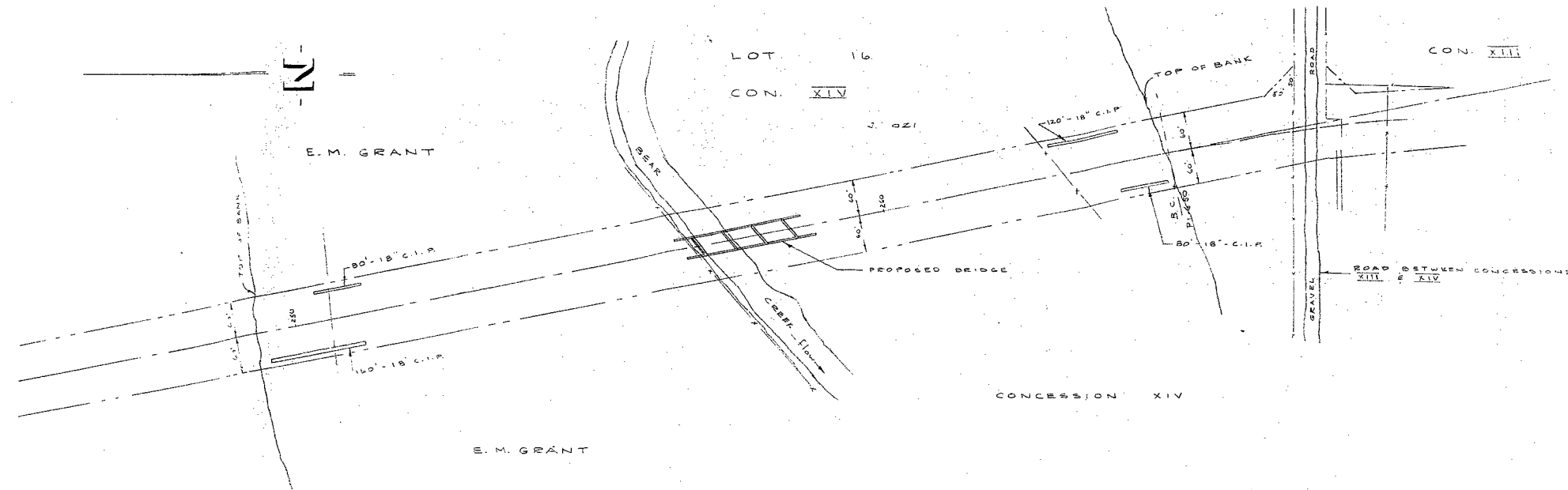
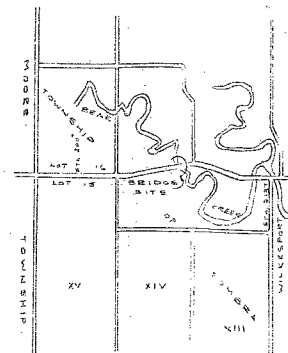


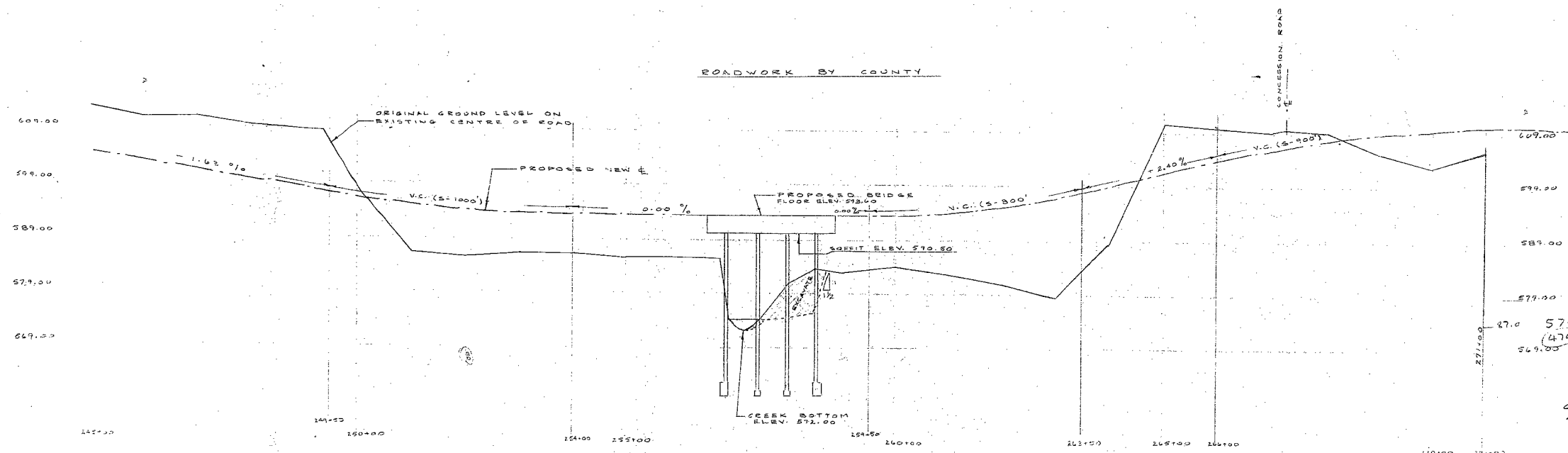
#61-F-248 M
BEAR CREEK
BRIDGE LOTS #15[#]16
CON XIV
LAMBTON
COUNTY



PLAN
SCALE: 1" = 100'



LOCATION PLAN
SCALE: 1/25" = 1 MI.



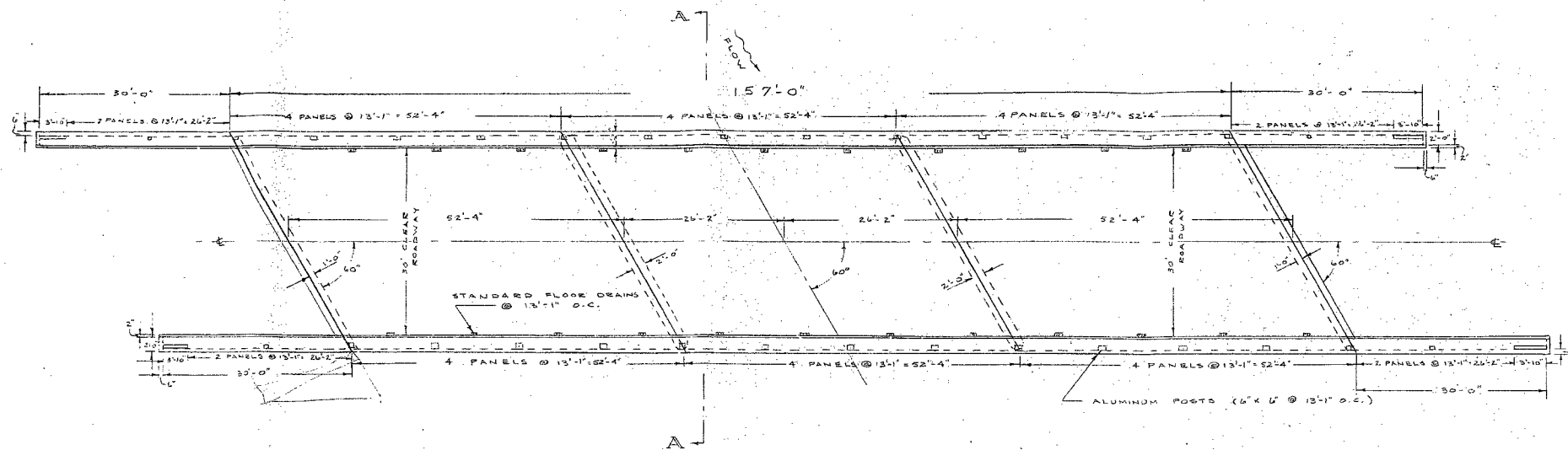
PROFILE
SCALE: HORIZ. 1" = 100'
VERT. 1" = 10'

PRELIMINARY PLAN
FORBHAM & CASE LIMITED
CONSULTING ENGINEERS
CHARTERED ENGINEERS
PLAN & PROFILE
BEAR CREEK BRIDGE (OVER
BEAR CREEK) TOWNSHIP OF
ROMBER, COUNTY OF LAMTON
JAN. 31, 1952
74-62109-1
LOT 16, CON. 14

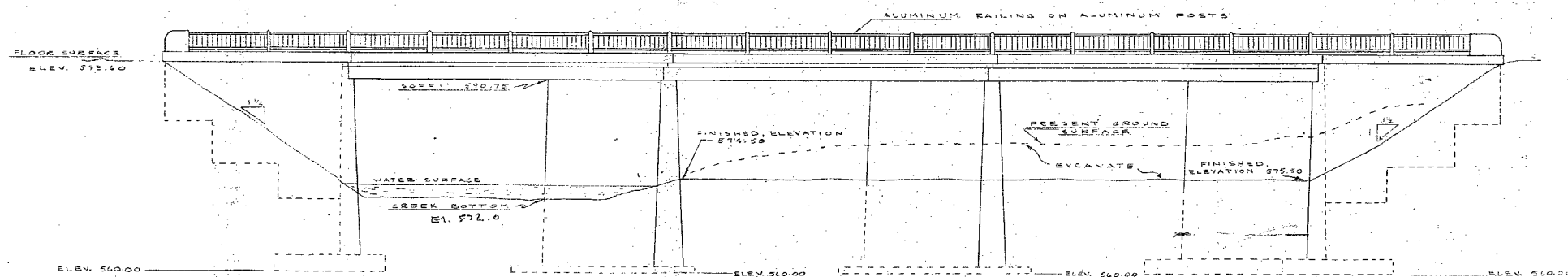


560

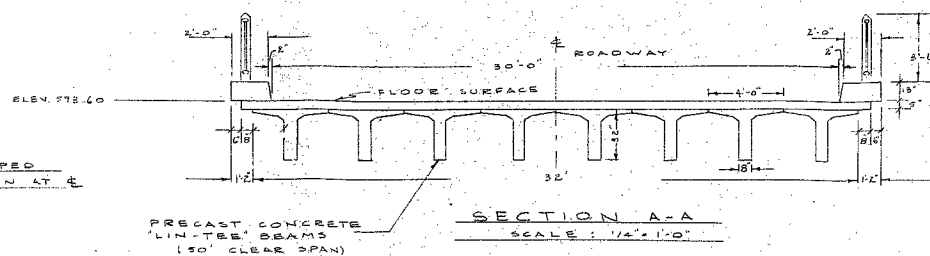
73



FLOOR PLAN
SCALE: 1" = 10'-0"



WEST ELEVATION
SCALE: 1" = 10'-0"



SECTION A-A
SCALE: 1/2" = 1'-0"

NOTE:
BEAMS TO BE STEPPED
TO PROVIDE 2" CROWN AT 4'

- DATA:**
- (1) SPECIAL FEATURES: WATERFALLS, DAMS, EXCEPTIONAL FLOODS, ICE, DRIFTWOOD, SLIDING BANKS, ETC. SOME ICE & DRIFTWOOD.
 - (2) (A) UPSTREAM: DOWNSTREAM BRIDGES (GIVE LOCATION, LENGTH, HEIGHT ABOVE N.H.W.L., NET CROSS-SECTIONAL AREA AT HIGH WATER & ESTIMATED AGE):
UPSTREAM: 157'-0" SPAN, ROAD AT LOT 17, 37' CLEAR SPAN, 30'-0" IS AT N.H.W.L. 1200-32-ET AT HIGH WATER 30 YEARS OLD.
DOWNSTREAM: SELMAN BRIDGE, 157'-0" SPAN, ROAD AT LOT 17, 37' CLEAR SPAN, 30'-0" IS AT N.H.W.L. 1200-32-ET AT HIGH WATER 6 YEARS OLD.
(B) REASONS WHY THESE BRIDGES ARE, OR ARE NOT, FAIR INDICATIONS OF SIZE OR PROPOSED BRIDGE: UPSTREAM BRIDGE, THE ROAD APPROACH IS LOW & MUCH WATER FLOWS OVER ROAD. SELMAN BRIDGE CARRIES A LITTLE MORE WATER BUT IS COMPARABLE.
 - (3) REASONS FOR CHANGES IN HEIGHT OR LENGTH FROM THAT OF OLD BRIDGE:
NO OLD BRIDGE AT THIS SITE, SAFETY & FLOOR LEVEL APPROX SAME ELEVATION AS SELMAN BRIDGE.
 - (4) IS DITCH, STREAM, OR RIVER GRADIENT LIABLE TO BE LOWERED? NO.
 - (5) NAVIGATION CLEARANCE REQUIRED, IF ANY: NONE.
 - (6) RAILWAY CLEARANCE REQUIRED, IF ANY: NONE.
 - (7) IF STRUCTURE IS OVER OR UNDER A RAILWAY HAS APPROVAL BEEN OBTAINED:
(A) FROM RAILWAY CO.: NO.
(B) FROM BOARD OF TRANSPORT COMMISSIONERS: NO.
 - (8) HAS APPROVAL BEEN OBTAINED UNDER NAVIGABLE WATERS PROTECTION ACT? NO.
 - (9) IS A TEMPORARY DETOUR REQUIRED? NO.
WHO WILL BUILD IT? NO.
WHO WILL MAINTAIN IT? NO.
 - (10) INFORMATION & EVIDENCE OF EXTREME FLOODING WAS OBTAINED FROM OBSERVATIONS & LOCAL RESIDENTS AND REFLECTS HIGHEST WATER ELEVATION IN THE AREA OF THIS CONSTRUCTION TO BE 585' AND THE LOWEST WATER ELEVATION TO BE 573'.
 - (11) ROAD DESIGN INFORMATION:
ESTIMATED A.D.T.: 51/82
DESIGN SPEED: 30 M.P.H.
STOPPING SIGHT DISTANCE: 1400 FEET

STRUCTURE DATA

- (1) NET SPAN LENGTH & TYPE OF BRIDGE: 127' ON C/L - 127' AT RIGHT ANGLES TO DIRECTION OF FLOW, NET AREA 1230-32-ET, 3 SIMPLE SPANS, LIN-TIE, POURED IN PLACE SLAB & SOLID FIELDS, PIERS & ABUTMENTS ON SPREAD FOOTINGS.
- (2) ROADWAY WIDTH ON BRIDGE: 30 FT.
- (3) NUMBER & WIDTH OF SIDEWALKS: NONE
- (4) SKEW ANGLE: CENTER LINE OF OPENING IS SKEWED 30° FROM RIGHT ANGLES.
- (5) TOTAL LENGTH & TYPE OF PILING: NONE
- (6) APPROX. VOLUME OF CONCRETE: NOT CALCULATED CU. YDS.
- (7) APPROX. WEIGHT OF STR. STEEL: NOT CALCULATED TONS.
- (8) APPROX. WEIGHT OF REINFORCEMENT: NOT CALCULATED TONS.
- (9) APPROX. VOLUME OF APPROACH DIRT 100' EACH SIDE OF STRUCTURE: NOT CALCULATED CU. YDS.
- (10) DRAINAGE AREA: 332 HYDROLOGY DEPARTMENT 50 MILES.

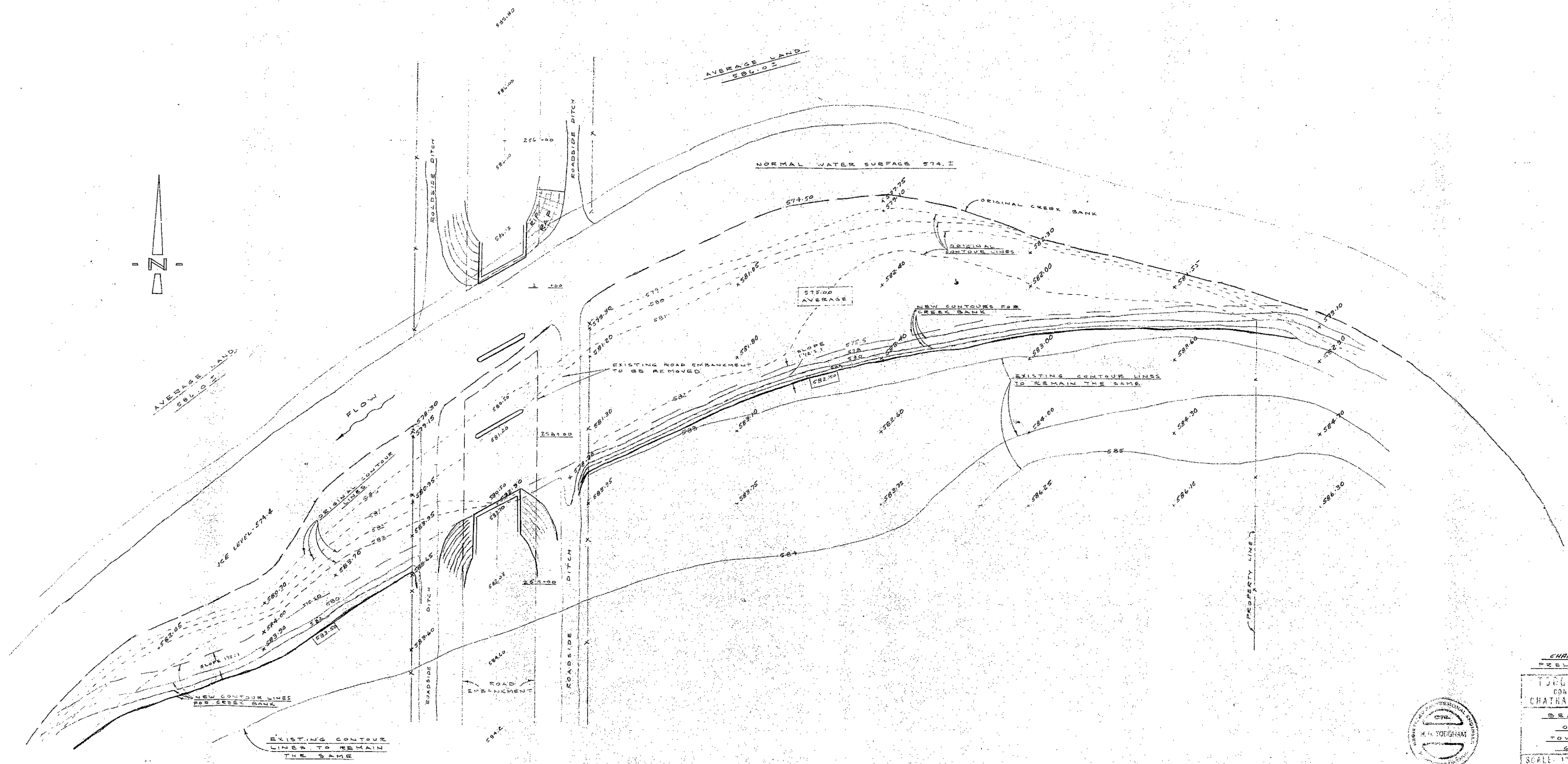
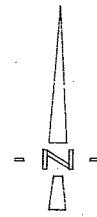
FIELD INVESTIGATION MADE
OCTOBER & NOVEMBER 1961

BY: H. H. Tiedeman
SURVEY ENGINEER



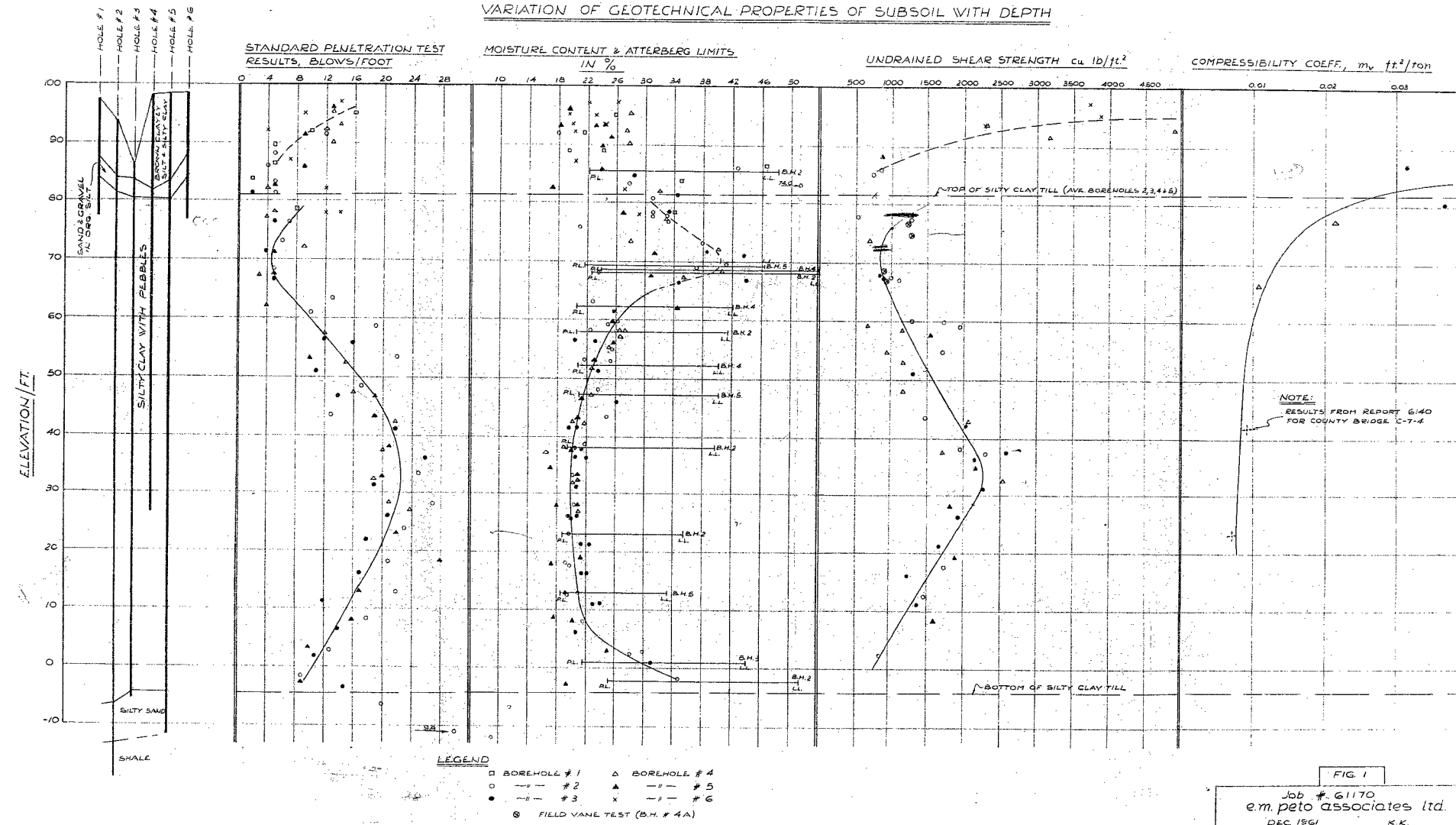
FORBES & CASE LIMITED
CIVIL ENGINEERS
CATHAM
685-710
BEAR CREEK BRIDGE
OVER BEAR CREEK
TOWNSHIP OF SOMERS
COUNTY OF LAMARCA
AS SHOWN
JAN. 21, 1962
BY: H. H. Tiedeman 62109-2

PRELIMINARY PLAN



CHANNEL IMPROVEMENT PRELIMINARY PLAN	
J. J. O'CONNELL & CASE LIMITED CONSULTING CIVIL ENGINEERS CHATHAM — ONTARIO	
BEAR CREEK BRIDGE OVER BEAR CREEK TOWNSHIP OF SOMERSET COUNTY OF LANARK	
SCALE 1" = 50'	DATE JAN. 31, 1960
CHECKED BY 277	62109-3

VARIATION OF GEOTECHNICAL PROPERTIES OF SUBSOIL WITH DEPTH



Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Division,
(Foundation Section)

March 22, 1962.

REVIEW OF FOUNDATION REPORT BY
E. M. PETG ASSOC., LTD., and
PRELIMINARY PLANS -
Bridge Office Ref. BA 1349.

Attention: Mr. K. L. Kleinsteinber,
Municipal Bridge Liaison Engr.

Re: Proposed New Bridge at Bear Creek,
Lots 15/16, Con. XIV, Sombra Twp.,
Lambton Co., District # 1

We have reviewed the Foundation Report and the Preliminary Plans for the above structure. Our comments are as follows:-

The Foundation Report contains sufficient information to enable the Consultant to design spread footings. If this type of footing is selected, the formation level may be raised to el. 564.0. The design loads should be in accordance with the recommendations made by the Soils Consultant. The dimensions of the footings and the proposed design loads are not shown on the Preliminary Plans; therefore, we are unable to determine whether or not the Bridge Consultant has followed these recommendations.

In view of the relatively low bearing capacity of the subsoil, we are of the opinion that a piled foundation should be considered in this case. 12 BP at 74 'H' piles driven to bedrock, could provide a design load of 70 tons/pile. In this way, a rigid foundation could be designed to support a continuous structure. A structure such as this could probably be more economical than the simply supported structure at present proposed by the Bridge Consultant. For a piled foundation, the formation level for the footings should be at el. 568.0, the minimum depth necessary for scour protection.

If you have any further queries in connection with this matter, please contact this Office.

KGS/MdeF
cc: Foundations Office
Gen. Files.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.
Per:

K. G. Selby
K. G. Selby,
SR. PROJECT FOUNDATION ENGR.

OFFICE LOCATION -
DOWNSVIEW AVE.,
KEELE ST. - HIGHWAY 401
TORONTO, ONTARIO.



ONTARIO
DEPARTMENT OF HIGHWAYS

POSTAL ADDRESS -
DEPARTMENT OF HIGHWAYS
PARLIAMENT BUILDINGS,
TORONTO 5, ONTARIO.

Bridge Hydrology Section,
March 8, 1962.

MEMORANDUM TO:

Mr. K. L. Kleinsteinber,
Municipal Bridge Liaison Eng.,
Department of Highways,
Administration Building,
DOWNSVIEW, Ontario.

RE: Lambton County Bridge
Sombra Twp., Lot 16, Con. 14
Bear Creek N.E. of Wilkesport
Mun. Dist. #1 - SW621
W.O. 360-cl-139

For the watershed area of 216 square miles, the proposed right-angled opening of 130' is adequate.

With regard to the location of the bridge relative to the channel, the principle of opening out the channel on the south bank is sound, since the bulk of the flood overflow crosses the south flood plain. However, in view of the erosion of the north river bank at and upstream from the proposed bridge it is recommended that the north abutment be moved slightly further back, to about 10' north of the position shown on drawing 62109-3.

Scour protection

If the bridge is to be on piles, the bottom of footings could be 4' below stream bed.

If spread footings are to be used, these should be preferably 8' below the bed, at elevation 504. This depth could be reduced to 6' or 7' if there were a reasonable certainty that ice jamming will not occur at the bridge. However, in view of the nature of Bear Creek, some jamming does appear possible.

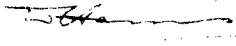
....2

- 2 -

Lambton County Bridge
Bear Creek N. E. of Wilkesport
W.O. 366-81-139

To reduce the possibility of jamming at the bridge, cutwaters with steel angle nosings should be provided at the upstream ends of the piers.

JDH/ea


J. D. Harris
for L. Wilkie,
Bridge Hydrology Engineer.

OVER



ONTARIO
DEPARTMENT OF HIGHWAYS

Bridge Division.

Memo to Mr. A. Stermac, Principal
Foundation Engineer,
Materials & Research Section

Date February 9, 1962.

From G. C. E. Burkhardt

Subject County of Lambton, Sombra Twp.
Bear Creek Bridge
Lot 15/16, Conc. A14

Report File # BA1349

Attached please find a copy of the foundation report, by E. M. Peto Associates Limited, and a copy of the preliminary plans for your comments.

The designer of structure, Mr. H. H. Tougham, P. Eng., has shown the pier and abutment foundations at elevation 500. He is not completely satisfied that this is necessarily the proper elevation for the footings. Mr. H. H. Tougham would be pleased to have your comments and suggestions in this particular matter.

100-500-100-100

GCEB/ea

G. C. E. Burkhardt
G. C. E. Burkhardt,
for A. L. Kleinsteiber,
Municipal Bridge Liaison Eng.

To be reviewed later
when further info is received
from Mr. Kleinsteiber.

15-2-62

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

15-2-62

OVER



Stream Bed 572.0

Bottom of footings 560.0

Bottom of footing need not be below 566.0

5338

Piles el. 568.0

REVO

CONDITION OF 40110403

BA 1349

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,
Toronto 19, Ontario.

Our Job Number 61170

Russell 9 - 1126.

December 18th, 1961.

Messrs. Todgham and Case,
Consulting Engineers,
151 Thames Street,
P. O. Box 336,
Chatham, Ontario.

Dear Sirs,

Re: Site Investigation for Bear Creek
Bridge, County of Lambton
(County Bridge C-7-6)

We have pleasure in submitting three (3) copies of our Report No. 61170 on the above site investigation. One (1) additional copy has been forwarded directly to Mr. O. VanDeurs, County Engineer.

As no data concerning the design loads and foundation dimensions of the proposed bridge was available, we have carried out bearing capacity and settlement calculations for foundations placed at several elevations, making certain assumptions as to the foundation dimensions. Should the final dimensions materially differ from the assumed figures, we will gladly re-calculate the safe bearing capacity and the theoretical settlements.

We have treated independently the possibility of founding the bridge on spread footings, friction piles and end-bearing piles resting on dense sand or bedrock. Due to the considerable depth of excavation which would be necessary for the construction of spread footings, piled foundations may prove more economical.

The problems connected with the construction of embankments are briefly discussed in the Report.

The elevations, referred to in the report, are based on an arbitrary benchmark of assumed elevation 100.0; the position of this reference point has been described in the report, in order that the given elevations may be tied in to the Geodetic Datum. *ADD 486.86 TO CONVERT TO CANADIAN GEODETIC DATUM.*

Yours very truly,

E. M. PETO ASSOCIATES LTD.



E. M. Peto, P. Eng.

RK/ap

THE COUNTY OF LAMBTON

C/O TODGHAM AND CASE,
CONSULTING ENGINEERS

SOILS REPORT

BEAR CREEK BRIDGE (C-7-6)
NEAR WILKESPORT, ONTARIO.

E. M. PETO ASSOCIATES LTD.

1287 Caledonia Road,
Toronto 19, Ontario.

TABLE OF CONTENTS

	<u>Page Number</u>
A. INTRODUCTION	1
B. GENERAL INFORMATION	2
1. Positions & Depth of Testholes	2
2. Drilling Operations	4
3. Soil Testing	5
C. SITE and GEOLOGY	7
D. SOIL CONDITIONS	8
E. WATER CONDITIONS	24
F. ENGINEERING CONSIDERATIONS and CONCLUSIONS	25
1. Summary of Subsoil Conditions	28
2. Bridge Foundation	28
(a) Spread Footings	29
(b) Friction Piles	44
(c) End-bearing Piles	52
3. Embankments	52

Appendix "A" Standard Procedure

Appendix "B" Soil Test Results

Borehole Logs

Site Plan and Profile

A. INTRODUCTION

The work described in this report was authorized verbally by Mr. H. H. Todgham, Consulting Engineer, on behalf of the County of Lambton.

In connection with realignment of road No. 7 in the Township of Sombra, County of Lambton, a new bridge will be constructed over the Bear Creek. The Client's reference number of the bridge is C-7-6 .

It is expected that the bridge will have a total length of 140 ft., consisting of three equal spans of approximately 47 ft. These will be simply supported and constructed of pre-stressed concrete sections. Abutments and piers will be solid reinforced concrete and the abutments will be closed.

At the proposed crossing the Bear Creek was approximately 55 ft. wide, but is known to be subject to considerable flooding.

B. GENERAL INFORMATION

1. Positions and Depth of Test Holes

Six test holes were drilled at the site, along the centre line of the proposed realigned road. The positions of the holes, in relation to the road chainage, were as follows:-

<u>Borehole No.</u>	<u>Chainage</u>	<u>Elevation</u>	<u>Depth</u>
1	259 + 80	97.3	20'-0"
2	257 + 80	93.9	112'-0"
3	257 + 40	93.9	92'-0"
4	256 + 75	93.6	71'-6"
5	256 + 35	98.8	110'-2"
6	254 + 35	98.9	21'-6"

Boreholes 2 and 5 were nearer the positions of the abutments of the bridge, while boreholes 3 and 4 were at the approximate locations of the piers. Boreholes 1 and 6 were drilled to investigate the subsoil conditions under the approach embankments, which will be up to 20 ft. high.

Originally, the four borings under the bridge structure and the boring under the high embankment were specified to be put down to refusal on shale bedrock, which was expected to commence approximately 90 ft. below ground level.

B. GENERAL INFORMATION - Cont'd

However, test holes 2, 3 and 5 have indicated a practically uniform subsoil conditions, so that it was not considered necessary to sink test hole 4 to a depth greater than 71 ft. 6 in., at which it was terminated. Similarly, test holes 1 and 6, under the proposed embankments, were interrupted at a depth of 20 ft. and 21 ft. 6 in. respectively, after the longitudinal continuity of the soil stratification was proved.

The subsoil profile was similar to that determined at the Wilkesport Bridge, (County Bridge C-7-4), the site investigation for which was described in our Report No. 6140 of June, 1961.

The positions of the present test holes are shown on the enclosed site plan and profile. At the time of preparing the report, we have not received the road profile, so that the profile plotted on the enclosed drawing is based on the elevations at the positions of the test holes determined by our field engineer.

The elevations are referred to an arbitrary datum, of assumed elevation 100.00 ft. This datum was taken as the top of the iron bar at the eastern fence bordering the road allowance, on the northern side of the creek. As a check, a reference point, in the form of a nail in eastern fence anchor post, south of the

B. GENERAL INFORMATION - Cont'd

creek, was measured; its elevation was established as 95.42. The elevation of surface of Bear Creek at the time of the site investigation was 86.8.

2. Drilling Operations

The field work was carried out by our drilling units Nos. 4 and 6, between October 26th and November 10th, 1961.

Test holes 1, 2, 3 and 4 were performed by unit No. 4, while test hole number 5 was drilled by unit No. 6. Test hole 5 was sunk to the depth of 106'-6" by unit number 6, between October 28th and November 2nd, 1961, and was completed to the depth of 110'-2" on November 7th, by crew No. 4.

In addition, in-situ vane tests were carried out by crew No. 4, in test hole 4a, located 10 ft. north of test hole 4; the vane tests were performed in the soft to firm silty clay, between the depths of 21 ft. and 32 ft. below ground level.

B. GENERAL INFORMATION - Cont'd

Our standard drilling and sampling procedures were followed, as outlined in the enclosed Appendix "A". BX and 4 inch diameter casing was used as required.

Details of the soil conditions found in the test holes are shown on the enclosed borehole logs. A simplified soil profile, deduced from the logs, is included on the drawing.

3. Soil Testing

a) Field Tests

Standard penetration tests were performed at regular intervals in all test holes. The results are entered on the borehole logs, which also contain the in-situ moisture contents, determined on standard split spoon samples of soil.

The penetration resistance and moisture content distribution is plotted against elevation in Fig. 1. In addition, the penetration resistance is shown plotted against the undrained shear strength in Fig. 2.

Vane tests were performed in test hole 4a, between the depth of 21 and 32 ft., after laboratory examination of samples from the initial test holes disclosed that within this depth the silty clay is sufficiently free of pebbles. Lower down, the clay generally possessed a considerable pebble con-

B. GENERAL INFORMATION - Cont'd

tent, frequently exceeding 1 inch in diameter, so that vane tests would probably overestimate the true shear strength.

b) Laboratory Tests

The following tests were performed in the soil mechanics laboratory.

Atterberg Limits,

Particle size distribution,

Unconfined compression tests,

Undrained triaxial compression tests,

Consolidation tests.

The relatively large number of soil identification tests was required in order to determine to what extent the silty clay stratum can be considered as uniform and what reliance can be placed on the established shear strength profile when calculating the necessary length of piles.

The results of the above tests are included in Appendix "B", while the variation of the main geotechnical properties with depth is plotted on Fig. 1.

C. SITE and GEOLOGY

The site of the proposed bridge C-7-6 is situated about 1-1/2 miles north-east of Wilkesport, in the Township of Sombra, County of Lambton. It is approximately a mile north of the bridge C-7-4 over the Sydenham River, for which we have carried out a site investigation described in our Report Number 6140 of June, 1961.

The new bridge is to carry road No. 7 over the Bear Creek, which at the time of the site investigation was approximately 55 ft. wide at the proposed crossing. A flood plain extended about 700 ft. to the north and south of the creek. The flood plain is about 30 ft. lower than the surrounding, generally very flat, topography. According to local reports, the plain is flooded every spring.

Geologically, the area is located within the St. Clair Clay Plain where glacial processes have deposited a mantle of clayey till over a shale bedrock. The test holes have disclosed that the clayey till stratum is approximately 85 ft. thick.

D. SOIL CONDITIONS

Details of the soil conditions found in the test holes are given on the appended borehole logs. A simplified subsoil profile, as determined from the logs, was plotted on the enclosed drawing in the form of a longitudinal section through the line of test holes.

The borings have disclosed that the subsoil profile can be considered^a as uniform over the section investigated.

The main soil types, in the order of occurrence, were as follows:-

- a) Brown silty clay or clayey silt,
- b) Sand and gravel with organic silt,
- c) Silty clay, changing into silty clay with pebbles (Clay till),
- d) Silty sand and silt with broken shale and natural gas,
- e) Solid black shale bedrock.

The above-listed soil types occurred at the locations of the bridge structure and embankments as follows (see also the Soil Profile on the enclosed drawing).

D. SOIL CONDITIONS - Cont'd

<u>Soil Type</u>	<u>Elevation</u>		<u>Thickness</u>
	<u>Top</u>	<u>Bottom</u>	
(a)	98.9 to 86.9 (ground level)	88.9 to 82.4	2'6" to 14'8"
(b)	88.9 to 82.4	84.5 to 80.8	1'4" to 4'0"
(c)	84.5 to 80.8	-3.6 to -4.2	84'10" to 87'6"
(d)	-2.6 to -4.2	-11.5 to -13.1	7' approx.
(e)	-13.1 (proved by coring in BH 2) -11.5 (indicated by refusal in BH 5)		

It is most likely that the new bridge will be founded either on footings placed in the silty clay, (soil type c), or on piles. Frictional piles would be located in the silty clay stratum, while end-bearing piles would rest on the dense silty sand (stratum d) or on the shale bedrock.

The embankments will probably be built on top of the existing strata, so that each of the layers a, b & c will be stressed by the embankment loads.

The main geotechnical properties of the various soil types listed above will now be described in turn.

D. SOIL CONDITIONS - Cont'd

a) Brown silty clay or clayey silt.

This soil type was found immediately under a thin layer of topsoil, and extended to the greatest depth of 14 ft. 8 in. in borehole 5. Its minimum thickness was observed in borehole 3, located just north of the creek, where it extended only to a depth of 2 ft. 6 in.

This stratum was generally variable in composition, consisting in parts of sandy and silty clay and in parts of silt or clayey silt. Organic matter was present in the form of enclosures of decayed vegetation, and the organic content increased with depth. The stratum was brown throughout, having been desiccated. The upper parts are of firm consistency, while the material becomes softer with depth. In the upper portions, standard penetration test results (N-values) ranged between 12 and 16 blows per foot, while lower down they dropped to 5 blows per foot.

The undrained shear strength, measured as one half of the unconfined compressive strength, was between 2800 and 5000 lb./sq.ft. above the elevation 83, dropping to less than 1000 lb./sq.ft. at a greater depth.

D. SOIL CONDITIONS - Cont'd

On account of the highly variable composition, the moisture content results were very scattered, ranging at random from 18.0% to 28.0%. The higher moisture contents were associated with more clayey portions of the stratum, while the lower ones were in seams of silt or sandy silt.

The standard penetration resistance, shear strength and moisture contents have been plotted against elevation in Fig. 1. An Atterberg Limit result is included on the above graph, indicating that the material is near its plastic limit.

The results of this test were as follows:-

B. H. & SA. No.:	2/7
Depth:	7'-8'6"
Liquid Limit:	49.7%
Plastic Limit:	22.1%
Plasticity Index:	26.6%
Natural Moisture Content:	21.0%

D. SOIL CONDITIONS - Cont'd

b) Sand and gravel with organic silt

A layer of fine to coarse sand, with or without gravel, and mixed with a highly organic silt was encountered in all test holes, between the elevations 89.9 and 80.8; its thickness ranged from 1 ft. 4 in. in borehole 4 to approximately 4 ft. in borehole 6.

A very similar, organic, sandy layer was encountered on the north side of Sydenham River approximately one mile to the south, and has been described in our Report No. 3140 of June, 1961, on the site investigation for the Sydenham River bridge C-7-4.

The organic, sandy layer was generally very soft, and was waterlogged. It can be assumed to be in hydraulic continuity with the water in Bear Creek, the surface of which was above this layer.

Standard penetration test results performed in the organic sand gave N-values ranging from 2 to 5, confirming the loose character. The moisture contents varied from 17.2% to 46.7%, the higher values being associated with the higher organic content.

D. SOIL CONDITIONS - Cont'd

On account of its loose character and the high proportion of decayed vegetable matter, this layer will be subject to long-term settlements under any applied loads. It will also be a source of hydrostatic uplift water pressures on the bottom of the desiccated, brown silty clay and silt stratum, described as soil type (a).

c) Silty clay, and silty clay with pebbles (Clay till)

This stratum is the main overburden over the shale bedrock in the area. It was encountered in all six test holes immediately below the above-described organic sand layer. In test holes 2, 3 and 5 it was found to have a mean thickness of approximately 85 ft., while the remaining borings were terminated in it.

It may be relevant to note that during the site investigation for the Sydenham River bridge we have encountered the clay till stratum also in part under the organic sand, and that the thickness of the till at that site was very similar to the present case. Therefore, we have attempted to correlate the geotechnical

D. SOIL CONDITIONS - Cont'd

properties of this material at the two sites in order to obtain a better appreciation of the conditions with the minimum amount of laboratory testing. However, since this material is likely to support the foundations of the bridge, either in the form of spread footings or piles, it was necessary to study its geotechnical properties in a certain detail, particularly in view of the considerable settlement observable on a relatively new reinforced concrete bridge, supported on steel H-piles, which is located between the two new bridge sites.

The main geotechnical properties of the silty clay, as affecting the design of bridge foundations, will now be discussed.

(1) Composition

Three grain size distribution curves, determined on typical samples of the silty clay from various depths, have been plotted on Fig. 4 b, to illustrate the classification, based on visual description of the soil samples, as entered on the borehole logs.

The upper layers of this stratum were found to possess a high clay fraction, which reached 45% in the sample tested from a depth of 18 ft. in borehole 2. The top part of this stratum was relatively free of pebbles.

D. SOIL CONDITIONS . Cont'd

Lower down, commencing near the elevation 65, the clay fraction began to decrease gradually and the material included grits and pebbles. The grain size distribution was typical of a glacial clayey till. At the elevation 55, a sample analysed had a 34% clay content, with 14% of gravel. Near the elevation 15, the clay content decreased to 26%, and the gravel fraction was 16%.

In the bottom 15 feet of the stratum, the material was more plastic, and although no grainsize tests were performed, the clay fraction can be assumed to increase, while a decrease in stone content was observed.

The stones embedded in samples of clay from the zone between the elevations 65 and 10 frequently reached 1.5 inches in diameter.

The percentage of gravel was considered sufficiently high to make the validity of in-situ vane tests below the depth of about 35 ft. problematical and consequently such tests were not attempted.

D. SOIL CONDITIONS - Cont'd

(ii) Consistency and Shear Strength

The variation of geotechnical properties of the silty clay with depth is presented on Fig. 1.

As the results from all four deep test holes follow a common pattern of distribution with depth, the silty clay stratum can be regarded as uniform horizontally. It appears permissible to assume the average curves, plotted on Fig. 1, as applicable throughout the project, both under the bridge foundations and under the embankments.

The distribution of consistency and strength with depth was as follows.

Near its boundary with the overlying organic sand layer, the silty clay was relatively firm, although the results of standard penetration tests (N-values) and the water contents were rather scattered. The N-values ranged from 4 to 14, while the average water content was 30%. The undrained shear strength near the elevation 75 was 1250 lb./sq.ft.

D. SOIL CONDITIONS - Cont'd

The silty clay gradually softened with depth down to the elevation 70. At this level, the N-values dropped to an average of 4 blows per foot, the shear strength fell to 850 lb./sq.ft. and the moisture content rose to an average of 40%, which is near the middle of the plastic range of the material.

Below the elevation 70, the strength of the silty clay increased progressively, with a corresponding decrease in the natural moisture content. The average of the N-values increased to 22 blows per foot near the elevation 35, and the undrained shear strength at a similar elevation reached a mean value of 2300 lb./sq.ft. The average moisture content dropped at this elevation to 23%. The increase in strength was accompanied by a steady decrease of the liquid limit and of the plasticity index, while only a slight decrease in the plastic limit was observable.

Below the elevation 30, the silty clay till again became progressively weaker. The N-values steadily fell to an average of 10 blows per foot at the elevation 0.0, and the undrained shear strength dropped to about 1000 lb./sq.ft. at this elevation. The loss of strength with depth below the elevation 30 was accompanied by a rise in the moisture content; however, down to the elevation +8 this increase was only slight, but became more pronounced between the elevations 8 and -4. In this bottom layer of the stratum, the

D. SOIL CONDITIONS - Cont'd

average moisture content increased from 21% to 35%. Also, considerable increase of the liquid limit below the elevation 10 was observed. At the elevation -2, the liquid limit reached 51.4%, while the plastic limit increased from a low of 18.4% at the elevation 13 to 25.1% at the elevation -2. This change in plasticity characteristics is caused by the increased clay fraction of the material in the bottom layers.

Below the elevation -3.6 to -4.2, the silty clay was followed by the dense silty sand which contained natural gas and water under pressure.

Comparison of the distribution with depth of consistency of the silty clay till at this site with that of the site of bridge C-7-4, (Graph No. 1 of our Report No. 6140), shows that a similar zone of maximum shear strength exists at both sites roughly in the middle of the silty clay till layer. Below this zone, the shear strength at both sites was found to decrease steadily down to the bottom of the stratum. However, no zone of minimum shear strength, corresponding to that detected at the present site near the elevation 70, was observed in the upper portions of the silty clay layers at the site of bridge C-7-4.

D. SOIL CONDITIONS - Cont'd

Although the pattern of distribution of the consistency and shear strength at the two sites was similar, with the above exception, the standard penetration test results in the strongest zone at the present site were considerably higher than at bridge C-7-4. Also, the average moisture content, plotted on Fig. 1, is about 2.5% lower than the corresponding average at the previous site, while the plasticity of the clay in the present case is somewhat smaller. However the distribution of the undrained shear strength is roughly the same at both sites.

In view of this, it may be possible that the higher standard penetration resistance at the presently investigated site is due to a higher proportion of stones or to their larger size, while the lower moisture content and plasticity index is caused by a bigger silt fraction. The shear strength need not be materially affected by these variations in composition.

In Fig. 2 the undrained shear strength of the silty clay was plotted against the standard penetration resistance near the depth from which the compression test samples were recovered. A roughly linear relationship is observable and can be approximately expressed by the following equation, in which the scatter of the results is considered.

D. SOIL CONDITIONS - Cont'd

$$c_u = 90N \pm 30 N \text{ lb./sq.ft.}$$

Where: c_u = Undrained Shear Strength

N = No. of blows in the Standard Penetration Test

In order to determine the sensitivity of the clay, which may cause a loss of strength on remoulding by the action of driving of piles, a number of unconfined compression tests were performed on remoulded samples from various depths, the undisturbed strength of the samples having been previously determined. The relationship between the undisturbed and the fully remoulded shear strength is plotted on Fig. 3, and indicates a linear relationship, with a mean sensitivity of about 1.2 for samples from below the elevation 70. This value is in agreement with the range of sensitivity of 1.2 to 2.9 determined on samples of the silty clay till at bridge No. C-7-4.

In the weak zone near the elevation 70, the silty clay was considerably more sensitive.

D. SOIL CONDITIONS - Cont'd

3. Compressibility

Consolidation tests were carried out on four undisturbed samples of the silty clay from between the elevations 78 and 60, where the compressibility is probably highest. The results are plotted on Figs. 6 a, b, c and d, in the form of void ratio - log pressure curves. The compressibility, consolidation and permeability coefficients for the various loading stages of the tests are included on the graphs in tabular form. The shape of the void ratio - log pressure curves indicates that the silty clay can be considered as lightly overconsolidated.

The compressibility coefficient, m_v , deduced from the consolidation tests for a load increase of 1 ton/sq. ft. in excess of the estimated existing effective overburden pressure, was plotted against elevation on Fig. 1. The compressibility in the soft zone near the elevation 70 was relatively high, reaching 0.037 ft.sq./ton. Below this soft zone, the compressibility was found to decrease.

In view of the similar properties of the middle and lower portions of the silty clay stratum at this site and at the site of bridge No. C-7-4, it appears permissible to use the two results of consolidation tests presented in our Report No. 6140 for the

D. SOIL CONDITIONS - Cont'd

estimation of settlement in the lower portions of the silty clay till stratum at the present site. The corresponding compressibility coefficients from our Report No. 6140 are included on Fig. 1.

No values of compressibility are available below the elevation 15, where the moisture content and liquid limit were found to increase sharply, so that a considerable increase in the compressibility coefficient can be expected. However, from a comparison of the relationship between the moisture content and the plastic properties of the clay of this bottom soft layer with the very similar relationship in the soft zone near the elevation 70, from which consolidation test results are available, it appears permissible to use these results for the estimation of the settlement in the bottom 10 ft. of the silty clay stratum.

Undrained triaxial compression tests were carried out on undisturbed samples of the silty clay from several depths, to determine the stress-strain relationship for the purpose of "immediate" settlement calculations. The modulus of deformation, as deduced from the loops of stress-strain curves in unloading and reloading cycles, was found to be as follows:-

D. SOIL CONDITIONS - Cont'd

B.H. / SA No.	4/11B	4/18	3/20
Depth	24'-6"	43'-6"	44'-6"
Modulus of Deformation ton/sq. ft.	90	187	163
Undrained Shear Strength lb./sq. ft.	695	950	2040
Water Content %	27.9	24.5	19.1

d) Dense Silty Sand

A stratum of silty sand was encountered below the silty clay till. It commenced near the elevations 3.6 to 4.2 and continued to the top of the black shale bedrock, near the elevation - 13.

The silt and sand stratum contained stones, often in the form of broken angular fragments of black shale. Test hole 3 was terminated at the elevation -5.1, where refusal was encountered probably due to a boulder. In test hole 5, refusal occurred at the elevation -11.5, where either a boulder or the shale bedrock was struck.

The dense sand stratum was dark grey to black, the sand having the appearance of a finely fragmented shale.

D. SOIL CONDITIONS - Cont'd

The grain size varied from fine to coarse, and included a proportion of silt and silty clay. Standard penetration tests performed in this layer gave results of 20 and 32 blows per foot in the upper portions of this stratum and 98 near the lower boundary.

The sand contained natural gas and water under pressure, which came up violently in the test holes.

e) Shale Bedrock

Black shale bedrock commenced at the elevation -13.1 in test hole 2, where it was proved by 5 ft. 11 ins. of diamond drilling. The shale was found to be horizontally bedded, and was solid and very hard. It provides a very sound basis for any end-bearing piles.

The shale was not proved by diamond drilling in other test holes, but refusal in test hole 5 at the elevation -11.4 indicated that the surface of the shale can probably be considered as roughly horizontal throughout the site. Moreover, at the site of bridge C-7-4 located roughly one mile to the south, the surface of the black shale was proved by diamond drilling in three test holes and was found to be practically horizontal.

D. SOIL CONDITIONS - Cont'd

It will be possible to compare the elevation of the bedrock at the two sites when the geodetic elevations of the present site will be available, but it appears that the shale commences at both sites at comparable levels.

E. WATER CONDITIONS

The ground water table at the site is controlled by the water level in Bear Creek. At the time of the site investigation, the water surface was at the elevation 86.8, but, similarly as the near-by Sydenham River, the Bear Creek is known to be subject to considerable flooding.

The depth of the creek was not determined.

The layer of silty organic sand with gravel, which was described as soil type (b) and which occurred roughly between the elevations ^{89 and 91,} can be assumed to contain ground water in hydraulic continuity with the water in Bear Creek. The uplift pressures in this organic sand layer on the bottom of the brown silty clay, described as soil type (a), which overlies this organic sand layer, can be assumed as equal to the height of water in the creek, under steady flow conditions. However, after a rapid drawdown from ~~high~~ flood

E. WATER CONDITIONS - Cont'd

level, excess pore pressures could exist in the organic sand layer for a time, particularly at some distance from the creek. This possibility should be taken into account in embankment stability considerations.

Some water under pressure was found in the dense silty sand layer resting immediately over the shale bedrock. In test hole 2, water overflowed casing to 3 ft. 7 in. above ground level. Thus artesian conditions exist below the silty clay till stratum.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS

1. Summary of Subsoil Conditions

Details of the subsoil conditions found in the six test holes are given on the borehole logs included with this report. A simplified subsoil profile was prepared, and is presented on the enclosed drawing. The elevation and thickness of the main soil types are tabulated on page 9 of the report, the elevations being referred to a temporary bench mark of assumed elevation of 100.0. The position of this bench mark is described on page 3.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

At the site of the proposed bridge, the subsoil consisted of the following strata:

A layer of firm, brown, silty and sandy clay commenced under a thin layer of organic topsoil and continued down to an average elevation 84. The thickness of this layer depended on the grade elevation at the positions of individual test holes, and varied from 2 ft. 6 in. in test hole 3 to 16 ft. 2 ins. in test hole 4. The clay was firm in its upper portions, but became progressively softer with depth. It contained some organic matter, particularly in the lowerpart of the deposit.

The brown silty clay was resting on a layer of fine to coarse silty sand with some gravel, containing a very high proportion of organic matter. This layer was at the bridge site 1 ft. 4 in. to 3 ft. 4 in. thick, occurring between the elevations 81 and 84. The organic sand layer contained ground water.

The organic sand was followed by a grey silty clay, with a stone content which progressively increased with depth. This stratum, identified as a silty clay till of glacial origin, is the main overburden over bedrock in the area. At the present site it was approximately 65 ft. thick.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

The silty clay was found to have a soft zone near the elevation 70, where the average shear strength was only about 850 lb./sq. ft. The material became stronger with depth, reaching a maximum average strength of about 2300 lb./sq. ft. near the elevation 35. Thereafter, the silty clay again became progressively softer, and near the bottom of the stratum the shear strength dropped to about 1000 lb./sq. ft.

The silty clay till was followed by an approximately 10 ft. thick layer of dense silty sand containing natural gas and water under pressure, which in turn was followed by the shale.

Hard black shale bedrock was found at a depth of 107 ft. in test hole 2, corresponding to the elevation -13.1. Test holes 1 and 6 were drilled on the anticipated line of the embankments, 200 ft. south and north of the probable positions of the two abutments respectively. A similar stratification as at the bridge site was confirmed to a depth of 21 ft. by these holes, with the exception that the organic layer was more silty and less sandy, and was thicker in test hole 1 and thinner in test hole 6 than at the site of the bridge. It was also at a slightly higher elevation.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

For the consideration of soil problems connected with the bridge foundation design and the construction of the embankments, the subsoil can be regarded as uniform horizontally over the section investigated. The average engineering properties of the subsoil, as given in Chapter D and illustrated graphically on Fig. 1, can be considered applicable both to the bridge structures and to the embankments.

The test holes have disclosed that down to the elevation 70 the subsoil possesses a relatively low bearing capacity and has high compressibility characteristics, partly due to the soft layers of the silty clay and partly due to the presence of a layer of sand with organic content.

2. Bridge Foundations

The following types of foundations for the bridge piers and abutments have been considered:

- a) Spread footings, or mats
- b) Friction Piles,
- c) End-bearing piles, resting on bedrock.

Each of the possible solutions will now be discussed.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

a) Spreading footing, or mat foundations

We do not know the exact proposed locations of the bridge piers and abutments, and, as the elevation of the existing grade at the site varies considerably, in the following discussion it will be assumed that the structures will be located at or near the positions of the test holes. We will consider the two piers and the two abutments individually.

i) South Abutment

Chainage 258 + 00

Elevation 93.9 (test hole 2)

Assuming that this abutment is located sufficiently far from the river's edge so as not to be affected by scouring, it could theoretically be constructed on a mat foundation placed 3 ft. below the existing grade, i.e. at the elevation 91. To protect it against frost action, the foundation would have to be covered by approximately 5 ft. of backfill.

Placed at this high elevation, the foundation would be taking advantage of the firm, brown crust of the silty clay which was found to continue to 5 ft. below the existing grade. However,

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

below this depth, the clay became softer, while at a depth of 9 ft. 10 in., a 2 ft. 8 in. thick layer of soft organic silt, or gravel with clay was encountered.

On account of the existence of the organic layers, the maximum bearing capacity at the elevation 91 would not be more than 1.0 tons per sq. ft. Even this limited loading would undoubtedly lead to considerable settlements, the bulk of which would be in the organic layer. The amount of settlement would depend on the size of the mat, but could amount to several inches.

The settlement of the completed abutment could be considerably reduced if the constructed mat foundation was loaded for a period of one or two months with a surcharge, preferably exceeding the final bridge loads to be transmitted through the abutment.

} Is this
practical
at all??

In this way, the bulk of the settlement of the subsoil would occur prior to the construction of the bridge, although, due to the organic content of the subsoil with which secondary settlements are associated, some vertical movement would continue for a period after completion of the bridge, and suitable provisions to allow for such settlements should be included in the bridge design.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

Because of the limited bearing capacity and the hazard of settlements, as well as due to some possible danger of locating foundation at a shallow depth in an area which is subject to flooding, it would appear considerably more preferable to construct the abutment at a greater depth. However, due to the occurrence of the soft clay and the organic sand and silt, no suitable foundation level is available above the elevation 79. The allowable bearing capacity increases progressively with depth below this elevation.

As we have not been supplied with data regarding the likely dead and live loads of the bridge and we do not know the required size of a mat foundation, we have carried out bearing capacity and settlement calculations assuming two possible sizes of footings, namely 50 x 12 ft. and 30 x 8 ft.

The computations were carried out for foundation elevations 79, 75, 70 and 65. In the following table, we have summarized the results of these calculations, giving the allowable uniformly distributed total pressure under the foundation at each

**F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd**

of the above elevations. We have also included the value of the theoretical settlement, both for a net stress increase of 1.0 tons/sq. ft. in excess of the existing estimated overburden pressure, as well as for an uniformly distributed working pressure equal to the maximum allowable bearing capacity.

Ground elevation ~ 84.0

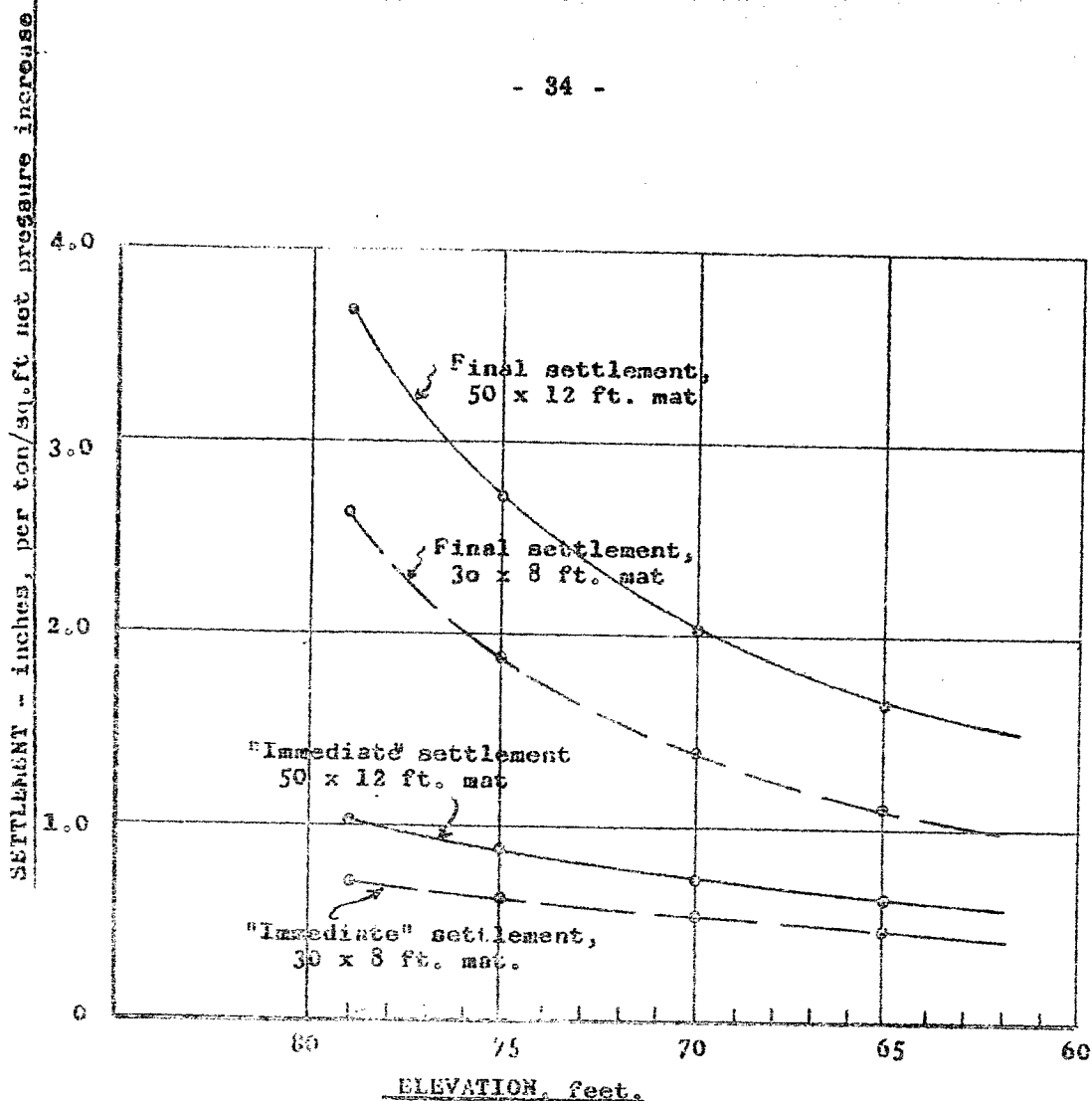
South Abutment - 50 x 12 ft. mat.

Elevation:	ft.	79	75	70	65
Estimated existing overburden pressure:	tons/sq. ft.	0.66	0.90	1.20	1.50
Allowable pressure increase:	t./sq. ft.	0.84	0.85	1.30	1.75
Total allowable pressure:	t./sq. ft.	1.50	1.75	2.50	3.25
Settlement per ton/sq. ft. of net pressure increase (inches)	Immediate	1.02	0.98	0.73	0.60
	Consolidation	2.66	1.83	1.29	1.0
	Total	3.68	2.71	2.02	1.60
Settlement for total pressure equal to allowable bearing capacity. (inches)	Immediate	0.86	0.75	0.95	1.05
	Consolidation	2.24	1.56	1.67	1.75
	Total	3.10	2.31	2.62	2.80

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

South Abutment - 30 x 8 ft. mat.

Elevation:	ft.	79	75	70	65
Estimated existing overburden pressure	t./sq. ft.	0.66	0.90	1.22	1.50
Allowable pressure increase	t./sq. ft.	0.94	1.10	1.30	1.75
Total allowable pressure	t./sq. ft.	1.60	2.0	2.5	3.25
Settlement per ton/sq. ft. of net pressure in- crease (inches)	Immediate Consoli- dation Total	0.71 1.91 2.62	0.62 1.26 1.88	0.54 0.85 1.39	0.45 0.65 1.10
Settlement for the total allowable pressure (inches)	Immediate Consoli- dation Total	0.67 1.80 2.47	0.68 1.39 2.07	0.71 1.11 1.82	0.76 1.10 1.87



Variation of theoretical settlement of foundation mats with elevation.

Note: Settlements are per ton/sq.ft. of net stress increase from the existing estimated total overburden pressure.

GRAPH I - SOUTH ABUTMENT.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

In the above Graph I we have plotted the theoretical settlement of the 60 x 12 ft. and the 30 x 8 ft. mats, against the elevation of the mats. The settlements plotted on this graph are for a net pressure increase of 1.0 ton/sq.ft. in excess of the estimated existing overburden pressure. For a net stress increase other than 1.0 ton /sq.ft. the theoretical settlement can be deduced from these graphs by simple proportion. In this way, the estimated settlement for the allowable bearing pressures have been obtained, as given in the preceding table.

"Immediate" and final long-term settlements have been treated individually; the "immediate" settlements will occur mostly during construction and ^{soon} after the application of the full working load, while the long-term settlements will be developed over a number of months. The total settlement is the sum of the immediate and consolidation settlements.

If the footings are placed at elevations other than given in the preceding table, the likely settlements can be obtained by interpolation from the included graphs, between the elevations 60 and 79. The allowable bearing capacity can be interpolated between the tabulated values at the various elevations.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

From the above tables and graphs it may be concluded that, for an adequate bearing capacity with tolerable settlements, the footings would have to be founded at an elevation not higher than 70, and preferably lower. This would entail a considerable depth of excavation, and it may prove more economical to build the bridge on piles.

ii) South Pier

Chainage 257 + 40

Elevation 86.9 (test hole 2)

The ground elevation at the likely position of the south pier was 7 feet lower than at the south abutment. Consequently, the stiff brown clay crust at the present position is only 2 ft. 6 in. thick and it will be necessary to construct the pier on a mat foundation, placed below the organic sand and silt layers, and at a sufficient depth to protect the footings against the possible effects of scouring. For these reasons, it would not be possible to construct the foundation at an elevation higher than 75.

As for the south abutment, we have considered the bearing capacity and settlement of the south pier constructed on a mat of dimensions 50 x 12 ft. and 30 x 8 ft. The allowable bearing

**F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd**

capacity, the settlements for a net stress increase of 1.0 ton/sq. ft. in excess of the existing overburden pressure and the settlements under the allowable bearing capacity are tabulated below, for foundation elevations 75, 70, and 65. Also, the settlement per ton/sq. ft. of net stress increase has been plotted against the elevation on the following Graph II.

South Pier - 50 x 12 ft. mat.

Elevation	ft.	75	70	65
Estimated existing overburden pressure	ton/sq. ft.	0.60	0.90	1.20
Allowable pressure increase	ton/sq. ft.	0.65	1.10	1.55
Total allowable pressure	ton/sq. ft.	1.25	2.0	2.75
Settlement per ton/sq. ft. of net pressure increase (inches)	Immediate	0.93	0.78	0.64
	Consolidation	1.97	1.38	1.03
	Total	2.90	2.16	1.67
Settlement for the total allowable pressure (inches).	Immediate	0.60	1.23	1.83
	Consolidation	0.69	1.44	2.13
	Total	1.29	2.67	3.96

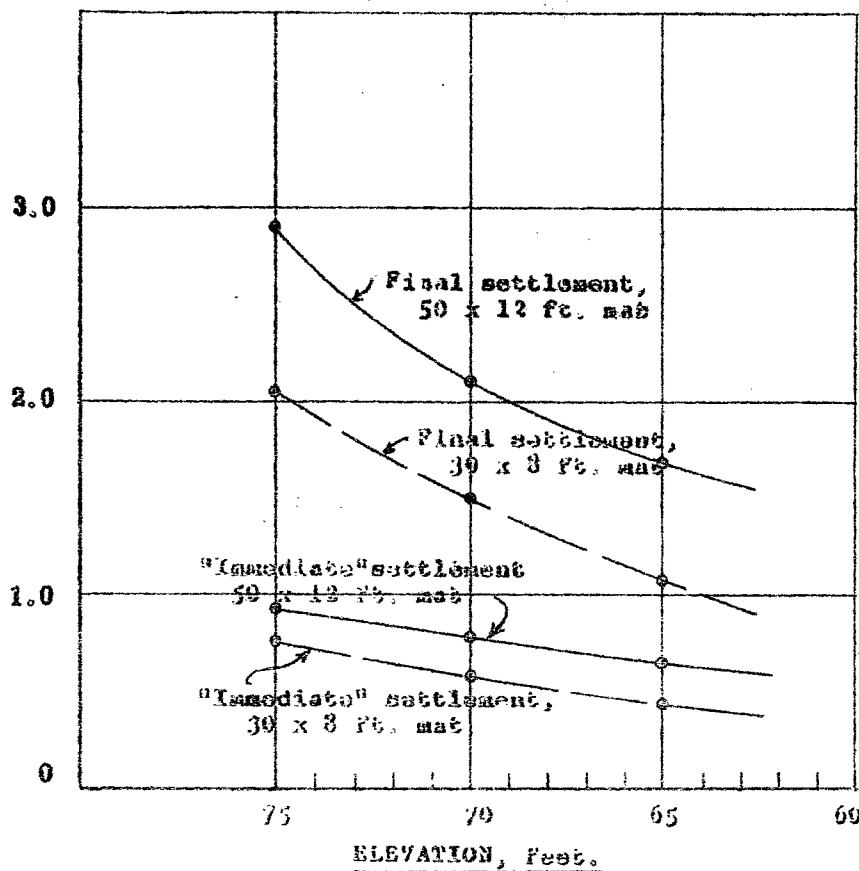
F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

South Pier - 30 x 8 ft. mat.

Elevation	ft.	75	70	65
Estimated existing over-burden pressure	ton/sq. ft.	0.60	0.90	1.20
Allowable pressure increase	ton/sq. ft.	0.90	1.20	1.80
Total allowable pressure	ton/sq. ft.	1.50	2.10	3.0
Settlement per ton/sq. ft. of net pressure increase (inches)	Immediate	0.66	0.58	0.42
	Consolidation	1.37	1.20	0.65
	Total	2.03	1.78	1.07
Settlement for the total allowable pressure (inches)	Immediate	0.60	0.69	0.76
	Consolidation	1.23	1.44	1.17
	Total	1.83	2.13	1.93

The settlements for the 50 x 12 ft. and 30 x 8 ft. mats have been plotted against the elevation on Graph II, for a net pressure increase of 1.0 ton/sq. ft. in excess of the existing estimated over-burden pressure.

SETTLEMENT - inches per ton/sq. ft. of net pressure increase



Variation of theoretical settlement of foundation mats with elevation

Notes: Settlements are per ton/sq. ft. of net pressure increase from estimated existing overburden pressure

GRAPH II - SOUTH PIER

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

Again it may be concluded that for a sufficient bearing capacity with tolerable settlements, the foundation would have to be put down at or below the elevation 70, and a piled foundation may prove more economical.

iii) North Pier

Chainage 256 + 75

Elevation 98.6 (test hole 4)

Test hole 4 was drilled on top of a bank, rising 12 ft. above the water level in the creek; the existing ground fell sharply in the direction of the creek. Thus, although a 16 ft. thick, firm crust of silty clay was present in this test hole, the pier will probably have to be founded below the creek bed and below the organic sand and silty layer. For considerations similar to the case of the south pier, we do not consider it advisable to construct the north pier at an elevation higher than 75, and adequate bearing capacity may only be available below the elevation 70.

The bearing capacity and settlements of foundations of the north pier would be practically the same as for the case of the south pier, and the values tabulated on pages 37 and 38 for various elevations of the south pier, can be applied to the north pier.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

iv) North Abutment

Chainage 256 + 35

Elevation 98.8 (test hole 5).

If this abutment is to be constructed near the position of test hole 5, it would be located approximately 50 ft. from the edge of the creek. Unless the creek is considerably widened in the direction of the northern abutment, this structure could theoretically be located 2 to 3 ft. below the existing grade, taking advantage of the brown, firm silty clay crust. However, similarly as in the case of the south abutment, due to the presence of the organic sand and silt layer, here between the depths of 14'-8" and 18 ft., below the existing grade, we would not recommend a higher foundation pressure than 1.0 ton/sq. ft. in order to limit the settlements to a tolerable amount.

As the above pressure will probably be inadequate,
it will again be necessary to place the abutment foundations at an
elevation not higher than 79. The allowable capacity at the various
levels below the elevation 79, for the northern abutment will be
0.5 ton/sq. ft. higher than at the south abutment, due to the fact
that the ground level is higher and the overburden pressure

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

component of the allowable bearing capacity is also higher at any elevation. Thus, the values of the total allowable pressure, tabulated on page 32 for the south abutment for the 50 x 12 ft. and page 23 for 30 x 8 ft. mats, can be increased by 0.5 ton/sq. ft.

of the case of the north abutment. For instance, the total allowable pressure at the elevation 70 would be 3.0 ton/sq. ft. both for 50 x 12 ft. and 30 x 8 ft. mats, as compared with 2.5 ton/sq. ft. at the south abutment.

The settlements are unaffected by these higher loads, and will be approximately the same as for the south abutment, as tabulated on pages 32-33 and plotted on Graph I on page 34. In fact, due to the higher grade on this side of the creek, the settlements at comparable elevations for the same net pressure increase will be a little smaller than for the case of the south abutment, but the differences would be too small to warrant a separate analysis, in view of all the more serious uncertainties involved in the computations.

v) Construction Problems

Assuming that the foundations will be placed in the silty clay till stratum below the elevation 79, excavations 20 to 30 feet deep may be required. Due to the presence of ground water,

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

which would probably flow freely from the creek through the organic silt and sand with gravel stratum between the elevations 81 and 84, it will be necessary to perform the excavations inside sheeting, driven into the grey silty clay stratum, which underlies the water-bearing organic sand layer.

In the case of the two abutments, the silty clay crust, which was 9 ft. 10 in. and 14 ft. 8 in. thick in test holes 2 and 5 respectively, is sufficiently strong to stand unsupported to a depth of at least 7 ft., and it may be sufficient to drive sheeting only from below this depth, if this is practical.

A cushion of granular material, at least 6 inches thick should be provided immediately below the footings, to form a suitable working surface for the placement of concrete. Great care should be taken not to allow access of free water to the excavated formation grade below the foundations, as this would lead to softening and increased settlements.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

b) Pile Foundation - Friction Piles.

(i) Bearing Capacity

Because of the considerable depth of excavation required to reach subsoil possessing adequate bearing capacity with tolerable settlements, it may be more economical to construct the bridge on piles.

As no design data on the bridge loads was available at this stage, we have carried out preliminary calculations of bearing capacity and settlements of piles, making the following assumptions:

1. The distribution of undrained shear strength with depth was assumed as the mean curve plotted in Fig. 1. This was confirmed by comparison with the shear strength profile at the site of bridge No. C-7-4, and with the shear strength-moisture content relationship of similar clay till, determined during several site investigations in South-West Ontario.
2. Factors proposed by M. J. Tomlinson in 1957, relating the adhesion of clay on driven piles to the undrained shear strength of the clay, have been applied. As a consequence, a mean adhesion of 700 lb./sq.ft. can be assumed to be available between

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

the elevations 80 and 30.

3. A 14 x 14 inch steel H-pile was considered and, in accordance with the D.H.O. practice, the perimeter effective in mobilizing skin friction was taken as $3/4$ of the perimeter of a 14 x 14 inch square section, i.e. 42 inches.

4. Only the length of pile embedded below the elevation 80 was considered effective in mobilizing skin friction. The contribution to bearing capacity from any layers of soil above this elevation was neglected as a safety measure.

5. End-bearing capacity below the pile toes was neglected as an additional security against a possible existence of soft pockets of clay immediately below the piles.

The calculations have shown that with a factor of safety of 2, for a working load of 30 tons per pile, the pile toes would have to be set at the elevation 30. At the elevation 40, with a factor of safety of 2 the maximum allowable load per pile would be 25 tons.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

However, in view of the presence of softer, and more compressible layers of clay in the lower part of the silty clay stratum, it would appear to be advantageous to place the pile toes at an elevation not lower than 40. At this elevation the toes would be placed in the zone of the maximum shear strength, which was established as existing near this elevation in the silty clay till stratum. (Fig. 1).

Although, for a factor of safety of 2, the load per pile, based on the average adhesion of 700 lb./sq.ft., would be only 25 tons per pile at the elevation 40, we consider it permissible to design the piles for a working load of 30 tons at this elevation. Assuming the mean adhesion to be 700 lbs. per sq. ft., a load of 30 tons per pile at the elevation 40 would include a factor of safety of 1.5. However, we consider that after the piles have been driven, the silty clay will reconsolidate, and the adhesion will gradually increase. Consequently, the factor of safety will rise with time; to reach the desirable value of 2.0, the adhesion would have to rise to approximately 950 lbs. per sq. ft. As the shear strength of the stratum was mostly well in excess of this value, the required adhesion can be safely expected to develop after reconsolidation.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

It is difficult to predict the time required for the adhesion to increase to the required value, but it seems reasonable to assume that it will be reached within 12 months of driving the piles, in view of the relatively high coefficient of consolidation, determined by consolidation tests. Between the time of completion of the bridge, when the factor of safety with respect to full loading would be 1.5, and the time when the adhesion reached 950 lbs./sq. ft. for a factor of safety of 2, it would be advisable to restrict the live loads using the bridge to 1/2 of the design value. The period of duration of this restriction can be specified as 12 months.

Impractical

It can be concluded from the above considerations that, for an allowable working load of 30 tons per pile, the piles could be set at the elevation 40 provided that the live loads are restricted to half of the design value for a period of 12 months after completion of the bridge. Should it, however, be required to have a factor of safety of 2 for a load of 30 tons per pile immediately after construction, the piles would have to be set at the elevation 30. However, the long-term settlement of piles at the elevation 30 would be higher, due to the greater proximity of the soft clay layers below the pile toes.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

(ii) Settlement of Piles

The settlement under a pile group supporting the piers and abutments is directly related to the number of piles in the group, their spacing and the final depth of penetration, which are unknown at the present. However, we have carried out an estimate of the settlements, assuming a pile group of the dimensions 40 x 12 ft.; this would correspond to a group of 30 piles, in three rows of ten, carrying a total load of 900 tons with 30 tons per pile. The pile toe level was taken as the elevation 40.

The settlement of the group of piles was calculated by two methods. In the first instance, it was assumed that the whole load of the pier or abutment supported by the pile group is applied at a depth equal to $2/3$ of the penetration of the piles, and that this load is uniformly distributed at this depth over the area of the pile group. The settlement of the pier or abutment, deduced on the above assumptions, was calculated as 2.3 inches.

The settlement was recalculated by another method, which assumes that the total load carried by the pile group is applied on a horizontal plane at the level of the pile toes, assuming a spread of stress of 1 horizontal to 4 vertical along the length of

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

the piles effective in mobilizing skin friction, i. e. from the elevation 80. On this assumption, the settlement of the pile group, supporting a total load of 900 tons, was calculated as 1.5 inches.

It is difficult to know which of the two estimates is more correct, as little is known of the distribution of stresses with depth in a group of friction piles in clay. However, we consider that a total settlement of 1.5 to 2 inches should be allowed for in the bridge design.

The settlement of a group of piles of other dimensions than assumed above, or for a different pile penetration, would have to be re-calculated. Also, both the bearing capacity and the settlement would have to be reviewed if it was desired to use a pile type other than the steel H-pile of 14 x 14 inch section, which was assumed in the above bearing capacity and settlement calculations.

(iii) Pile Driving Considerations

Hard pile driving conditions in the clay till can be anticipated, and it may be necessary to interrupt the driving to allow dissipation^{of} pore-pressures which may build up during the driving operations and result in increased resistance. The piles would have to be re-driven after a few days.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

If steel H-piles are employed, we would like to draw your attention to the following important considerations, which we have already mentioned in our Report No. 6140 for the bridge C-7-4.

There is a danger of gaps, or voids, forming adjacent to the pile along its length in the clay till, which can be subsequently filled with water, causing softening of the surrounding clay and consequent loss of adhesion. Such voids may be formed by the transverse vibration of piles during the hard driving operations, by the displacement of stones by the pile toe, and due to air being drawn in during the driving. The danger of formation of such voids is particularly acute in the case of piles with an H-section, due to the large surface area and the presence of sharp corners between the flanges and web.

The danger of softening of the clay around the piles could be particularly great if water is drawn into any such gaps from the river or from the permeable organic sand and silt layer.

All this is in contradiction to the stated earlier i.e. difficulty in driving apart from that, something like that has never been experienced driving DHC pile a number of piles were driven.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

We recommend therefore that if H-piles are used, they should be driven inside a tube, the purpose of which would be to cut off water from the creek or from the organic sand layer. Such a tube would have to be driven at least a foot into the top of the relatively impermeable silty clay till, which begins near the elevation 81. Any free water should be pumped out of such tubes before the piles are driven.

Should a method of installation of piles be used that involves pre-boring, lining would have to be provided at least to the top of the clay till in order to cut off water from the river and from the permeable organic sand layer. All free water should be pumped out from the pre-bored hole before driving the pile.

In view of the above discussed possibility of softening of clay around the piles if water accumulates in any voids, piles with a circular section may be considered safer than H-piles.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

(c) End Bearing Piles.

The problem of settlements would be completely eliminated if the bridge was founded on piles resting on the hard shale bedrock. The dense sand began at a mean elevation -5, the shale following (test hole 2) at the elevation -13.1. The bedrock was proved by diamond drilling to be very solid, and to possess a very high bearing capacity. The allowable bearing capacity of piles driven to or below the elevation -7 to -10, would be determined by the structural strength of the piles themselves. Pre-boring, with the hole cased to below the organic sand, could be considered as an alternative to driving the piles.

We would draw your attention to the existence of natural gas and water under pressure in the silty sand stratum immediately over the shale bedrock.

3. Embankments

We understand that the bridge approach embankments will be up to 20 ft. high. The embankments can be constructed on the existing grade, though the thin surficial layer of organic topsoil should preferably be removed to limit settlements.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

(i) Embankment Stability

There is no apparent danger of slip failure in the foundation strata for embankments with slopes up to 2:1, with the following reservation.

Pore-water pressures, which will build up during construction of the embankments, will dissipate in the direction of the embankment toes through the relatively permeable organic sand and silt layer, and may create temporary local instability at the toes. Should any signs of toe failure appear, stability would probably be restored by providing pressure relief holes. A row of auger holes could be drilled opposite the endangered portion of the embankment a few feet from the toe, about 10 feet apart. The holes should penetrate to the organic sand and silt layer which, according to the results of test holes 1, 2, 5 and 6, was at 10 to 16 feet below the existing grade.

However, due to the presence of the strong crust of brown silty clay above the organic silt and sand layer, the danger of any such failures appears remote, unless excess pore pressures existed in this ^{organic} layer following drawdown of high flood waters.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

(ii) Settlement of Embankments

Theoretically, a settlement of between 6 and 9 inches can be expected under a 20 ft. high embankment, due to the presence of the organic layer and of the soft portions of the silty clay till strata. However, it has been observed on other projects that the stiff crust of brown clay, which is also present at this site, acts as a raft, by spreading the weight of the embankment over a larger area. In this way, the pressures transmitted to the more compressible layers underlying the stiff crust are reduced, leading to considerably smaller settlement than estimated from theoretical considerations.

However, a settlement of several inches undoubtedly will occur under the higher parts of the embankments. It would be advisable to construct the embankment several months before completion of the bridge, so that most of the settlement will take place before the project is finished. The road surface should only be laid when the bulk of the settlement has taken place and the grade has been restored, where required.

F. ENGINEERING CONSIDERATIONS
and CONCLUSIONS - Cont'd

Care should be taken to compact carefully the backfill between the abutments and the embankment, to avoid the formation of a step.

(iii) Embankment Material.

As far as can be judged from the test holes drilled at the site, the firm brown silty clay, found immediately below the surficial layer of organic topsoil, can be used for constructing the embankment. A grainsize distribution curve of a typical sample from the depth of 4 to 5 ft. in test hole 1 is included in Fig. 4a. It consisted of 20% of clay, 49% of silt and 31% of fine to medium sand.

Due to the high silt and clay content, the material should be considered as susceptible to frost heave, and a layer of granular material, 6 to 12 inches thick, should be included under the pavements.

Any borrow pits should be located sufficiently far from the embankments so as not to endanger the stability of the slopes.

E. M. PETO ASSOCIATES LTD.

C. F. Freeman.

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

RK/ap

APPENDIX "A"

STANDARD PROCEDURE

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 augers, 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 inch dia. x 60° 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 or pumped out during the work as necessary, 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 levels.

Borehole logs are prepared giving details of the soil description and conditions as recorded in the field. These logs form the basis of the soil profile which indicates the general stratigraphy assumed to exist between the boreholes as represented by the borehole logs.

The boreholes are normally set out by the Field Engineer, who also records the ground elevations referred to a temporary bench mark or known reference point. If the Client has been responsible for setting out the boreholes and recording their ground elevations this is stated in the preamble to the report.

A plan is drawn up from drawings supplied by the Client or his representatives, showing the locations of the boreholes and the T.P.M. where applicable.

Normally, the standard penetration blows and the natural moisture contents are plotted against elevation as a graph, and these graphs form part of the appendices, together with laboratory test results, details, ground water readings and other soil characteristics which can be best illustrated in graphical form.

APPENDIX "B"

SOIL TEST RESULTS

UNCONFINED COMPRESSION TEST DATA SHEETS

Borehole No.	Sample No.	Depth, Ft.	Nat. Water Content %	Wet Density, p.c.f.	Dry Density, p.c.f.	Void Ratio	Undisturbed ¹ / _u /c Shear strength, p.s.f.	Remoulded Shear Strength, p.s.f.
2	5A	9'-9'8"	74.0	125	72	--	810	--
2	5B	9'6"-10'	27.6	129	101	--	730	--
2	8B	15'6"-16'	31.0	121	92	0.82	520	405
2	8C	16'-16'8"	33.0	119	89	0.89	1280	405
2	12A	24'-24'6"	41.4	113	80	1.10	970	370
2	12B	24'6"-25'	37.0	115	84	1.05	1090	1020
2	15A	34'-34'6"	26.2	126	100	0.63	1620	480
2	16B	34'6"-35'	25.0	126	101	0.68	1940	1450
2	18	39'6"-40'	25.5	124	99	0.70	1700	--
2	21B	48'6"-50'	24.7	124	99	0.69	1460	1300
2	22	54'6"-55'	23.5	126	103	0.64	1940	--
2	23	55'-56'6"	21.8	129	106	0.59	2380	2600
2	27	75'-76'6"	19.3	133	111	0.52	1730	1660
2	28	80'-81'6"	19.5	133	111	0.52	1440	1220
2	30	90'-91'6"	26.2	125	97	0.73	840	--
3	10A	19'-19'0"	48.5	113	76	1.21	810	284
3	10B	19'6"-20'	44.0	113	79	1.15	890	284
3	14	29'6"-30'	20.4	130	108	0.57	1540	--
3	17	34'-36'6"	22.4	131	107	0.53	1300	--
3	22	43'6"-50'	21.7	132	109	0.55	2300	--
3	23	50'-51'6"	20.6	135	113	0.49	2160	--
3	24	55'-56'6"	18.3	131	111	0.53	2300	--
3	25	60'-61'6"	21.2	130	107	0.58	1950	--
3	26	65'-66'6"	21.5	135	111	0.52	1660	--
3	27	70'-71'6"	21.3	134	110	0.53	1230	--
3	28	75'-76'6"	23.3	132	107	0.56	1360	--
4	20	45'-46'6"	23.8	125	101	0.67	1150	--
4	22	50'-51'6"	22.6	129	103	0.60	1150	--
4	23	59'-56'6"	20.1	123	110	0.54	2020	--
4	25	65'-66'6"	21.1	134	110	0.54	2540	--
5	3	5'-6'6"	23.2	127	103	0.63	4900	--
5	4	7'-8'6"	23.2	127	102	0.66	3170	--

"Continued"

UNCONFINED COMPRESSION TEST DATA SHEETS

Borehole No.	Sample No.	Depth, Ft.	Nat. Water Content %	Wet Density, p.c.f.	Dry Density, p.c.f.	Void Ratio	Undisturbed u/c Shear strength, p.s.f.	Remoulded Shear Strength, p.s.f.
5	15	40'6" - 41'	25.4	127	101	0.67	350	700
5	16	41' - 41'6"	27.3	125	98	0.72	1120	1000
5	21	60' - 61'6"	18.8	130	109	0.53	1700	1580
5	22	65' - 66'6"	20.9	134	110	0.52	2160	2010
5	23	70' - 71'6"	20.9	133	110	0.53	1800	2010
5	24	80' - 81'6"	21.2	134	110	0.52	1870	--
5	26	90' - 91'6"	20.4	133	110	0.53	1580	--
6	2	1' - 2'6"	22.1	120	98	0.72	3740	--
6	4	3' - 4'6"	23.1	125	107	0.67	3880	--
6	5	5'4" - 5'8"	24.3	126	101	0.67	2300	--

FIELD VANE TESTS

E. H. 4 a (10 ft. North of B. H. 4)

<u>Depth ft.</u>	<u>Shear Strength, p.s.f.</u>	
	<u>Undisturbed</u>	<u>Remoulded</u>
21'9" - 22'6"	1220	250
23'3" - 24'0"	1250	330
29'9" - 30'6"	880	150
31'3" - 32'0"	900	440

APPROXIMATE RELATIONSHIP
BETWEEN
NO OF BLOWS, S.P.T. AND UNDRAINED SHEAR STRENGTH
OF SILTY CLAY TILL

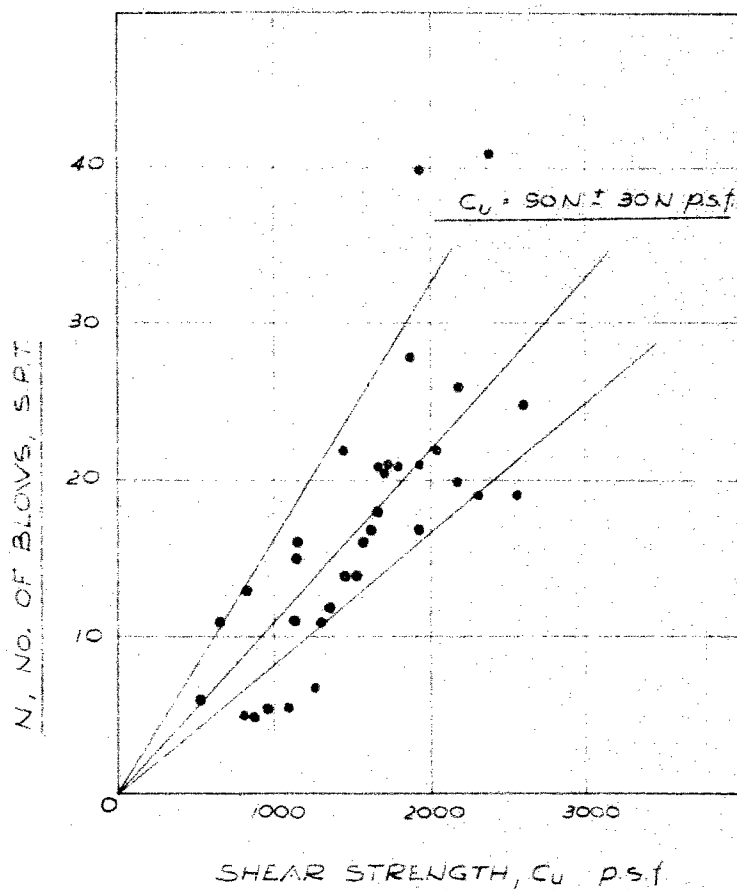


Fig. 2

JOB # 61170
e.m. peto associates ltd
DEC 1961
K.K

RELATIONSHIP BETWEEN 'UNDISTURBED' AND REMOULDED UNDRAINED SHEAR STRENGTH OF SILTY CLAY TILL

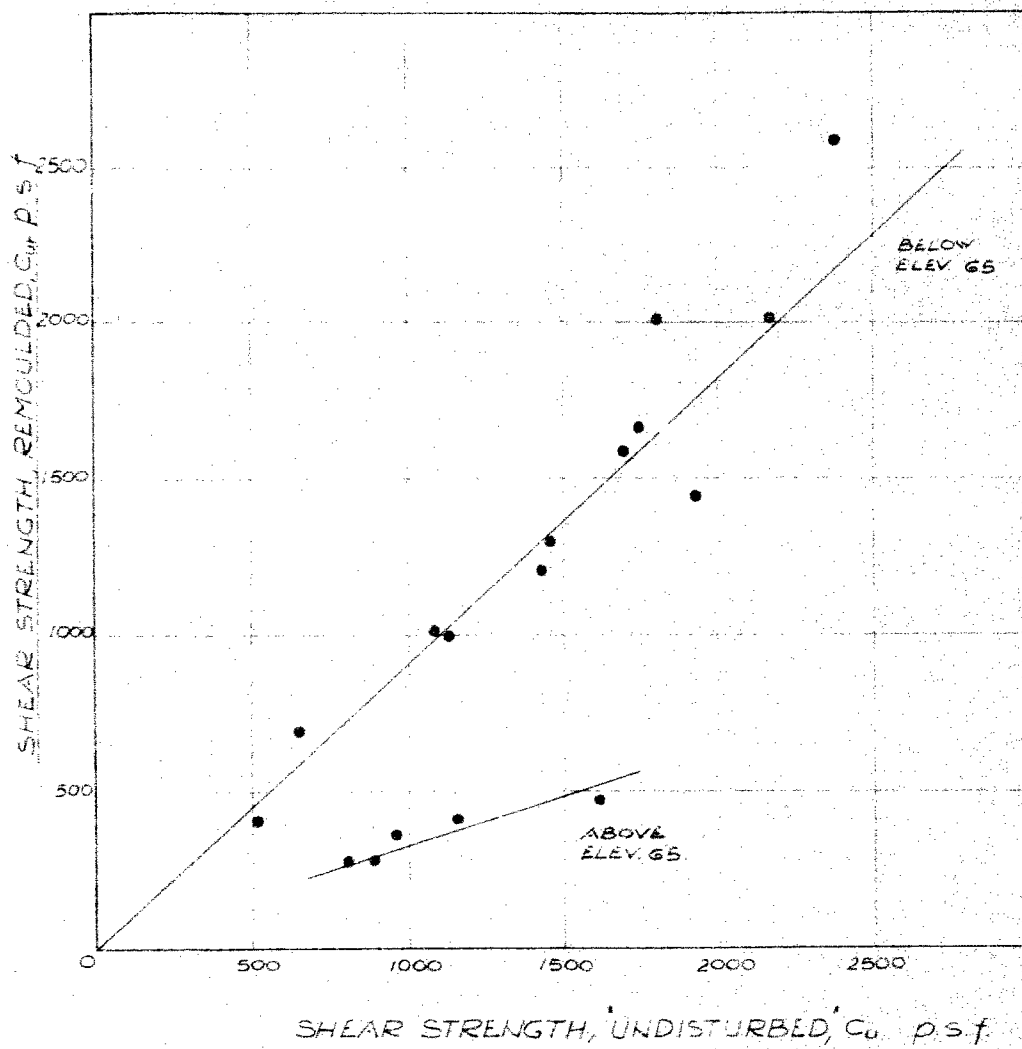
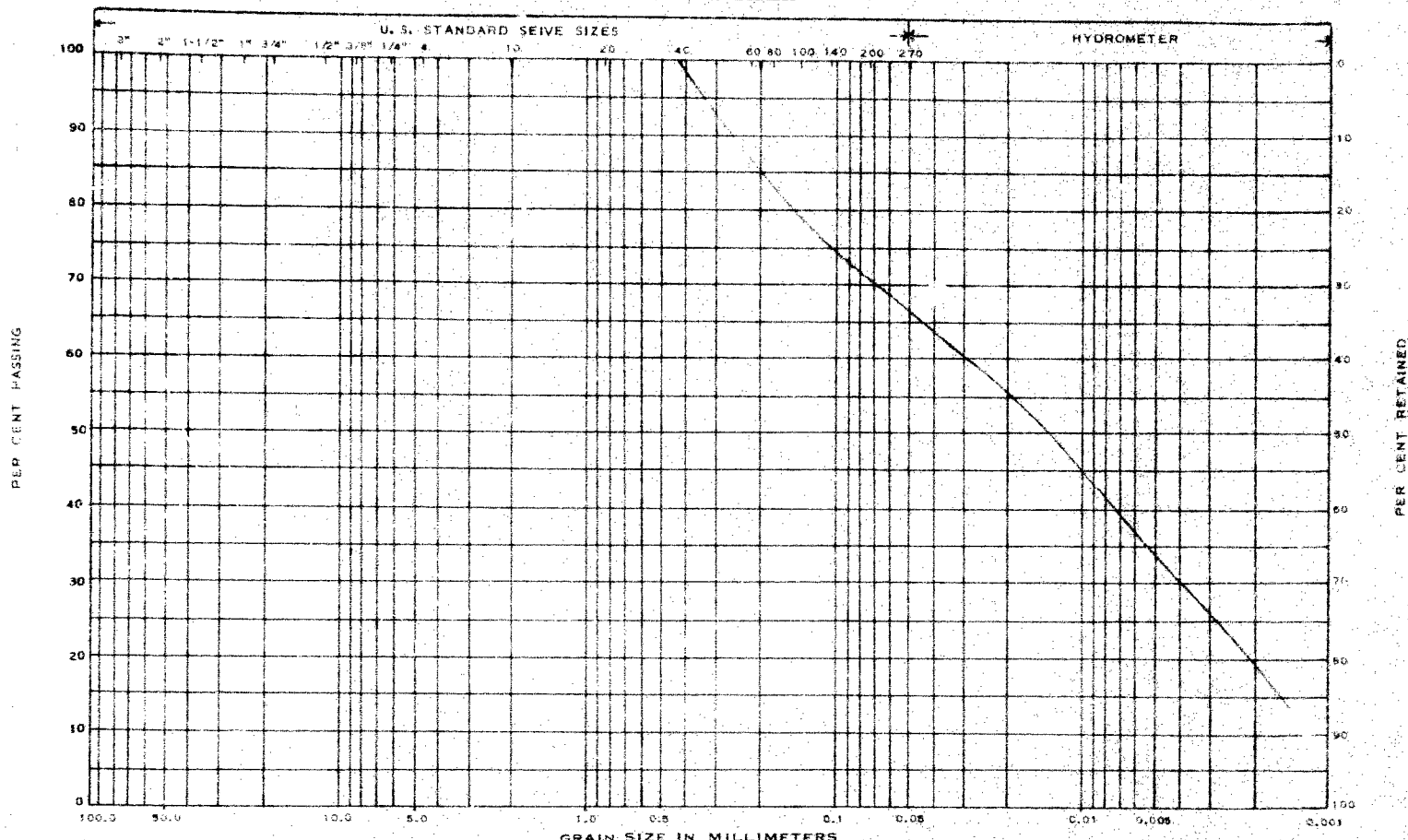


Fig. 3

e. m. peto associates ltd.

Toronto 19, Ontario



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

MASS. INST. OF TECH. CLASSIFICATION

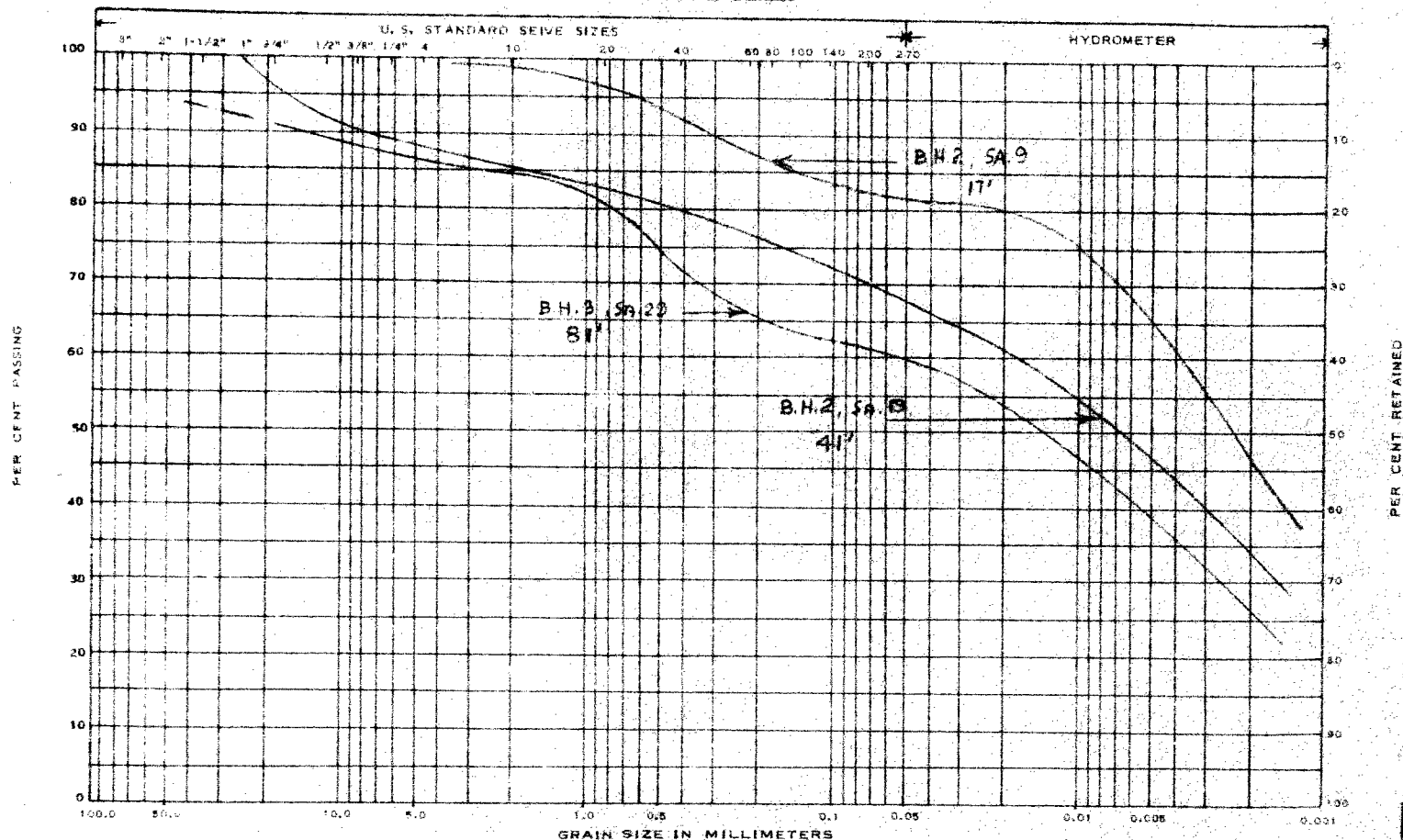
JOB NAME BEAR CREEK BRIDGE JOB NO. 01170 HOLE NO. 1 SAMPLE NO. 6
 DEPTH 4'-5' ELEVATION _____ REMARKS Brown, sandy and clayey silt.

GRAIN SIZE DISTRIBUTION

FIG. 4a

e. m. peto associates ltd.

Toronto 18, Ontario



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

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME BEAR CREEK BRIDGE JOB NO. 61170 HOLE NO. As shown SAMPLE NO. As shown

DEPTH As shown ELEVATION As shown REMARKS Silty Clay Till.

GRAIN SIZE DISTRIBUTION

FIG. 4b

e. m. peto associates ltd.

Toronto 19, Ontario

LIQUID LIMIT TEST

FLOW LINE CHARTS

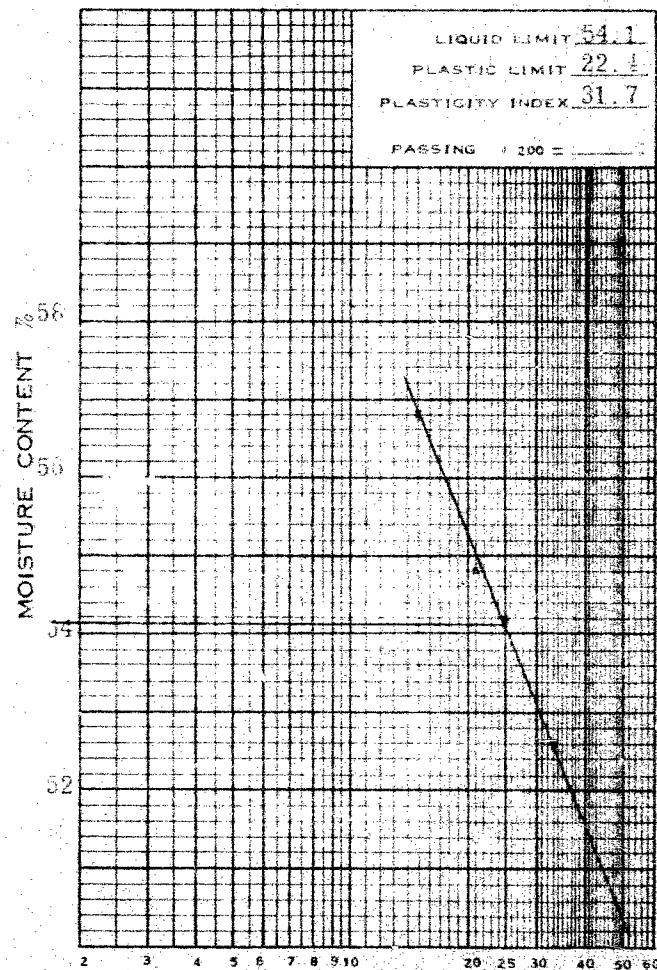
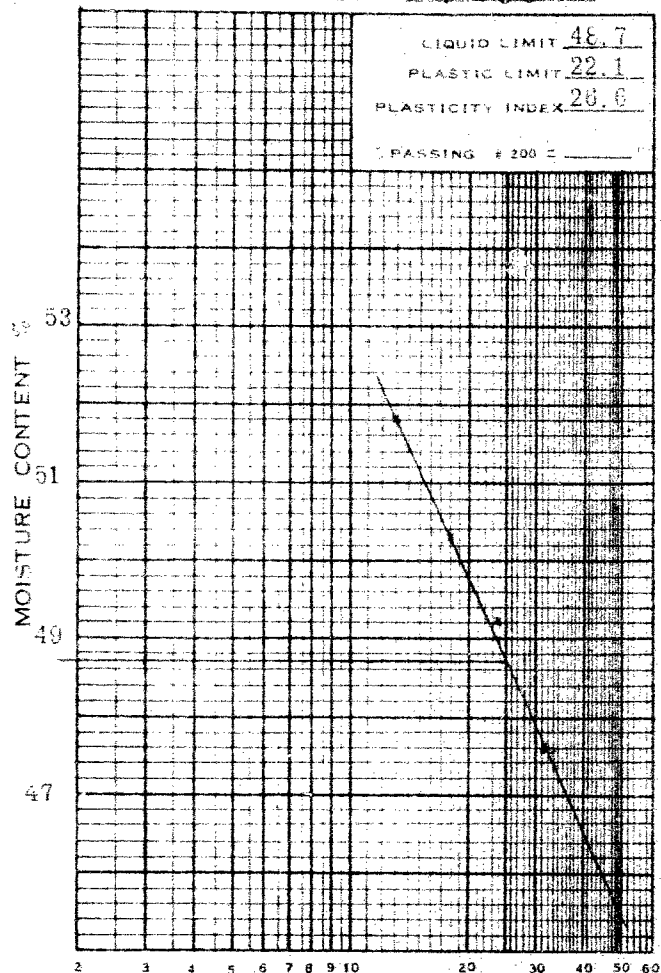
JOB No. 61170 PROJECT Bear Creek Bridge

SAMPLE FROM #2 SA 7

SAMPLE FROM #2 SA 13

DEPTH 7' - 8' 6"

DEPTH 25' - 26' 6"



e. m. peto associates ltd.

Toronto 19, Ontario

LIQUID LIMIT TEST

FLOW LINE CHARTS

JOB No 61170 PROJECT Bear Creek Bridge

SAMPLE FROM #5 SA 14

SAMPLE FROM #4 SA 14

DEPTH 30' - 31'6"

DEPTH 30' - 31'6"

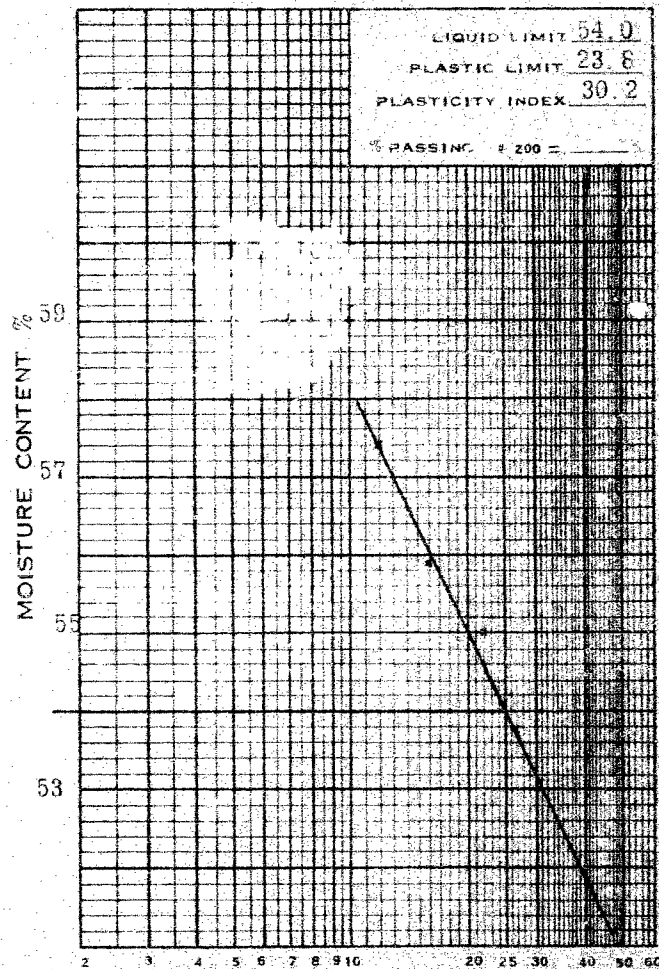
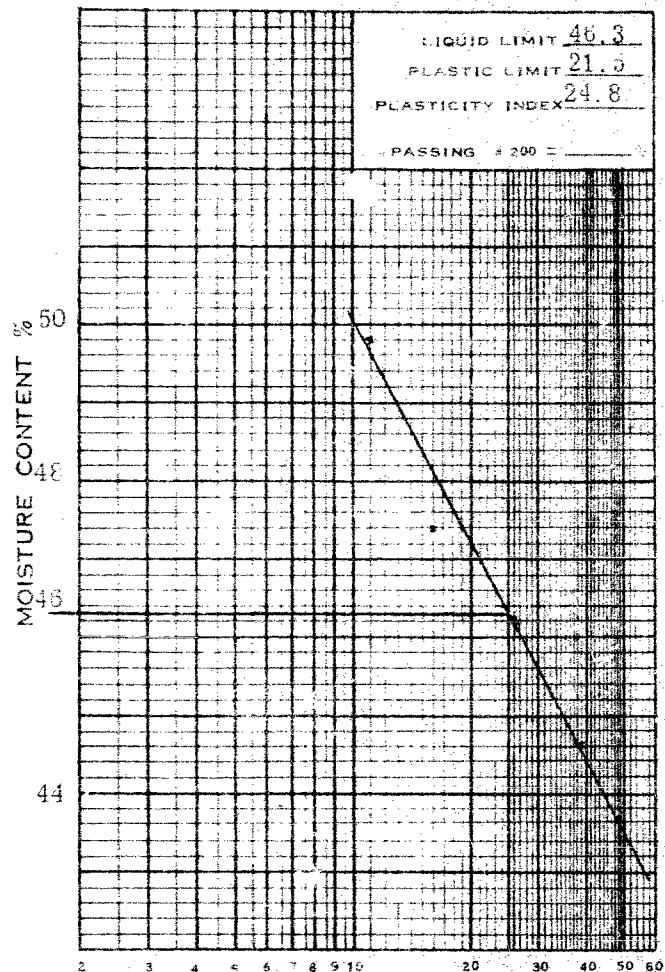


Fig. 57

e. m. peto associates Ltd.

Toronto 18, Ontario

LIQUID LIMIT TEST

FLOW LINE CHARTS

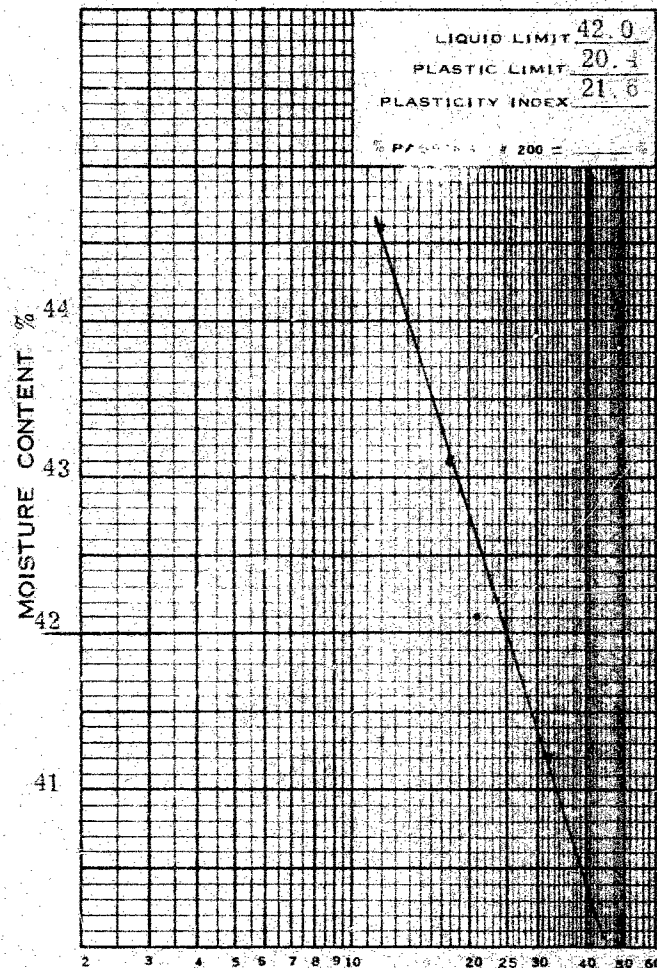
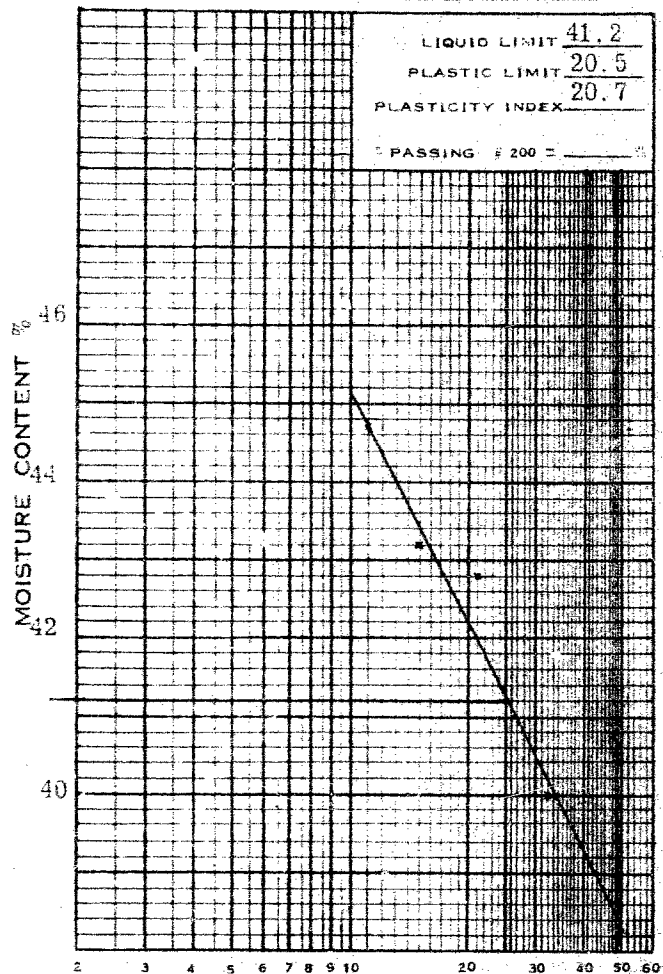
JOB No. 61170 PROJECT Bear Creek Bridge

SAMPLE FROM #2 SA 17

DEPTH 35' - 36'6"

SAMPLE FROM #4 SA 16

DEPTH 35' - 36'6"



NO. OF BLOWS (LOG SCALE)

Fig. 50

e. m. peto associates ltd.

Toronto 19, Ontario

LIQUID LIMIT TEST

FLOW LINE CHARTS

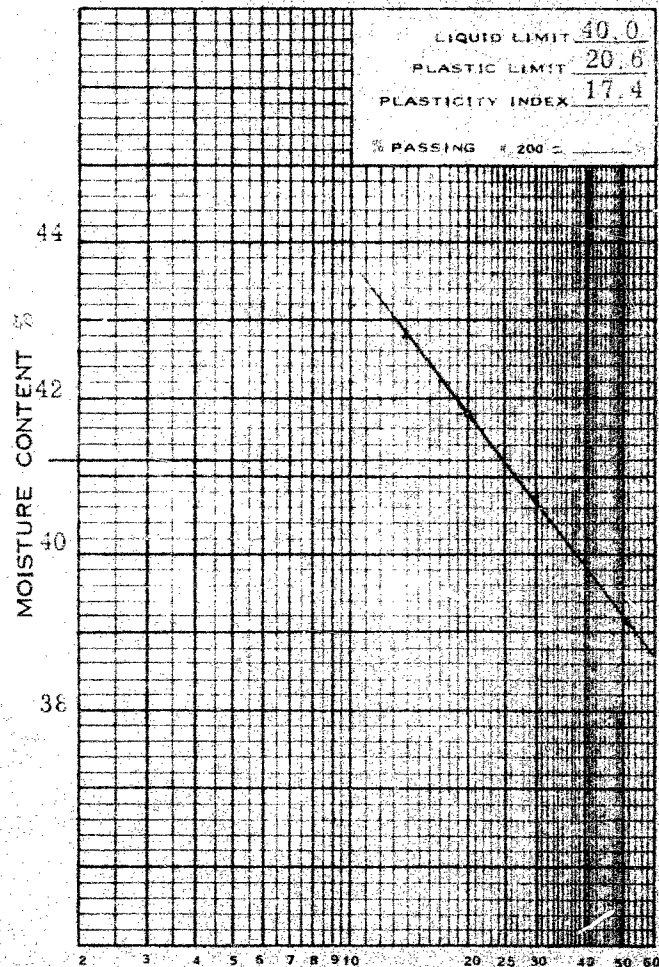
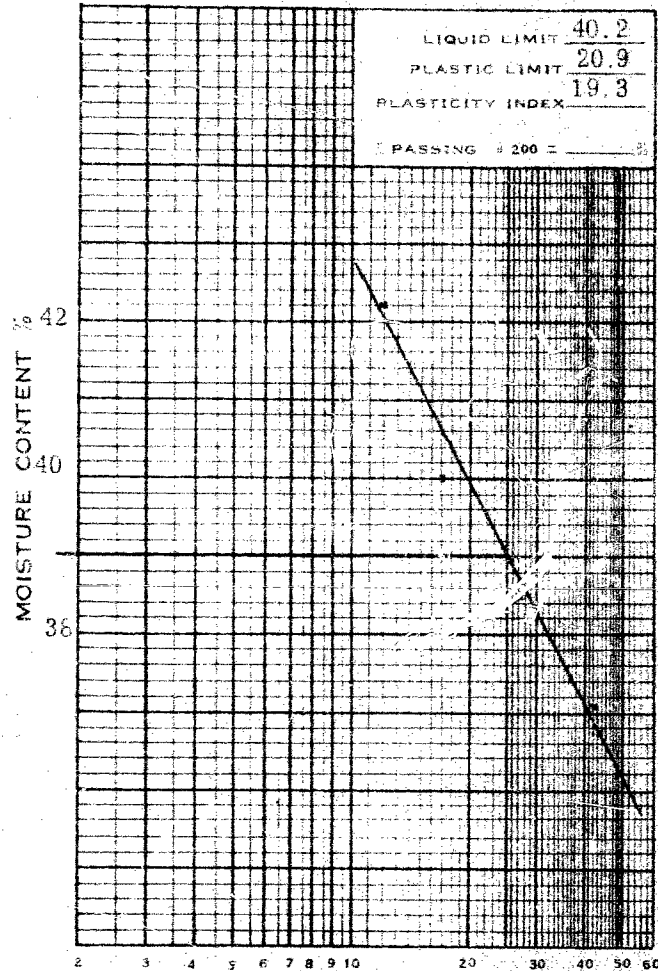
JOB No. 61170 PROJECT Bear Creek Bridge

SAMPLE FROM #5 SA 19

SAMPLE FROM #4 SA 20

DEPTH 51'6" - 53'0"

DEPTH 45' - 46'6"



NO. OF BLOWS (LOG SCALE)

e. m. paton & associates ltd.

Toronto 19, Ontario

LIQUID LIMIT TEST

FLOW LINE CHARTS

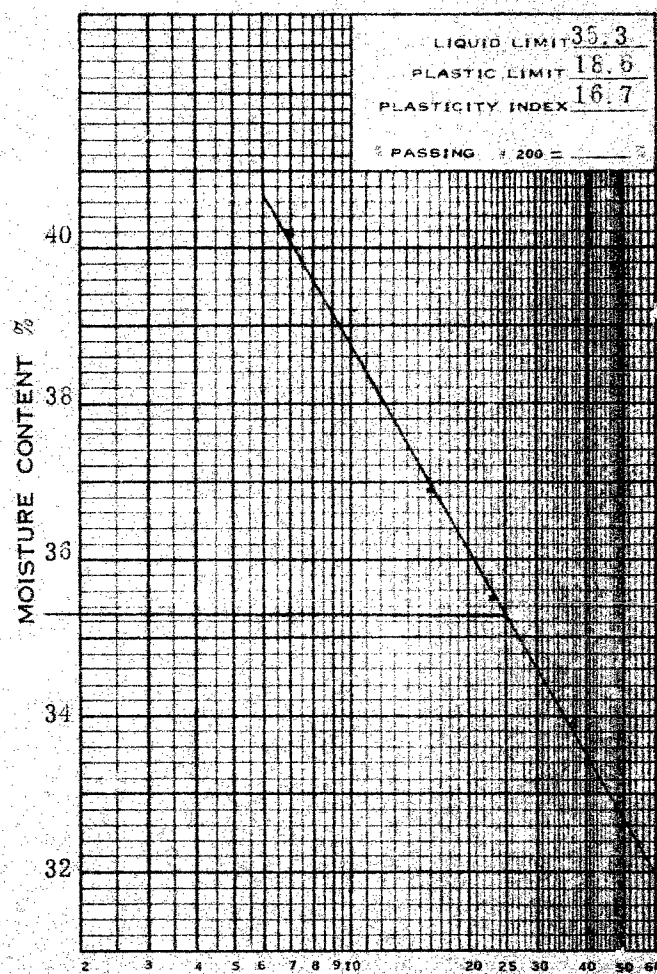
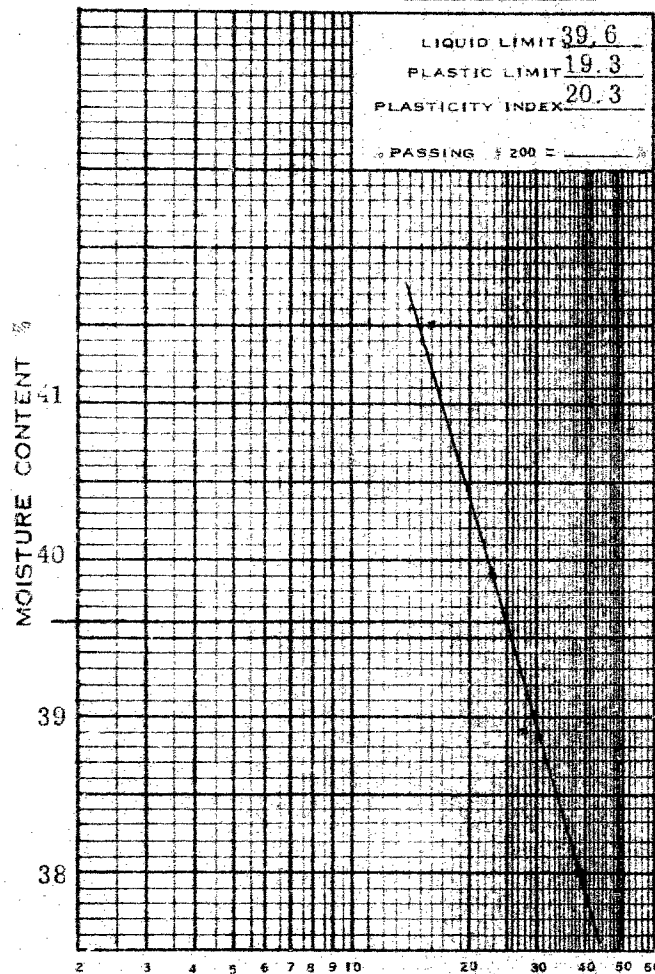
JOB No. 61170 PROJECT Bear Creek Bridge

SAMPLE FROM E.H. 2 SA 23

SAMPLE FROM #2 SA 26

DEPTH 55'6" - 56'6"

DEPTH 70' - 71'6"



NO. OF BLOWS (LOG SCALE)

Fig. 5 e

e. m. peto associates ltd.

Toronto 19, Ontario

LIQUID LIMIT TEST

FLOW LINE CHARTS

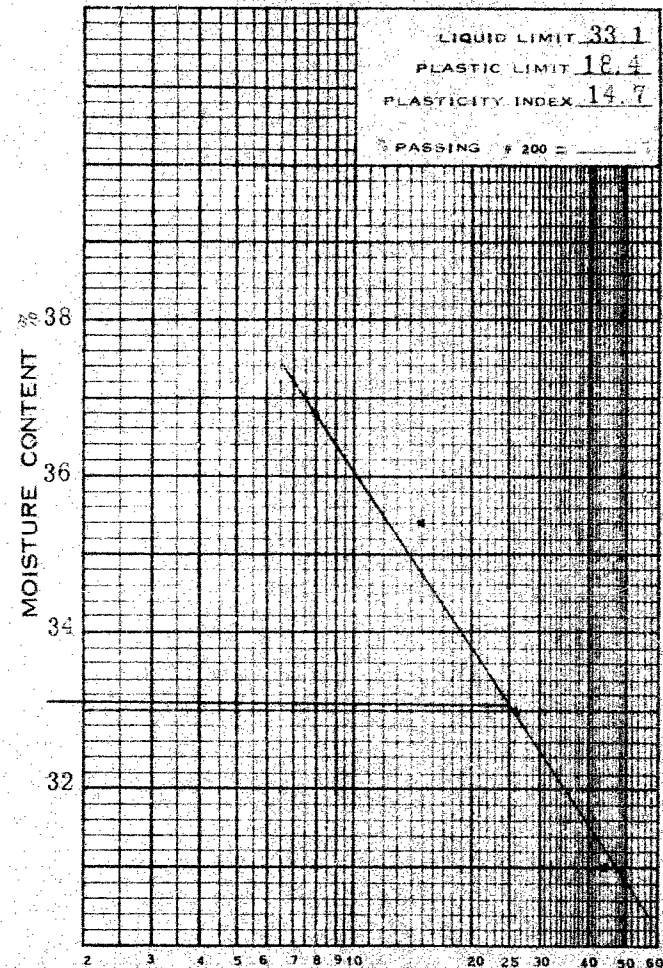
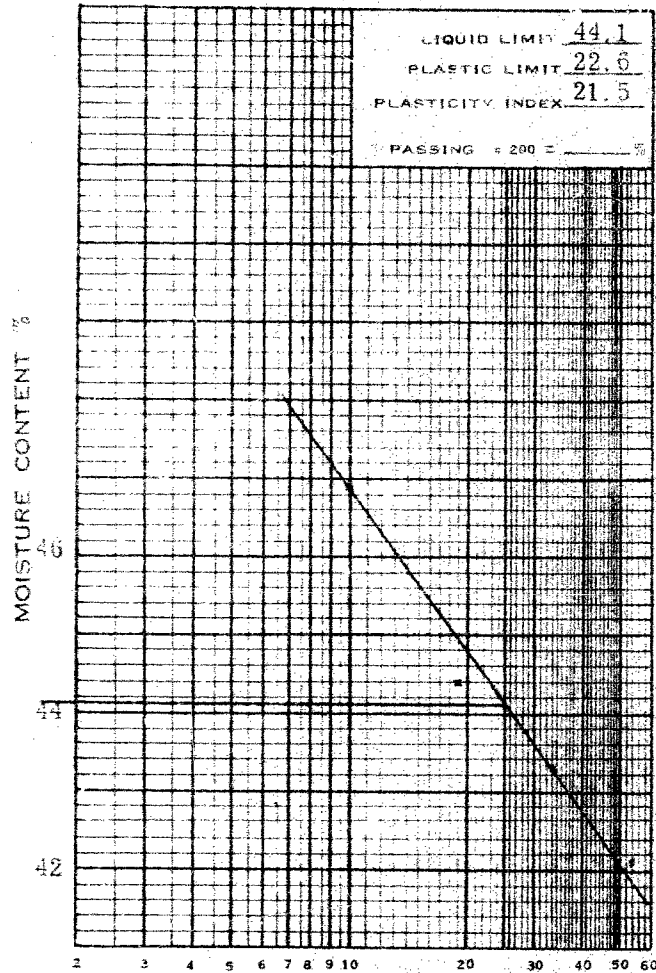
JOB No. 61170 PROJECT Bear Creek Bridge

SAMPLE FROM B.H. #3 SA 30

DEPTH 85' 6" - 86' 0"

SAMPLE FROM #5 SA 25

DEPTH 85' - 86' 6"



NO. OF BLOWS (LOG SCALE)

Fig. 57

e. m. peto associates ltd.

Toronto 19, Ontario

LIQUID LIMIT TEST

FLOW LINE CHARTS

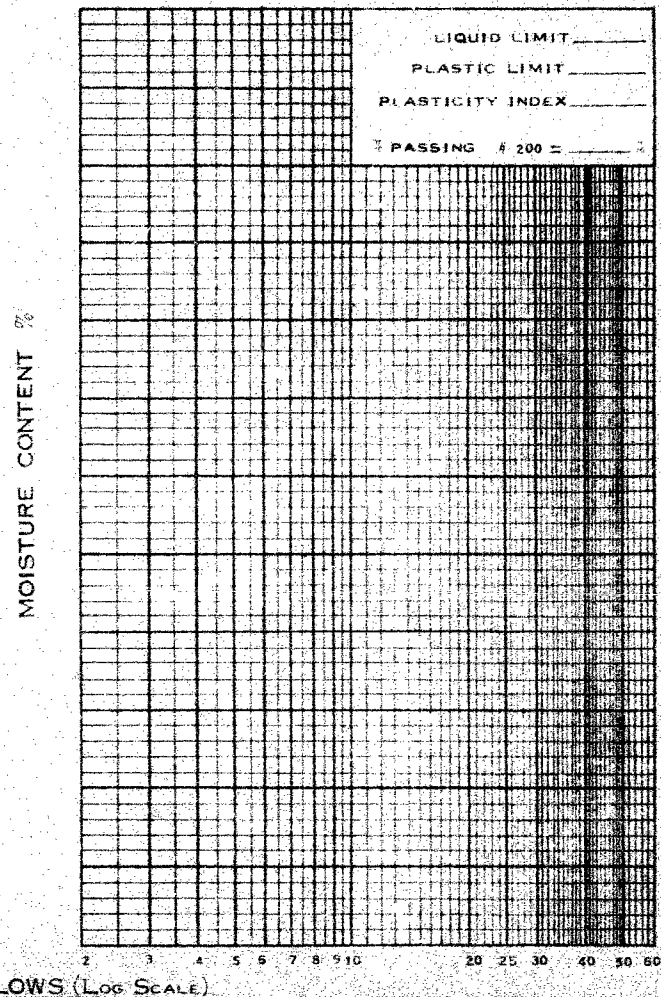
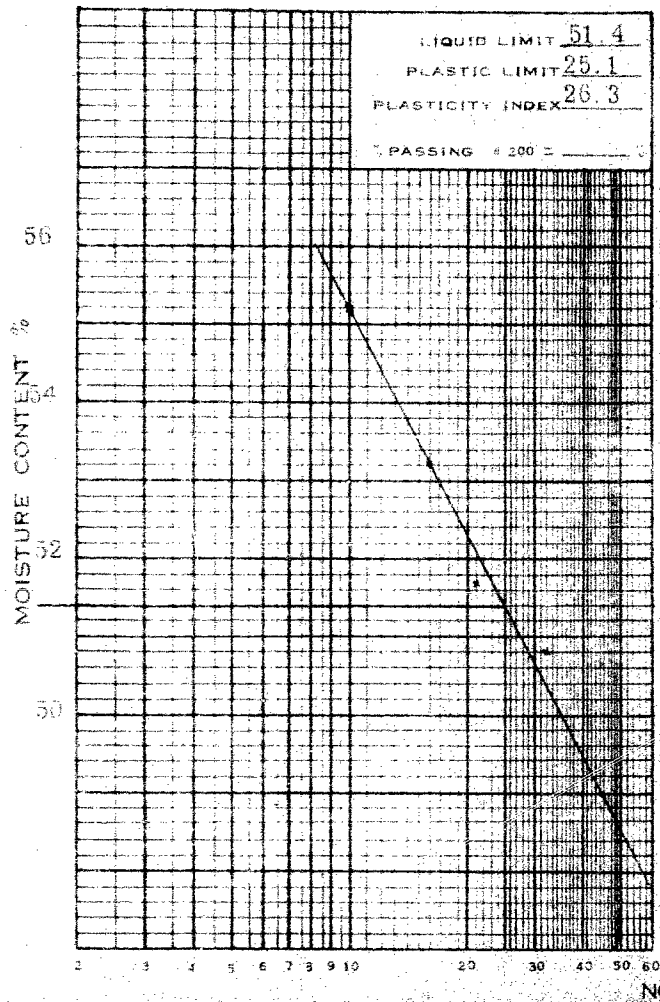
JOB NO. 61170 PROJECT Bear Creek Bridge

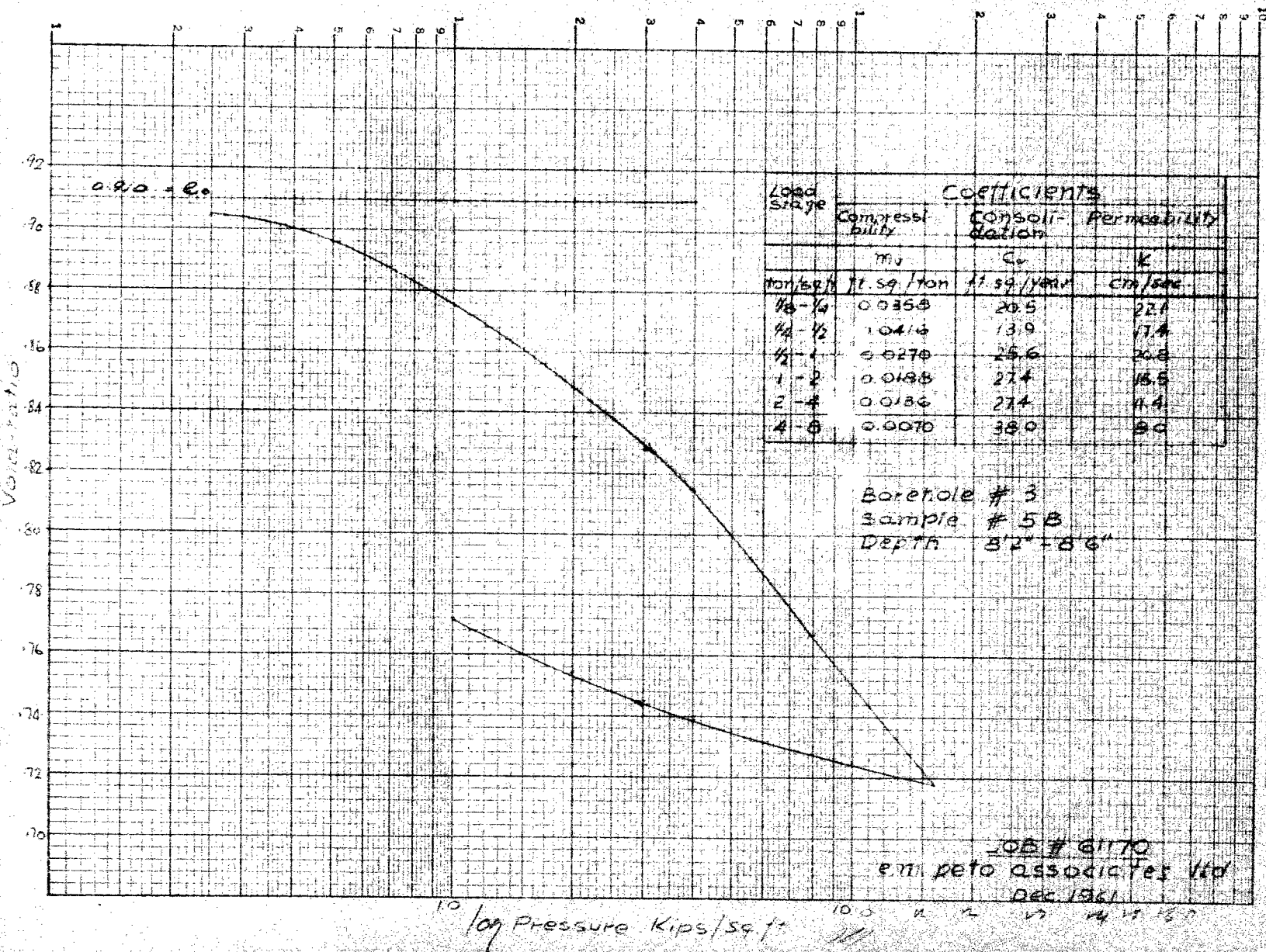
SAMPLE FROM 42 SA 31

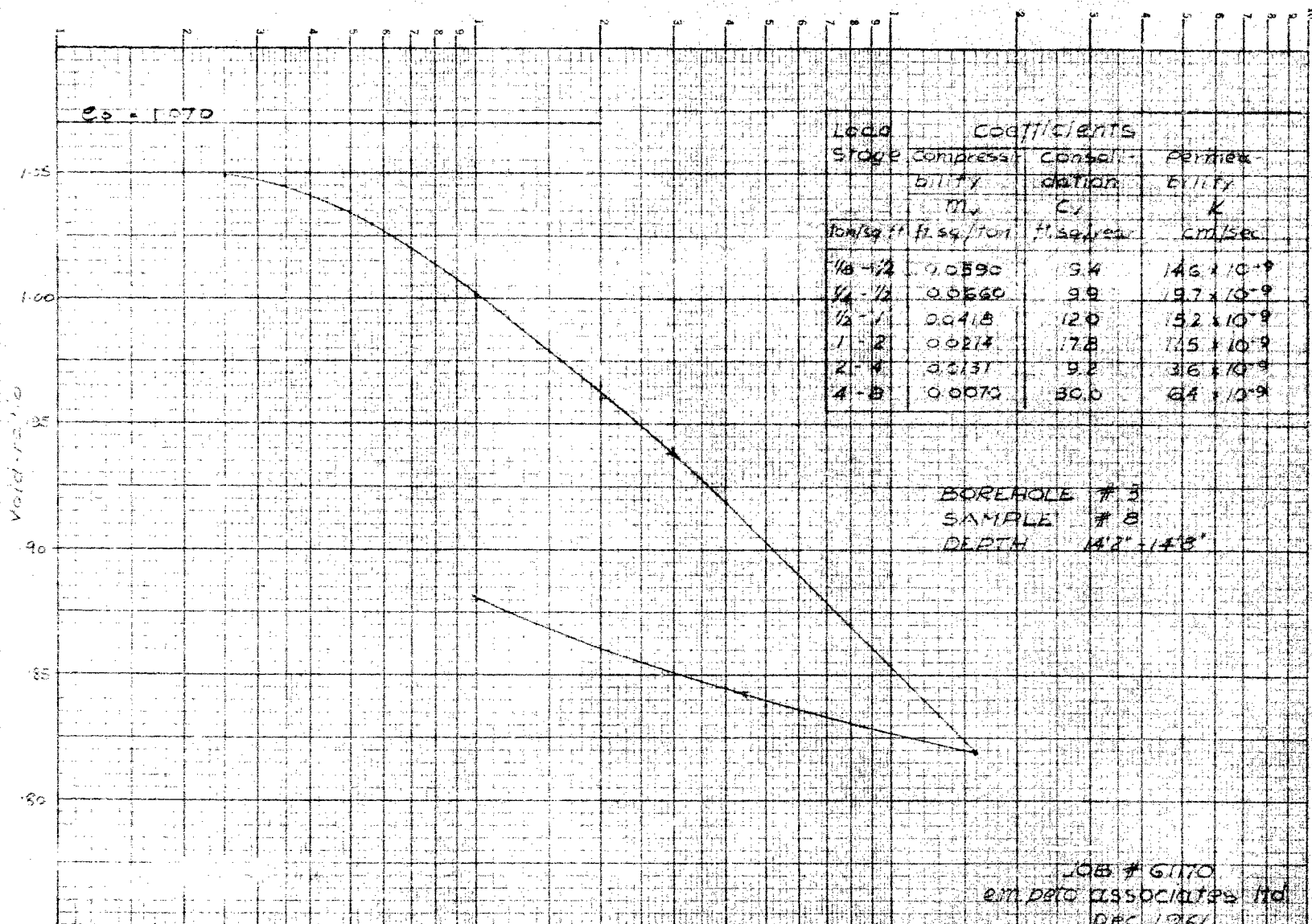
DEPTH 95'-98'6"

SAMPLE FROM _____

DEPTH _____







Pressure kips/sq. ft.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

FIG 69

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO





BOREHOLE LOG

Job Name Bear Creek Bridge (C-7-0)Job No. 61170Borehole No. 6Client County of LambtonCasing 4"Boring Date October 20th, 1961Elevation ArbitraryCompiled By R. K.Checked By S. B.

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	Unit Weightpcf	WATER LEVELS & REMARKS
			98.9 0'0"						
Organic topsoil	Black				1	C.S.			
Clayey silt or very silty clay	Brown	Firm to Stiff			2	S.S.	14	26.1	
Very silty clay					3	C.S.			
Ditto	Ditto	Firm			4	S.S.	9	19.5	
			5'0"						
Silty clay, layers of fine silty sand	Ditto	Soft			5	S.S. L.		19.9	
					6	S.S.	4	20.3	
									W.T. approx. 8'6"
			10'0"						
Fine to medium sand with organic silt	Black	Loose			7	3"S. L.	7	20.4	
			14'0"						
			15'0"						
Silty clay occasional pebbles	Grey	Soft to Firm			8	3"S. L.			No free water below 14'
						S.S.	12	27.1	
			20'0"						
As above	As above	As above			9	3"S. L.			
			21'6"			S.S.	14	29.1	

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

TEST HOLE TERMINATED AT 21'6"

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Bear Creek Bridge (C-7-6)Job No. 61170Borehole No. 5Client County of LambtonCasing 4" & BXBoring Date October 28, 30, 31 November 1,
2, and 7, 1961Elevation ArbitraryCompiled By R. K.Checked By S. B.

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.	Original Moisture Content	WATER LEVELS & REMARKS
			88.8						
Organic topsoil	Black		0'0"		1	C.S.			
Clayey silt	Brown	Firm	0'8"		2	S.S.	13	19.5	
Clayey silt, some shells	Brown	Firm	5'0"		3	S.S.	14	18.3	
Very silty clay	Brown	Firm			4	S.S.	9	23.9	
			10'0"		5	2" S. L. Tapped			W. T. approx. 10'0"
Silty clay, some organic inclusions	Brown	Soft to firm			6				
	Grey		14'0"		7	S.S.	9	23.8	
			14'8"						
Fine to coarse sand with organic silt	Black	Very loose to Loose			8	S.S.	5	17.2	
			18'0"						
Silty clay, occasional small pebbles.	Grey	Soft	20'0"		9	S.S.	5	26.9	
			25'0"						
Silty clay	Grey	Soft			10	2" S. L. Tapped			
					11				
					12	S.S.	5	31.3	
Silty clay	Grey	Soft	30'0"		13	S.S.	5	30.8	
			35'0"						
Ditto	Ditto				14	2" S. L. Tapped			
			40'0"						
Silty clay, some grits	Grey	Firm			15	2" S. L.			
					16				
					17	S.S.	12	25.7	
Silty clay with pebbles	Grey	Firm	45'0"		18	S.S.	19	23.1	Flaky structure commens
			50'0"						

$e_0 = 1.120$

Load 57 kg	Coefficients		
	Compressi- bility mv	Consoli- dation C_v	Permea- bility k
	in sq/in	in sq/year	cm/sec
1-1	0.064	61.0	12.5×10^{-9}
1-2	0.023	47.9	12.8×10^{-9}
2-4	0.014	29.5	10.6×10^{-9}
4-8	0.009	25.0	5.2×10^{-9}

Borehole # 4
Sample # 13
Depth 29'2" - 29'6"
Grey silty clay

JOSEPH SITO
engineer
Dec 1961

Pressure $kil = 1.22 f$

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

FIG. 6C

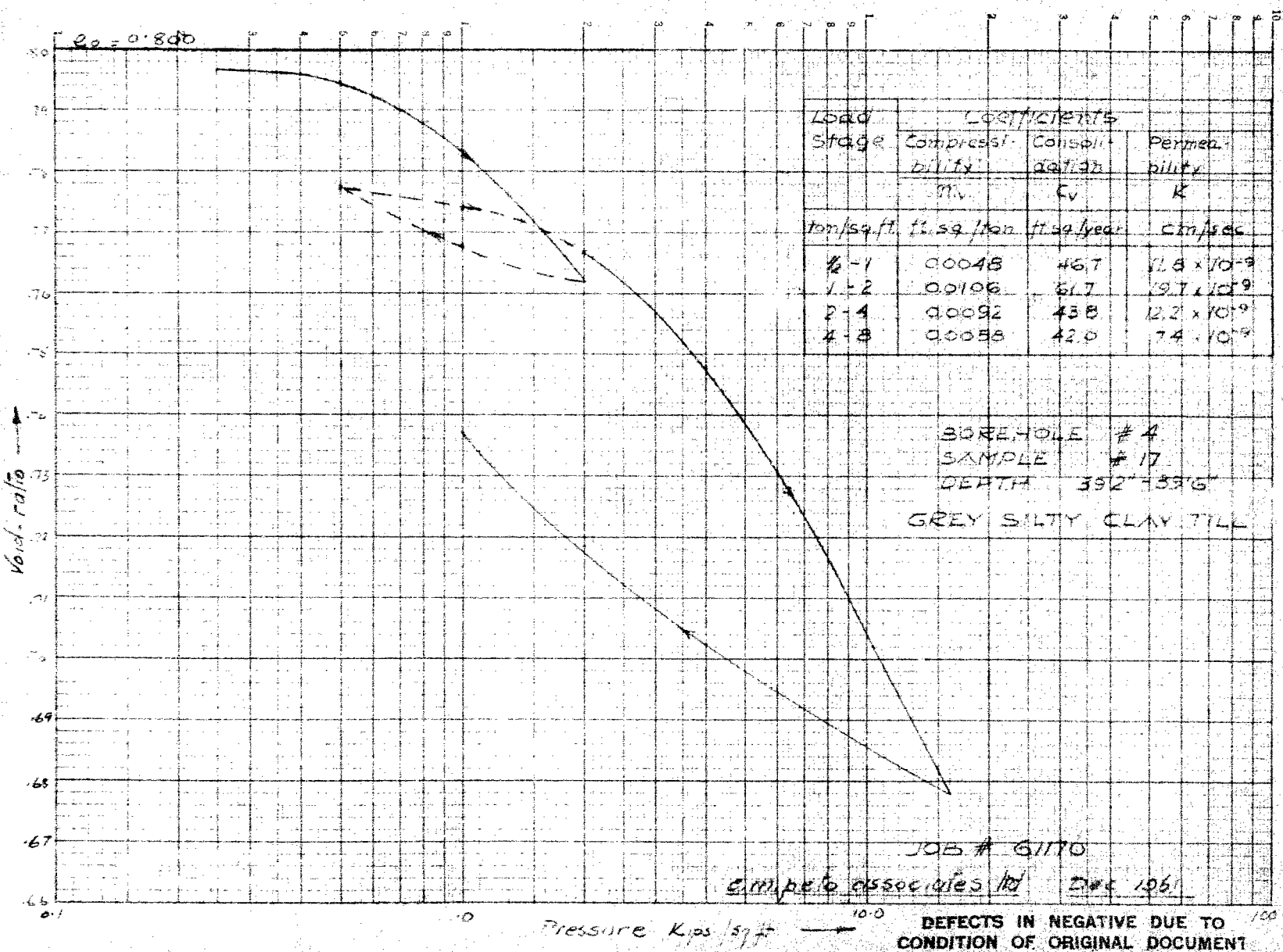


FIG. 5d

DEFECTS IN NEGATIVE DUE TO
 CONDITION OF ORIGINAL DOCUMENT





e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Bear Creek Bridge (C-7-6) Job No. 61170 Borehole No. 2
 Client County of Lambton Casing BX Boring Date October 26-29th, 1961
 Elevation Arbitrary Compiled By R. K. Checked By S. B.

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	COLOR	Density of Compaction	Depth Elevation	Legend	Sample No and Condition	Sample Type	No. of Blows per Ft	Water Level in Casing	WATER LEVELS & REMARKS
			93.9 0'0"						
Silty and clayey loam	Brown				1	C.S.			
Partly sandy silt, partly clayey silt.	Brown	Firm			2	S.S.	12	18.0	
			5'0"						
Silty clay with plant roots and a medium sand seam	Mottled brown	Soft			3	S.S.	5	19.5	Sand seam
Silty clay with pebbles	Black		7'3"		4	S.S.	4	42.8	W. T. at 6'9"
Partly v. soft organic clay with pieces of wood, partly fine gravel with clay	Dark grey to Black	Very soft	9'10"		5A 5B	2"S. L. Pushed			
			12'6"		6	S.S.	5		
Silty clay	Grey	Very soft			7	S.S.	5	31.0	
			15'0"						
Silty clay	Grey	Soft			8A 8B 8C 9	2"S. L. Pushed S.S.	7	21.0	
			20'0"						
Silty clay	Grey	Soft			10A 10B 10C 11	2"S. L. Pushed S.S.	6	38.0	
			25'0"						Grits start at 23'6"
Silty clay	Grey	Soft			12A 12B 13	2"S. L. Pushed S.S.	5	23.7	
			30'0"						
Silty clay with grits and some sand	Grey	Soft to firm			14A 14B 15	2"S. L. Pushed S.S.	13	22.8	
			35'0"						
Silty clay with pebbles	Grey	Soft to firm			16A 16B 17	2"S. L. Pushed S.S.	19	22.5	Flaked structure of clay Pebbles up to 3/4"
			40'0"						
As above	Grey	As above			18A 18B 19	2"S. L. Tapped S.S.	22	21.7	
			45'9"						Softer layer 42' to 54'
As above	Grey	Soft to firm			20A 20B 21A 21B 22	2"S. L. S.S.	17	23.5	
			50'0"						
As above	Grey	As above			23A 23B 24	2"S. L. Tapped S.S.	13		

As above	Grey	Soft to firm	50'0"	20	S.S.	17	23.5	
As above	Grey	As above	50'0"	21A 21B	S.S. L S.S.	Tapped 13		
As above	Grey	Firm to stiff	55'0"	22 23	2"S. L S.S.	Tapped 41	20.5	Stiffening at 54 ft.
As above	Grey	Firm to stiff	60'0"	24	S.S.	25	20.3	
As above	Grey	As above	65'0"	25	S.S.	27	20.5	
As above	Grey	As above	70'0"	26	S.S.	23	19.7	Stones up to 1-1/2"
As above	Grey	As above	75'0"	27	S.S.	21	19.3	
As above	Grey	As above	80'0"	28	S.S.	22	21.1	
As above	Grey	Firm	85'0"	29	S.S.	18	21.8	Softer
As above	Grey	Firm	90'0"	30	S.S.	13	30.1	
As above	Grey	Soft	95'0"	31	S.S.	9	35.9	Less grits & pebbles below 93 ft. Very pronounced, shiny flake structure
Fine to coarse silty sand with some fine gravel (broken up shale)	Black	Compact	100'0"	32	S.S.	20	11.8	Hit natural gas at 100 ft.
Mainly fine, very silty sand with pebbles	Black	Extremely dense	105'0"	33	S.S.	98	9.3	
Shale	Black	Solid	107'0"		R.C. R.C. R.C. R.C.			Refusal at 107'0" Diamond drilled 107'-107'7" D.D. 107'7"-108'4" 4' recovery 4' 8" recovery D.D. 108'4"-109'2" 2'-7" recovery D.D. 109'2"-112'11" 3'9" recovery (100%)
			112'11"					
TEST HOLE TERMINATED AT 112'11"								
Casing at 107', hole at 112'11", water overflowed casing to 3'7" above G. L.								

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

-13-

-15-





e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Bear Creek Bridge C-7-6) Job No. 61170 Borehole No. 3
 Client County of Lambton Casing 4" Boring Date Nov. 1
 Elevation Arbitrary Compiled By R. K. Check'd By S. B.

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.	Unit Weight (pcf)	WATER LEVELS & REMARKS
			86.9						
			0'0"						
Silty & clayey topsoil with roots	Dark grey		1'0"		1	C.S.			
Silty clay loam with plant roots	Dark brown		2'6"		2	C.S.			
Medium to coarse sand with organic silt and pieces of wood	Black	Very loose			3	S.S.	2	28.5	
Organic clayey sand	Black	Very soft	5'2"		4	S.S.	2	34.3	
Silty clay and 1/4" stone	Grey				5A	3" S.L. Tapped		33.0	
			10'0"		5B				
Silty clay with occasional grits	Grey	Soft			6	S.S.	5		Grits up to 1/8"
					7A	2" S.L. Pushed			
					7B				
			15'0"		8A	3" S.L. Tapped		28.4	
					8B				
As above	Grey	Soft			9	S.S.	4	42.6	Very soft after remoulding
			20'0"		10A	2" S.L. Pushed			
					10B				
As above	Grey	Soft			11	S.S.	5	34.7	
			25'0"		12	3" S.L. Tapped			Darker, more pebbles.
Silty clay with pebbles	Grey	Soft to firm			13	S.S.	40	25.9	Flaky Structure.
			30'0"		14	2" S.L. Tapped			
Silty clay with pebbles	Grey	Firm			15	S.S.	16	23.3	
			35'0"		16	3" S.L. Tapped			
As above	Grey	Firm			17	S.S.	11	23.8	Much grits and pebbles.
			40'0"		18	2" S.L. Tapped			
As above	Grey	Firm			19	S.S.	14	26.2	
			45'0"		20	3" S.L. Tapped			
As above	Grey	Firm			21	S.S.	22	20.8	
			50'0"		22	2" S.L. Tapped			
As above	Grey	Firm			23	S.S.	26	22.4	

			25'0"	12	3"S. L. Tapped			Darker, more pebbles.
Silty clay with pebbles	Grey	Soft to firm		13	S.S. 10	25.9		Flaky Structure.
			30'0"	14	2"S. L. Tapped			
Silty clay with pebbles	Grey	Firm		15	S.S. 16	23.3		
			35'0"	16	3"S. L. Tapped			
As above	Grey	Firm		17	S.S. 11	23.8		Much grits and pebbles.
			40'0"	18	2"S. L. Tapped			
As above	Grey	Firm		19	S.S. 14	26.2		
			45'0"	20	3"S. L. Tapped			
As above	Grey	Firm		21	S.S. 22	20.3		
			50'0"	22	2"S. L. Tapped			
As above	Grey	Firm		23	S.S. 26	22.4		
			55'0"	24	S.S. 19	21.0		
As above	Grey	Firm		25	S.S. 21	21.0		
			60'0"	26	S.S. 18	22.6		
As above	Grey	Firm		27	S.S. 17	22.3		
			75'0"	28	S.S. 12	24.3		Stones up to 1-1/2"
As above	Grey	Firm		29	S.S. 14	20.9		
			80'0"	30	S.S. 11	31.3		
As above	Grey	Soft		31	S.S. 15	14.0		
Silty clay	Grey	Soft	90'6"	32	S.S. 215/3"			Refusal at 91'9"
Sandy and clayey silt and soft	Dark grey to black		92'0"		100/0"			Total refusal at 92'0": 100
Shale								blows, not moving

TEST HOLE TERMINATED AT 92 ft.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT





e. m. beto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Bear Creek Bridge (C-7-b) Job No. 61170 Borehole No. 4
Client County of Lambton Casing BX Boring Date November 8 & 9th, 1961
Elevation Arbitrary Compiled By R. K. Checked By S. B.

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 COPE

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.	Water Level in Casing	Water Levels & Remarks
			98.6 0'0"						
Organic silty clay	Dark brown		1'2"		1	C.S.			
					2	C.S.			
Clayey silt, or very silty clay	Brown	Firm			3	S.S.	13	28.0	
			5'0"						
Very silty clay, some organic matter	Brown	Firm			4	S.S.	12	27.4	
(occasional very thin peat layers & decayed shells)					5	S.S.	13	27.6	
As above	Brown		10'0"						
As above	Brown	Firm			6	S.S.	12		Softer at 10'2"
As above	Brown	Soft to firm	13'6"		7	S.S.	7	27.3	Steady W. L. at 11'5"
As above	Grey		15'0"		8A	3"S. L.	Tapped		change to soft silty clay
As above	Grey	Very soft	16'2"		9	S.S.	4	31.9	
Clayey sand with a high organic content	Black	Very soft	17'6"						
			20'0"			3"S. L.			
Silty clay	Grey	Very soft			10	S.S.	4	33.8	
						3"S. L.			
			25'0"			2"S. L. Pushed			
Silty clay	Grey	Soft to firm			11A				
					12	S.S.	9	40.2	
						3"S. L. Tapped			
As above	Grey	Very soft to soft	30'0"		13A				
					14	S.S.	3	35.3	
			35'0"			3"S. L. Tapped			
As above	Grey	Soft			15A				
					16	S.S.	4	34.4	
			40'0"			3"S. L. Tapped			Grits and pebbles begin at 37'3"
Silty clay with pebbles	Grey	Soft to firm			17A				
					18	S.S.	12	26.5	Flaky structure
						2"S. L. Tapped			Stones up to 1 inch
As above	Grey	Firm	45'0"		19				
					20	S.S.	15	22.8	
			50'0"			2"S. L. Tapped			
As above	Grey	Firm			21				
					22	S.S.	16	27.0 22.6	
			55'0"			2"S. L.			Stone pushed ahead of S. L.
					23	S.S.	22	21.9	No sample 1" stone
			60'0"						
Silty clay with numerous grits and pebbles	Grey	Soft			24	S.S.	20	16.5	
			65'0"						
Silty clay with grits and pebbles	Grey	Firm			25	S.S.	19	20.4	
			70'0"						
As above	Grey	Soft	71'0"		26	S.S.	24	21.1	Stones up to 1 inch.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

BOREHOLE TERMINATED AT 71'0"

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 6140 - 61170

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

January 26th, 1962.

Tougham and Case Ltd.,
Consulting Civil Engineers,
P. O. Box 386,
151 Thames Street,
Chatham, Ontario.

Attention: Mr. H. B. Tougham.

Dear sir,

Wilkesport Bridge - 6140
Bear Creek Bridge - 61170

We refer to the two queries addressed to Mr. H. B. Litchins on his recent visit to your office.

The first was in respect of the varying distribution of pressure below the 16ft. (approx.) wide cantilevered abutment wall footings for the Wilkesport Bridge.

We have checked the local shear condition over the outer 5 feet width strip, where the contact pressure will be highest on these footings and compared this with the overall bearing capacity for the footing. As a result of this study we can confirm that at elevation 560.0 the net allowable bearing capacity is approximately 1.1 tons per sq. foot both for local stability under the 1/3 width of the abutment and for overall stability.

The net maximum concentrated edge pressure can go up to 1.23 tons.

The second query was in respect of the possibility of raising the footings at Bear Creek Bridge above elevation 560.0.

PAGE TWO

If we assume a footing width of 16 feet and a length of 40 feet the values given for the 50 x 12 feet mat on pages 32 and 37 will be applicable for all practical purposes, and the settlement conditions for the same size mat, shown graphically on pages 34 and 39 of report 61170 will also apply within close limits. Therefore the footings may be raised provided the net increase in bearing value is decreased in order to comply with these values.

Generally the effect of raising the footing elevations will only have a slight effect on the net allowable increase in pressure because the average unraised shear strength does not decrease to any great extent above elevation 560.0.

Raising the footings however will increase the total settlement as shown on Graphs I and II. Summarizing the position it would appear that footings on the south side may be raised to elevation 566.0 and on the north side to elevation 562.0, provided the increased settlement can be accommodated and that scour does not present overriding considerations.

Turning now to the verification of soil conditions on the line of the existing bridge at Wilkesport, we have been in touch with the Principal Foundation Engineer at the D.P.O. regarding this. As a result of our discussion it has been agreed that initially a boring with a dynamic probe test will be put down at each bank and subject to the conditions and stratification at these locations confirming similarity with the conditions on the line of the original investigation no further work will be required.

In view of this we proposed driving two borings, one on each bank, to a depth of 40 feet approximate elevation 545.0 together with a dynamic penetration test at each boring, to this end we are arranging to set out the borings on this site to-day with the object of putting a crew on the site on Monday the 29th January.

Lastly, in connection with your request for wall pressures on the abutment walls for the Wilkesport Bridge, it would be of assistance to us in arriving at these if we could be supplied with a sketch showing the existing profile and final proposals at these walls, particularly such items as final grade, depth of removal of poor material, footing elevation and the approximate profile of the bulk fill behind the bridge walls before the granular fill is placed.

PAGE

THREE

We trust our replies to your queries on these two sites have clarified the conditions for you and that you are in agreement with our action in respect to the Wilkesport Bridge.

Yours very truly,

E. M. PETO ASSOCIATES LTD.



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

CFF/ap

