

63 - F - 246 M

OTTER CREEK BRIDGE

LOT 24, CON. V

SOMBRA TWP.

Mr. K. L. Kleinsteinber,
Municipal Bridge Liaison Engr.,
Bridge Division.

Foundation Section,
Materials & Testing Div.,
Room 107, Lab. Bldg.

Attn: Mr. G.C.E. Burkhardt,
Mun. Bridge Checking Engr.

March 10, 1965

Your Memo -- Mar. 10/65

County of Lambton,
Bridge over the Otter Creek,
Township of Sombra,
Lot 24, Con. V,
Sombra-Chatham Gore Townline,
Structure Site No. 13-28,
Your File No. BA 1965

With reference to the above-mentioned report and your memo of March 10, 1965, we would like to submit the following comments for your consideration:

The subsoil conditions are rather unfavourable. According to the test carried out, the shear strength of the subsoil below the footing elevation is between 500 and 1,000 p.s.f. We feel that an allowable pressure of 1,500 p.s.f. is all that can be permitted. Apparently, this is what the bridge consultant has used in his design.

If the proposed changes regarding the wing walls can be carried out in such a manner that the load on the footing is not increased and the footing is not widened (because widening would tend to increase settlements), then such alteration should be carried out.

AGE/Mdef
Encl.

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office
Gen. Files

P.S. -- Returned herewith is Drawing by
J. A. Monteith Associates Ltd.

MEMORANDUM

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

FROM: Bridge Division,
Downsview, Ontario.

DATE: March 10, 1965.

OUR FILE REF.

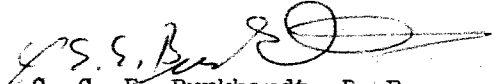
IN REPLY TO

SUBJECT: County of Lambton,
Bridge over the Otter Creek,
Township of Sombra,
Lot 24, Con. V,
Sombra-Chatham Gore Townline,
Structure Site No. 13-28,
Our File No. BA 1965.

Attached please find one copy of the Foundation Report, by E. M. Peto Associated Limited, one copy of the plan with our comments and copy of a letter from the designer.

We would appreciate it very much, if you could review the report, especially in respect of our proposed change in footing lay-out, at your earliest convenience.

GCEB/1m


G. C. E. Burkhardt, P. Eng.,
Municipal Bridge Checking Engineer.

B.H. 1765

E. M. PETO ASSOCIATES LTD.

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

Job No. 6330

June 10th, 1963.

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

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

Re: Subsoil Investigation,
Otter Creek Bridge,
near Wallaceburg, Ontario.

Gentlemen:

We enclose five copies of our Report No. 6330 on the above
site investigation.

The subsoil consists of a silty clay with pebbles, which
possesses a fair strength near the ground surface but becomes progressively
weaker with depth.

A black shale bedrock, which provides an excellent support
for end-bearing piles, commenced in test hole 1 at a depth of 75.4 ft
(elevation 24.4). It is covered, apart from the clay strata, by a 5 ft thick layer
of very dense, shaly silt till.

Whether the bridge should be founded on spread footings or on piles driven to bedrock, will probably be decided on the basis of considerations of economics. The allowable bearing capacity of spread footings in relation to the anticipated settlement is illustrated graphically on page 14. The depth of the footings will depend on scour protection considerations, but the allowable bearing capacity decreases with depth, so that there is a distinct advantage in placing the footings as high as possible. Also, the structure and abutments should be as light as possible.

No test hole was performed in the location of a possible pier, but the subsoil conditions below such a pier are most likely to be very similar to those established in the two test holes.

Should end-bearing piles prove more economic than a footing foundation, steel H-piles would probably be well suited for this site; they would obtain excellent support on the shale bedrock, but, because of the anticipated hard resistance to driving in the 5 ft thick layer of dense shaly silt till immediately above the bedrock, it is recommended to protect the pile toes with reinforcing caps. The elevation of the surface of bedrock is likely to be similar throughout the site, so that the setting level for driven piles can be confidently specified. Care should be taken not to shatter the shale by overdriving.

We consider the report to be complete within your terms of reference; however, we would gladly provide additional assistance should you wish to discuss further any of its contents. We would also welcome an opportunity of re-examining the bearing capacity and settlement aspects, should a shallow footing design be adopted and when the footing levels, dimensions, and design pressure have been decided.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

RK:sb

THE COUNTY OF LAMBTON.

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

SUBSOIL INVESTIGATION

OTTER CREEK BRIDGE
NEAR WALLACEBURG, 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 verbally on February 27th, 1963, by Mr. G. Ingram, of J. A. Monteith Associates Ltd., Consulting Engineers.

An existing bridge on Otter Creek, approximately 5.0 miles east-north-east of Wallaceburg, Ontario, is to be replaced by a larger structure. The site is located on a gravel road, which has an east-west orientation and forms the southern line of Lambton County. Otter Creek is approximately 35 feet wide at the site, narrowing to about 30 ft at the bridge. The average direction of flow is from the north-east to the south-west.

The existing bridge has a span of 30 ft and consists of steel beams simply supported on concrete abutments; it appeared to be in a satisfactory condition.

The new bridge, for which the subsoil investigation was required, is to have a span of 50 ft.

B. GENERAL INFORMATION:

1. Two test holes were performed in the positions indicated on the enclosed site plan, which is based on a site sketch supplied by the Consulting Engineers. The test holes were set out in the field by Mr. G. Ingram, who subsequently provided the ground elevations, referred to the middle of the existing bridge deck as a datum of assumed elevation 100.0. The elevations are entered on the borehole logs, site plan and subsoil profile.
2. Prior to commencement of the field work, we were informed that the new bridge was to have a span of 30 ft, which is the same as the existing structure. Based on this information and on the evidence of satisfactory condition of the present bridge, test hole 1 was initially put down to a depth of 41.5 ft below the existing grade; test hole 2 was terminated at a depth of 23 ft, which was only deep enough to prove the uniformity of the subsoil conditions within the depth influenced by the bridge loads.

This work was performed by our drilling unit No. 7, between March 18th and 20th, 1963.

Subsequent to completion of the above field work, we were informed that the new bridge is to have a span of 50 ft, and not 30 ft as was originally assumed. As laboratory testing of soil samples from the original field work indicated that the bearing capacity, with relation to allowable settlement, may be inadequate for footing foundations for the larger

B. GENERAL INFORMATION:

2. (Cont'd)

structure, it was decided to perform additional field work to determine the position of a dense stratum, capable of supporting end-bearing piles.

The additional field work was performed by our drilling unit No. 3, between May 24th and 27th, 1953. It consisted of reborer test hole 1, without sampling, to the original depth of 41.5 ft, and continuing this test hole with normal sampling procedures, until refusal was reached on the surface of the shale bedrock at a depth of 75.4 ft. The shale was then proved by 10 ft of diamond drilling.

3. The following soil mechanics tests were performed in our laboratory.

- Water content determinations,
- Unconfined compression tests, with volumetric analysis,
- Atterberg limit tests,
- Grain size distribution,
- Oedometer consolidation tests,
- Consolidated, undrained-triaxial compression tests with pore water pressure measurements, for determination of Skempton's pore pressure parameter A.
- Undrained triaxial compression tests, at slow rate of strain, to determine Modulus of Linear Deformation (Young's Modulus).

The water contents are entered on the borehole logs, together with the field standard penetration tests. The remaining tests are included in the Appendix.

C. SOIL CONDITIONS:

Details of subsoil conditions encountered in 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 presented on the drawing and indicates the inferred contact levels between the various subsoil strata.

The distribution of main geotechnical properties of the subsoil with depth is included on Fig. 1, where the in-situ water content and Atterberg limits, standard penetration test resistance, undrained shear strength and bulk density are plotted against elevation.

The following main soil types were identified at the site:

- Crushed stone fill
- Firm to soft silty clay with pebbles
- Very soft clay
- Dense shaly till
- Black shale bedrock.

Each of the above soil types will now be described in turn.

a) Crushed stone fill

An 3 inch thick layer of crushed stone and sand fill was encountered below the existing grade in both test holes, forming the shoulders of the gravel road.

C. SOIL CONDITIONS: (Cont'd)

b) Silty clay with pebbles

This deposit, of lacustrine character, forms the main overburden over bedrock in the area. It was found to extend to a depth of 60.5 ft, where it was followed in turn by a 10 ft thick layer of very soft highly plastic clay, 5 ft of till and the shale bedrock.

The silty clay stratum had the following properties.

1. Appearance

The clay was dark brown to mottled brown and gray to a depth of 5 ft below the existing grade and within this depth was of soft to firm consistency. The upper layers were a man-made fill forming the existing embankments.

Between a depth of 5 and 8 ft, the clay was firmer and of light brown and grey colour; it was also more silty, but included plastic clay layers and was very fissured by desiccation.

Below a depth of about 8 ft, in both test holes the colour changed to a dark grey, which continued to the bottom of the deposit. Below the mottled brown crust, the material appeared more uniform and less plastic than in the uppermost layers.

C. SOIL CONDITIONS:

b) Silty clay with pebbles

1. Appearance (Cont'd)

Atterberg limit test results are plotted on Fig. 1 and are tabulated in Table A. Below the stiff crust the clay was found to have quite uniform plastic properties, the average of which were as follows:

Liquid Limit	36%
Plastic Limit	18%
Plasticity Index	18

A typical grain size distribution curve is included on Fig. 3. The sample of clay from a depth of 22 ft included 23% of clay and 40% of silt, the remainder being sand and pebbles. Black shale fragments of angular shape were encountered in several samples, as described on the borehole logs.

2. Consistency and shear strength

As shown on Fig. 1 the water content of the material is quite high in the softer and more plastic layers of the mottled brown and grey crust. The water content reaches a minimum value of 19% to 22% between elevations 82 and 86; below this level, a gradual increase in water content with depth is observable.

C. SOIL CONDITIONS:

b) Silty clay with pebbles

2. Consistency and shear strength (Cont'd)

Compared to the Atterberg limits, the natural water content is in the lower or middle portions of the plastic range of the material.

Standard penetration test results were quite scattered in the upper, fissured layers of the clay, but indicate a gradual decrease in penetration resistance with depth.

The undrained shear strength, assumed equal to one half of unconfined compressive strength was found to drop from over 1000 lb/sq. ft above elevation 84 to under 500 lb/sq. ft at a greater depth.

To enable a more thorough interpretation of the shear strength results, the undrained shear strength was plotted against water content on Fig. 2. All the test results are included and a very good relationship is apparent.

The average curve of undrained shear strength, plotted on Fig. 1, takes also into account the water content - shear strength relationship. This curve has been adopted in bearing capacity calculations.

C. SOIL CONDITIONS:

b) Silty clay with pebbles

2. Consistency and shear strength (Cont'd)

By comparing the measured undrained shear strength with that indicated by the relationship between the plasticity index and the ratio of undrained shear strength to effective overburden pressure, (Also plotted on Fig. 1) it would appear that the upper layers of the deposit, above elevation 70, are over-consolidated but the overconsolidation ratio decreases with depth, approaching unity near elevation 70. From this data it would also appear that the minimum undrained shear strength is unlikely to be less than 500 lb/sq. ft, occurring near elevation 63. The slightly lower values determined below this depth may be caused by sample disturbance.

3. Compressibility

Two oedometer consolidation tests were performed on undisturbed samples of the material, and the results are presented in the form of log pressure - void ratio curves on Fig: 4 a and 4 b. The curves indicate that the material in the field is only very lightly over-consolidated, but it must be observed that results of the tests may be affected by the presence of numerous pebbles in the clay, because of which the samples were probably partly disturbed.

C. SOIL CONDITIONS:

b) Silty clay with pebbles

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3. Compressibility (Cont'd)

On the basis of the test results, the coefficient of volume change, m_v was found to have an average value of 0.017 sq.ft/ton, for a stress increment of 1.0 ton/sq. ft in excess of the estimated existing effective overburden pressure at the depths from which the samples were extracted.

The coefficients of consolidation, c_v for the various load stages of the tests in loading and unloading are tabulated in Table B.

The Modulus of Linear Deformation E, (Young's Modulus) was measured on two undisturbed samples by a special undrained triaxial compression test; the results are tabulated in Table C. An average value of E of 45.0 ton/sq. ft was assumed in the settlement analyses on the basis of these results.

c) Very soft clay

A deposit of much softer clay of high plasticity was encountered in test hole 1 at a depth of 60.3 ft, and extended to a depth of 70.3 ft below the existing grade. Standard penetration tests in this layer resulted in only 2 blows per foot. Water contents of 44.0 and 46.1 were recorded, which is near the Liquid Limit. An undisturbed sample

C. SOIL CONDITIONS:

c) Very soft clay (Cont'd)

recovered from a depth of 63 ft was too soft for compression testing.

An Atterberg limit test was performed on a sample of this material from a depth of 66 ft and the following results were obtained.

Liquid Limit	47.6%
Plastic Limit	22.4%
Plasticity Index	25.2

This layer can be considered as highly compressible and its presence could result in some settlement of a friction pile foundation, supported above it. For this reason, friction piles are not recommended at this site.

d) Shaly till

Dense subsoil was encountered at a depth of 70.3 ft, and had the form of a clayey silt with some sand and with a dense accumulation of angular fragments of black shale. The matrix of the material was plastic and was at a moisture content higher than the plastic limit.

This deposit probably consists mostly of the underlying shale, which has been broken up and redeposited by a glacier.

C. SOIL CONDITIONS: (Cont'd)

e) Shale Bedrock

The black shale bedrock was encountered in testhole 1 at a depth of 75.4 ft (elevation 24.4). The shale was proved by 10.1 ft of diamond drilling, the test hole terminating in it at a depth of 85.5 ft. The core recovery was 91%.

The cores were typical of the Ordovician black shale bedrock which underlies most of Lambton County. The bedrock can be considered an excellent medium for the support of end-bearing piles, the allowable bearing capacity of which would be very high.

D. WATER CONDITIONS:

No measurable seepage of ground water was reported in test holes to a depth of 70 ft, but slow water seepage was observed when the test hole penetrated into the dense shaly silt till and into the shale bedrock.

The phreatic surface in the subsoil probably corresponds to the boundary between the mottled brown and the gray clay, which occurs at the approximate elevation 91.5. This corresponds closely with the water level in the creek, which was at elevation 92.7.

D. WATER CONDITIONS: (Cont'd)

Because of the absence of pervious seams, it appears virtually certain that no free ground water, other than minor seepage, will be encountered in excavations once the infiltration of creek has been prevented.

E. CONCLUSIONS AND RECOMMENDATIONS:

1. Summary of subsoil conditions.

The subsoil consists of a silty clay with pebbles to a depth of 60.5 ft, followed by 10 ft of very soft clay, and by 5 ft of dense shaly till; the solid, black shale bedrock commences at a depth of 75.5 ft (elevation 24.4) and is suitable for the support of end-bearing piles, designed to a high bearing capacity. The clayey overburden possesses a firm brown crust extending to a depth of 8 ft, below which the material becomes progressively softer with depth.

The most favourable design would consist of as light as possible footings placed at the highest possible level, so that the pressure bulb should be mostly confined to the upper, stiffer portions of the clay deposit.

E. CONCLUSIONS AND RECOMMENDATIONS:

1. Summary of subsoil conditions (Cont'd)

Should the limited bearing capacity and the settlement considerations preclude a footing foundation, end-bearing piles driven to the shale bedrock are recommended.

No free ground water was encountered in the upper layers of subsoil, indicating that the excavations for the footings can be performed practically dry.

2. Footing foundations

The level at which the bridge foundations, in the form of spread footings, could be placed will probably depend on the anticipated scour depth and on the distance of abutments from the stream.

As stated earlier, it would be preferable to construct the footings as high as possible, (provided that they are safe from scouring and frost action), so that advantage can be taken of the highest bearing capacity within the stiffer, upper layers of the clay.

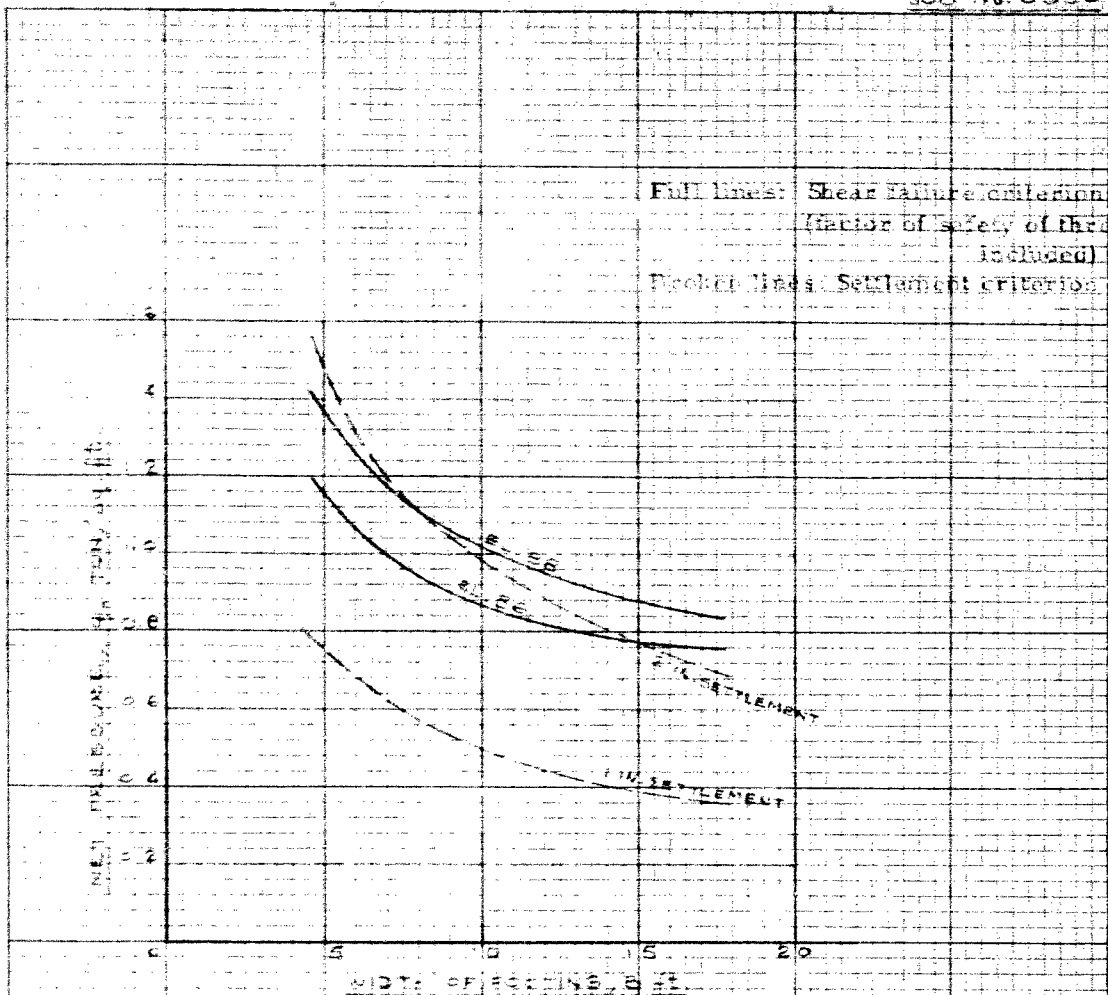


FIG. 6 CURVES OF ALLOWABLE BEARING CAPACITY OF FOOTINGS AT ELEVATIONS 35 AND 35.5 FOR FACTOR OF SAFETY OF THREE (IRRESPECTIVE OF SETTLEMENT).

Net pressure for 1.0 and 2.0 in. total settlement for various footing widths.

Note: See p. 11 for explanation of diagram.

E. CONCLUSIONS AND RECOMMENDATIONS:

2. Footing foundations. (Cont'd)

For the purposes of bearing capacity calculations, the level of footing foundations was assumed to be between 86 and 88. The allowable net bearing capacity for various widths of footings is plotted on page 14 (solid line curves); factor of safety of three against shear failure is included. The overburden component of bearing capacity, equal to the least anticipated weight of overburden above the footing can be added to the net allowable pressure obtained from the diagram.

The diagram on page 14 includes also curves of net pressure increase which will theoretically result in a final settlement of the bridge abutment of 1.0 and 2.0 inches (broken line curves).

Both the shear and the permissible settlement criteria curves assume that the footings will have a length of 30 ft; any likely departure from this assumed length will have a relatively small effect on the allowable pressure and anticipated settlement, since the width of the footing is the more critical variable in the calculations.

E. CONCLUSIONS AND RECOMMENDATIONS

The curves indicate that, for instance, for a 5 ft wide footing, which can be tolerated to settle 1.0 inch, the allowable net pressure increase is 0.75 ton/sq.ft. If a larger settlement than 1.0 inch is considered permissible, the maximum net pressure for footing at elevation 86 will be 1.15 ton/sq.ft, and for footing at elevation 88, it would be 1.35 ton/sq.ft, these pressures being determined by the safety against shear failure criterion. The anticipated settlement in such cases would be obtained by direct proportion of the 1.0 inch settlement at a net pressure of 0.75 ton/sq.ft.

The settlements were calculated by the method proposed by Skempton and Bjerrum in "A Contribution to Settlement Analysis of foundations on Clay", Géotechnique 1957. By this method the settlement is assumed to consist of two independent parts, namely the immediate or "elastic" settlement, and the long-term or consolidation settlement. The immediate settlement was calculated on the basis of the elastic theory, assuming Poisson's Ratio of 0.5 under undrained conditions. The average value of Modulus of Linear Deformation, E of 45.0 ton/sq.ft, determined experimentally, was employed.

E. CONCLUSIONS and RECOMMENDATIONS - Cont'd

The long-term consolidation settlement was calculated assuming that the coefficient of volume change m_v , is constant with depth and has an average value of 0.017 sq.ft/ton. This value was used in conjunction with Skempton's pore pressure parameter A of 0.18, determined experimentally for the corresponding estimated stress range.

The actual values of settlement, for different widths of footings, were found to be as follows:-

Footings of Length 30Ft, between Elevation 86 and 88.
Inches of Settlement per ton/sq.ft net stress increase.

<u>Footing Width, Ft</u>	<u>Immediate</u>	<u>Long-Term</u>	<u>Total</u>
5	1.60	0.55	2.15
10	2.40	0.83	3.23
15	2.98	1.07	4.05

The settlement for a net pressure other than 1.0 ton/sq.ft can be obtained by direct proportion of the above values.

E. CONCLUSIONS and RECOMMENDATIONS - Cont'd

It will be observed from the above table that the immediate, elastic settlement forms by far the major portion of the total theoretical settlement. It is possible that this portion of settlement is over-estimated because of an under-estimate of the value of E, caused by possible sampling disturbance of the very stony clay. Also, it is to be expected that a large portion of the immediate settlement will occur as soon as the abutments or piers are constructed and will be completed by the time the construction of the super structure is commenced. On the logic of these considerations, only one half of the values of immediate settlement given on the above table was considered as affecting the structure and is included in the total final settlement curves presented on page 14 . This halving of the calculated theoretical elastic settlement is purely arbitrary but is considered justified in order to give a more realistic estimate of the potential settlement that may effect the structure as a whole.

3. Pile foundation.

Should the allowable pressure in conjunction with anticipated settlements be insufficient for an economical design of footing foundations, the bridge should be supported on piles.

E. CONCLUSIONS and RECOMMENDATIONS - Cont'd

Friction piles, supported within the silty clay with pebbles stratum, should be designed on the basis of the assumed frictional bearing capacity, equal to the product of the adhesion of clay on piles embedded below elevation 92, and of the perimeter of pile effective in skin friction. The estimated average adhesive strength of clay on piles is 500 lb per sq. ft of pile surface area effective in mobilizing adhesion. A factor of safety of at least two should be included in the bearing capacity of piles, calculated by the above method.

The friction pile solution suffers from the drawback that the silty clay stratum becomes progressively softer with depth and, moreover, a 10 ft thick layer of highly plastic, very soft and compressible clay is present below a depth of 60 ft; some settlement of a friction pile foundation supported above this layer would therefore have to be expected.

In view of the above, it will probably be uneconomical to use friction piles on this project.

E. CONCLUSIONS and RECOMMENDATIONS - Cont'd

End-bearing piles, driven to the shale bedrock, which commences near elevation 24.4, appear preferable. The piles will be relatively easy to drive down to elevation 30, near which a deposit of very dense shaly silt till was found to commence. However, we would recommend that the piles are driven through this deposit a further 5 ft until refusal on the shale bedrock is obtained. The allowable bearing capacity of piles driven to bedrock would be very high and determined by the structural strength of the piles.

A sudden and considerable increase in pile driving resistance is anticipated when the deposit of dense silt till is reached.

4. Excavations

The excavations for footings can probably be performed without any difficulty, since the subsoil is relatively soft and no free ground water was encountered in the test holes. Walls of excavations will theoretically stand unsupported in an almost vertical cut, although normal safety precautions are recommended, particularly in view of the fissured structure of the upper layers of the clay stratum.

E. CONCLUSIONS and RECOMMENDATIONS - Cont'd

Because of the relatively low shear strength of the clay below the footing level, every precaution should be taken not to disturb the subsoil below the footings during excavation and construction operations. The last one to two feet should preferably be excavated by hand, in order to limit the possible disturbance.

The clay should not be allowed to soften due to contact with free water, which must be kept away from the excavations. Should it be unavoidable to retain open footing excavations for any length of time, it is recommended to protect the grade against water by an impervious seal, in the form, for instance, of a thin layer of lean concrete or bituminous coating. Alternatively, the last six to twelve inches of subsoil could be left unexcavated until the last possible moment before construction of the footings.

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

5. Embankments

The subsoil is relatively compressible and appreciable settlement could develop under heavy fill, applied over a large area. However, it is not anticipated that the road grade will be raised significantly at this site.

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

Report prepared by:

E. M. PETO ASSOCIATES LTD.

R. Kulesza

R. Kulesza, P. Eng.

C. F. Freeman

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

RK/ap

Our Job Number 6330

10th June 1963.

APPENDIX "A"

LABORATORY TEST RESULTS

TABLE A

ATTERBERG LIMIT TESTS

S. H. / Sa. No.	Depth, ft.	Liquid Limit, %	Plastic Limit, %	Plasticity Index	In-situ water content %
1 / 8	16	37.0	19.0	18.0	20.1
1 / 12	26	36.5	18.4	18.1	21.6
1 / 18	41	36.7	18.6	18.1	23.7
1 / 24	51	24.9	17.9	7.0	33.3
1 / 29	61	36.9	17.5	19.4	-
1 / 31	65	47.6	22.4	25.2	46.1
2 / 7	18	35.4	18.2	17.2	23.2

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TABLE 'B'

UNCONFINED COMPRESSION TESTS

Hole No.	Sample No.	Depth, ft.	Nat. M. C. %	Densities, p. c. f.		Void Ratio e	% Strain at failure	u/c Shear Strength p. s. f.
				Wet	Dry			
1	7	12'0" - 13'6"	18.8	125.3	105.5	0.60	20.0	2360
1	9	17'6" - 18'0"	20.0	130.4	108.8	0.55	20.0	1050
1	13	27'6" - 28'0"	21.1	129.7	107.0	0.58	20.0	390
1	15	32'6" - 33'0"	27.1	126.5	99.5	0.69	20.0	275
1	17	37'0" - 37'6"	24.5	128.0	102.9	0.64	20.0	356
2	4	10'0" - 11'6"	18.5	130.3	110.0	0.53	20.0	2520
2	5	12'0" - 13'6"	19.9	127.4	106.4	0.53	20.0	1585
2	6	15'0" - 16'0"	21.0					850
1	22	47'6" - 48'0"	25.8	125.7	100.0	0.69	20.0	372
1	23	48'0" - 48'6"	25.9	125.7	100.0	0.69	20.0	534

TABLE 'C'

UNDRAINED TRIAXIAL COMPRESSION TESTSMODULUS OF LINEAR DEFORMATION, E (YOUNG'S MODULUS)(Undrained Shear Strength = C_u)

E. H. / Sal. No	Depth, ft.	Bulk Density lb/cu. ft.	Water Content %	Cell Pressure lb/sq. in.	E ton/sq. ft.	C_u ton/sq. ft.	E/ C_u	Compressive stress range lb/sq. in.
1 / 11	22.5	132.5	22.0	30	47.1	0.36	131	0 - 5.5
1 / 17	37.5	123.5	27.2	45	43.5	0.17	264	0.1 - 2.6

Note: E was obtained from average slope of stress-strain curve loops in successive loading and unloading cycles over the compressive stress range given above. The rate of strain was 0.03 in./min.

TABLE 'D'

COEFFICIENTS OF CONSOLIDATION, c_v

From oedometer tests

<u>B. H. / Sa. No.</u>	<u>Depth, ft.</u>	<u>Load stage tons/sq. ft.</u>	<u>c_v sq. ft./year</u>
2 / 6	15'8" - 15'0"	1/8 - 1/4	51.0
		1/4 - 1/2	29.9
		1/2 - 1	51.5
		1 - 2	39.3
		2 - 4	51.1
		4 - 8	58.4
		8 - 2	142.3
		2 - 1	67.9
		1 - 1/2	35.7
2 / 8	20'3" - 21'6"	1/8 - 1/4	21.9
		1/4 - 1/2	20.8
		1/2 - 1	26.8
		1 - 2	39.7
		2 - 4	51.1
		4 - 8	64.7
		8 - 2	148.7
		2 - 1	72.3
		1 - 1/2	27.0

GEOTECHNICAL SOIL PROPERTIES

APPENDIX

FIG.

1

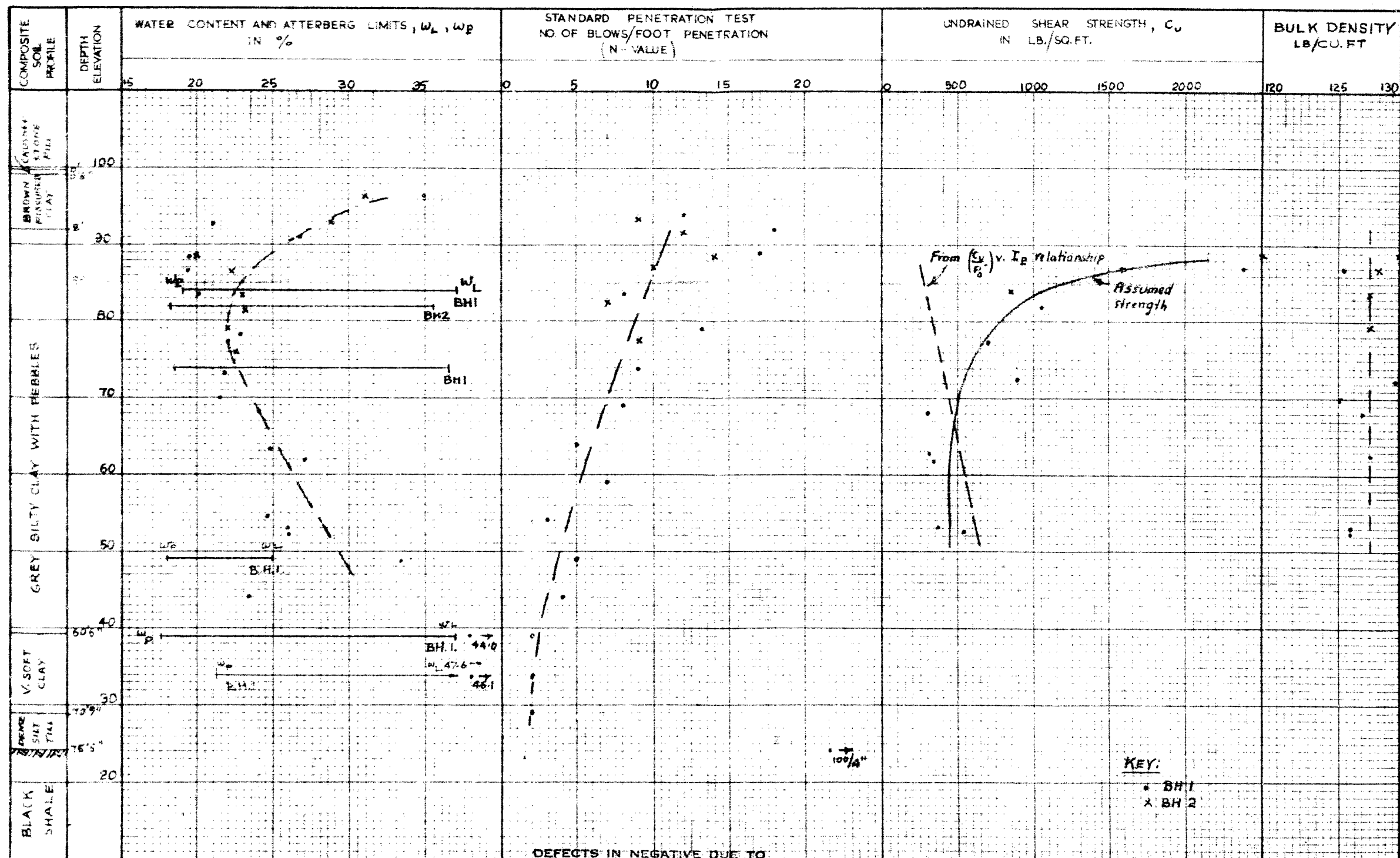


FIG. 2.

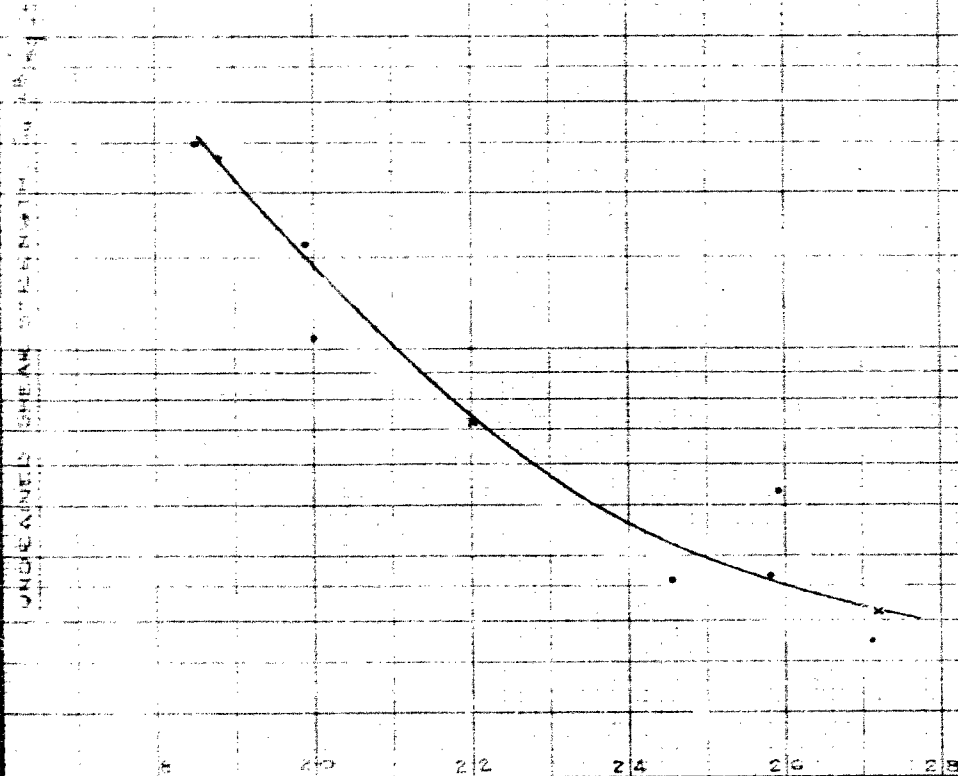
RELATIONSHIP BETWEEN UNDRAINED
SHEAR STRENGTH AND WATER CONTENT

Silty clay with some sand and pebbles

$w_L = 38\%$ $w_p = 13\%$

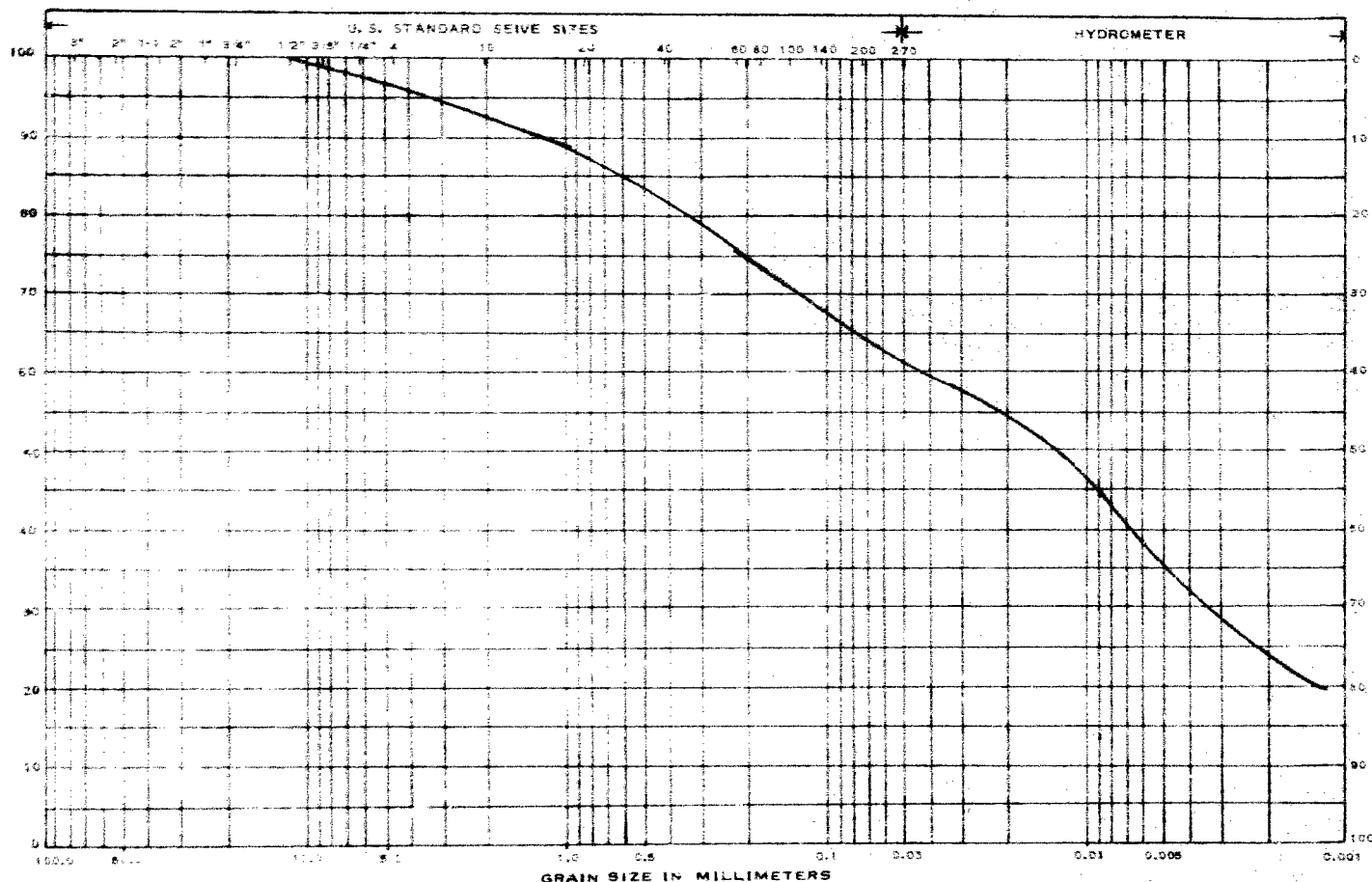
UNDRAINED SHEAR STRENGTH (lb./sq. ft.)

WATER CONTENT IN %



e. m. peto associates ltd.

Toronto 19, Ontario



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

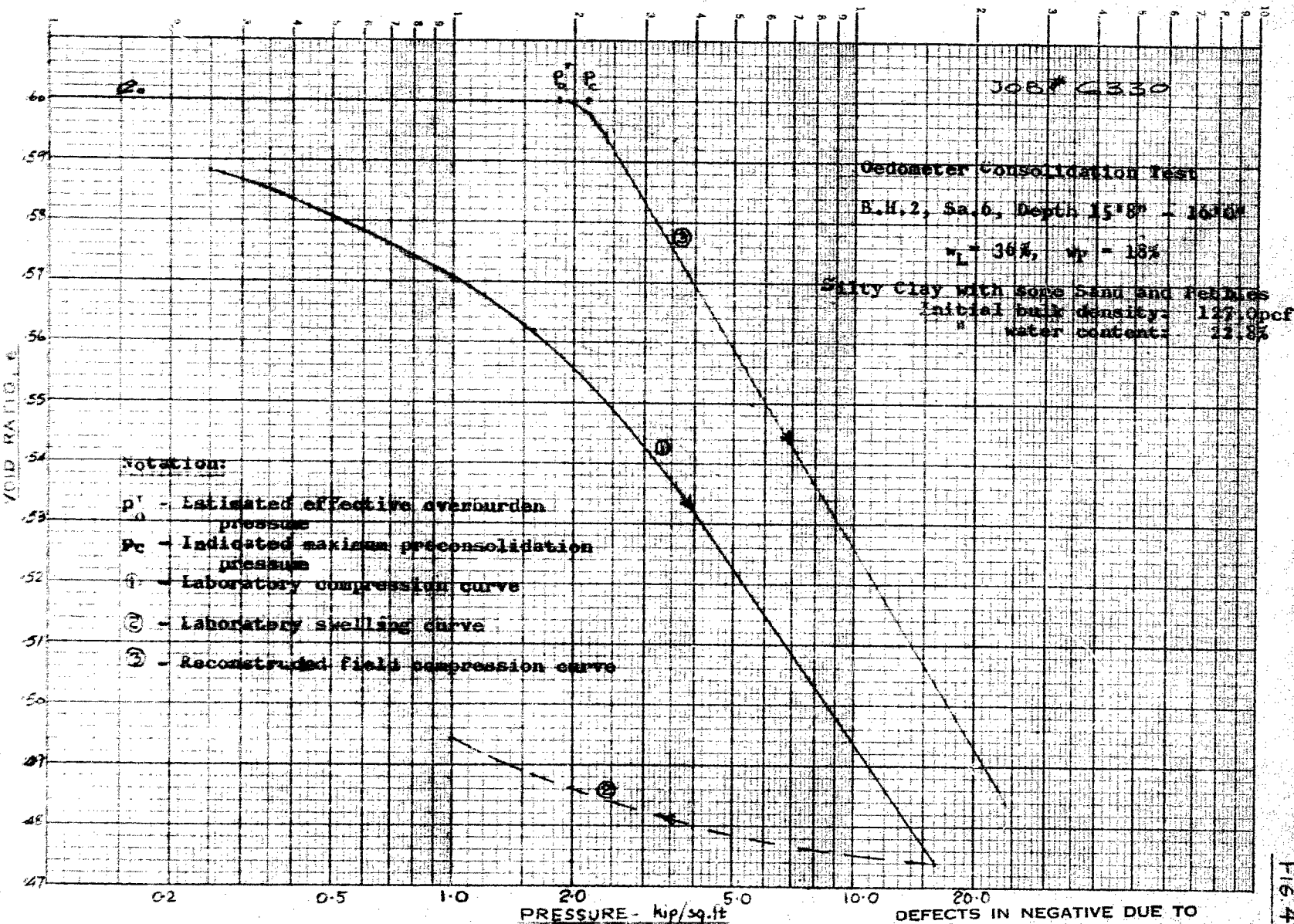
MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Otter Creek Bridge JOB NO. 6330 HOLE NO. 2 SAMPLE NO. 9
 DEPTH 21'6"-23' ELEVATION _____ REMARKS Silty clay with sand and pebbles

GRAIN SIZE DISTRIBUTION

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

E 16.3



DEFECTS IN NEGATIVE DUE TO
 CONDITION OF ORIGINAL DOCUMENT

JOB # 6330

Oedometer Consolidation Test

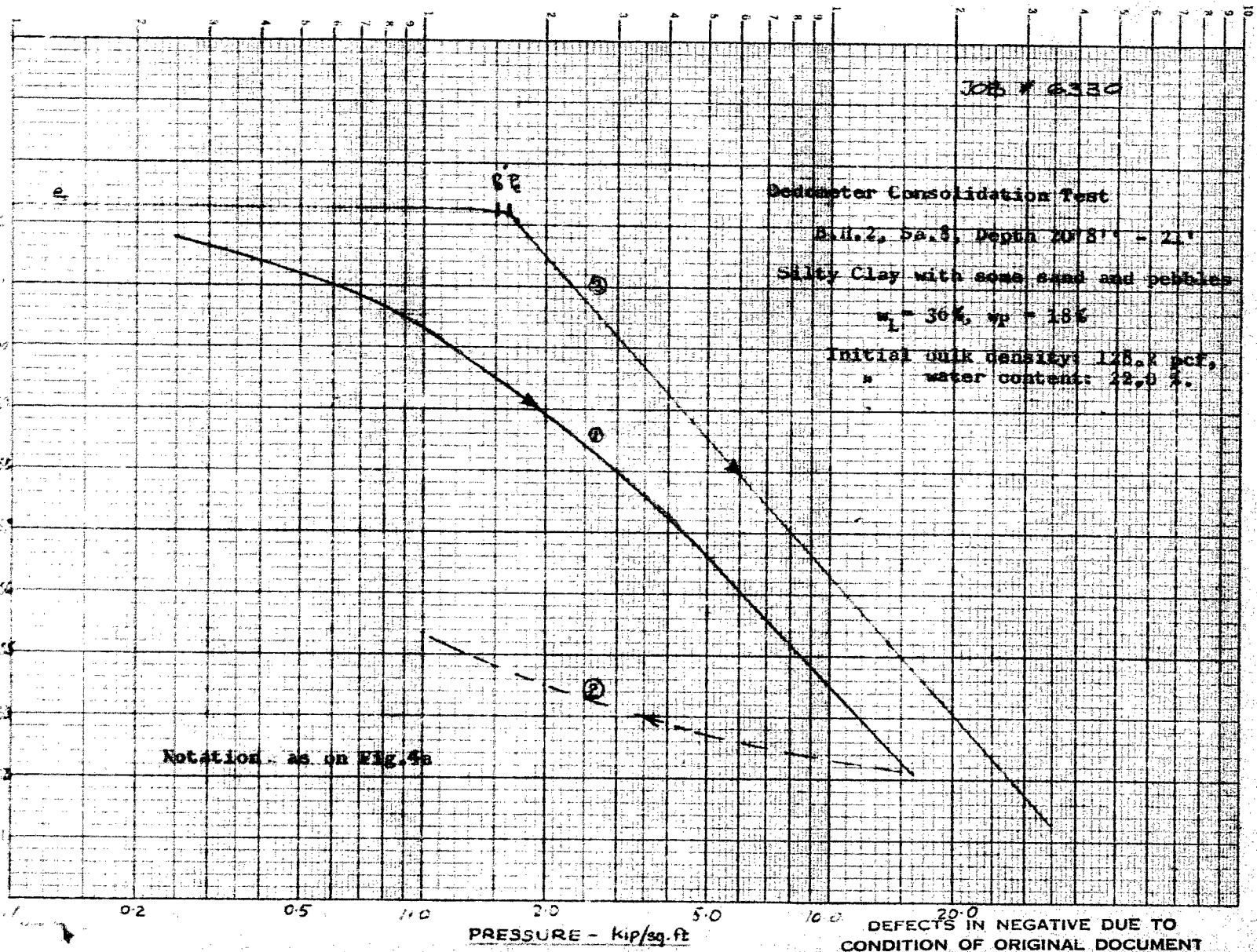
Ball. 2, S.A. 3, Depth 20' 8" - 21'

Silty Clay with some sand and pebbles

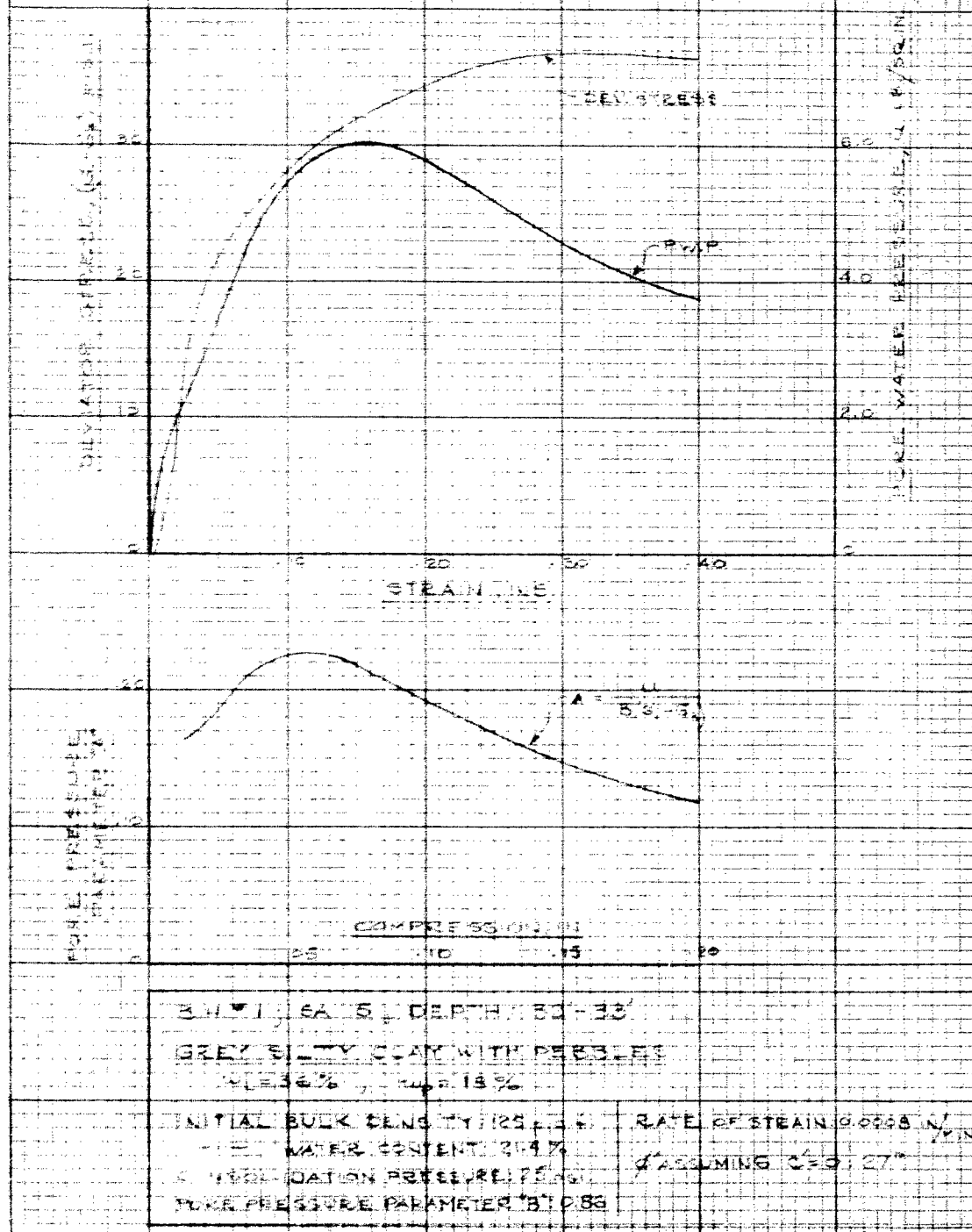
$w_L = 36\%$, $w_p = 18\%$

Initial bulk density: 125.2 pcf,
 water content: 22.0 %

Notation as on Fig. 4b



CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST WITH PORE PRESSURE MEASUREMENT



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SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

BOREHOLE LOG

Job Name Otter Creek Bridge

Job No. 6550

Borehole No. 1

Client County of Lambton

Casing 1" to 2" MX to 15'





Boring Date March 18 & 19, 1963.

Elevation 99.8 (ARBITRARY)

Compiled By K. J.

Checked By A. P.

SAMPLE CONDITION

-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

SAMPLE TYPE

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

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- 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

DEPTH (Feet)	DESCRIPTION	COLOR	Consistency or Classification	Depth Elevation	Sample No. (and Condition)	Sample Type	No. of Blows per Ft.	W.T.P.L.	WATER LEVEL & REMARKS
0' - 8"	Crushed stone fill				1	CS			
	Very fissured silty clay with organic matter	Dk. mottled brown			2	SS	(18)*	24.8	
	Very fissured silty clay	Dk. brown	Soft to firm						* Frozen to 3' - 3", No pebbles below 4 ft
	Partly fissured silty clay	Mottled brown & grey	Firm		3	SS	12		
	Silty clay with pebbles & some sand	Ditto	V. stiff	8' - 5"	4	SS	7 7/6"	21.0	
		Gradually be- coming grey		10' - 0"			11 7/6"		Moist till from 7' - 10"
	Ditto	Dk. grey	Stiff		5	SS	17	19.5	Just W.T.P.L.
	Ditto	Ditto	Ditto		6	SS	16	19.3	W.T.P.L.
					7				
	Ditto	Ditto	Firm		8	SS	8	20.1	Samples include angular fragments of black shale
					9	2" SL			
				20' - 0"					
	Ditto	Ditto	Soft to firm		10	SS	13	22.8	W.T.P.L.
					11	2" SL			
									Fragment of black shale in SL tip
	Ditto	Ditto	Ditto		12	SS	9	21.6	W.T.P.L.
					13	2" SL			
				30' - 0"					
	Ditto	Ditto	Ditto		14	SS	8	24.0	
					15	2" SL			
	Ditto	Ditto	Soft		16	SS	5	24.8	
					17	2" SL			Test hole performed dry, no free water encountered
	Ditto	Ditto	Ditto	41' - 6"	18	SS	7 7/5	23.7	
								23.8	
	Ditto	Ditto	Ditto		20	SS	3	24.6	
					21				
					22	2" SL			
					23				
				50' - 0"					

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

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CONDITION OF ORIGINAL DOCUMENT

Job Name Otter Creek Bridge
County of Lamson
Client J. L. A. Monteith Assoc. Ltd
Elevation 99.5 (ARBITRARY)

Job No. 6330
Casing 4" 10.15 ft
Comp'd By R. K.

Borehole No. 2
 Boring Date March 19 & 20, 1963
 Checked By A. P.

SAMPLE CONDITION



UNDISTURBED

**FAIR**

DISTURBED



LOST

SAMPLE TYPE

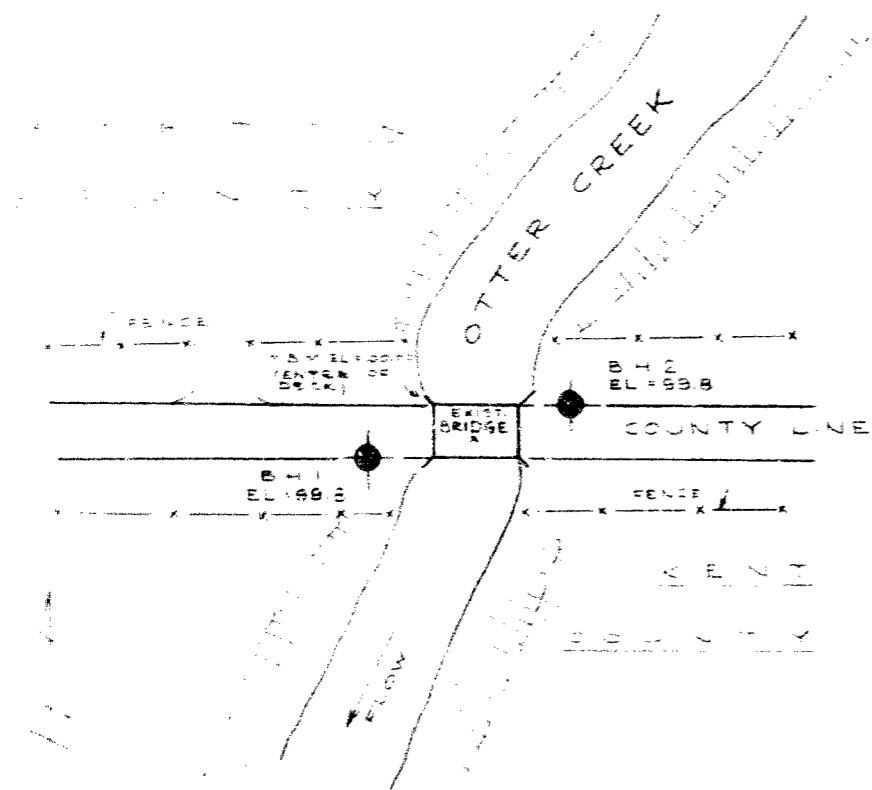
A.S. AUGER SAMPLE
C.S. CASING SAMPLE
S.S. 2" STANDARD SPLIT TUBE SAMPLE
S.L. SPLIT BARREL WITH LINERS
S.T. THIN-WALLED SHELLY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

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.	DRYER THAN PLASTIC LIMIT
A.P.L.	ABOUT PLASTIC LIMIT

SOIL DESCRIPTION	COLOR	Consistency	Depth Elevation	Legend	Sample No. and Depth	Sample Type	No. of Blows per Ft.	W.C. Specific Grav.	WATER LEVELS & REMARKS
Crushed stone fill			0'-0"						
Very fissured silty clay	Dr. brown	Soft to firm	0'-8"		1	SS	(15)	31.1	Frozen to 4'-2" Moist from 4'-6"
Fissured plastic clay	Mottled light grey & yellow- brown	Firm			2	SS	9 2/6"	28.7	
Ditto	Ditto	Ditto	8'-2"		3	SS	6/6"		
Silty clay with pebbles and some sand.	Dr. grey						6/6"	26.7	Softer at 8'-2"
		Firm to soft			4	SS	14	12.5	
Ditto	Ditto	Ditto			5	SS	10	23.1	W. T. P. L.
			15'-0"		6	3'SL			
Ditto	Ditto	Firm			7	SS	7	23.2	
					8	3'SL			Test hole performed dry. No free water encountered
Ditto	Ditto	Ditto	23'-0"		9	SS	9	26.4	

TEST HOLE TERMINATED AT 23 FT



SKETCH SHOWING
BOREHOLE LOCATIONS

SCALE: 50' TO 1"

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.

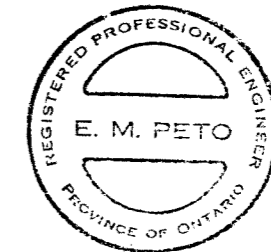
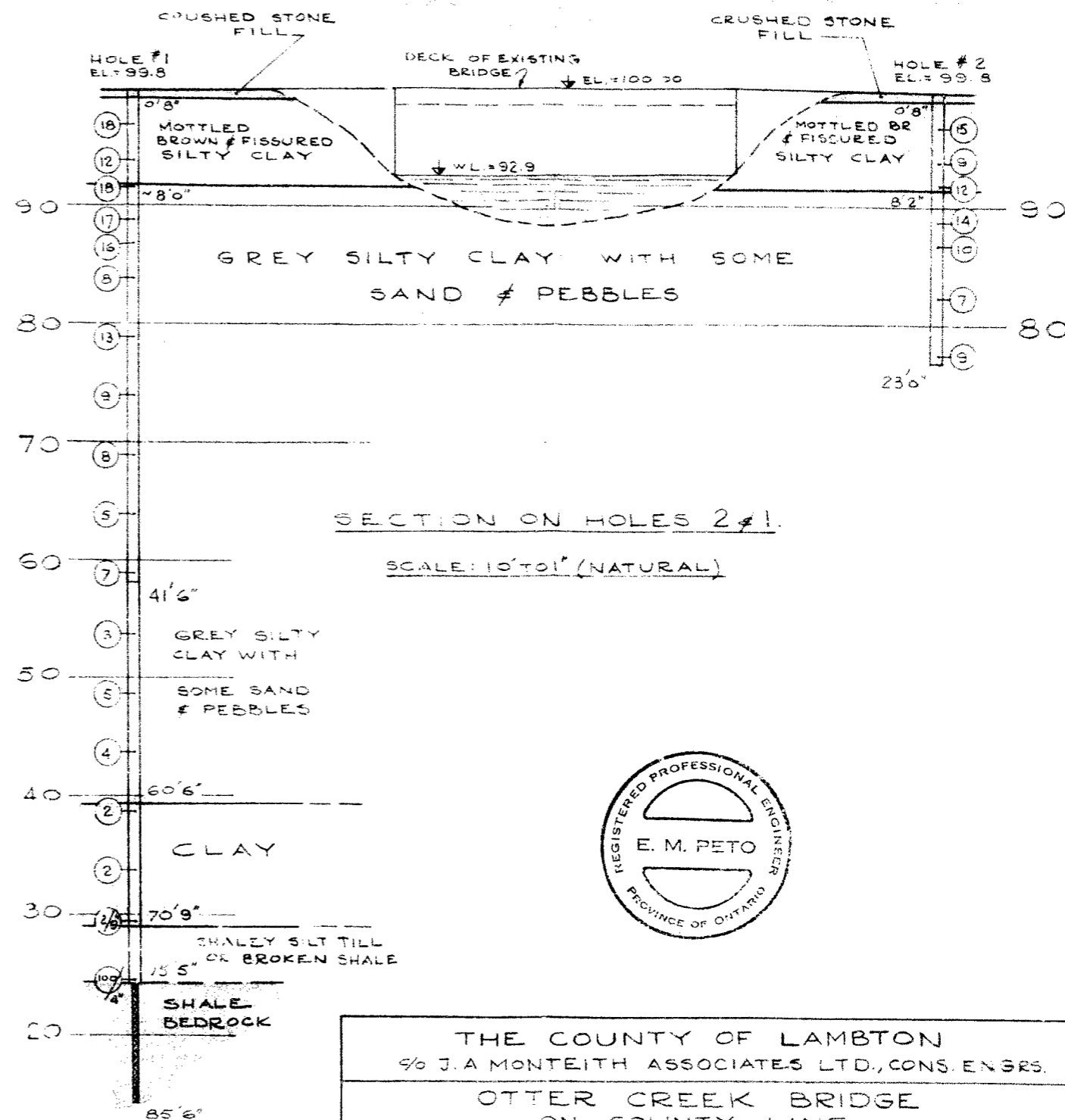
LEGEND:

- BOREHOLE
- ⑤ BLOWS/FOOT (SPT)

NOTE:

SEE BOREHOLE LOGS
FOR COMPLETE SOIL
DETAILS

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT



THE COUNTY OF LAMBTON			
9% J. A. MONTEITH ASSOCIATES LTD., CONS. ENGRS.			
OTTER CREEK BRIDGE ON COUNTY LINE			
PREPARED BY: e.m. peto associates ltd.			
JOB No. 6330	MAY 1963	DWN. BY: W.G.	CHECKED BY: RK