settlement, as given on this figure, refers to the theoretical consolidation settlement. Thus for an 18 feet height of embankment a settlement in the order of 2.5 inches may be expected.

8. No bottom heave is likely to occur for excavations to about 20 feet depth. Excavations could be made in an unsupported vertical cut, but due to the presence of the sand and silt seams bracing is advisable. The uppermost layer of silty fine sand to sandy silt may be sloped back for the duration of the construction at a slope of 1 vertical to 1.5 horizontal.

VI. OBSERVATIONS AND CONCLUSIONS

9. Due to the position of the water level, water control measures will be required for a spread footing design. As the sand seams were found to be water bearing cutting them off from the general excavation area is recommended. This could be achieved by close sheeting extended into the grey silty clay layer. However, due to the impermeable character of this deposit the influx of water into excavations will be moderate and may be dealt with by pumping from a sump installed slightly lower than the general excavation level.

10. The soils at this site are generally frost susceptible and should be not used as a backfill immediately adjacent to the structural members. In addition, care should be taken not to expose the foundation soils to weathering or freezing as such conditions may lower the bearing value. Therefore, foundations should be installed as soon as the excavation level is reached.

E. M. PETO ASSOCIATES LTD.,

Report Prepared by:

B. Lewicki

B. Lewicki, P. Eng.

C. F. Freeman

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

FL/vm

Job Number 63184

December 5th, 1963.

T A B L E IA T T E R B E R G L I M I T S

Testhole	Depth	Elevation	Liquid Limit	Plastic Limit	Plasticity Index		Natural Water Content
					in	per cent	
1	7'0"-8'6"	96.2	40.6	22.4	18.2		18.9
1	16'6"-18'0"	86.7	32.2	19.8	12.4		27.3
1	26'6"-28'0"	76.7	33.2	20.2	13.0		23.1

TABLE II

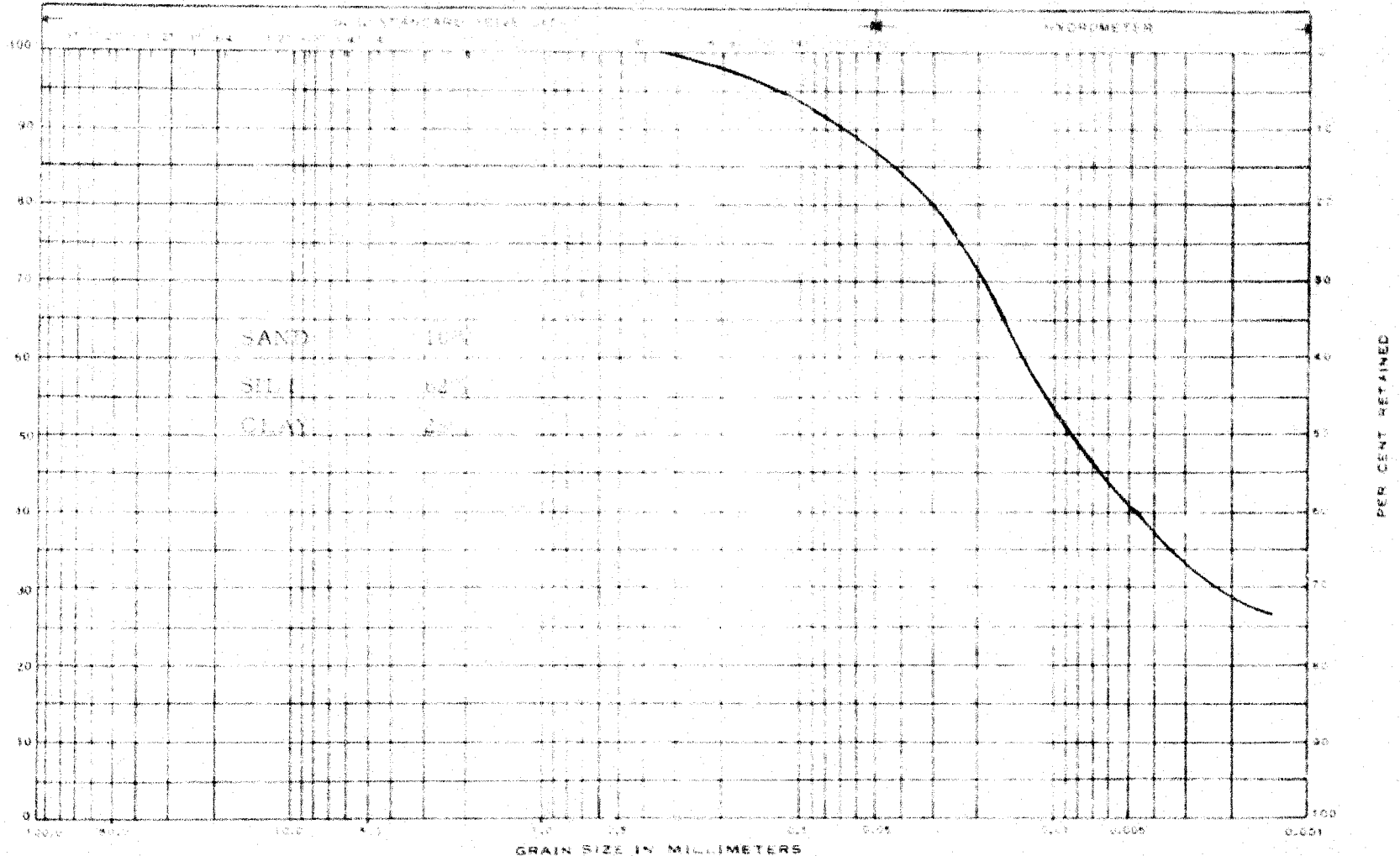
UNCONFINED COMPRESSION TESTS AND VOLUMETRIC ANALYSES

Job No. 6484

Borehole #	Sample #	Depth	Elevation	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.
2	7	11'-11'6"	98.4	22.4	130.2	106.3	100.0	0.60	6.0	810
2	10	15'-6"-16'	93.9	23.7	124.0	100.5	90.0	0.70	20.0	1180
2	11	16'-16'6"	93.4	16.5	117.0	100.3	65.0	0.70	4.0	1780
2	16	21'-21'6"	88.4	23.6	122.5	104.5	100.0	0.64	16.0	1300
2	17	27'-6"-28'	81.9	26.6	120.0	93.3	100.0	0.77	12.0	1620
2	18	28'-28'6"	81.4	24.6	120.0	96.2	100.0	0.67	20.0	1620
2	21	33'-33'6"	76.4	34.0	121.0	90.0	100.0	0.92	10.0	1620

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Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
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MASS. INST. OF TECH. CLASSIFICATION

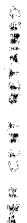
JOB NAME Hardy Creek Bridge JOB NO. 63184 HOLE NO. 1 SAMPLE NO. 7

DEPTH 10'-11'6" ELEVATION 93.2 REMARKS Clayey Silt

GRAIN SIZE DISTRIBUTION

Fig. 1

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT



CLASS INST OR TECH CLASSIFICATION

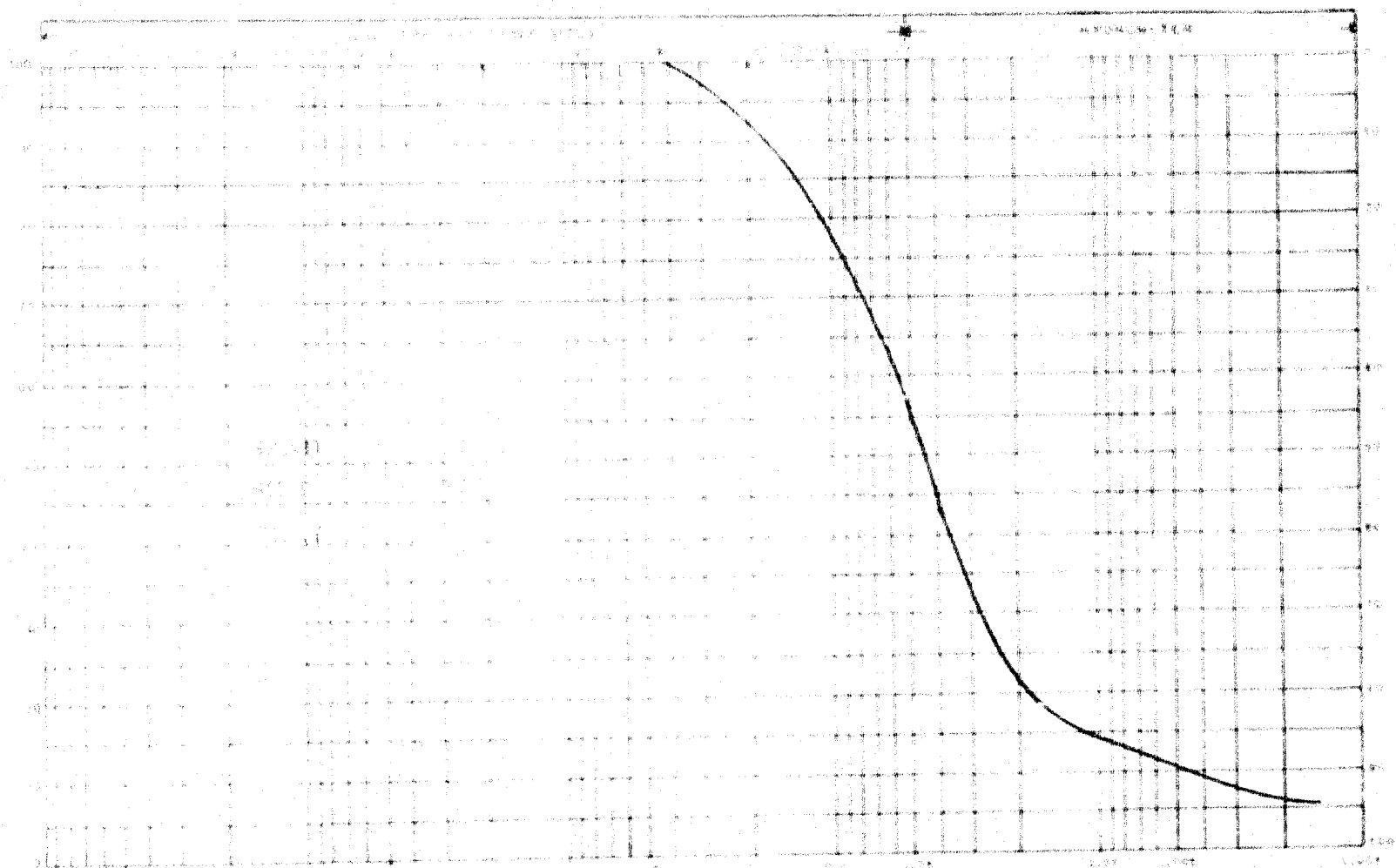
DEPTH	ELEVATION	REMARKS
31.0-33	71.7	Clay Silt

Fig. 2

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Toronto 19, Ontario

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT



GRAIN SIZE IN MILLIMETERS

COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
-------------	-----------	-----------	-------------	-----------	-----------	------

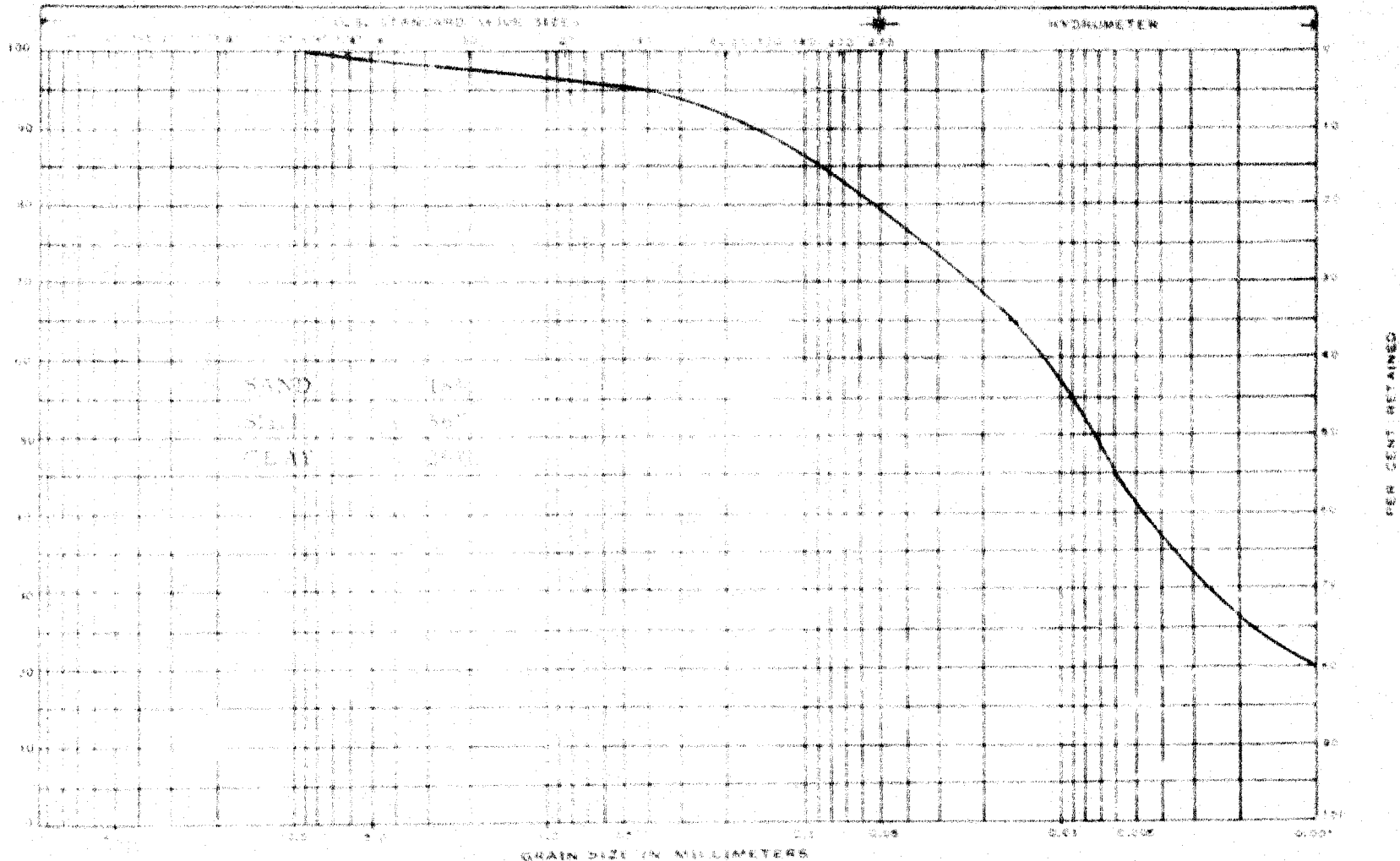
MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Harvey Creek Bridge JOB NO. 63164 HOLE NO. 2 SAMPLE NO. 12
 DEPTH 1.00 ELEVATION 91.3 REMARKS Sandy Silty

GRAIN SIZE DISTRIBUTION

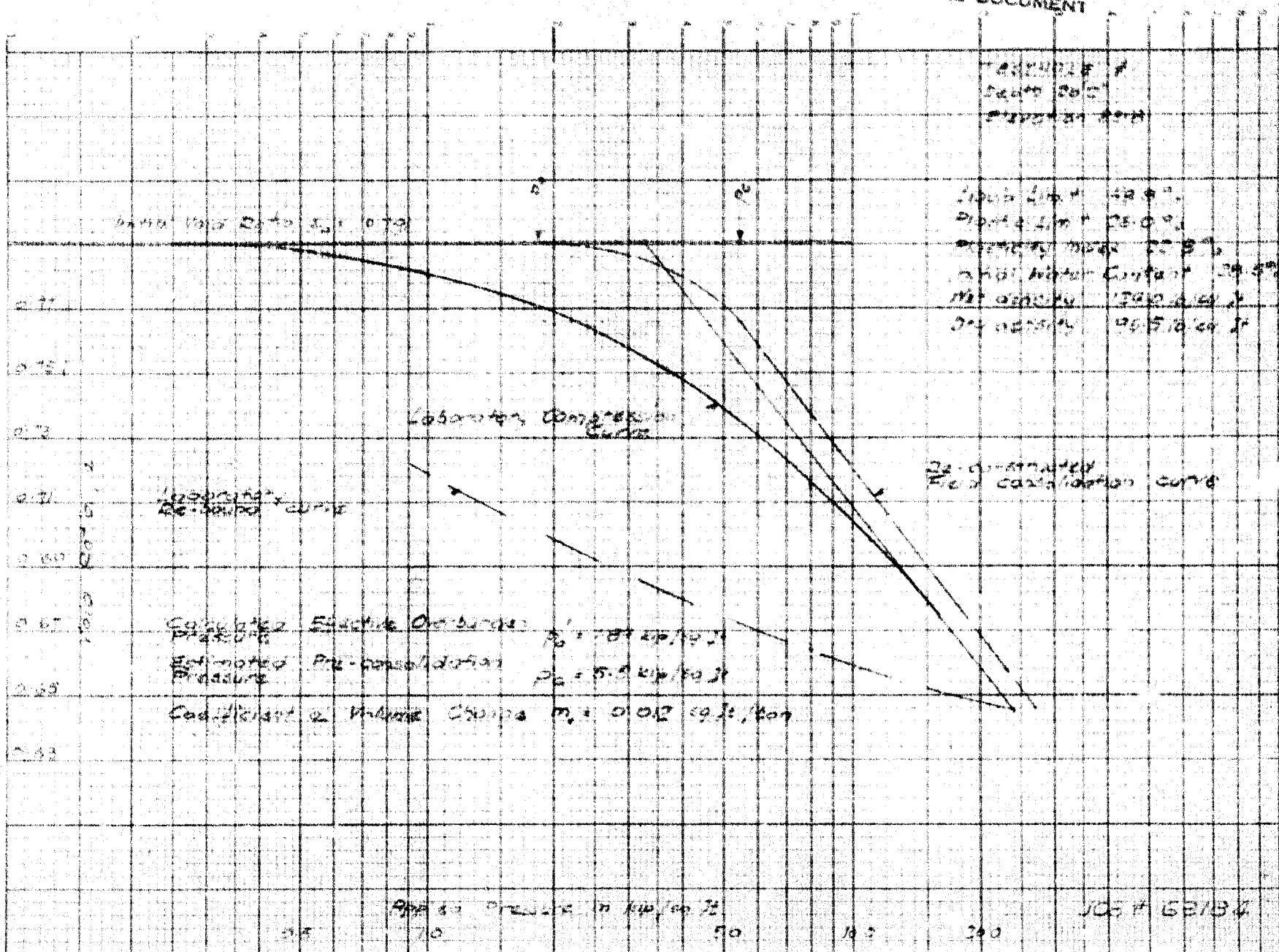
Fig. 3

e. m. pelo associates ltd. DEFECTS IN NEGATIVE DUE TO
 Toronto 19, Ontario. CONDITION OF ORIGINAL DOCUMENT

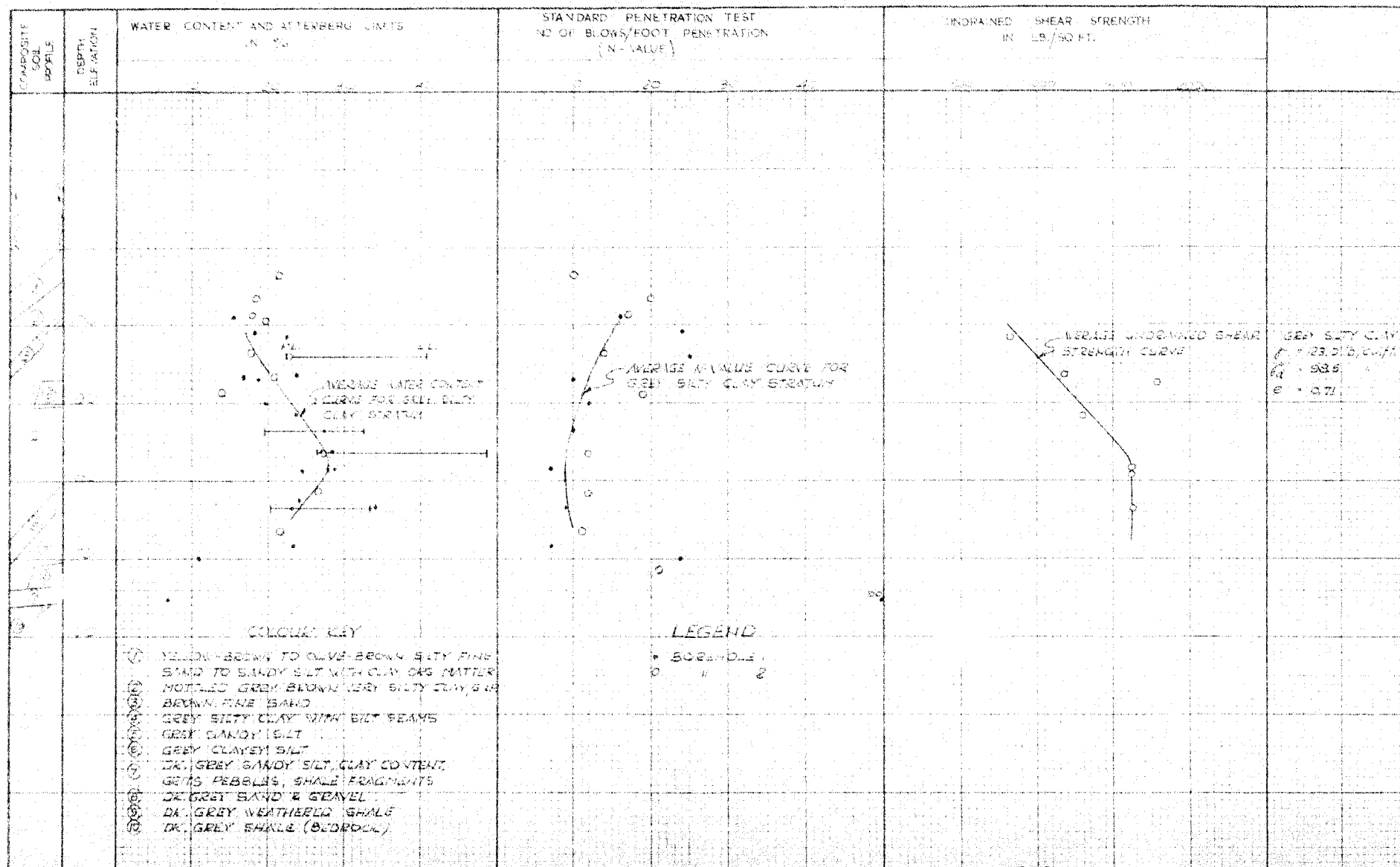


GRAIN SIZE DISTRIBUTION		MASS. INST. OF TECH. CLASSIFICATION		GRAIN SIZE DISTRIBUTION		MASS. INST. OF TECH. CLASSIFICATION	
GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
Hardy Creek Bridge		53184		2		22	
DEPTH 35.36' ELEVATION 73.9		REMARKS Clayey Silt (SIH: MH)					

Fig. 4



GEOTECHNICAL SOIL PROPERTIES



SETTLEMENT VERSUS NET APPLIED PRESSURE FOR VARIOUS FOOTING SIZES AND FOUNDATION ELEVATIONS.

MEASURED AMOUNT OF SETTLEMENT

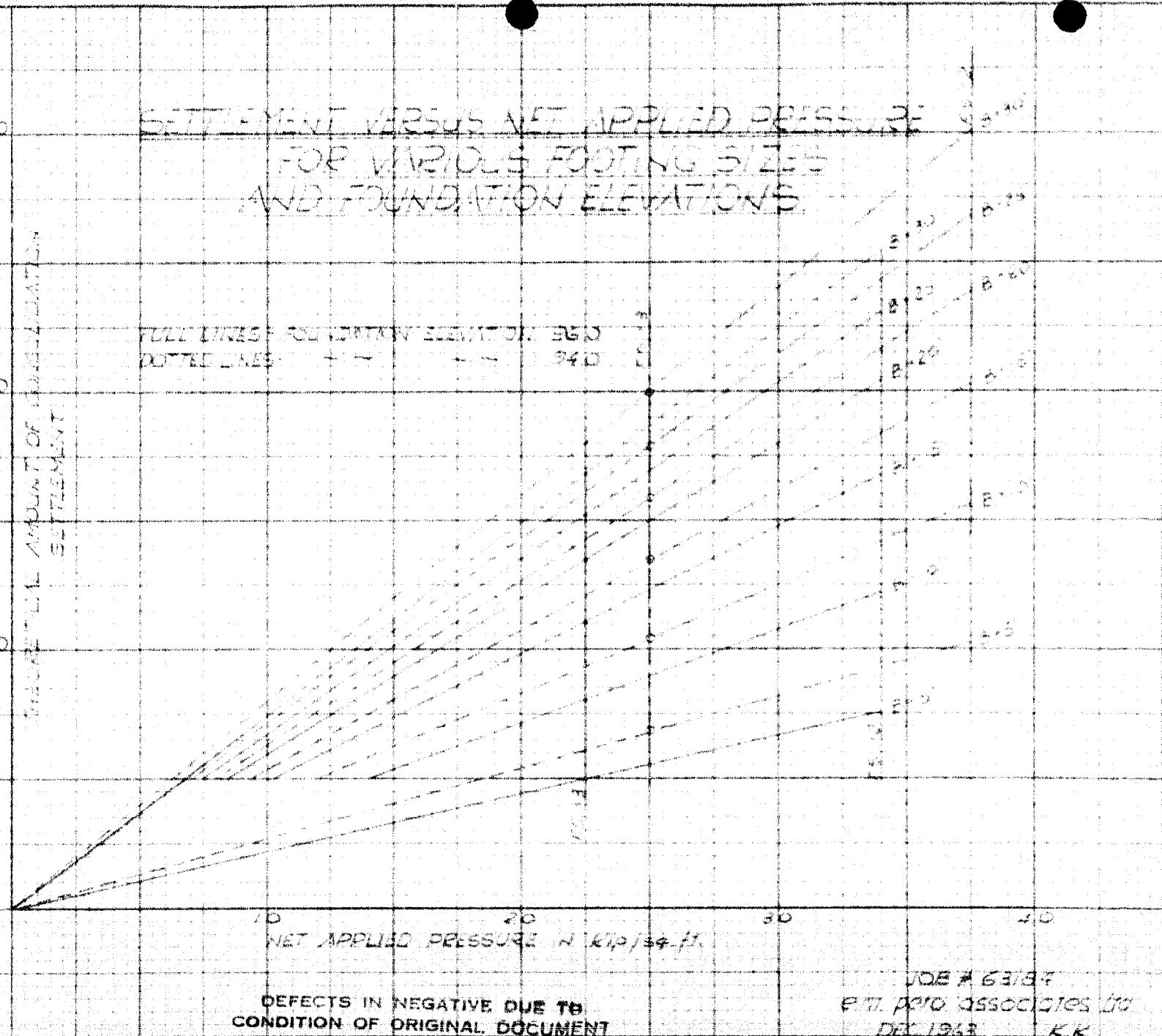
FULL LINES FOUNDATION ELEVATION 36.0
DOTTED LINES " " " " 34.0

NET APPLIED PRESSURE IN KIP/34-ft.

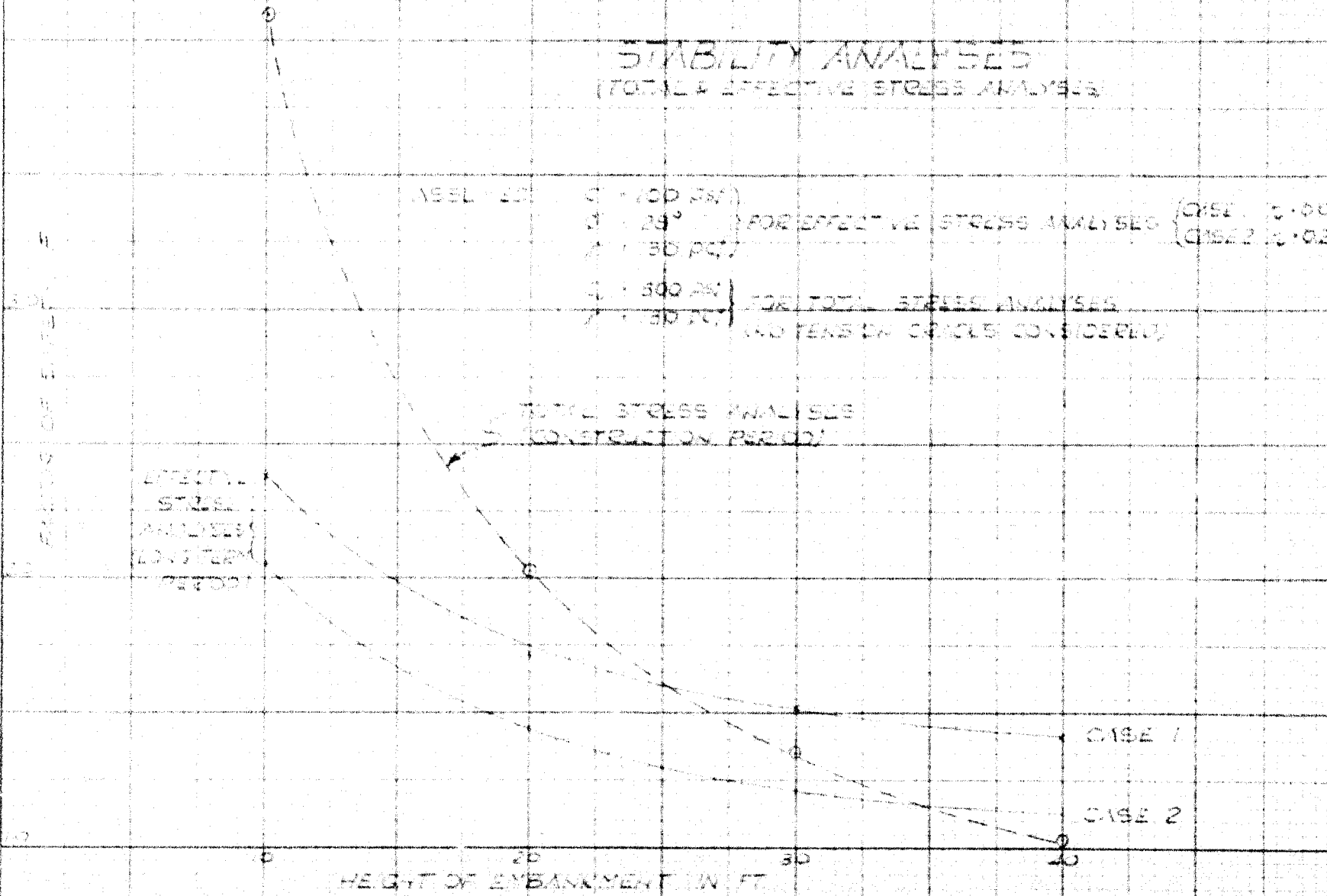
DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

JOB # 63184
P.T. POLO ASSOCIATES LTD
DEC 1963 K.K.

FIG. 7



STABILITY ANALYSES (TOTAL & EFFECTIVE STRESS ANALYSES)



ASSUMPTIONS: $C = 100 \text{ PSF}$
 $\phi = 29^\circ$ FOR EFFECTIVE STRESS ANALYSES (CASE 1: 1.00, CASE 2: 1.02)
 $\gamma = 120 \text{ PCF}$
 $C_u = 500 \text{ PSF}$ FOR TOTAL STRESS ANALYSES
 $\gamma = 120 \text{ PCF}$ (NO TENSION CRACKS CONSIDERED)

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CONDITION OF ORIGINAL DOCUMENT

OB. # 63184
 QM. DETO. 03500 CTS L/U
 DEC. 1967 R.A.

CONSOLIDATION SETTLEMENT FOR VARIOUS HEIGHT OF EMBANKMENT

45

30

20

10

CONSOLIDATION SETTLEMENT IN INCHES

HEIGHT OF EMBANKMENT IN FT

JOB # 6882
S.M. P&O ASSOCIATES, LTD.
OCT 1983

FIG. 9





e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hardy Creek Bridge Job No. 11151 Borehole No.
 Client Township of Brooke Casing 1" & 3" Boring Date Oct 25 1963
 Elevation 104.0 Compiled By E. J. L. Checked By

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.	W.T.P.L.	D.T.P.L.	WATER LEVELS & REMARKS
Silty fine sand, org. matter	Yellow brown		9.10		1	CS				Almost dry
Ditto	Brown				2	CS				Slightly moist
Fine sand	Ditto	Compact			3	SS	16	15.2		Tightly moist
Very silty clay, g. & p.	Mottled grey & brown	Very stiff			4	SS	23	15.3		Moist D.T.P.L.
Silty clay, grits	Ditto	Ditto			5	SS	35	22.0		Moist D.T.P.L.
Fine sand, some clay content	Brown				6	SS				Water bearing seam
Silty clay, odd grit	Grey	Stiff			7	SS	19	14.4		Slightly W.T.P.L.
					8	SS				
					9	SS				
					10	SS				
Ditto	Ditto	Ditto			11	SS				
					12	SS	14	14.7		S.T.P.L.
					13	SS				
					14	SS				
Silty clay, seams of silt	Grey	Ditto			15	SS	19	21.2		W.T.P.L.
					16	SS				
					17	SS				
					18	SS				
					19	SS				
Ditto, increased clay content	Grey	Stiff			20	SS	7	12.6		W.T.P.L.
					21	SS				
					22	SS				
					23	SS				
Ditto, more silt seams	Ditto	Firm to stiff			24	SS	9	23.1		W.T.P.L.
					25	SS				
					26	SS				
					27	SS				
Clayey silt	Grey	Firm			28	SS	5	24.0		W.T.P.L.
					29	SS				
Sandy silt, clay content	Dr. grey	Compact			30	SS	24	10.4		Moist
grits, pebbles, fragments of shale					31	SS				
Changing to					32	SS				
Weathered shale	Dr. grey	Disintegrated			33	SS	31.0	6.7		Moist
					34	SS				
Shale (fissile)	Dr. grey				35	SS				
					36	SS				
					37	SS				
					38	SS				
					39	SS				
					40	SS				
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					100	SS				

DEFECTS IN NEGATIVE DUE TO
 CONDITION OF ORIGINAL DOCUMENT

Borehole "Hardy Creek Bridge"

Revised 4/63
 Print Shop No. 15

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hardy Creek Boring Job No. 109.7 Borehole No. 109.7
 Client Township of Pelee Casing 109.7 Boring Date 10/26/11
 Elevation 109.7 Compiled By 109.7 Checked By 109.7

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

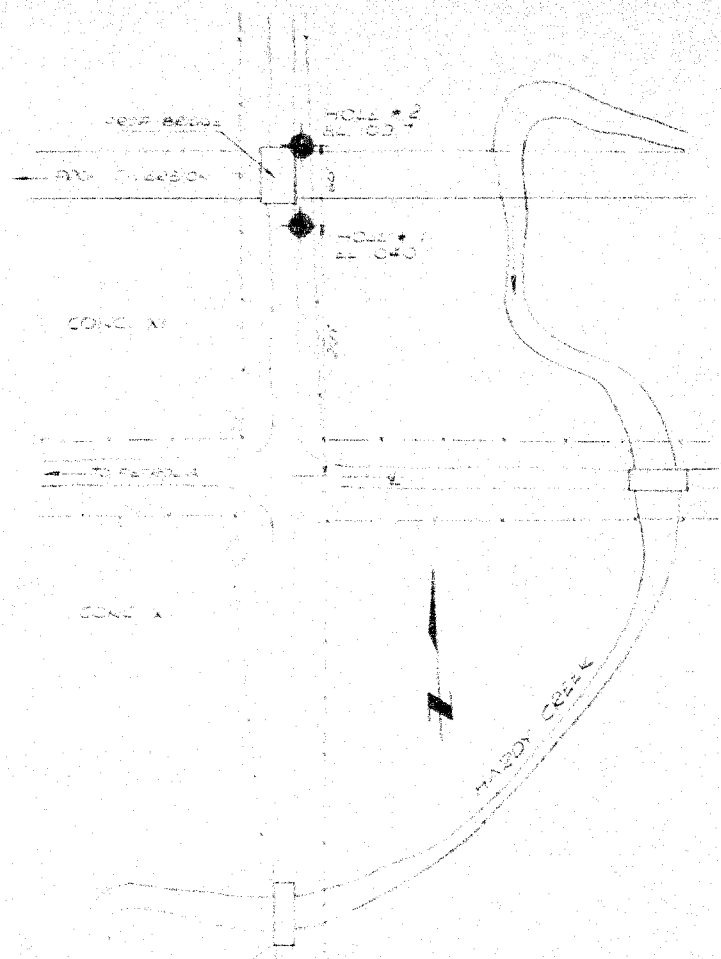
- V.T. IN SITU VANE SHEAR TEST
 M. MOIST
 W.L. WATER LEVEL IN CASING
 W.T. GROUND WATER TABLE IN SOIL
 W.T.P.L. WETTER THAN PLASTIC LIMIT
 D.T.P.L. DRIER THAN PLASTIC LIMIT
 A.P.L. ABOUT PLASTIC LIMIT

SOIL DESCRIPTION	COLOR	Consistency	Depth (feet)	Sample Type	Water Level & Remarks
Sandy silt, org. matter	Olive brown	Loose to	1.0	A.S.	Slightly moist.
Ditto	Ditto	Loose to	2.0	A.S.	Slightly moist.
Sandy clay, org. matter	Yellow brown	Compacted	2.5	A.S.	W.T.P.L.
Very silty clay, s. & p.	Mottled grey	Very stiff	3.5	A.S.	Much D.T.P.L.
Very silty clay, s. & p.	Grey	Stiff to very stiff	4.5	A.S.	About F.L.
Sand (8'6" - 9'6")			5.5	A.S.	
Silty clay, odd grit	Grey	Stiff to very stiff	6.5	A.S.	W.T.P.L.
Sandy silt	Grey	Compacted	7.5	A.S.	Saturated Water bearing seam
Silty clay, seams of silt	Grey	Stiff	8.5	A.S.	Started using wash
Clayey silt with layers of silty clay	Grey	Stiff	9.5	A.S.	W.T.P.L.
Sandy silt, clay content, grits, pebbles, fragments of shale	Dark grey	Stiff	10.5	A.S.	W.T.P.L.
Fine to coarse sand, gravel, clay fragments of shale	Dark grey	Stiff	11.5	A.S.	W.T.P.L.
Weathered shale	Dark grey	Stiff	12.5	A.S.	W.T.P.L.

DEFECTS IN NEGATIVE DUE TO
 CONDITION OF ORIGINAL DOCUMENT

Test hole terminated at

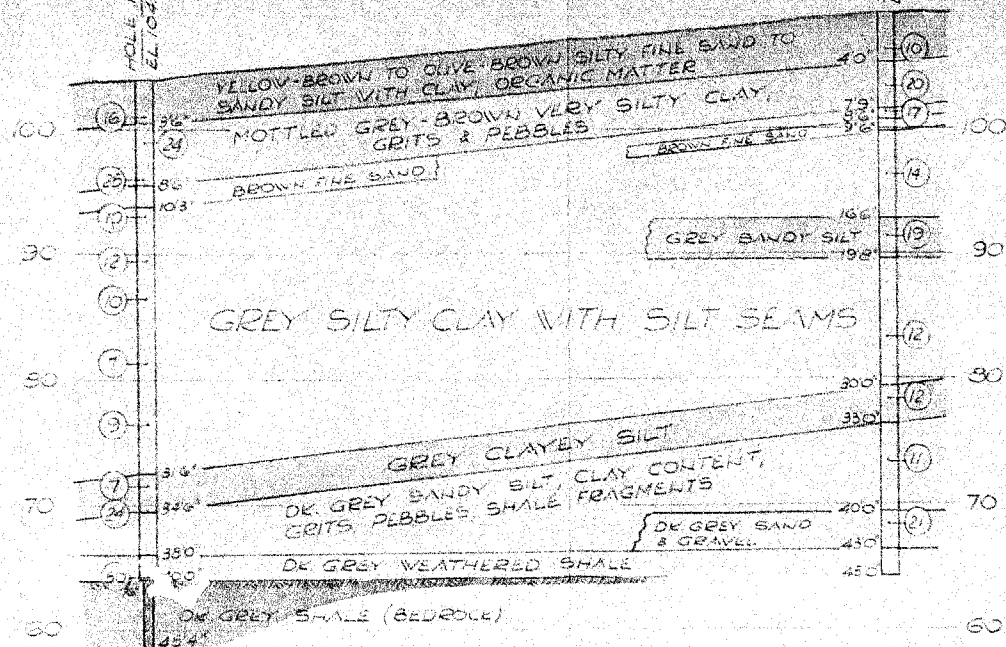
Water Sample No. 25



SITE PLAN

SCALE: 100' TO 1"

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT



SECTION THROUGH HOLES 1 & 2

SCALE 10' TO 1" (NATURAL)

LEGEND

- BOREHOLE
- BLOW'S/FOOT SPT

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.



TOWNSHIP OF BROOK

% J.A. MONTEITH ASSOCIATES LTD.

HARDY CREEK BRIDGE (#6)

PREPARED BY
e.m. peto associates ltd

JOB NO 63164	DATE DEC. 1963	DWN BY K.K.	CHECKED BY E.L.
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