

#63-F-274m

STEWART

CREEK BRIDGE

PLYMPTON

Twp.

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 6390

1287 caledonia road
TORONTO 19, ONTARIO.
Telephone: 789-1126

July 12th, 1963.

The Township of Plympton,
c/o J. A. Monteith Associates Ltd.,
Consulting Engineers,
P.O. Box 579,
Petrolia, Ontario.

Attention: Mr. G. Ingram, P. Eng.

Gentlemen:

Re: Subsoil Investigation,
Stewart Creek Bridge,
Plympton Township, Ontario.

Following submission of our Report No. 6390 on the above investigation, we were informed by the Consulting Engineers that the bridge is to have a span of 45 ft and that it is proposed to adopt elevation 610 as the footing level; the footings have the dimensions 10 x 30 ft. The net foundation pressure (dead + live) would be about 1.1 ton/sq ft.

On the basis of the geotechnical properties included in the Report, and maintaining the various assumptions on which the original calculations were based, we have revised the bearing capacity and settlement of a footing designed to the above dimensions. The figures depend to some extent on the embedment depth, D of the footing (this should be the least value), which is uncertain. The following results were obtained:

	<u>D = 10 ft</u>	<u>D = 20 ft.</u>
Ultimate net bearing capacity, ton/sq. ft.	2.40	2.32
Allowable net bearing capacity, ton/sq. ft. (for $F = 3$)	0.80	0.94
Immediate settlement, in.	0.78	0.68
Long-term settlement, in.	0.85	0.74
Total final settlement, in.	1.63	1.42

The settlements are for a net stress increase of 1.0 ton/sq. ft in excess of the existing net overburden pressure.

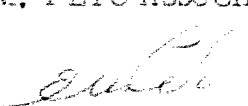
An overburden component, γD , where γ is the density of material above footing level, can be added to the net allowable bearing capacity, and will not result in additional settlement if γD does not exceed existing effective overburden pressure, which at elevation 10 was estimated to equal 1.0 ton/sq. ft.

As has been discussed in the Report, only about one-third of the indicated settlement is likely to develop after the structure has been completed. Also as discussed in the Report, the bearing capacity estimate is likely to be over-pessimistic, for the reasons given.

It is felt that, with the proposed net pressure of 1.1 ton/sq. ft, a sufficient factor of safety against shear failure will be retained resulting in tolerable settlements, provided that the constructional precautions suggested in the Report are followed.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

RK;sb

E. M. PETO ASSOCIATES LTD.

Job No. 6390

1287 Caledonia Road,
Toronto 19, Ontario.
RUssell 9-1126-7

June 13th, 1963.

SITE 14-293

The Township of Plympton,
c/o J. A. Monteith Associates Ltd.,
Consulting Engineers,
P.O. Box 579,
Petrolia, Ontario.

Attention: Mr. G. Ingram, P. Eng.

Gentlemen:

Re: Subsoil Investigation,
Stewart Creek Bridge,
Plympton Township, Ontario.

We have pleasure in submitting five copies of our
Report No. 6390 on the above investigation.

Field standard penetration test results were in the range
of 15 to 20 blows per foot below the anticipated footing level; these are
relatively high values for a clay stratum, and indicated that a footing
foundation would probably be adopted for the proposed bridge of only 30
ft span. Consequently, the test holes were terminated at a depth of 31.5 ft,
within which no stratum capable of supporting an alternative, end-bearing
pile foundation was encountered.

Although subsequent laboratory tests indicated a lower strength of subsoil than was concluded from the field penetration data, nevertheless, it is felt that a spread footing design is practicable. The bearing capacity is discussed in the Report, in the light of the anticipated settlement.

Should you wish to discuss further any aspects of this investigation, we would be very pleased to provide additional assistance.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,


E. M. Peto, P. Eng.

RK:sb

THE TOWNSHIP OF PLYMPTON,

C/O J. A. MONTEITH ASSOCIATES LTD.,
CONSULTING ENGINEERS.

SUBSOIL INVESTIGATION

STEWART CREEK BRIDGE,
PLYMPTON TOWNSHIP, ONTARIO.

E. M. PETO ASSOCIATES LTD.,

1287 Caledonia Road,
Toronto 19, Ontario.

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APPENDIX "A" LABORATORY TEST RESULTS

BOREHOLE LOGS

SITE PLAN and SUBSOIL PROFILE

A. INTRODUCTION:

The work described in this Report was authorized by J. A. Monteith Associates Ltd., Consulting Engineers, by letter dated May 9th, 1963.

A site investigation was required for a bridge, of approximate span 30 ft, which is to cross a proposed new channel of the Stewart Creek; the rearrangement of the creek bed and road alignments will eliminate the necessity for several bridges in the area.

The site is located at the intersection of two gravel roads, at a point located approximately 5 miles north of Reece's Corner and about 5 miles west-south-west of Forest, Ontario.

The test holes were put down at a site located several hundred feet from the present channel of Stewart Creek.

B. GENERAL INFORMATION:

1. Two test holes were performed at the site, in the positions indicated on the enclosed drawing. This drawing is based on a site sketch supplied by the Consulting Engineers, who also provided the ground elevations at the positions of the test holes. The test holes were set out in the field by Consulting Engineers' Mr. G. Ingram, who met our drilling crew at the site.

B. GENERAL INFORMATION: (Cont'd)

2. The field work was performed by our drilling rig unit No. 7, between May 22nd and 23rd, 1963. Our standard drilling and sampling procedures were followed.

3. Both test holes were terminated at a depth of 31.5 ft below the existing grade, having encountered very similar subsoil conditions, consisting of a silty clay till which was stiff near ground surface but progressively softening with depth. On the basis of standard penetration test results, it was felt that the depth of the test holes was sufficient for the proposed span of the new bridge.

4. The following soil tests were performed in our laboratory.

- Water content determinations
- Atterberg limit tests
- Grain size distribution
- Unconfined compression tests with volumetric analyses
- Undrained triaxial compression tests, with and without pore pressure measurement
- Consolidation tests.

The results of all tests are included in the Appendix.

C. SUBSOIL CONDITIONS:

Details of the subsoil conditions encountered at the test holes are described on the borehole logs, while a simplified subsoil profile, in the form of a section through the test holes, is included on the drawing and shows the inferred contact between the various subsoil strata.

Apart from a surficial layer of sand and gravel fill, extending to a depth of 2.2 to 3.0 ft below the existing grade and forming the present road structure, the subsoil was found to consist of a stratum of silty clay with pebbles, typical of the clay overburden of glacial origin which overlies this part of Lambton County.

The clay has a very stiff, desiccated crust of mottled brown and grey colour, extending to a depth of 9.5 to 11.5 ft below the existing grade. In the upper portion of this crust, the material is fissured.

Below the brown crust, the clay is of grey colour and of stiff to firm consistency, becoming firm to soft with depth. Grain - size distribution curves of two typical samples are included on Figs. 2a and 2b.

The distribution of geotechnical properties of the subsoil with depth is illustrated on Fig. 1, where the natural water content with relation to the Atterberg Limits, as well as standard penetration test results and the undrained shear strength are plotted against elevation.

- 2 -

C. SUBSOIL CONDITIONS: (Cont'd)

It will be observed from Fig. 1 that the natural water content is near the plastic limit in the upper portions of the stratum, while lower down it is generally in the lower half of the plastic range, corresponding to a firm consistency.

The undrained shear strength is found to drop from over 3000 lb/sq. ft above elevation 617, to less than 1,000 lb/sq. ft below elevation 610. The average curve of undrained shear strength plotted on Fig. 1, used in the bearing capacity calculations, was plotted taking into account also the relationship between the undrained shear strength and the water content.

It may be noted that the standard penetration test results at this site give an unduly optimistic indication of the undrained shear strength of the silty clay till. The average N-values of at least 15 blows per foot below elevation 610 are high for a material with an undrained shear strength of between 600 and 1,000 lb/sq. ft. The relatively high penetration resistance is probably caused by the presence of pebbles. On the other hand, however, the laboratory test results may underestimate the true shear strength of the clay, because of possible sampling disturbance caused by pebbles.

C. SUBSOIL CONDITIONS: (Cont'd)

Consolidation characteristics of the silty clay till were studied by means of three oedometer tests, the results of which are included in the Appendix in the form of void ratio-log pressure curves (Figs. 3a, b, c). Coefficients of consolidation c_v for the various load stages of the tests are included in tabular form.

As the test on a sample from the depth of 18 ft in test hole 1 appeared to give a rather low result of compressibility, the test was repeated on a sample from similar level in test hole 2. The indicated preconsolidation pressure p_c and the value of the coefficient of volume change m_v for a stress increment of 1.0 ton/sq. ft in excess of existing overburden pressure was identical, (0.0047 sq. ft/ton) and both test results are included in the Appendix. The compressibility of the upper, over-consolidated layers of the clay was thus found to be quite low, but it increases with depth to a relatively high value of $m_v = 0.0150$ sq. ft/ton at a depth of 23 ft.

The Modulus of Linear Deformation, E (Young's Modulus) was obtained from two special undrained triaxial compression tests, and the results are tabulated in the Appendix. The average value of E was taken as 110 ton/sq. ft and was used in the calculation of the immediate, or elastic part of the total settlement below footings.

C. SUBSOIL CONDITIONS: (Cont'd)

The average value of Skempton's pore pressure parameter A over the estimated stress range to be applied by structure, was taken as 0.4, on the basis of the two consolidated-undrained triaxial compression tests, the results of which are included in the Appendix. These results were used in conjunction with the compressibility measured in the oedometer tests for the calculation of the long-term, "consolidation" settlement of footings.

D. WATER CONDITIONS:

No free ground water was encountered in either test hole at any depth. Although insignificant seepage from perched ground water pockets contained in any pervious lenses is possible, it appears certain that no major ground water problem will be encountered during excavation for bridge footings and for the new stream channel.

E. CONCLUSIONS AND RECOMMENDATIONS:

1. Summary of subsoil conditions

Apart from a surficial layer of sand and gravel fill, the subsoil consists of a silty clay with pebbles, which was penetrated in both test holes to a maximum depth of 31.5 ft below the existing grade. The clay possesses a very stiff, desiccated, upper crust extending down to the average elevation 618. Below this crust, the stratum becomes progressively softer with depth (See drawing and Fig. 1).

2. Bridge Foundations.

a) General Considerations

It was assumed that for the proposed bridge of 30 ft span, the subsoil has adequate bearing capacity to allow spread footing foundations.

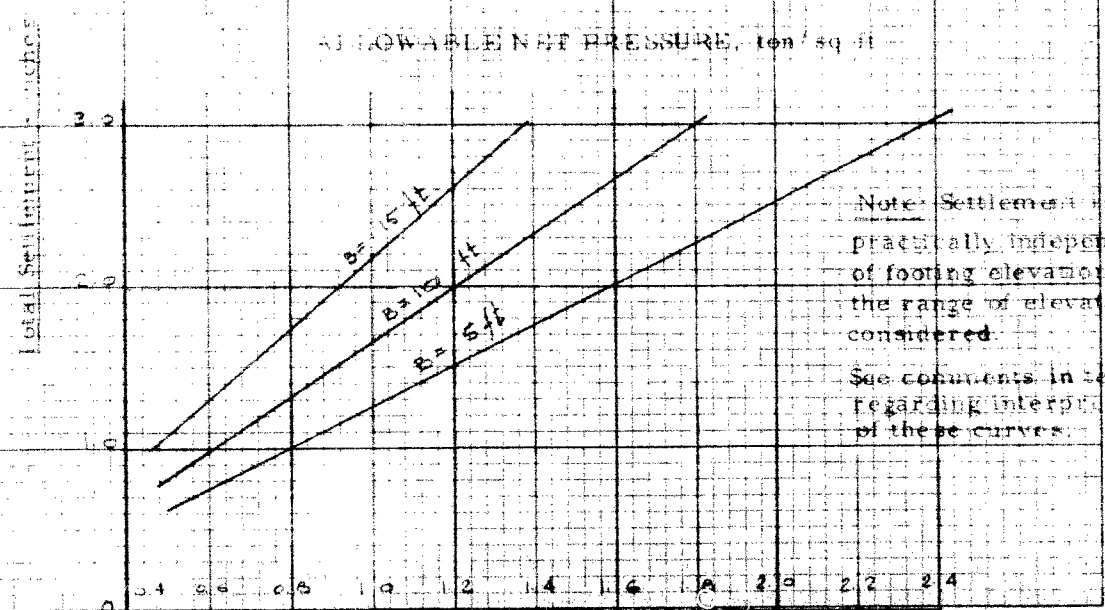
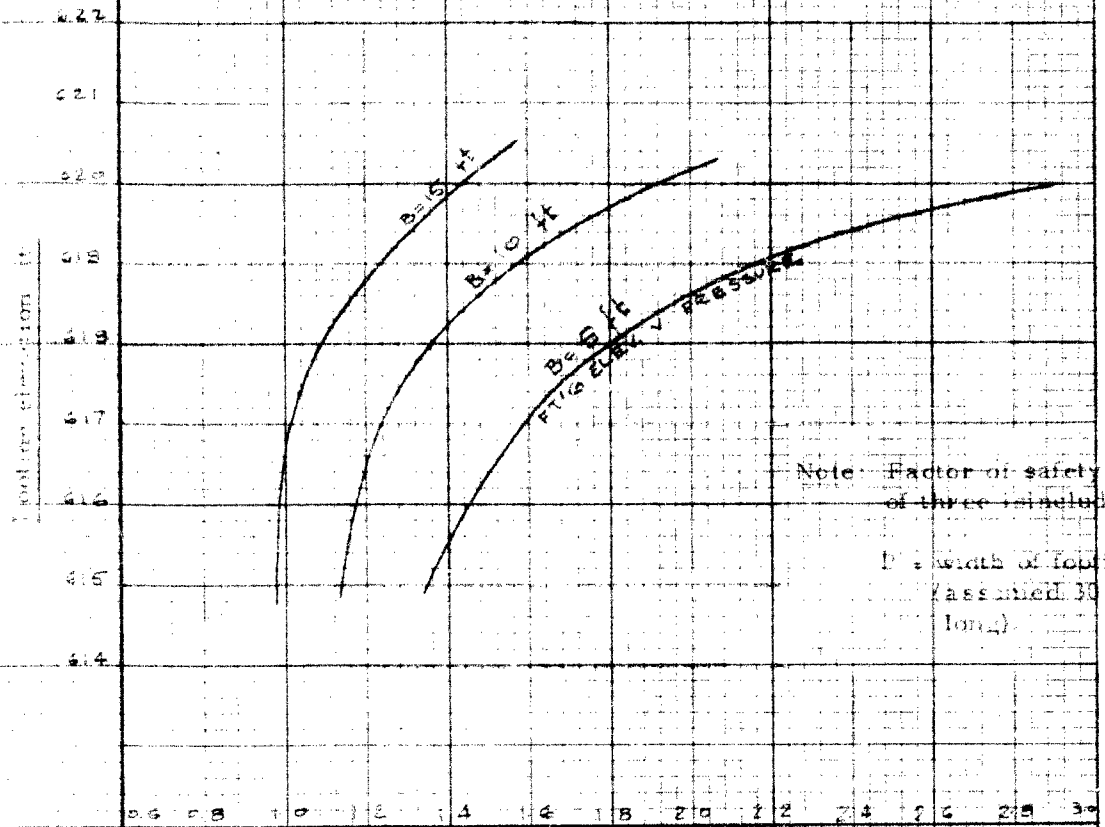
The footing level will depend on the potential scouring considerations and on the distance of abutments from the water's edge. This information was not available at this time, and it was assumed that the footings would be located between elevations 615 and 620.

Because of the fact that the subsoil becomes progressively softer with depth, there is a distinct advantage of keeping the footings at as high a level as possible, in order to take advantage of the higher bearing capacity of the upper layers of subsoil.

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STEWART CREEK BRIDGE
BEARING CAPACITY AND SETTLEMENT CURVES



See comments in text regarding interpretation of these curves.

E. CONCLUSIONS AND RECOMMENDATIONS: (Cont'd)

2. Bridge Foundations

b) Bearing Capacity and Settlement

The bearing capacity and theoretical settlement of footings of a assumed length 30 ft and of width varying from 5 to 15 ft was calculated, over a range of possible footing levels between elevations 620 and 615. The results are presented on page 3 in the form of curves of allowable net pressure (for a factor of safety of three) for various footing widths and elevations, and in the form of relationships between the net pressure increase and the total theoretical settlement.

The allowable pressure is for the net stress increase only; an overburden component, equal to the anticipated least weight of overburden above footing level (including following scouring) can be added to the net bearing capacity obtained from the curves, and will not affect the calculated settlement if the presently existing overburden pressure is not exceeded.

The footings were assumed to be 30 ft long; any likely departure from this assumed figure will have a small effect on the bearing capacity and settlement considerations.

E. CONCLUSIONS AND RECOMMENDATIONS: (Cont'd)

2. Bridge Foundations

b) Bearing Capacity and Settlement (Cont'd)

The calculated settlements were as follows, for a unit pressure increase: (see page 8 for actual settlements)

FOOTING OF LENGTH 30 ft. BETWEEN ELEV. 615 and 620

Inches of settlement per ton/sq. ft net stress increase

<u>Footing width, ft.</u>	<u>Immediate ("elastic")</u>	<u>Long-term ("Consolidation")</u>	<u>Total final</u>
5	0.66	0.60	1.26
10	0.98	0.70	1.68
15	1.24	0.96	2.20

It will be observed that the "elastic" settlement forms the major portion of the final settlement. As this settlement, as well as a part of the consolidation settlement, will develop during construction of the abutments, it is unlikely to affect adversely the elements of the bridge sensitive to settlements. It is considered a fair assumption that only about one-third of the above final settlement is likely to develop after the structure is completed. In addition, an arbitrary assumption was made that the compressible clay extends to elevation 550; if the thickness of the clay is appreciably smaller than was assumed, the calculated settlements would be reduced.

E. CONCLUSIONS AND RECOMMENDATIONS: (Cont'd)

2. Bridge Foundations

b) Bearing Capacity and Settlement (Cont'd)

The undrained shear strength of the subsoil, as measured and used in the bearing capacity calculations, was found to be rather low compared to similar material at corresponding water content at other sites in the area which this Company has investigated. The indicated strength may be affected by any sampling disturbance, particularly in view of the presence of pebbles, and may be lower than the true in-situ strength.

In view of the above facts, it is felt that the bearing capacity and settlement curves given on page 3, based on the measured soil properties and on the specified assumptions, give an unduly pessimistic impression of the potential bearing capacity of subsoil at this site. In our opinion, footings of width 5 to 6 ft, placed not lower than elevation 613, can be safely designed to a net pressure of 2.0 ton/sq. ft. The corresponding settlement actually developing under the completed structure is unlikely to exceed 1.0 to 1.5 inch.

E. CONCLUSIONS AND RECOMMENDATIONS: (Cont'd)

3. Excavations and Backfilling

No free ground water was encountered in the test holes, and it is expected that bridge footing and the new channel excavations can be performed under very favourable subsoil conditions. Walls of footing excavations will theoretically stand unsupported in an almost vertical cut during the probable period of construction; however, normal safety precautions are recommended, particularly in view of the fissured structure of the upper layers of the clay stratum.

Every precaution should be taken not to disturb the subsoil below the footings during excavation and construction operations. The clay should not be allowed to soften due to contact with free water, and it is recommended to protect the excavated grade against the weather, should it be unavoidable to retain open footing excavations for any length of time.

It is not considered practicable to use the excavated clay as a backfill behind abutments; a good granular material, which will allow a high standard of compaction, should be imported for this purpose.

E. CONCLUSIONS AND RECOMMENDATIONS: (Cont'd)

4. Embankments

Bridge approach embankments of small height are unlikely to settle appreciably, although a heavy fill placed over a large area could be expected to undergo considerable settlement.

The excavated clay, if used in new embankments, must be considered as susceptible to frost heave, and an adequate granular blanket should be included under pavements as a protective measure against damage due to frost heave.

E. M. PETO ASSOCIATES LTD. ,

C. F. Freeman

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

RK:sb

Job No. 6390

June, 1963.

Report Prepared By:

R. Kulesza

R. Kulesza, P. Eng.

APPENDIX "A"

LABORATORY TEST RESULTS

TABLE A

UNCONFINED COMPRESSION TEST RESULTS

Hole No.	Sample No.	Depth, feet	Nat. M. C. %	Densities, p. c. f.		Void ratio e	% Strain at Failure	u/c Shear Strength p. s. f
				Wet	Dry			
1	4	10'0" - 11'6"	18.2	135.3	114.3	0.47	20.0	3258
1	9	22'0" - 22'6"	21.1	129.5	107.0	0.57	20.0	858
1	9	22'6" - 23'0"	22.2	133.5	109.0	0.55	20.0	890
1	12	27'0" - 27'6"	21.4	131.1	108.0	0.56	20.0	567
1	12	27'6" - 28'0"	21.5	134.3	110.7	0.52	20.0	751
2	4	10'0" - 11'6"	15.3	139.0	120.3	0.40	20.0	2955
2	9	20'0" - 21'6"	20.6	141.0	117.0	0.44	20.0	937
1	7	16'6" - 18'6"	18.7	132.0	111.2	0.52	20.0	600
2	12	26'0" - 28'6"	21.1	126.5	104.4	0.61	20.0	600

TABLE B
UNDRAINED TRIAXIAL COMPRESSION TESTS

DETERMINATION OF MODULUS OF LINEAR DEFORMATION, E

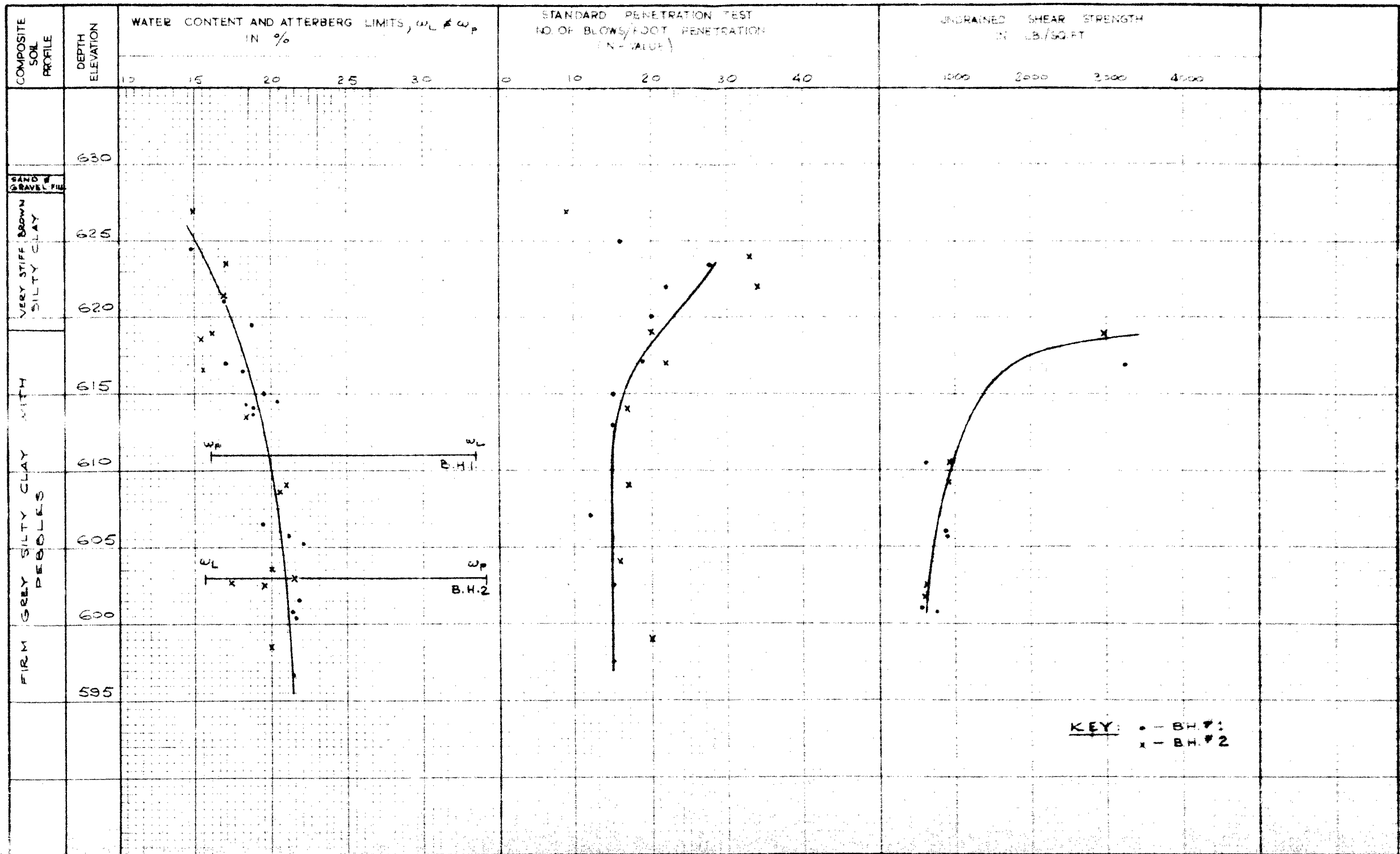
(YOUNG'S MODULUS)

Undrained Shear Strength = C_u

<u>BH/ Sa. No.</u>	<u>Depth</u>	<u>Bulk Density</u> <u>lb/ sq. ft</u>	<u>Water</u> <u>Content, %</u>	<u>Cell Pressure</u> <u>lb/ sq. in.</u>	<u>E</u> <u>ton/ sq. ft.</u>	<u>C_u</u> <u>ton/ sq. ft.</u>	<u>E/C_u</u>	<u>Range of compressive</u> <u>stress lb/ sq. in.</u>
1 / 7	18	129.0	18.4	15	107	0.46	232	1.0 - 3.0
2 / 12	27	129.5	19.6	20	113	0.32	352	0.5 - 5.0

Note: E was obtained from average slope of stress-strain curve loops in successive loading and unloading cycles at a rate of strain of 0.03 in/min.

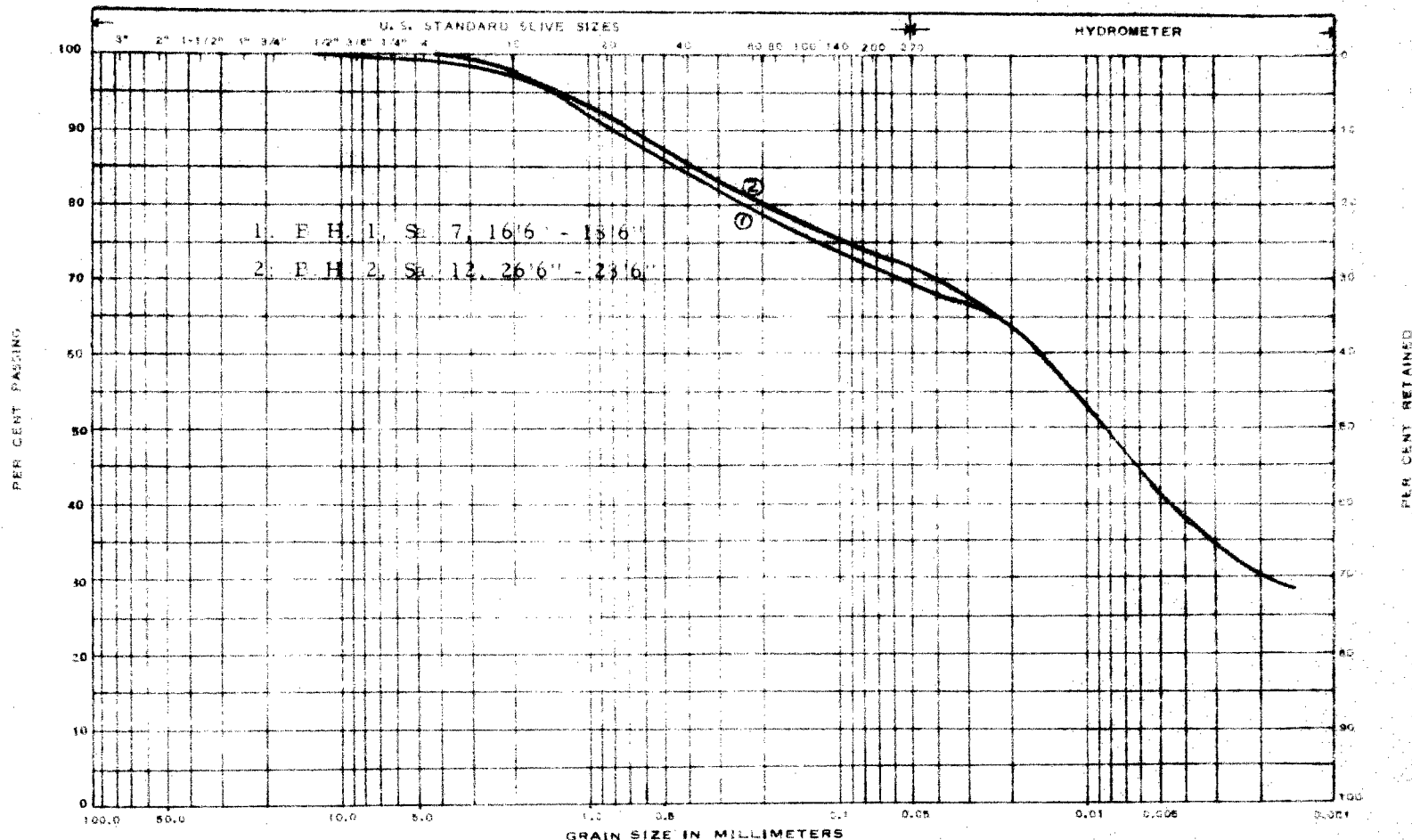
GEOTECHNICAL SOIL PROPERTIES



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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

As shown

JOB NAME Stewart Creek Bridge

JOB NO. 6397

HOLE NO. SAMPLE NO.

DEPTH As shown

ELEVATION

REMARKS

Silty clay with sand and pebbles (silty clay till)

GRAIN SIZE DISTRIBUTION

Fig. 2

ODNOMETER CONSOLIDATION TEST

Job No. 5378

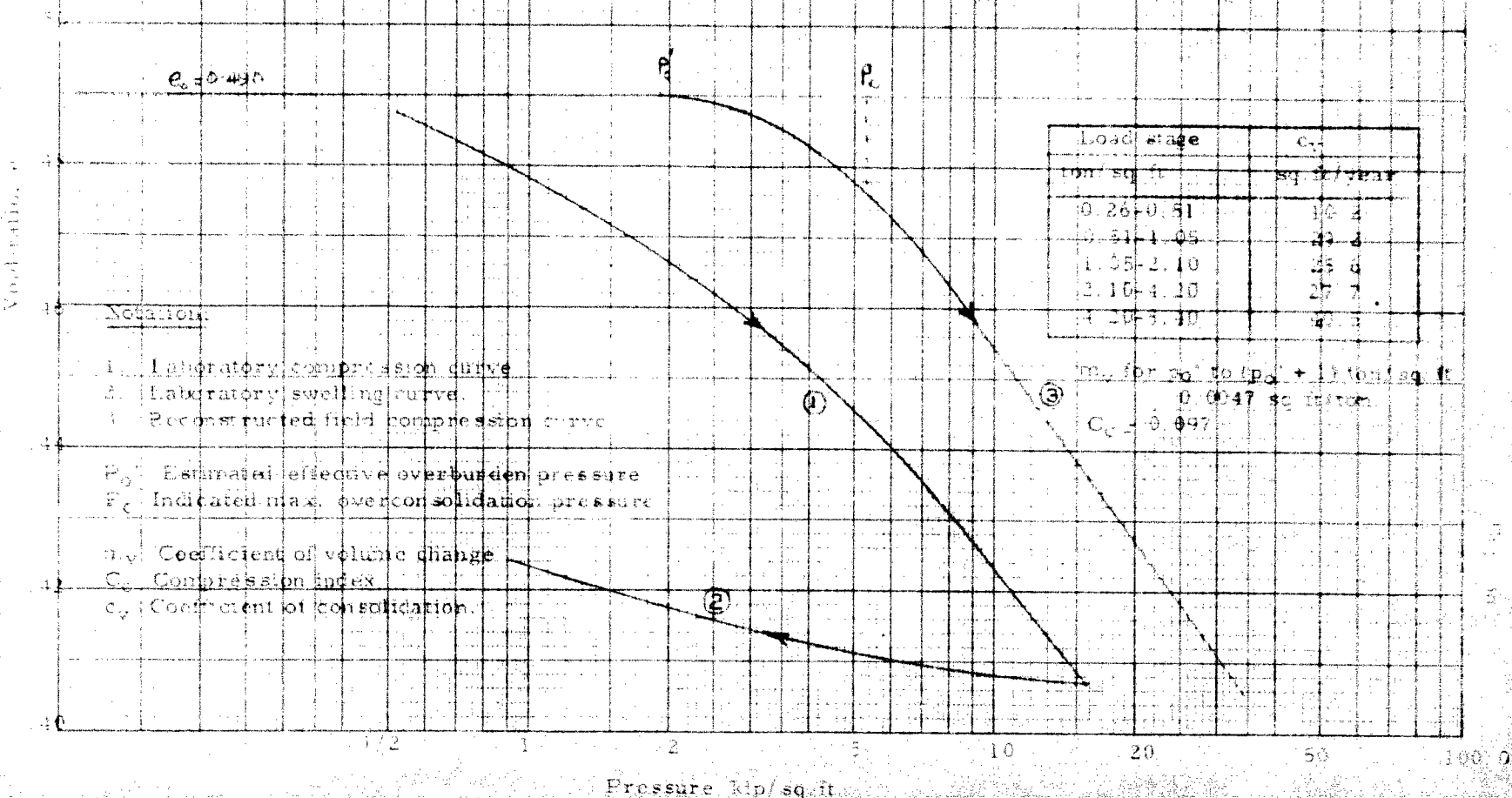
B. H. 1. Sa 7. Depth 15 ft.

Silty clay with pebbles

$W_L = 34\%$, $W_P = 10\%$

Initial bulk density 150.8 lb/cu. ft.

Initial water content 15.5%



16389

OEDOMETER CONSOLIDATION TEST

$e = 0.510$

$P_{o'} \& P_c$

P. H. 2 Sa 12 Depth 24 ft

Shy clay with pebbles

WL - 44% Wp - 16%

Initial bulk density 130.6 lb/cu ft

Initial water content 16.4%

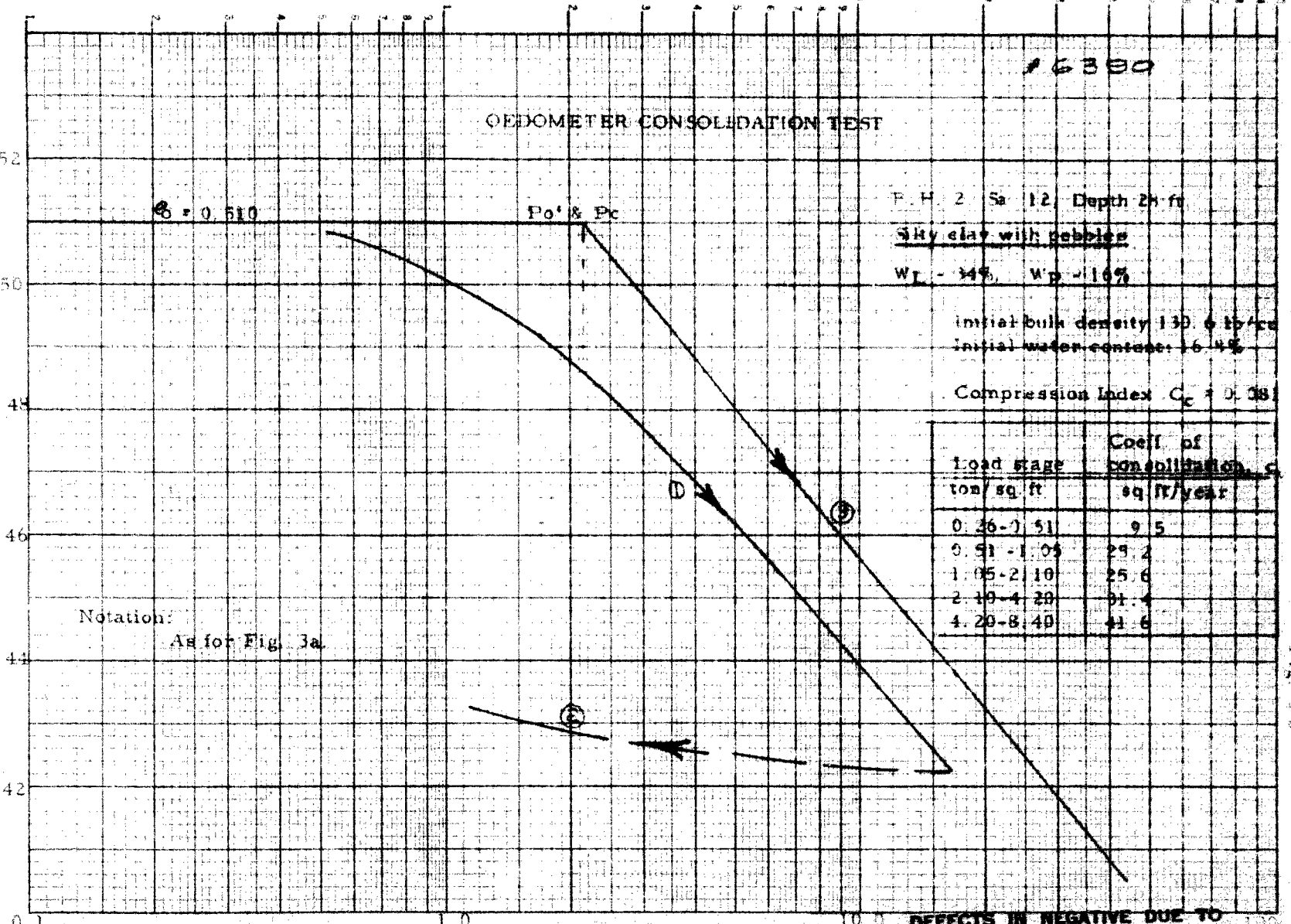
Compression Index $C_c = 0.38$

Load stage ton/sq. ft	Coeff. of consolidation, c_v sq. ft/year
0.26-0.51	9.5
0.51-1.05	29.2
1.05-2.10	25.6
2.10-4.20	31.4
4.20-8.40	41.6

Notation:

As for Fig. 3a

Void ratio



Pressure, K/sq. ft

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FIG. 3b

ODDOMETER CONSOLIDATION TEST

Job No 6390

T. H. 2, Sa. 7, Depth 17.5 ft.

Clay with pebbles

Initial bulk density = 136.2 lb/cu. ft.

Initial water content = 17.7%

$e_0 = 0.4850$

Load stage	c_v
ton/sq ft	ft/year
1/2 - 1	72.0
1 - 2	38.5
2 - 4	55.6
4 - 8	56.0

m_v for p_0' to $(p_0' + 1)$ ton/sq ft.
0.0047 sq ft/ton

$C_c = 0.060$

Notation: As on Fig. 3a.

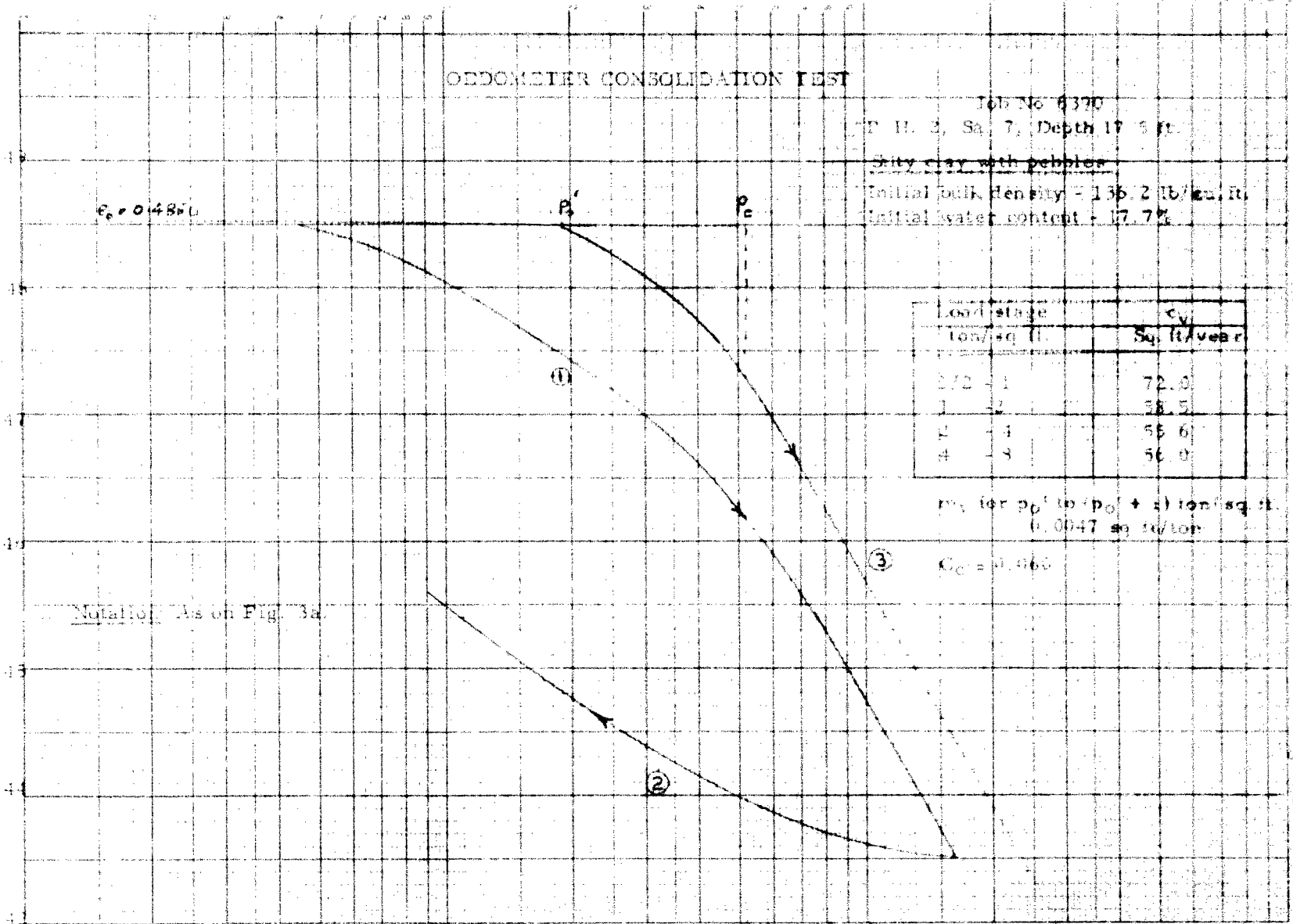
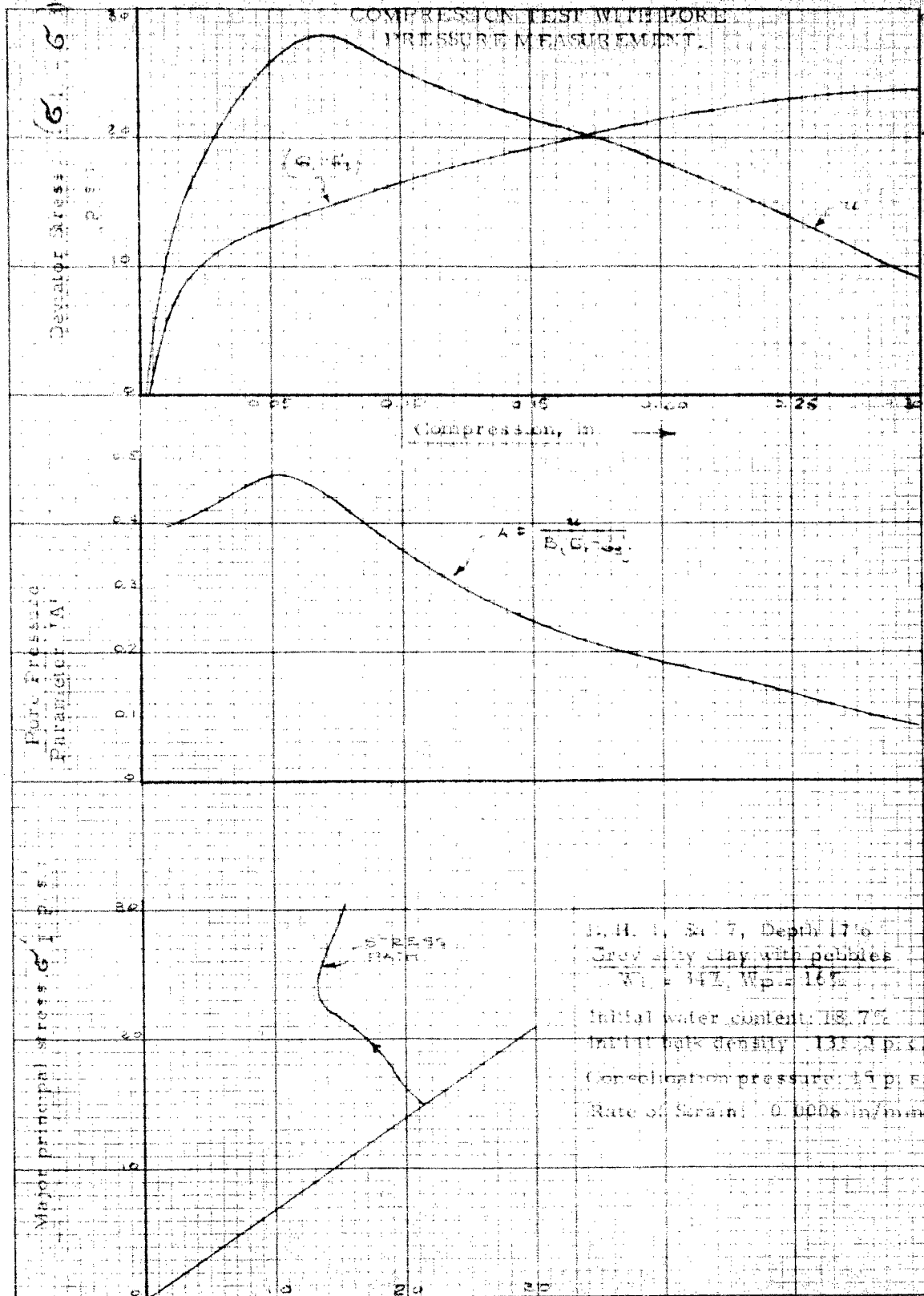


Fig. 3c

CONSOLIDATED-UNDRAINED TRIAXIAL

COMPRESSION TEST WITH PORE
PRESSURE MEASUREMENT.



$\sqrt{2} \times$ (minor principal stress), $\sqrt{2} \times 5$ p.p.s.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Stewart Creek Bridge Job No. 6300 Borehole No. 1
 Client Twp. of Plympton Casing 10 to 5 ft DN to 25 ft Boring Date May 22nd, 1965
 Elevation 928.0 Compiled By RK Checked By S.P.

SAMPLE CONDITION

SAMPLE TYPE

ABBREVIATIONS



UNDISTURBED



FAIR



DISTURBED



LOST

A.S. AUGER SAMPLE
 C.S. CASING SAMPLE
 S.S. 2" STANDARD TEST SAMPLE
 S.L. SELF BARREL SAMPLE
 S.T. THIN-WALLED SAMPLE
 W.S. WASH SAMPLE
 R.C. ROCK CORE

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. WETTER THAN PLASTIC LIMIT
 D.T.P. DRIER THAN PLASTIC LIMIT
 A.F.L. ABOVE PLASTIC LIMIT

SOIL DESCRIPTION	COLOUR	DEPTH (ft)	DIAGRAM	TESTS	WATER LEVELS & REMARKS
Silty sand and gravel fill	Brown	0 to 10			Moist
Silty clay with pebbles fissured	Mottled brown - grey	10 to 11'6"		SS	11.7
Ditto	Ditto	11'6" to 12'0"		SS	12.3
Silty clay with pebbles	Ditto	12'0" to 13'0"		SS	13.5
Silty clay with pebbles	Grey-brown	13'0" to 14'6"		SS	14.2
Ditto	Grey	14'6" to 15'0"		SS	15.1
Ditto	Ditto	15'0" to 16'0"		SS	16.5
Ditto	Ditto	16'0" to 17'0"		SS	17.7
Ditto	Ditto	17'0" to 18'0"		SS	18.1
Ditto	Ditto	18'0" to 19'0"		SS	19.5
Ditto	Ditto	19'0" to 20'0"		SS	20.2
Ditto	Ditto	20'0" to 21'0"		SS	21.1
Ditto	Ditto	21'0" to 22'0"		SS	21.7
Ditto	Ditto	22'0" to 23'0"		SS	23.1
Ditto	Ditto	23'0" to 24'0"		SS	24.1
Ditto	Ditto	24'0" to 25'0"		SS	25.1
Ditto	Ditto	25'0" to 26'0"		SS	26.1
Ditto	Ditto	26'0" to 27'0"		SS	27.1
Ditto	Ditto	27'0" to 28'0"		SS	28.1
Ditto	Ditto	28'0" to 29'0"		SS	29.1
Ditto	Ditto	29'0" to 30'0"		SS	30.1
Ditto	Ditto	30'0" to 31'0"		SS	31.1

No free water in test hole.

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

Test Hole terminated at 31 ft 0 in.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Stewart Creek Bridge Job No. 6390 Borehole No. 2
 Client Twp. of Plympton Casing 4" to 5", PX to 25' Boring Date May 22, and 23, 1963.
c/o J. A. Monteith Associates Elevation 630.0 Compiled By R. K. Checked By S. P.

SAMPLE CONDITION



UNDISTURBED



FAIR



DISTURBED



LOST

SAMPLE TYPE

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

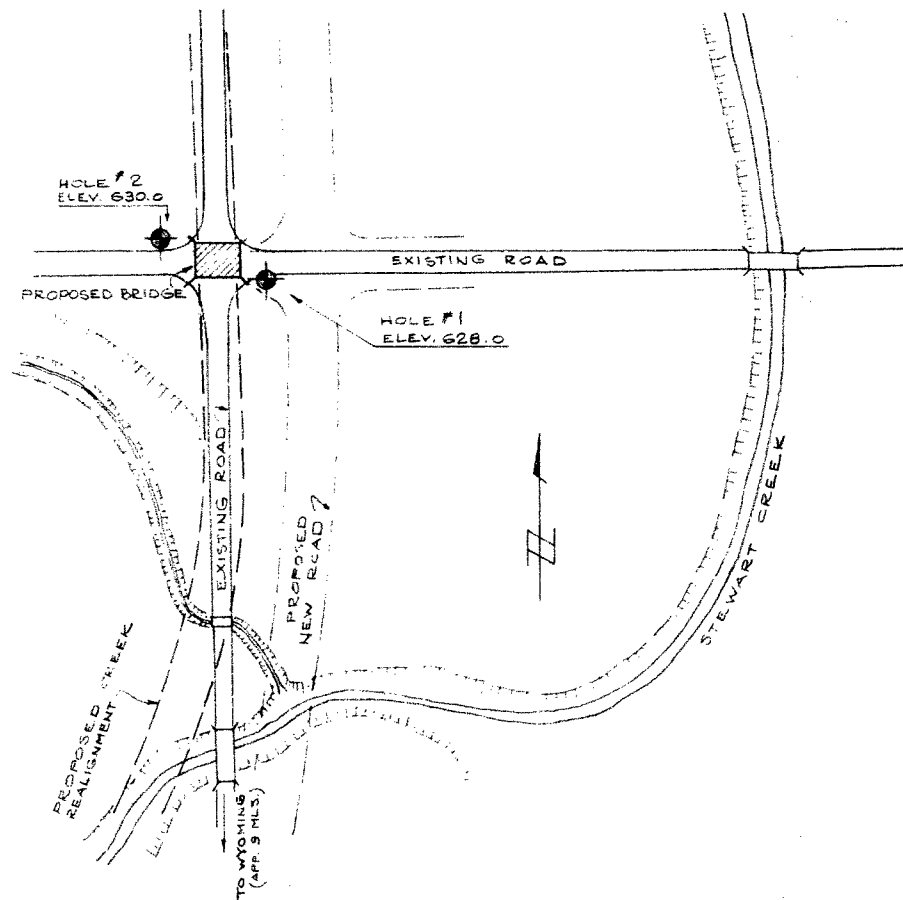
ABBREVIATIONS

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

SOIL DESCRIPTION	COLOUR	Depth or Consistency	Depth Elevation	Logical	Sample No. and Location	Sample Type	No. of Blows per Ft.	WATER LEVELS & REMARKS
Silty sand and gravel fill	Brown		0' 0"		1	CS		Moist.
Silty clay with some sand and pebbles, fissured	Mottled brown	Stiff	2' 0"		2	SS	14.9	
Ditto	Ditto	V. Stiff			3	SS	33	D.T.P.L.
Silty clay with pebbles	Ditto	Ditto	8' 0"		4	SS	34	Just D.T.P.L.
Ditto	Grey brown		9' 6"		5	SS	20	W.T.P.L.
Ditto	Grey	Stiff	10' 0"		6	SS	22	
Ditto	Ditto	Ditto			7	SS	17	
Ditto	Ditto	Ditto			8	SS	17	
Ditto	Ditto	Ditto			9	SS	17	
Ditto	Ditto	Ditto			10	S.T.		
Ditto	Ditto	Firm	20' 0"		11	SS	16	
Ditto	Ditto	Soft to firm			12	ST	21.1	No free water in test hole.
Ditto	Ditto	Ditto			13	SS	20	

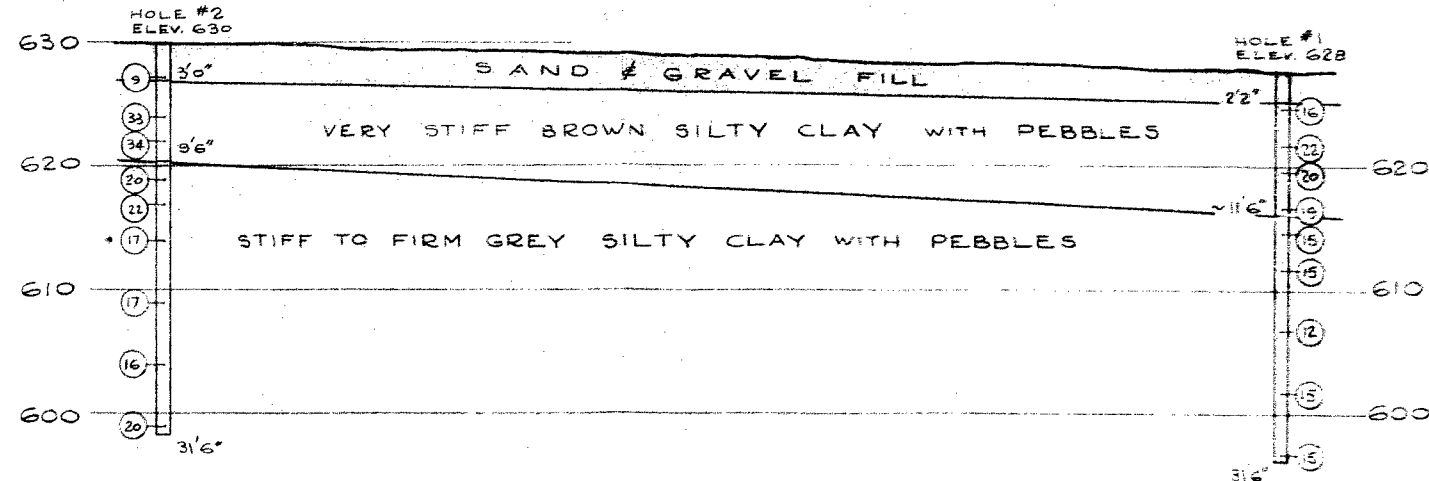
DEFECTS IN NEGATIVE DUE TO
 CONDITION OF ORIGINAL DOCUMENT

Test Hole Terminated at 31 ft 6 in



SKETCH SHOWING BOREHOLE
LOCATIONS

SCALE: 100' TO 1"



SECTION ON HOLES 2 #1

SCALE: 10' TO 1" (NATURAL)

LEGEND

- - BOREHOLE
- ③ - BLOWS/FOOT (S.P.T.)

NOTE:

SEE BOREHOLE LOGS FOR
COMPLETE SOIL DETAILS.

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



THE TOWNSHIP OF PLYMPTON
c/o J.A. MONTEITH ASSOCIATES LTD.

STEWART CREEK BRIDGE.

PREPARED BY:
e.m. peto associates ltd.

JOB No. 6390 | JUNE 1963 | DWN. BY W.G. | CHECKED BY: HJ