

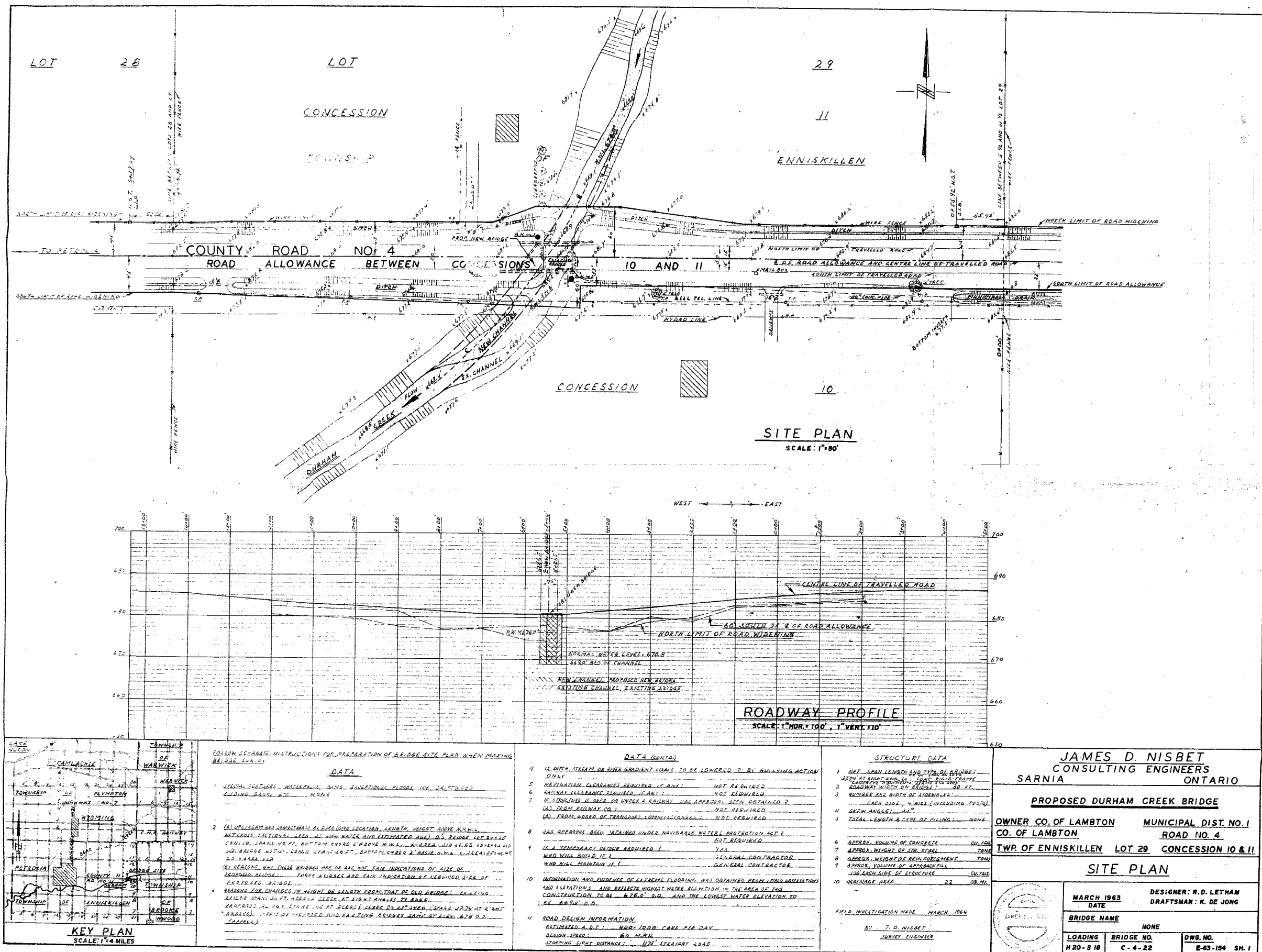
#63-F-247 M

DURHAM CREEK

BRIDGE C. 4.22.

LOT 29, CON. X / X1

ENNISKILLEN TWP.



MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

FROM: G. C. E. Burkhardt

DATE: March 25, 1964.

OUR FILE REF.


IN REPLY TO

SUBJECT: County of Lambton,
Durham Creek Bridge,
Twp. of Enniskillen,
Lot 29, Con. X/XI
Structure Site No. 15-144
Our File No. BA 1791

Attached please find one copy of the Foundation Report, by E. M. Peto Associates Limited, and one copy of the Preliminary Plans for your comments.

We would appreciate it very much if we could have your comments on or before April 3, 1964.

GCER/es


G. C. E. Burkhardt,
for K. L. Kleinsteinber,
Municipal Bridge Liaison Engineer.

Basically no comment. The graphical relationship presented on fig 8 seems to be an oversimplification. A safe net load of 20 T/sq ft should not create any problems.
By phone April 1, 1964. *AS Stermac*

B.A. 1791

F. M. PETO ASSOCIATES LTD.

Job No. 63182

1287 Caledonia Road,
Toronto 19, Ontario.
789-1126-7

November 22nd, 1963

The County of Lambton,
c/o James D. Nisbet,
206 Water Street,
Sarnia, Ontario.

63-1791-247.4

Attention: Mr. J. D. Nisbet

Re: Soil Investigation,
Bridge C. 4. 22

Gentlemen:

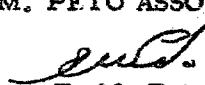
We have pleasure in forwarding to your four copies of our soil investigation report. Two copies have been sent directly to Mr. O. VanDeurs, the County Engineer.

We believe that the information contained in the following report is complete, and will permit you to proceed with the foundation design.

Should you, however, have some questions arising from this report, please do not hesitate to call on us.

Yours very truly,

F. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

BL:sb

THE COUNTY OF LAMBTON,

C/O JAMES D. NISBET,
CONSULTING ENGINEERS.

SOIL INVESTIGATION

BRIDGE C.4.22.

E. M. PETO ASSOCIATES LTD.

1287 Caledonia Road,
Toronto 19, Ontario.

SYNOPSIS

The reconstruction of the present bridge over the Bear Creek on County Road 4 (Bridge No. C. 4. 22) will involve widening and reuse of the existing steel joists. The existing abutments will be demolished, and replaced by a new wider structure, using the old superstructure.

The minimum recommended foundation depth is at least 4 feet below creek bottom which is at elevation 669.5. The soils underlying the foundation depth (elevation 665.5) are CI soils. Differentiation was made between individual clay strata at the depth investigated.

At the minimum proposed foundation depth (665.5) a spread footing design is possible and the net allowable bearing values for various footing width are given on Fig. 8.

The resulting settlements are thought to be of minor importance (maximum consolidation settlement of about 3 inches for 20 feet wide abutment and the net allowable bearing value of 1.8 ton/sq.ft), as the bridge is a simply supported structure and may safely take the calculated settlements.

No constructional problems are anticipated for a spread footing design.

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Synopsis.

Report.

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TABLE I Atterberg Limits

TABLE II Unconfined compressive tests and volumetric analyses

FIG. 1, 2 and 3 Grain size distribution curves

FIG. 4, 5 and 6 Consolidation test results

FIG. 7 Geotechnical soil properties

FIG. 8 Net allowable bearing values for various footing widths
and amounts of settlement.

FIG. 9 Time - Settlement relationship

BOREHOLE LOGS (2)

SITE PLAN AND PROFILE

A. INTRODUCTION:

We were authorized verbally on September 19th, 1963 by the Consulting Engineers to conduct a soil investigation at the present location of bridge No. C. 4. 22 on County Road 4, in the County of Lambton.

We understand that it is the intention to widen the present 30 feet wide bridge to about 50 feet. The present span of 36 feet will be maintained. It is proposed to remove the concrete deck and the handrails, and retrieve all steel joists. This will be then followed by demolition of the existing abutments. Following construction of new abutments, the deck will be widened by the addition of two new steel beams used in conjunction with the salvaged superstructure.

Although the bridge is of fairly recent origin, no information is available on the foundation depth.

The present soil investigation consisted of drilling two test holes at diagonally opposite corners of the existing bridge.

B. GENERAL INFORMATION:

1. The location of the test holes in relation to the existing bridge is shown on the attached site plan.

Based on the results of these test holes a simplified soil profile is shown on the drawing.

2. The elevations as given in this report refer to elevation 680.85 as denoted for the blue cross situated on the north-east abutment. (10 inches south of north face, and 9 inches east of west face of abutment).

3. The results of laboratory tests are given on Fig. 1 to 6, and Tables I & II.

4. The graphical representation of the natural moisture contents, Atterberg Limits, N-values (number of blows per foot penetration as obtained in Standard Penetration Test), and the undrained shear strength versus elevation is given on Fig. 7.

5. The results of the analyses interpreted as allowable bearing values and settlement are shown on Fig. 8 and 9.

C. SITE:

The present bridge is located on County Road 4 east of Petrolia. The existing grade on both sides of the bridge over the Bear Creek is fairly flat. The creek itself is located in a small valley, with the eastern banks higher and steeper than these on the west side. The maximum estimated height of the banks is about 9 to 10 feet above the creek water level.

At the time of this investigation County Road 4 on both sides of the existing bridge was being newly graded. Some fresh fill was added, especially near the existing bridge. The new fill was placed on the sides of the road thus widening the existing road.

D. SOIL CONDITIONS:

Underlying the surficial strata of fill and topsoil there were deposits of cohesive soils. Starting from the uppermost they were:

- i) mottled grey-brown very silty clay with grits and pebbles.
- ii) grey silty clay with grits and pebbles.
- iii) Grey silty clay
- iv) dark grey silty clay with grits and pebbles, and
- v) dark grey very silty clay with grits and pebbles.

The distribution of natural water contents, Atterberg Limits and N-values of these deposits with depth is shown on Fig. 7. A brief description of each soil type follows:

D. SOIL CONDITIONS: (Cont'd)

i) Surficial strata (fill, and topsoil)

Fill was met at test hole 1, which was located on the extended shoulder of the present roadway. It terminated at a depth of 7 ft 10 ins. below grade. The fill consisted of silty fine to fine sand, intermixed with some clay and minor organic matter. Present also were pebbles and stones. The fill was generally very loose to loose and moist.

At test hole 2, located away from the present fill section, the surficial layer consisted of organic sandy loam, 2 feet in thickness.

ii) Cohesive deposits

1. The first cohesive layer encountered was grey-brown very silty clay. This layer contained grits and pebbles. The lower limit of the desiccated grey-brown silty clay was established at 14.0 feet below grade at test hole 1 and at 10.0 feet below grade at test hole 2. The N-values varied between 14 and 20, decreasing with depth. Thus based on the N-values the stratum could be described as stiff to very stiff. The natural water contents were about 20%. The Atterberg Limits were:

Liquid Limit	41.9%
Plastic Limit	21.5%, and
Plasticity Index	20.4%.

Based on these results, the grey-brown silty clay layer was drier than the plastic limit. According to the Casagrande's classification system it is a CI clay.

D. SOIL CONDITIONS:

ii) Cohesive Deposits

1. (Cont'd)

The grading curve, which may be assumed to be typical for this deposit, is shown on Fig. 1.

2. Underlying the desiccated portion of the cohesive deposits there was a grey silty clay layer with grits and pebbles. The terminal depth of this layer was estimated to be at about elevation 660.

The N-values were between 12 and 17, thus the layer was stiff to very stiff. The natural water contents were much higher than for the overlying stratum, and approached a value of 26%. The Liquid Limit was 35%, and the Plastic Limit was 19%, the deposit was therefore about plastic limit at testhole 1, and wetter than plastic limit at test hole 2. Basically, this deposit is similar to the overlying stratum.

The other average values were:

Wet density $\gamma = 130.0$ lb/cu. ft.

Dry density $\gamma_d = 108.0$ lb/cu. ft.

Void ratio $e = 0.57$

From the result of a consolidation test (Fig. 4) it is seen that this deposit is only slightly pre-consolidated. The value of the Coefficient of Volume Change was

$$m_v = 0.017 \text{ sq. ft./ton.}$$

The coefficient of consolidation $c_v = 0.0095$ sq. in./min.
and the coefficient of permeability $k = 3.15 \times 10^{-7}$ in./min.

D. SOIL CONDITIONS:

ii) Cohesive Deposits

2. (Cont'd)

The undrained shear strength of this deposit, although over 2300 lb/sq. ft cannot be viewed separately from the general trend of the shear strength distribution with depth. This feature is discussed in paragraph 3, "Undrained Shear Strength".

3. Grey silty clay was located between about elevation 660 and elevation 647. This layer was practically free of grits and pebbles. The N-values were about 11 and generally constant throughout the depth of the deposit. The natural water contents exhibited a considerable scatter in values. Natural water contents as high as 35% was recorded on samples taken for the unconfined compressive tests, see Table II. The lower limit of the water contents was about 20 to 26%. This considerable scatter is due to the presence of silt seams in the matrix of the deposit.

The Atterberg Limits were:

Liquid Limit	34.4%
Plastic Limit	19.5%, and
Plasticity Index	14.9%.

D. SDIL CONDITIONS:

ii) Cohesive Deposits

3. (Cont'd)

Accordingly, the deposit was in all instances wetter than the plastic limit, and only in isolated cases approached the liquid limit. The Liquidity Index, however, as calculated for the sample taken from test hole 1, 25.8 feet below grade, was only 0.14. Thus the soil was only slightly wetter than its plastic limit at this depth.

The average densities for the deposit are:

Wet density $\gamma = 122.5 \text{ lb/uc. ft.}$
Dry density $\gamma_d = 96.0 \text{ lb/cu. ft.}$

Other average values which may be appropriated for this deposit are:

Void ratio	$e = 0.80$	
Coefficient of Volume change		$m_v = 0.015 \text{ sq. ft/ton}$
Coefficient of Consolidation		$c_v = 0.0117 \text{ sq. in/min.}$
Coefficient of Permeability		$k = 3.35 \times 10^{-7} \text{ in/min.}$

4. Underlying the grey silty clay layer was a dark grey silty clay deposit with grits and pebbles. Generally this deposit had similar average N-values of about 11, as the overlying layer and thus the same stiffness. The natural moisture contents were also in the same range as for the overlying deposit. The Atterberg Limits were also similar with a liquid limit of 37.7%, and a plastic limit of 21.7%, and plasticity index of 16.0%.

D. SOIL CONDITIONS:

ii) Cohesive Deposits

4. (Cont'd)

Other values are:

Wet density $\gamma = 125.3$ lb/cu ft.
Dry density $\gamma_d = 100.0$ lb/cu. ft.
Void ratio $e = 0.73$

Coefficient of Volume Change $m_v = 0.013$ sq. ft/ton
Coefficient of Consolidation $c_v = 0.0182$ sq. in/min.
Coefficient of Permeability $k = 5.15 \times 10^{-7}$ in/min

5. The last stratum at the depth investigated was a dark grey very silty clay. The N-values started to increase with depth, and the natural water contents to decrease. The average natural water content was about 19%. A typical grading curve of this material is shown on Fig. 3.

The average values, as determined in laboratory were:

Wet density $\gamma = 131.9$ lb/cu. ft.
Dry density $\gamma_d = 107.6$ lb/cu. ft.
Void ratio $e = 0.61$.

D. SOIL CONDITIONS: (Cont'd)

3. Undrained shear strength

The distribution of the undrained shear strength with depth is shown on Fig. 7; from this graph it is evident that the shear strength decreases from about 2600 lb/sq. ft at elevation 663 (i. e. in the grey silty clay layer with grits and pebbles) to a minimum of 1300 lb/sq. ft. at elevation 647. Thus a minimum shear strength occurs at about the interstratification of the grey silty clay and the dark grey silty clay with grits and pebbles.

From elevation 647 the undrained shear strength increases, and at the interstratification with the dark grey very silty clay reaches a value of 2000 lb/sq. ft.

In the dark grey very silty clay the shear strength is assumed to be constant with depth and the average value is 2690 lb/sq. ft.

E. WATER CONDITIONS:

Due to an insufficient period of observation no definite conclusions could be reached regarding the position of ground water table. During the drilling operations no free water was encountered in the upper 4 cohesive deposits, i. e. until the stratum of dark grey very silty clay was encountered. The water level as observed during one hour period of time after the completion of test hole 1 was constant at about 25.5 feet depth below grade.

F. OBSERVATIONS AND CONCLUSIONS:

1. Allowable Bearing Values

As the present foundation depth is unknown the foundation depth given is the minimum assumed to be compatible with frost and scour protection requirements. The elevation of the creek bottom was found to be at 669.5, thus allowing for a minimum of 4 feet of cover, the minimum recommended foundation elevation is 665.5.

Based on an assumed width of abutment of 50 feet, and the shear strength distribution as shown on Fig. 7, the allowable bearing values were calculated for various width using Skempton's formula.

The values so obtained give the allowable bearing values from shear strength consideration only, and include a factor of safety of 3 against shear failure.

Settlement analyses were made for various footing sizes and the applied net pressures. These calculations were made using the coefficients of Volume change values as given in the Soil Conditions. In addition a value of pore pressure parameter $A = 0.3$ was assumed (based on the paper by L. G. Soderman, T. C. Kenney, and A.K. Loh "Geotechnical Properties of Glacial Clays in Lake St. Clair Region of Ontario" for a silty clay deposit of similar characteristics). Consolidation settlement was then calculated, according to A. W. Skempton and L. Bjerrum "A Contribution to the Settlement Analysis of Foundation on Clay".

F. OBSERVATIONS AND CONCLUSIONS:

1. Allowable Bearing Values (Cont'd)

The results of these analyses are presented in Fig. 8. On this figure the net allowable bearing values are given for various footing widths and various amounts of consolidation settlement. Thus for a 4 feet wide abutment and 1.5 inches of consolidation settlement the allowable net increase in pressure above the existing overburden pressure is 2.35 ton/sq. ft. The maximum permissible net increase in pressure for a 4 feet wide abutment is 2.6 ton/sq. ft (as given by the shear strength consideration) but with a consolidation settlement of about 1.65 inches to be expected. Similarly for a 6 feet wide abutment the maximum permissible net increase in pressure is 2.4 ton/sq. ft, but the consolidation settlement will be in the order of 2.0 inches.

As mentioned above the values of allowable bearing values as given on Fig. 8 refer to the net allowable bearing values. The total allowable bearing value will be composed of the net allowable bearing value plus the minimum effective overburden pressure.

F. OBSERVATIONS AND CONCLUSIONS: (Cont'd)

2. Time Settlement Relationship

The time-settlement relationship was based on the average value of the coefficient of consolidation for all the cohesive layers. See Fig. 9. From this graph it may be seen that 50% of settlement expressed as degree of consolidation, will take place in about 3 years, and 80% in just over 8 years.

3. Alternative foundation scheme

A pile foundation may be considered as an alternative solution for the site. The piles will be of friction and end-bearing type; based on the shear strength profile shown on Fig. 7, and using the relationship between the undrained shear strength and the adhesion as established by M. L. Tomlinson ("Relationship of Observed Adhesion to Cohesive Strength of Clay" Proc. 4th Int. Conf. Soil Mechanics and Foundation Engr., London 1957), the following are the allowable loads per pile of unit cross section.

<u>Pile tip at elevation</u>	<u>Allowable Load per Pile (in kips)</u>
647	19.7
645	21.7
640	26.9
635	33.5

F. OBSERVATIONS AND CONCLUSIONS: (Cont'd)

4. Embankments

It is not anticipated that the height of the embankment will be higher than the present one, which is, according to the results of testhole 1 about 8 feet.

However, based on the available shear strength, an embankment with side slopes of 1 vertical to 2 horizontal, of about 34 feet in height may be placed. The settlement resulting from such an embankment was not calculated since it was not thought that such an embankment will be placed at the site.

5. Constructional Problems

Excavations in the cohesive deposits to a depth of about 19 feet may be made in an unsupported vertical cut. The surficial deposits of fill and topsoil may be either removed, or sloped back, at a slope of 1 vertical to 1.5 horizontal for the duration of construction.

No bottom heave of the excavations for spread footing design to the required foundation depth will occur.

No elaborate water control measures will be required at the site, apart, probably, from control of seepage water resulting from surface water or precipitation.

F. OBSERVATIONS AND CONCLUSIONS: (Cont'd)

6. Backfill

The use of a clean, granular fill, as backfill material, adequately compacted is recommended. The use of a granular blanket placed at the foundation depth is recommended. in order, to provide uniform bearing over the whole foundation area.

E. M. PETO ASSOCIATES LTD.,

C. F. Freeman

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

BL:sb

Job No. 63182

November, 1963.

Report Prepared By:

B. Lewicki

B. Lewicki, P. Eng.

TABLE I

ATTERBERG LIMITS

Test Hole	Depth	Elev.	in per cent			W	L.I.
			L.L.	P.L.	P.I.		
1	10'0"-11'6"	669.6	41.9	21.5	20.4	19.4	Negative
1	15'0"-16'6"	664.6	35.3	19.1	16.2	19.0	Ditto
1	25'0"-26'6"	654.6	34.4	19.5	14.9	21.6	0.14
1	35'0"-36'6"	644.6	37.7	21.7	16.0	25.4	0.23

$$LI = \frac{W - PL}{LL - PL}$$

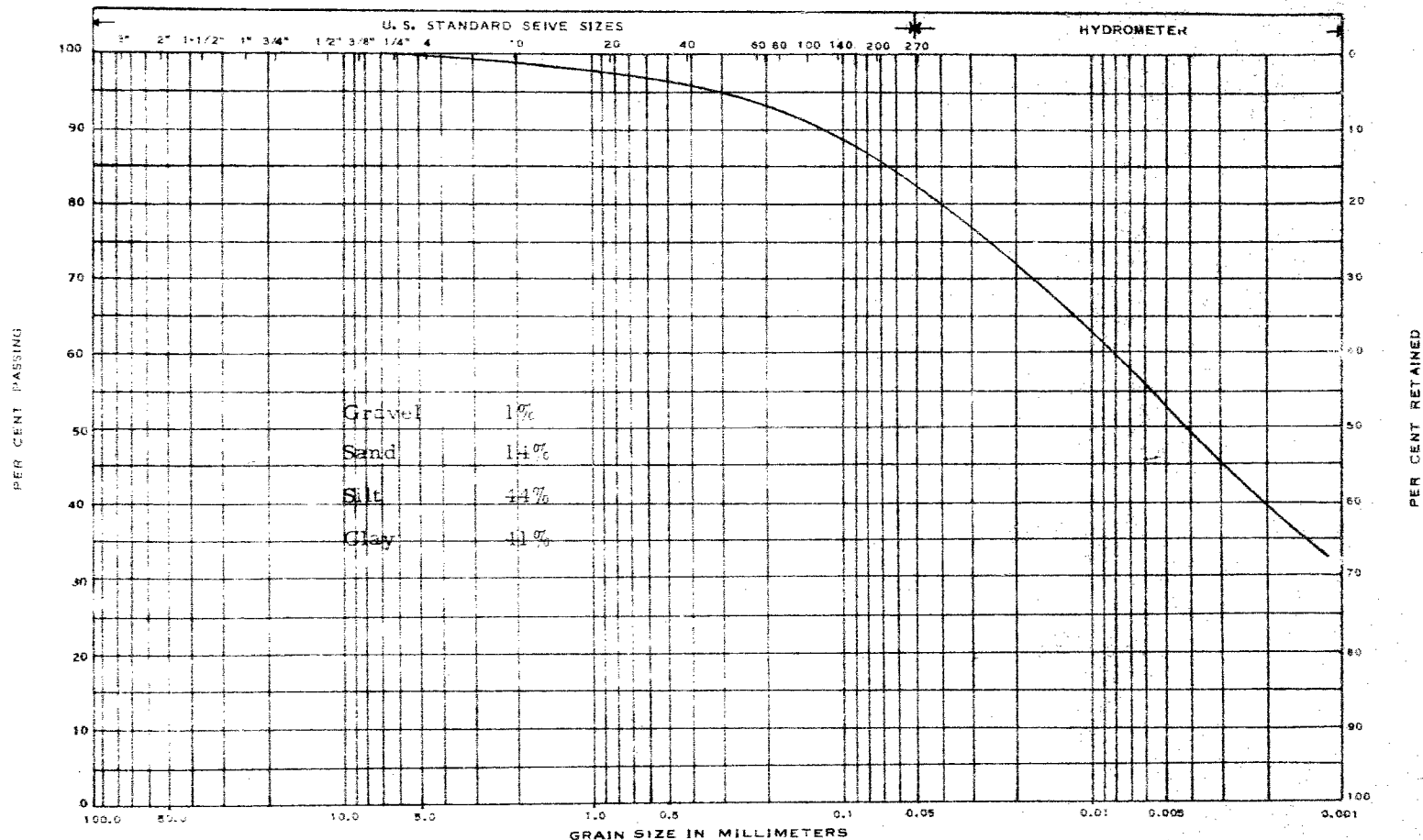
TABLE I I

UNCONFINED COMPRESSION TESTS AND VOLUMETRIC ANALYSES

Hole No.	Sample No.	Depth	Elevation	Nat. M. C. %	Densities, p. c. f.		Degree of Saturation, %	Void ratio, e	% Strain at Failure	u/c Shear Strength psf
					Wet	Dry				
1	9	17'6" - 18'0"	662.5	21.8	130.0	107.0	100.0	0.59	20	2600
1	10	18'0" - 18'6"	662.0	20.6	133.0	110.0	100.0	0.56	20	2300
1	12	22'6" - 23'0"	657.5	25.6	124.5	99.0	97.0	0.73	20	2300
1	13	23'0" - 23'6"	657.0	22.2	129.5	106.0	100.0	0.61	20	1950
1	15	27'6" - 28'0"	652.5	30.2	116.0	89.0	90.0	0.92	15	1700
1	16	28'0" - 28'6"	652.0	35.1	120.0	89.0	100.0	0.96	20	1700
1	19	33'0" - 33'6"	647.0	28.3	123.0	96.0	100.0	0.78	20	1300
1	21	37'0" - 38'6"	642.6	14.8	135.0	117.7	92.0	0.43	20	1510
2	28	32'6" - 33'0"	641.1	34.6	121.5	90.5	100.0	0.93	20	1860
2	29	33'0" - 33'6"	640.6	30.1	119.5	91.5	97.0	0.84	20	1620
2	31	35'6" - 36'0"	638.1	25.5	132.0	105.2	100.0	0.69	20	2780
2	32	36'0" - 36'6"	637.6	19.5	131.8	110.0	100.0	0.53	20	2600

e. m. peto associates ltd.

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 Bridge C. 4. 22 JOB NO. 63182 HOLE NO. 2 SAMPLE NO. 4

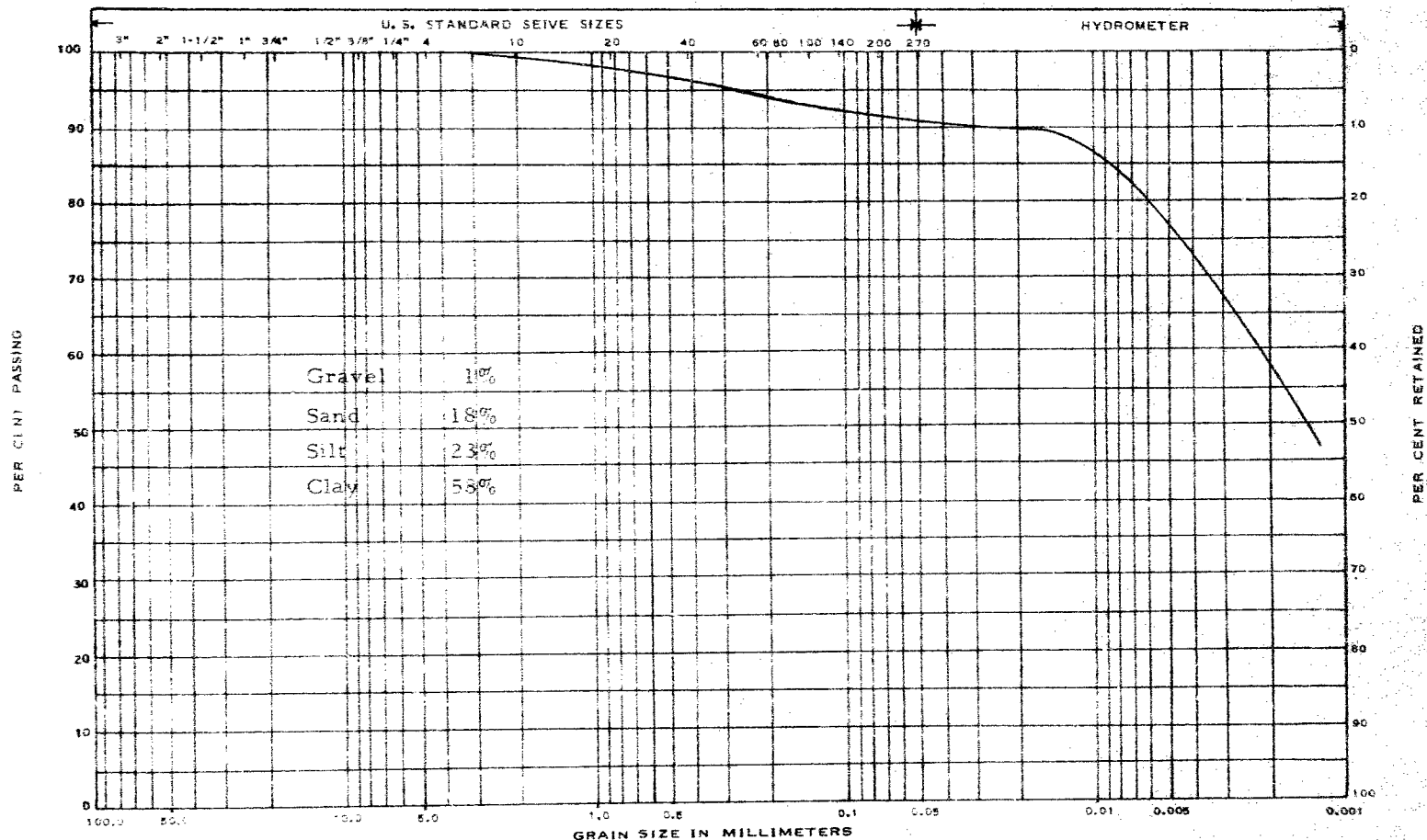
DEPTH 5'-6'6" ELEVATION 668.0 REMARKS Silty clay (silty clay till)

GRAIN SIZE DISTRIBUTION

Fig. # 1

e. m. peto associates ltd.
Toronto 19, Ontario

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PER CENT RETAINED

Fig. #2

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

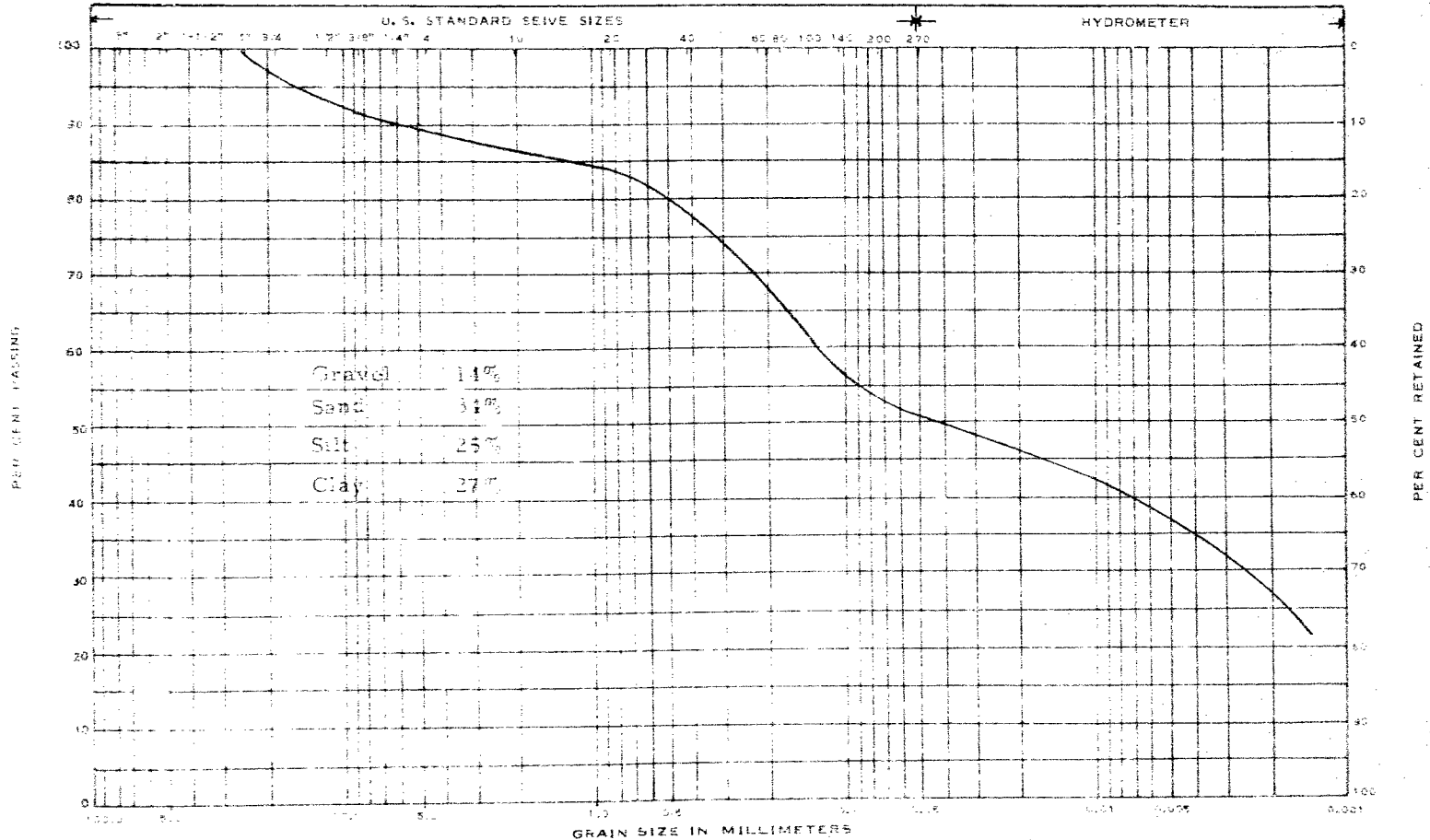
MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Bridge C. 4. 22 JOB NO. 63182 HOLE NO. 2 SAMPLE NO. 20
 DEPTH 20'-21'6" ELEVATION 653.0 REMARKS Clay (Clay till)

GRAIN SIZE DISTRIBUTION

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Toronto 19, Ontario

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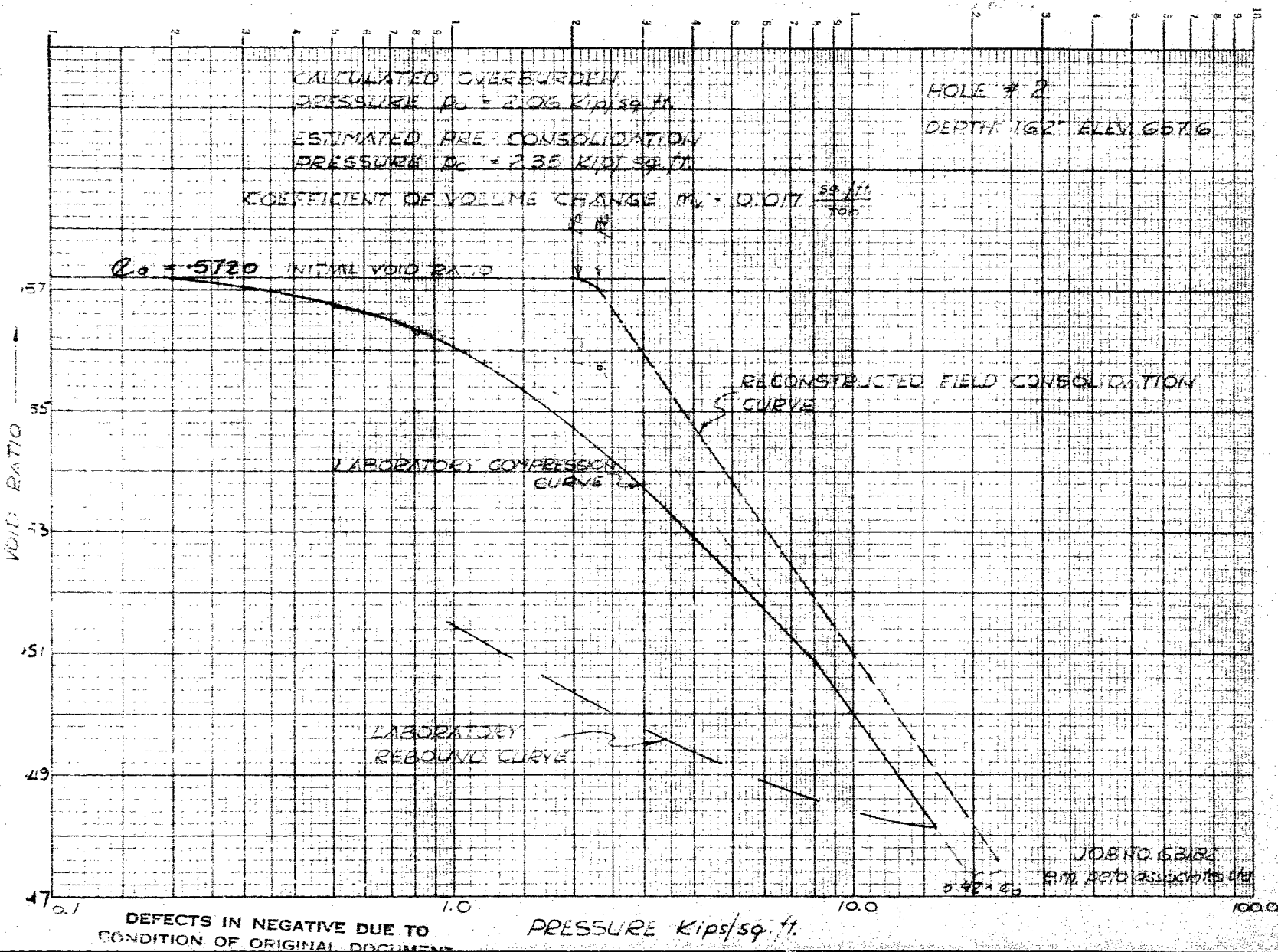
ST. 105	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
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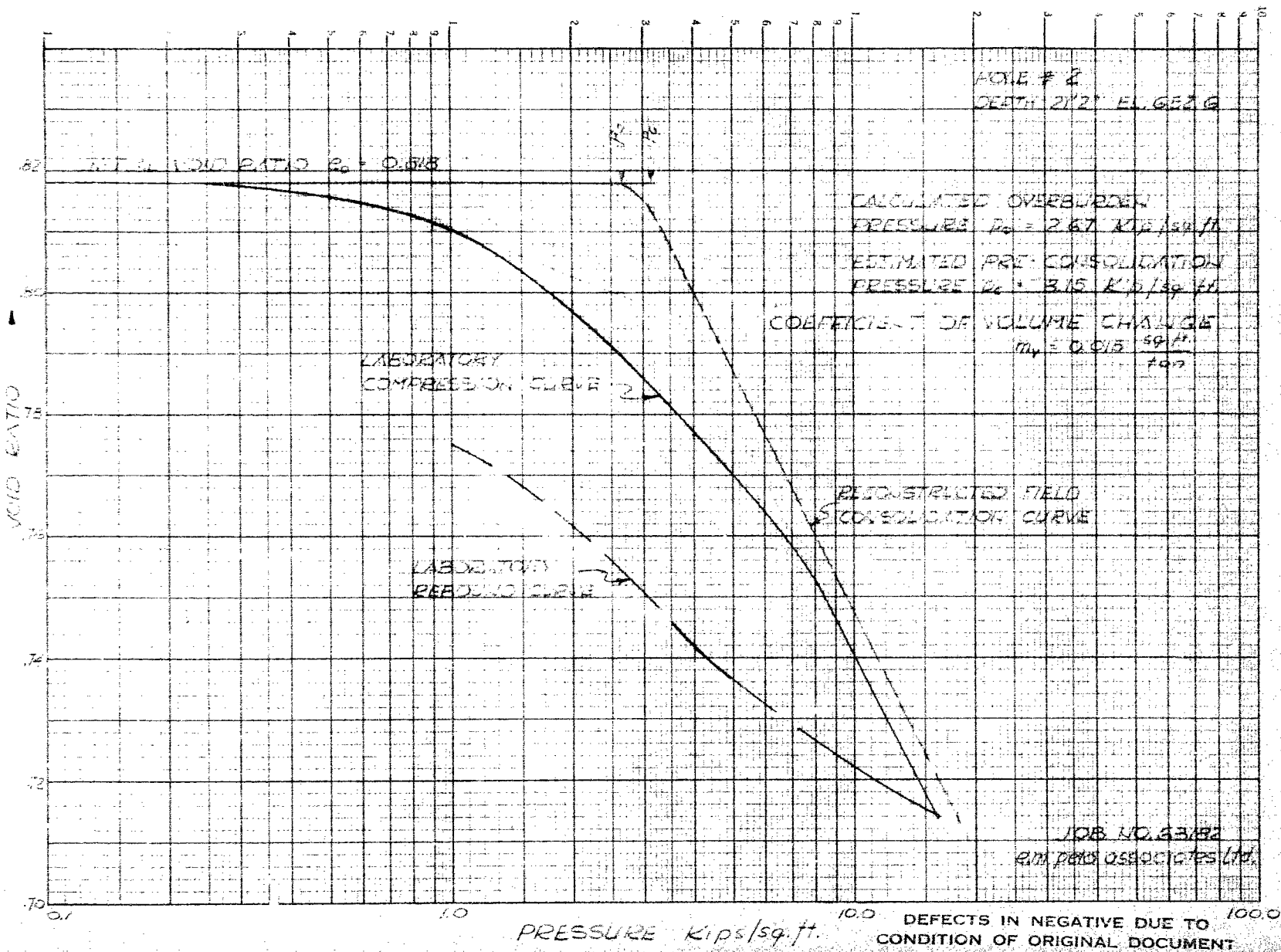
MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Bridge C. 4. 22 JOB NO. 63182 HOLE NO. 2 SAMPLE NO. 32
 DEPTH 37'-38'6" ELEVATION 636.0 REMARKS Clay till (clayey sand)

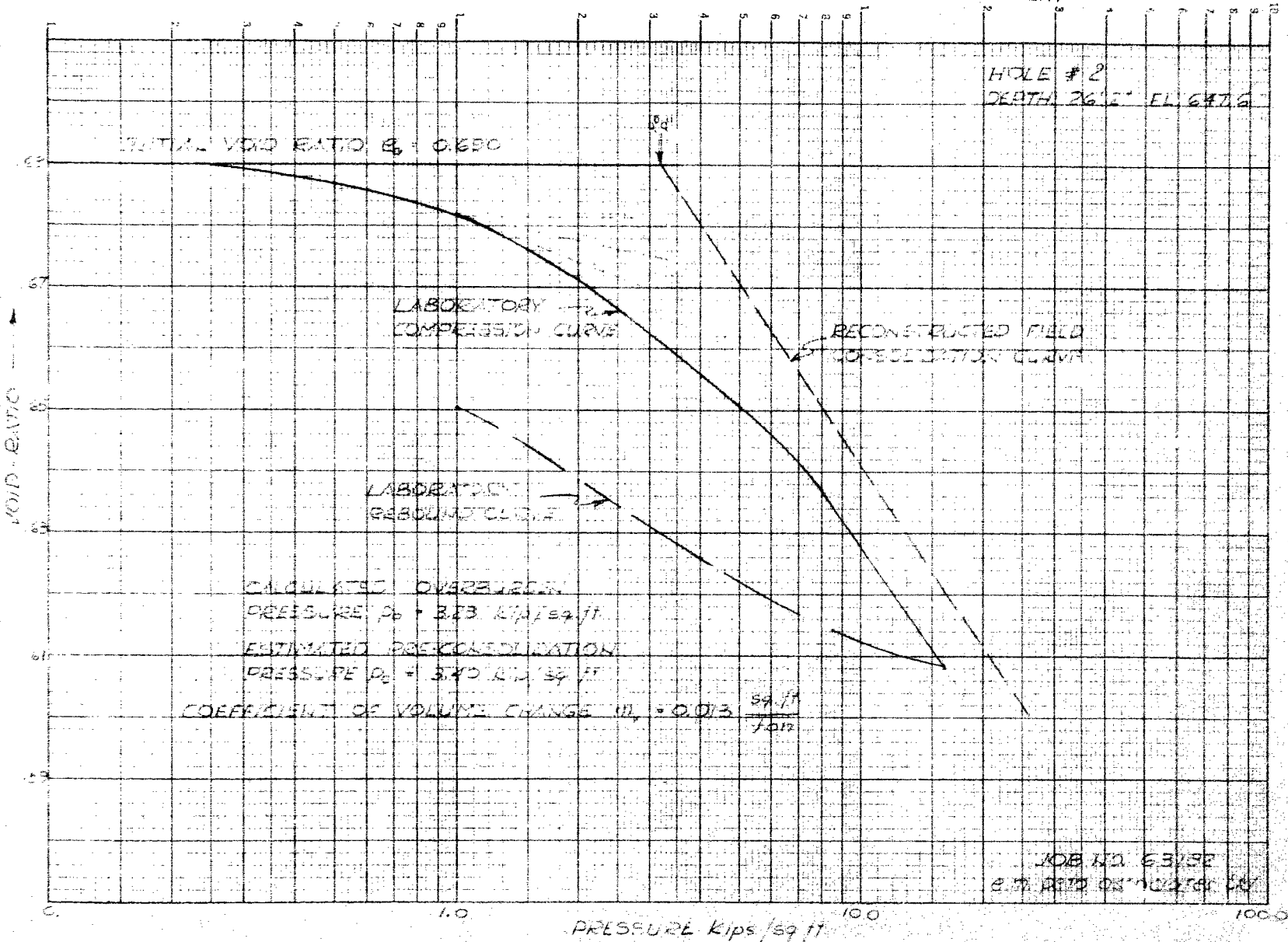
GRAIN SIZE DISTRIBUTION

Fig. 10

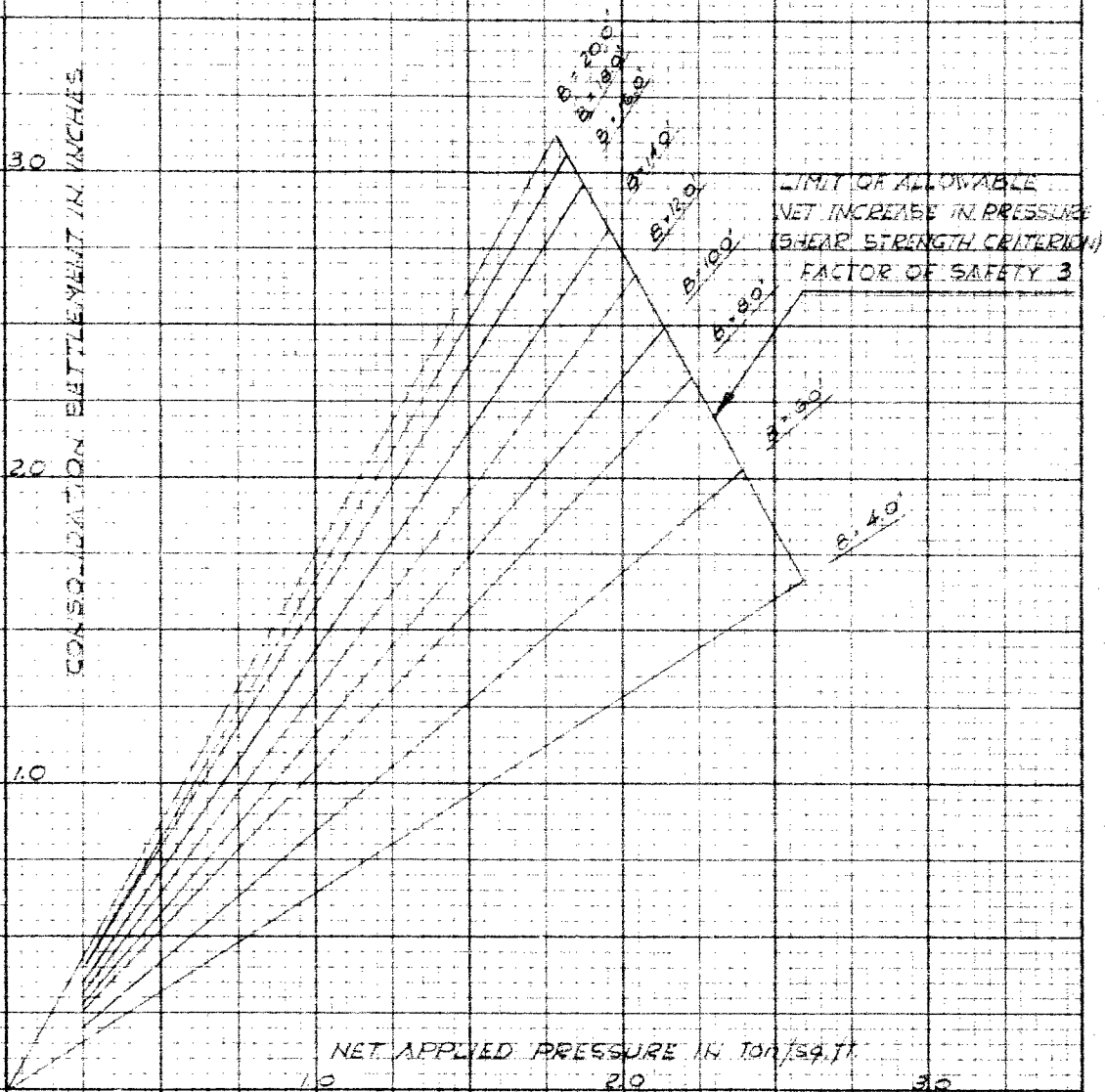




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NET ALLOWABLE BEARING VALUE FOR VARIOUS WIDTHS AND AMOUNTS OF SETTLEMENT




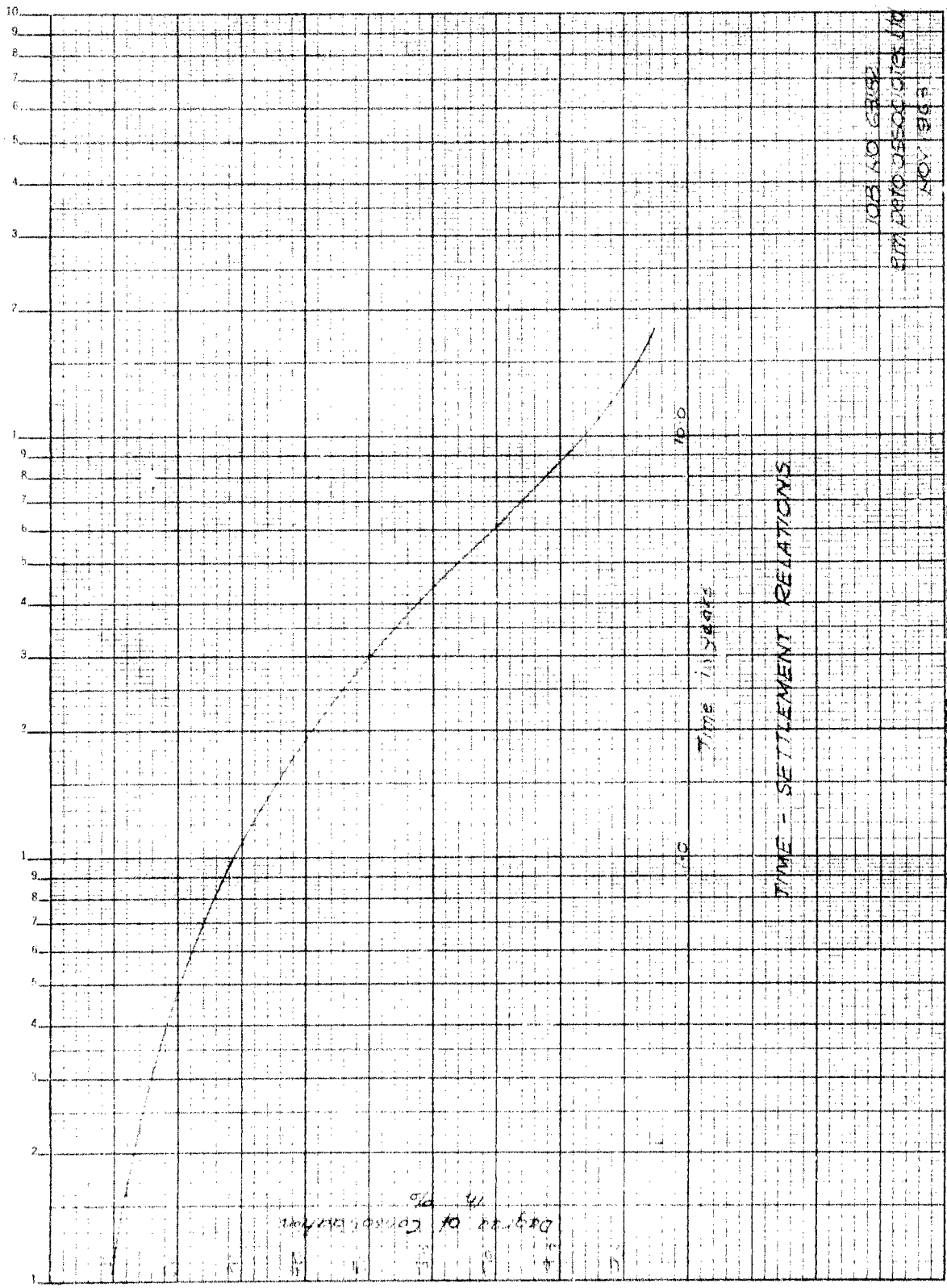
JOB NO. 6378

E.M. Peto Associates Ltd.

NOV. 1963

K.K.


SEMI-LOGARITHMIC 359-71
 KEUFFEL & ESSER CO. JERSEY CITY, N.J.
 10 CYCLES X 70 DIVISIONS



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 NOV 1968

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE

BOREHOLE LOG

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CONDITION OF ORIGINAL DOCUMENT

Job Name Bridge C-4.22

Job No. 63182

Borehole No.

Client The County of Lambton

Casing 4" & BX

Boring Date

Elevation 6803

Compiled By B. 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. BLOCK 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

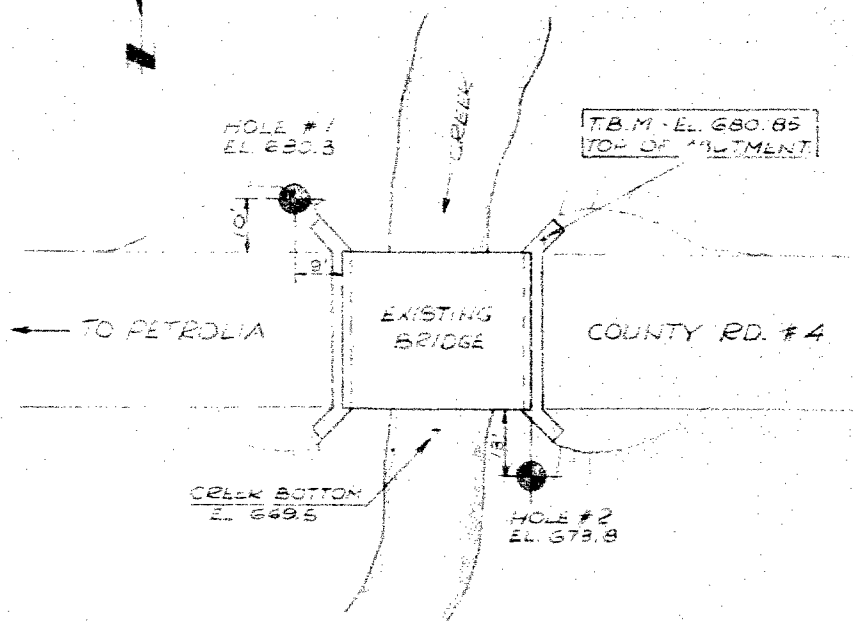
A.P.L. ABOUT PLASTIC LIMIT

[illegible]

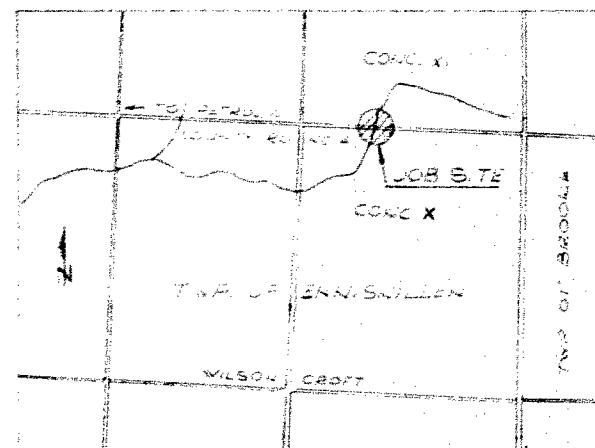
SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth / Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Unit Weight (pcf)	WATER LEVELS & REMARKS
			0'0"						
					1	CS			
Silty fine sand mixed very silty clay (fill)	Mixed brown	Very loose to loose			2	SS	4	8.6	Moist.
Fine sand, pebbles & stones	Ditto	Very loose	5'0"		3	SS	3	6.9	Moist.
Some org. matter (fill)									
Very silty clay, roots	Mottled grey brown	Firm to stiff	7'10"		4	SS	9	21.0	D.T.P.L.
			10'0"						
Ditto, grits & pebbles	Ditto	Stiff to very stiff			5	SS	17	19.4	Much D.T.P.L.
					6	2"SL		18.3	
Silty clay, grits odd pebbles	Grey		14'0"		7	2"SL			Slightly D.T.P.L.
					8	SS	17	19.0	About P.L.
					9	2"SL			
					10	2"SL			
			20'0"						
Silty clay	Grey	Stiff			11	SS	12	20.3	W.T.P.L.
					12	2"SL			
					13	2"SL			
			25'0"						
Ditto, odd pebbles, lamina of silt	Ditto	Ditto			14	SS	12	21.6	W.T.P.L.
					15	2"SL			
					16	2"SL			
			30'0"						
Silty clay	Ditto	Ditto			17	SS	13	26.3	W.T.P.L.
					18	2"SL			
					19	2"SL			
			35'0"						
Silty clay, grits, odd pebbles	Dk. grey	Ditto			20	SS	11	25.1	W.T.P.L.
					21	2"SL			
			40'0"						
Ditto	Ditto	Ditto	41'6"		22	SS	11	31.5	Much W.T.P.L.

Test Hole Terminated at 41' 6"

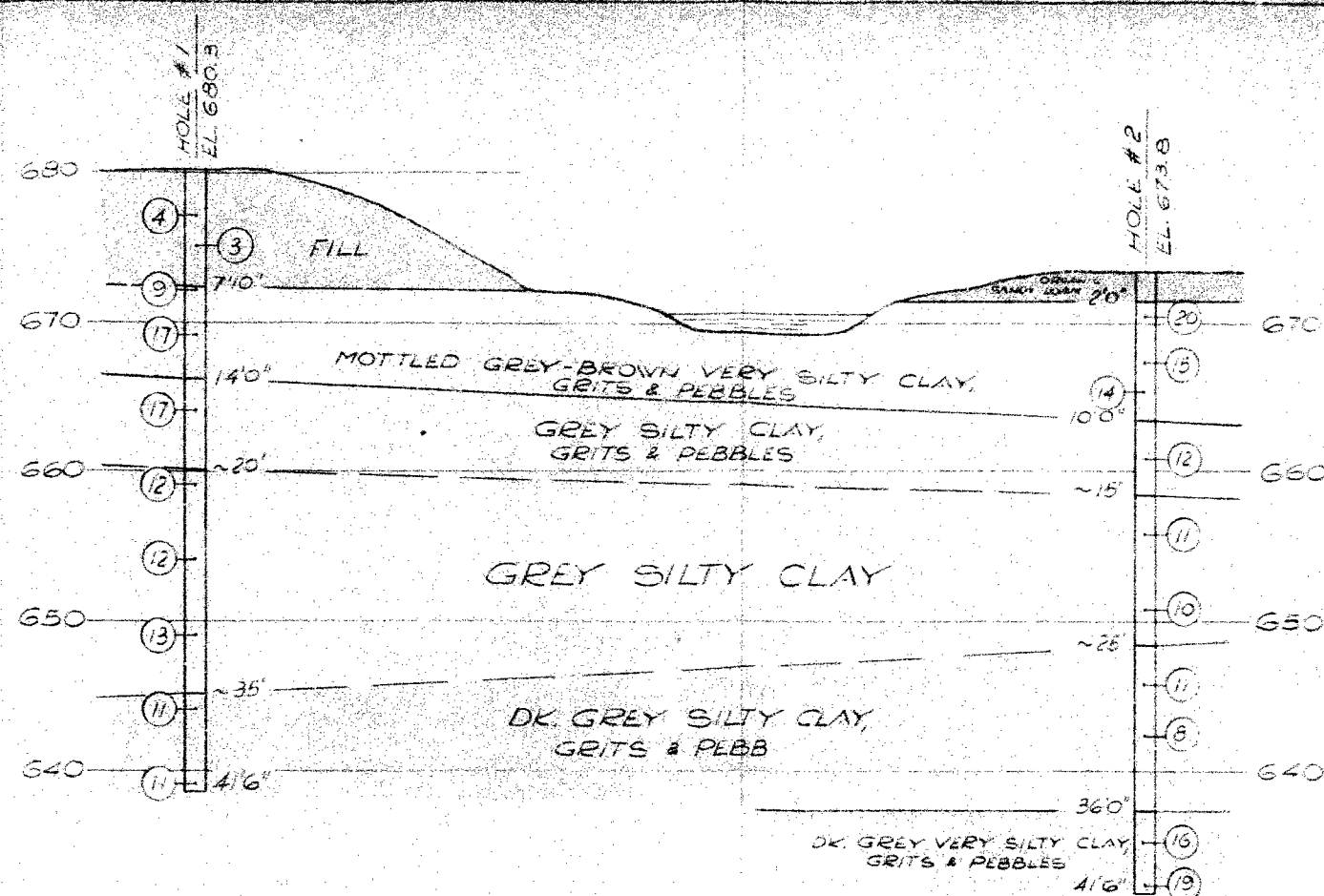
'Test Hole Terminated at 41'6"



SITE SKETCH
(NOT TO SCALE)



KEY PLAN
SCALE: 1/4" TO 1 MI.



SECTION THROUGH HOLES 1 & 2

HOR.: NOT TO SCALE

SCALES:

VERT.: 10' TO 1"

LEGEND

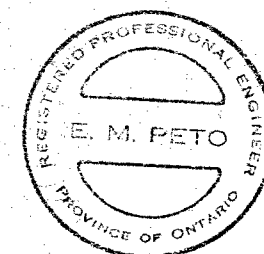
- BOREHOLE
- ⑬ BLOWS/FOOT S.P.T.

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.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT



THE COUNTY OF LAMBTON
% JAMES D. NISBET, CONS. ENGINEER

BRIDGE C. 4. 22 (#4)

PREPARED BY
e.m. peto associates ltd.

JOB NO.	DATE	DWN. BY	CHECK'D BY
63/82	NOV. 1963	K.K.	B.L.