

#60-F-265C

W.P. # 42-60

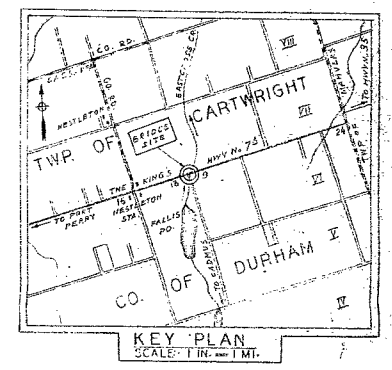
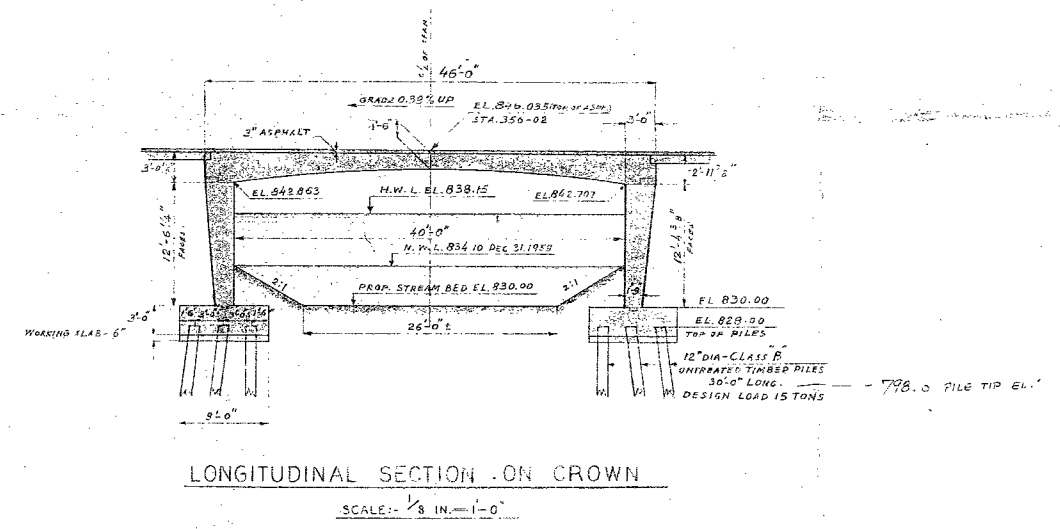
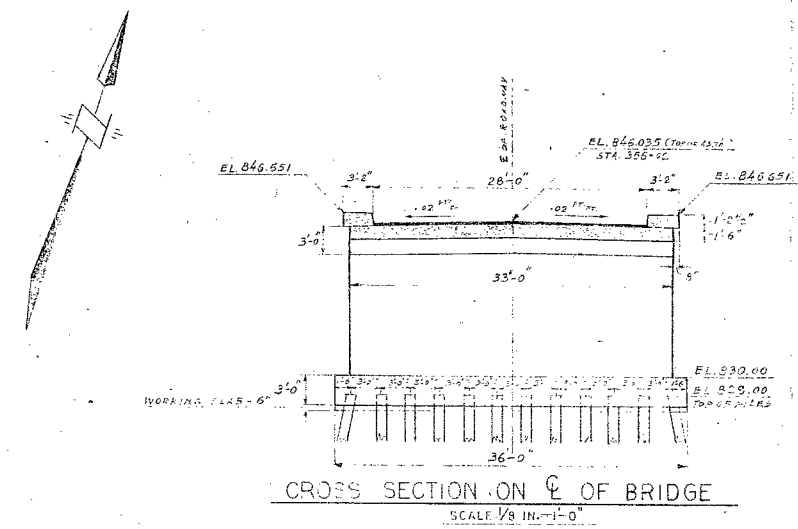
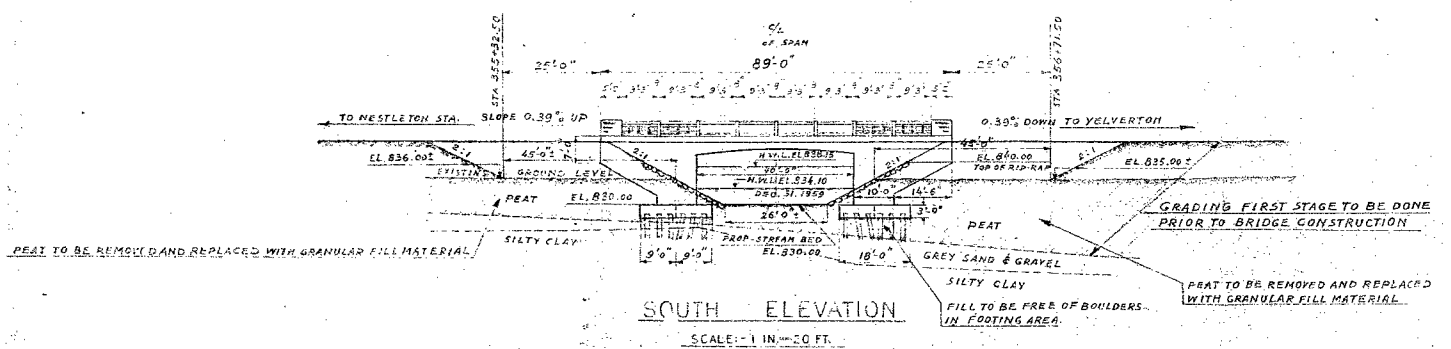
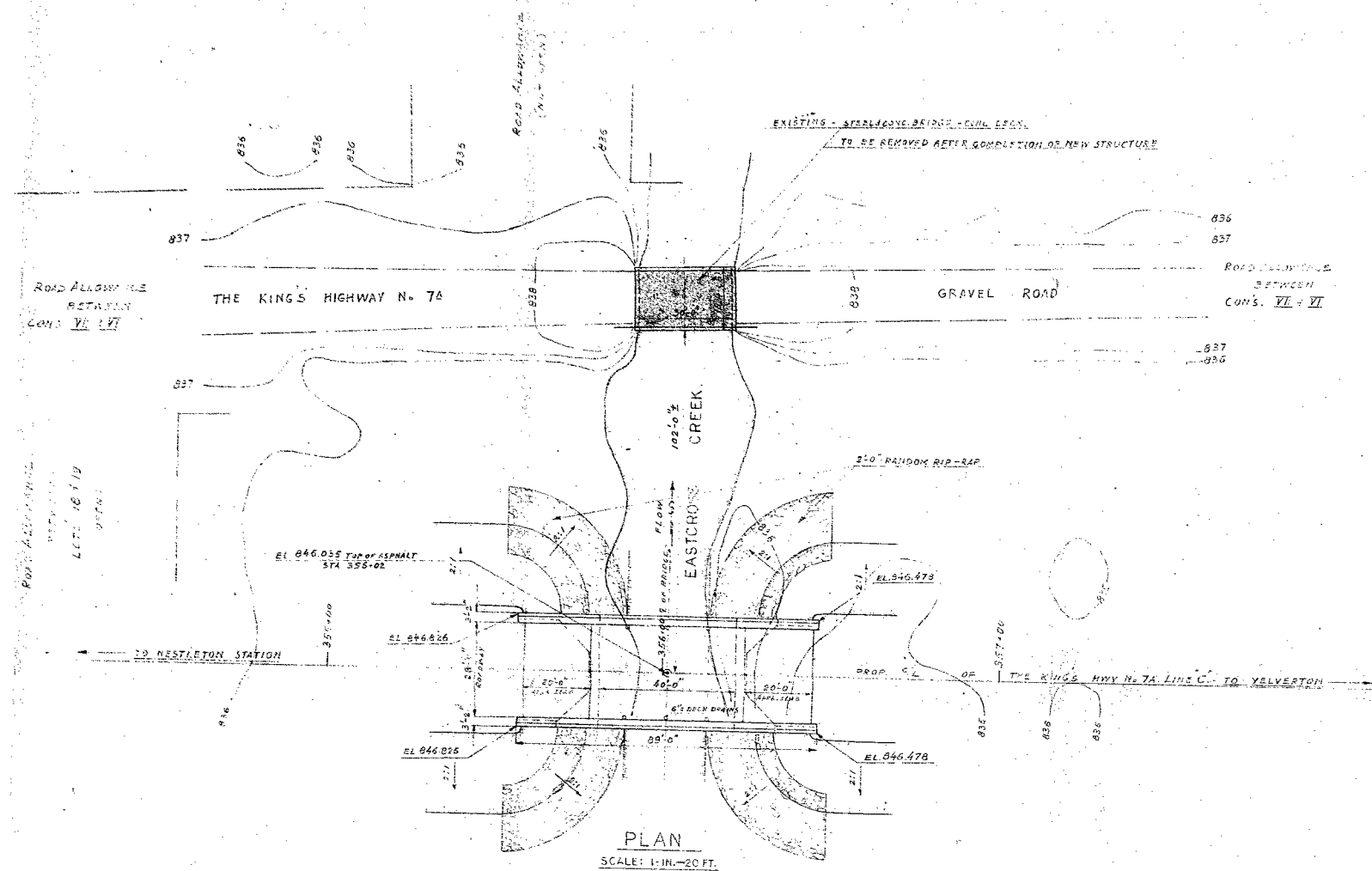
Hwy. # 7-A

EAST CROSS

CREEK BRIDGE







W.P. 42-60

DEPARTMENT OF HIGHWAYS-ONTARIO  
BRIDGE OFFICE-TORONTO

EASTCROSS CREEK BRIDGE  
(CADMUS CREEK)

THE KING'S HIGHWAY No. 7A DIST. No. 7  
CO. DURHAM  
TWP. CARTWRIGHT LOT. 19 CON. VI & VII

PRELIMINARY PLAN

APPROVED

BRIDGE ENGINEER DESIGN ENGINEER

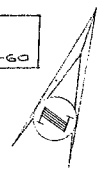
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DRAWING	CHECK	LOADING	NUMBERS
DATE	DATE	DATE	DATE

DATE: AUGUST 1960

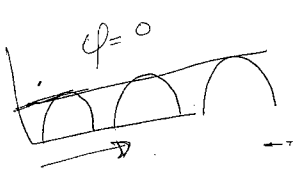
D-4541-P2



Δ V ELEV 828.6  
TOP OF SOUTH END OF CULVERT  
(CV: 253+27) SEE CLIENT'S PLAN W.P. 42-60



LOT 19  
CON. VII



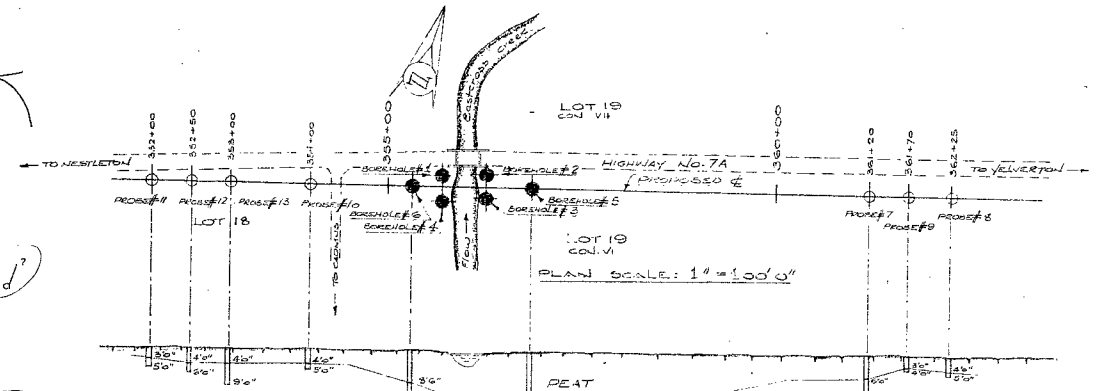
LOT 18  
CON. VI

TO NESTLETON HIGHWAY NO. 7A TO YELVERTON

LOT 18

PLAN SCALE: 1" = 20' 0"

LOT 19  
CON. VI

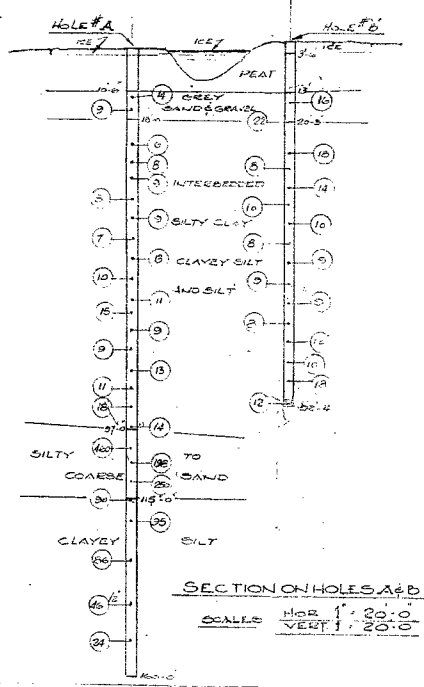


SOIL PROFILE ALONG PROPOSED 1  
SCALE: HOR. 1" = 100' 0"  
VERT. 1" = 20' 0"

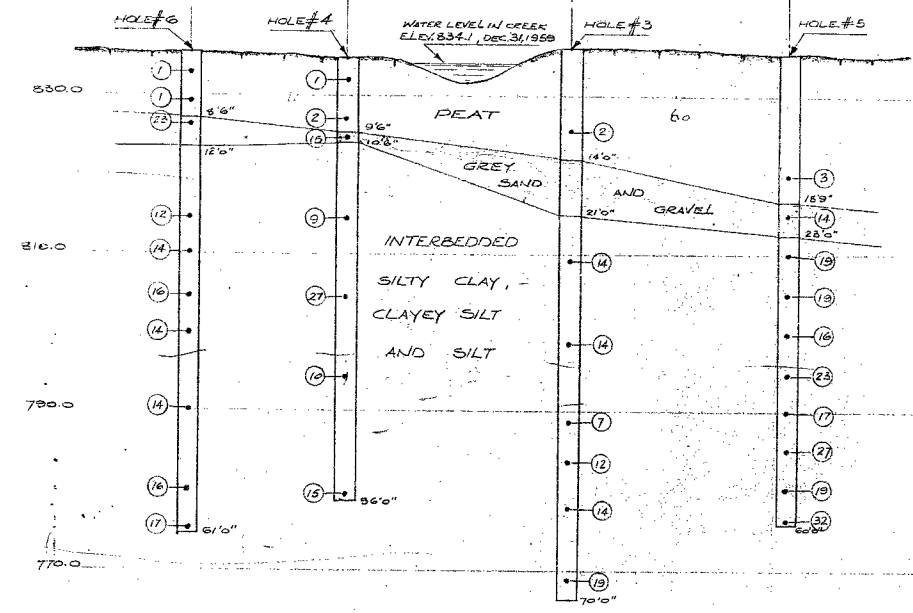
LEGEND

- BOREHOLE
- BLOKS/FOOT
- PROBEHOLE

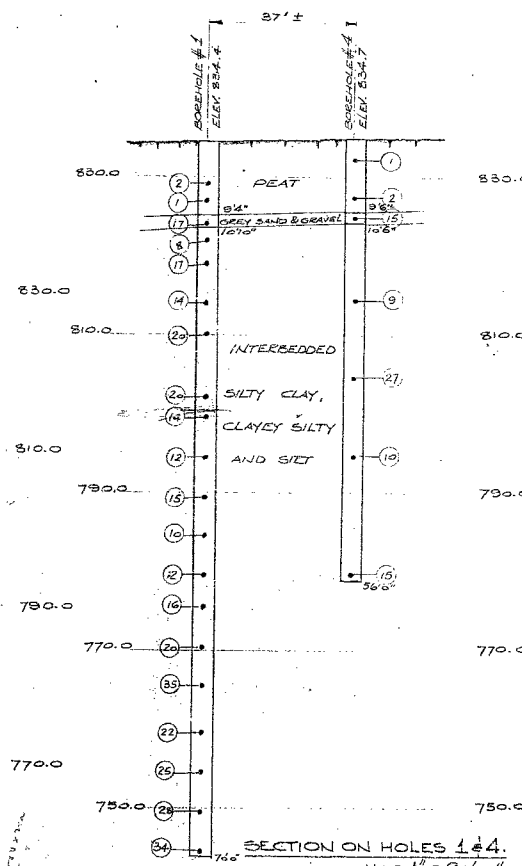
NOTE:  
SEE BOREHOLE LOGS  
FOR COMPLETE SOIL DATA



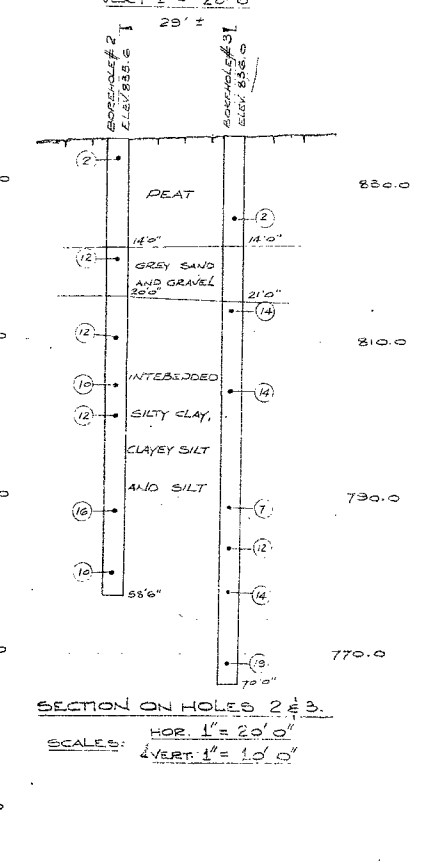
SECTION ON HOLES A & B  
SCALE: HOR. 1" = 20' 0"  
VERT. 1" = 20' 0"



SOIL PROFILE THROUGH HOLES 6, 4, 3 & 5  
SCALE: HOR. 1" = 20' 0"  
VERT. 1" = 10' 0"

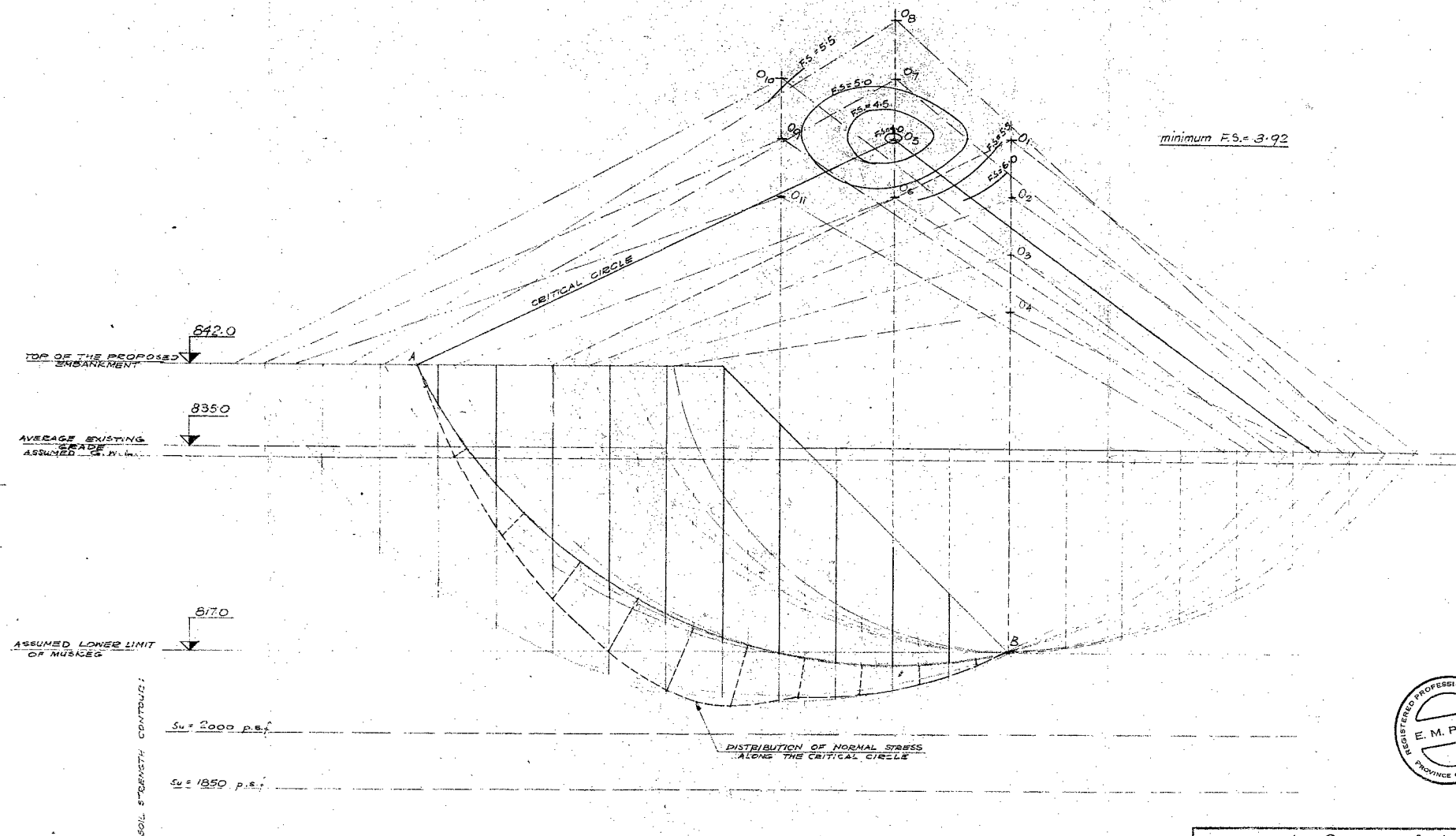


SECTION ON HOLES 1 & 4  
SCALE: HOR. 1" = 20' 0"  
VERT. 1" = 10' 0"



SECTION ON HOLES 2 & 3  
SCALE: HOR. 1" = 20' 0"  
VERT. 1" = 10' 0"

A	20/1/60	HOLES N & BARDED	C.J.V.
REVISION	DATE	REMARKS	BY
e.m. peto & associates Ltd.			
SOIL SITE INVESTIGATION			
AT			
EAST CROSS CREEK BRIDGE			
FOR			
DEPARTMENT OF HIGHWAYS OF ONTARIO			
OUR JOB No. 59249		DATE JAN. 11, 1960	
CLIENTS PLAN No. W.P. 42-60		PER G.T.T.	



SCALE:  
 HORIZONTAL: 1 inch = 6 feet  
 VERTICAL: 1 inch = 2000 p.s.f.  
 NORMAL STRESS: 1 inch = 2000 p.s.f.

# SLOPE STABILITY ANALYSIS



<b>e.m. peto &amp; associates Ltd.</b>	
SOIL SITE INVESTIGATION AT EAST-CROSS CREEK (SLOPE STABILITY) FOR DEPT. OF HIGHWAYS OF ONTARIO	
OUR JOB No. 58249	DATE 30/12/60
CLIENTS PLAN No.	PER. E. L.

Mr. A. M. Towe,  
Bridge Engineer.  
Materials & Research Section.

June 13, 1960.

FOUNDATION INVESTIGATION -- by  
E. M. Peto Associates, Ltd.

Attention: Mr. G. McCombie.

Re: East Cross Creek Bridge, Highway 7-A,  
Township of Cartwright, District No.7,  
W.P. 42-60.

---

We have reviewed the above mentioned report submitted by E. M. Peto and Associates, and have subsequently discussed, with the Consultants, a number of points and recommendations contained in the report. On the basis of these discussions and clarifications, we are herewith giving you the conclusions and recommendations which you should follow in your future design work:-

1. The stratigraphy of the investigated site can be considered to be fairly uniform. The main soil types, in succession, as they were encountered are:-

- a) peat.
- b) sand and gravel.
- c) interbedded silty clay, clayey silt and silt.
- d) silty to coarse sand.
- e) silt.

2. The water conditions were found, as follows:-

- a) a perched water table exists on top of the relatively impermeable layer of interbedded silty clay and clayey silt. The water table was about 1 ft. below ground level at the time of the investigation. Due to the presence of the creek which is known to carry much higher water levels than that encountered during the investigation, the adjacent area may be subjected to flooding.

cont'd. /2 ...

2. (cont'd.) ...

b) Artesian water was observed to exist at different depths in the stratum of interbedded silty clay, clayey silt, and silt, with the artesian head well above the present grade. Artesian water was also encountered in the layer of silty to coarse sand in Borehole 'A'. No artesian water was encountered to at least 40 ft. below ground level.

3. Because of the above mentioned ground and water conditions, it is recommended that the peat layer be excavated under the approach fill in accordance with Highway Standard DD-406. This excavation should also be carried out under and beyond the abutments and possible intermediate pier locations. The excavated material should be replaced with a fine-grained granular material at abutment and pier locations and the structure be founded on piles. Timber displacement piles driven to approximate elevation 800 and capable of carrying a safe load of 20 T/pile, are recommended. The elevation of the pile cap should be governed by frost and/or scour action criteria.
4. It is recommended that the excavation and replacement of the material be done first, and that a possible period of 6 months be allowed for the consolidation of this material to take place.

Upon completion of your preliminary design, we would appreciate reviewing the problem of excavation and replacement of the peat layer in the area of the structure.

If there are any other questions or problems you would like to discuss in the meantime, please feel free to call on our Office.

L. G. Soderman,  
PRINCIPAL FOUNDATIONS ENGINEER

Per:

AS/EGeF  
Attach.

cc: Messrs. A. M. Toye (2)  
H. A. Tregaskes  
D. G. Ramsay  
I. Campbell  
G. Wetherall  
T. J. Kovich  
A. Watt

  
(A. Stermac,  
FOUNDATIONS OFFICE ENGR.)

Foundations Office  
Gen. Files.

YOUR REFERENCE:-

OUR REFERENCE:-

**59249**

**1287 caledonia road,  
TORONTO 19, ONTARIO.  
RUssell 9-1126**

**8th June, 1960.**

**Department of Highways of Ontario,  
Materials and Research Section,  
Parliament Buildings,  
Toronto 2, Ontario.**

**Attention: Mr. A. Rutka, P. Eng.**

**re: Foundation Investigation Report  
East Cross River, Highway 7A  
W.P. 42-60, Port Hope, Ontario  
Our reference No. 59249**

**Dear Sirs:**

**In reply to your letter, dated the 11th May, 1960, regarding the data presented in the foundation report on the above site. After a careful review of the data presented in this report, we have come to the following conclusions:**

**1. Field Work**

**The main purpose of the two additional test holes was to determine the consolidation characteristics of the stratum of interbedded silty clay, clayey silt and silt. During the original investigation it was found that ordinary sampling methods and handling produced considerable sample disturbance. For this reason, additional care was exercised in obtaining and handling the samples during the subsequent investigation. The other purpose of the additional investigation was to find, if possible, a dense bearing stratum for an end-bearing type of pile. The presence of such a layer was positively established at test hole A only after considerable difficulty was experienced in sinking this test hole, due to Artesian water conditions. Similar, although worse Artesian conditions, were encountered at hole B and it was thought best not to aggravate this situation by releasing the Artesian pressure at this test hole, for this reason test hole B was terminated at a depth of 92'4".**

*3. Summary  
of test  
results  
see p. 341  
of report  
attached  
in envelope  
with this  
letter  
is complete*

## II. Strength Tests and Allowable Bearing Values

The shear strength determinations were carried out during the original (1st part) investigation. The type tests employed in arriving at the shear strength characteristic of the soil were

- (a) Unconfined compressive tests
- (b) Field vane tests
- (c) Laboratory vane tests
- (d) Triaxial shear tests

As mentioned in our report (Part I, page 7) the laboratory vane test results had to be discarded due to considerable influence of sample disturbance. They were included only for the purpose of illustrating the influence of sample disturbance on the results of shear strength.

The triaxial shear tests, although designated as UU-tests in our report (Part I, page 8) actually were allowed to consolidate for 24 hours in the laboratory before the samples were sheared. The rough estimation indicates that about 90% of consolidation did take place before vertical load was applied. This apparent discrepancy between the reported procedure and the actual was discussed between our Messrs. Lewicki and Freeman and your Messrs. L. G. Soderman and A. Stermac, and although we realise that in a strict sense the test does not represent a CU test; at the same time, the values of shear strength as calculated from the shear parameters as obtained from these tests show fairly good agreement, having in mind the type of soil stratum under consideration, between the shear values as obtained from other tests. (See graph of shear strength versus elevation Appendix III).

The field vane tests were carried out in test holes 2 and 3. Apart from the three initial test results of test hole 3 which were conducted in the layer of peat (from 3'0" to 13'6") the remainder of the tests were carried out in the layer of interbedded silty clay, clayey silt and silt.

The vane test gives only the value of the undrained shear strength and thus, the material best suited to be tested should have very low permeability so as not to allow any dissipation of pore pressure during the test. The vane strength values were found to be reliable when applied to the problems, using  $\phi = 0$  method of stability analysis.

In the "Symposium on Vane Shear Testing of Soils" A. S. T. M. Special Technical Publication No. 193 - in a paper by W. C. Hill "Vane In-Place Soil Shear Device Developed and Applied by Oregon State Highway Department" on page 37 of the above publication, a table of comparative soil shear values from vane shear test and direct shear test are given.



The material tested had a value of  $\phi$  varying from  $4.5^\circ$  to  $37.0^\circ$  with a "unit cohesion" varying from 0 to 2.5 p.s.f. According to these results, fairly good agreement was obtained for various types of soil between the shear values as calculated from the direct shear and the field vane values.

In the case under the consideration, i.e. the layer of inter-bedded silty clay, clayey silt and silt, it was found that the material had a low permeability and for practical reasons, it may be assumed that there will be no dissipation of pore pressures, thus the vane shear test was considered to give quite reliable values.

On the graph of shear strength versus elevation (Part I, Appendix III) an attempt was made to find the best shear strength distribution between the shear values as obtained from various tests. A line was drawn indicating the assumed average undrained shear strength distribution with depth. This curve gives the following  $S_u$  values (as given in the report)

Elevation	815 - 2300 p.s.f.
	810 - 2100 p.s.f.
	805 - 1850 p.s.f.
	800 - 1600 p.s.f.
	795 - 1350 p.s.f.
	792 - 1200 p.s.f.
	788 - 1200 p.s.f.
	780 - 2000 p.s.f.

As may be seen, the assumed average curve does follow the trend of the shear strength results as obtained from various tests, and for this reason it is considered that it does fairly represent the shear profile of the subsoil.

The shape of the shear strength profile indicates a normally loaded deposit with the stiffer upper crust reaching down to elevation 792.

#### Revised Allowable Bearing Values for continuous footings

The allowable bearing values are calculated using the following equation:

$$q_a = \frac{q_u N_c}{F.S.}$$

where  $S_u$  = undrained shear strength. This was taken to be the average over a depth equivalent to 2/3 of the footing width from the shear strength profile as given in appendix III, Part I, with a maximum undrained shear strength above elevation 815 of 2300 p.s.f.

$N_c$  - as obtained from the graph of  $N_c$  vs  $D/B$  according to Skempton's empirical relation

$$N_c = 5 \left( 1 + \frac{D}{5B} \right) \left( 1 + \frac{B}{5L} \right)$$

$D$  - the foundation depth

The foundation depth was assumed to be equal to the height of material overlying the foundation elevation of the same unit density as the material underlying the foundation elevation. In the case of the conditions encountered at this site in the area of the proposed East abutment, i.e. area of test holes B, 2 and 3 - the lower limit of the peat layer was assumed to be at elevation 822, i.e. 14.3 feet below the existing grade, having a unit density of 65.5 lbs. per cu. ft.

The equivalent height of material, having density of 126 lbs. per cu. ft. (as taken for the layer of interbedded silty clay, clayey silt and silt) is then

$$14.3 \times 65.5 = \text{equivalent height} \times 126$$

$$\text{equivalent height} = \frac{14.3 \times 65.5}{126} = 7.4 \text{ feet}$$

which at the same time represents the foundation depth at elevation 822.0.

Thus the following table may be set out:

#### East Abutment

Proposed found Elevation	Found Depth (D) (ft.)	Found Width B (ft.)	D/B	$N_c$	Average $S_u$ p.s.f.	$q_a$ p.s.f.
822	7.4	5	1.48	6.47	2300-	4960
		10	0.74	5.73	2300	4390
		15	0.49	5.49	2250	4120
		20	0.37	5.37	2150	3860

East Abutment - Cont'd.

Proposed found. Elevation	Found. Depth D (ft.)	Found. Width B (ft.)	D/B	$N_c$	Average $S_u$ p. s. f.	$q_a$ p. s. f.
820	9.4	5	1.88	6.87	2300	5270
		10	0.94	5.93	2300	4550
		15	0.63	5.62	2200	4120
		20	0.47	5.46	2100	3830

West Abutment:

The foundation depth was found to be 6.1 feet for the proposed foundation elevation at 824 thus the following table of allowable bearing values may be given:

Foundation Elevation	Found. Depth D (ft.)	Found Width B (ft.)	D/B	$N_c$	Average $S_u$ p. s. f.	$q_a$ p. s. f.
824	6.1	5	1.22	6.22	2300	4770
		10	0.61	5.51	2300	4300
		15	0.41	5.41	2300	4150
		20	0.30	5.30	2200	3890
822	8.1	5	1.62	6.61	2300	5070
		10	0.81	5.81	2300	4460
		15	0.54	5.54	2250	4080
		20	0.41	5.41	2150	3830

As may be seen for the larger footing sizes, the allowable bearing values decrease. This decrease is firstly due to the decrease in the ratio of D/B and, consequently in the  $N_c$  - value, and secondly because of the decrease in the average undrained shear strength over the range of depth considered, which was equivalent to two thirds of the width of a continuous footing.

The detailed calculation therefore, will be as follows (East Abutment, Foundation Elevation 820)

Continuous footing  $B = 5.0$  ft.  
 Foundation depth  $D = 9.4$  ft.  
 Therefore  $D/B = 1.88$   
 and from the graph, the corresponding  
 $N_c$  - value is 6.87

The average  $S_u$ :  
(taken from the assumed average shear strength profile)

at elevation 820 is 2300 p.s.f.

$$\text{at elevation } 820 - \frac{5 \times 2}{3} = 817.7$$

and at elevation 817.7 the average undrained shear strength is 2300 p.s.f.

Allowable bearing value (F.S. = 3)

$$q_a = \frac{6.87 \times 2300}{3} = 5270 \text{ p.s.f.}$$

For continuous footing  $B = 20$  feet

$D/B = 0.47$  and the corresponding  
 $N_c$  value = 5.46

The average  $S_u$  at elevation 820 is 2300 p.s.f.

The average  $S_u$  at elevation  $(820 - \frac{20 \times 2}{3})$

$$= 806.7 \text{ where } S_u = 1900 \text{ p.s.f.}$$

Therefore average  $S_u = 2100$  p.s.f.

The allowable bearing value  $q_a = \frac{5.46 \times 2100}{3} = 3830$  p.s.f.

Thus, as may be seen for the 20 feet wide continuous footing the allowable bearing value is 3830 p.s.f. as compared with 5270 p.s.f. for 5 feet wide continuous footing.

The allowable bearing values as calculated above represent the net bearing value i. e. they do not take into account the effect of existing overburden pressure.

#### Pile foundation

The total bearing capacity of friction piles is the sum of the 'friction' capacity and the point bearing capacity.

$$\text{Then } Q = Q_f + Q_p$$

where  $Q_f$  - friction capacity

$Q_p$  - point bearing capacity

### Pile foundation - Cont'd.

The frictional component of bearing capacity is calculated according to equation:

$$Q_f = A_s \times S_a$$

where  $A_s$  - the surface area of the pile  
 $S_a$  - cohesive strength of the soil

(See "Use of Strength Data in Pile and Spread Footing Design" by Roberts M. I. T. - Summer Course, 1959).

The end bearing capacity portion of the total bearing value is calculated as follows:

$$Q_p = A_p \times N_c \times S_u$$

where  $A_p$  - cross sectional area of the pile  
 $N_c$  - 9.0  
 $S_u$  - undrained shear strength of the soil

The triaxial shear tests gave a minimum value for the cohesion of 430 p. s. f. This value was recommended as the cohesive strength of the soil for use in the equation for the frictional component of the total bearing value of the friction piles.

For piles acting in a group (where piles are spaced less than 4 times the diameter of the pile) the ultimate bearing capacity of the pile group, as a whole should be considered, in the light of the load apportioned to each pile when it is acting as one of a group, compared with the load bearing capacity of a pile acting as a single unit. The value in the former case cannot exceed the value derived in the latter case. Various methods in estimating the effect of grouping on pile bearing capacity are available (E. P. Chellis "Pile Foundation" - Theory-Design- Practice" Chapter 6).

### III. Consolidation Tests and Settlement Analyses

The variable character of the layer of interbedded silty clay, clayey silt and silt made it extremely difficult to arrive at one definite conclusion regarding the consolidation characteristics of this stratum. As could be seen, the preconsolidation pressures ranged from 1900 p. s. f. to 10,140 p. s. f. In some instances the preconsolidation pressures, as obtained according to Casagrande's graphical procedure, coincided with the calculated overburden pressures indicating the normally loaded character of the deposits.

### III. Consolidation Tests and Settlement Analyses - Cont'd.

This also was evident from the shear strength profile. But in some cases a definite preconsolidation was obtained. We are of the opinion that the quite variable character of the deposit, and possibly some "precompression" effect during the sampling, are responsible for such inconsistencies.

It was clear from the start that the settlement will have to be computed either from the uncorrected average  $e$ -log  $p$  curve, or from the averaged values of the coefficient of Volume Decrease versus applied average pressures curve as the construction of the field virgin consolidation curve which will represent the consolidation characteristics of the whole layer would be practically impossible.

Further, in arriving at the average  $e$ -log  $p$  curve an arithmetical procedure in arriving at the average values was followed, but for the  $m_v$  versus applied pressures curve - the average curve was fitted best graphically. Thus the settlement calculations using both methods (i. e. change in void ratio method and the coefficient of volume decrease method) would give answers which will not give the same results, although from theoretical consideration they should agree closely.

Furthermore, it is generally accepted in soil mechanics that one of the most difficult tasks is the prediction of the amount of the final settlement. Examination of the settlements of many structures by S Skempton, Peck and MacDonald ("Settlement Analysis of Six Structures in Chicago and London" Proc. Inst. C.E., Part I, Volume 4, July 1955) then Oboiling and Gibson ("Settlement Studies on Structures in England" - Conf. Calculated and observed Stresses and Displacement, I.C.E. 1955) and MacDonald and Skempton (Survey of Comparisons between Calculated and Observed Settlements of Structures on Clay - Conf. calculated and observed stresses and displacement, I.C.E. 1955) has shown that the error involving the computation of the final settlement is of order - 27% to + 57%.

Having in mind the type of deposits investigated, it is our opinion that although two seemingly different settlement values were given from two types of settlement calculation they do give a reasonable indication of the amount of settlement to be expected, i. e. the settlement will be in order of 6 to 12 inches for the embankment, 4 to 8 inches for the footing design, and about 6 inches for the friction pile design.

#### Revised theoretical amounts of settlement

##### I. Embankment

- Assumptions (1) Height of the proposed embankment :  
H = 25 ft. (It was assumed that the whole peat layer will be displaced)  
Width of the embankment : B = 50 ft.  
Slope : 1 vertical to 4 horizontal



Revised theoretical amounts of settlement - Continued.

1. Embankment

Assumptions: (ii) Unit weight of embankment

$$\gamma = 125 \text{ lbs. per cu ft.}$$

Thus: the contact pressure at assumed foundation elevation 817.0

(soil profile assumed as encountered at test hole 5)

$$25 \times 125 = 3000 \text{ p.s.f.}$$

- (iii) Settlement is calculated at the centre of the proposed embankment.
- (iv) The lower limit of the compressible stratum at elevation 738, i.e. the depth of the compressible layer is 79 feet.
- (v) The vertical stress distribution was calculated using Westergaard's graphs.
- (vi) The  $\Delta p_{av}$  was calculated using Simpsons equation:

$$\Delta p_{av} = \frac{1}{3} (p_t + 4 p_m + p_b)$$

where  $p_t$  - stress at the top portion

$p_m$  - stress at the midsection under consideration

$p_b$  - stress at the bottom portion

- (vii) Coefficient of volume decrease  $M_v$  was taken from the graph of  $M_v$  versus average applied pressures for the various  $\Delta p_{av}$
- viii) The settlement is then calculated according to the following relations

$$\Delta s = p_{av} \Delta M_v \times \Delta h$$

The total theoretical amount of settlement as calculated from the  $M_v$  relation (see the detailed settlement analysis) is 12.4 inches.

For the settlement from the change in void ratio, the following assumptions were made:

- i) The C.W.L. is 1 ft. below the existing grade which was assumed to be at 835.8.

ii) The densities were:

for peat layer 35.5 lbs. per cu. ft.  
layer of interbedded silty clay, clayey silt  
and silt, 126 lbs. per cu. ft.

iii) The void ratios for corresponding pressures were  
taken from the uncorrected average e-log p curve

iv) The settlements were calculated according to the  
following relation:

$$\Delta s = \frac{e_0 - e}{1 + e_0} \Delta h$$

The theoretical amount of primary settlement calculated according to the  
above equation was found to be 9.45 inch.

Thus, as may be seen, a difference in the amounts of  
settlement is obtained using two methods of computation. The reason  
for discrepancy was explained in preceding paragraph, but in addition  
in the above computation a stress release of 113.5 p.s.f., due to placing  
the embankment at elevation 817.0, was considered. (This stress release  
arises from the removal of the original overburden pressure).

### 3. Spread Footing Design

The settlement for the spread footing design is calculated  
according to both methods. In addition to the assumptions made for the  
case of the embankment - the following additional assumptions are taken  
for the spread footing design:

- i) An assumed foundation elevation of 823.0 (Area of  
test holes A & B)
- ii) An assumed footing size of 15 x 60 feet
- iii) An allowable bearing value of:
  - (a) West abutment 4020 p.s.f.
  - (b) East abutment 4120 p.s.f.Thus an average bearing value of 4100 p.s.f. is taken  
for both abutments.

The above values were taken from the revised allowable  
bearing values as given in preceding section.

The amount of total consolidation settlements were calculated:

- (a) from  $M_v$  relation -  $s = 4.9$  inch
- (b) from e-log p curve -  $s = 8.75$  inch

### 3. Friction Pile Foundation Design

- Assumptions:
- (1) Existing grade elevation 836
  - (2) Cut off at elevation 820
  - (3) Length of friction pile 40' 0"
- Thus, the elevation at the tip of the piles 786.
- Diameter of pile 18 inches

- (4) Bearing capacity of pile:  
 Total bearing capacity  $Q = Q_f + Q_p$   
 where  $Q_f = A_s \times c$ , and  
 $Q_p = N_c \times S_u \times A$

$A_s$  - skin area of embedded pile

$c$  - 430 p.s.f. as obtained from tests

$N_c$  = 9 as for deep footings

$S_u$  = 1200 the minimum undrained shear strength of the soil

$A$  - cross sectional area of the pile

$$\text{Thus: } Q_f = 1.5 \times 40 \times A \times 430$$

$$= 81000 \text{ lbs.}$$

$$Q_p = 9 \times 1200 \times \frac{1.5^2}{4} = 19000 \text{ lbs.}$$

$$\text{Total } Q = 81.0 + 19.0 = 100$$

$$\text{and with F.S.} = 2, \text{ the } q = \frac{100}{2} = 50k \text{ per pile}$$

Assuming further that these will be 5 rows @ 15 piles each the total load per pier will be:

$$= 50 \times 45 = 2250 \text{ kips}$$

$$= 1125 \text{ tons}$$

Due to effect of pile group, the assumed efficiency will be say 80%, thus the total load will be 1800 kips and further assuming an area of 12 x 30 ft. the equivalent uniform load may be assumed to be

$$\frac{1800}{12 \times 30} = 1.87 \text{ k/ft.}^2$$

- (5) In estimating the settlement, the  $q = 1.87 \text{ k/ft.}^2$  is applied at 2/3 of pile length i.e. at elevation  $820 - \frac{40 \times 2}{3}$   
 $= 820 - 26.7 = 793.3$   
 say elevation 793.0

3. Friction Pile Foundation Design - Cont'd.

- (6) The vertical stress distribution of the applied pressure of 1.87 at elevation 793 is distributed with depth as for the conventional footings using Boussinesq equation.
- (7) The amounts of the theoretical consolidation settlements are:
  - (a) from  $M_v$  - relation 4.50 inch
  - (b) from e-log p relation 1.35 inch

Yours very truly,  
E. M. PETO ASSOCIATES LTD.

*C. F. Freeman*

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

BL/vs

# e. m. peto associates ltd.

YOUR REFERENCE:.

OUR REFERENCE:.

59249

1287 caledonia road.

TORONTO 19, ONTARIO.

RUssell 9-1126

8th June, 1960.

Department of Highways of Ontario,  
Materials and Research Section,  
Parliament Buildings,  
Toronto 2, Ontario.

Attention: Mr. A. Futka, P. Eng.

re: Foundation Investigation Report  
East Cross River, Highway 7A  
W.P. 42-60, Port Hope, Ontario  
Our reference No. 59249

Dear Sirs:

In reply to your letter, dated the 11th May, 1960, regarding the data presented in the foundation report on the above site. After a careful review of the data presented in this report, we have come to the following conclusions:

## 1. Field Work

The main purpose of the two additional test holes was to determine the consolidation characteristics of the stratum of interbedded silty clay, clayey silt and silt. During the original investigation it was found that ordinary sampling methods and handling produced considerable sample disturbance. For this reason, additional care was exercised in obtaining and handling the samples during the subsequent investigation. The other purpose of the additional investigation was to find, if possible, a dense bearing stratum for an end-bearing type of pile. The presence of such a layer was positively established at test hole A only after considerable difficulty was experienced in sinking this test hole, due to Artesian water conditions. Similar, ~~although water was not~~ <sup>artesian</sup> conditions, were encountered at hole B and it was thought best not to aggravate this situation by releasing the Artesian pressure at this test hole, for this reason test hole B was terminated at a depth of 92'4".

## II. Strength Tests and Allowable Bearing Values

The shear strength determinations were carried out during the original (1st part) investigation. The type tests employed in arriving at the shear strength characteristic of the soil were

- (a) Unconfined compressive tests
- (b) Field vane tests
- (c) Laboratory vane tests
- (d) Triaxial shear tests

As mentioned in our report (Part I, page 7) the laboratory vane test results had to be discarded due to considerable influence of sample disturbance. They were included only for the purpose of illustrating the influence of sample disturbance on the results of shear strength.

The triaxial shear tests, although denoted as UU-tests in our report (Part I, page 8) actually were allowed to consolidate for 24 hours in the laboratory before the samples were sheared. The rough estimation indicates that about 90% of consolidation did take place before vertical load was applied. This apparent discrepancy between the reported procedure and the actual was discussed between our Messrs. Lewicki and Freeman and your Messrs. L. G. Soderman and A. Stermac, and although we realise that in a strict sense the test does not represent a CU test; at the same time, the values of shear strength as calculated from the shear parameters as obtained from these tests show fairly good agreement, having in mind the type of soil stratum under consideration, between the shear values as obtained from other tests. (See graph of shear strength versus elevation Appendix III).

The field vane tests were carried out in test holes 2 and 3. Apart from the three initial test results of test hole 3 which were conducted in the layer of peat (from 3'0" to 13'6") the remainder of the tests were carried out in the layer of interbedded silty clay, clayey silt and silt.

The vane test gives only the value of the undrained shear strength and thus, the material best suited to be tested should have very low permeability so as not to allow any dissipation of pore pressure during the test. The vane strength values were found to be reliable when applied to the problems, using  $\phi = 0$  method of stability analysis.

In the "Symposium on Vane Shear Testing of Soils" A. S. T. M. Special Technical Publication No. 198 - in a paper by W. C. Hill "Vane In-Place Soil Shear Device Developed and Applied by Oregon State Highway Department" on page 37 of the above publication, a table of comparative soil shear values from vane shear test and direct shear test are given.



The material tested had a value of  $\phi$  varying from  $4.5^{\circ}$  to  $37.0^{\circ}$  with a "unit cohesion" varying from 0 to 2.5 p.s.f. According to these results, fairly good agreement was obtained for various types of soil between the shear values as calculated from the direct shear and the field vane values.

In the case under the consideration, i.e. the layer of inter-bedded silty clay, clayey silt and silt, it was found that the material had a low permeability and for practical reasons, it may be assumed that there will be no dissipation of pore pressures, thus the vane shear test was considered to give quite reliable values.

On the graph of shear strength versus elevation (Part I, Appendix III) an attempt was made to find the best shear strength distribution between the shear values as obtained from various tests. A line was drawn indicating the assumed average undrained shear strength distribution with depth. This curve gives the following  $S_u$  values (as given in the report)

Elevation	815 - 2300 p.s.f.
	810 - 2100 p.s.f.
	805 - 1850 p.s.f.
	800 - 1600 p.s.f.
	795 - 1350 p.s.f.
	792 - 1200 p.s.f.
	788 - 1200 p.s.f.
	780 - 2000 p.s.f.

As may be seen, the assumed average curve does follow the trend of the shear strength results as obtained from various tests, and for this reason it is considered that it does fairly represent the shear profile of the subsoil.

The shape of the shear strength profile indicates a normally loaded deposit with the stiffer upper crust reaching down to elevation 792.

#### Revised Allowable Bearing Values for continuous footings

The allowable bearing values are calculated using the following equation:

$$Q_a = \frac{S_u \cdot N_c}{F.S.}$$

where  $S_u$  = undrained shear strength. This was taken to be the average over a depth equivalent to 2/3 of the footing width from the shear strength profile as given in Appendix III, Part I, with a maximum undrained shear strength above elevation 815 of 2300 p.s.f.

$N_c$  - as obtained from the graph of  $N_c$  vs  $D/B$  according to Skempton's empirical relation

$$N_c = 5 \left( 1 + \frac{D}{5B} \right) \left( 1 + \frac{B}{5L} \right)^{3/4}$$

$D$  - the foundation depth

The foundation depth was assumed to be equal to the height of material overlying the foundation elevation of the same unit density as the material underlying the foundation elevation. In the case of the conditions encountered at this site in the area of the proposed East abutment, i.e. area of test holes B, 2 and 3 - the lower limit of the peat layer was assumed to be at elevation 822, i.e. 14.3 feet below the existing grade, having a unit density of 65.5 lbs. per cu.ft.

The equivalent height of material, having a density of 126 lbs. per cu. ft. (as taken for the layer of interbedded silty clay, clayey silt and silt) is then

$$14.3 \times 65.5 = \text{equivalent height} \times 126$$

$$\text{equivalent height} = \frac{14.3 \times 65.5}{126} = 7.4 \text{ feet}$$

which at the same time represents the foundation depth at elevation 822.0.

Thus the following table may be set out:

#### East Abutment

Proposed found Elevation	Found Depth (D) (ft.)	Found Width B (ft.)	D/B	$N_c$	Average $S_u$ p.s.f.	$q_a$ p.s.f.
822	7.4	8	1.48	3.47	2300	4980
		10	0.74	5.73	2300	4390
		15	0.49	5.49	2250	4120
		20	0.37	5.37	2150	3860

East Abutment - Cont'd.

Proposed found. Elevation	Found. Depth D (ft.)	Found. Width B (ft.)	D/B	$N_c$	Average $S_u$ p. s. f.	$q_a$ p. s. f.
820	9.4	5	1.88	6.87	2300	5270
		10	0.94	5.93	2300	4530
		15	0.63	5.62	2200	4120
		20	0.47	5.46	2100	3830

West Abutment:

The foundation depth was found to be 6.1 feet for the proposed foundation elevation at 824 thus the following table of allowable bearing values may be given:

Foundation Elevation	Found. Depth D (ft.)	Found Width B (ft.)	D/B	$N_c$	Average $S_u$ p. s. f.	$q_a$ p. s. f.
824	6.1	5	1.22	5.22	2300	4770
		10	0.61	5.61	2300	4300
		15	0.41	5.41	2300	4150
		20	0.30	5.30	2200	3890
822	8.1	5	1.62	6.61	2300	5070
		10	0.81	5.81	2300	4460
		15	0.54	5.54	2250	4080
		20	0.41	5.41	2150	3880

As may be seen for the larger footing sizes, the allowable bearing values decrease. This decrease is firstly due to the decrease in the ratio of  $D/B$  and, consequently in the  $N_c$ - value, and secondly because of the decrease in the average undrained shear strength over the range of depth considered, which was equivalent to two thirds of the width of a continuous footing.

The detailed calculation therefore, will be as follows (East Abutment, Foundation Elevation 820)

Continuous footing  $B = 5.0$  ft.

Foundation depth  $D = 9.4$  ft.

Therefore  $D/B = 1.88$

and from the graph, the corresponding

$N_c$ - value is 6.87

The average  $S_u$  : at elevation 820 is 2300 p.s.f.  
 (taken from the assumed average shear strength profile)

$$\text{at elevation } 820 - \frac{5 \times 2}{3} = 817.7$$

and at elevation 817.7 the average undrained shear strength is 2300 p.s.f.

Allowable bearing value (F.S. = 3)

$$q_a = \frac{6.87 \times 2300}{3} = 5270 \text{ p.s.f.}$$

For continuous footing  $B = 20$  feet

$D/B = 0.47$  and the corresponding  
 $N_c$  value = 5.46

The average  $S_u$  at elevation 820 is 2300 p.s.f.

The average  $S_u$  at elevation  $(820 - \frac{20 \times 2}{3})$

$$= 808.7 \text{ where } S_u = 1900 \text{ p.s.f.}$$

Therefore average  $S_u = 2100$  p.s.f.

The allowable bearing value  $q_a = \frac{5.46 \times 2100}{3} = 3830 \text{ p.s.f.}$

Thus, as may be seen for the 20 feet wide continuous footing the allowable bearing value is 3830 p.s.f. as compared with 5270 p.s.f. for 5 feet wide continuous footing.

The allowable bearing values as calculated above represent the net bearing value i. e. they do not take into account the effect of existing overburden pressure.

#### Pile foundation

The total bearing capacity of friction piles is the sum of the "friction" capacity and the point bearing capacity.

$$\text{Then } Q = Q_f + Q_p$$

where  $Q_f$  - friction capacity

$Q_p$  - point bearing capacity

### Pile foundation - Cont'd.

The frictional component of bearing capacity is calculated according to equation:

$$Q_f = A_s \times S_a$$

where  $A_s$  - the surface area of the pile  
 $S_a$  - cohesive strength of the soil

(See "Use of Strength Data in Pile and Spread Footing Design" by Roberts M.I.T. - Summer Course, 1959).

The end bearing capacity portion of the total bearing value is calculated as follows:

$$Q_p = A_p \times N_c \times S_u$$

where  $A_p$  - cross sectional area of the pile  
 $N_c$  - 9.0  
 $S_u$  - undrained shear strength of the soil

The triaxial shear tests gave a minimum value for the cohesion of 430 p.s.f. This value was recommended as the cohesive strength of the soil for use in the equation for the frictional component of the total bearing value of the friction piles.

For piles acting in a group (where piles are spaced less than 4 times the diameter of the pile) the ultimate bearing capacity of the pile group, as a whole should be considered, in the light of the load apportioned to each pile when it is acting as one of a group, compared with the load bearing capacity of a pile acting as a single unit. The value in the former case cannot exceed the value derived in the latter case. Various methods in estimating the effect of grouping on pile bearing capacity are available (R.D. Chellis "Pile Foundation" - Theory-Design- Practice" Chapter 6).

### III. Consolidation Tests and Settlement Analyses

The variable character of the layer of interbedded silty clay, clayey silt and silt made it extremely difficult to arrive at one definite conclusion regarding the consolidation characteristics of this stratum. As could be seen, the preconsolidation pressures ranged from 1900 p.s.f. to 10,140 p.s.f. In some instances the preconsolidation pressures, as obtained according to Casagrande's graphical procedure, coincided with the calculate overburden pressures indicating the normally loaded character of the deposits.

### III. Consolidation Tests and Settlement Analyses - Cont'd.

This also was evident from the shear strength profile. But in some cases a definite preconsolidation was obtained. We are of the opinion that the quite variable character of the deposit, and possibly some "precompression" effect during the sampling, are responsible for such inconsistencies.

It was clear from the start that the settlement will have to be computed either from the uncorrected average  $e$ -log  $p$  curve, or from the averaged values of the coefficient of Volume Decrease versus applied average pressures curve as the construction of the field virgin consolidation curve which will represent the consolidation characteristics of the whole layer would be practically impossible.

Further, in arriving at the average  $e$ -log  $p$  curve an arithmetical procedure in arriving at the average values was followed, but for the  $m_v$  versus applied pressures curve - the average curve was fitted best graphically. Thus the settlement calculations using both methods (i. e. change in void ratio method and the coefficient of volume decrease method) would give answers which will not give the same results, although from theoretical consideration they should agree closely.

Furthermore, it is generally accepted in soil mechanics that one of the most difficult tasks is the prediction of the amount of the final settlement. Examination of the settlements of many structures by S Skempton, Peck and MacDonald ("Settlement Analysis of Six Structures in Chicago and London" Proc. Inst. C.E., Part I, Volume 4, July 1955) then Oboiling and Gibson ("Settlement Studies on Structures in England" - Conf. Calculated and observed Stresses and Displacement, I.C.E. 1955) and MacDonald and Skempton (Survey of Comparisons between Calculated and Observed Settlements of Structures on Clay - Conf. calculated and observed stresses and displacement, I.C.E. 1955) has shown that the error involving the computation of the final settlement is of order - 27% to + 57%.

Having in mind the type of deposits investigated, it is our opinion that although two seemingly different settlement values were given from two types of settlement calculation they do give a reasonable indication of the amount of settlement to be expected, i. e. the settlement will be in order of 9 to 16 inches for the embankment, 4 to 8 inches for the footing design, and about 5 inches for the friction pile design.

#### Revised theoretical amounts of settlement

##### 1. Embankment

- Assumptions (i) Height of the proposed embankment :  
H = 25 ft. (It was assumed that the whole peat layer will be displaced)  
Width of the embankment : B = 50 ft.  
Slope : 1 vertical to 2 horizontal



Revised theoretical amounts of settlement - Cont'd.

1. Embankment

Assumptions: (ii) Unit weight of embankment

$$\gamma = 125 \text{ lbs. per cu ft.}$$

Thus: the contact pressure at assumed foundation elevation 817.0

(soil profile assumed as encountered at test hole 5)

$$25 \times 125 = 3000 \text{ p.s.f.}$$

- (iii) Settlement is calculated at the centre of the proposed embankment.
- (iv) The lower limit of the compressible stratum at elevation 738, i.e. the depth of the compressible layer is 79 feet.
- (v) The vertical stress distribution was calculated using Westergaard's graphs.
- (vi) The  $\Delta p_{av}$  was calculated using Simpsons equation:

$$\Delta p_{av} = \frac{1}{6} (p_t + 4 p_m + p_b)$$

where  $p_t$  - stress at the top portion

$p_m$  - stress at the midsection  
under consideration

$p_b$  - stress at the bottom portion

- (vii) Coefficient of volume decrease  $M_v$  was taken from the graph of  $M_v$  versus average applied pressures for the various  $\Delta p_{av}$
  - viii) The settlement is then calculated according to the following relations
- $$\Delta s = p_{av} \times M_v \times \Delta h$$

The total theoretical amount of settlement as calculated from the  $M_v$  relation (see the detailed settlement analysis) is 13.4 inches.

For the settlement from the change in void ratio, the following assumptions were made:

- i) The G.W.L. is 1 ft. below the existing grade which was assumed to be at 835.5.

ii) The densities were:

for peat layer 65.5 lbs. per cu. ft.  
layer of interbedded silty clay, clayey silt  
and silt, 126 lbs. per cu. ft.

iii) The void ratios for corresponding pressures were  
taken from the uncorrected average e-log p curve

iv) The settlements were calculated according to the  
following relation:

$$\Delta s = \frac{e_0 - e}{1 + e_0} \Delta h$$

The theoretical amount of primary settlement calculated according to the  
above equation was found to be 9.45 inch.

Thus, as may be seen, a difference in the amounts of  
settlement is obtained using two methods of computation. The reason  
for discrepancy was explained in preceding paragraph, but in addition  
in the above computation a stress release of 119.5 p.s.f. due to placing  
the embankment at elevation 817.0, was considered. (This stress release  
arises from the removal of the original overburden pressure).

## 2. Spread Footing Design

The settlement for the spread footing design is calculated  
according to both methods. In addition to the assumptions made for the  
case of the embankment - the following additional assumptions are taken  
for the spread footing design:

- i) An assumed foundation elevation of 922.0 (Area of  
test holes A & B)
  - ii) An assumed footing size of 15 X 60 feet
  - iii) An allowable bearing value of:
    - (a) West abutment 4080 p.s.f.
    - (b) East abutment 4120 p.s.f.
- Thus an average bearing value of 4100 p.s.f. is taken  
for both abutments.

The above values were taken from the revised allowable  
bearing values as given in preceding section.

The amount of total consolidation settlements were calculated:

- (a) from  $M_v$  relation -  $s = 9.0$  inch
- (b) from e-log p curve -  $s = 5.75$  inch

### 3. Friction Pile Foundation Design

- Assumptions: (1) Existing grade elevation 835  
 (2) Cut off at elevation 820  
 (3) Length of friction pile 40'0"  
 Thus, the elevation at the tip of the piles 780.0  
 Diameter of pile 18 inches

- (4) Bearing capacity of pile:  
 Total bearing capacity  $Q = Q_f + Q_p$   
 where  $Q_f = A_s \times c$ , and  
 $Q_p = N_c \times S_u \times A$

$A_s$  - skin area of embedded pile  
 $c$  - 430 p.s.f. as obtained from tests  
 $N_c$  = 6 as for deep footings  
 $S_u$  = 1200 the minimum undrained shear strength of the soil  
 $A$  - cross sectional area of the pile

$$\text{Thus: } Q_f = 1.5 \times 40 \times 430 = 81000 \text{ lbs.}$$

$$Q_p = 6 \times 1200 \times \frac{1.5^2}{4} = 19000 \text{ lbs.}$$

$$\text{Total } Q = 81.0 + 19.0 = 100$$

and with F.S. = 2, the  $q = \frac{100}{2} = 50k$  per pile

Assuming further that these will be 3 rows of 15 piles each the total load per pier will be:

$$Q = 50 \times 45 = 2250 \text{ kips}$$

$$= 1125 \text{ tons}$$

Due to effect of pile group, the assumed efficiency will be say 60%, thus the total load will be 1350 kips and further assuming an area of 12 x 60 ft. the equivalent uniform load may be assumed to be 1350

$$\frac{1350}{12 \times 60} = 1.87 \text{ k/ft.}^2$$

- (c) In estimating the settlement, the  $q = 1.87 \text{ k/ft.}^2$  is applied at 2/3 of pile lengths i.e. at elevation  $820 - \frac{40 \times 2}{3} = 820 - 26.7 = 793.3$   
 Say elevation 793.0

3. Friction Pile Foundation Design - Cont'd.

- (5) The vertical stress distribution of the applied pressure of 1.87 at elevation 783 is distributed with depth as for the conventional footings using Boussinesq equation.
- (7) The amounts of the theoretical consolidation settlements are:
  - (a) from  $M_v$  - relation 4.50 inch
  - (b) from e-log p relation 1.85 inch

Yours very truly,  
E. M. PETO ASSOCIATES LTD.

*C. F. Freeman.*

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

BE/vs

May 11, 1960.

E. M. Peto & Associates, Ltd.,  
1287 Caledonia Rd.,  
Toronto, Ontario.

Re: Foundation Investigation Report  
East Cross River, Highway 7-A,  
W.P. 42-60, Port Hope, Ontario.  
Your Reference No. 59249.

Dear Sirs:-

This is to advise you that the Foundations Sub-section of the Materials & Research Section, have reviewed the above foundation report. This review has resulted in our general conclusion that the data presented in this report is technically inadequate. The details which led to this conclusion, have been discussed with your Messrs. Lewicki, Freeman, and Hitchins. These details are enumerated under the following headings:-

1. FIELD WORK:

As a supplement to the initial drilling program, two additional boreholes were authorized. The purpose of these borings was to obtain data on the compressibility of the substrata and also to define the vertical extent and continuity of a dense layer of sand at depth.

The depth of the underlying sand layer was proven adequately in Borehole A. Borehole B was terminated at least 5 feet above the upper horizon of this layer, as determined in Hole A. This boring should not have been terminated until the continuity of the sand stratum had been established. There is no appreciable difference in artesian conditions reported on Holes A and B and the reason given for stopping "B" above the sand layer is difficult to reconcile.

cont'd. /2 ...

## II. LABORATORY TESTING:

### (A) Strength Tests.

Shear strength measurements have been carried out using the following reported tests: (1) Unconfined Compression Tests - 6 in number, (2) Field Vane Tests - 13 in number, (3) Laboratory Vane Tests - 13 in number, and (4) Undrained Triaxial Tests with Pore Pressure Measurements - 6 in number.

Comments on the above test results are as follows:-

(1) The 13 laboratory vane test results were entirely disregarded due to reported sample disturbance.

(2) Undrained unconsolidated triaxial tests with p.w.p. measurements have been reported and these tests have been used to determine values of shear strength parameters in terms of effective stresses. This is, in principle, an erroneous procedure for determining these parameters. For saturated cohesive soils  $C'$  and  $\phi'$  can only be determined by consolidated undrained tests. The subsequent relationships between  $C'$ ,  $\phi'$  and depth, which have been plotted, are consequently meaningless.

(3) Field vane strength measurements have been reported at elevations where the plasticity index of the soil was found to be 5 or less. Results obtained from in-situ vane tests are based upon the assumption that the soil into which the vane is inserted will fail under  $\phi = 0$  conditions. That this condition can be assumed for soil of such low plasticity, has not been substantiated by research or case studies, and the results presented cannot be used with confidence.

### (B) Oedometer Tests.

A total of 12 void ratio vs. log p curves have been determined and reported. In order to estimate settlements on cohesive subsoil strata, the estimator must know whether or not the deposit is normally consolidated or overconsolidated. This conclusion has not been indicated in the report. The use of void ratio vs. log p data varies for each case. Preconsolidation pressures vary from a reported value of 1,900 to 10,140 p.c.f. An average void ratio vs. log p curve has been drawn indicating a most probable preconsolidation pressure of 3000 p.c.f. It is not made clear in the report that either a corrected void ratio vs. log p curve is recommended for settlement calculations or that the uncorrected average lab. curve should be used. Also:-

cont'd. /3 ...

## II. LABORATORY TESTING: (cont'd.) ...

(B) (cont'd.) ...

(1) Different settlement values are given in the report based upon two computation procedures which should, in principle, give sensibly the same result.

(2) The units given for  $M_v$  on Page 4, Part II, are incorrect.

(3) Statements to the effect that the stratum investigated will have a "fairly rapid rate of settlement", and "these values of coefficient of permeability place the layer investigated in the range of low to practicably impermeable materials", are contradictory.

## III. BEARING CAPACITY EVALUATIONS:

### Footings:

In the determination of the allowable bearing capacity of spread footings founded at Elev. 822' or below, the value of the average undrained shear strength below the footing for a depth equal to the footing width, must be used. (Ref. Skempton - 1951 - Bearing Capacity of Clays). These undrained shear strength values are given in the report - i.e., 2293 p.s.f. at Elev. 815.5', 73,000 p.s.f. at Elev. 808.1', and 2210 p.s.f. at Elev. 807.7' - the second value must obviously be in error. The two remaining values indicate an average  $C = 2250$  for the stratum above Elev. 807.7'. On Page 7 of Part I, last paragraph, and on Page 10, Part I, the value of undrained shear strength of 2100 for Elev. 810' has been quoted. This value cannot be reconciled with the factual data in the report.

On Page 11, Part I, allowable bearing pressures are quoted for footings 5, 10 and 15 feet in width. These values show a reduction in bearing capacity with width. Computations made on review of these values are as follows:-

$$\text{Elev. 822} \quad B = 5' \quad \ell = 30' \quad D = 0$$

$$C = 2250 \text{ p.s.f.}$$

$$N_c = (2 + \pi) (1 + \frac{B}{\ell}) (1) = 5.3$$

$$q_{\text{net allowable}} = \frac{2250 \times 5.3}{3} \approx 4000 \text{ p.s.f.}$$

$$\text{Elev. 822} \quad B = 10' \quad \ell = 30' \quad D = 0$$

$$N_c = 5.45 \quad C = 2250$$

$$q_{\text{net allowable}} = \frac{5.45 \times 2250}{3} = 4100 \text{ p.s.f.}$$

cont'd. /4 ...

### III. BEARING CAPACITY EVALUATIONS: (cont'd.) ...

#### Footings: (cont'd.) ...

The effect of increasing the width from 5 feet to 10 feet is to increase, rather than decrease the bearing capacity as is shown on Page 11, Part I. This effect is independent of whether bearing capacities are reported in terms of gross or net footing pressures. A decrease in capacity is a result of decrease in shear strength below the footing and this is not consistent with data presented in the report.

#### Piles:

The procedure for design of friction piles in cohesive soils is to determine the capacity by summing two contributing components. One component is due to the mobilization of embedded shaft adhesion and the second component is due to point resistance - (Ref. M. J. Tomlinson, ICSM - 1956). The value of shear strength to be used in this computation, is the appropriate undrained strength.

On Page 3, Part I of the report, the design procedure for friction piles is suggested, and the use of only the strength parameter  $C'$  - i.e., apparent cohesion in terms of effective stresses, is proposed. This procedure is unprecedented in the soil mechanics literature, and leads to the obvious questions: (1) Why is  $\phi'$  not used, as well, and (2) Since  $C'$  is only obtained in overconsolidated clays, what procedure should be used for designing piles in normally consolidated clays where  $C' = 0$ .

### IV. EMBANKMENT STABILITY:

Comments on stability analyses carried out, are as follows:-

(1) The type of embankment material is not mentioned in the report. A shear strength of 2000 p.s.f. in the fill has been used; this assumption is not justified if cohesive type fill is used.

(2) No consideration given to possible formation of tension crack in embankment.

(3) The average shear strength of the subsoil is of the order of 1500 p.s.f. Based upon bearing capacity consideration, alone, the ultimate load that could be supported is of the order of 3 tons/sq.ft. The embankment loading is only 2/3 of this value. With this obvious situation, the need to carry out 11 trial circles, appears to be a needless expenditure of time and effort.



V. CALCULATIONS:

The absence of a typical calculation showing the procedure for either bearing capacity evaluation or settlement estimate, made it difficult for our Section to check the values given in the report.

---

You will appreciate that we are responsible for the recommendations upon which substructure elements and earth or rock-filled embankments are designed. In order to fulfill this responsibility, it is imperative that the reports submitted to us by soils consultants, be factually correct and that the conclusions presented in the reports be based upon established principles in the field of soil mechanics.

We would welcome clarification or rebuttal of the foregoing comments; through discussion, we will gain a better understanding of the technical competence of your staff members and you, in turn, will gain a better understanding of our requirements pertaining to site investigations.

LGS/MdeF

Yours very truly,



A. Rutka,  
A/MATERIALS & RESEARCH ENGINEER

# COMMENTS

I GENERAL

II FIELD WORK

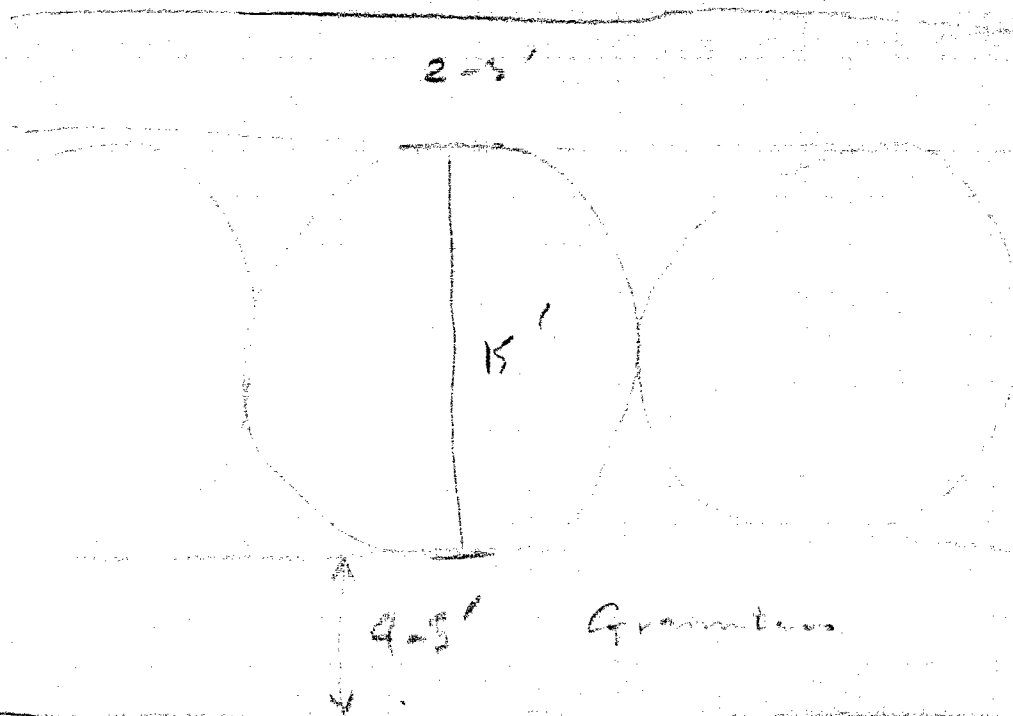
III LABORATORY WORK

IV INTERPRETATION, CONCLUSIONS AND  
RECOMMENDATIONS

1. BEARING CAPACITY

2. SETTLEMENTS

3. EMBANKMENT STABILITY



①

OVER

## COMMENTS

ON THE REPORT

EASTCROSS CREEK BRIDGE  
TOWNSHIP OF CARTWRIGHT

SUBMITTED BY E. M. PETO & ASSOCIATES.

### I General

The Report should have given data on field and laboratory work <sup>as well as</sup> ~~and~~ conclusions and recommendations based on these results. The recommendations should have contained proposals for the foundation of the structure i.e. the allowable bearing capacity of the soil for a <sup>kind or kinds of</sup> particular foundation, with respect to shear failure and settlement, and the <sup>stability analyses</sup> ~~checking of the stability of~~ the approach embankment.

Our comments given below refer to the above mentioned questions of the Report

### III Laboratory works

a) Values of  $c'$  and  $\phi'$  cannot be obtained from <sup>the</sup> (unconsolidated undrained type of test).

b) It is very strange that 13 lab. vane tests were done and all had to be discarded because of the considerable sample

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~~The carrying out and the interpretation of the standard penetration test does not seem to be right. The sampler cannot be driven <sup>into the ground</sup> for two feet without having the material compacted in the cylinder and thus breaking a full cylinder into the ground. For such 2 feet of penetration the number of blows for such two feet of penetration cannot be divided by two to obtain the number of blows for the standard penetration test.~~

~~The carrying out and the interpretation of the standard penetration test does not seem to be right. If the sampler is driven into the ground for more than 6 feet after 6 inches of initial penetration, the cylinder is~~

The standard requires for a penetration of 18 inches of which the first 6 inches take care for the disturbance of the bottom of the borehole. Any further penetration does not correspond to the standard and cannot therefore be interpreted as such.

difficult conditions existed.

⊗ TURN PAGE S.V.P.

#### IV Interpretation and conclusions

##### 1) Bearing capacity

The interpretation of shear strength values is not correct because of

a) ~~the~~ Shear strength values expressed in terms of effective stress cannot be compared and interpreted with those expressed in terms of total stress.

~~as all the values tests were identical~~

↑  
The ~~average~~ shear strength  $\sigma$  elevation curve given in the Report ~~does not~~ <sup>is</sup> represent an average because of the reasons stated above and earlier. Because of that ~~the~~ values chosen for <sup>the</sup> calculation of the bearing capacity of the soil are not representative and the calculations not satisfactory and realistic.

b) The application of the value in materials of such a low plasticity ( $PI = 5$  or less) as occur below elevation 790 is not justified and the use of such results can be misleading.

The recommendation for the computation of the bearing capacity of friction piles on the basis of  $c'$  value is wrong.

OVER

(4)

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MS. Consideration of Part 17. 1. 1. 1.

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### 3. Embankment stability

The assumption that the shear strength of the fill will be 2,000 lb/sq ft is wrong because such a strength is unobtainable in a cohesive material which has to be placed under water as is the case here.

The submitting of 11 stability analyses of which the most unfavourable (with the assumptions made) gives a factor of safety of 3.92 is not justified, and the time spent is wasted.



## 2. Settlement

Different settlement values are given in the Report with the explanation that they were computed on the basis of the void <sup>ratio</sup> change and <sup>the coefficient of volume decrease</sup> ~~the~~ ~~values~~ respectively. ~~These two~~ Since these two are interrelated the same settlement values should have been obtained.

The settlement analysis was carried out without the modification ~~for~~ i.e. correction of the oedometer results.

The dimensions of the coefficient of volume decrease are wrong.

Statements that "the stratum investigated will have a fairly rapid rate of settlement" and "These values (coeff. of permeability) place the layer investigated in the range of low to practically impermeable materials" are contradictory and misleading.

To relate the ~~difference of~~ <sup>difference of</sup> the coefficients of volume decrease to the ~~difference of~~ <sup>difference of</sup> ~~the~~ ~~coefficients of permeability~~ is not justifiable.

To relate the variation in rates of volume decrease to the difference in permeability ~~is~~ ~~has~~ is not justifiable because there is no relation whatsoever between these two phenomena.

OVER

E. M. PETO ASSOCIATES LIMITED  
SOIL ENGINEERS

1287 CALEDONIA ROAD  
TORONTO 19, ONTARIO

WORK ORDER

DATE ISSUED: Dec. 7th. 1959 CLIENT'S ORDER NO: Letter to follow OUR ORDER NO: 59249

CLIENT: Department of Highways of Ontario.

Soil & Foundation Engineering Branch, Downsview, Ontario.

W.P. 42-60

Hwy. 7A between Port Perry

JOB NAME: Culverts on Easterns Creek Bridge  
Township of Cartwright

LOCATION: 5 Hwy. 15.

OPERATOR: L. Gailis

ENGINEER: I.G. Bowie MH 9-1555

REPORTS: Final to Client INVOICES: 3 to Client CHARGES: Standard charges for  
D.R.O.

OPERATOR TO NOTE:

BX HOLES/DEPTHS:

Total of six (6) holes, 3 of which go to rock. 4 of the test  
holes are located at the abutments and 2 on the centreline.

4" HOLES/DEPTHS:

To be determined on site.

2" S.S. SAMPLES AT 2-3', 5-6', 7-8', 10-11', 12-13', 15-16', AND AT 5' INTERVALS AND/OR

ADDITIONAL SAMPLES AS FOLLOWS:

CASING (BAG) SAMPLES AT: 0'0" - 2'0"

2" BRASS LINERS AT: 9' - 10'0", 14' - 15', 19' - 20', 24' - 25', 29' - 30'

3" BRASS LINERS AT: At 5 foot intervals below anticipated footing elevation.

MOISTURE CONTENT TINS AT: All Split Spoon & Split Line samples.

HOLES DRY TO: As far as practicable.

CHECK WATER LEVELS a) DAILY ALL HOLES: Yes b) BEFORE PULLING CASING: Yes

c) AFTER PULLING CASING: Yes d) AT TIMED INTERVALS AFTER BAILING: 5, 10 & 20  
minutes.

Yes NOTE SPECIFICALLY ALL CHANGES, i.e. COLOUR, MATERIAL, DENSITY,  
(SOFTENING OR STIFFENING)

9 hours on site

WORK HOURS/DAY: 9 hours on site PUMP REQUIRED: Yes HOSE FOOTAGE:

SPECIAL REMARKS: Take shear vane equipment. Locate water bearing seams or  
strata. Dutch Cone to be driven at each hole. Use new 3" diameter x 60"  
Cones.

(SEE OVER)

OVER

ENGINEER TO NOTE:

1. SET OUT: Yes LEVEL: Yes TEST HOLES: \_\_\_\_\_
2. YOU SHOULD BE ON SITE AT: With crew AND REMAIN FOR: throughout job.
3. PAY SPECIAL ATTENTION TO: Depth of rock and type.  
Send print of first borehole log to Mr. K. Pocher, as soon as possible.
4. CONTACT MR. \_\_\_\_\_ AT \_\_\_\_\_  
ON COMPLETION OF \_\_\_\_\_
5. VERBAL REPORT BY: \_\_\_\_\_ COMPLETED REPORT BY: Definite  
Jan. 8th. 1959.
6. ESTIMATED FIELD TIME: \_\_\_\_\_
7. LABORATORY BUDGET: \_\_\_\_\_

GENERAL INFORMATION

Bridge is located less than half way ~~between~~ to Hwy/35 from Junction to Casarrea. Work must continue on West side. Expect to find rock along - via which over sand. The centre holes are to check substantial settlement and stability. Rock coring will only be carried on depending on condition of overburden in respect of foundation bearing and settlement characteristics. Dutch Cone to be driven before each hole. If soft clays are found the shear strength profile must be determined by Vane Test as well as Laboratory Test on undisturbed samples. Then 4" casing will be required for both Vane Test and Consolidation samples.

Holes # 1 & 6 for bank stability.

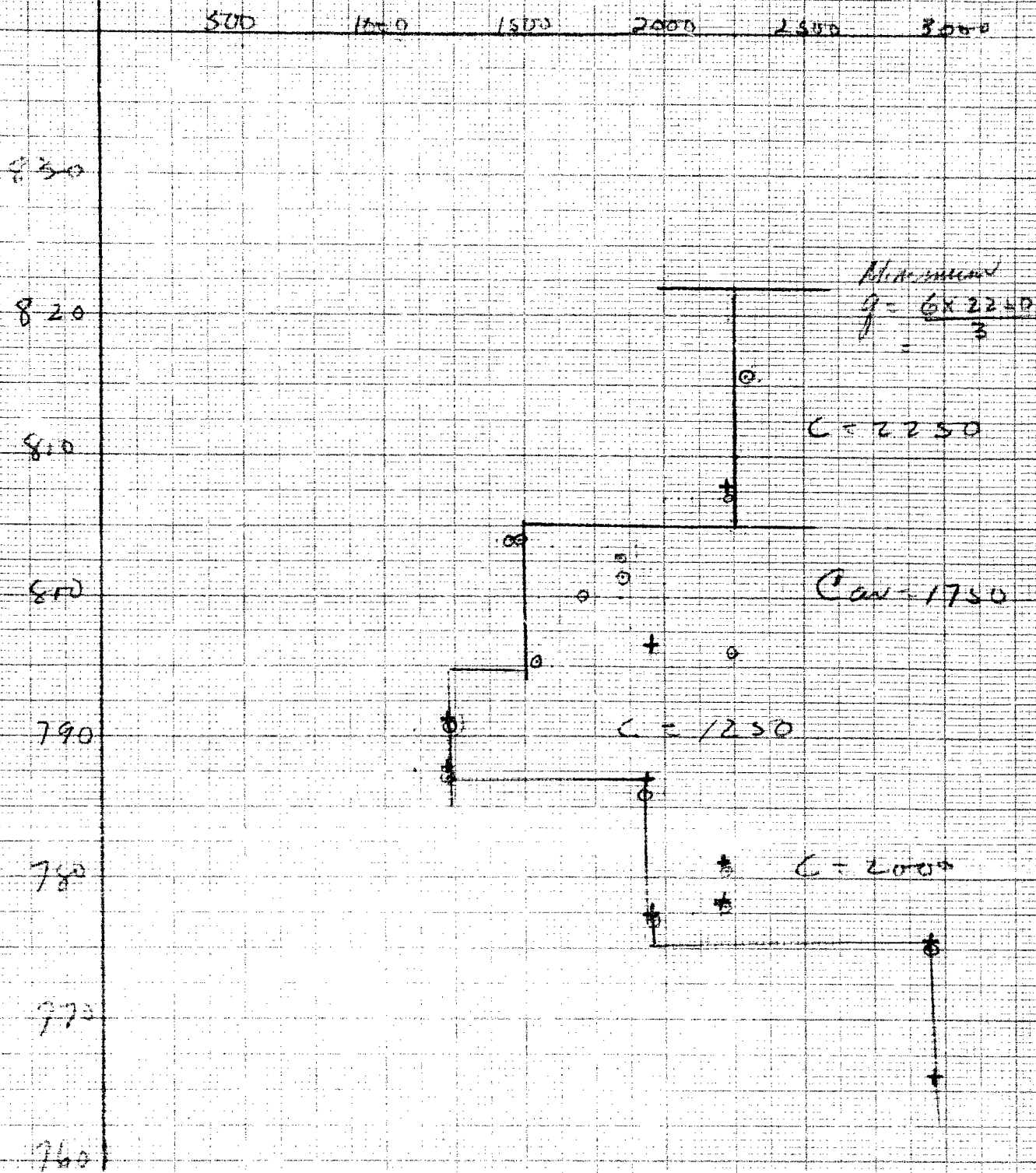
Hole # 2 to confirm conditions at # 3

Hole # 4 to confirm conditions at Hole # 1

Hole # 1 & 3 to rock. (Check before with Dutch Cone)

Assuming holes # 1 & 1

are similar.



**e. m. peto associates ltd.**

YOUR REFERENCE:- W. P. 42-60

OUR REFERENCE:- 59249

1287 caledonia road,

TORONTO 19, ONTARIO.

RUssell 9-1126

September 1st, 1960

The Department of Highways of Ontario,  
Materials & Research Section,  
Parliament Buildings,  
Toronto 2, Ontario.

Attention: Mr. A. Rutka, P. Eng.

Gentlemen:

Re: Foundation Investigation Report  
East Cross River Highway  
near Port Hope, Ontario.

We wish to confirm the verbal observations made at a meeting on 9th June, 1960 in your offices, at which time your letter of 11th May, 1960 and our reply dated 8th June, 1960, were discussed. Your Messrs. Soderman, Peaker and Stermac, and our Mr. B. Lewicki were present, together with the writer.

i) FIELD WORK

D. H. O. comments are correct. We agree that the report should have stated more clearly the reasons for stopping Hole B at a shallow depth.

Certain oral information was not included in the report since E. M. Peto Associates Ltd., considered it was non-essential to the report. After discussion, this viewpoint was appreciated and the matter was not pursued further.

ii) LABORATORY TESTING

(a) Shear Strength Tests

(1.) Laboratory Vane Tests - D.H.O. observation correct.

The 13 laboratory Vane test results were entirely disregarded due not only to (a) sample disturbance, but also (b) on the grounds of probability. Only two results out of the thirteen were reasonably undisturbed and these could have been included.

(2.) Undrained Consolidated Triaxial Tests -

The D. H. O. statement is in principle correct.

We confirm that we arrived at shear strengths from the shear envelope. The method of approach was discussed at some length, but Mr. Soderman stated that he thought this approach was incorrect; but the difference between the two methods was not resolved. It was agreed that for future calculations, the consolidation should be carried out under  $K_0$  conditions.

In the subject under discussion, the tests taken were more of the CU type than of the UU type, and the results were shown.

The usage of the equation representing the shear strength  $S = C + (p-u) \tan \phi$  was assumed to be valid for the calculation of the shear strength with depth. Mr. Soderman did quote a paper that this was incorrect, but we had felt that the  $K_0$  approach was purely a research one and not widely applied by soil engineers at the present time.

### (3) Field Vane Strength Measurements -

Our main reason for using vane tests was because there were clear indications of impervious conditions and the tests were therefore considered justified. It is worth mentioning here that the samples were very difficult to obtain.

Mr. Soderman stated that a good strength/depth relationship had not been proven below 795.00. This was agreed.

Mr. Soderman said that there was insufficient corroboration between undrained tests to substantiate the use of vane in materials with a low Plasticity Index, particularly below 795.00. He considered it incorrect in principle to use  $c'$ ,  $\phi'$  strengths, because the consolidation would be different.

It was agreed that the use of Indices values is questionable. It was suggested that for soils with low Plasticity Index, however, that the soil was extremely variable, and since there appeared numerous clay layers, albeit thin, the vane was used.

### (b) Oedometer Tests

In essence E. M. Peto Associates Ltd. omitted to state in the report whether the soil was normally consolidated or overconsolidated. This is correct. The curve used was an uncorrected laboratory curve. Mr. Soderman felt that this was wrong in this particular instance, since the soil was apparently normally consolidated.

In reply, Mr. Lewicki stated that his main reason for not correcting the curve was based on the extreme variability of the materials.

There was no question regarding the heterogeneous assortment and stratification of the soils (silty clay, clayey silt, and silt). A wide variation in the Preconsolidation pressures was obtained and we did not consider that there was sufficient evidence to prove that the soil was in fact normally consolidated.

The uncorrected e-log p curve was used only because the materials were so mixed.

It was agreed that for normally consolidated clays the curve should be corrected.

With regard to the question of settlement, it was agreed at the meeting by E. M. Peto Associates Ltd. representatives, that there was no explanation in the report why the settlement would be rapid, even though impermeable layers are mentioned. Mr. Soderman stated that the permeability values given are apparently in the silt range, and would thus appear to be permeable.

Mr. Lewicki in reply stated, that if the statements were lifted from the report piecemeal they are obviously contradictory.

He went on to say that the initial samples exhibited quite a range of permeability from pervious to impervious. The "k" values were obtained from Cv values and therefore, they should agree. However, due to the great variability of the material, although a rapid rate of consolidation may be expected, the permeability may still, in his opinion, range from low to impermeable. He contended that the settlement would be fairly rapid because of the many layers available for the dissipation of pore water, but agreed that the reasons were not given in the report for arriving at these conclusions. The statement that the material is in the range of low permeability to virtually impervious applies strictly to the higher applied pressures. This was not stressed in the report.

### iii) BEARING CAPACITY EVALUATIONS

#### Footings

There was a difference of opinion in the choice of the shear strength value for the calculation of the bearing value at the proposed footing elevation. We had no criticism to make of Mr. Soderman's choice of shear strength of 2250 lbs./sq. ft. compared with the 2100 lbs./sq. ft. given in our report. The reason we chose the lower figure was because we felt that this figure was substantiated by a greater number of results at the lower elevation. It was quite true that no actual shear value was available at the proposed footing elevation, but the assumption was made that it could not be lower than 2100 lbs./sq. ft.

In the original and the revised bearing value calculations, a value of depths was assumed. In the discussion, it was agreed that the peat layer overlying the footing level will not help appreciably to resist the shear failure and therefore, D should be regarded as 0.

### iv) EMBANKMENT STABILITY


- (1.) We assumed that cohesive material would not be used in the embankment. We also assumed that the shear strength of the material to be placed would not be less than the shear strength of the underlying soil. These assumptions were made since we did not know what material would be used for the fill.

- (2) We clearly agree with Mr. Soderman's contention that 11 trial circles were not necessary since there was an adequate factor of safety. We erred on the conservative side mainly because we were anxious to provide contours of factors of safety, as requested in previous jobs.

We trust that these views are on the lines discussed.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,

  
D. H. Hitchins.

DHH/ajm



23-62-66.

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*met*

THE DEPARTMENT OF HIGHWAYS OF ONTARIO

*23-62-66*

*Sept 1 to*  
*Sept 1 -*  
*4 m*

*N.P. 42-60*

SOILS REPORT

*1.1*

EASTRESS CREEK BRIDGE

TOWNSHIP OF HARTWRIGHT

*1.1.1.1*

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  - 2. 59249A (A & B)
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# e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 58245

1287 caledonia road,  
TORONTO 19, ONTARIO.  
RUssell 9-1128

April 1, 1961.

Department of Highways of Ontario,  
Soils & Foundation Engineering Branch,  
c/o Parliament Buildings,  
Ottawa, Ontario.

Attention: Mr. E. Peaker, P. Eng.

re: Soil Site Investigation  
Wassross Creek Bridge  
Township of Carleton Place  
O.P. 42-80

Dear Sirs:

We have pleasure in submitting herewith ten (10) copies of our soil report.

The following report is divided into two parts:

- |          |  |
|----------|--|
| Part I:  | covers the initial investigation which did establish the soil stratification and the shear characteristics of the subsoil,                               |
| Part II: | the additional investigation comprised of two test holes 1 and 2 required in order to obtain more detailed consolidation characteristics of the subsoil. |

The report covers a detailed description of the soil conditions encountered, the various properties and shear strength of the soil (Part I) and the detailed analysis of the consolidation characteristics of the stratum of interest (silty clay, clayey silt and silt) (Part II). Following each part there is a section dealing with the observations and conclusions of that part of report. For your convenience, we summarize below the observations and conclusions of the report as a whole.

1. The soil conditions, specifically the soil stratification appears to be uniform over the site investigated. The following main types of soil were encountered:

- (a) peat
- (b) sand and gravel
- (c) interbedded silty clay, clayey silt and silt
- (d) silty to coarse sand
- (e) silt

The detailed description of the location and the characteristics of the first three strata is given in Part I and of the silty to coarse sand and silt, in Part II of this report.

The soil profiles as given on the attached site plan show all these soil strata.

2. The water conditions were found to be as follows:
- (a) a perched water table exists on top of the impermeable layer of interbedded silty clay, clayey silt and silt, i.e. in the layer of peat. The ground water level was, at the time of the investigations, about 1 foot below the existing grade. Due to the presence of the creek, which is known to carry much higher water levels than encountered during the investigation, the adjacent area may be subjected to flooding.

- (b) artesian water was observed to exist at different depths in the stratum of interbedded silty clay, clayey silt and silt, with the artesian head well above the present grade.

Artesian water was also encountered in the layer of silty to coarse sand in the area of test hole ...

The detailed water level readings for each part of the report are given in Appendix III of Part II of the report.

3. Bridge Structure

- (a) It is understood that a single span bridge of some 20 feet span is proposed over the Lasticas Creek.

In the following report an analysis for the shear strength and consolidation and compression characteristics of the soil was made for different types of foundations.

3. Bridge Structure: Cont'd.

(b) Footing design

The allowable bearing values for different proposed foundation elevations are given in Part I of this report. The allowable bearing values, depending on foundation depth and foundation width, were varying from 9000 p.s.i. to 2400 p.s.i. These allowable bearing values take into account only the shear characteristic of the soil (Factor of Safety of 3 against shear failure).

The settlement analysis did show that with the above bearing values and an assumed foundation of 15 feet by 60 feet at foundation elevation 822 a minimum settlement of 3.73 inches may be expected. The maximum settlement will probably be 7.8 inches.

The reduction of bearing values will decrease only slightly the amount of settlements, and for 7.8 kips per sq.ft. bearing value, a settlement of 5.52 inches was calculated.

(c) Pile Foundation

1. Friction piles

A friction pile foundation may be employed which will permit much higher bearing values. The length of the friction piles may be obtained by using the minimum value of the cohesive shear parameters as obtained from triaxial tests with pore water measurement. The minimum value was

$$c = 432 \text{ p.s.i.}$$

Due to danger of penetrating the source of artesian water at lower depths of the stratum of interbedded silty clay, clayey silt and silt, a limitation as to the length of the friction piles is recommended. The adverse effect of the artesian pressure on a friction pile is well known, and in addition to reducing the bearing capacity of such a foundation, the additional settlements in the lower reaches of the interbedded silty clay, clayey silt and silt layer, if the artesian pressure is reduced, may contribute to total settlements which will be far in excess of those calculated.

### 3. Bridge Structure

-4-

#### (c) Pile Foundation

##### 1. Friction piles. Cont'd.

A settlement calculation of a friction pile foundation, assumed to have the same peripheral area as the footing foundation and with piles some 28 feet in length, gave as a theoretical settlement of 5.25 inches. Thus it may be seen that from a settlement aspect there is little difference between a spread footing and friction pile type of foundation.

Hence, the only advantage of a friction pile foundation is the increased bearing capacity obtained by this type of foundation design.

##### 2. Bearing piles

The additional investigation did show the presence of an extremely dense stratum of silty to coarse sand, at a depth of 57'8" below grade in test hole ... Assuming that the stratification is more or less horizontal - the use of bearing piles is possible.

The problem associated with the artesian water at these depths will have to be overcome, if a bearing pile foundation is contemplated; i.e. the piles have to be designed to resist the hydrostatic uplift pressures, and secondly the piles will have to be designed against possible buckling, since the hydrostatic pressure will remove any lateral soil support normally available to such piles. In addition, the piles will be subjected to negative friction arising from the settlement of the stratum carrying the embankment loads and from the release of the artesian pressure.

#### 4. Embankment

- (a) We understand that the proposed approach embankment will be some 7 feet above the existing grade.
- (b) The peat deposit as found at the site investigated will have to be either completely removed, or partially removed and the rest displaced by any method as described in the observations and conclusions at the end of Part I of the report.

4. Embankment: Cont'd.

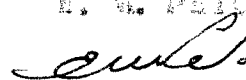
- (c) A stability analysis carried out, assuming that the whole of the peat deposit will be removed and thus the embankment will be 25 feet high, showed that the embankment with side slopes 1 in 1 is stable with a Factor of Safety in excess of 3.
- (d) The settlement analysis performed for the above embankment gave theoretical amounts of settlement of 16.2 inches. The total settlement will be completed in 6.5 years.
- (e) In view of the fairly long term settlement the placing of additional surcharge fill is proposed (subject to the stability analysis) and thus reduce considerably the time required for 100% consolidation.

The height of the additional embankment required to produce 16.2 inches of settlement in the required time may be easily calculated from the attached graphs of  $e$ -log  $p$  curve and  $C_v$  versus  $p$  curve.

- 5. In order to minimize the settlement effect due to the embankment loads on the bridge structure, placing the embankment fill first is proposed followed by the addition of the safe surcharge fill; following this, there will be a lapse of time sufficiently long to allow the major part of the settlement to occur; after which the bridge (spread footing or pile foundation) may be constructed.
- 6. In view of the large amounts of settlement, consideration may be given to the construction of culverts at the above site. If the installation of one culvert is not sufficient to handle the water flow, a series of culverts may be installed which incorporated with the embankment will lessen the differential settlements, which otherwise have to be expected if a bridge structure is constructed.
- 7. A floating type of structure may be considered for this site. This will eliminate or largely reduce the settlement of the bridge, however, this solution does not remove the settlement of the embankment.

We trust we have covered all the points connected with the placing of an embankment and bridge structure at the above site. Should you have some questions in connection with this report, however, we shall be very glad to be of further service.

Yours very truly,  
E. M. PETO ASSOCIATES LTD.



E. M. Peto, P. Eng.

BL/vs  
encl.



**Figure 1**

Diagram illustrating the experimental setup for measuring the effect of temperature on the rate of reaction between hydrogen peroxide and potassium iodide.

The diagram shows a test tube containing a mixture of hydrogen peroxide ( $H_2O_2$ ) and potassium iodide ( $KI$ ). The test tube is placed in a water bath at a specific temperature. A gas syringe is connected to the test tube to measure the volume of oxygen gas produced during the reaction.

The reaction is represented by the following equation:

$$H_2O_2(aq) + 2KI(aq) \rightarrow I_2(aq) + 2KOH(aq) + O_2(g)$$

The experiment involves measuring the time taken for a fixed volume of oxygen gas to be produced at different temperatures, allowing the calculation of the rate of reaction.

Job No. 59249

Client's Ref. No.

Date January 12th, 1960

Report on  
SOIL SITE INVESTIGATION  
EASTCROSS CREEK BRIDGE  
TOWNSHIP OF CARTWRIGHT

for

THE DEPARTMENT OF HIGHWAYS OF ONTARIO

1. INTRODUCTION:

The investigation was authorized by Mr. A. Rutka, P. Eng. Acting Materials and Research Engineer, Materials and Research Section, Ontario Department of Highways, in a letter dated December 8th, 1959.

It was proposed to put down six test holes; three on each side of the Eastcross Creek. Two test holes on each side were to be close to the Creek in order to determine the soil stratigraphy and its characteristics in the vicinity of the abutments of the proposed bridge (Test holes 1, 2, 3, and 4). The two remaining boreholes (5 and 6) were to be put on the centre line of the proposed realignment of Highway 7A at locations further away from the proposed bridge in order to provide information for the proposed approach embankment. Each test hole was preceded by a Dutch Cone penetration test. In addition 7 probe holes were put down (three on the East side and four on the West side of the Creek) to find the depth of the peat stratum.

2. PROGRAMME OF WORK:

December 11th, 1959: Field engineer arrived on the site. Test holes located and elevations determined. Field crew and equipment moved onto site. Testhole 1 commenced.

December 12th, 14th and 15th, 1959: Test hole 1 continued.

December 16th, 1959: Test hole 1 completed. Test hole 4 commenced.

December 17th, 1959: Test hole 4 continued.

December 18th, 1959: Test hole 4 completed. Test hole 6 commenced.

December 19th, 1959: Test hole 6 continued.

December 21st, 1959: Test hole 6 completed. Test hole 5 commenced.

## 2. PROGRAMME OF WORK: (Cont'd)

December 22nd, 1959: Test hole 5 completed. Test hole 2 commenced.  
December 23rd, 1959: Test hole 2 continued. Senior Engineer visited the site.  
December 24th, 1959: Test hole 2 continued.  
December 28th, 1959: Test hole 2 completed. Test hole 3 commenced.  
December 29th and 30th, 1959: Test hole 3 continued.  
December 31st, 1959: Test hole 3 completed. Water levels checked. Crew and equipment moved off the site.  
January 5th, 1960: Crew and equipment returned to site. Field Engineer located probe holes. Probe holes 7 to 13 completed. Field crew left the site.

## 3. GENERAL INFORMATION:

- a) The site plan, accompanying this report shows the location of the test holes and the assumed soil sections.
- b) Since no other Bench Mark could be found, the elevations as shown on the attached site plan, in the borehole logs and the following report were derived from the elevation of the top of the opening of the South end of the culvert at Chainage 359 + 97. The drawing of D.H.O. No. C -506-3 (Profile) gives the elevation of the top of the opening of this culvert at elevation 836.6.
- c) The detailed description of the soils encountered is given on the individual borehole logs at the end of the report.
- d) The graphical representations of the field test results: Standard Penetration Test versus Elevation; Dynamic Cone Penetration Test versus Elevation and the Natural Moisture Contents versus Elevation are given in Appendix I.
- e) Appendix II contains the laboratory test results:
  - 1. Atterberg Limits.
  - 2. Mechanical analyses.
  - 3. Shear strength tests, and
  - 4. Consolidation Tests.are given in detail and also in a graphical form versus elevation.
- f) Various shear strength relationships as derived from shear strength tests are given in Appendix III.
- g) The test holes were put down in accordance with our standard procedure as outlined in Appendix IV.

#### 4. SITE AND GEOLOGY:

The site is located on Highway 7A approximately 6-1/2 miles ~~West~~ East of Highway 35.

The bridge is situated in the bottom of the Eastcross Creek valley running in a South-Northerly direction. The Eastcross creek flows Northward, parallel to the East shore of Lake Scougog until it joins the Scougog river some 5 miles South of Lindsay.

The entire area is underlain by a Cobourg formation of late Trenton age. In its lower part it consists of dark grey and greenish grey, fine-grained, argillaceous, fossiliferous limestone, whereas the upper part (uppermost few feet) comprise a denser, grey limestone. There are numerous shale partings throughout the Cobourg formation.

The streams such as Pigeon and East Cross head in the Oak Ridges moraine and flow North to join the Trent River System. These streams flow mainly on either outwash gravels or Scotomberg Lake deposits. The valleys in which these streams flow reach a maximum width of a mile and a depth of about 250 feet. The presence of outwash in some of the valleys, and the fact that the present streams are small; suggest that these valleys were formed by some glacial drainage. The deposits therefore as found in these valleys are of glacio-fluvial or glacio-lacustrine origin. Most of these deposits are poorly stratified silts and clays, although varved clays have also been found in a few localities.

#### 5. SOIL CONDITIONS:

During the present investigation the following soil strata were encountered.

- a) Peat
- b) Grey sand and gravel, and
- c) Interbedded silty clay, clayey silt and silt.

The details of the characteristics and the location of each stratum are as follows:

- a) Peat

This uppermost organic deposit was found over the whole area investigated. The depth of muskeg deposit was found to be:

Test hole 1 - 9'4"  
Test hole 2 - 14'0"  
Test hole 3 - 14'0"  
Test hole 4 - 9'6"  
Test hole 5 - 18'9"  
Test hole 6 - 8'6"

## 5. SOIL CONDITIONS:

### a) Peat (Cont'd)

Thus the depth of muskeg increased in the Easterly direction and was deepest at test hole 5. In order to determine the depth of this deposit in the area which will be affected by the approach embankment several probe holes were sunk East and West of the proposed bridge (See section through the probe holes and test holes on the attached site plan). It was found that the depth of peat decreased more in the Westerly direction than in the Easterly.

The depth of this deposit in the probe holes was as follows:

Probe hole #10 - 4'0"  
 Probe Hole #13 - 4'0"  
 Probe hole #12 - 4'0"  
 Probe hole #11 - 3'0", and

in the Easterly direction;

Probe hole #7 - 3'0"  
 Probe hole #9 - 3'0", and  
 Probe hole #6 - 4'0".

In order to be able to apply the Radforth Classification System the surface growth covering the muskeg area was observed. It was wooded by spruce, poplar, cedar and birch which were over 15 ft. in height. According to the Radforth Classification System, this designates the "A - Coverage Type", and according to the topographic classification an "m" Contour Type. The Munsell reference formula was mostly 10YR 3/2.

The natural moisture content of the peat was found to vary between 1689% and 208%.

The standard penetration test results gave N-values varying between 1 and 2.

A field vane test conducted on the peat in test hole 3 gave an average value for the shear strength of 600 p.s.f.

A consolidation test performed on a sample taken from 2'3" below grade in test hole 4 (elevation 825.45) gave the following results.

Bulk density  $\gamma_w = 65.5$  lbs. per cu. ft.  
 Dry density  $\gamma_d = 9.5$  lbs. per cu. ft.  
 Void ratio  $e = 6.55$   
 Porosity  $n = 87\%$   
 Degree of Saturation  $S = 100\%$ , and  
 The compression index  $C_c = 1.53$

$w\% = 690\%$

## 5. SOIL CONDITIONS: (Cont'd)

### b) Grey Sand and Gravel

Underlying the peat deposit a thin layer of grey sand and gravel was met. The depth of this layer was found to vary especially West of the proposed bridge, where the thickness of the deposit was only 1 foot.

The detailed position of the grey sand and gravel was established as follows:

Test hole 1 from 9'4" to 10'10" below grade.  
Test hole 2 from 14'0" to 21'0" below grade.  
Test hole 3 from 14'0" to 21'0" below grade.  
Test hole 4 from 9'8" to 10'8" below grade.  
Test hole 5 from 18'8" to 23'0" below grade.  
Test hole 6 from 8'8" to 12'0" below grade.

Thus, the grey sand and gravel stratum dipped in the Easterly direction, and was thickest just East of the Hagteross bridge (Test hole 2 and 3).

The natural moisture content varied between 9.0% (test hole 5) and 17.5% (test hole 4). The latter value of the moisture content was probably affected by the presence of the underlying silty clay stratum.

Generally the grey sand and gravel stratum was compact with an N-value varying between 12 and 17, and an average N-value of 15.

### c) Interbedded silty clay, clayey silt and silt.

Following the thin deposit of sand and gravel, there was an extensive stratum of interbedded silty clay, clayey silt and silt. The stratification between the separate soil fractions was found to be highly irregular, with the stratification planes in all directions.

Sampling was found to be extremely difficult, and in many instances the samples were lost. The sample disturbance was observed to be of great magnitude. To be able to obtain more reliable results some of the tests were performed directly on the site, very shortly after extraction of the samples. The irregular stratification made the recovery of "representative samples" especially for consolidation tests, extremely difficult.

## 5. SOIL CONDITIONS:

### c) Interbedded silty clay, clayey silt and silt (Cont'd)

The nature of the variation of the separate soil fractions may be seen in the results of the mechanical analyses. The tests performed on samples from borehole 1 produced the following results:

Depth below Grade	Percent of			Textural Classification
	Sand	Silt	Clay	
12' - 13'	6	79	11	Silt
32' - 33'	2	28	70	Clay
45'6" - 46'6"		86	14	Silt

After obtaining the following results no further gradation tests were conducted, as it was considered that the results would lie between the limits as obtained in test hole 1. Thus, as may be seen, the material composing the lowest stratum contained layers of clay and silt with some intermediate seams such as clayey silt.

The Atterberg Limit tests showed that the natural moisture content of the material, in the upper portions of the stratum, was wetter than the plastic limit, but drier than the liquid limit. With increasing depth the moisture content approached, and even was in excess of, the liquid limit (test hole 5 elevation 791 to 789). The detailed test results were as follows:

Test Hole	Depth Ft.	Elevation	Nat. M.C.	Percent of		
				L.L.	P.L.	P.I.
1	20' - 21'	815.9	20.3	21.5	14.8	6.7
5	25' - 26'	810.0	21.8	20.4	15.2	5.2
5	30' - 31'	805.0	33.4	52.3	25.8	27.2
1	35' - 36'	800.9	23.7	34.5	18.0	16.5
5	35' - 36'	800.0	24.4	35.5	18.3	17.2
5	40'6" - 41'6"	794.5	27.9	37.3	18.9	28.4
5	45' - 46'	790.1	23.4	22.7	18.5	5.2
5	50' - 51'	785.0	22.9	20.2	17.5	2.7
5	55' - 56'	780.0	22.3	21.2	18.9	2.5
5	59' - 60'	775.0	23.1	23.0	17.6	5.4

## 5. SOIL CONDITIONS:

### c) Interbedded silty clay, clayey silt and silt (Cont'd)

Further, it may be seen (see graph of Atterberg's Limits and Average curve of natural moisture contents versus elevation in Appendix II) that the Plasticity Index increases to elevation 805 where it has a value of 27.2% and then decreases to elevation 780 with a minimum observed Plasticity Index of 2.3%. The maximum value of the Liquid Limit was observed at elevation 805 just where the natural moisture content reached its peak with a value of 38.5% (average moisture content). The corresponding value of Liquid Limit was 52.2%. According to Casagrande's Classification system the interbedded silty clay, clayey silt and silt stratum is, at its upper portion, an SF-CL soil changing to CH-silt at some 25 to 30 feet below grade, then a further change occurred to a CH-silt and gradually transforming to an SF-CL silt, and even an ML-silt, at some 50 feet below the existing grade.

The natural moisture contents were found to be fairly uniform throughout the area investigated at the same horizontal places. With depth a variation was observed. The moisture content at first tended to increase to elevation 805, from a value of about 20% (average value) to about 30.5% at elevation 805. From this depth a gradual decrease in moisture content was observed to elevation 790, where an average moisture content of 21.5% could be assumed. From this depth a nearly constant moisture content of about 21.5% with depth was observed.

To define the shear strength profile a number of different shear strength tests were conducted. The results of these tests are given in graphical form in Appendix II. It may be seen that the results as obtained from the unconfined compressive tests, the field vane tests and the laboratory triaxial tests agree fairly well.

The laboratory undrained vane test results had to be discounted to the considerable influence of sample disturbance.

From the graph of shear strength versus elevation it may be seen that the shear strength decreases with depth to elevation 782, then a nearly constant shear strength value to elevation 787 exists, and from there on again a decrease in shear strength is evident. The value of the shear strength (undrained shear strength) may thus be given as follows:

at elevation 810 - 2100 p.s.f., decreasing  
at elevation 782 - to 1000 p.s.f. and constant  
to elevation 787; from elevation 787 increases  
resulting at elevation 750 a value of 2000 p.s.f.



## B. SOIL CONDITIONS:

### c) Interbedded silty clay, clayey silt and silt (Cont'd)

The ratio of  $\sigma_u/\sigma_o$  (undrained shear strength over overburden pressure) versus Plasticity Index, P.I. showed that with increasing P.I. the ratio of  $\sigma_u/\sigma_o$  also increased. For a P.I. of 5, the ratio of  $\sigma_u/\sigma_o$  was found to be 0.6, and for a P.I. of 10 the  $\sigma_u/\sigma_o$  was 1.0.

The ratio of  $\sigma_u/\sigma_o$  plotted versus elevation showed a similar trend as the shear strength versus elevation, i.e. a decrease was observed to elevation 792, the ratio of  $\sigma_u/\sigma_o$  at this point was 0.55. From elevation 792 the increase of  $\sigma_u/\sigma_o$  with depth was then seen.

The decrease in shear strength at elevation 792 could also be seen from the results of the Standard Penetration tests, but not to the same degree. Generally, the N-value at elevation 815 varied between 9 and 14 with an average N-value of 12 increasing to 16 at elevation 810 and decreasing to 14 at elevation 792. A very slight increase in overall density from elevation 792 could be observed, although a value of 7 blows per foot penetration was also recorded at test hole 3 at elevation 788.5. This apparent very low N-value was due to the presence of "thicker" seams of clay at this point. The difference in density between the separate seams of clay, and silt respectively was very pronounced at the lower depths of this stratum.

Three sets of triaxial shear tests were performed on samples taken from test hole 6. The tests were the unconsolidated undrained type with pore water measurements, i.e. U.U.-test.

<u>Depth</u>	<u>Elevation</u>	<u>c</u>	<u><math>\phi'</math></u>
20'0"	815.5	1730	28°
30'9"	804.5	590	25°
39'7"	795.5	430	24°

Plotting these values versus elevation it may be seen that the c'-value approaches asymptotically a value of 420 p.s.f. which may be assumed to be a minimum. Similarly, the minimum  $\phi'$ -value may be assumed to be 23°.

As mentioned previously in the report, the sample disturbance and the stratification did not allow a sufficient number of consolidation tests to be performed. Only two test results were thought to be least affected by these factors, and these results were as follows:

## 5. SOIL CONDITIONS:

c) Interbedded silty clay, clayey silt and silt (Cont'd)

Test Hole	Depth	Elevation	$\gamma_w$	$\gamma_d$	Void Ratio, $e$	Porosity, $n\%$	Compression Index $C_c$
4	27'3"	807.45	131.0	107.0	0.600	37.5	0.055
4	37'2"	797.50	126.0	107.0	0.665	40.0%	0.091

According to these results the soil is of slight to medium compressibility.

## 6. WATER CONDITIONS:

The following water level readings were taken during the investigation.

Hole #	Date	Time	Depth of Hole	Depth of Casing	Other Circumstances	Depth to water
1	Dec. 11th		3'0"	2'0"		6"
		4:00 p.m.	6'0"	5'0"		14"
			10'0"	8'0"		None
			13'0"	12'0"	After bailing out Fine sand backing up 2 to 3 feet, in casing.	None
			16'0"	15'0"		None
	Dec. 12th		21'0"	20'0"		None
			31'0"	30'0"		None
		11:50 a.m.	31'6"	30'0"		None
		1:15 p.m.	31'6"	30'0"		None
		6:00 p.m.	41'0"	40'0"		None
	Dec. 14th	7:50 a.m.	41'0"	40'0"		None
	Dec. 15th				Artesian flow about 6" head above surface.	
					Artesian flow continued through the day	
	Dec. 16th	8:00 a.m.	56'0"	75'0"	Slight Artesian flow	
		10:00 a.m.	51'0"	75'0"	10 - 12 ft. Artesian head.	
					30 ft. casing left in the test hole.	
					Artesian flow continued throughout the period of the investigation.	
2	Dec. 28th	10:00 a.m.	58'6"	46'0"		None
	Dec. 29th	8:00 a.m.	58'6"	46'0"		1'0"
3	Dec. 31st	8:00 a.m.	70'0"	57'0"		1'0"
4	Dec. 16th	11:45 a.m.	56'0"	32'0"		26'8"
		1:10 p.m.	56'0"	32'0"		24'0"
		2:00 p.m.	56'0"	20'0"		24'0"
5	Dec. 22nd	5:45 p.m.	60'0"	48'0"		None
6	Dec. 21st	11:10 p.m.	61'0"	34'0"		15'0"
		11:30 a.m.	61'0"	34'0"		17'0"

## 6. WATER CONDITIONS: Cont'd.

According to the above observations the following conclusions may be reached regarding the water conditions:

1. A perched water table exists on top of the impermeable stratum of interbedded silty clay, clayey silt and silt, which at the time of the investigation was 1 ft. below the existing grade.
2. In the area of test hole 1, an artesian water condition existed, with the probable Artesian water source some 45 feet below the existing grade. The Artesian head was about 57 feet, i.e. 12 feet above the existing surface.
3. The grey sand and gravel layer is fully saturated due to the character of the overlying deposit.

## 7. OBSERVATIONS AND CONCLUSIONS:

### a) General

1. The soil conditions at the site investigated were found to be reasonably uniform:

Underlying the peat deposit varying in depth from 8'6" to 18'9" there was a thin stratum of grey sand and gravel followed by an extensive stratum of interbedded silty clay, clayey silt and silt.

The last mentioned stratum exhibited very irregular stratification between the separate soil fractions.

2. The undrained shear strength of the stratum of the interbedded silty clay, clayey silt and silt was found to decrease with depth from a value of 2100 p.s.f. at elevation 810 to 1200 p.s.f. at elevation 792, which represents the minimum undrained shear strength of this stratum.

The minimum shear parameters at elevation 795 were found to be:

$$c' = 420 \text{ p.s.f. and} \\ \phi' = 35^\circ$$

3. A perched water table was found to exist on top of the impermeable stratum of interbedded silty clay, clayey silt, and silt, which at the time of this investigation was 1 foot below the existing grade.

## 7. OBSERVATIONS AND CONCLUSIONS:

### b) Bridge Structure: Cont'd.

#### (b) West Abutment

Proposed Foundation Elevation	Footing Width Ft.	Allowable Bearing Value p. s. f.
826	5	4200
	10	3800
	15	3700
	20	3500
824	5	4300
	10	3900
	15	3800
	20	3500
822	5	4500
	10	4000
	15	3900
	20	3500
820	5	4700
	10	4100
	15	3700
	20	3400
818	5	5000
	10	4300
	15	3800
	20	3500

3. If higher bearing values are required than given above for the footing design - friction pile foundation may be considered.

The unit value of skin friction for piles may be assumed to be uniform for the entire embedded length, and for the soil condition encountered may be taken as 480 p. s. f. (minimum cohesive strength encountered). The presence of artesian water in the area investigated, below elevation 760, limits the length of the friction piles. The hydrostatic uplift pressures resulting from artesian pressures will reduce skin friction to negligible amounts (to far less than the hydrostatic up lift on the piles) and as a result the piles may rise.

The neutral pressures at the bottom of the interbedded silty clay, clayey silt and silt layer cannot exceed the total pressures, since such a condition would cause the uplift of the entire mass of the overlying deposit.

## 7. OBSERVATIONS AND CONCLUSIONS:

### a) General

3. An artesian water condition exists at the site. In the area of test hole 1 the artesian water source was encountered some 45 feet below the existing grade, whilst in the area of test hole A (see part II of this report) it was found at 56 feet and again at 87 feet below grade; in the area of test hole 3 it was some 82 feet below the existing grade.

### b) Bridge Structure

1. We understand that a single span bridge structure is proposed of approximately 40 feet span.

2. The allowable bearing values from the shear strength consideration of the subsoil (Factor of Safety of 3 against shear failure) may be taken as follows:

#### (a) East Abutment

Proposed Foundation Elevation	Footing width Ft.	Allowable Bearing Value p. s. f.
322	5	4300
	10	3900
	15	3800
	20	3700
320	5	4700
	10	4100
	15	3700
	20	3400
318	5	4500
	10	4300
	15	3900
	20	3500
316	5	4500
	10	4300
	15	3700
	20	3400
314	5	4200
	10	4200
	15	3800
	20	3400

## 7. OBSERVATIONS AND CONCLUSIONS:

### b) Bridge Structure: Cont'd.

3. At any depth the intergranular pressure will be equal to the overburden pressure minus the artesian pressure.

Any change in the conditions which lead to a decrease in the artesian pressure, may cause large amount of settlement, and thus may produce negative skin friction.

The most suitable piles to resist uplift are those with the largest perimeter. If H piles are used the effective friction surface should be taken equal to the outside bounding dimensions. Core stoppers are used sometimes in the webs of H piles to reduce uplift action.

The use of tapered piles is not recommended since they may not fill the hole after an upward movement.

4. Due to the presence of an extremely dense sand layer at 97 feet below the existing grade, as encountered in test hole A (see part II of the report), i. e. at elevation 738, a bearing pile type of foundation may be used, provided due attention is given to the existing artesian pressures, and its effect on the bearing capacity of the piles.

Neglecting the effect of the artesian pressures, the allowable bearing value of the soil at elevation 738 may be taken as 4.0 tons per sq. ft.

### c) Embankment

1. We understand that an approach embankment is proposed which will be some 7 feet above the existing grade. The assumed elevation of the top of the embankment is 942.2.

2. As the site is overlain by the unstable peat deposit, it is essential either to remove it or to ensure complete consolidation before the fill is placed.

3. The following methods are recommended:

- (a) the total excavation of the peat deposit
- (b) partial excavation and then partial displacement
- (c) total displacement

(a) This is the soundest method. The filling of the material should be kept close behind the excavation in order to reduce the possibility of the sides of the excavation slipping. The removal of peat deposit may be accomplished, either by using excavators, or by using suction pumps.

## 7. OBSERVATIONS AND CONCLUSIONS:

### (c) Embankment: Cont'd.

- (b) For depths greater than 10 feet and particularly where the peat is very fibrous, excavation to a depth of about 10-12 feet is recommended, followed by displacement of the lower part of the peat by the superimposed weight of the soil in the embankment. If the lower part of the peat is too solid to flow under the weight of the embankment it may be necessary to use jetting.

The displacement may be accomplished by any blasting method (trench-shooting, toe shooting or bog-blasting method).

4. A stability analysis of the proposed embankment was carried out (see the attached drawing of slope stability analysis). In the stability analysis, the following assumptions were made:

- (a) The potential sliding surface will be a cylinder.
- (b) The analysis is two dimensional.
- (c) The shear strength (undrained shear strength) is constant within each layer and as defined by the shear strength contour of the soil.
- (d) At the moment of failure the shear strength is completely mobilized at every point along the sliding surface.
- (e) The undrained shear strength of the embankment is equal to 2000 p. s. f.
- (f) The full pore pressures within the soil mass will develop when the full surcharge of the fill is placed.
- (g) The minimum Factor of Safety will exist at the end of the construction period.
- (h) As a rapid rate of loading will exist, the assumption is made that no drainage will take place and thus the shear strength of the soil is represented by its undrained shear strength.
- (i) The ground water table in the embankment will be at the same level as in the surrounding peat deposit, i. e. at elevation 834.0. Further, the resisting effect of the peat layer at the toe of the embankment was completely neglected in the stability analysis. The analysis gave a minimum Factor of Safety for the 21 feet high embankment, with side slopes of 1 in 1, of 3.62. The analysis showed further that the minimum equilibrium shear strength, i. e. the shear strength required for Factor of Safety = 1, was 510 p. s. f.

PART II



## PART II

### 1. INTRODUCTION:

The results of the first investigation did not permit an extensive and detailed analysis of the consolidation characteristics of the subsoil, especially of the interbedded silty clay, clayey silt and silt stratum. After consultation with Mr. K. Peaker, Professional Engineer, of the Department of Highways of Ontario, an additional investigation was called for in order:

- (1) to determine the consolidation characteristics of the stratum of interbedded silty clay, clayey silt and silt, and
- (2) to find, if possible, a dense bearing stratum for an end-bearing pile-type of foundation.

The time of this additional investigation coincided with the most adverse weather conditions. A raft had to be used in order to permit the drilling operations in view of sudden variations in temperatures. The fluctuation in temperature put on and

SOIL CONDITIONS:

The additional investigation consisted of two test holes A and B, one on each side of the Easteross Creek at the proposed location of the bridge abutments. The location of the test holes is shown on the attached site plan, together with the assumed soil profile.

As may be seen from the attached soil profile and the bore-hole logs the soil conditions at test holes A and B were very similar to those previously encountered.

The stratum of interbedded silty clay, clayey silt and silt extended to a depth of 97'6" at test hole A. The N values and the natural moisture contents were of the same order as described in part I of this report. A decrease in the N value was observed at a depth of about 50 feet below the existing grade (elevation 795) which corresponds to the observation regarding the shear strength distribution with depth as described in Part I of this report.

The additional investigation did show the presence of an extremely dense sand stratum. This stratum was found to exist between 97'6" and 115'0" below the existing grade in test hole A. The N values were as high as 480 at the upper limit of this layer, decreasing to 90 at its lower boundary. The layer of fine to coarse sand, containing some gravel and even stones, grey in colour, was the source of the artesian water. The presence of water may also be seen from the values of the natural moisture contents which were varying between 14.7% and 17.6%.

# SOIL CONDITIONS: Cont'd.

Underlying the sand stratum, from 115 feet below grade, a layer of grey silt with small layers of clayey silt, and seams of fine sand was encountered. The silt stratum was very hard with the N values decreasing with depth. A maximum N value of 95 was recorded at 120'0" below the grade. At a depth of 148'8" it decreased to 23, (48 blows per 2 feet of penetration) with the decreasing N value the natural moisture content was found to increase. At the upper boundary of the silt stratum the natural moisture content of 19.7% was recorded. At a depth of 141'6" the moisture content increased to 22.3%. At 133'0" below the existing grade, the silt stratum changed to clayey silt with grits and pebbles. This layer was similar in character to the interbedded silty clay, clayey silt and silt layer as found to 97'3" depth.

In test hole B no sand layer was encountered as in test hole A. Extremely difficult water conditions (artesian water) made the advancement of the test hole practically impossible, and test hole B was terminated at a depth of 92'4".

## WATER CONDITIONS:

Similarly to the previous investigation, a perched water table was established in the peat layer. The water level was 3" below the existing grade at test hole A, and 1'3" below the existing grade at test hole B.

Artesian water was observed at the following depths:

<u>Test Hole A</u>	55'0" with artesian head of 61 feet
67'10"	" " " 73 feet
72'0"	" " " 78 feet
97'0"	" " " 103 feet
109'6"	" " " 114 feet
116'0"	" " " 120 feet
144'0"	" " " 147.5 feet
<u>Test Hole B</u>	92'0" " " " 97'0"

Accordingly, the artesian water level is some 5 feet above the existing grade, i. e. at elevation 846.

The seams of impervious layers did cut off the artesian water. The greatest flow of artesian water arose from a source at 97'3" below grade at test hole A, i. e. at the upper limit of the sand stratum, and at 84'0" at test hole B.

The detailed water level readings are included in the Appendices.

## CONSOLIDATION CHARACTERISTICS:

As mentioned before the main object of the additional investigation was to establish the consolidation characteristics of the stratum of interbedded silty clay, clayey silt and silt.

The sampling in testholes A and B was carried out extremely carefully and the samples were handled and transported to the laboratory with the utmost care. The consolidation tests were conducted immediately after receiving the samples. In spite of such precautions, it was still found that considerable disturbance of the samples existed. Numerous tests were carried out only on samples which were thought to be the least disturbed.

The results of 11 consolidation tests are given in tabular form and the graphical form in the Appendix to this report.

An analysis of each relationship is made as follows:

### (a) e-log p relation

The e-log p curves for various tests have been plotted on one graph in order to illustrate the variation in the e-log p relation.

As may be seen, the initial void ratios vary from 0.825 to 0.500 with  $C_c$  values between 0.113 and 0.086.

A definite preconsolidation pressure could be observed on some of the e-log p curves. The estimated preconsolidation pressures varied, depending on the depth of the sample, from 1100 p.s.f. to 10,140 p.s.f.

The other reason for plotting all the e-log p curves on one graph was to determine whether there was some relationship with depth. No such relationship in the e-log p curves, and subsequently, in the  $C_c$  values was seen. It may be seen that the  $C_c$  value is practically independent of depth and for all practical purposes remains constant.

The average  $C_c$  value for the interbedded silty clay, clayey silt and silt layer was found to be 0.084.

In order to arrive at some practical e-log p curve which would enable a settlement analysis to be carried out, an average e-log p curve was derived. This curve is given in the appendix. It may be seen that the average e-log p curve for the whole layer is very similar to the e-log p curve of test no. 9. Casagrande's method of estimating the preconsolidation pressure gave 3000 p.s.f. as the "average" preconsolidation pressure with a minimum probable preconsolidation pressure of 1130 p.s.f.

## CONSOLIDATION CHARACTERISTICS:

### (a) e-log p relation: Cont'd.

The Compressive Index  $C_c$  was 0.082 which was very close to the average  $C_c$  obtained arithmetically.

The initial void ratio of 0.867 is the most probable void ratio of the material.

### (b) Coefficient of volume decrease $M_v$ versus applied pressures

The coefficient of volume decrease  $M_v$  in square foot per minute has been plotted for the average applied pressures for various tests and is shown on graph no. 3 in the Appendix.

It may be seen at once that the value decreases with increasing pressure. Secondly three different "areas" of the rate of the decrease could be seen:

- a) tests 1, 3, 4, 7, 8 and 12
- b) tests 5 and 11
- c) tests 2, 6 and 10.

The group of tests 1, 3, 4, 7, 8 and 12 show a much larger rate of decrease of  $M_v$  with increasing pressure, with the  $M_v$  values as high as 0.0455 sq. ft. per minute (test no. 4, average pressure 0.25 kips per sq. ft.) decreasing to 0.0008 sq. ft. per minute (Test no. 4, average pressure 20 kips per sq. ft.)

The second group of tests (tests nos. 5 and 11) has a nearly linear decrease of  $M_v$  with the increasing pressure on the log p graph, with  $M_v$  values at average pressure of 0.25 kips per sq. ft. of 0.0080 (test 5) decreasing to 0.0011 at the average applied pressure of 20 kips per sq. ft. The last group of tests (tests nos. 2, 6 and 10) shows the lowest rate of decrease.

The attempt to find a relationship of the various groups with any other factor such as depth, the natural moisture content, rate of load application and others, did not produce any definite results. The various rates of decrease are thought to be only due to the difference in the permeability of the samples and thus to the difference in clay or silt content.

From the results of 11 tests an average  $M_v$  versus p curve was constructed and this is shown in graph No. 4. It may be seen that at the average applied pressure of 0.75 kips per sq. ft.  $M_v = 0.0115$  sq. ft. per kips dropping to 0.0011 sq. ft. per kips at  $p = 20$  kips per sq. ft.

## CONSOLIDATION CHARACTERISTICS: Con'd.

### (c) Coefficient of consolidation $C_v$ in sq. inches per minute versus applied pressures

The coefficient of consolidation was calculated using Taylor's fitting method (square root of time fitting method). The various coefficients of consolidation are given in the tabular form in the individual consolidation test results.

No graph is given of the variation of  $C_v$  for various applied pressures for the individual tests as it was found that the scatter was of such a nature that no definite conclusions could be reached regarding the variation of  $C_v$  with applied pressures. Some tests did show a decrease of  $C_v$  of nearly constant rate with the increasing pressure, others an increase, and still another group, an initial decrease, followed by an increase of  $C_v$  with the applied pressure. From the available data an average graph of  $C_v$  versus applied pressures was drawn and is represented on graph no. 5.

It may be seen that the  $C_v$  value decreases from a value of  $53.65 \times 10^{-3}$  sq. inch per minute at an average pressure of 0.75 kips per sq. ft., to  $31.19 \times 10^{-3}$  sq. inch per minute at the average pressure of 3.0 kips per sq. ft.

From this point a slight increase of  $C_v$  follows to a point represented by the average pressure of 12 kips per sq. ft. where  $C_v = 37.82 \times 10^{-3}$  sq. inches per minute and then again  $C_v$  decreases and at  $p = 20$  kips per sq. ft. has a value of  $28.48 \times 10^{-3}$  sq. inches per minute.

These values of the coefficient of consolidation indicate that any settlement will be fairly rapid and will probably be completed inside ten years duration.

### (d) Coefficient of Permeability $K$ , versus the average applied pressures

From the values of the coefficient of consolidation and the coefficient of volume decrease the coefficient of permeability,  $K$  could be calculated. These values are given firstly in the appended tables of the consolidation test results and secondly are represented graphically for various tests on graph no. 6.

apart from the results of tests 8 and 11, good agreement was obtained between the remainder of the tests. Choosing the arbitrary border line between the impervious and pervious soils as being  $10^{-4}$  cm/second, i.e.  $2.36 \times 10^{-3}$  in/min., it may be seen that all the samples tested represent the impervious material.

## CONSOLIDATION CHARACTERISTICS:

### (d) Coefficient of Permeability K, versus the average applied pressures : Cont'd.

apart from the logical decrease of permeability with the increasing pressure, the soil is of very low permeability, and tests 8 and 11 represent a soil which is practically impermeable.

The average curve of the coefficient of permeability versus applied pressures is given on graph no. 7.

### (e) Unit compression versus applied pressures

The relationship between the compression obtained in the individual load increments to the initial height of the sample, i.e. the unit compression was calculated for each test and is given in the tabular form in the results of the consolidation tests.

These values have also been plotted for various applied pressures and are shown on graph no. 8.

Two groups may be observed to exist in the  $\Delta H/H$  versus  $p$  relationship.

1. Group of tests 2, 5, 6 and 10 and
2. Group of tests 1, 3, 4, 7, 8, 9, and 12

Some relationship may be observed between the various groups of unit compression versus pressure and the various groups of the coefficient of volume decrease versus applied pressure. From the relationship of  $M_v$  versus  $p$ , we have seen that the group represented by tests nos. 2, 5 and 10 have the slowest rate of the decrease with increasing pressure. The same group (tests nos. 2, 5, 6 and 10) with addition of test no. 10 shows much reduced increase of the unit compression values with the increasing applied pressures. Otherwise, the rest of the tests show very good agreement. The maximum obtained unit compression was 12.41% (test no. 12, applied pressure 24 kips per sq.ft.

The average  $\Delta H/H$  versus  $p$  curve is represented on graph No. 8.

The unit compression versus applied pressure curve serves in obtaining the Modulus of Compression  $M_c$  and therefrom the settlement. The stress-strain relationship for the soil may be written, using Hook's law in the following form:

$$p = \frac{\Delta H}{H} M_c$$

## CONSOLIDATION CHARACTERISTICS:

### (e) Unit compression versus applied pressures: Cont'd.

Therefore, the settlement  $\Delta H = \frac{A}{H} \times \frac{H}{M} = \frac{A}{M}$

and by a closer approximation  $P_{final} = \frac{\Sigma A}{M}$

where  $A$  is the area of stress distribution (pressure area), i. e. area between the vertical line through the point where the settlement is considered and the line of the vertical stress due to applied load as determined at any depth by Boussinesq or some other method.

Thus to calculate the settlement using Modulus of Compression - the pressure area is divided into smaller areas for a closer approximation. The average pressure of the individual areas is then determined - and from the graph of Modulus of Compression,  $M$  versus applied pressures (graph no. 10) the corresponding Modulus of Compression  $M$  is obtained. Using the equation given above, the settlement for each increment area is then calculated and the summation gives then the total settlement.

## OBSERVATIONS AND CONCLUSIONS:

1. The variable character of the interbedded silty clay, clayey silt and silt stratum manifested itself in the values of the initial void ratios.

The initial void ratios varied between 0.825 and 0.899 with the average initial void ratio of 0.867.

The compression index,  $C_c$ , was found to be more consistent than the initial void ratio and the average value of the compression index was found to be 0.082 with a maximum  $C_c$  of 0.119 and a minimum  $C_c$  of 0.055. No definite relationship between the compression index and depth was established.

The coefficient of volume decrease was found to lie between 0.0455 sq. ft. per minute and 0.0008 sq. ft. per minute, decreasing with the increasing applied pressure. A "grouping" of tests was observed with a similar rate of decrease of the coefficient of volume decrease with increasing pressure, but apart from a similar grouping observed in the relationship between the unit compression versus applied pressure no other specific relation could be established.



OBSERVATIONS AND CONCLUSIONS: Cont'd.

1. The coefficient of consolidation with a minimum value of  $0.11 \times 10^{-3}$  sq. inches per minute suggests that the stratum investigated will have a fairly rapid rate of settlement.

The values of the coefficient of permeability were found to vary between  $3.38 \times 10^{-3}$  sq. inches per minute to  $0.010 \times 10^{-3}$  sq. inches per minute. These values place the layer investigated in the range of low to practically impermeable materials.

2. A settlement analysis using both the change in void ratio due to applied surcharge and the coefficient of volume decrease gave the following amounts of theoretical settlements.

(a) Embankment

It was assumed that the whole of the peat layer will be displaced and thus the height of the proposed embankment will be 25 feet. The width of the embankment was assumed to be 50 feet with a side slope 1 horizontal to 1 vertical ( $45^\circ$  slope) as established by the slope stability analysis carried out in Part I of this report.

The settlement equation using change in void ratio gave a total theoretical settlement 9.0 inches.

The settlement calculated from the  $M_v$  relationship gave a corresponding settlement at 16.2 inches.

The time required to obtain 100% degree of theoretical consolidation was found to be 5.5 years.

(b) Bridge structure

1. Footing design

It was assumed that the footings will be placed at elevation 822, i. e. on top of the thin sand and gravel layer, overlying the stratum of interbedded silty clay, clayey silt, and silt. An allowable bearing value of 3.8 kips per sq. ft. was taken for calculation purposes (the bearing value as given in observations and conclusions at the end of Part I of this report.)



## OBSERVATIONS AND CONCLUSIONS:

2.

### (b) Bridge Structure

#### 1. Footing design: Cont'd.

The change in void ratio equation gave a theoretical amount of settlement of 3.73 inches and using the coefficient of volume decrease and a bearing value of 4.0 kips per sq. ft. gave a settlement of 7.9 inches. Decreasing the bearing value to 3.0 kips per sq. ft. still produced a settlement of 5.92 inches.

#### 2. Pile foundation

The assumption was made that a friction pile foundation will be used and the tip of the piles will be at elevation 782. Assuming also that a load of 2.0 kips per sq. ft. will be (bearing capacity of the pile) acting at a point  $1/3$  from the pile points (elevation 784), the settlement analysis gave the following results:

(a) from the change in void ratio  
settlement: 6.25 inches

(b) from the coefficient of volume decrease  
settlement: 5.45 inches

3.

From the results of the settlement analyses, it is evident that any structure placed upon the existing soil will settle appreciably. Of course, the friction pile foundation will produce less settlements but still the order of the settlement does not warrant the use of friction pile foundation except solely for the purpose of increasing the allowable bearing values.

The danger of inducing the negative friction on the piles, if not sufficient time is allowed for the consolidation of the soil to take place due to weight of the embankment, and further the danger of piercing the artesian water source which also will produce appreciable settlements in addition to the settlements due to surcharge, makes the use of a friction pile foundation a very doubtful solution.

The advantage of a friction pile foundation may lie in the disassociating the induced pressure bulbs in the soil if battered piles are introduced, but the extent of the action of the vertical stresses due to the embankment loads are such that they will completely nullify the advantages of using battered piles.

OBSERVATIONS AND CONCLUSIONS: Cont'd.

4. The settlement due to the embankment may be speeded up if a surcharge fill is placed above the proposed embankment level and at some later date, removed. In this way, the danger of inducing the negative friction on piles will be greatly reduced and the waiting period before installing friction piles shortened.

Thus it follows that the embankment should be constructed first and after allowing for reasonable settlement to take place, the bridge structure.

5. The installation of vertical sand drains in order to increase the rate of settlement may be considered. But here again, the danger associated with the presence of Artesian water should be carefully studied before this solution is adopted.

E. M. PETO ASSOCIATES LTD.

*C. F. Freeman*

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





BL/vs

PART I

APPENDICES

e. m. peto associates ltd.  
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO  
BOREHOLE LOG

Job Name Eastcross Creek Bridge Job No. 59249 Borehole No. 1  
Client Dept. of Highways Of Ont. Casing BX Boring Date Dec. 11th-15th, 1959  
Datum Geodetic Compiled By I. C. B. Checked By C. R. F.

SAMPLE CONDITION		SAMPLE TYPE		ABBREVIATIONS	
	UNDISTURBED	A.S.	AUGER SAMPLE	V.T.	IN SITU VANE SHEAR TEST
	FAIR	C.S.	CASING SAMPLE	C.	SOIL SHEAR STRENGTH LBS/SQ.FT.
	DISTURBED	S.S.	2" STANDARD SPLIT TUBE SAMPLE	W.L.	WATER LEVEL IN CASING
	LOST	S.L.	SPLIT BARREL WITH LINERS	W.T.	GROUND WATER TABLE IN SOIL
		S.T.	THIN-WALLED SHELBY TUBE SAMPLE	W.T.P.L.	WETTER THAN PLASTIC LIMIT
		W.S.	WASH SAMPLE	D.T.P.L.	DRIER THAN PLASTIC LIMIT
		R.C.	ROCK CORE		

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Water Level	WATER LEVELS & REMARKS
			0'0"			S.S.	Dutch Cone		
duck	Dk. Brown	V. Soft	834.4		1	S.S.	0		Saturated
duck	Dk. Brown				2	C.S.	0		W.T. 1'2" Creek level
duck	Dk. Brown-Black	V. Soft			3	S.S.	Pushed	1	330.0 Saturated
			5'0"						
duck and decayed wood	Dk. Brown	V. Soft			4	S.S.	2	3	514.0 Saturated
as above	Dk. Brown	V. Soft			5	S.S.	1	3	526.0 Saturated
as above	Dk. Brown		9'4"		6A	S.L.	15	9.8	
sand and gravel	Grey		10'10"		7	S.S.	17	17	13.0 Saturated
as above	Grey	Compact			8	S.S.	8	10	18.5 Saturated M.W.T.P.L.
silty clay	Grey	Firm to Stiff				S.L.	15		(Sand 6% Silt 79% Clay 15%)
silty clay	Grey	Stiff to V. Stiff	16'0"			S.S.	17	18	
Silt	Grey				9	A.S.	15	20.0	Saturated.
			20'0"						
Layers of sandy silt and clayey silt	Grey	Stiff to V. Stiff			10	S.S.	14	15	20.3 Saturated P.I. 6.7 M.W.T.P.L.
Fine silt	Grey				11	A.S.	20		Saturated
Clayey silt-layers of sandy silt	Grey	V. Stiff	25'0"		12A	S.L.	16	20.5	
					12B	S.S.	20	17	21.0 Saturated M.W.T.P.L.
Clayey silt/silty clay	Grey				13				
			30'0"		14	A.S.	21	26.8	
Clayey silt	Grey				15	S.L.	24	39.2	Sand 2%
as above	Grey	V. Stiff			15B		49		Silt 28%
			35'0"		16	S.S.	20	28	30.2 Saturated Clay 70%
as above	Grey						35		M.W.T.P.L.
			40'0"		17	S.S.	14	47	23.7 Saturated M.W.T.P.L.
as above	Grey	Stiff to V. Stiff					49		L.L. 14.5
							50		P.L. 18.0
							51		P.I. 16.5%
as above	Grey	Stiff to V. Stiff			18	S.S.	12	45	26.2 Saturated Silt 26% Clay 14%
			45'0"		19A	S.L.	57		Slight Artesian Flows from 40'0" Onward
as above	Grey	Stiff to V. Stiff			19B	S.S.	15	57	22.7 Saturated M.W.T.P.L.
Silty clay	Grey				20		66		
as above	Grey	Stiff to V. Stiff			21	S.S.	10	70	24.0 Saturated M.W.T.P.L.
Silty clay/clayey silt	Grey	Stiff					80		
			55'0"		22A	S.L.	12	105	Saturated M.W.T.P.L.
Silty clay	Grey	Stiff			22B		145		
					22C		140		
as above	Grey	Stiff to V. Stiff	60'0"		23	S.S.	10	155	22.9 Saturated M.W.T.P.L.
							250		
							300		
							324		
as above, small stones	Grey	V. Stiff	65'0"		24	S.S.	20	315	24.7 Saturated Getting Stiffer
							280		M.W.T.P.L.
							290		
Silty clay	Grey	Hard	70'0"		25	S.S.	35		23.5 Saturated, but drier than previously
			75'0"		26	S.S.	22		28.1 Saturated W.T.P.L.
Silty clay	Grey	Alternate Soft and Stiff layers							
			80'0"		27	S.S.	25		23.0 Saturated W.T.P.L.
Silty clay/clayey silt some stones	Grey	As above							
			85'0"		28	S.S.	28		22.2 Saturated W.T.P.L.
as above	Grey	As above							
			90'0"		29	S.S.	34		
Silty clay, trace of grits	Grey	Hard							

BOREHOLE TERMINATED AT 91'0"

# BOREHOLE LOG

Borehole No. 2  
Boring Date Dec. 22nd-28th, 1959  
Checked By C. F. F.

### ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST  
C. SOIL SHEAR STRENGTH LBS/SQ.FT.  
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

SOIL DESCRIPTION	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	Natural Moisture Content	WATER LEVELS & REMARKS
			0' 0"			S.S.	Dutch Cone		
			835.6		1 X	C.S.	3		
Peat	Dk. Brown	V. Loose			2 X	S.S.	2	0	1114.0 Saturated
			5' 0"					1	
Peat, decayed wood	Dk. Brown				3 X	S.S.	3	208.0	Saturated
								3	
								3	
			10' 0"					3	
Peat	Dk. Brown				4 X	S.S.	4	613.0	Saturated
								5	
			14' 0"					10	
			15' 0"					21	
		Loose to Compact				S.S.	12	24	
								22	
								not	
Gravel (1-1/2" down)	Grey				5 X	S.S.	22	22	Moist
Gravel	Grey		20' 0"		6 X	A.S.	21	22.6	
Layers of silty clay and clayey silt (Interbedded)	Grey							27	
								30	
								46	
			25' 0"			S.S.	29	20.9	
								31	
As above	Grey	Stiff			8 X	S.S.	12	30	21.3 M.W.T.P.L.
								28	
								26	V.T. did not shear at
								19	870 lbs. per sq.ft.
			30' 0"		9A	S.L.		18	
As above	As above				9B			30	25.2
As above	As above	Stiff			10 X	S.S.	10	28	29.0 M.W.T.P.L.
								30	
								26	V.T. did not shear at
								26	2950 lbs./sq.ft.
As above	As above	Stiff	35' 0"		11 X	S.S.	12	28	M.W.T.P.L.
								28	
								35	V.T. : C = 2950#/sq.f.t
								35	
			40' 0"					38	
								51	
								48	
As above	As above				12A	S.L.		64	24.2
					12B			68	
			45' 0"					65	V.T.: C = 1,230#/sq.f.t
								66	
								67	
As above	As above	Stiff to V. Stiff			11 X	S.S.	10	64	20.7 W.T.P.E.
								59	V.T.C. * 1,970 #/sq.f.t
			50' 0"					50	
As above	As above				14A	S.L.			119.7
					14B				
			55' 0"						
As above	As above	Stiff			15 X	S.S.	10		23.0
									V.T.: C = 2210#/sq.ft.
			58' 0"						V.T.: C = 1970#/sq.ft.
									HOLE TERMINATED AT 58' 0"



SOIL ENGINEERING SERVICE - TORONTO, ONTARIO





# BOREHOLE LOG

Job Name Eastercross Creek Bridge Job No. 59249 Borehole No. 4  
Client Dept. of Highways of Ont. Casing 4" Pipe Boring Date Dec. 16th-18th, 1959  
Datum Geodetic Compiled By S. B. Checked By B. L.

### SAMPLE CONDITION

**SAMPLE TYPE**

### ABBREVIATIONS

	UNDISTURBED
	FAIR
	DISTURBED
	LOST

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

V.T.	IN SITU VANE SHEAR TEST
C.	SOIL SHEAR STRENGTH LBS/SQ.FT.
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

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
Existing Ground			0'0"				S.6.	Dubhh	
Peat	Dk. Brown to Black		834.7		1		0		Saturated
Peat, decayed wood	As above	V. Loose			2	S.S.	1 0	216.0	Saturated
			5'0"				2		
		V. loose			3	3"S. L.	3	542.0	
						S.S.	2 4		
						3" S. L.	4		
Fine sand, pebbles & stones trace of peat, pockets of silt	Olive Grey	Compact	9'6"		4	3"S. L.	5	504.0	
			10'6"		5	S.S.	15	20	17.5 Quite moist.
							17		
							14		
							14		
							37		
			15'0"				40		
Layers of silty clay and silt.	Lt. Grey				6	3"S. L.	40		
							35		
							39		
			20'0"				28		
Layers of silty clay and silt	Lt. Grey	Firm to Stiff			7	S.S.	9 29	24.3	W.T.P.L.
							34		
							40		
							24		
			25'0"				12		
							36		
As above	As above				8	3" S. L.	31	23.3	
							29		Stiffening at 27'6"
							29		
			30'0"				29		
Silty clay	Grey	V. Stiff			9	S.S.	27 39	31.6	W.T.P.L.
							46		
As above	As above				10	S.S.	48		W.T.P.L.
							44		
			35'0"				42		Turning softer at 35'0"
							40		
As above	As above.				11	3"S. L.	41	23.8	
							45		
							43		
			40'0"				45		
Layers of silty clay, silt, and clayey silt.	Lt. Grey	Stiff			12	S.S.	10 46	21.5	W.T.P.L.
							48		
							50		
							58		
			45'0"				53		
As above	As above.				13	3" S. L.	50	30.3	W.T.P.L.
							49		
							50		
							49		
As above	As above		50'0"		14	A.S.	54		W.T.P.L.
							64		
					15	3"S. L.	70	21.7	
							60		
							55		
			55'0"				60		
As above	As above	Stiff to V. Stiff			16	S.S.	15 69		W.T.P.L.
							65		
							65		
							67		
							70		
									TEST HOLE TERMINATED AT 50'0"

# e. m. peto associates ltd.

## SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

### BOREHOLE LOG

Job Name Eastercross Creek Bridge Job No. 59249 Borehole No. 5  
 Client Dept. of Highways Casing BX Boring Date Dec. 21 & 22nd, 1959  
 Datum Geodetic Compiled By S.B. Checked By B.L.

**SAMPLE CONDITION****SAMPLE TYPE****ABBREVIATIONS**

UNDISTURBED

FAIR

DISTURBED

LOST

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

V.T. IN SITU VANE SHEAR TEST  
 C. SOIL SHEAR STRENGTH LBS/SQ.FT.  
 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

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVEL / REMARKS
			0'0"			St. Pen. Test	Cone Test	
Decayed wood, organic matter	Dk. brown to Black		8'35.5"		1	C.S.	Test	Wet
Peat					1	S.S.	Pushed	Saturated
As above	As above		5'0"		2	S.S.	Pushed	376.0 N.M.C. Sat.
As above	As above	V. Loose to Loose	10'0"		3	S.S.	5	61689.0 Saturated
As above	As above	Loose	15'0"			S.S.	3	
			18'9"					
			20'0"		4	S.S.	14	229.0 Wet
Fine to medium sand pebbles and stones	Olive grey	Compact	23'0"					
			25'0"		5	S.S.	19	3221.6 Silt - moist
Layers of silt, clayey silt and silty clay	Grey	V. Stiff	30'0"		6	S.S.	19	34W.T.P.L.
Silty clay	Grey	V. Stiff	35'0"		7	S.S.	16	30W.T.P.L.
Silty clay, seams of clayey silt	Grey	Stiff to V. Stiff	40'0"		8	2"S.L.		3726.5
Silty clay	Grey	V. Stiff	45'0"		9	S.S.	23	4827.9 W.T.P.L.
Alternate layers of soft silty clay and stiff clayey silt.	Lt. Grey	Stiff to V. Stiff	50'0"		10	S.S.	17	46M.W.T.P.L.
As above	As above	V. Stiff	55'0"		11	S.S.	27	7622.9 M.W.T.P.L.
As above	As above	V. Stiff			12	S.S.	19	6622.3 M.W.T.P.L.
As above	As above	V. Stiff to Hard	60'0"		13	S.S.	32	6123.7 Silty clay M.W.T.P.L.
								23.1 Clayey silt.

TEST HOLE TERMINATED AT 60'0"



**e. m. peto associates ltd.**  
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO  
BOREHOLE LOG

Job Name Eastcross Creek Bridge Job No. 59249 Borehole No. 0  
Client Dept. of Highways of Ont. Casing BX Boring Date Dec. 18th-21st, 1959  
Datum Geodetic Compiled By S. B. L. Checked By B. L.





SAMPLE CONDITION		SAMPLE TYPE			ABBREVIATIONS				
	UNDISTURBED	A.S. AUGER SAMPLE	C.S. CASING SAMPLE	S.S. 2" STANDARD SPLIT TUBESAMPLE	S.L. SPLIT BARREL WITH LINERS	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.S. WASH SAMPLE	R.C. ROCK CORE	Y.T. IN SITU VANE SHEAR TEST
	FAIR								C. SOIL SHEAR STRENGTH LBS/SQ.FT.
	DISTURBED								W.L. WATER LEVEL IN CASING
	LOST								W.T. GROUND WATER TABLE IN SOIL
									W.T.P.L. WETTER THAN PLASTIC LIMIT
									D.T.P.L. DRIER THAN PLASTIC LIMIT

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
			0'0"				St. Cone Pen. Test.		
Decayed wood, organic matter	Dk. Brown		8'35.5		1	C.S.	1		Wet
Peat					2	S.S.	1		
Peat	Dk. Brown	V. Loose							Saturated
			5'10"						
					3	S.S.	1		
Peat, decayed wood	Dk. Brown to Black	V. Loose							Saturated
			8'6"						
Fine sand, trace of peat	Olive grey	Compact			4	S.S.	23	15.8	Quite Moist
Fine to medium sand pebbles, pockets of grey silt.	Olive grey	Dense	12'0"		5	S.S.	43	35	Quite moist.
								50	
								13.1	
								32	
								16	
			15'0"					43	
								51	
								64	
								56	
								42	
			20'0"		6	2" S.L.		70	20.0
Layers of silty clay and clayey silt	Grey	Stiff			7	S.S.	12	47	M.W.T.P.L.
								50	
								52	
								54	
			25'0"					54	Washwater used after 21'6" depth
Layers of silty clay (soft) Clay silt (Stiff) Some Silt	Grey	Stiff			8	S.S.	14	57	22.6 M.W.T.P.L.
								48	
								43	
								49	
			30'0"		9	2" S.L.		42	23.8
Silty clay	Grey	Stiff to V. Stiff			10	S.S.	10	45	M.W.T.P.L.
								45	
								45	
			35'0"					48	
Silty clay, seams of clayey silt	As above	Stiff			11	S.S.	14	52	28.4 M.W.T.P.L.
								51	
								55	
								54	
			40'0"		12	2" S.L.		58	20.8
As above	As above							63	20.8
								80	
								70	
								67	
			45'0"					73	
Layers of silty clay and Silt	Lt. Grey	Stiff			13	S.S.	14	70	21.6 M.W.T.P.L.
								74	
								62	
								60	
			50'0"					62	
As above	As above				14	2" S.L.		200	Note: Cone penetration test results at 50-51 ft. probably high due to rods being frozen during the night
								155	
								140	
								115	
			55'0"					122	
As above	As above	Stiff to V. Stiff			15	S.S.	16	109	23.5 W.T.P.L.
								84	
								80	
								82	
								90	
			60'0"						
As above	As above	Stiff to V. Stiff	61'0"		16	S.S.	17		24.3 W.T.P.L.
			</						

**e. m. peto associates ltd.**  
**SOIL ENGINEERING SERVICE - TORONTO, ONTARIO**  
**BOREHOLE LOG**

Job Name Eastcross Creek Bridge Job No. 59249 Borehole No. Probe to find depth of muck  
 Client Dept. of Highways of Ont. Casing BX Boring Date Jan. 5th, 1960  
 Datum  Compiled By I.G.B. Checked By C.F.P.

**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  
 C. SOIL SHEAR STRENGTH LBS/SQ.FT.  
 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

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Relative Moisture Content	WATER LEVELS & REMARKS
Probe 10: 200 ft. West of bridge, centre line of proposed road.			0'0"			BX			
Muck			4'0"			X3			
			5'0"			X0			
Sand						X0			
						X6			
Probe 11: 400 ft. W. of bridge, centre line of proposed road			0'0"						
Silty topsoil and decayed vegetation			3'0"			X6			
						X5			
Sand			5'0"			X5			
						X20			
						X30			
Probe 12: 350 ft. W. of bridge, centre line of proposed road			0'0"						
Silty topsoil and decayed vegetation			4'0"			X4			
						X3			
						X2			
Sand			6'0"			X3			
						X3			
						X20			
Probe 13: 300 ft. W. of bridge, centre line of proposed road.			0'0"						
Silty topsoil and decayed vegetation			4'0"			X2			
						X3			
						X5			
Silty fine sand,			9'0"			X10			
						X20			
						X15			
						X25			
						X30			

**e. m. peto associates ltd.**  
**SOIL ENGINEERING SERVICE - TORONTO, ONTARIO**  
**BOREHOLE LOG**

Job Name Eastcross Creek Bridge Job No. 59249 Borehole No. Probes to find depth of  
 Client D.H.O. Casing BX Boring Date Jan. 5th, 1960 muck  
 Datum ..... Compiled By T.G.B. Checked By C.E.F.

**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
- C. SOIL SHEAR STRENGTH LBS/SQ.FT.
- 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

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
Probe 7: 520 ft. East of bridge centre line of proposed road			0'0"			BX Casing			
						X2			
						X1			
Muck down to 6 ft.						X0			
			5'0"			X1			
			6'0"			X2			
Grey sand below 6 ft.						X6			
						X10			
						X12			
			10'0"			X12			
Probe 8: 625 ft. East of bridge, centre line of proposed road			0'0"			X2			
Silty topsoil and decayed vegetation down to 4 ft.						X4			
			4'0"			X2			
Grey & Brown sand			5'0"			X20			
Probe 9: 570 ft. East of bridge, centre line of proposed road			0'0"			X2			
Silty topsoil and decayed vegetation						X2			
			3'0"			X2			
Grey brown sand.			4'0"			X15			

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO





## BOREHOLE LOG

<b>Job Name</b> Eastern Creek Bridge	<b>Job No.</b> 56245A	<b>Borehole No.</b> 7A
<b>Client</b> Dept. of Highways of Ontario	<b>Casing</b> 4" Pipe	<b>Boring Date</b> February 2nd-25th, 1960.
<b>Drawn</b>	<b>Compiled By</b> U. J. V.	<b>Checked By</b>

[illegible]

e. m. peto associates ltd.  
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO  
BOREHOLE LOG

Job Name: 100' Deep Water Borehole Job No.: 1920 Proposed Use: Water  
Client: City of Hamilton Casing: 4" IPS Borehole Date: Feb. 25, 1960  
Datum: Geoid Compiled By: E. M. Peto Checked By:

SAMPLE CONDITION		SAMPLE TYPE		ABBREVIATIONS	
	UNDISTURBED	A.S. AUGER SAMPLE		S.T.	IN SITU VANE SHEAR TEST
	FAIR	C.S. CASING SAMPLE		C.	MIN. SHEAR STRENGTH (K.G./SQ. FT.)
	DISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE		W.L.	WATER LEVEL IN CASING
	LOST	S.L. SPLIT BARREL WITH LINERS		W.T.	GROUND WATER TABLE IN SOIL
		S.T. THIN-WALLED SHELL BY TUBE SAMPLE		W.T.P.L.	WETTER THAN PLASTIC LIMIT
		W.S. WASH SAMPLE		D.T.P.L.	DRIER THAN PLASTIC LIMIT
		R.C. ROCK CORE			

SOIL DESCRIPTION	COLOR	Quantity or Consistency	Depth Elevation	Logged	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Moisture Content (%)	WATER LEVELS & REMARKS
Snow 18", ice 12" water 12"			0'0"						
			3'7"						
			9'4"						
			9'9"						
Mudlap	Dark Brown		10'0"						Saturated
			13'0"						
			15'0"						
Fine to Medium sand with grey gravel & stones					1	S.S.	15	12.4	Wet to Saturated
Very fine sandy silt	Grey		20'3"		2	S.S.	22	22.4	Wet to Saturated, Feb. 25th
			25'0"						
					3	S.S.	20	21.5	
As above with layers of clayey silt	Grey		30'0"		4	S.S.	18	20.7	Wet and W.T.P.L.
					5	2"S. L. tapped	5	32.5	
Silty clay with layers of clayey silt	Grey		35'0"		6	S.S.	8	32.5	W.T.P.L.
					7	2"S. L. tapped	1	23.3	
Silty clay with seams of silt	Grey		40'0"		8	S.S.	7	23.6	W.T.P.L.
					9	3"S. L. tapped	1	25.8	
Clayey silt with layers of silty clay containing seams of silt	Grey		45'0"		10	S.S.	10	25.9	W.T.P.L.
					11	3"S. L. tapped	1	23.5	
Clayey silt with seams of silt	Grey		50'0"		12	S.S.	10	24.1	M.W.T.P.L. Feb. 27th
					13	3"S. L. tapped	1	22.2	
Clayey silt with layers of silty clay	Grey		55'0"		14	S.S.	8	23.4	M.W.T.P.L.
					15	3"S. L. tapped	1	22.8	
As Above	Grey		60'0"		16	S.S.	9	23.3	M.W.T.P.L.
					17	3"S. L. tapped	1	25.3	
Clay silt	Grey		65'0"		18	S.S.	9	22.6	M.W.T.P.L.
					19	3"S. L. tapped	1	23.5	
Silty clay with seams of silt	Grey		70'0"		20	S.S.	9	21.9	W.T.P.L.
					21	3"S. L. tapped	1	23.7	
As Above	Grey		75'0"		22	S.S.	8	25.8	W.T.P.L. Feb. 29th
					23	3"S. L. tapped	1	30.3	
Clayey silt with layers of silty clay, seams of fine sand and silt	Grey		80'0"		24	S.S.	10	23.2	W.T.P.L.
					25	3"S. L. tapped	1	26.5	
Clayey silt with layers of silty clay, seams of silt	Grey		85'0"		26	S.S.	10	26.6	M.W.T.P.L.
					27	3"S. L. tapped	1	25.1	
As Above	Grey		90'0"		28	S.S.	18	23.6	W.T.P.L.
					29	3"S. L. tapped	1	23.0	
As above	Grey				30	S.S.	12	28.6	W.T.P.L. March 1, 1960.

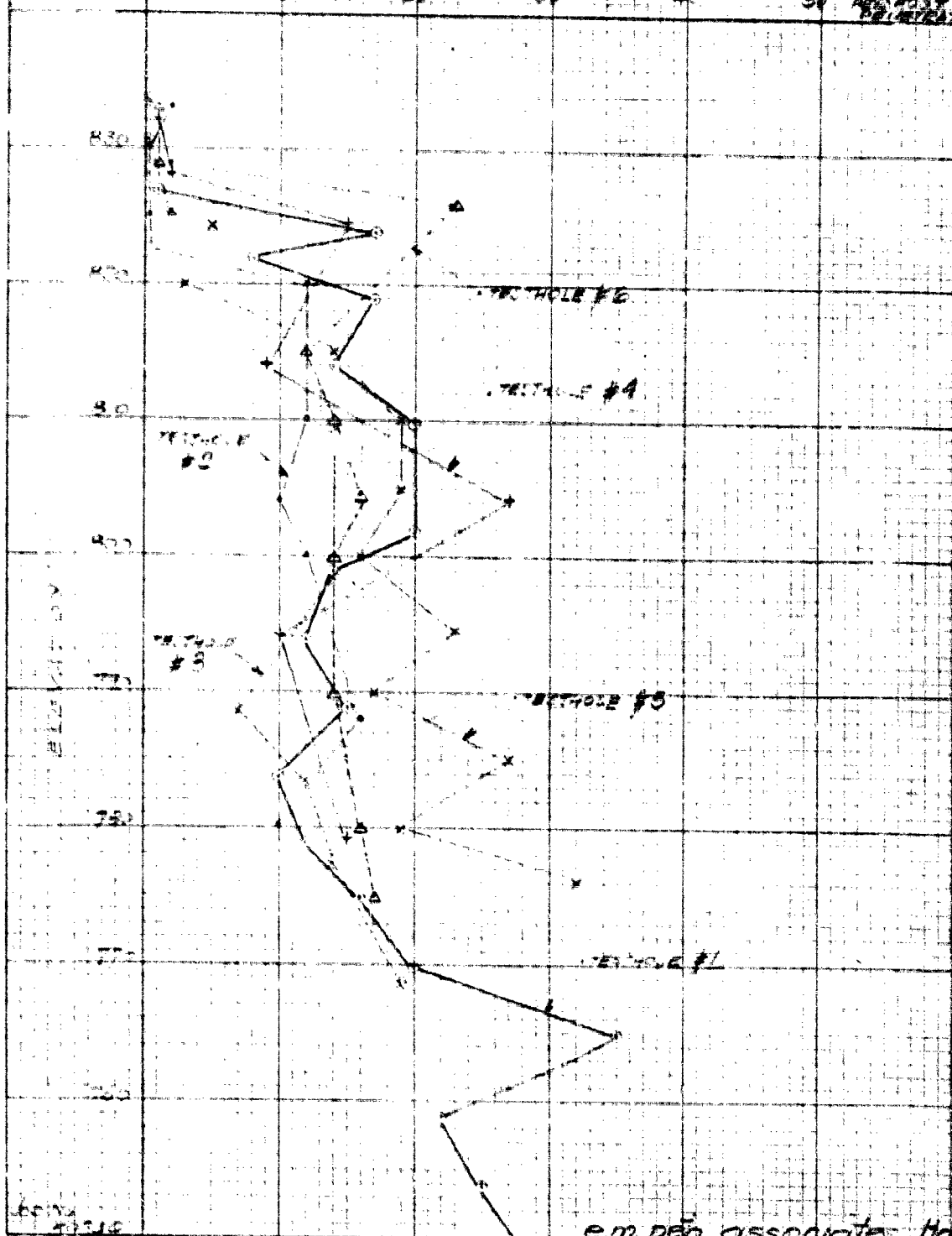
HOLE TERMINATED AT 92ft. 4 ins.

PRELIMINARY

REPORT

# STANDARD PENETRATION TEST RESULTS.

No. of blows  
per foot  
penetration



e.m.p.a. associates, Inc.

# DYNAMIC CONE PENETRATION TEST.

No. of blows / 100 penetration

DEPTH

(SYMBOLS)

- TESTHOLE #1
- TESTHOLE #2
- ×— TESTHOLE #3
- +— TESTHOLE #4
- \*— TESTHOLE #5
- ▲— TESTHOLE #6

Job No 59242

e.m.p.c. associates Hd.



# MOISTURE CONTENT VS. ELEVATION

NATURAL MOISTURE CONTENT IN %

10 20 30 40 50

820

810

800

790

780

770

760

ELEVATION

TIME

TESTHOLE #1

- #2
- △- #3
- x- #4
- #5
- #6

400 No

10219

emp. per associates Hcl

APPENDIX II

LABORATORY TEST RESULTS

# e. m. peto associates ltd.

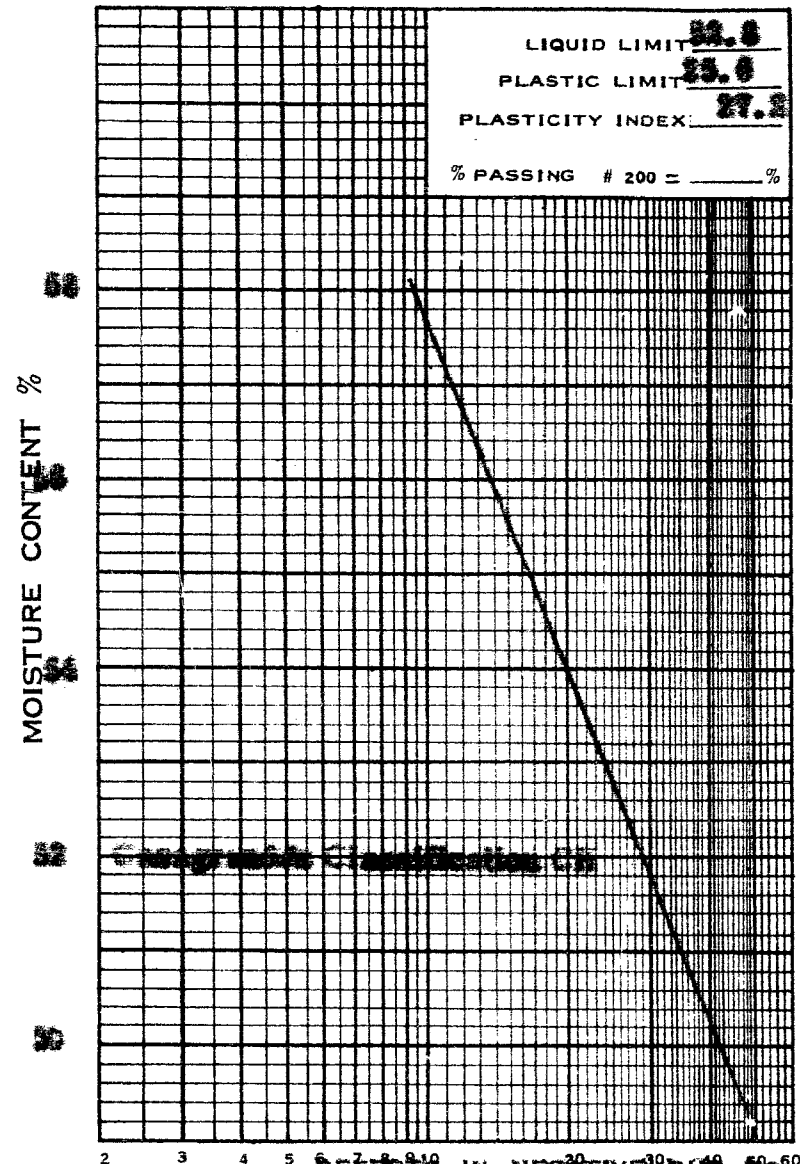
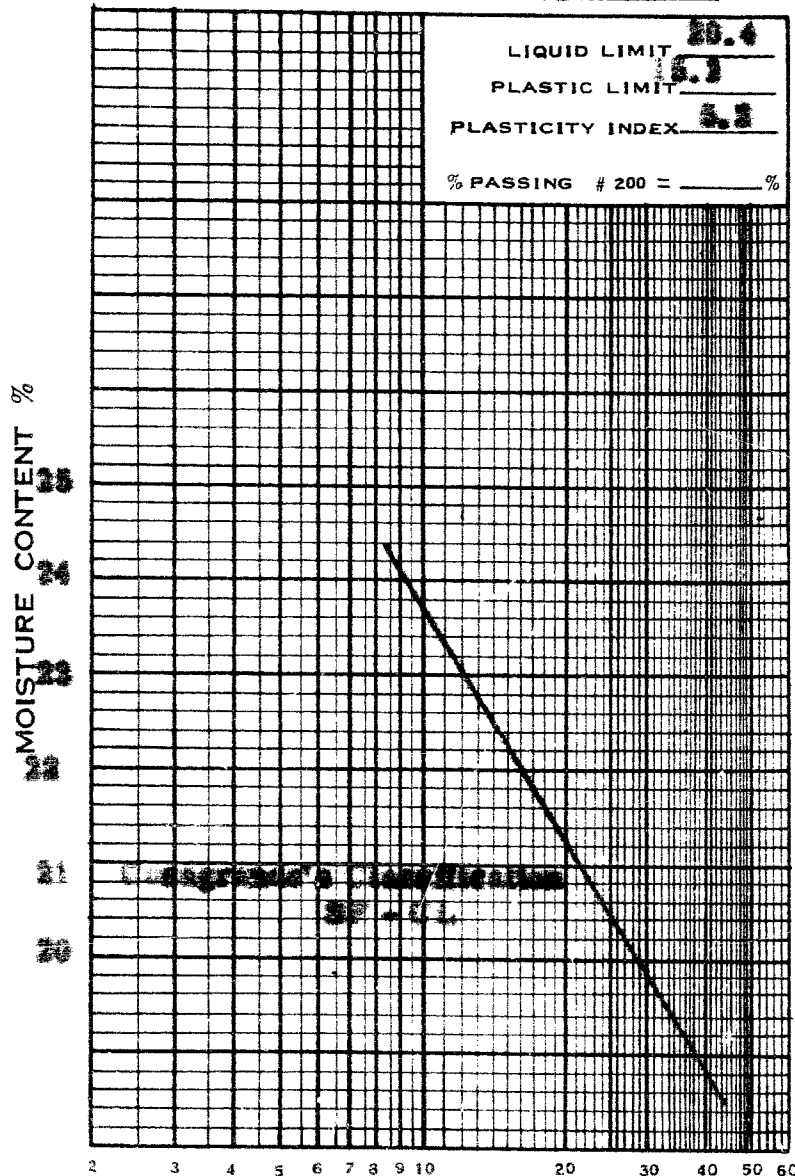
## SOIL TESTING LABORATORY

### LIQUID LIMIT TEST

### FLOW LINE CHARTS

JOB No. 50247 PROJECT Castroville Creek Bridge  
 SAMPLE FROM L.R. 5, Sample 1  
 DEPTH 20 - 25

SAMPLE FROM L.R. 5 Sample 6  
 DEPTH 30 - 31



DEFECTS IN NEGATIVE DUE TO  
 CONDITION OF ORIGINAL DOCUMENT

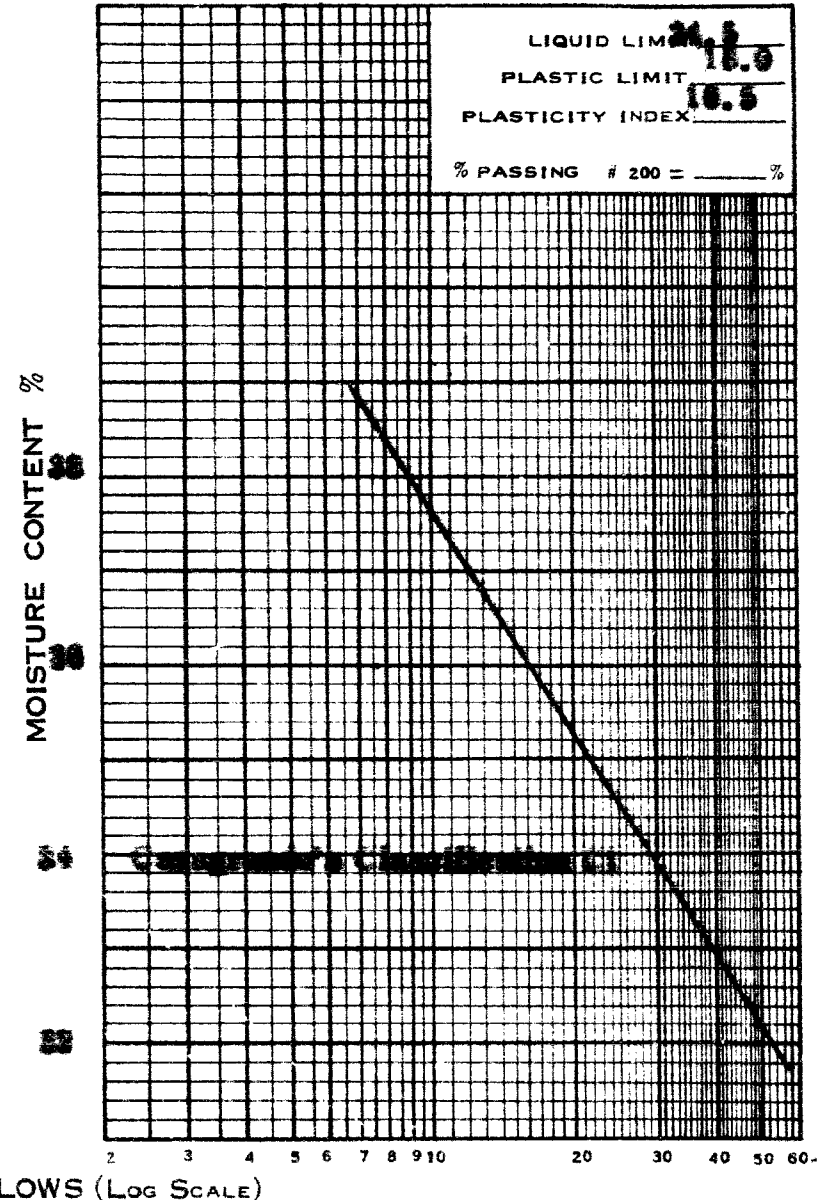
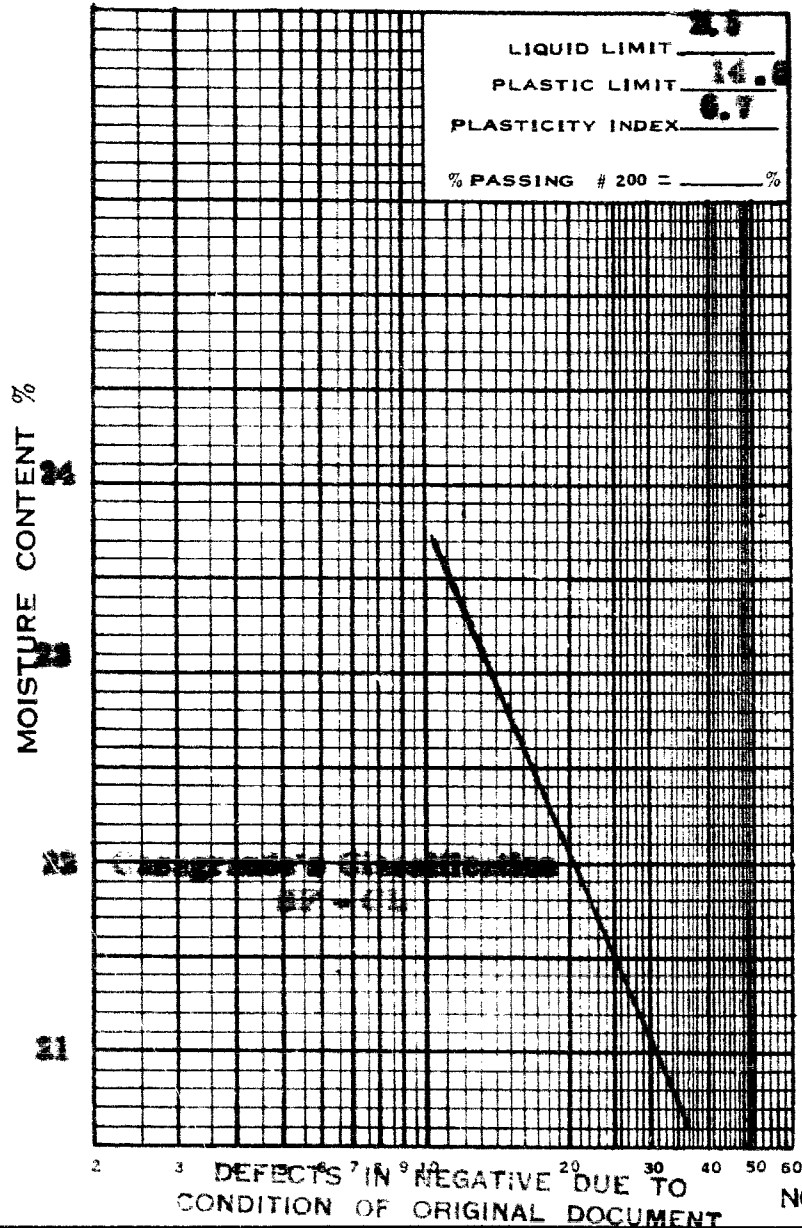
# e. m. peto associates ltd. SOIL TESTING LABORATORY

## LIQUID LIMIT TEST

## FLOW LINE CHARTS

JOB No. 50349 PROJECT Castroville Creek Bridge  
SAMPLE FROM S.H. 1 Sample 10  
DEPTH 20' - 21'

SAMPLE FROM S.H. 1 Sample 17  
DEPTH 20' - 21'



DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

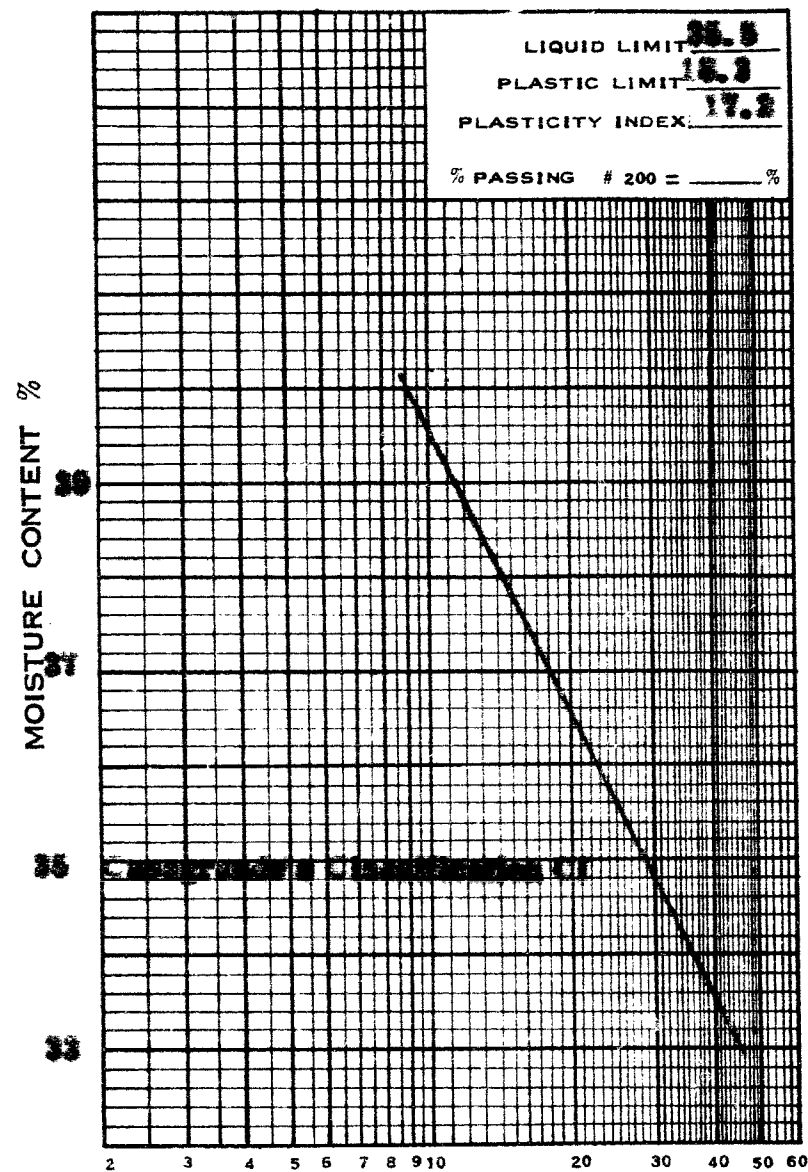
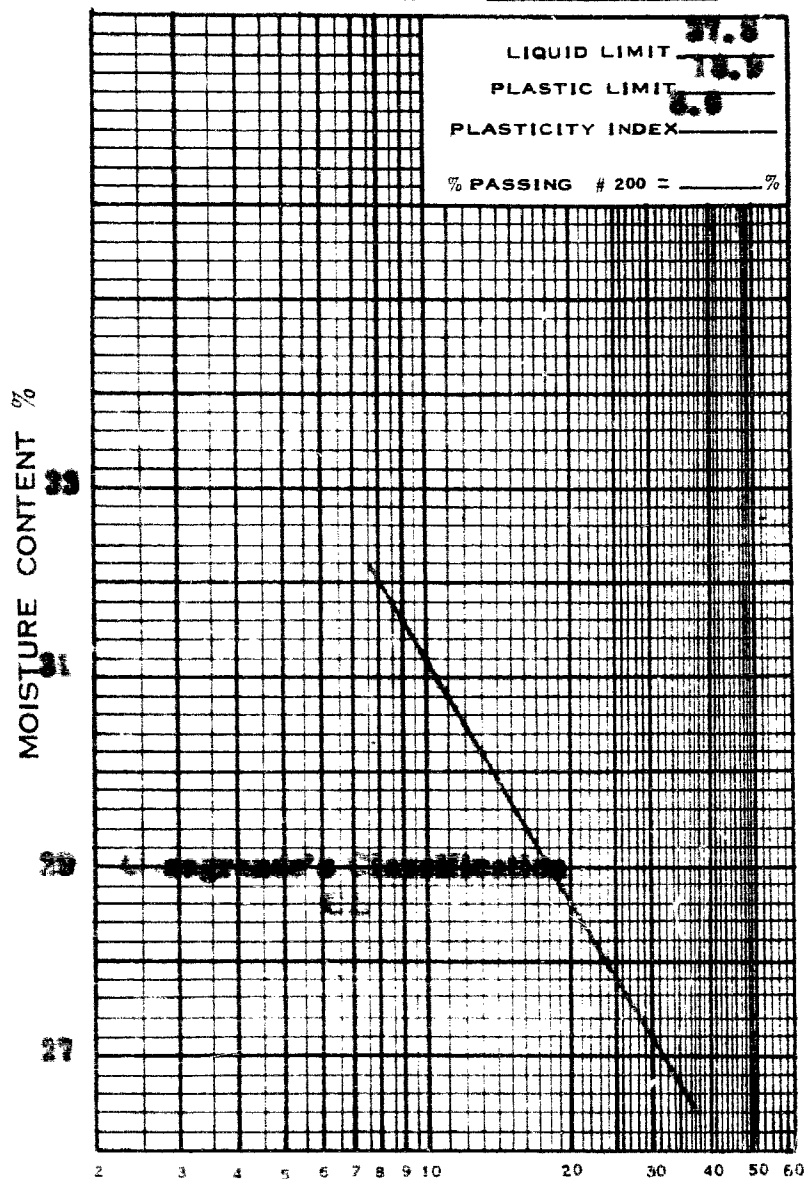
e. m. peto associates ltd.  
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

Job No. 58249 PROJECT Eastwood Creek Bridge  
SAMPLE FROM P.H. 5, sample 5  
DEPTH 45' - 47'

SAMPLE FROM P.H. 5, sample 7  
DEPTH 35' - 37'



NO. OF BLOWS (LOG SCALE)

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

e. m. peto associates ltd.  
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

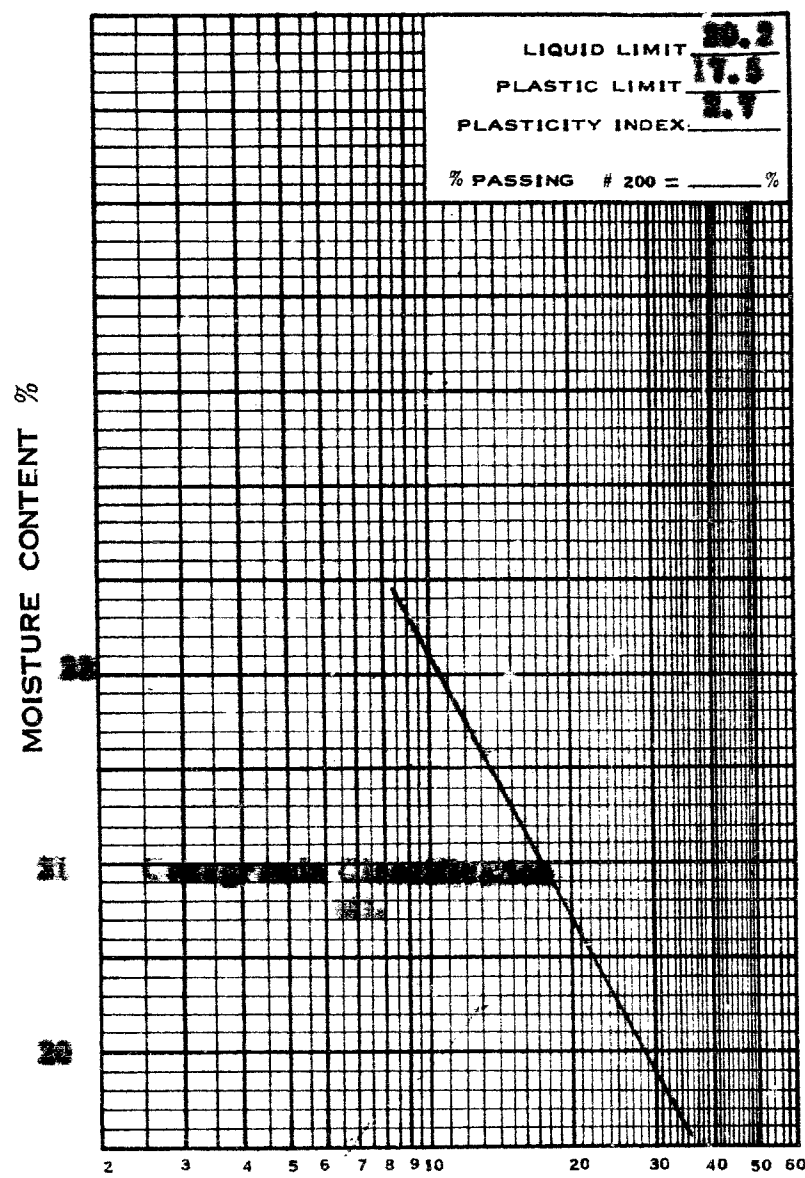
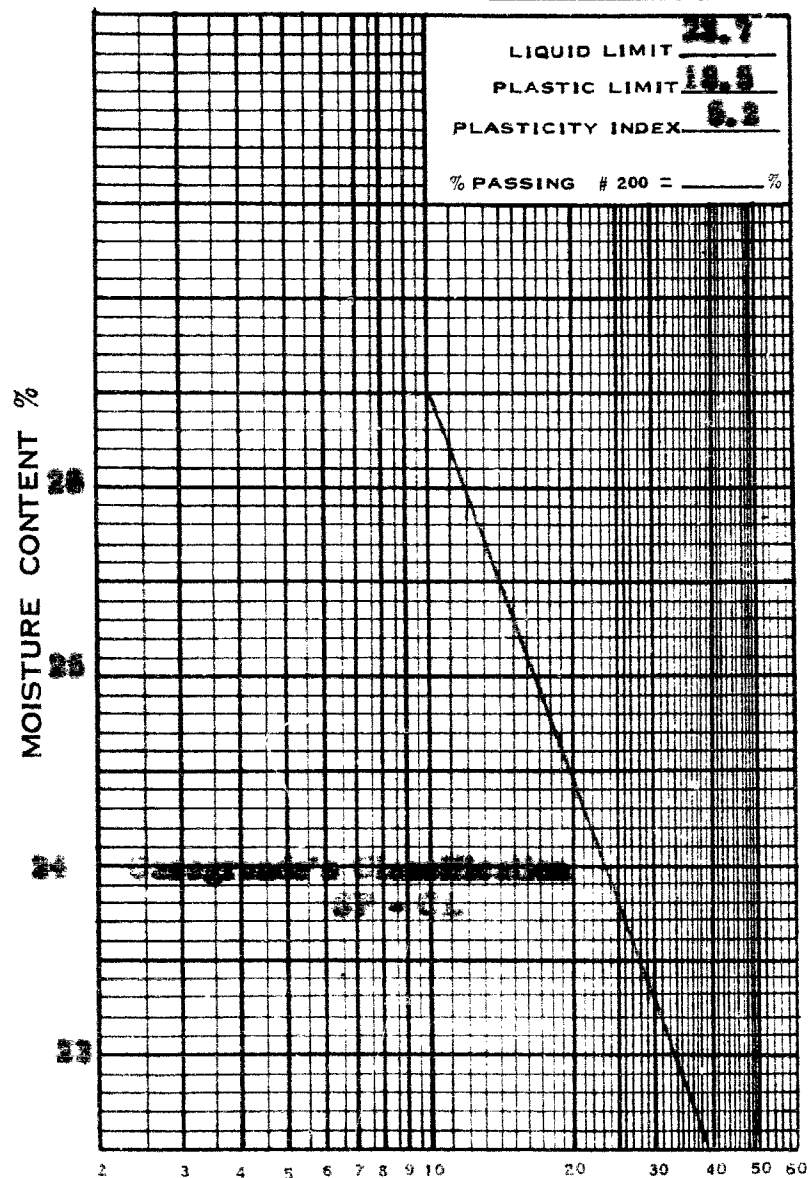
Job No. 53349 PROJECT Lester Creek Bridge

SAMPLE FROM A.B. 5, Sample 10

DEPTH 10' - 12'

SAMPLE FROM A.B. 5, Sample 11

DEPTH 10' - 12'



NO. OF BLOWS (LOG SCALE)

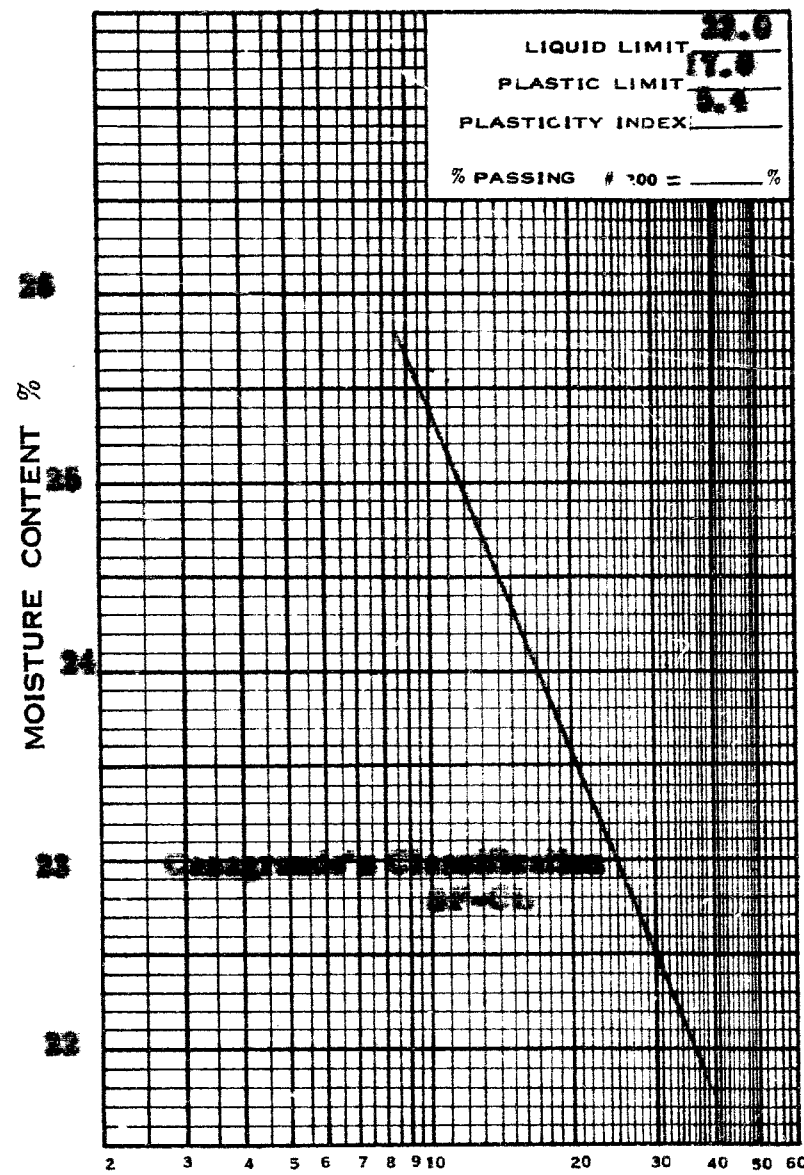
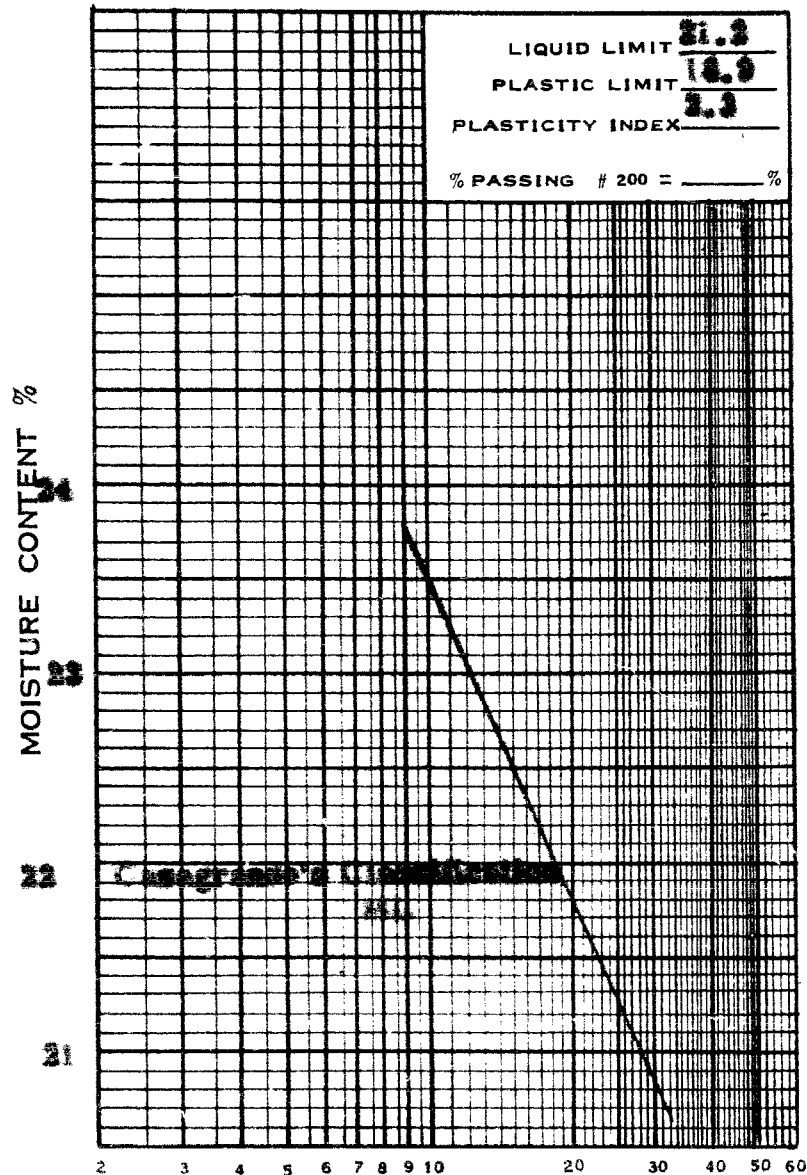
# e. m. peto associates ltd. SOIL TESTING LABORATORY

## LIQUID LIMIT TEST

## FLOW LINE CHARTS

JOB NO. 50249 PROJECT Hotcross Creek Bridge  
SAMPLE FROM E. H. 5, Sample 12  
DEPTH 10' - 10"

SAMPLE FROM E. H. 5, Sample 13  
DEPTH 10' - 10"

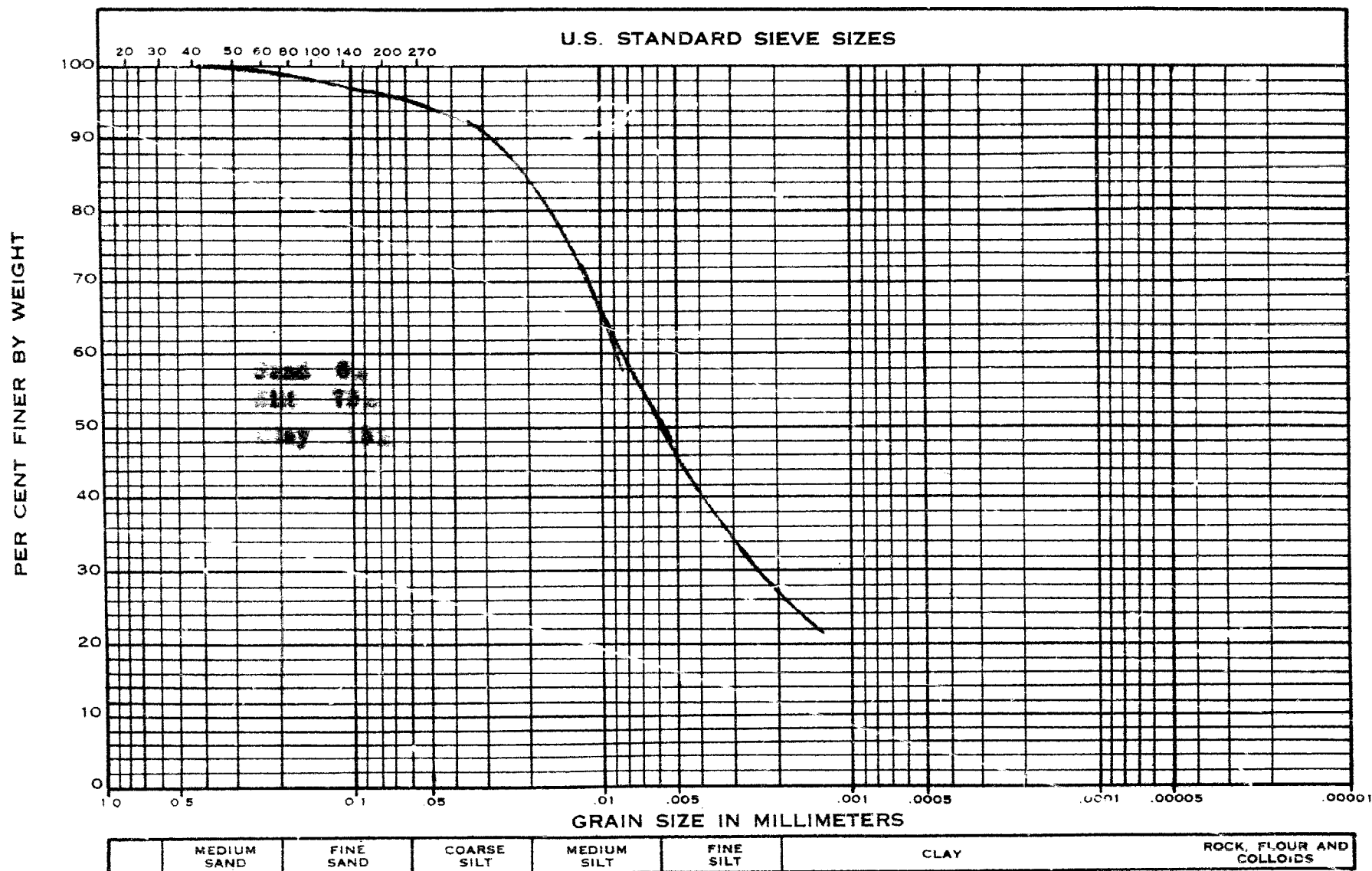


NO. OF BLOWS (LOG SCALE)



DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

## HYDROMETER GRAIN SIZE DISTRIBUTION DIAGRAM



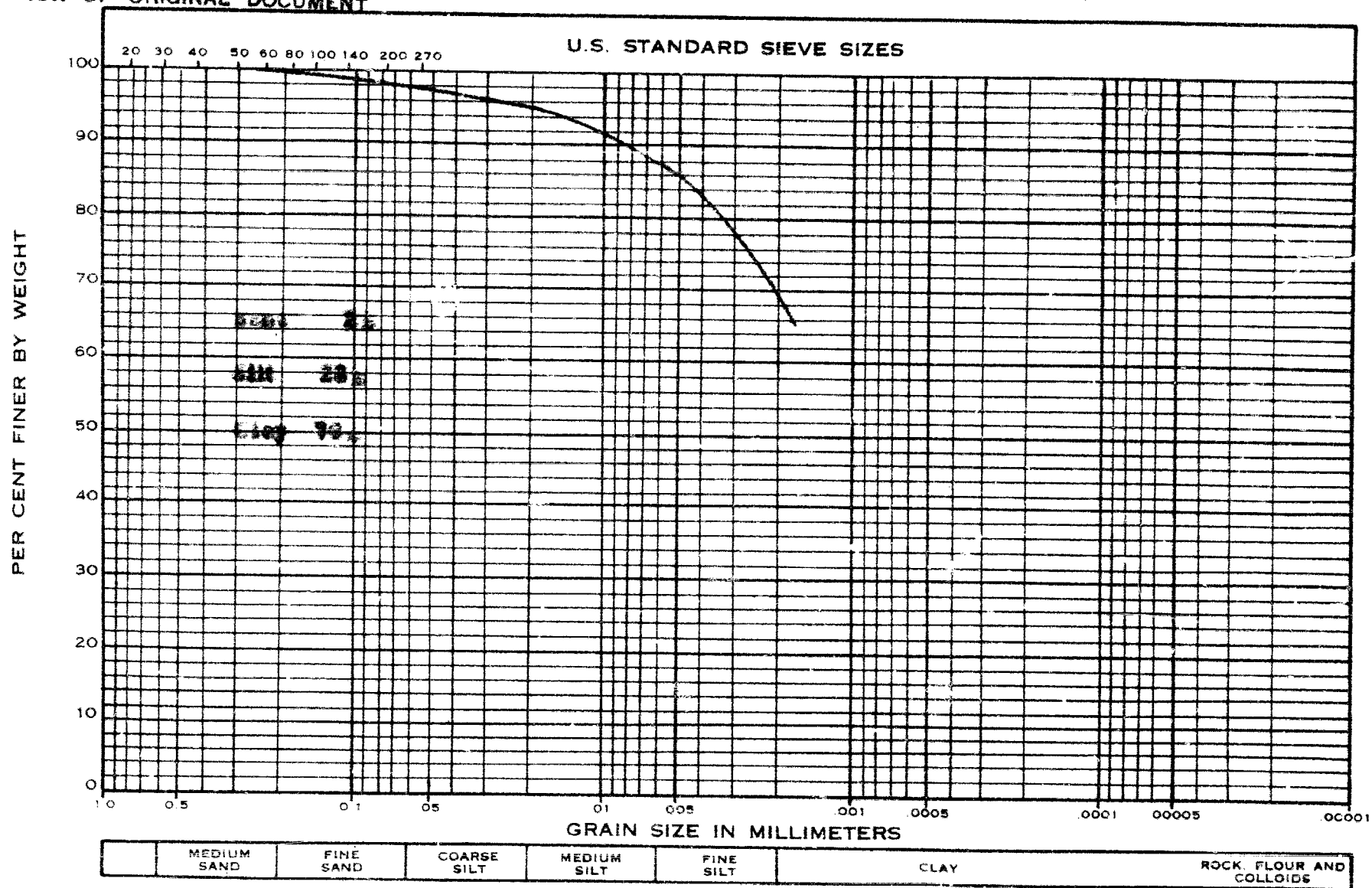
## M.I.T. CLASSIFICATION

JOB NAME Salcross Bridge JOB No. 5084 BOREHOLE No. 1 SAMPLE No. 1

DEPTH 1' - 3' ELEVATION \_\_\_\_\_ REMARKS Textural Classification - Silt



# DEFECTS IN NEGATIVE DUE TO HYDROMETER GRAIN SIZE DISTRIBUTION DIAGRAM CONDITION OF ORIGINAL DOCUMENT



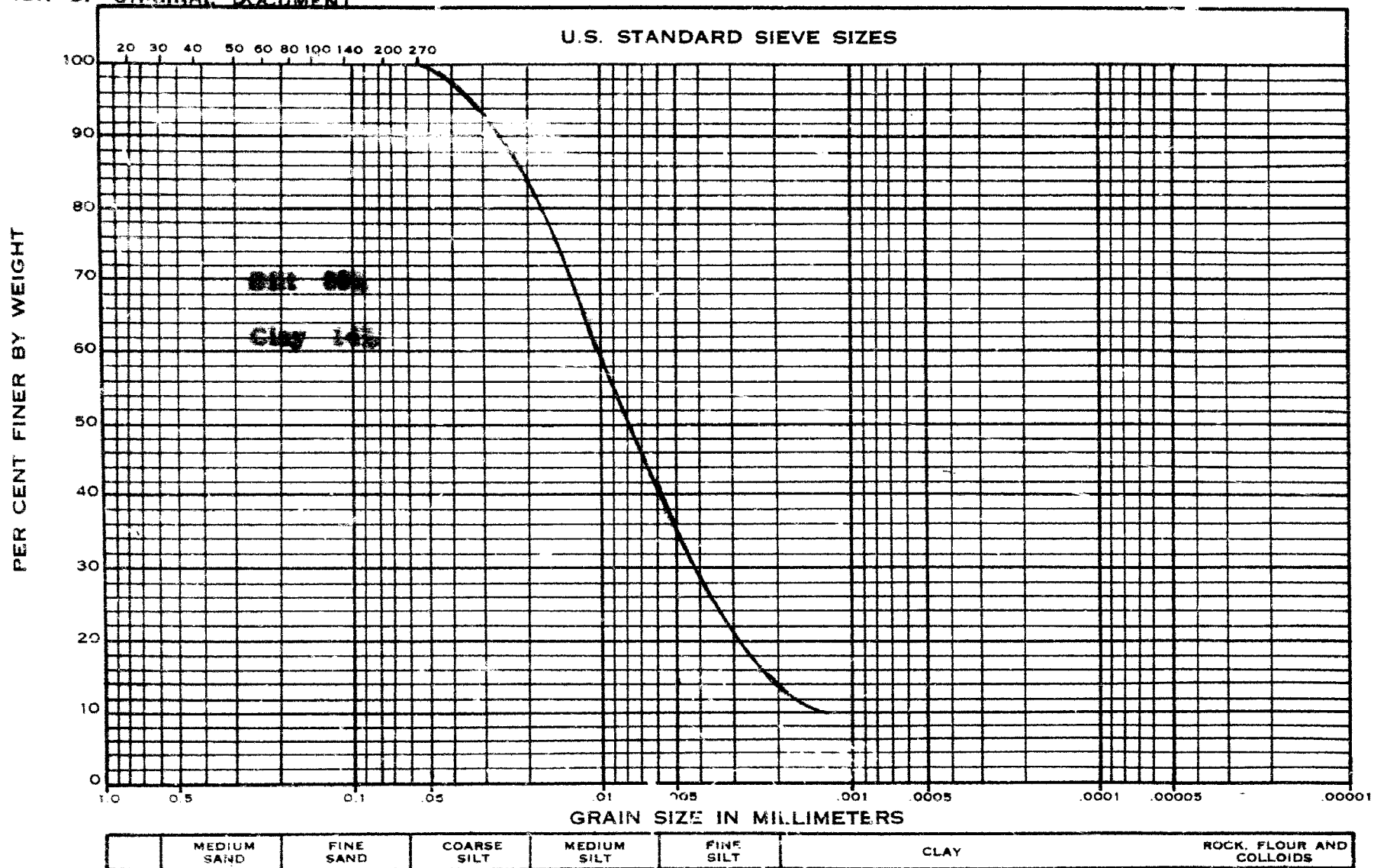
## M.I.T. CLASSIFICATION

JOB NAME Castroville Bridge JOB No 50349 BOREHOLE No 1 SAMPLE No 10

DEPTH 32' - 35' ELEVATION \_\_\_\_\_ REMARKS Textural Classification - Clay

# DEFECTS IN NEGATIVE DUE TO HYDROMETER GRAIN SIZE DISTRIBUTION DIAGRAM

CONDITION OF ORIGINAL DOCUMENT



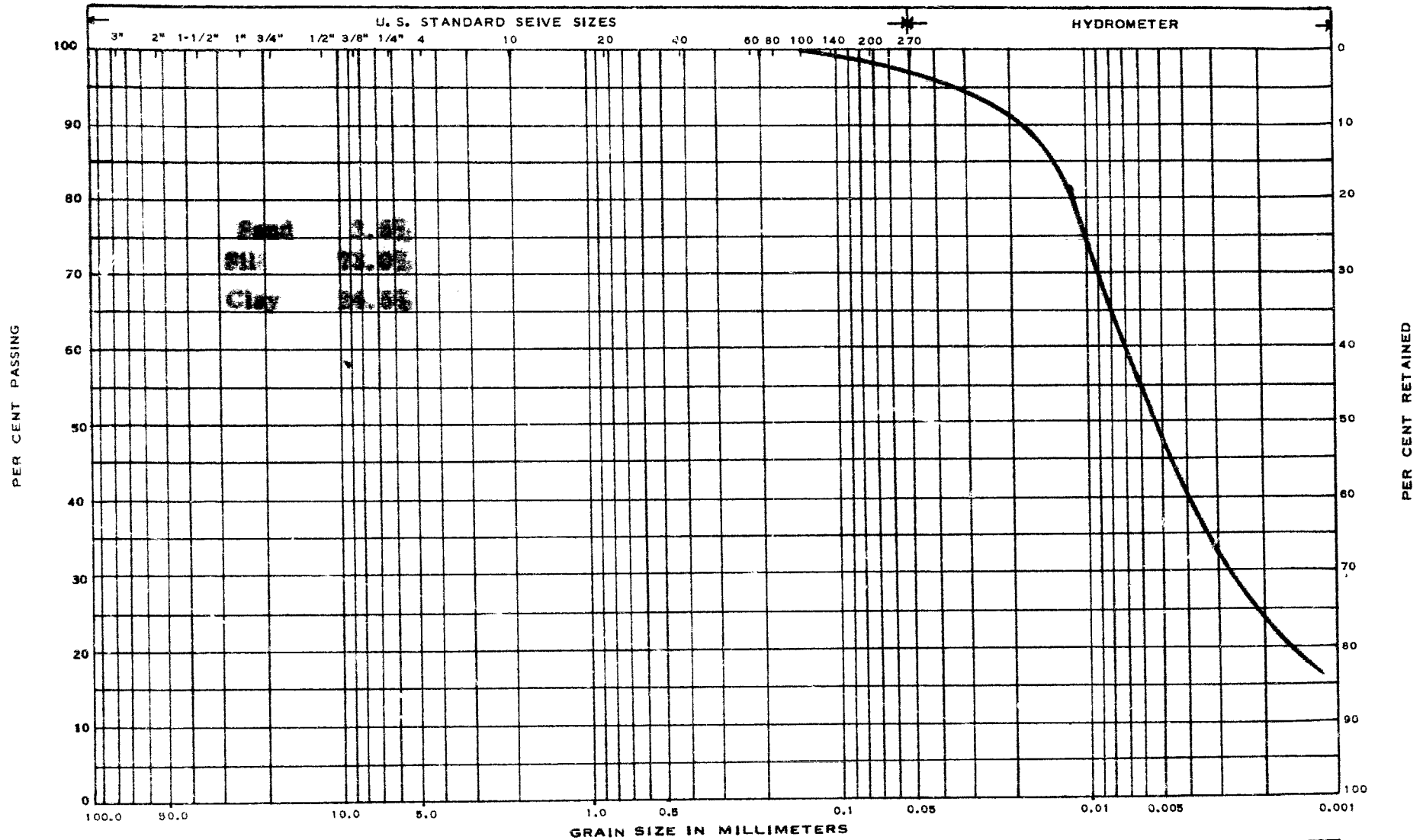
## M.I.T. CLASSIFICATION

JOB NAME Lepteros Bridge JOB No. 50340 BOREHOLE No. 1 SAMPLE No. 20

DEPTH 45' - 48' ELEVATION \_\_\_\_\_ REMARKS General classification - Silt

e. m. peto associates ltd.

Toronto 19, Ontario



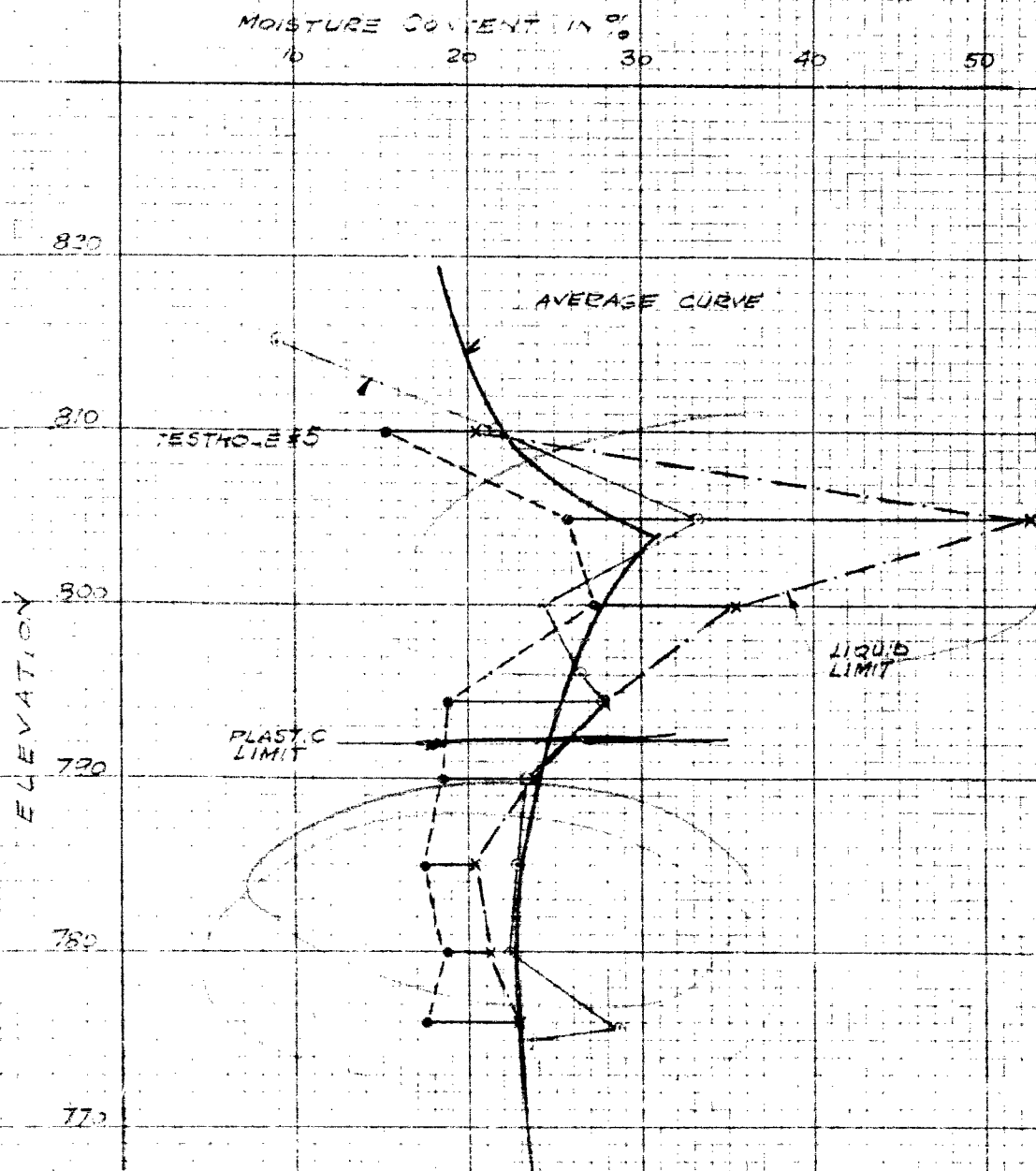
STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
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MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Assessors Bridge Port Perry JOB NO. 80240 HOLE NO. 4 SAMPLE NO. 17 B

DEPTH 22'4" ELEVATION 37'8" REMARKS Textural Classification Clayey silt

# ATTERBERG LIMITS & MOISTURE CONTENT VERSUS ELEVATION



Job No 59249

e.m. p'so associates Pa.

APPENDIX III

SHEAR STRENGTH RESULTS

# UNCONFINED COMPRESSION TEST RESULTS

Job No. 59249

Hole Sample No.	No.	Depth	Elevation Ft.	Nat. M.C.	Densities, p.c.f.		Degree of Saturation	Void Ratio, e	% Strain at Failure	w/c Shear Strength p.s.f.	Description
					Wet	Dry					
1	15A	30'6"-31'	803.4	33.3	122.0	91.5	100	.843	20.0	1805	Grey silty clay)
	15B	31'-31'6"	802.9	33.3	123.5	92.5	100	.824	13.3	1885	Grey silty Clay)
											) Friable
3	6	26'6"-27'6"	809.0				100		20.0	1300	Test performed in field
	10	30'6"-31'6"	804.6		124.4		100		20.0	1440	as above
	11	35' - 36'	800.1		131.5		100		20.0	1730	as above
	13	47' - 48'	788.1				100		20.0	575	as above

# FIELD VANE TEST

<u>Test Hole #</u>	<u>Depth</u>	<u>Elevation</u>	<u>Shear Strength p.s.f.</u>
3	3' - 3'6"	832.7	598
	7' - 7'6"	828.7	650
	13' - 13'6"	822.7	650
	28' - 28'6"	807.7	2210
	39' - 39'6"	796.7	1960
	48' - 48'6"	787.7	1230
	54' - 54'6"	781.7	2210
	60' - 60'6"	775.7	2950
	69'6" - 70'	766.3	2950
2	44' - 44'6"	791.3	1230
	48'6" - 49'	786.9	1960
	56'6" - 57'	778.7	2210
	53' - 58'6"	777.3	1960

# LABORATORY VANE TESTS

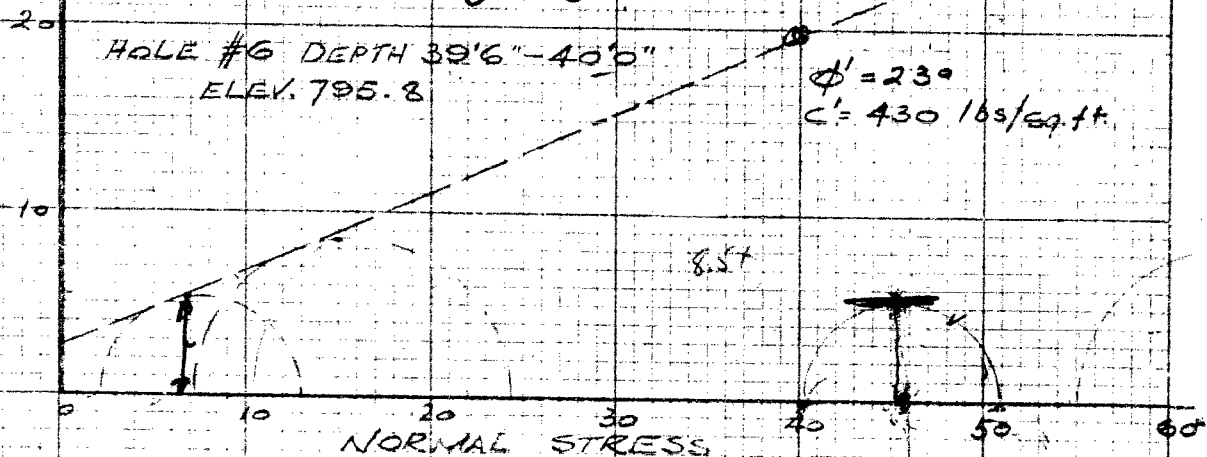
<u>Test Hole #</u>	<u>Depth</u>	<u>Elevation</u>	<u>Shear Strength in p.s.f.</u>
2	29'8"-30'0"	805.6	925 925 775 1000
2	30'0"-30'6"	805.3	1525 1573 925 1000
4	36'6"-37'0"	798.0	510 650
2	42'6"-43'0"	792.9	600
4	51'6"-52'0"	782.4	590 620



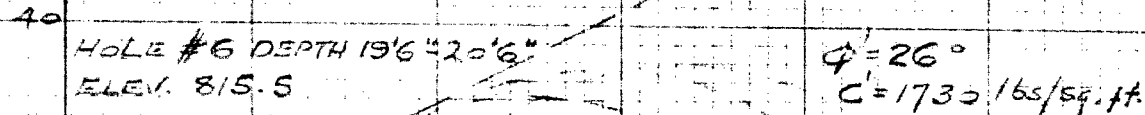
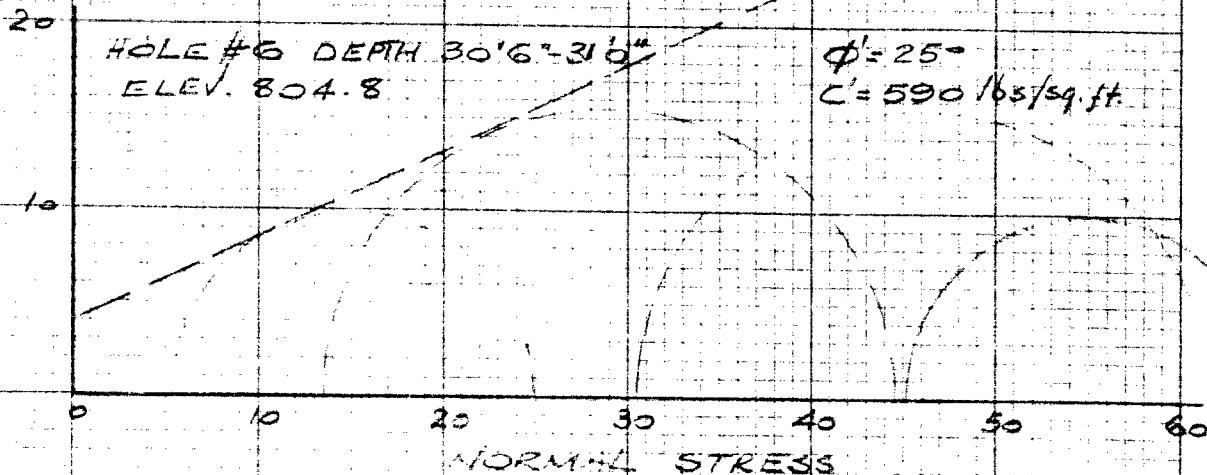
# TRIAXIAL SHEAR TEST RESULTS.

U.U

9" x 1"



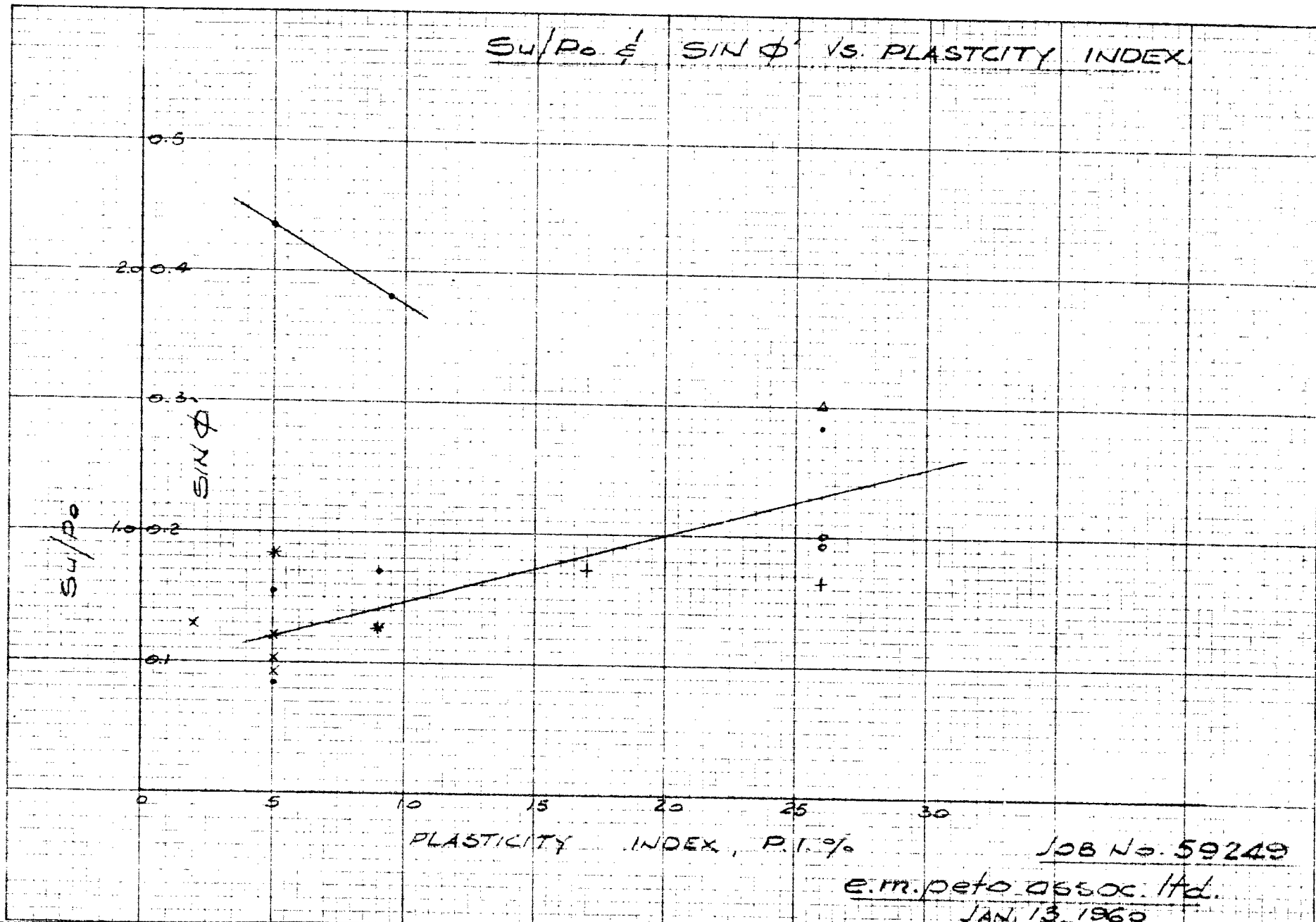
SHEAR STRESS



Job No. 59249

G.M. Peto Associates Ltd.  
JAN 13, 1960

$S_u/p_o$  &  $\sin \phi'$  VS. PLASTICITY INDEX

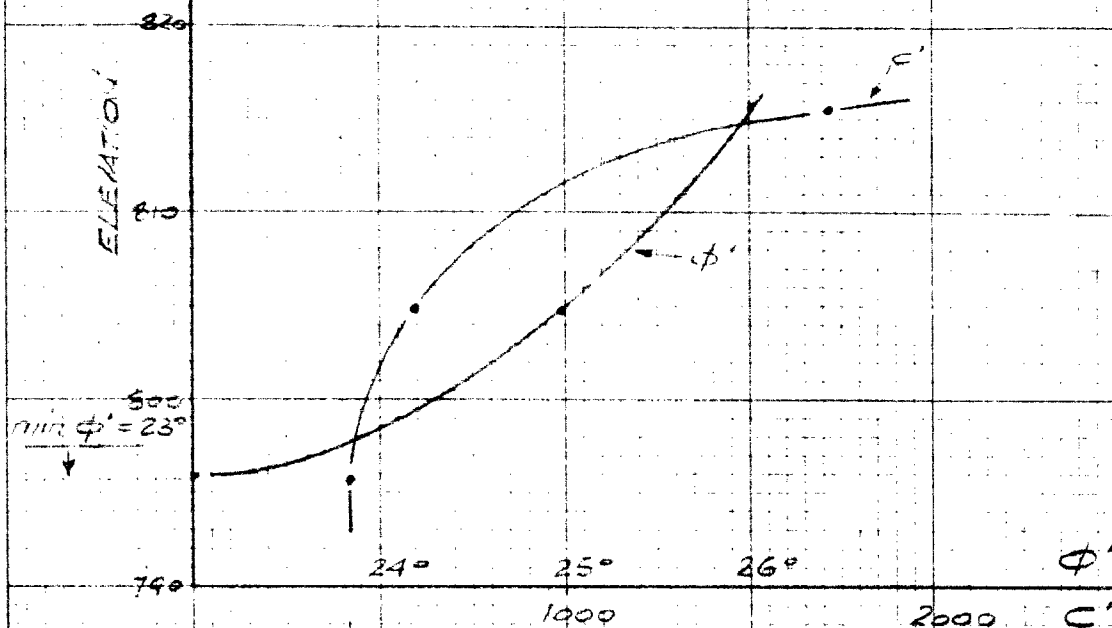
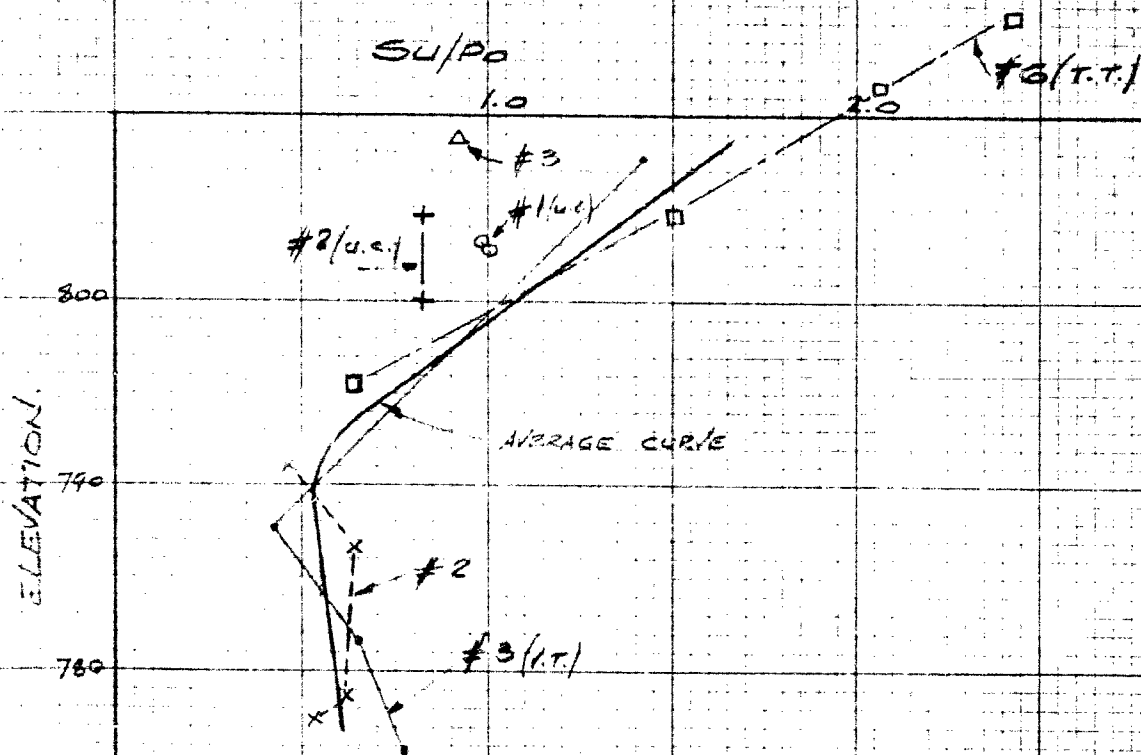


JOB No. 59249

e.m. peto assoc. ltd.

JAN. 13, 1960

SU/P<sub>0</sub> 15. ELEVATION.



$\min C' = 420 \text{ p.s.f.}$

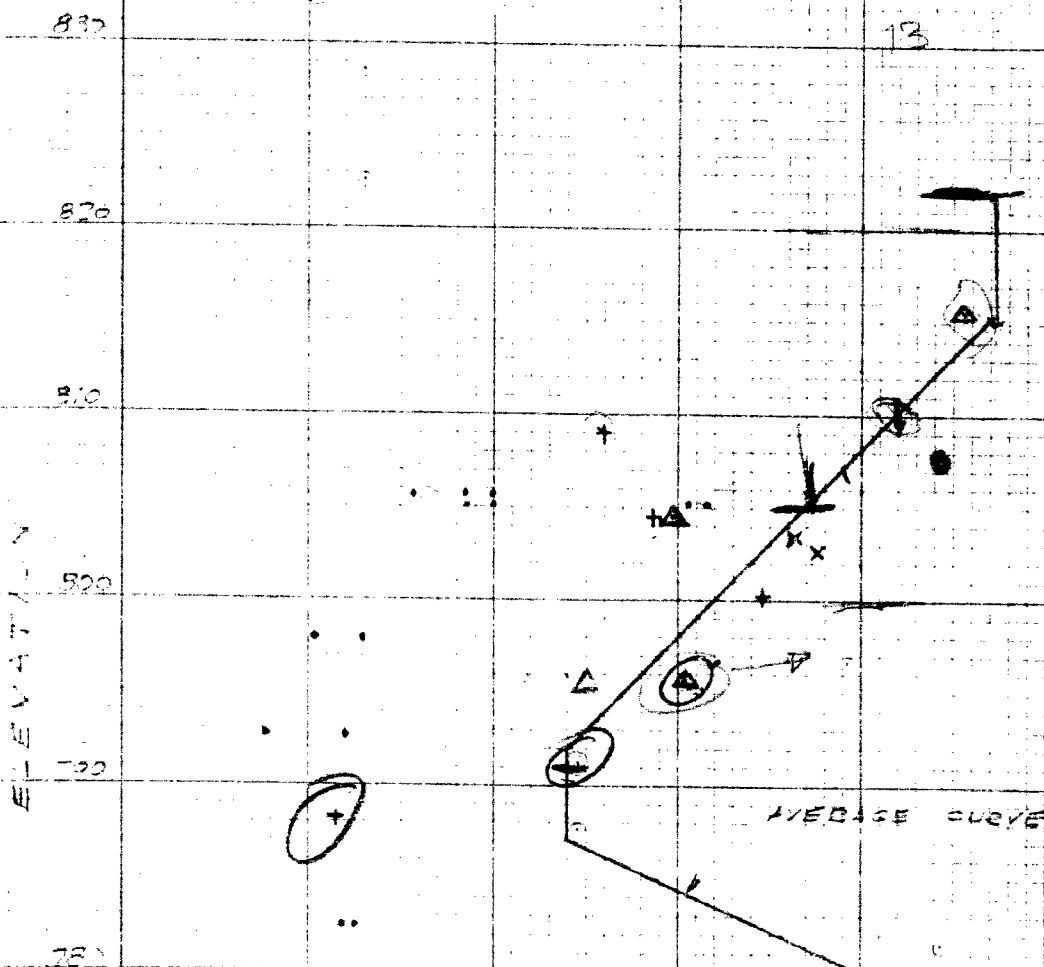
Job No. 59249

e.m. peto assoc. ltd.

JAN. 13, 1960

# SHEAR STRENGTH VS. ELEVATION.

SHEAR STRENGTH IN P.C.F.  
500 1000 1500 2000 2500



*chemical values  
not put up U.U.*

FIELD VANE TEST RESULTS: TESTHOLE #3  
TESTHOLE #2

- LABORATORY VANE TEST RESULTS
- UNCONFINED COMPRESSIVE TEST RESULTS
- TRIAXIAL SHEAR TEST RESULTS

empeto associates Inc.

## TABLE OF VARIOUS RELATIONSHIPS

Test Hole No.	Depth	Elevation	$s_u$ p.s.f.	P. L.	P. E.	Overburden $p_o$	$s_u$ $p_o$	$\phi'$
3	28'3"	807.7	2210	15	5	1550	1.42	
	39'3"	796.7	1960	18	9	2325	0.85	
	48'3"	787.7	1230	19	5	2920	0.42	
	54'3"	781.7	2210	18	2	3400	0.65	
	60'3"	775.7	2950	18	5	3850	0.77	
2	27'3"	808.1	73000	15	5	1500	-	
	44'3"	791.3	1230	19	5	2700	0.46	
	48'9"	786.9	1960	18	2	3050	0.64	
	56'9"	778.7	2210	18	5	3690	0.61	
	58'3"	777.3	1960	18	5	3720	0.53	
1	30'9"	803.4	1805	26	26	1850	0.98	
	31'3"	802.9	1885	26	26	1880	1.00	
2	31'0"	804.6	1440	26	26	1750	0.83	
	35'6"	800.1	1790	18	17	2074	0.83	
3	27'0"	809.0	1300	15	5	1430	0.91	
6	20'0"	815.5	2293	15	5	950	2.41	26°
	30'9"	804.8	1495	26	26	1000	1.50	25°
	39'9"	795.8	1510	19	9	2400	0.83	23°

APPENDIX IV

SLOPE STABILITY ANALYSIS

PART II  
APPENDICES

APPENDIX I

CONSOLIDATION TEST RESULTS



SECRET

Test Hole No. \_\_\_\_\_

Death **27** - **28**

APPLIED PRESSURE KSF	H.C. %	S %	AVERAGE PRESSURE $\frac{P_1 + P_2}{2}$	VOID RATIO e	COEFF. OF COMPRES- SIBILITY $\frac{dV}{V dp}$	COEFF. OF VOLUME DECREASE $\frac{dV}{V}$	DENSITY CORRECTION $\frac{dH}{H_0}$ %	t 90 in min	COEFF. OF CONSOLID- ATION Cv Sq. in./min	COEFF. OF PERMEA- BILITY K in./min
0	27.0	100	0.00	0.761	0.0179	0.0180	0	1.5	12.00	10 <sup>-5</sup>
0.5			0.75	0.778	0.0180	0.0177	1.13	2.3	21.00	1.000
1.0			1.50	0.808	0.0220	0.0131	2.02	10.2	14.72	0.733
2.0			3.00	0.876	0.0180	0.0072	3.76	9.4	11.20	0.420
4.0			6.00	0.952	0.0093	0.0033	5.96	6.7	15.21	0.265
6.0			12.00	0.981	0.0044	0.0017	7.40	5.7	17.00	0.247
16.0			20.00	0.988	0.0020	0.0014	8.84	4.9	14.50	0.010
24.0			16.00	0.980			11.02			
6.0			5.00	0.987						
2.0			1.50	0.994						
1.0	21.0	100		0.615						

X  
124.5 lbs. per cu. ft.

Test No. 2 ..... Test Hole No. 4 ..... Depth 37' - 38'

APPLIED PRESSURE Ksf	M.C. %	S %	AVERAGE PRESSURE $\frac{P_1 + P_2}{2}$	VOID RATIO e	COEFF. OF CONSOLIDATION Cc	COEFF. OF VOLUME DECREASE mv	UNIT COMPRESSION AH/Ho %	t 90 in min.	COEFF. OF CONSOLIDATION Cv	COEFF. OF PERMEABILITY K
0	29.6	100		0.920						
			0.25	0.920	0.0020	0.0181	0	3.6	32.7	2.000
0.5				0.914			0.72	0.72	100.0	3.000
			0.75	0.914	0.0020	0.0044				
1.0				0.910			1.22	1.10	100.0	1.440
			1.5	0.910	0.0020	0.0027				
2.0				0.903			2.05	2.20	50.1	0.400
			2.0	0.903	0.0020	0.0019				
4.0				0.798			2.75	2.50	42.9	0.200
			4.0	0.798	0.0040	0.0022				
8.0				0.782			4.20	3.90	25.0	0.271
			12.0	0.782	0.0030	0.0015				
16.0				0.780			6.41	3.25	20.0	0.234
			20.0	0.780	0.0024	0.0014				
24.0				0.757			8.55			
				0.751						
4.0				0.751						
2.0				0.775						
1.0	28.0	100		0.730						

112.5 lbs. per cu. ft.

APPLIED PRESSURE K.S.F.	W.C. % 9%	S %	AVERAGE PRESSURE $\frac{P_1 + P_2}{2}$	VOID RATIO e	COEFF. OF COMPRESSION CONSOLIDITY $\frac{e_0 - e}{P_2 - P_1}$	COEFF. OF VOLUME DECREASE $\frac{\Delta V}{V_0}$	UNIT COMPRESSION $\frac{\Delta H}{H_0}$ %	t 90 in min.	COEFF. OF CONSOLIDATION C <sub>v</sub> $\frac{C_u}{C_c}$	COEFF. OF PERMEABILITY K in/min
0	23.3	100	0.75	0.800	0.0040	0.0144	0	7.25	60.0	19.700
0.5			0.75	0.807	0.0030	0.0130	2.21	0.00	10.75	1.300
1.0			1.5	0.806	0.0102	0.0080	2.25	0.00	12.17	0.470
2.0			2.0	0.801	0.0145	0.0090	4.73	10.0	10.51	0.003
4.0			5.0	0.800	0.0083	0.0041	6.02	2.0	11.20	0.250
5.0			10.0	0.803	0.0094	0.0024	8.06	5.0	10.00	0.150
10.0			20.0	0.800	0.0080	0.0013	10.00	7.0	12.10	0.000
20.0				0.800			11.00			
2.0				0.844						
1.0				0.888						
0.5	18.0	100		0.900						

*h* = 120.0 lbs. per cu. ft.

Test No. 1000000

Test Hole No. . . . .

Depth 270-275

APPLIED PRESSURE KSF	M.C. %	S %	AVERAGE PRESSURE $\frac{P_1 + P_2}{2}$	VOID RATIO e	COEFF. OF COMPRE- SSIBILITY $\frac{e}{V_s \Delta p / K}$	COEFF. OF VOLUME DECREASE $\frac{\Delta V}{V_s \Delta p / K}$	UNIT COMPRESSION $\frac{\Delta H}{H_0}$ %	t 90 17 min.	COEFF. OF CONSOLID- ATION Cv $\frac{S_{90}}{S_{90} / \text{min}}$	COEFF. OF PERME- ABILITY K $\frac{1}{\text{min}}$
0	22.0	100		0.600						
0.5			0.25	0.544	0.0720	0.0448	0	14.0	8.40	1.000
1.0			0.75	0.544	0.0320	0.0141	2.50	7.5	15.19	1.100
2.0			1.5	0.532	0.0140	0.0061	4.50	4.5	23.70	1.201
3.0			2.0	0.522	0.0081	0.0036	5.25	3.5	25.10	0.846
4.0			3.0	0.506	0.0050	0.0023	6.36	2.5	40.73	0.500
6.0			6.0	0.506	0.0025	0.0013	8.10	2.25	42.30	0.398
10.0			10.0	0.490	0.0011	0.0006	9.50	4.2	22.04	0.005
24.0			24.0	0.481			10.50			
5.0				0.485						
2.0				0.495						
0.5	10.1	100		0.504						

151.0 lbs. per cu. ft.

Test No. .... Test Hole No. .... Depth 37' - 38'

APPLIED PRESSURE KSF	MC %	S %	AVERAGE PRESSURE $\frac{P_1 + P_2}{2}$	VOID RATIO e	COEFF. OF COMPRES- SIBILITY $\frac{\Delta V}{V \Delta p} \text{ ft}^3/\text{K}$	COEFF. OF VOLUME DECREASE $\frac{\Delta H}{H_0}$ %	UNIT COMPRESSION $\Delta H/H_0$ %	t 90 in min.	COEFF. OF CONSOLID- ATION Cv $\frac{\text{cm}^2}{\text{min}}$	COEFF. OF PERMEA- BILITY K $\frac{\text{in}}{\text{min}}$
0	19.6	100	0.28	0.500	0.0120	0.0120	0	4.6	35.00	1.000
0.5			0.75	0.494	0.0140	0.0061	0.71	6.4	30.41	1.241
1.0			1.50	0.488	0.0079	0.0047	1.35	8.5	32.00	0.780
2.0			3.00	0.481	0.0080	0.0041	2.31	6.1	27.00	0.577
4.0			6.00	0.469	0.0035	0.0026	3.37	4.0	21.00	0.305
8.0			12.00	0.454	0.0018	0.0016	4.97	2.5	41.00	0.146
16.0			24.00	0.439	0.0016	0.0011	6.69	1.5	46.00	0.230
24.0				0.423			8.30			
32.0				0.420						
40.0				0.430						
48.0				0.441						
56.0										
64.0										
72.0										
80.0										
88.0										
96.0										
104.0										
112.0										
120.0										
128.0										
136.0										
144.0										
152.0										
160.0										
168.0										
176.0										
184.0										
192.0										
200.0										
208.0										
216.0										
224.0										
232.0										
240.0										
248.0										
256.0										
264.0										
272.0										
280.0										
288.0										
296.0										
304.0										
312.0										
320.0										
328.0										
336.0										
344.0										
352.0										
360.0										
368.0										
376.0										
384.0										
392.0										
400.0										
408.0										
416.0										
424.0										
432.0										
440.0										
448.0										
456.0										
464.0										
472.0										
480.0										
488.0										
496.0										
504.0										
512.0										
520.0										
528.0										
536.0										
544.0										
552.0										
560.0										
568.0										
576.0										
584.0										
592.0										
600.0										
608.0										
616.0										
624.0										
632.0										
640.0										
648.0										
656.0										
664.0										
672.0										
680.0										
688.0										
696.0										
704.0										
712.0										
720.0										
728.0										
736.0										
744.0										
752.0										
760.0										
768.0										
776.0										
784.0										
792.0										
800.0										
808.0										
816.0										
824.0										
832.0										
840.0										
848.0										
856.0										
864.0										
872.0										
880.0										
888.0										
896.0										
904.0										
912.0										
920.0										
928.0										
936.0										
944.0										
952.0										
960.0										
968.0										
976.0										
984.0										
992.0										
1000.0										

$\gamma = 121.5 \text{ lbs. per cu. ft.}$

Test No. .... Test Hole No. .... Depth 425 - 430

APPLIED PRESSURE Ksf	M.C. %	S %	AVERAGE PRESSURE $\frac{P_1 + P_2}{2}$	VOID RATIO e	COEFF. OF COMPRE- SSIBILITY $\frac{dv}{v dp}/K$	COEFF. OF VOLUME DECREASE $\frac{dV}{V dp}/K$	UNIT COMPRESSION $\frac{dH}{H dp}$ %/K	$\bar{c}_{90}$ in min.	COEFF. OF CONSOLID- ATION CV $\frac{dV}{V dp}/min$	COEFF. OF PERMEA- BILITY K in/min
0	22.9	100		0.620						
			2.5	0.610	0.0110	0.0046	0	1.0	22.7	1.207
1.0			1.5	0.600	0.0070	0.0044	1.25	1.7	27.5	1.540
2.0			3.0	0.602	0.0094	0.0022	1.89	1.7	34.0	0.783
4.0			6.0	0.595	0.0050	0.0016	2.52	1.8	31.8	0.514
6.0			12.0	0.583	0.0021	0.0013	3.60	1.8	35.0	0.403
10.0			20.0	0.568	0.0016	0.0012	5.03	2.0	46.1	0.282
24.0				0.553			7.01			
6.0				0.565						
3.0				0.580						
1.0	20.1	100		0.574						

$\gamma = 126.0$  lbs. per cu. ft.

Test No. **7** Test Hole No. **574 - 575** Depth **574 - 575**

APPLIED PRESSURE Ksf	M.C. %	S %	AVERAGE PRESSURE $\frac{P_1 + P_2}{2}$	VOID RATIO e	COEFF. OF COMPRES- SIBILITY $\frac{dv}{v \Delta p}/k$	COEFF OF VOLUME DECREASE $\frac{\Delta H}{H_0}$ %	DRIFT COMPRESSION $\frac{\Delta H}{H_0}$ %	t 90 in min.	COEFF OF CONSOLID- ATION Cv $\frac{cm^2}{min}$	COEFF OF PERMEA- BILITY K $\frac{in}{min}$
0	24.4	100.0	0.5	0.540	0.0050	0.0022	0	16.0	10 <sup>-3</sup>	10 <sup>-3</sup>
1.0			1.5	0.538	0.0100	0.0035	2.41	7.1	12.22	0.002
2.0			3.0	0.575	0.0085	0.0054	4.51	4.0	22.45	0.000
4.0			5.0	0.565	0.0046	0.0035	5.92	2.0	24.00	0.043
8.0			12.0	0.538	0.0022	0.0015	7.00	2.25	44.99	0.200
16.0			20.0	0.520	0.0012	0.0003	8.51	3.70	26.00	0.152
24.0				0.510			9.45			
4.0				0.514						
2.0				0.502						
1.0	12.3			0.527						

✓ = 120.0 lbs. per sq. ft.





Test No. 10 Test Hole No. 2 Depth 350' - 394'

[illegible]

Test No. 11 Test Hole No. 1 Depth 7' 0" - 7' 6"

APPLIED PRESSURE KSF	M.C. %	S %	AVERAGE PRESSURE $\frac{P_1 + P_2}{2}$	VOID RATIO e	COEFF. OF COMPRE- SSIBILITY $\frac{e_v}{e}$	COEFF. OF VOLUME DECREASE $\frac{\Delta V}{V}$	DRY COMPRESSION $\frac{\Delta H}{H_0}$ %	t 90 in min.	COEFF. OF CONSOLIDATION C <sub>v</sub> $\frac{S_{90}}{S_{90} + S_{100}}$	COEFF. OF PERMEABILITY K in/min.
0	24.6	100.0	0.0	0.750	0.0140	0.0041	0	10.0	11.12	0.470
1.0			1.0	0.716	0.0100	0.0030	2.00	6.0	12.61	0.366
2.0			3.0	0.706	0.0090	0.0033	3.00	5.4	13.10	0.308
4.0			5.0	0.696	0.0085	0.0036	4.00	11.4	6.15	0.183
6.0			12.0	0.682	0.0082	0.0035	6.00	6.1	12.45	0.168
18.0			22.0	0.656	0.0085	0.0035	8.00	10.0	9.60	0.075
24.0				0.606			11.00			
6.0				0.623						
1.0	23.2			0.630						

w = 125.4 lbs. per cu. ft.

Depth 999 - 974

APPLIED PRESSURE Ksf	WATER % S	S %	AVERAGE PRESSURE $\frac{P_1 + P_2}{2}$	VOID RATIO e	COEFF. OF COMPRE- SSIBILITY 8V5qft/K	COEFF. OF VOLUME DECREASE MV5qft/K	UNIT COMPRESSION $\Delta H/H_0$ %	t 90 in min.	COEFF. OF CONSOLID- ATION CV Sqft/min.	COEFF. OF PERMEA- BILITY K in/min.
0	20.0	100	0.5	0.715	0.0020	0.0042	0	10.1	10.0	1.0
1.0			1.5	0.713	0.0020	0.0043	1.75	2.0	17.3	1.375
2.0			2.0	0.704	0.0100	0.0060	4.57	4.0	21.0	0.612
4.0			3.0	0.684	0.0060	0.0051	0.20	2.0	25.0	0.305
6.0			12.0	0.665	0.0032	0.0020	2.43	2.0	32.4	0.246
10.0			20.0	0.637	0.0012	0.0011	10.01	2.0	33.1	0.184
24.0				0.623			12.01			
3.0				0.631						
2.0				0.587						
1.0	22.7			0.663						

$\gamma_w = 123.5$  lbs. per cu. ft.

APPENDIX II  
GRAPHICAL REPRESENTATION  
OF  
VARIOUS CONSOLIDATION RELATIONSHIPS

**e. m. peto associates ltd.**

YOUR REFERENCE:-

OUR REFERENCE:- 59249

1287 caledonia road,  
TORONTO 19, ONTARIO  
RUssell 9-1126

June 10th, 1960.

Mr. A. Rutka,  
Acting Materials and Research Engineer,  
Materials and Research Section,  
Department of Highways of Ontario,  
Parliament Buildings,  
Toronto, Ontario.

Re: East Cross Creek

Dear Sir:

We enclose herewith the settlement analysis on the above mentioned job which were omitted from our report which was submitted to your office yesterday.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,



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

Sb

## SPREAD FOLDING DESIGN

FOOTING SIZE 15 x 60 ft.  
CONTACT PRESSURE: 4100 psf

e. m. peto associates ltd,

## SETTLEMENT ANALYSIS

- 1.
- Magnitude of the consolidation settlement

**Under the centre point of the footing.**

Name of Job East Cross Creek Date June 9th. 1960 Job No. 59249

[illegible]

\* This value of  $m_v$  was interpolated from the given  $m_v$  versus  $p$  curve as the curve was drawn from 0.6 ksf applied pressure only.

e. m. peto associates ltd.

NAME OF JOB Eastcross Creek

DATE June 9th. 1960

JOB NO. 59249

\* Applied pressure:  $4100 - 104 = 3996$  psf.

e. m. peto associates ltd.

### 1. Magnitude of the consolidation settlement

[illegible]



e. m. peto associates ltd.

NAME OF JOB East Cross Creek

DATE June 9th. 1960

JOB NO. 59249

[illegible]

\* Applied pressure 3000 - 127.5 \*

# FRICTION PILE FOUNDATION

e. m. peto associates ltd.

## SETTLEMENT ANALYSIS

### 1. Magnitude of the consolidation settlement

Under the centre point of pile foundation

(12 ft. x 60 ft.)

June 9th. 1960

Name of Job Eastcross Creek

Date

Job No.

**59249**

Elevation	Depth $\Delta H$	Calculation of the Influence Factor					$\Delta p$	$\Delta p_{av}$	Coefficient of Volume Decrease $m_v$	$\Delta h$	Settlement $\Delta S$
		Z	B = 6 ft. B/2	L = 30 ft. L/2	$I_{\alpha}/4$	$I_{\alpha}$					
1	2	3	4	5	6	7	8	9	10	11	12
793.0	5.0	0			.250	1.000	1870				
788.0		5	1.200	6.000	.218	0.872	1630				
783.0	5.0	10	0.600	3.000	.156	0.624	1165	1592	0.0074	10.0	0.118
775.5	7.5	17.5	0.343	1.715	.099	0.396	741				
768.0	7.5	25.0	0.240	1.200	.068	0.272	508	773	0.0113	15.0	0.131
753.0	15.0	40.0	0.150	0.750	.037	0.148	277				
738.0	15.0	55.0	0.109	0.545	.023	0.092	172	298	0.0140 *	30.0	0.125
										AS ==	0.374 ft.
											= 4.5 in.

This value of  $m_v$  is the assumed value which according to the tables of consolidation tests is a reasonable value for the average pressure of 0.3 k/sq. ft.

e. m. peto associates ltd.

SETTLEMENT ANALYSIS  
Under the centre point of pile foundation

DATE \_\_\_\_\_

JOB NO. 59249

[illegible]

# SPREAD FOOTING DESIGN

FOOTING SIZE 15 x 60 ft.  
CONTACT PRESSURE: 4100 psf

e. m. peto associates ltd.

## SETTLEMENT ANALYSIS

### 1. Magnitude of the consolidation settlement

Under the centre point of the footing.

Name of Job East Cross Creek Date June 9th. 1960 Job No. 59249

Elevation	Depth $\Delta H$	Calculation of The Influence Factor					$\Delta p$	$\Delta p_{av}$	Coefficient of Volume Decrease $m_v$	$\Delta H$	Settlement L.S.
		Z	B=	L=	$I_{\sigma}/4$	$I_{\sigma}$					
			$B/2$	$L/2$							
1	2	3	4	5	6	7	8	9	10	11	12
822.0		0			0.250	1.000	4100				
811.0	11.0	11.0	0.681	2.730	0.168	0.672	2760				
	11.0							2774	0.0051	22.0	0.311
800.0	10.0	22.0	0.341	1.363	0.093	0.372	1525				
790.0	10.0	32.0	0.234	0.939	0.063	0.252	1030				
780.0	20.0	42.0	0.179	0.715	0.043	0.172	705	1058	0.0095	20.0	0.201
760.0	22.0	62.0	0.121	0.484	0.023	0.092	377				
		82.0	0.092	0.366	0.014	0.056	230	407	0.0140 *	42.0	0.239
										AS =	0.751 ft.
											= 9.0 in.

\* This value of  $m_v$  was interpolated from the given  $m_v$  versus  $p$  curve as the curve was drawn from 0.6 ksf applied pressure only.

e. m. peto associates ltd.

NAME OF JOB Eastcross Creek

DATE June 9th. 1960

JOB NO. 59249

\* Applied pressure:  $4100 - 104 = 3996$  psf.

e. m. peto associates ltd.

### 1. Magnitude of the consolidation settlement

[illegible]

e. m. peto associates lid.

NAME OF JOE East Cross Creek

DATE June 9th. 1960

JOE NO. 59249

[illegible]

\* Applied pressure 3000 - 127.5 \*

# FRICTION PILE FOUNDATION

e. m. peto associates ltd.

## SETTLEMENT ANALYSIS

### 1. Magnitude of the consolidation settlement

Under the centre point of pile foundation

(12 ft. x 60 ft.)

Name of Job Eastcross Creek

Date June 9th. 1960

Job No. 59249

Elevation	Depth $\Delta H$	Calculation of the Influence Factor					$\Delta p$	$\Delta p_{av}$	Coefficient of Volume Decrease $m_v$	$\Delta h$	Settlement $\Delta S$
		Z	B = 6 ft. L = 30 ft.		$I_{\sigma/4}$	$I_{\sigma}$					
			B/2	L/2							
1	2	3	4	5	6	7	8	9	10	11	12
793.0	5.0	0			.250	1.000	1870				
788.0		5	1.200	6.000	.218	0.872	1630				
783.0	5.0	10	0.600	3.000	.156	0.624	1165	1592	0.0074	10.0	0.118
775.5	7.5	17.5	0.343	1.715	.099	0.396	741				
768.0	7.5	25.0	0.240	1.200	.068	0.272	508	773	0.0113	15.0	0.131
751.0	15.0	40.0	0.150	0.750	.037	0.148	277				
738.0	15.0	55.0	0.109	0.545	.023	0.092	172	298	0.0140 *	30.0	0.125
										AS ==	0.374 ft.
											= 4.5 in.

This value of  $m_v$  is the assumed value which according to the tables of consolidation tests is a reasonable value for the average pressure of 0.3 k/sq. ft.



**FRICTION PILE FOUNDATION**

e. m. peto associates ltd.

## SETTLEMENT ANALYSIS

Under the centre point of pile foundation

NAME OF JOB Eastcross Creek

DATE \_\_\_\_\_

JOB NO. 59249

[illegible]

# SPREAD FOOTING DESIGN

FOOTING SIZE 15 x 60 ft.  
CONTACT PRESSURE: 4100 psf

e. m. peto associates ltd.

## SETTLEMENT ANALYSIS

### 1. Magnitude of the consolidation settlement

Under the centre point of the footing.

Name of Job East Cross Creek Date June 9th. 1960 Job No. 59249

Elevation	Depth $\Delta H$	Calculation of the Influence Factor					$\Delta p$	$\Delta p_{av}$	Coefficient of Volume Decrease $m_v$	$\Delta h$	Settlement AS
		Z	B=	L=	$I_{\sigma/4}$	$I_{\sigma}$					
			$B/2$	$L/2$							
1	2	3	4	5	6	7	8	9	10	11	12
822.0		0			0.250	1.000	4100				
811.0	11.0	11.0	0.681	2.730	0.168	0.672	2760				
	11.0							2774	0.0051	22.0	0.311
800.0	10.0	22.0	0.341	1.363	0.093	0.372	1525				
790.0	10.0	32.0	0.234	0.939	0.063	0.252	1030				
780.0	20.0	42.0	0.179	0.715	0.043	0.172	705	1058	0.0095	20.0	0.201
760.0	22.0	62.0	0.121	0.484	0.023	0.092	377				
		82.0	0.092	0.366	0.014	0.056	230	407	0.0140 *	42.0	0.239
										AS =	0.751 ft.
											= 9.0 in.

\* This value of  $m_v$  was interpolated from the given  $m_v$  versus  $p$  curve as the curve was drawn from 0.0 ksf applied pressure only.

e. m. peto associates ltd.

NAME OF JOB      Eastcross Creek

DATE **June 9th. 1960**

JOB NO. 59249

\* Applied pressure:  $4100 - 104 = 3996$  psf.

e. m. peto associates ltd.

### 1. Magnitude of the consolidation settlement

[illegible]

# EMBANKMENT

e. m. peto associates ltd.

## SETTLEMENT ANALYSIS

NAME OF JOB East Cross Creek

DATE June 9th. 1960

JOB NO. 59249

Elevation	Depth D.H., ft	Density, 10/cu.ft.	Over- burden pressure Po psf	Calculation of vertical stress Influence Factor IG					Pressure Analysis Data				Settlement Computation				
				Z	B/2	L/2	I <sub>0</sub> /4	I <sub>0</sub>	Applied Pressure ΔP	Stress Release Pa	Average Po	Average E <sub>av</sub> (ΔP=Pa)	e <sub>0</sub>	e	Δe	Δh	ΔS
835.5			0														
	18.5	65.5															
817.0			127.5	0			.250	1.000	2872.5	*							
	9.0	126.0															
808.0			700	9.0	5.55		0.249	0.996	2860								
	8.0	126.0															
800.0			1208	17.0	2.94		0.246	0.984	2820		663	3518	0.647	0.621	.026	17.0	0.268
	10.0	126.0															
790.0			1844	27.0	1.85		0.238	0.952	2730								
	10.0	126.0									1844	4134	0.634	0.617	.017	20.0	0.208
780.0			2480	37.0	1.35		0.225	0.900	2580								
	20.0	126.0															
760.0			3752	57.0	0.88		0.195	0.780	2240								
	22.0	126.0									3815	6050	0.619	0.607	.012	42.0	0.312
738.0			5151	79.0	0.63		0.163	0.652	1870							ΔS =	0.788 ft.
																=	9.45 in.

\* Applied pressure 3000 - 127.5 \*

# FRICTION PILE FOUNDATION

e. m. peto associates ltd.

## SETTLEMENT ANALYSIS

### 1. Magnitude of the consolidation settlement

Under the centre point of pile foundation  
(12 ft. x 60 ft.)

Name of Job Lastcross Creek      Date June 9th. 1960      Job No. 59249

Elevation	Depth $\Delta H$	Calculation of the Influence Factor					$\Delta p$	$\Delta p_{av}$	Coefficient of Volume Decrease $m_v$	$\Delta h$	Settlement $\Delta S$
		Z	B = 6 ft. L = 30 ft.		$I_{\sigma'/4}$	$I_{\sigma'}$					
			B/2	L/2							
1	2	3	4	5	6	7	8	9	10	11	12
793.0	5.0	0			.250	1.000	1870				
788.0		5	1.200	6.000	.218	0.872	1630	1592	0.0074	10.0	0.118
783.0	5.0	10	0.600	3.000	.156	0.624	1165				
775.5	7.5	17.5	0.343	1.715	.099	0.396	741				
768.0	7.5	25.0	0.240	1.200	.068	0.272	508	773	0.0113	15.0	0.131
753.0	15.0	40.0	0.150	0.750	.037	0.148	277				
738.0	15.0	55.0	0.109	0.545	.023	0.092	172	298	0.0140 *	30.0	0.125
										AS ==	0.374 ft
											= 4.5 in.

This value of  $m_v$  is the assumed value which according to the tables of consolidation tests is a reasonable value for the average pressure of 0.3 k/sq. ft.

FRICTION FILL FOUNDATION

e. m. peto associates ltd.

## SETTLEMENT ANALYSIS

Under the centre point of pile foundation

NAME OF JOB Eastcross Creek

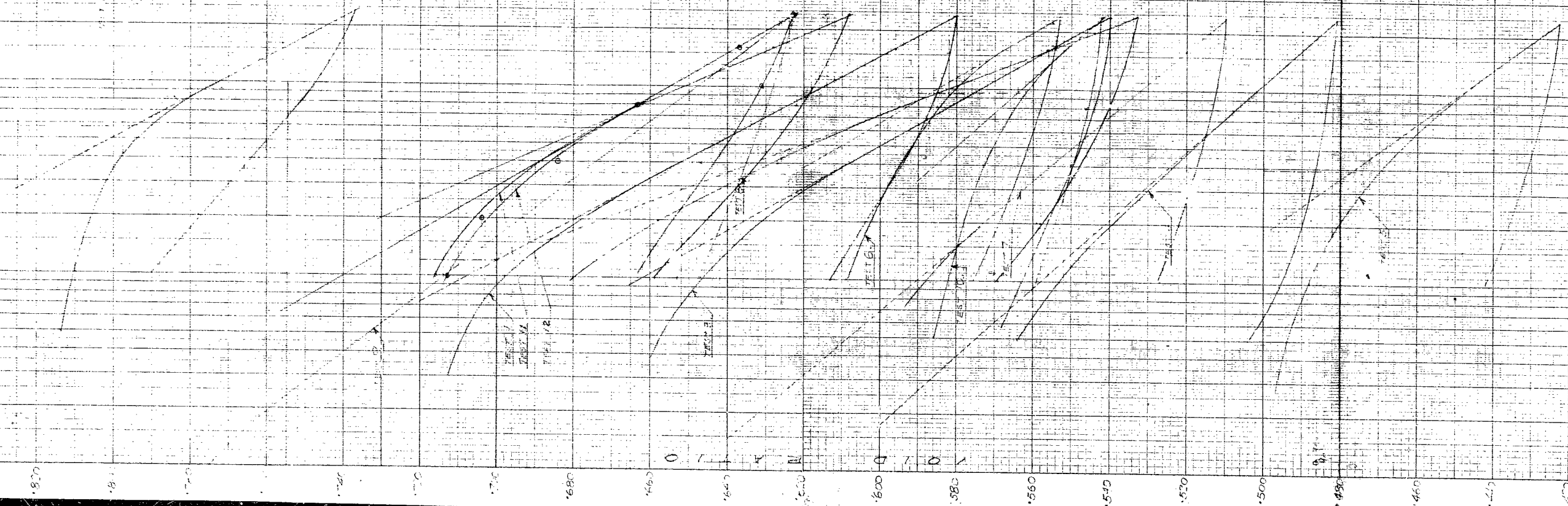
DATE \_\_\_\_\_

JOB NO. 59249

[illegible]

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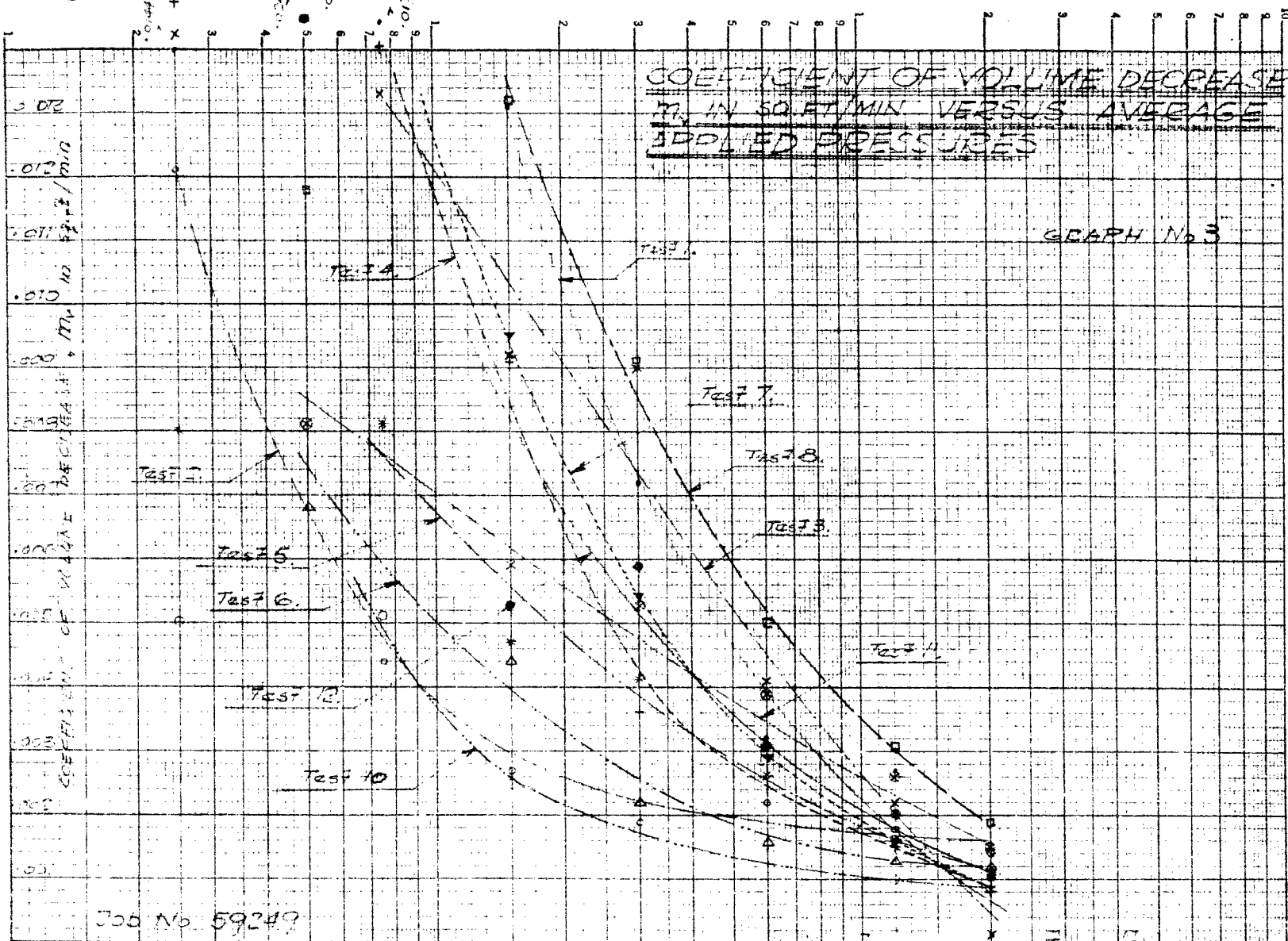
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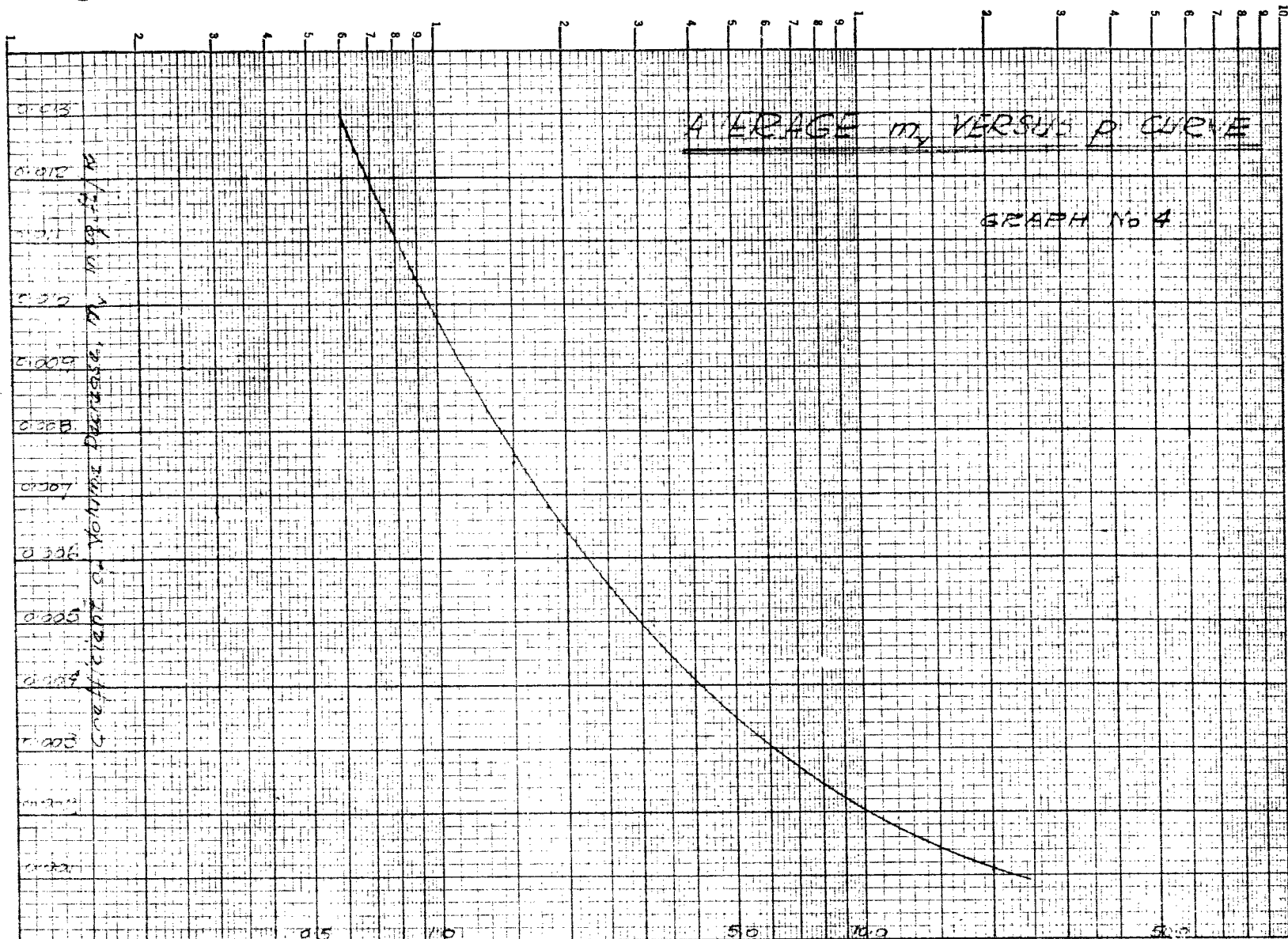


COEFFICIENT OF VOLUME DECREASE  
 $M_v$  IN SQ. FT./MIN. VERSUS AVERAGE  
APPLIED PRESSURES

GRAPH No. 3

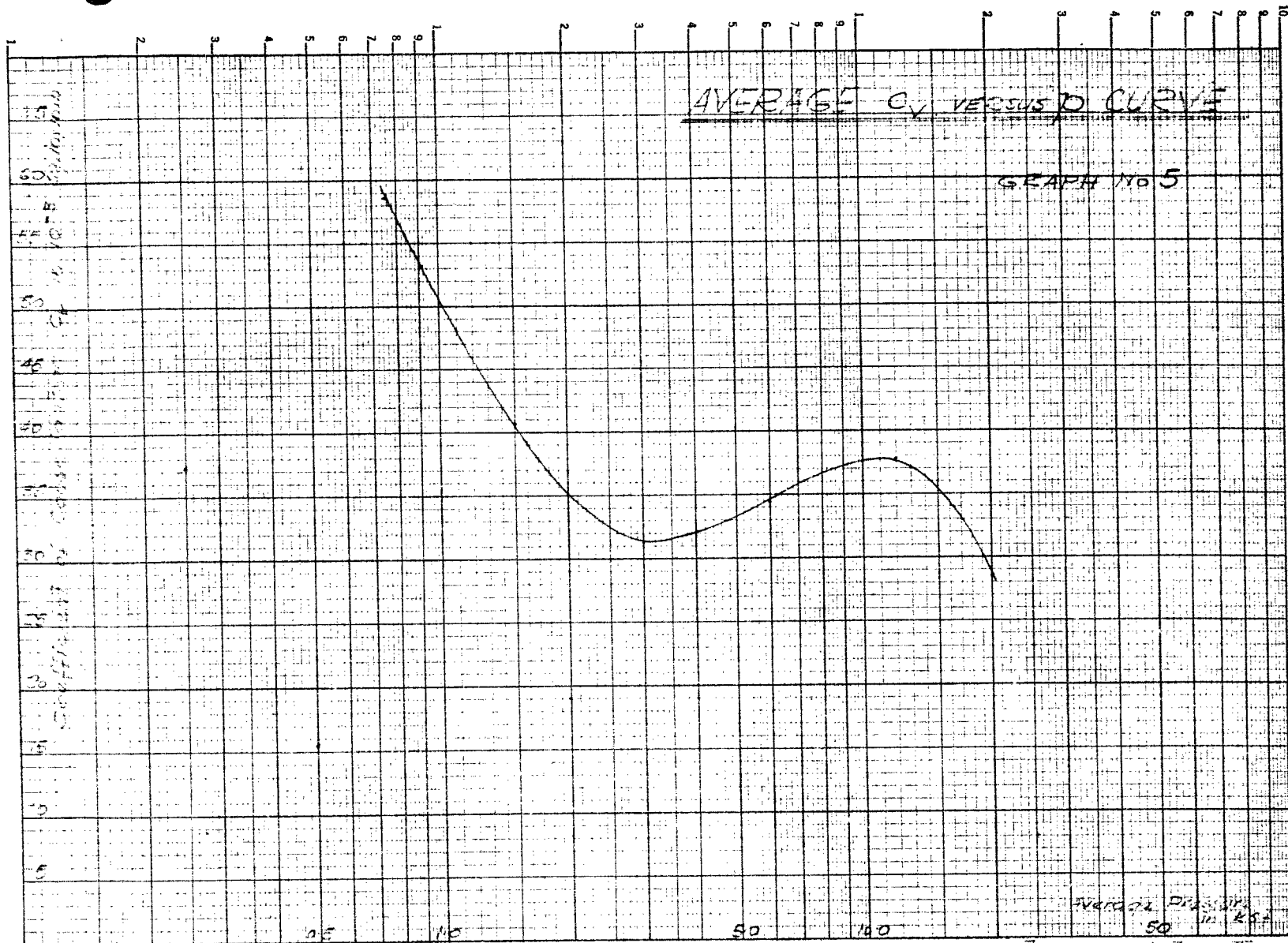


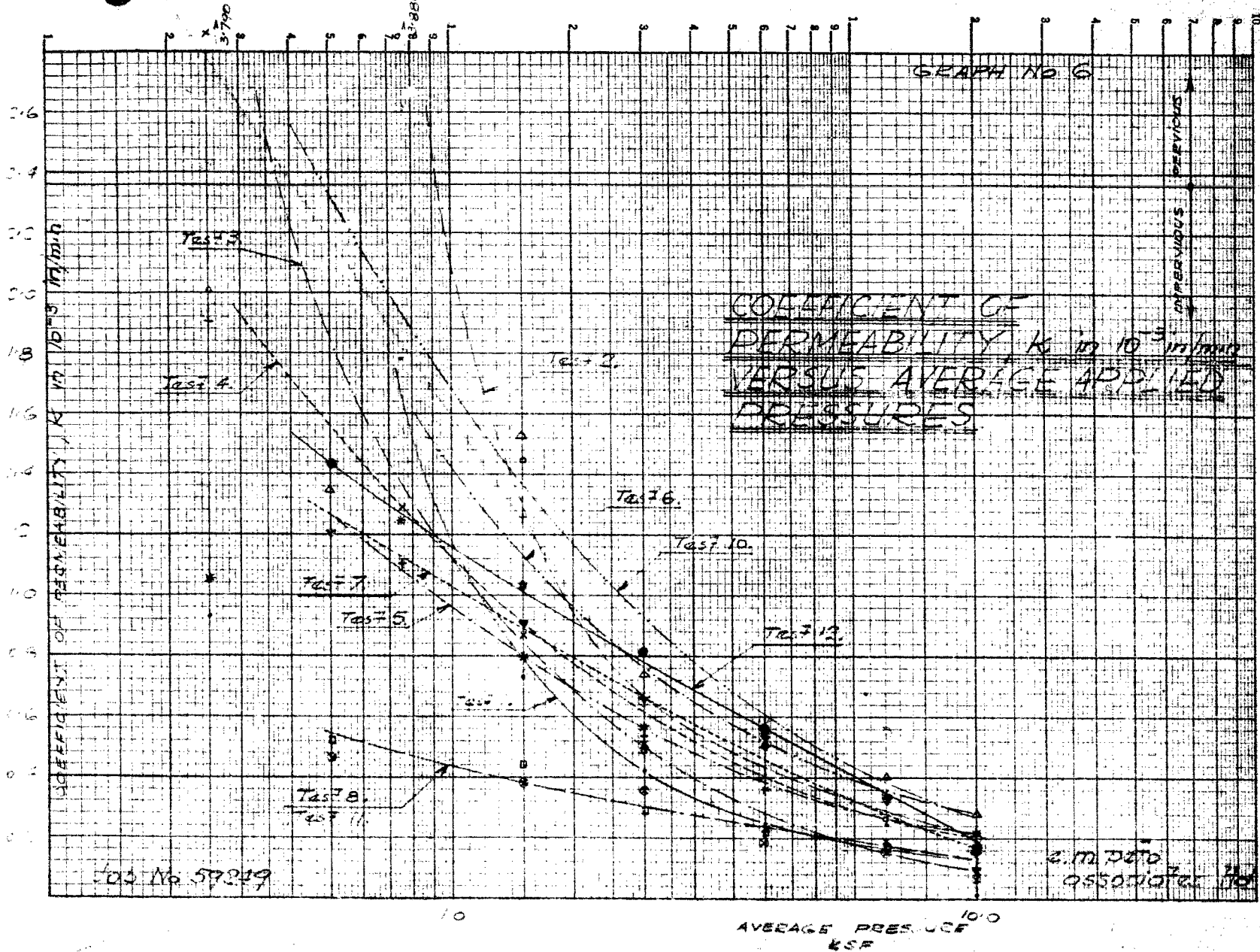
50 L.M. 50.00 ASSOCIATES



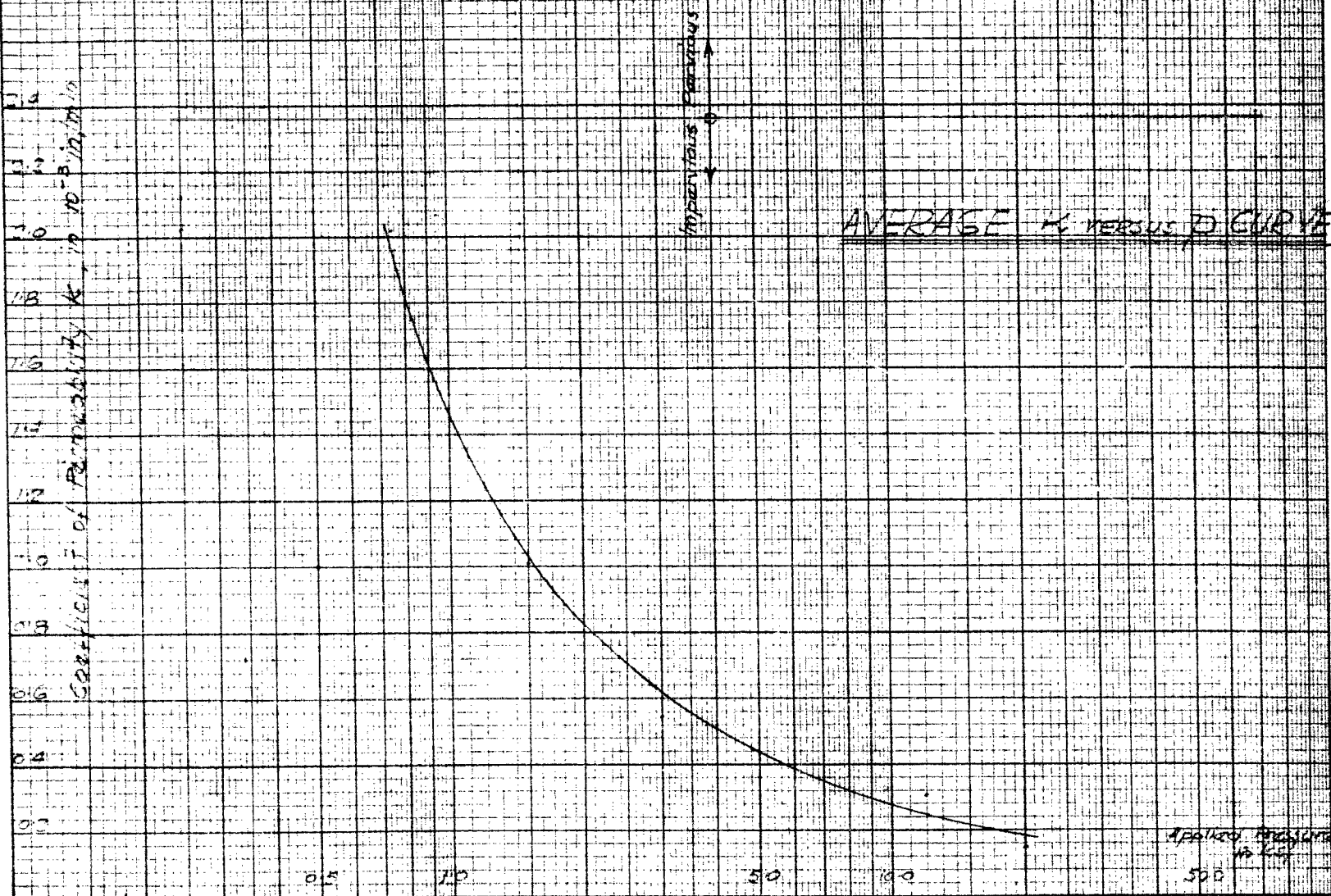
Job No 50249

Average Applied Pressure in ksf  
S.M. DEPTO associates TPA





GRAPH No. 7

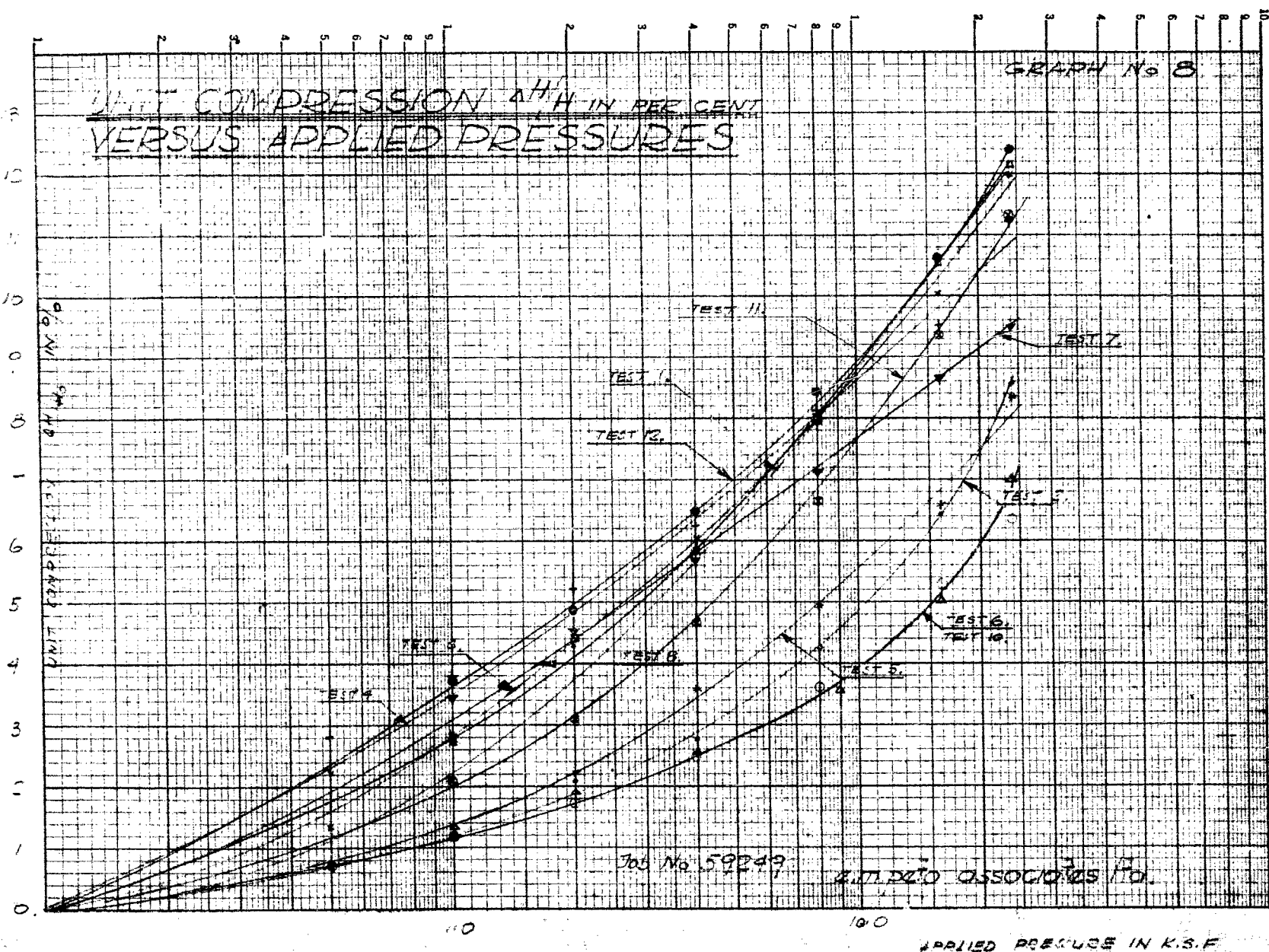


Job No. 59220

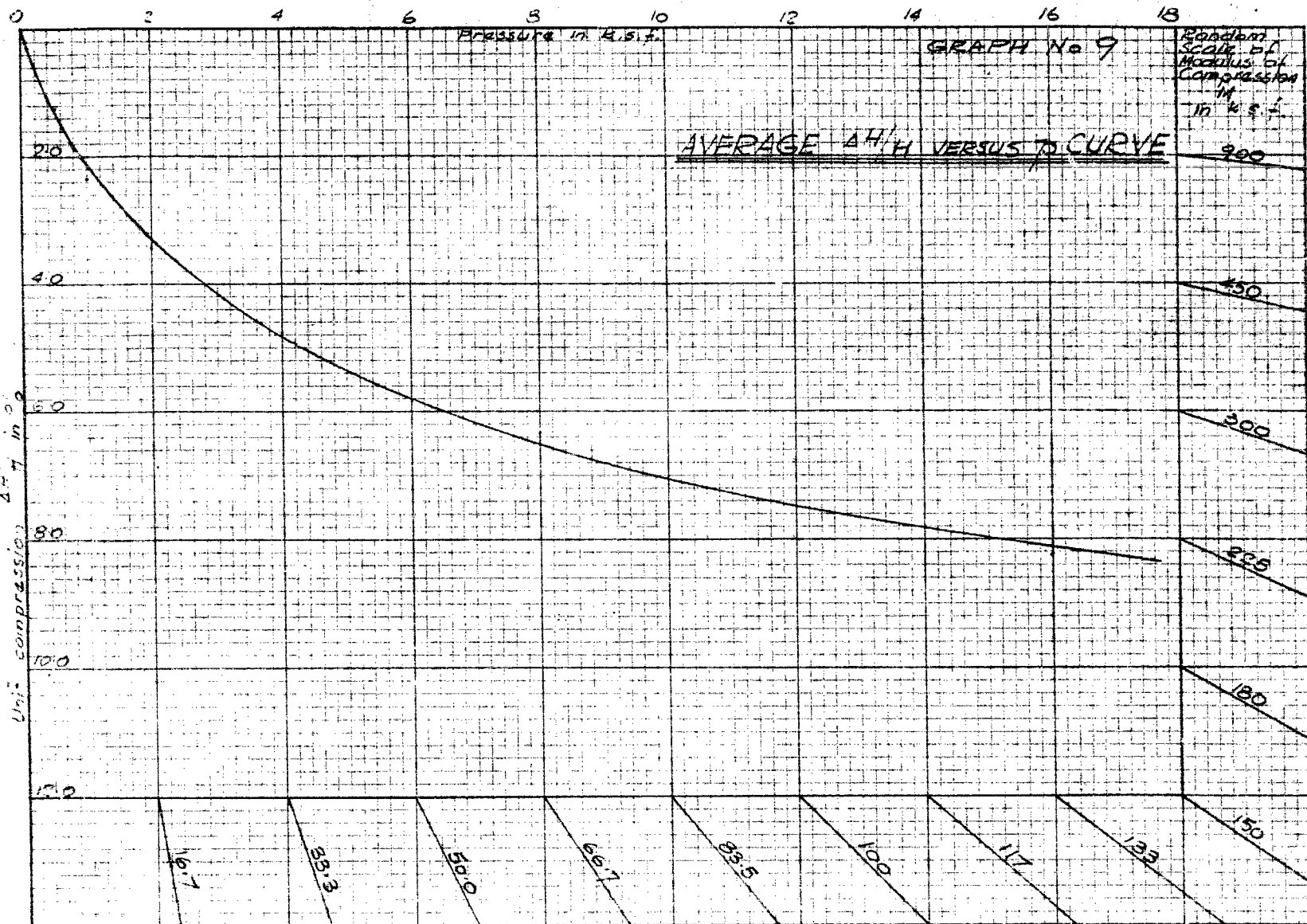
C.M. DEFO ASSOCIATES - Q

GRAPH No 8

UNIT COMPRESSION  $\Delta H/H$  IN PER CENT  
VERSUS APPLIED PRESSURES





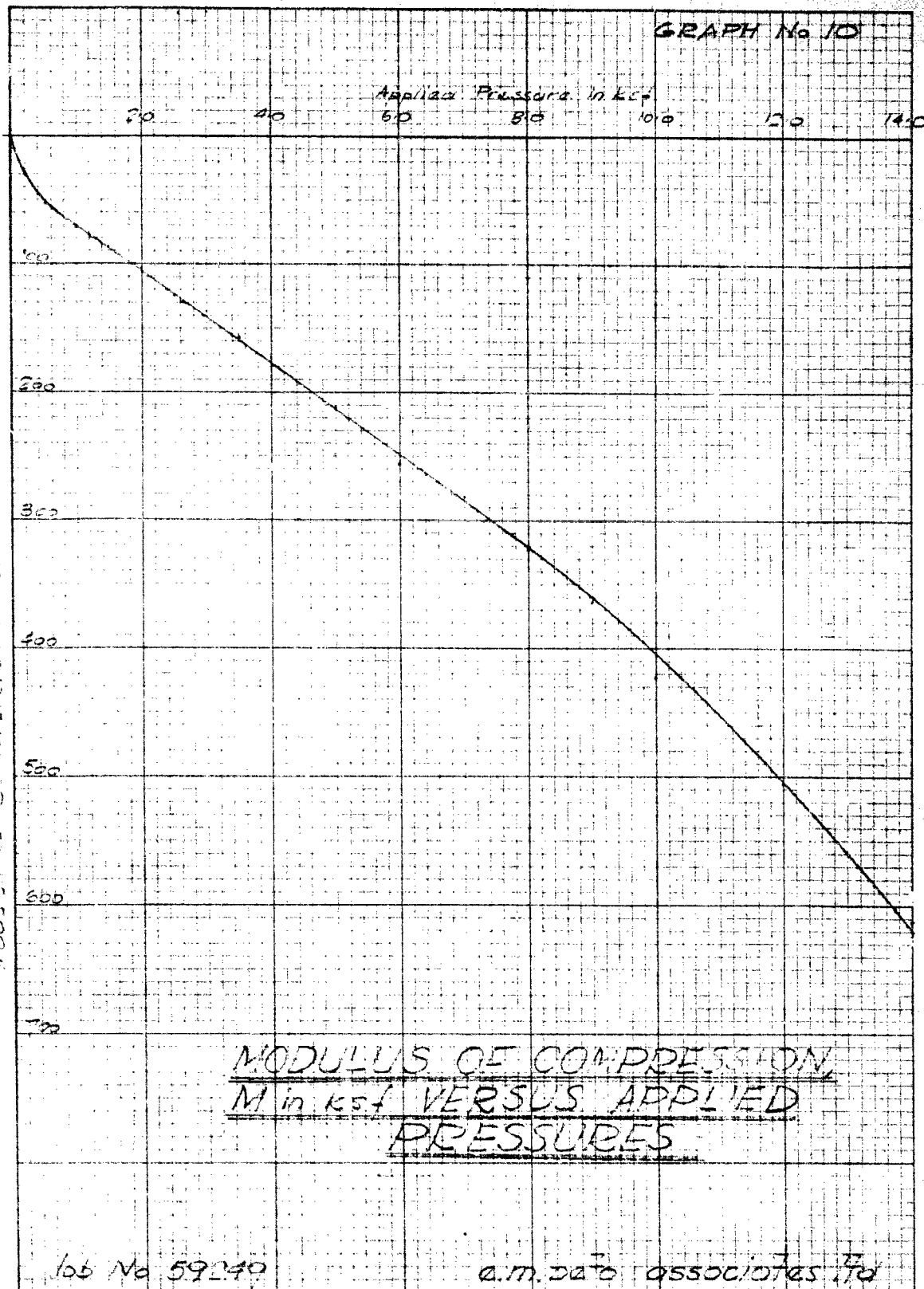


Job No 59249

e.m. 2270 associates, Inc.

GRAPH No 10

Modulus of Compression in Ksf





APPENDIX III

WATER LEVEL READINGS

# APPENDIX III

## Detailed Water Level Readings

Taken at Test Holes A and B

### Forehole A

Date	Time	Depth		Other Circumstances	Depth to Water
		Hole	Casing		
Feb. 3	9:30 a.m.	2'0"	None	(18" of ice)	1'0"
	9:35	2'0"	None		6"
	9:35	2'0"	None		3"
	10:20	12'0"	11'6"		None
	10:25	13'0"	11'6"		None
	10:55			Hole caved in at 9'6"	
	3:05 p.m.	24'8"	22'0"	Hole bailed out to 20'0"	20'0"
	3:10	24'8"	22'0"		18'0"
	3:15	24'8"	22'0"	Hole caved in at 22'3"	15'0"
	3:25	22'3"	22'0"		14'8"
	3:40	22'3"	22'0"		12'9"
	3:55	22'3"	22'0"		11'3"
	4:10	22'3"	22'0"		10'2"
	4:25			Hole caved in at 21'6"	
				Hole bailed out to 15'7"	15'7"
	4:30	21'6"	27'0"		15'7"
	4:35	21'6"	27'0"		15'7"
	5:30	28'0"	27'0"		Wet in bottom
Feb. 4	8:00 a.m.	28'0"	27'0"		21'9"
	10:20	34'4"	27'0"		28'0"
	10:28	34'4"	27'0"		26'0"
	10:45	34'4"	27'0"		24'4"
	11:00	34'4"	27'0"		22'9"
	11:15	34'4"	32'0"		19'0"
	11:30	34'4"	32'0"		19'0"
	12:30 p.m.	39'4"	32'0"	after pulling S.S. out	36'0"
	12:55	39'4"	32'0"		36'0"
	1:20	39'4"	32'0"		36'0"
	2:20	44'4"	32'0"	after pulling S.S. out	39'10"
	2:30	42'4"	32'0"		38'6"
	2:40	44'4"	32'0"	Hole caved in at 42'10"	37'11"
	2:55	42'10"	32'0"	Hole caved in at 42'7"	38'6"
	3:25	42'7"	32'0"		34'4"
	3:40	42'7"	32'0"		33'6"
	3:55	42'7"	32'0"		32'9"
	4:15	42'7"	32'0"		31'8"
	5:30	45'0"	44'1"	after bailing out	42'3"

Borehole A (Cont'd)

Date	Time	Depth		Other Circumstances	Depth to Water
		Hole	Casing		
Feb. 5	7:15 a.m.	42'4"	44'1"	Hole caved in at 42'4"	28'0"
	9:40	47'0"	44'1"		42'6"
	9:55	47'0"	44'1"		42'4"
	12:10 p.m.	52'0"	44'1"	After pulling the wash rods out	4'6"
	12:35	52'0"	44'1"		3'5"
	12:50	52'0"	44'1"		2'10"
	1:40	50'4"	44'1"	Hole filled up with water Water overflowing the top of casing very slowly Put additional 5 feet of casing above existing grade	
	1:50	50'4"	44'1"		
	2:10	50'4"	44'1"		
	2:20	50'4"	44'1"		+ 3' (*)
	2:30	50'4"	44'1"		+ 8"
	2:50	50'4"	44'1"		+ 13"
	3:25	50'4"	44'1"		+ 19"
	3:35				+ 22"
	4:45	64'4"	44'1"		
				Water level after pulling the s.s. out	14'6"
Feb. 6	7:50 a.m.	64'4"	44'1"		+ 1'7"
	12:20 p.m.	67'10"	65'0"	After withdrawing the s.s.	26'9"
	12:25	67'10"	65'0"		16'4"
	12:35	67'10"	65'0"		14'9"
	12:45	67'10"	65'0"		12'8"
	1:20	67'10"	65'0"	Water level in creek rose above the casing Put additional 5 feet of casing above existing grade Water level rising very slowly	0
	1:50	67'10"	65'0"		
	2:00	67'10"	65'0"		+ 5'0"

(\*) Note '-' sign designates F.L. above the existing grade.

Borehole A (Cont'd)

<u>Date</u>	<u>Time</u>	<u>Hole</u>	<u>Depth</u>	<u>Other Circumstances</u>	<u>Depth to Water</u>
			<u>Casing</u>		
Feb. 8	8:00 a.m.	87'10"	65'0"	Water overflowing the casing which is 5 ft. above existing grade.	+ 5'0"
	3:30 p.m.	72'4"	70'0"	Put additional 10 ft. of casing above existing grade.	+ 7'0"
Feb. 9	8:00 a.m.	72'4"	70'0"	Water overflowing the top of casing	+ 10'0"
	3:30 p.m.	72'4"	70'0"	After withdrawing the wash rods	3'0"
	3:31 p.m.	72'4"	70'0"		2'10"
	3:32	72'4"	70'0"		2'3"
	3:33	72'4"	70'0"		1'8"
	3:34	72'4"	70'0"		0'7"
	3:35	72'4"	70'0"		0'1"
	3:36	72'4"	70'0"		+ 0'4"
	3:37	72'4"	70'0"		+ 0'5"
				While driving the casing water kept overflowing. Water flow cut off when casing was driven past 77'0" depth	
	5:30	87'4"	86'0"	after driving the casing	0
					+ 0'4"
Feb. 10	8:00 a.m.	87'4"	86'0"		
	2:00 p.m.	97'4"	91'0"	After withdrawing the s.s.	7'0"
	2:01	97'4"	91'0"		6'7"
	2:02	97'4"	91'0"		4'8"
	2:03	97'4"	91'0"		4'1"
	2:04	97'4"	91'0"		3'7"
	2:05	97'4"	91'0"		2'10"
	2:25	97'4"	91'0"	Water overflowing at a fast rate (6 gal. per 12 sec.)	+ 6'0"
	3:00			While driving the casing water flow was cut off when casing driven past 92'8" depth.	
				When casing past 96'6" depth water started to overflow the top of the casing at 5'6" above existing grade at a rate of 6 gals. per 12 sec.	
Feb. 16	3:00 p.m.	109'6"	103'0"	Water overflowing slowly the top of the casing.	+ 5'0"

Borehole A (Cont'd)

<u>Date</u>	<u>Time</u>	<u>Depth</u> <u> Hole      Casing</u>	<u>Other Circumstances</u>	<u>Depth to</u> <u>Water</u>
Feb. 17	8:00 a. m.	116'0"    103'8"	Water overflowing the top of the casing at a rate of 2 gals. per 10 sec.	+ 5'0"
	8:30		Water flow cut off when casing driven past 105'0" depth.	
	3:00 p. m.	143'10"    115'0"	Water overflowing the casing which is 3'0" above existing grade	+ 3'0"
	5:17	143'10"    115'0"	Water level after withdrawing the s. s.	2'11"
	5:18	143'10"    115'0"		2'0"
	5:19	143'10"    115'0"		1'2"
	5:20	143'10"    115'0"		0
	5:21	143'10"    115'0"		+ 0'8"
	5:22	143'10"    115'0"		+ 3'2"
	5:22	143'10"    115'0"	Overflowing the top of the casing	+ 3'6"
Feb. 18	8:00	143'10"    115'0"	Overflowing the top of the casing at the rate of 2 gals. per 35 sec. Gas bubbles observed which burn with a red flame.	

Borehole B

Date	Time	Hole	Depth		Other Circumstances	Depth to Water
			Casing			
Feb. 26	2:15 p.m.	2'0"	None			1'6"
	2:30	9'0"	None			1'6"
	5:25	21'0"	20'0"			2'8"
Feb. 27	8:00 a.m.	21'0"	20'0"			1'6"
	12:10 p.m.	37'8"	31'3"			9'10"
	12:20	37'8"	31'3"			9'8"
	12:45	37'6"	31'3"			9'8"
	2:20	41'0"	37'0"	After bailing out		41'0"
	2:35	41'0"	37'0"			40'2"
	6:00	47'4"	47'6"			10'10"
Feb. 29	8:00 a.m.	47'4"	47'6"			11'3"
	12:10	57'4"	53'0"			12'6"
	12:35	57'4"	53'0"			11'10"
	5:30	72'4"	69'0"	After withdrawing the s.s.		22'2"
Mar. 1	8:00 a.m.	72'4"	69'0"			10'6"
	12:30 p.m.	81'4"	80'0"	after withdrawing the S. L.		14'0"
	12:50	81'4"	80'0"			13'2"
	4:45	92'4"	90'0"	After withdrawing the S. S.		15'2"
	4:55	92'4"	90'0"			10'9"
	5:10	92'4"	90'0"			5'6"
	5:50	92'4"	90'0"	Water overflowing the top of the casing		0'0"
Mar. 2	8:00 a.m.	93'2"	90'0"	Water overflowing the top of the casing which is 5'0" above existing grade. Overflowing Overflowing at the rate of 1 gal. per min.		+ 5'0"
		93'2"	85'0"			+ 5'0"
	10:30 a.m.	93'2"	78'0"			+ 2'0"

METHOD OF OPERATION

The field investigation work is carried out by means of a skid-mounted diamond drill rig.

Standard sampling procedures are followed. Casing is driven and cleaned, either by tubes or by wash water.

Samples are recovered ahead of the casing at frequent intervals, with either a 2 inch or 3 inch O.D. split barrel sampling tube, Shelby tube, or split barrel sampling tube fitted with brass liners and special sharp cutting nose.

The standard penetration test results are recorded when sampling with the regular 2 inch O.D. split barrel sampler, these being the number of blows of a 140 pound hammer falling 30 inches, required to drive the sampling tube a distance of one foot into undisturbed soil.

The Dutch cone probe test is made by driving the drill rods into the ground with a 2-1/4" - 90° cone tip. The number of 4200 inch pound blows per foot of penetration are recorded, as in the standard penetration test.

Where required, "in situ" shear strength tests are made ahead of the casing, using modified Acker vane test equipment.

Disturbed samples are visually classified in the field, sealed in sample jars, and are re-examined, and tested as necessary, in the soils laboratory. Undisturbed samples are returned to the laboratory for later examination and testing, as required.

The test holes are bailed at the end of the day and on completion. Subsequent water level readings are taken for the duration of the field work. Water pressure readings are recorded when Artesian water conditions are encountered. Moisture content samples are recovered at frequent intervals to assist in the soil classification and the interpretation of water table results.