

64-F-257M

LONG'S CREEK  
BRIDGE

DAWN TWP

BN 1876

**E. M. PETO ASSOCIATES LIMITED**

Job Number 6423

1287 Caledonia Road,  
Toronto 19, Ontario,  
789-1126,  
26th March, 1964.

The County of Lambton,  
c/o Todgham & Case Limited,  
Consulting Civil Engineers,  
151 Thames Street,  
Chatham, Ontario.

64-F-257M

Attention: Mr. H. H. Todgham, P. Eng.

Gentlemen:

Re: Subsoil Investigation,  
Long's Creek Bridge,  
Lot 21, Concession Road 12-13,  
Township of Dawn,  
County of Lambton.

We have pleasure in submitting four copies of our Report No. 6423 on the above subsoil investigation. One additional copy has been forwarded directly to Mr. O. van Dears, Lambton County Engineer.

Our analysis has indicated that spread footing type foundation, placed near elevation 78.0 should prove satisfactory, although some differential settlement is to be anticipated between the two abutments. The conditions for an alternative pile foundation are very favourable at the site, but in view of the design of the bridge which we understand will not be sensitive to some settlement, the presumably more expensive piles will probably be unnecessary.

No difficulties with construction of the bridge and embankments or with excavation of foundations and new creek channel are envisaged.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,

A handwritten signature in dark ink, appearing to read 'E. M. Peto', with a long, sweeping horizontal stroke extending to the right.

E. M. Peto, P. Eng.

RK/vm

**THE COUNTY OF LAMBTON  
C/O TODGHAM & CASE LIMITED  
CONSULTING CIVIL ENGINEERS**

**SUBSOIL INVESTIGATION  
LONG'S CREEK BRIDGE  
LOT 21, CONCESSION ROAD 12-13  
TOWNSHIP OF DAWN  
COUNTY OF LAMBTON**

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

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TABLE A: Atterberg Limits

TABLE B: Unconfined Compression Test Results

TABLE C: Modulus of Elasticity Test Results

FIGURE 1 - Grainsize distribution curves

FIGURES 2a, b - Consolidation test results

FIGURES 3a, b - Triaxial - pore pressure test results

BOREHOLE LOGS (2)

DRAWING: Key plan, site plan, subsoil profile,  
geotechnical properties diagram.

## **A. INTRODUCTION**

The work described in this report was authorized by Mr. H. H. Todgham, by letter dated 29th January, 1964.

In connection with the realignment of Long's Creek, a new bridge is to be constructed to replace an existing structure, which is inadequate for the present day traffic conditions. A subsoil investigation was required for the design of foundations.

The new bridge will have a span of 35 ft., and the deck is to be simply supported on abutments, which it was hoped to place on spread footings located near elevation 78.

The new road grade is to be raised by a few feet, the highest fill being about 7.5 ft. at some 100 ft. north of the existing bridge.

Additional data concerning the project was supplied by the Consulting Engineer on Drawing No. 64101-1 and in letters dated January 29th and March 5th, 1964.

**B. GENERAL INFORMATION**

1. The site is located on a road which runs north-south between Concessions 12 and 13, in Lot 21, Township of Dawn, County of Lambton. The crossing is situated approximately 2.5 miles to the north-west of Florence, Ontario. Location of the site is indicated on the key plan, included on the enclosed drawing.

2. The terrain surrounding the site is almost flat, and Long's Creek flows near the southern edge of a shallow valley, the floor of which extends to approximately 10 feet below the general level of the area. While the existing road grade falls by 3 or 4 feet to the north of the bridge, it rises to the level of surrounding terrain immediately to the south of the existing structure.

The proposed new bridge is to be constructed to the south of the present structure, in such a way that the northern abutment will be located in the position of the present channel of Long's Creek.

3. Two testholes were put down at the site, in the positions indicated on the enclosed site plan. Both testholes were terminated in the shale bedrock, at the respective depths 43.8 and 46.0 feet, after proving the bedrock by diamond drilling.

**B. GENERAL INFORMATION - Cont'd.**

The testholes were set out by our field engineer, who also measured ground elevations, which were referred to Client's temporary bench mark of assumed elevation 93.53, in the form of a nail in hydro pole located approximately 15 feet to the south-west of the southern end of the western wing wall of the existing bridge. The elevations are entered on the drawing and on the borehole logs.

4. The field work was performed in second part of February, 1964, by our field unit No. 3, employing a standard skid-mounted diamond drilling rig. Our standard drilling and sampling procedures were followed.

5. Laboratory testing of soil samples consisted of water content measurements, Atterberg Limit and grain size distribution tests for soil classification purposes, compression tests for determination of shear strength and elasticity, and oedometer consolidation tests and consolidated triaxial compression tests with pore water pressure measurements for the determination of consolidation characteristics of the clay.



### C. SUBSOIL CONDITIONS

Details of the soil conditions encountered in the testholes are described on the borehole logs, while a simplified profile, in the form of a section through the testholes, is included on the drawing and shows the inferred levels of contact between the various strata. The main geotechnical properties of subsoil are plotted against elevation on the drawing, next to the subsoil profile.

Shale bedrock was encountered at a depth of 40.2 and 38.5 feet in boreholes 1 and 2, the elevation of bedrock surface being 50.0 and 52.4 respectively. The shale was proved by diamond drilling and found to be solid and sound. It provides an excellent basis for end-bearing piles.

The shale was overlain by a 3 to 4 ft. thick sheet of dense silty and sandy till; the lower layers of the till contained much broken shale.

The main overburden at the site, extending to the top of the dense till at an average depth of 36 ft. below the grade, is a grey clay deposit, of firm consistency, which possesses a firm to stiff, desiccated brown crust extending to a depth 7.5 to 14 ft. below the existing grade. The clay has a layered structure

C. SUBSOIL CONDITIONS - Cont'd.

and contains pebbles, as well as occasional silt seams. It is believed that this deposit originated in a post-glacial lake which covered this area after the retreat of the Wisconsin glacier. The material is slightly overconsolidated, probably mainly by desiccation.

On the north side of the bridge, testhole 2 disclosed that the surficial layers of subsoil, up to a depth of 8 ft. consist mostly of a clayey fill with silt and some organic matter. In contrast, testhole 1 disclosed that the desiccated crust of the clay stratum extends almost to the ground level.

Because the clay stratum may be required to support bridge foundations in the form of spread footings, geotechnical properties of this material were studied in some detail by means of laboratory testing. The variation with depth of standard penetration resistance, water content, Atterberg Limits and undrained shear strength is plotted on the Geotechnical Properties diagram, included on the drawing. Compressibility and pore pressure characteristics of the clay are represented on Figures 2 and 3, while typical grain size distribution is shown on Figure 1.

C. SUBSOIL CONDITIONS - Cont'd.

It may be noted that the undrained shear strength was somewhat higher in testhole 2 than in testhole 1. However, the lower strength in testhole 1 was not reflected by standard penetration test results nor by water contents, which were similar in both borings. There is a possibility that the lower strength results in testhole 2 were caused by sampling disturbance due to a different method of sampling (mostly 3 in. dia. liners compared to 2 in. dia. liners in testhole 1). However, for calculation of allowable bearing capacity of foundations, the average value of 900 lb/sq. ft. , (corresponding to results of testhole 2) was employed for the northern abutment, and a value of 1200 lb/sq. ft. for the southern abutment.

**D. WATER CONDITIONS**

Because of the low permeability of the subsoil above elevation 54, only very slow seepage was observed in testholes. However, when the broken shale layers immediately above the bed-rock were reached, ground water rose rapidly in testhole casing to elevation 84.5, or to approximately 6 ft. below the existing grade.

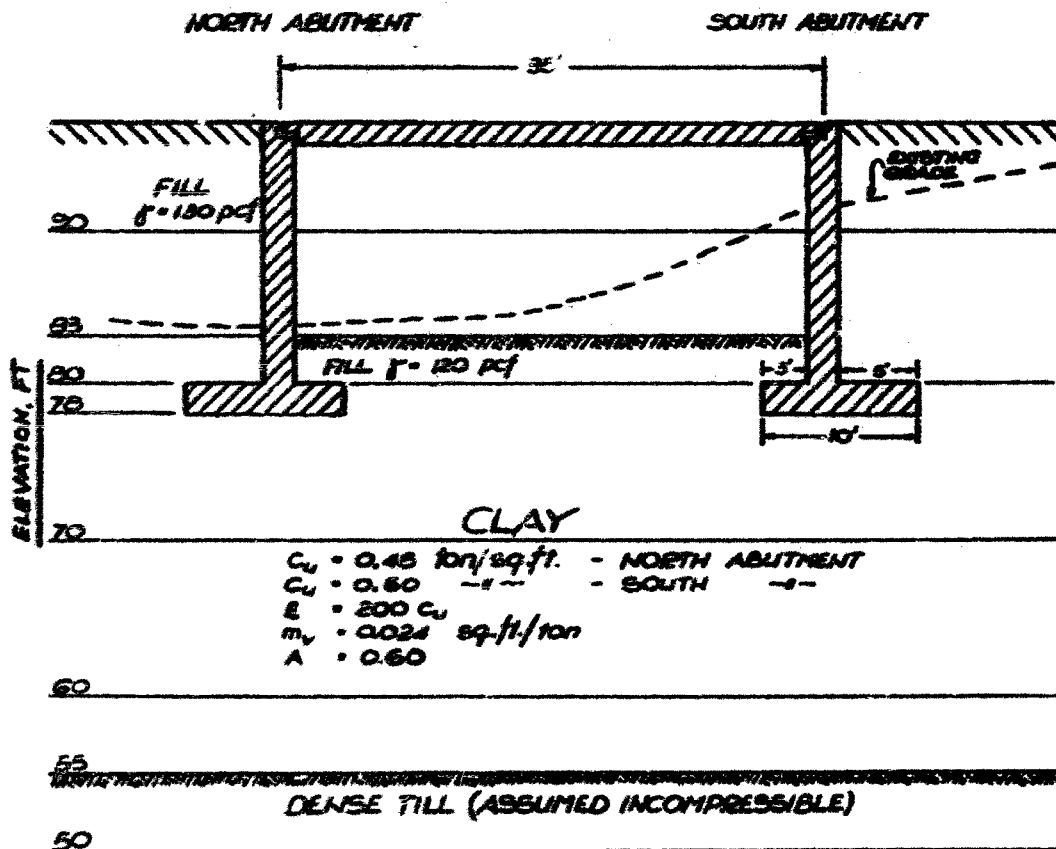
It is anticipated that only minor seepage of ground water will occur into excavations through the surficial, silty layers and that the inflow from Long's Creek has been cut off. Provided that water does not find an outlet to the excavation floor through any holes put down to the pervious till, the dewatering of excavations should not present any problems.

## **E. CONCLUSIONS AND RECOMMENDATIONS**

### **1. Bridge Foundations - Spread Footings**

In our opinion, it is feasible to support the bridge on spread footings located at elevation 78.0, as is proposed by the Consulting Engineers, although the bearing capacity of the subsoil is not high and some differential settlement between the two abutments may take place.

According to the layout proposed by the Consulting Engineers, the southern abutment will be located over the slope of the shallow valley and only a minor raising of embankment will take place. On the other hand, the northern abutment is to be constructed in the position of the present creek channel, where the subsoil is not consolidated under the weight of any fill and, moreover, appeared to be softer in testhole 2 than in testhole 1. Because new fill will be placed adjacent to the northern abutment, the footing will settle not only under the weight of the structure but also under the stresses transmitted to the soil below the footing by the adjacent fill. For these reasons, some differential movement between the two abutments is expected; moreover, calculations have indicated that an increase in the width of the footing (to 15 ft.), will not result in a reduction of potential settlement. Reasonable lowering of the footing level will not decrease the settlement materially.



### SYMBOLS

- $c_u$  - UNDRAINED SHEAR STRENGTH
- $E$  - MODULUS OF ELASTICITY
- $m_v$  - COEFFICIENT OF VOLUME CHANGE
- $A$  - SKEMPTON'S PORE PRESSURE PARAMETER

CONFIGURATION AND SOIL PROPERTIES ASSUMED  
IN BEARING CAPACITY AND SETTLEMENT ANALYSES

**E. CONCLUSIONS AND RECOMMENDATIONS - Cont'd.**

The bearing capacity and theoretical settlements were calculated using the dimensions and loading details supplied by the Consulting Engineers in letter dated March 5th, 1964. Because the abutment will have a monolithic form, the front wall and the wing walls footing being continuous, it was assumed, for the purposes of settlement analysis, that the bridge loads will be spread over one half of the area of the wing wall footings, in addition to the front wall footing. Further, it was assumed that the footing will have a cantilever form, and for an overall width of 10 ft., the heel was assumed to be 5 ft. wide, and the toe 3 ft. wide. The assumed configuration and soil properties used in bearing capacity and settlement analyses are presented on page 9.

Results of the calculations were as follows:

**a) - North Abutment**

Estimated gross pressure transmitted by equivalent footing:	1.25 ton/sq. ft.
Allowable bearing capacity, for factor of safety of three against shear failure:	1.16 ton/sq. ft.
Factor of safety after scour, assuming that overburden has been removed to top of footing at elevation 80.0 and gross pressure is 1.25 ton/sq. ft. :	2.0

**E. CONCLUSIONS AND RECOMMENDATIONS - Cont'd.**

It is considered that the design is safe with respect to stability against shear failure.

The settlements at the centre of the abutment were calculated to be as follows:

	<u>Settlement, inches</u>	
	<u>Due to fill outside footi. g</u>	<u>Due to structure &amp; fill above footing</u>
Immediate ("elastic")	0	0.63
Long-term (consolidation)	0.42	1.51
Total (sum)	0.42	2.14

Adding the settlements due to both causes, the total final settlement of the centre of abutment is  $0.42 + 2.14 = 2.56$  in.

At least a part of the settlement due to the weight of the fill outside the footing would probably develop if the embankment fill was placed prior to construction of the bridge, thus partly precompressing the soil below the footing.



**E. CONCLUSIONS AND RECOMMENDATIONS - Cont'd.**

**b) - South Abutment**

Estimated gross pressure transmitted  
by equivalent footing: 1.25 ton/sq. ft.

Allowable bearing capacity, for factor  
of safety of three against shear failure: 1.45 ton/sq. ft.

Factor of safety after scour, assuming  
that overburden has been removed to  
top of footing at elevation 80.0: 3.0

The design is therefore safe with respect to stability  
against shear failure.

In settlement analysis, because of the small thickness  
of new fill outside this abutment, the settlement has not been sub-  
divided as for the case of the north abutment; the total settlement,  
due to bridge and fill, is theoretically as follows, for the centre of  
the south abutment:

	<u>Settlement, inches</u>
Immediate ("elastic")	0.20
Long-term (consolidation)	0.98
Total (sum)	1.18

The calculations have thus indicated that, theoretically,  
the differential settlement between the two abutments will be  
 $2.56 - 1.18 = 1.38$  in. In view of the simply-supported design of the  
bridge, the differential settlement can probably be considered as  
acceptable.

E. CONCLUSIONS AND RECOMMENDATIONS - Cont'd.

The above discussion assumes that the soil below elevation 78 in the position of the northern abutment will not be considerably inferior than in testhole 2, which was located some 30 ft. to the north of this abutment. Because of the possible variations in material immediately below the creek bed, in the position of the northern abutment, it will be necessary to inspect carefully the excavated footing formation grade, and to lower the footings should the examination indicate that any creek-deposited, soft materials extend to this level. It may be advisable to excavate firstly in the position of the south abutment, where the soil conditions are less doubtful, so that experience can be gained with the type of soil below footing level which should be expected below the northern abutment.

2. Bridge foundations - piles

As an alternative to spread footings, the bridge could be supported on end-bearing piles, resting on the shale bedrock, which was encountered at elevations 50.0 and 52.4 in testholes 1 and 2 respectively. Only minor variations from these levels are anticipated at the site, so that the length of piles can probably be predicted with high accuracy.

E. CONCLUSIONS AND RECOMMENDATIONS - Cont'd.

The bedrock cores recovered by diamond drilling were found to be solid and sound, and the shale incapable of supporting high pile loads, dependent only on manufacturer's specifications. Increased resistance to pile driving is to be anticipated near elevation 54 to 56, in the silt till stratum, which must not be mistaken for bedrock. However, care should be taken not to shatter the bedrock below pile tips by overdriving.

3. Embankments and cuts

The northern approach embankment to the existing bridge consisted mostly of a clayey fill containing much silt and some organic matter, including wood fragments. Because no major increase in grade level is anticipated, it is considered permissible to retain these materials in the reconstructed embankment. Although some minor settlements may occur, these should not be important in view of the secondary character of the highway.

The materials which will be recovered in excavations for footings and during creek channel realignment, judging from the two borings, will consist mainly of clay with seams of silt, of firm to stiff

E. CONCLUSIONS AND RECOMMENDATIONS- Cont'd.

consistency, and can be utilized in the construction of the new embankments, although any organic topsoil or pockets of obviously inferior material should be rejected. On account of the frost-susceptible character of the subsoil, a granular overlayer should be placed below pavements.

As far as could be judged from the testholes, no stability problem is envisaged with the embankments to the north of the new structure, where the maximum raising of existing grade is not to exceed 10 to 12 ft. However, any organic topsoil or soft surficial muck should be removed before placement of the new fill.

E. M. PETO ASSOCIATES LTD.,

Report prepared by:

*R. K. Kulesza*

R. Kulesza, P. Eng.

R. K. /vm

Job Number 6423

*C. F. Freeman*

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

26th March, 1964.

TABLE A

ATTERBERG LIMIT TESTS

B. H. / Sa. No.	Depth, ft	Liquid Limit W <sub>L</sub> %	Plastic Limit W <sub>P</sub> %	Plasticity Index I <sub>p</sub> %	In-situ Water content W %
1 / 13	21	39.4	19.0	20.4	26.7
1 / 21	31	42.5	22.4	20.1	34.4
2 / 8	13	44.2	20.3	23.9	28.7
2 / 19	27	38.2	19.0	19.2	36.0
2 / 20	27.5	41.5	22.2	19.3	35.5

TABLE B

UNCONFINED COMPRESSION TEST RESULTS

HOLE No.	Sample No.	Depth	M. C. %	Densities, p. c. f.		Void Ratio, e	% Strain at Failure	u/c Shear Strength, p. sf.	Remarks
				Wet	Dry				
1	7	12'6"-13'0"	26.7	127.0	100.3	0.74	20.0	3160	
1	8	13'0"-13'6"	25.4	129.0	102.8	0.71	9.0	2430	
1	10	17'0"-17'6"	20.4	136.5	113.0	0.57	16.0	1670	
1	19	27'6"-28'0"	34.9	117.0	87.0	0.99	15.0	1330	
1	20	28'0"-28'6"	34.4	116.5	86.6	1.00	15.0	1300	
1	23	32'6"-33'0"	26.3	124.0	98.0	0.77	8.0	1460	
1	24	33'0"-33'6"	31.2	122.0	93.0	0.87	10.0	1300	
2	6	12'0"-12'6"	26.6	128.5	101.3	0.74	20.0	1140	
2	17	23'0"-23'4"	24.9	128.0	102.5	0.69	18.0	920	
2	25	32'6"-33'0"	35.5	118.5	87.5	0.98	17.0	710	
2	14	22'0"-22'4"	27.6	126.2	99.0	0.75	20.0	810	
2	22	28'0"-28'4"	18.2	122.4	103.5	0.63	18.0	890	
2	24	32'0"-32'6"	26.6	119.2	94.2	0.81	20.0	760	
2	22	28'0"-28'4"	19.0	126.5	106.2	0.58	20.0	525	Remoulded
2	14	22'0"-22'4"	22.3	125.8	102.8	0.64	20.0	405	Remoulded
2	11	17'0"-17'4"	18.8	144.0	121.0	0.51	20.0	890	
1	11	17'6"-18'0"	19.3	134.3	112.5	0.52	11.0	1570	
1	12	18'0"-18'6"	20.5	133.2	110.5	0.55	10.0	1540	
1	15	22'6"-23'0"	28.8	125.0	97.5	0.78	10.5	1440	
1	16	23'0"-23'6"	31.5	120.0	91.4	0.85	9.5	1340	

TABLE C

UNDRAINED - TRIAXIAL COMPRESSION TESTS FOR  
DETERMINATION OF MODULUS OF LINEAR DEFORMATION  
(YOUNG'S MODULUS, E)

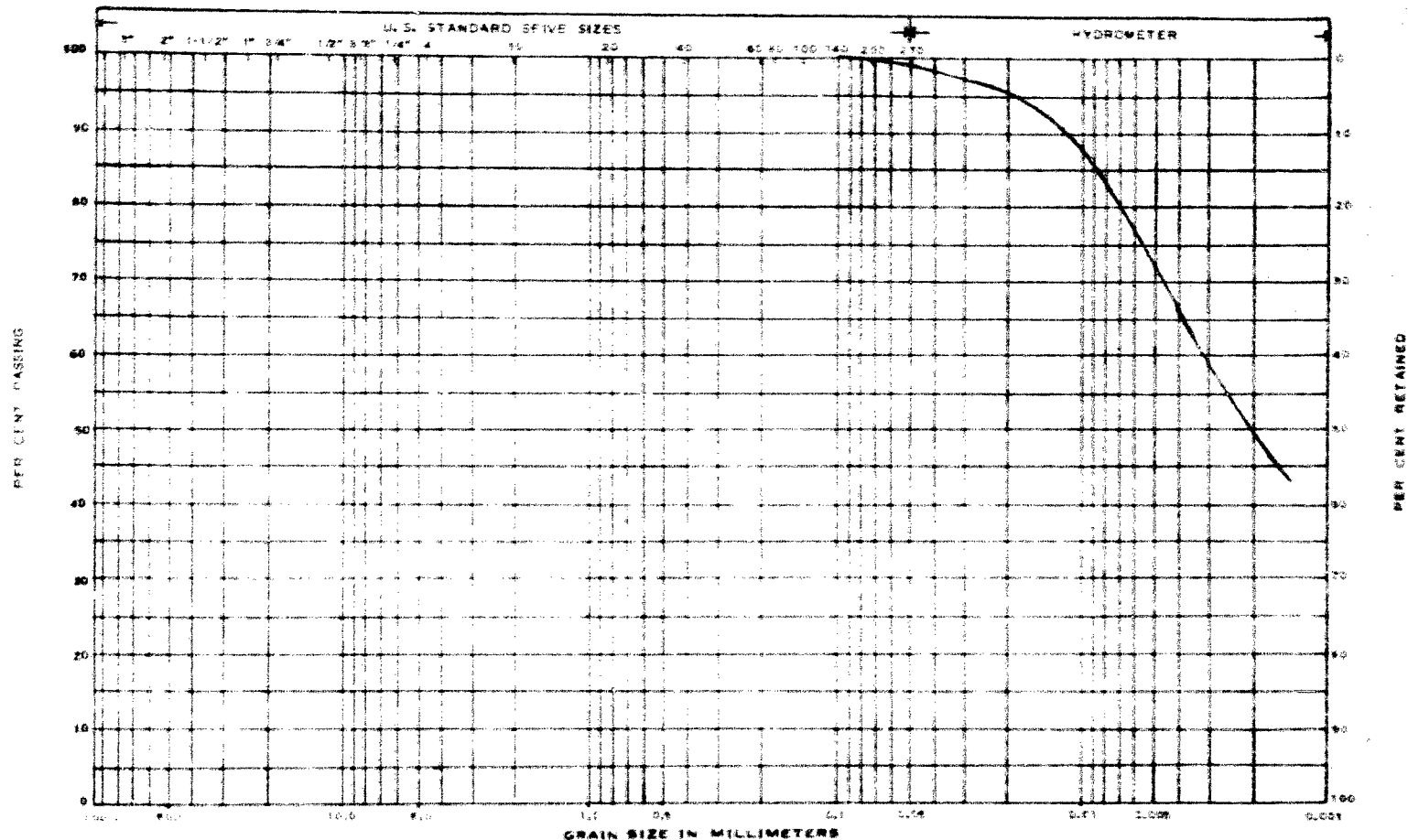
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EH/Sa. #	Depth ft	Water Content %	Bulk Density lb/cu. ft.	Modulus E ton/sq. ft.	Deviator Stress range ton/sq. ft.	Shear Strength Cu ton/sq. ft.	$\frac{E}{Cu}$
2 /	13	26.9	125.0	109	0.053 - 0.590	0.565	193
2 /	28	31.4	120.0	51.4	0.053 - 0.482	0.245	210

Note: The tests were performed at a rate of strain of 0.67 percent per min. and E was obtained from the average slope of stress-strain curve loops in repeated reloading and unloading cycles between the stated deviator stress limits. After four cycles, samples were stressed to failure to obtain Cu.

e. m. peto associates ltd.

Toronto 19, Ontario



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

JOB NAME Long's Creek Bridge JOB NO. 6423 HOLE NO. 2 SAMPLE NO. 13

DEPTH 20' 2" ELEVATION \_\_\_\_\_ REMARKS Grey Clay

GRAIN SIZE DISTRIBUTION

FIG 1



15556425

GEOMETER CONSOLIDATION TEST

BHC 3A B Depth 51

Grey Clay, 1700 g/100

$w_L = 44.7$ ,  $w_p = 20.7$

Initial water content: 28.7%

Soil density: 123.0 lb/cu ft

Estimated effective overburden  $p' = 0.66$  tons

LOAD STAGE	COEFFICIENTS	
	COMPRESSIBILITY	CONSOLIDATION
	$m_v$	$C_v$
load $p$	eq. ft./ton	sq. ft./yr
$1/8 - 1/4$	0.0390	40.2
$1/4 - 1/2$	0.0378	30.6
$1/2 - 1$	0.0378	20.9
$1 - 2$	0.0184	35.4
$2 - 4$	0.0116	42.0
$4 - 8$	0.0084	48.7
$p' \text{ (avg)}$	0.0141	35.9

VOLUME

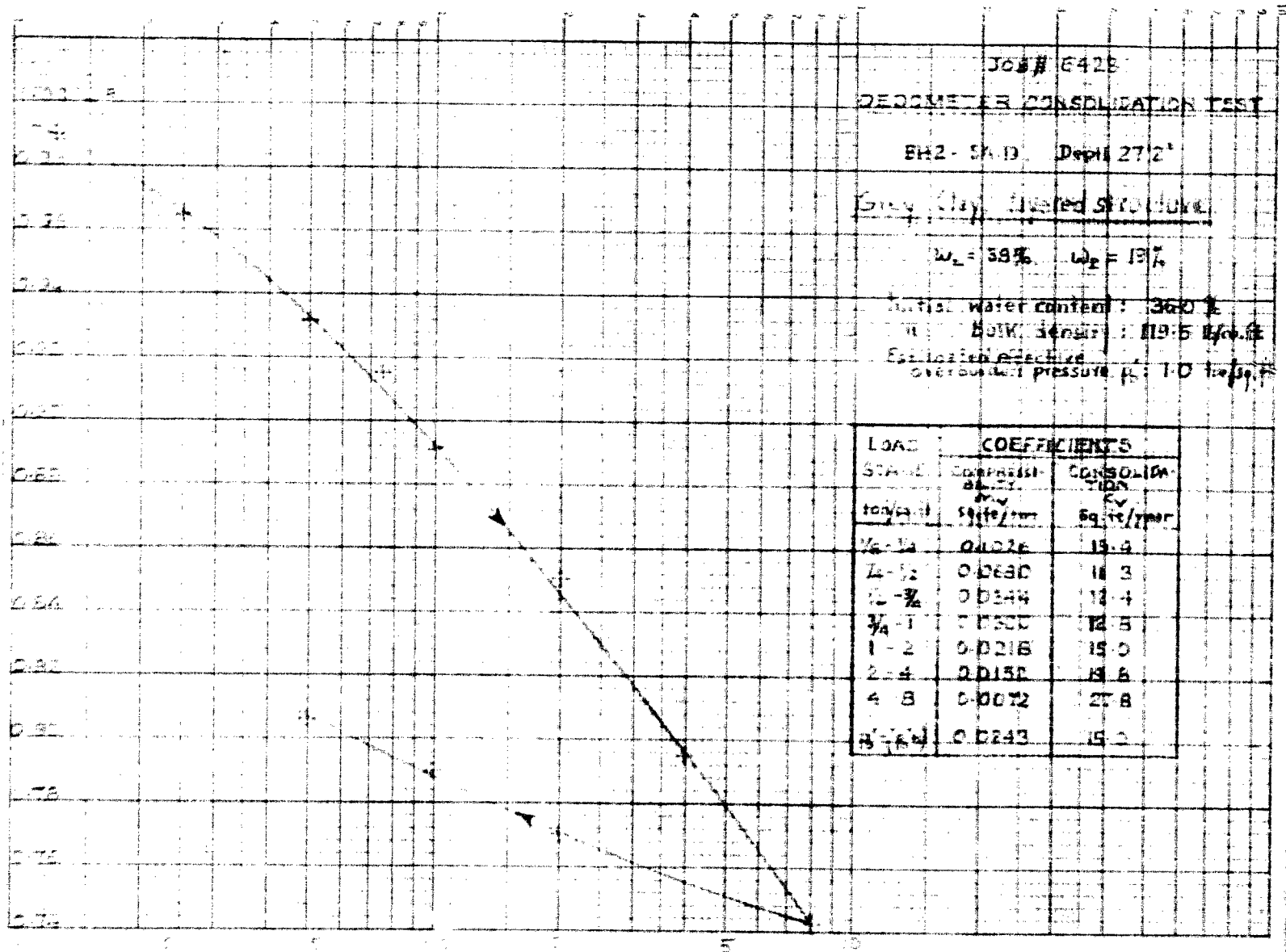
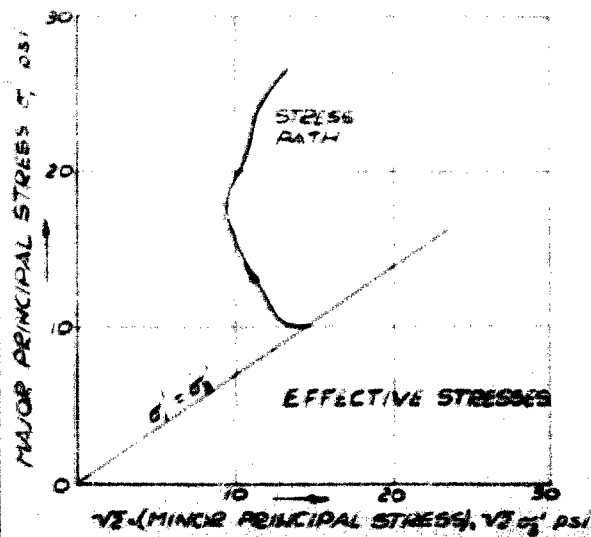
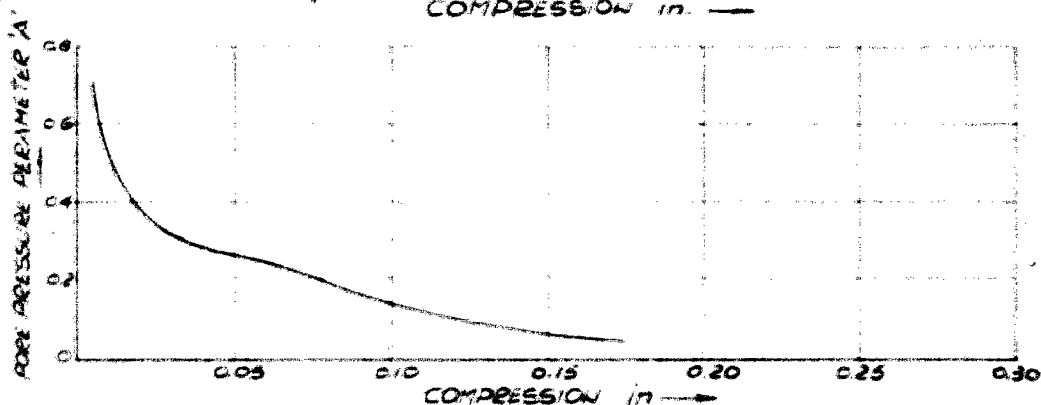
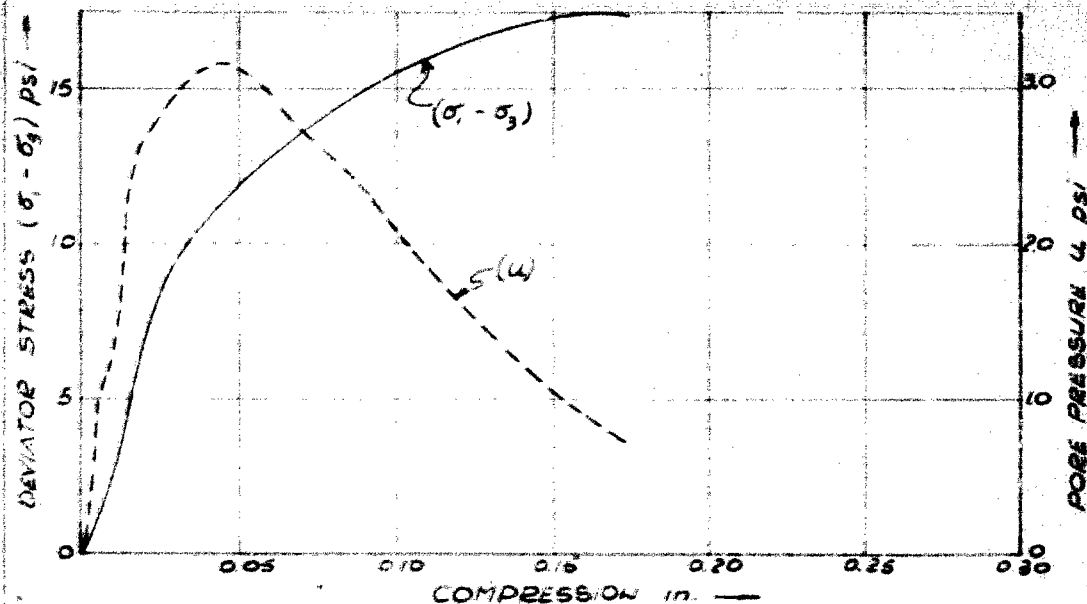


Fig. 2b

# CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST WITH PORE PRESSURE MEASUREMENTS



HOLE #2 SA 8 DEPTH 12'8"-13'0"  
 GREY CLAY WITH OCCASIONAL GRTS

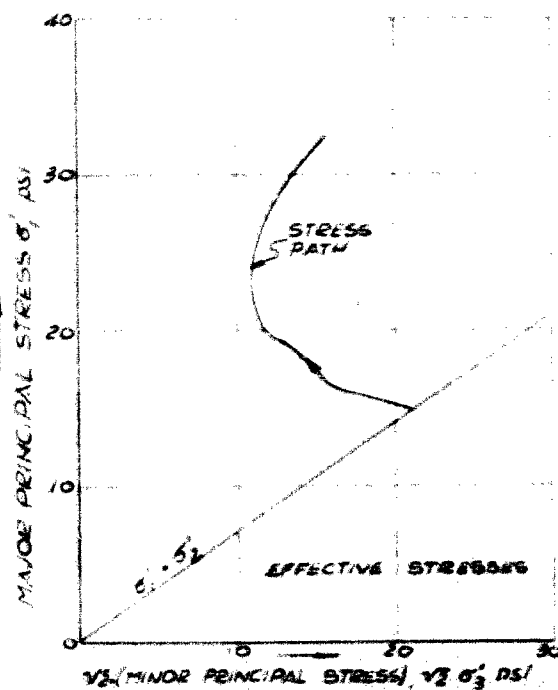
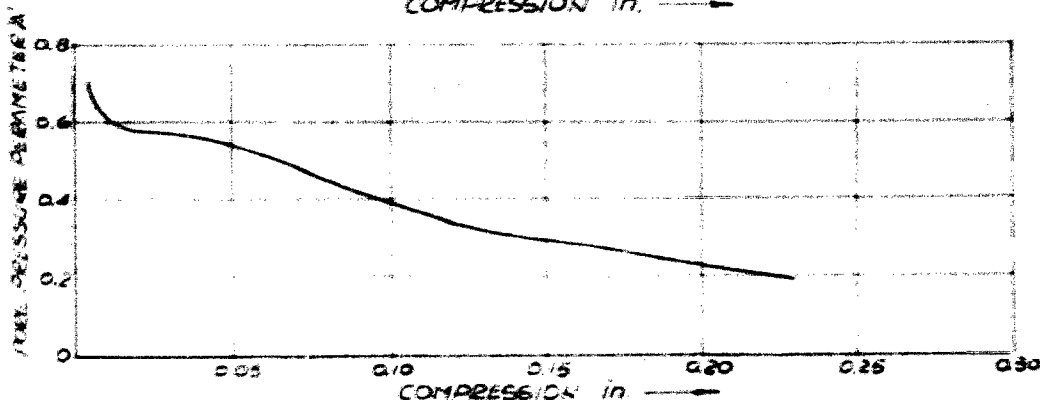
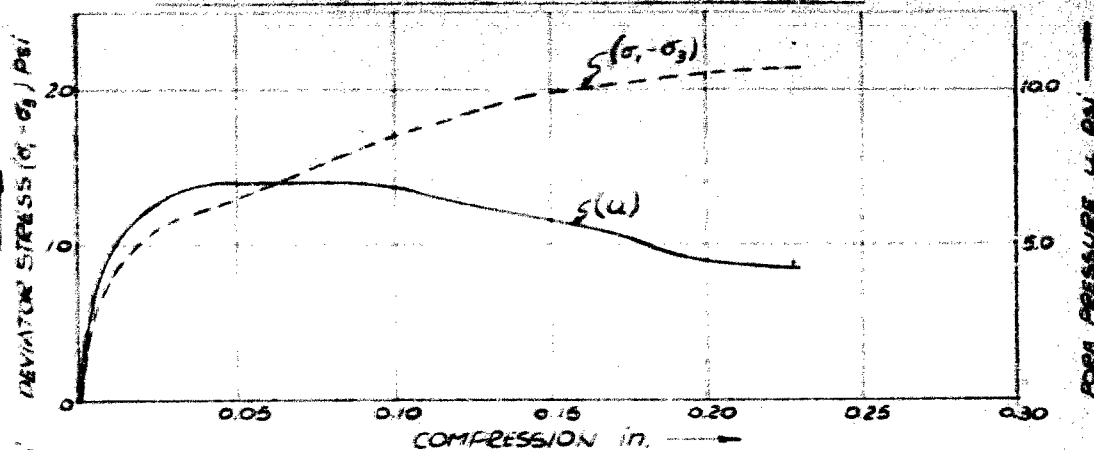
$w_L = 44.2\%$   $w_p = 20.3\%$

INITIAL WATER CONTENT 29.7%  
 INITIAL BULK DENSITY 123.2 pcf.  
 CONSOLIDATION PRESSURE 10 psi  
 (CONSOLIDATED AGAINST  
 BACK PRESSURE OF 30 psi)

PORE PRESSURE PARAMETER 'B' ASSUMED 1.0  
 RATE OF STRAIN: 0.02% PER MIN.

JOB # 6423  
 er: peto associates ltd.  
 MARCH 1964

# CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST WITH PORE PRESSURE MEASUREMENTS



HOLE #2 SA. 20 DEPTH 27'4"-28'0"

$w_L = 38.2\%$   $w_p = 19.0\%$

GREY CLAY WITH SOME GRITS

INITIAL WATER CONTENT = 35.5%

INITIAL BULK DENSITY = 116.0 pcf.

CONSOLIDATION PRESSURE = 15 psi.

(CONSOLIDATED AGAINST BACK)  
 (PRESSURE OF 30 PSI)

PORE PRESSURE PARAMETER 'B'

ASSUMED 1.0

RATE OF STRAIN: 0.02 % PER MIN.

JOB # 6423

e.m. peto associates ltd

MARCH 1964

# e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

## BOREHOLE LOG

Job Name Long's Creek Bridge  
 1st St. Con. Rd. 12.18  
 Client County of Lambton  
 100 Todgham and Case  
 Elevation 90.2

Job No. 4123  
 Casing 4" to 8" BX 15 3/16"  
 Compiled By R. K.

Borehole No. 1  
 Boring Date February 18, 1964  
 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. THINWALLED SHELBY TUBE SAMPLE  
 W.S. WASH SAMPLE  
 R.C. ROCK CORE

### ABBREVIATIONS

V.T. VANE TEST  
 M. MOK  
 W.L. WATER LEVEL IN CASING  
 W.T. GROUND WATER TABLE IN SOIL  
 W.T.P.L. DEETER THAN PLASTIC LIMIT  
 D.T.P.L. DETER THAN PLASTIC LIMIT  
 A.P.L. ABOUT PLASTIC LIMIT

SOIL DESCRIPTION	COLOUR	Consistency or Condition	Depth Feet	Legend	Sample Type	Blows per 6"	WATER LEVEL & REMARKS
			0'0"				
Clayey silt, some fine sand and grits, loam form	Dark yellow brown	Firm		1 2 3	CS CS SS	13	27.4
Clay with occasional pebbles	Brown	Stiff		4	SS	13	28.6
Mostly silt	Ditto	Compac	8'0"	5	SS	19	24.6
Clay with a few grits; neam of very silty clay with grits, partly fissured	Brown some grey fissures	Stiff	9'6"	6	SS	20	24.1
			14'0"	7	2" SL Tapped		
Silt with layer of very silty clay	Grey	Firm to Stiff		8	SS	13	32.4
				10 11	2" SL Pushed		
Clay, evidence of layered structure	Ditto	Firm		13	SS	7	26.7
				14	2" SL Pushed		W.T.P.L. Moderately sensitive
Clay with some pebbles, distinctly layered	Ditto	Ditto		17	SS	7	34.6
				18 20	2" SL		
Ditto	Ditto	Ditto	30'0"	21	SS	8	31.4
				22 23	2" SL		
Silt with sand and broken shale fragments, Till form	Dark grey to black	Dense	36'0" 34.2	25	SS	21	Blows 3 - 9 - 12 per 6"
Broken shale			39'0" 40'2"				
Shale with thin limestone bends	Grey	Solid, sound	50'0"		R.C.		Diamond drilled from 40'2" to 42'3" Core recovery: 25" (100%) Diamond drilled from 42'3" to 46'0", Core recovery: 43" (96%)
			46'0"				

Test Hole Terminated at 46 ft.

# e. m. peto associates ltd.

SOIL ENGINEERING SERVICE TORONTO, ONTARIO

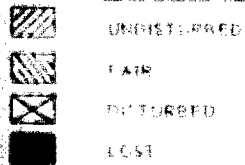
## BOREHOLE LOG

Job Name: Lamb's Creek Station  
 Lot 21, Conc. 21/12-13  
 Client: County of Lambton  
 Elevation: 200'

Lot No.: 421  
 Usage: 4" to 5" BX to 38' 6"  
 Completed By: R. B.

Borehole No.:  
 Boring Date: February 20, 21, 1964  
 Checked By: S. P.

### SAMPLE CONDITION



### SAMPLE TYPE

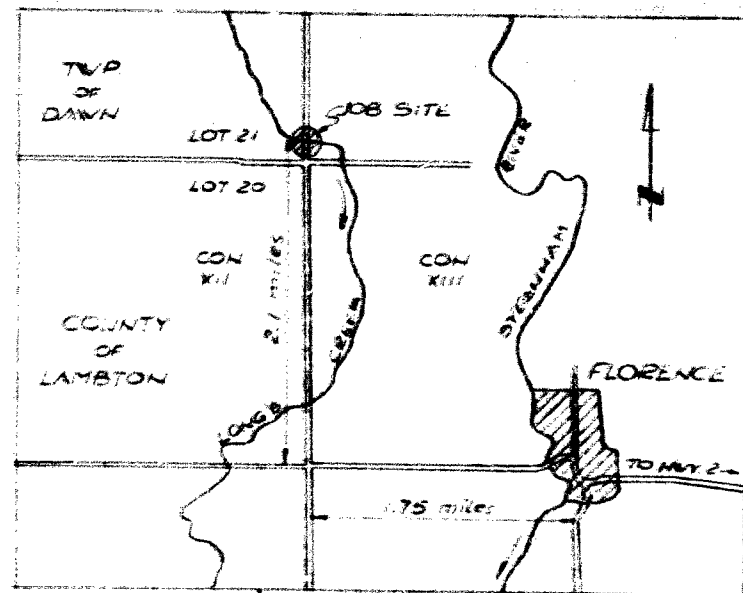
A.S. AUGER SAMPLE  
 C.S. CASING SAMPLE  
 S.S. 2" STANDARD SELF-TIME SAMPLE  
 S.L. SPLIT BARREL WITH LINERS  
 S.T. THIN-WALLED SHELBY TUBE SAMPLE  
 W.S. WASH SAMPLE  
 R.C. ROCK CORE

### ABBREVIATIONS

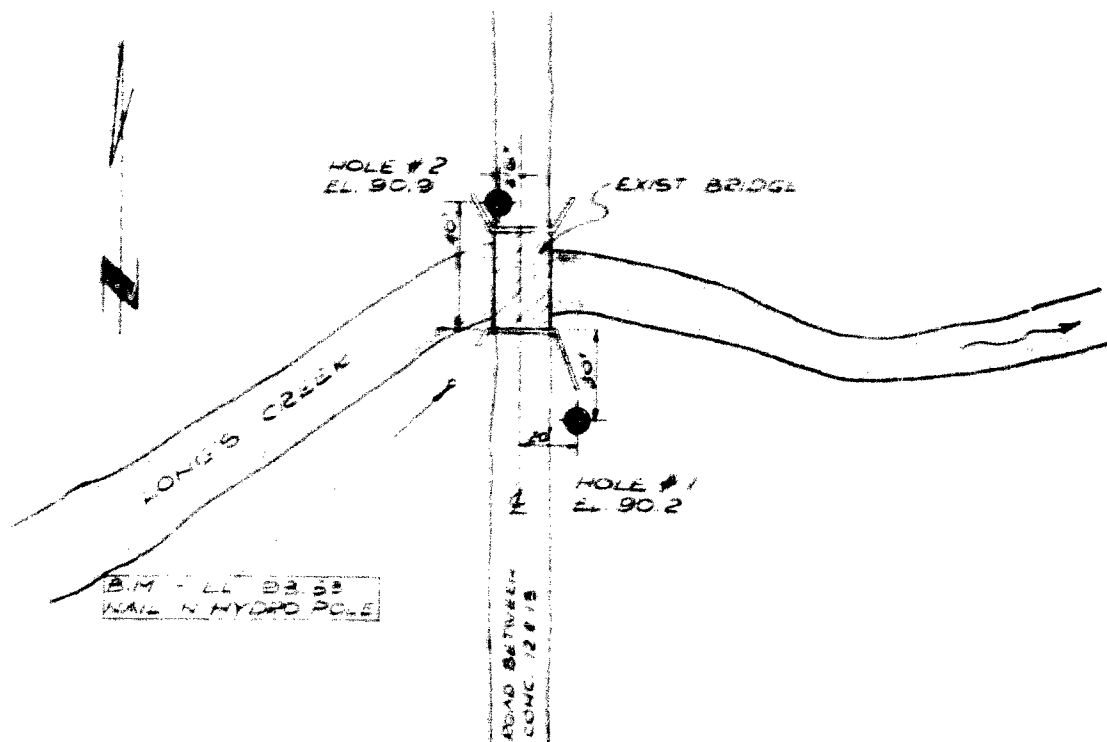
V.T. IN SITU VANE SHEAR TEST  
 M. MOIST  
 W.L. WATER LEVEL IN CASING  
 W.T. GROUND WATER TABLE IN SOIL  
 W.T.P.L. WETTER THAN PLASTIC LIMIT  
 D.T.P.L. DRIER THAN PLASTIC LIMIT  
 A.P.L. ABOUT PLASTIC LIMIT

SOIL DESCRIPTION	COLOR	Texture & Consistency	Depth (feet)	Lab. No.	Sample No. and Location	Sample Type	Moisture (%)	W.T. (%)	REMARKS
Sand and gravel fill	Brown		0' 0" - 1' 0"		1	CS			
Silt with fine sand, some clay and grits, loam-form. Very fissured, possibly till	Dr. yellow brown Firm		1' 0" - 2' 0"		2	SS	11	14.5	
As above, some wood fragments	Mottled brown and grey	Ditto	2' 0" - 3' 0"		3	SS	8	30.5	
Clay with occasional grits	Grey with brown lenses	Stiff	3' 0" - 4' 0"		4	SS	8	27.8	
Silty clay, some grits, layered	Grey	Firm	4' 0" - 5' 0"		5	SS	12	27.0	W.T.P.L.
As above, some silt seams	Ditto	Ditto	5' 0" - 6' 0"		6	3"SL			
Clay	Ditto		6' 0" - 7' 0"		7	SS	10	23.5	
Clay layered structure	Ditto		7' 0" - 8' 0"		8	3"SL			
Ditto	Ditto		8' 0" - 9' 0"		9	SS	7	31.5	
Ditto	Ditto		9' 0" - 10' 0"		10	3"SL			
Silt with sand, clay & grits	Dr. grey to black	Dense	10' 0" - 11' 0"		11	SS	8	30.4	
As above, with broken shale fragments	Black	Dense	11' 0" - 12' 0"		12	3"SL			
Shale with thin limestone bands	Grey	Solid, sound	12' 0" - 13' 0"		13	SS	6	34.6	
			13' 0" - 14' 0"		14	2"SL			Sample left in hole overnight
			14' 0" - 15' 0"		15	SS	21	10.4	
			15' 0" - 16' 0"		16	SS	31	20.3	
			16' 0" - 17' 0"		17				Refusal at 38' 6"
			17' 0" - 18' 0"		18	RC			Diamond drilled from 38' 6" to 43' 9", Recovery : 61" (96%)
			18' 0" - 19' 0"		19				
			19' 0" - 20' 0"		20				
			20' 0" - 21' 0"		21				
			21' 0" - 22' 0"		22				
			22' 0" - 23' 0"		23				
			23' 0" - 24' 0"		24				
			24' 0" - 25' 0"		25				
			25' 0" - 26' 0"		26				
			26' 0" - 27' 0"		27				
			27' 0" - 28' 0"		28				
			28' 0" - 29' 0"		29				
			29' 0" - 30' 0"		30				
			30' 0" - 31' 0"		31				
			31' 0" - 32' 0"		32				
			32' 0" - 33' 0"		33				
			33' 0" - 34' 0"		34				
			34' 0" - 35' 0"		35				
			35' 0" - 36' 0"		36				
			36' 0" - 37' 0"		37				
			37' 0" - 38' 0"		38				
			38' 0" - 39' 0"		39				
			39' 0" - 40' 0"		40				
			40' 0" - 41' 0"		41				
			41' 0" - 42' 0"		42				
			42' 0" - 43' 0"		43				
			43' 0" - 44' 0"		44				
			44' 0" - 45' 0"		45				
			45' 0" - 46' 0"		46				
			46' 0" - 47' 0"		47				
			47' 0" - 48' 0"		48				
			48' 0" - 49' 0"		49				
			49' 0" - 50' 0"		50				
			50' 0" - 51' 0"		51				
			51' 0" - 52' 0"		52				
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			54' 0" - 55' 0"		55				
			55' 0" - 56' 0"		56				
			56' 0" - 57' 0"		57				
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			60' 0" - 61' 0"		61				
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			62' 0" - 63' 0"		63				
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			64' 0" - 65' 0"		65				
			65' 0" - 66' 0"		66				
			66' 0" - 67' 0"		67				
			67' 0" - 68' 0"		68				
			68' 0" - 69' 0"		69				
			69' 0" - 70' 0"		70				
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			72' 0" - 73' 0"		73				
			73' 0" - 74' 0"		74				
			74' 0" - 75' 0"		75				
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			76' 0" - 77' 0"		77				
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			87' 0" - 88' 0"		88				
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			89' 0" - 90' 0"		90				
			90' 0" - 91' 0"		91				
			91' 0" - 92' 0"		92				
			92' 0" - 93' 0"		93				
			93' 0" - 94' 0"		94				
			94' 0" - 95' 0"		95				
			95' 0" - 96' 0"		96				
			96' 0" - 97' 0"		97				
			97' 0" - 98' 0"		98				
			98' 0" - 99' 0"		99				
			99' 0" - 100' 0"		100				

Test Hole Terminated at 43' 9"

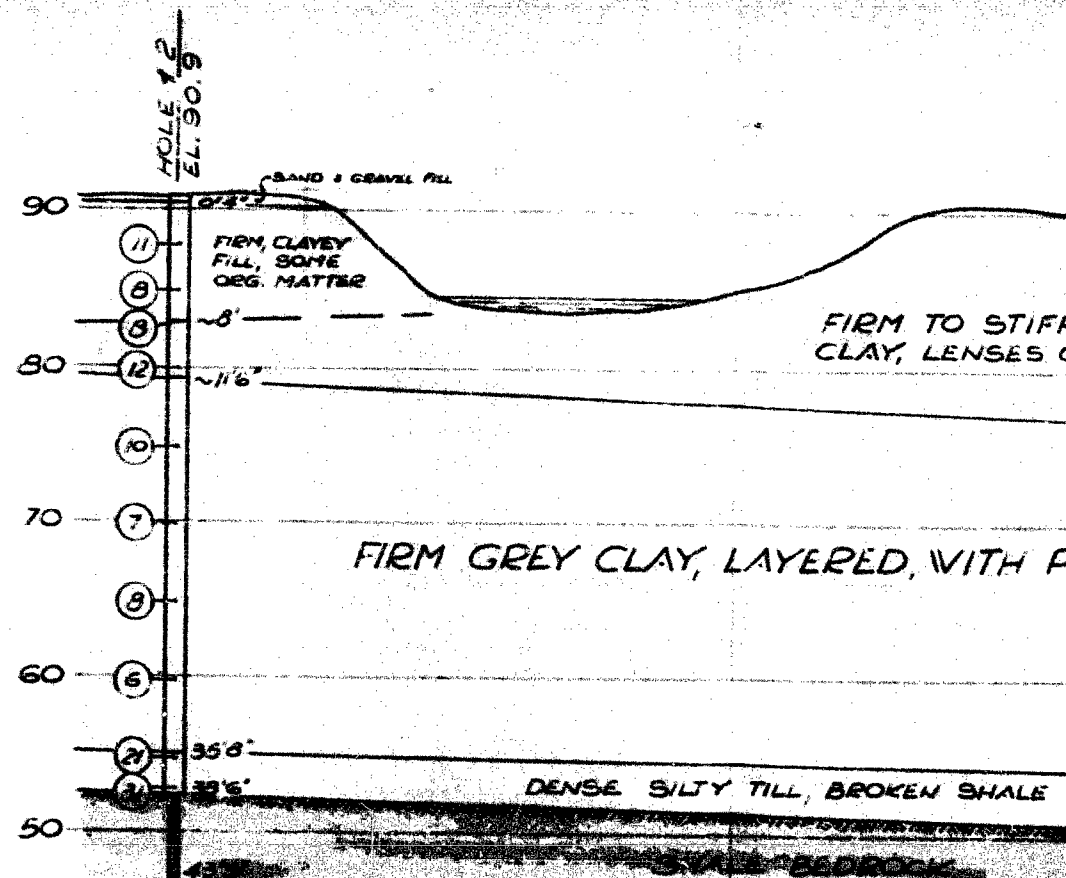


KEY PLAN  
SCALE: 1 MILE TO 1 INCH



SITE PLAN

SCALE: 50' TO 1"



SECTION THROUGH HOLES 2 & 12

SECTION SCALE: 10' TO 1" (NATURAL)

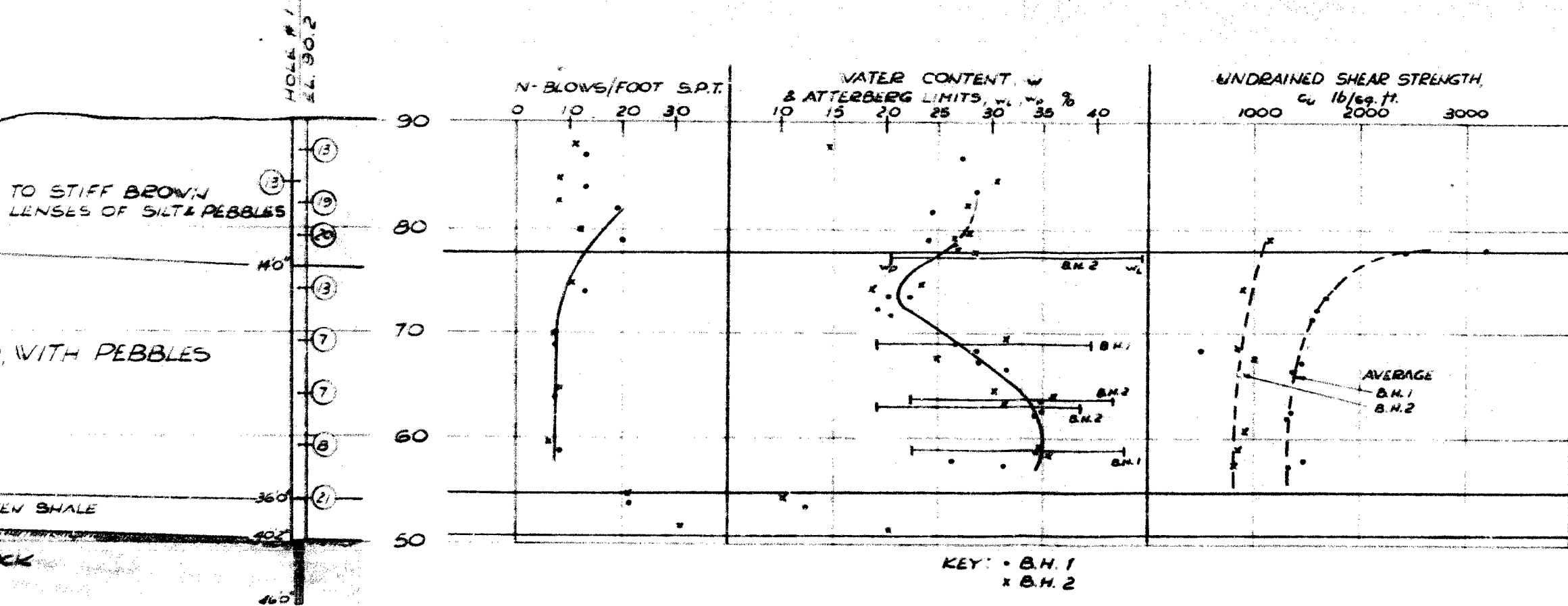
### LEGEND

● BOREHOLE

—○— BLOWS/FOOT S.P.T.

### NOTE

SEE BOREHOLE LOGS FOR COMPLETE SOIL DETAILS.



# GEOTECHNICAL PROPERTIES

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



THE COUNTY OF LAMBTON			
% TODGHAM & CASE LTD, CONSULTING CIVIL ENGINEERS			
LONG'S CREEK BRIDGE			
LOT 21, CONC. RD. 12-13			
PREPARED BY			
e.m. peto associates ltd.			
JOB NO. 6423	MARCH 1964	DWN. BY: K.K.	CHECK'D BY: K.K.