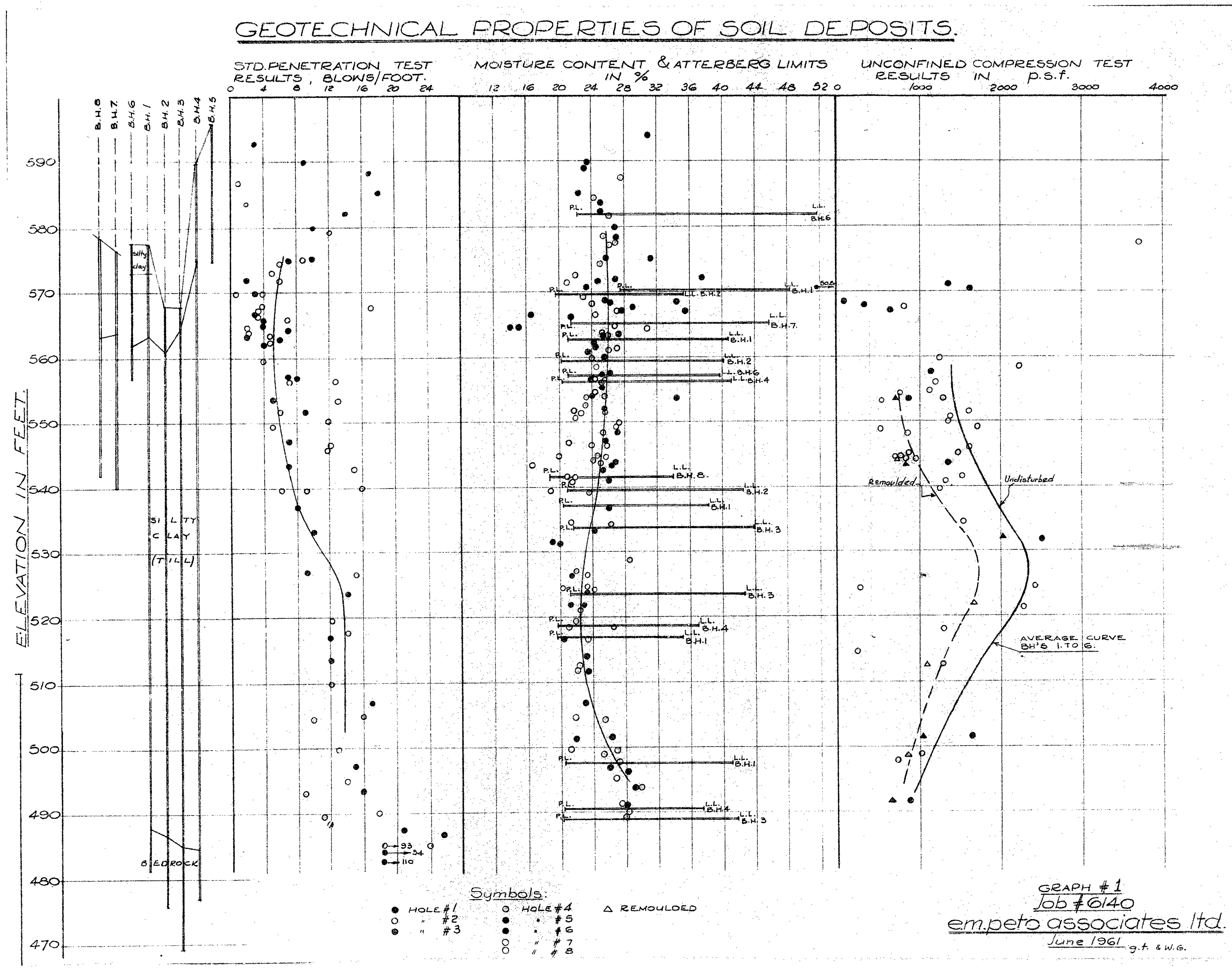
62-F-274 m
BEAR CREEK
BRIDGE #10
MOORE &
SOMBRA TWP.
LINE


GEOTECHNICAL PROPERTIES OF SOIL DEPOSITS.
STD. PENETRATION TEST RESULTS, BLOWS/FOOT.
MOISTURE CONTENT & ATTERBERG LIMITS IN %
UNCONFINED COMPRESSION TEST RESULTS IN p.s.f.
ELEVATION IN FEET.
B.H.8 B.H.7 B.H.6 B.H.1 B.H.2 B.H.3 B.H.4 B.H.5
silty clay
SILTY CLAY (TILL)
BEDROCK
Remoulded
Undisturbed
AVERAGE CURVE BH'S 1 TO 6.
Symbols:
HOLE #1
" #2
" #3
HOLE #4
" #5
" #6
" #7
" #8
REMOULDED
GRAPH #1
Job #6140
em.peto associates ltd.
June 1961
g.t. & W.G.

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A. Stermac
Principal Foundation Eng.,
Materials & Research Section,
Lab. Bldg.,

From: G.C.E. Burkhardt,
Bridge Division,

Date: November 13, 1962.

Our File Ref. #BA1531

In reply to

Subject: County of Lambton,
Bridge over Bear Creek,
Moore Twp., Lot 16, Con I,
Sombra Twp., Lot 17, Con XV,
Structure Site #15-182

Attached please find one copy of the Foundation Report, by E.M. Peto Associates Limited, for your comments.

The proposed bridge is a continuous three-span structure with an overall length of about 160 feet. The superstructure consists of steel beams and concrete deck supported on concrete abutments and concrete piers. The structure is founded on 14BP73 H-Piles, driven to EL-34±.

We would like to approve the plans before the end of this month and we would appreciate it very much, if we could have your comments within the next two weeks.

G.C.E. Burkhardt
G.C.E. Burkhardt,
for K.L. Kleinsteiber,
Municipal Bridge Liaison Engineer.

GCEB/dm

cc: Foundations Office (Rm. 110)

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

Attn: Mr. G.C.E. Burkhardt.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

November 15, 1962.

County of Lambton,
Bridge over Bear Creek,
Moore Twp., Lot 16, Con. I,
Sombra Twp., Lot 17, Con. XV,
Structure Site #15-182.
(Review of Foundation Report
by E. M. Peto Assoc., Ltd.)

We have reviewed the above report, and herewith submit our comments for your consideration:

Bedrock was not proven in this investigation and, therefore, a degree of uncertainty will be involved if end-bearing piles are used. Because of the consultant's experience and findings in this area, it is very likely that his assumptions are correct.

It appears to us that a certain misunderstanding is involved in the recommendation that pile driving be interrupted to allow for the dissipation of built-up pore pressures if driving becomes excessively difficult: An interruption in this particular soil would cause an increase - not a decrease - in bearing capacity and, therefore, interruption of the pile driving operations should not be allowed.

Should there be any other questions you would like to discuss, please feel free to call on our office.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeT

cc: Foundations Office
Gen. Files

BA-1531

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,
Toronto 19, Ontario.

RUssell 9 - 1126.

Our Job Number 62136

September 20th, 1962.

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

62 F 274 M

Attention: Mr. V.W. Ingram, P. Eng.

Gentlemen,

Soil Site Investigation
Township Bridge No. 10,
Bear Creek

We have pleasure in submitting three copies of our Report Number 62136 on the above site investigation. One additional copy has been forwarded directly to Mr. O. van Deurs, Lambton County Engineer.

The allowable bearing capacity under bridge piers, if supported on spread footings, would be limited to 1 ton/sq.ft. As this may prove insufficient for an economical design, we have directed our analysis mainly towards a pile foundation, and both end-bearing and friction piles were considered.

The end-bearing piles would obtain excellent support in the stratum of extremely dense, sandy silt with broken shale, which was encountered at a depth of 115 ft below the existing embankment. The black shale bedrock was not reached, but it can be assumed to commence within 5 to 10 ft below the surface of the extremely dense silty stratum. 2

If the piles are supported at or below the depth of 115 ft, the bearing capacity would be very high, with negligible settlements. However, some difficulty may be experienced in driving the piles through the whole thickness of the clayey till stratum, and preboring mayprove economical.

In our opinion, however, there is no need for end-bearing piles, which would have to be over one hundred feet long. Satisfactory support for the bridge can be obtained by the use of friction piles supported in the clay till stratum, and a suitable level to which the piles should be driven is the zone between elevations 0 and + 20.

Consideration could be given to supporting the piers on piles and the abutments on spread footings.

We believe the Report to be comprehensive within your terms of reference. However, we would gladly provide additional assistance should you wish to discuss further any of thepoints, and we would welcome an opportunity to re-examine the bearing capacity and settlement aspects when a foundation design will be available.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

A. Phillips

for E. M. Peto, P. Eng.

RK/ap

THE COUNTY OF LAMBTON

C/O J. A. MONTEITH & ASSOCIATES LTD.

CONSULTING ENGINEERS

SOILS REPORT

TOWNSHIP BRIDGE NO. 10,

BEAR CREEK

MOORE AND SOMBRA TOWNSHIP LINE

E. M. PETO ASSOCIATES LTD.

1287 Caledonia Road,
Toronto 19, Ontario.

TABLE OF CONTENTS

A. INTRODUCTION

The work described in this report was authorized verbally by Mr. O. van Deurs, Lambton County Engineer.

The site is located on the Moore-Sombra Township Line, approximately half a mile east of No. 15 Side Road, Moore Township. An existing narrow bridge, of steel trusses and of approximately 107 ft span, is inadequate for present day traffic, and the abutments show considerable cracking.

The new bridge is visualized as consisting of three spans, the centre span being 60 ft long and the side spans 48 ft each. According to the information obtained from Messrs. J. A. Monteith Associates Ltd., Consulting Engineers, the total weight of the bridge is anticipated to be 980 tons, of which the abutments will be 200 tons each and the piers 290 tons each. In addition, a total live load of approximately 200 tons is to be allowed for.

On the basis of preliminary information obtained from the test holes, it became apparent that the subsoil under the piers is not capable of withstanding concentrated spread footing loads in excess of 1 ton/sq.ft which could render this type of foundation economic; the site investigation and soil mechanics laboratory work was therefore directed to establishing information required for the design of a piled foundation. However, spread footings are also considered in the report.

B. GENERAL INFORMATION

1. Positions and Depths of Test Holes

Two test holes were drilled at the site, in the positions chosen and set out in the field by Mr. V. W. Ingram, of J. A. Monteith Associates Ltd., Consulting Engineers, who also provided ground elevations at thepositions of the test holes and a drawing showing the site plan and centre line profile(Drawing No. B-1907).

It was believed that the subsoil in the area consists of a stratum of clayey till, of generally firm to soft consistency, and approximately 90 feet thick. The clayey till is overlain by some alluvial deposits and backfill, and rests on top of a layer of extremely dense sandy silt with broken shale, which is approximately 5 to 10 ft thick and is followed by a black shale bedrock. The surface of the shale is known to be approximately horizontal in the area.

On the basis of these assumptions, test hole 1, put down on the west side of the creek, was aimed at proving the silty clay till stratum and reaching the extremely dense silt with broken shale or the shale bedrock. In fact, the till was found to continue to a depth of 115.2 ft below the existing grade; the extremely dense sandy silt with broken shale was encountered at this depth and was proved for a further 1.5 ft, whereupon the test hole was terminated.

B? GENERAL INFORMATION - Cont'd

Test hole 2, performed on the east side of the river, was taken only sufficiently deep to prove a uniformity of upper portions of the silty clay till stratum, and was terminated at a depth of 61.5 ft below the existing grade.

Both test holes were located a short distance towards the centre of the bridge from the proposed new abutments. The Client did not require test holes at the positions of the piers.

2. Ground Elevations

Ground elevations at the existing grade were measured by the Consulting Engineers, and supplied to us on Drawing No. B-1907.

The ground levels at test holes 1 and 2 were 82.4 and 78.8 respectively. These elevations are referred to a temporary bench mark, of assumed elevation 100.0, the position of which is known to the Consulting Engineers.

3. Drilling Operations

The field work was carried out by our drilling unit No. 6 between July 30th and August 8th, 1962.

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

B. GENERAL INFORMATION - Cont'd

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

4. Soil Testing

a) Field Tests

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

The penetration resistance and moisture content distribution is plotted against elevation, together with other geotechnical properties of the subsoil, on the enclosed drawing.

Vane tests were performed in test hole 1 at a depth of 50 ft and in test hole 2 at a depth of 38 ft, and the results are included in Appendix "B". Within this approximate depth, the clay was found to be sufficiently free of pebbles; lower down, however, the pebble content was generally high, the pebbles frequently exceeding 1 inch in diameter, so that vane tests would probably overestimate the true shear strength of the clay.

B. GENERAL INFORMATION - Cont'd

b) Laboratory Tests

The following tests were performed in the soil mechanics laboratory:

Moisture contents determinations
Atterberg Limits,
Particle size distribution,
Unconfined compression tests, undisturbed and remoulded, with volumetric analyses,
Undrained triaxial compression tests for determination of Young's Modulus,
Consolidation tests.

The soil identification tests were required in order to determine to what extent the silty clay till stratum can be considered as uniform and what reliance can be placed on the established shear strength profile when calculating the bearing capacity of piles.

The results of the above tests are included in Appendix "C", while the variation of the main geotechnical properties with depths is plotted on the Drawing.

C. SITE and GEOLOGY

The site of the proposed Township Bridge No. 10 is located on the Moore-Sombra Township Line, approximately one half mile east of No. 15 Side Road, Moore Township. The site is located approximately 4.5 miles south-west of Bridgen.

The Bear Creek, which flows in a northerly direction at the site, is approximately 75 feet wide and 3 ft deep at low water; (low water elevation 64.0 ±); under flood conditions, the high water level rises to elevation 79.0 ± , which is almost level with the existing bridge deck.

The ground rises to the west of the existing bridge, while to the east the grade falls several feet and the road swings to the south. The terrain, particularly to the east of the bridge, is densely wooded.

The road on which the existing bridge is situated has an unimproved, dirt surface.

Geologically, the area is located within the St. Clair Clay Plain, where glacial processes have deposited a mantle of clayey till over a shale bedrock. A layer of extremely dense silt and sand with broken shale intervenes between the clay till and the bedrock, and frequently contains some natural gas. The deepest test hole, No. 1, was terminated at a depth of 116.7 ft in the dense silt stratum, and it can be assumed that the shale bedrock is located only a short distance further down.

D. SOIL CONDITIONS

Details of the soil conditions found in the test holes are described on the appended borehole logs. A simplified subsoil profile is plotted on the enclosed drawing, in the form of a section through the two test holes.

The subsoil can be divided broadly into the following strata, in the order of occurrence.

(a) Silty clay fill
(b) Layers of brown silty clay, or sandy and clayey silt
(c) Grey silty clay with pebbles (clay till)
(d) Extremely dense sandy silt with broken shale.

By interpolating between results of test holes, as on the enclosed soil profile, it would appear that the creek bottom is located near the top of the grey siltyclay stratum. However, it is very likely that, in actual fact, several feet of the material have been eroded by scouring below the creek bottom and replaced by stream-carried deposits of sand and gravel, which may also contain organic matter and mud. The depth of such layers is unknown as no test holes were performed at the creek itself, but judging from our experience on other sites, the exact thickness of such deposits is not important.

D. SOIL CONDITIONS - Cont'd

The geotechnical properties of the various soil types listed above will now be described in turn. The properties of the grey siltyclay till were studied in some detail, since this stratum would be required to support friction pile foundations.

a) Silty clay fill

Both test holes were put down from the shoulder of the existing embankments, which were found to be built up of a silty clay with somepebbles, obviously derived from the deposit of silty clay till, which forms the main overburden over bedrock in this area.

The depth of the fill appeared to be approximately 6.3 ft in test hole 1 and 10.7 ft in test hole 2. However, the exact boundary between the fill and the original surficial weathered layers of subsoil is difficult to determine, because of a similar appearance of the two materials.

The fill was of a brown or mottled brown and grey colour and of generally firm or stiff consistency. Standard penetration test results were in the range of 16 to 22 blows per foot. Water contents ranged from 16.4% to 23.5%. The material was very weathered and fissured, and contained some organic matter in the fissures. The fissures were formed by cracking during the drying cycles.

D. SOIL CONDITIONS - Con't'd

b) Brown to brown-grey, silty clay or sandy and clayey silt

The above described fill was found to rest on top of a stratum of silty clay with some pebbles, which forms the upper, desiccated crust of the clay till stratum, or on layers of sandy and clayey silt with someplant roots and layers of clay, which may have originated as marsh deposits on the side of the creek. Generally, these layers were variable. They extended to a depth of 22.7 ft (elevation 59.7) in test hole 1, and to 17.8 (elevation 61.0) in test hole 2; the grey silty clay till followed at the above elevations.

Because of variability of the surficial deposits, the description of individual soil samples entered on the borehole logs should be consulted. Generally, the density was firm or stiff, with standard penetration test results in the range of 13 to 18 blows per foot. The exception was a much softer layer in test hole 2, between the depth of 14.7 ft and 17.9 ft, where a layer of very soft sandy clay was encountered, with a moisture content of 25.6% and 4 blows per foot standard penetration resistance.

An Atterberg Limit test was performed on a typical sample of the brown clay from a depth of 13 ft in test hole 1, and the following results were obtained:

Liquid Limit: 50.7% Plastic Limit: 22.4% Plasticity Index: 28.3

D. SOIL CONDITIONS - Cont'd

The natural moisture content was in the range of 21.6% to 28.1%, i.e. in the lower part of the plastic range of the material, corresponding to a generally firm or stiff consistency. The liquid limit, however, was considerably higher than of the underlying, main body of this silty clay till stratum, and may have been caused by chemical action associated with surface water percolation through the fissures to the water table, and by the organic content of the material.

c) Soft to firm grey silty clay with pebbles (Clay till)

This material forms the main overburden over the black shale bedrock in the area. It was encountered at the depth of 22.7 ft in test hole 1 (elevation 59.7) while in test hole 2 it was reached at a depth of 17.8 ft (elevation 61.0).

The above level of surface of this stratum marks the boundary with the overlying desiccated portion of the clayey till, of mottled brown colour and generally firmer consistency. It corresponds roughly to the low water level in the stream, and the boundary between the brown and the grey clay can be assumed as the phreatic surface in the subsoil.

D. SOIL CONDITIONS - Cont'd

The silty clay till was found to continue to a depth of 115.2 ft in test hole 1 (elevation 32.8), where it was followed by the extremely dense sandy silt with broken shale, which is known to overly the shale bedrock. Test hole 2 was terminated in the grey silty clay till at a depth of 61.5 ft and its lower boundary was not reached. However, from our experience at other sites in the area, it can be concluded that the thickness of the clay till (approximately 93 ft) is uniform at the site.

The main geotechnical properties of this stratum, as affecting the design of bridge foundations will now be described in turn.

1. Composition, Plasticity and Consistency

Although the grey silty clay till stratum can be considered, from the geological point of view, to form a single entity, in fact zones with somewhat varying characteristics were identified.

The variation of theproperties with depth can, to some extent, be observed on the Drawing, where standard penetration test results, water content and Atterberg Limits, and the undrained shear strength have been plotted against the elevation. The following zones in the silty clay till were identified.

D. SOIL CONDITIONS - Cont'd

(i) In the upper portions of the stratum, i.e. between elevations +60 and +40, the material was generally of soft to firm consistency, with standard penetration test results in the range of 8 to 15 blows per foot and a moisture content varying from 20.5% to 28.6%. In this zone the clay contained grits andpebbles, some up to 1.5 inches in diameter; however, the pebble content appeared to decrease with depth. Three typical grain size distribution curves, plotted on Fig. 1a, indicate that the material contains approximately 50% of clay, the remainder being mainly silt with some fine sand.

The plastic properties are illustrated on the Drawing: the Liquid Limit was between 40.5% and 42.5% and the Plastic Limit between 19.4% and 20%.

(ii) Between the elevations +40 and approximately +20, standard penetration test results were found to increase with depth, the average rising from about 12 to 20 blows per foot; at the same time, there is a tendency for a decreasing moisture content, although the scatter of results is very considerable. The plasticity properties are approximately similar as in the higher zone.

However, this layer appeared to contain very few pebbles; also, the structure in this zone was laminated, and some thin silt seams were observed.

D. SOIL CONDITIONS - Cont'd

(iii) Between the approximate elevations +20 and -25, the clay again contained pebbles, and was somewhat less plastic, as shown by the decrease of Liquid Limit with depth. A general tendency for an increase in standard penetration resistance with depth is observable, although the results are very scattered; however, the average of the results reaches about 30 blows per foot at the elevation -25. Grain size distribution curves on two typical samples of the material containing more pebbles and less clay are included on Fig. 1b, Appendix "B".

(iv) Between the approximate elevation - 25 and bottom of the stratum at - 32.8, the clay was found to be again more plastic, (Liquid limit rising to 41%) and was again laminated and contained fewer pebbles. The moisture content in this lowest layer was found to increase considerably to as high as 32.9% and 34.0%, while the standard penetration test results fell to 22 blows per foot.

The above variation ofproperties with depth was probably caused by different deposition environment. In addition, a scatter of test results within each of the above broadly defined zones indicates a generally heterogeneous character of the stratum.

D. SOIL CONDITIONS - Cont'd

2. Shear Strength

The undrained shear strength of the silty clay till at various depths was measured by unconfined compression tests, the results of which are tabulated in Appendix "B". The results are also plotted against elevation on the Drawing; a very considerable scatter is observed and between the elevations 61 and 20 the shear strength is in the range of 350 and 1300 lb/sq.ft, though the lower limit of most of the results is 900 lb/sq.ft.

It could be expected that the undrained shear strength profile would be similar to the variation of standard penetration test resistance, water contents and plasticity, as was described in the previous section. However, due to the scatter of the shear strength test results, no distinct pattern of distributionof the strength with depth is observable above the elevation 20. Below this level, there appears to be a consistent increase in strength, which would correspond to the observed high standard penetration resistance and lower moisture contents and plasticity, between the approximate elevations+20 and - 15. In this zone, the average undrained shear strength of the clay can be taken as 1000 lb/sq.ft, and this value can be used in calculating the point-bearing capacity of piles driven to elevations+20 to 0.

D. SOIL CONDITIONS - Cont'd

No information is available on the shear strength of the clay in the soft layer between the approximate elevations - 15 and - 32.8, as no undisturbed samples of the clay were recovered from this depth. However, from our amassed relationships between undrained shear strength on the one hand and the water content and standard penetration test results on the other hand for a similar silty clay till in this part of Lambton County, the undrained shear strength of this lowest zone of the stratum can be assumed as 500 to 700 lb/sq.ft.

A number ofunconfined compression tests were carried out on remoulded samples of the clay, and the results are tabulated, together with the corresponding undisturbed strength, in Appendix "B". It will be observed that the sensitivity was generally in the range of 1 to 2. In the lower and less plastic portions of the stratum, where the average undrained shear strength was of the order of 1000 lb/sq.ft, the sensitivity was near unity.

The skin friction component of bearing capacity of piles is calculated on the basis of adhesive strength of clay onpiles. For calculating the adhesion, we adopted the relationship between the undrained shear strength and adhesion of clay on piles proposed by M. J. Tomlinson, as presented in the Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, London, 1957. ("The Bearing Capacity of Piles Driven in Clay Soils").

D. SOIL CONDITIONS - Cont'd

The percentage adhesion and the adhesive strength of the clay are tabulated for each result of unconfined compression test on the test data sheets in Appendix "B".

The average value of adhesion was found to be 639 lb/sq. ft. For the calculationof the skin friction component of pile bearing capacity we propose to adopt the weighted value of adhesive strength of clay of 650 lb/sq.ft throughout the clayey till stratum below the elevation 55 for the case ofpiers and 65 for the case of abutments. Above these elevations, we consider that any adhesive component should be neglected, due to the possible scouring of piles supporting the piers and because of possible tension cracks through the brown, desiccated crust of silty clay under the abutments.

3. Compressibility

As the bridge will probably be supported on piles, we have only carried out consolidation tests on two samples of the silty clay till, from the lower portions of this stratum. The results, in the form of void ratio - log pressure curves, are included in Appendix "B" on Figs. 2a and 2b. From these tests, and on the basis of our amassed values of compressibility of the clay till in the area, it is proposed to employ average values of coefficient of volume change, m_v of 0.0075sq.ft/ ton between eleva-tions + 20 and -15, and 0.015 sq.ft/ton between elevations -15 and the bottom of the stratum, for the calculationof consolidation settlement

D. <u>SOIL CONDITIONS</u> - Cont'd

of the friction pile foundation.

Two triaxial compression tests were performed on samples of the silty clay till to determine the modulus of linear deformation (Young's modulus). The results of the tests are included in Appendix "B". The modulus was 69.5 tons/sq.ft at the depth of 55.5 ft and 110 tons/sq.ft. at a depth of 83.5 ft.

d) <u>Extremely dense silt with fine sand and broken shale</u>

This stratum was reached at a depth of 115.2 ft in test hole 1, (elevation-32.8). It was not reached in test hole 2, which was terminated at a considerably higher level.

This extremely dense stratum, in which a standard penetration test result of 122 blows per foot was recorded, is known to separate the overlying silty clay till from the black shale bedrock. This layer at other sites in the area was found to be roughly 5 to 10 ft thick.

The material is of dark grey to black colour, and consists mainly of silt mixed with fine sand and angular fragments of broken shale; the silt and sand also have the form of powdered shale. A moisture content of 10.1% was recorded in this deposit.

D. SOIL CONDITIONS - Cont'd

It can be assumed that no softer layers exist between this stratum and the underlying black shale bedrock, which probably commences a short distance below the bottom of test hole 1.

E. WATER CONDITIONS

Both test holes were put down from the existing embankment and at some distance away from the creek. The subsoil consisted of clay till of very low permeability and no significant seepage into test holes occurred while the holes were open. The ground water table at the site can be assumed as controlled by the average water level in the creek.

Judging by the change from the desiccated, brown portion of the silty clay till stratum to the grey layers, which commenced between the elevations 59.7 and 61.0, the phreatic surface is located approximately at this level.

The extremely dense sandy silt with broken shale, which underlies the silty clay till stratum, at other sites in the area has been found to contain ground water under a slight artesian pressure, and also some natural gas. At the present site, however, no artesian water was reported, although wash water was used for advancing the test hole below a depth of 78 ft. The wash water interfered with measurement of water level in the very dense silt stratum.

F. ENGINEERING CONSIDERATIONS and CONCLUSIONS

1. Summary of Subsoil Conditions

Both test holes were put down from the existing embankments, and the subsoil was found to consist of a stiff , silty clay fill to a depth of 6.3 ft in borehole 1 and 10.7 ft in borehole 2. The fill was followed by mottled brown and grey silty clay with pebbles, and layers of sandy silt, the density and strength of which progressively decreased with depth, and which extended to a depth of 22.7 ft in borehole 1 and 17.9 ft in borehole 2.

A stratum of soft to firm grey silty clay till followed, commencing near the elevation 61. This material forms the main overburden over the shale bedrock in the area. This stratum was approximately 92 ft thick, and rested on top of a deposit of extremely dense sandy silt with broken shale, which commenced at the elevation - 32.8. The deeper test hole, No. 1, was terminated in the latter material. From information obtained at other sites in the area, it is known that the extremely dense sandy silt with broken shale layer varies in thickness from 5 to 10 ft and rests directly on top of the black shale bedrock, the surface of which is known to be practically horizontal in this region.

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

The average strength of the silty clay till can be taken as approximately 850 lb/sq.ft between the elevations 60 and 20 and 1000 lb/sq.ft between elevations 20 and - 15.

The adhesive strength of the clay for the calculation of skin friction component of pile bearing capacity can be taken as 650 lb/sq.ft throughout the grey silty clay till stratum, i.e. between elevations 61 and - 32.8.

The distribution of geotechnical properties of the subsoil with depth is shown on the Drawing.

The following types of foundations for the new bridge will be considered in turn:

Spread footings,

End-bearing piles,

Friction piles.

2. Spread Footing Foundations

While the footings for the abutments, if located sufficiently far back from the channel, could be placed on the stiff brown crust of the silty clay till, spread footings for the support of piers would have to be located in the upper parts of the stratum of soft to firm grey silty clay till. In order to protect the footings against

F. **ENGINEERING CONSIDERATIONS & CONCLUSIONS**

possible scouring, the footing level would have to be at or below the elevation 55.

Assuming that the shear strength of the silty clay till under the creek is the same as in the test holes located on the banks, the allowable net bearing capacity of footings placed near the elevation 50 to 55 would be approximately 1.0 ton/sq.ft, if a factor of safety of three is employed. However, the clay may be softer under the river.

According to information received from the Consulting Engineers, the weight of the piers would be 290 tons each, while the total live load of the bridge is expected to be approximately 200 tons.

Assuming that the total dead and live load supported by the pier footing amounts to 350 tons, a footing 35 ft long and 10 ft wide or 30 ft long and 12 ft wide would be required to limit the bearing pressure to 1 ton/sq.ft.

Consolidation tests were not performed on samples of clay from the upper portions of the stratum, but from our experience at other bridge sites in this area, the total theoretical settlement of the footing designed with above dimensions and loading can be expected to be of the order of 1.5 inches.

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

Should the Client seriously consider adopting spread footing foundations, we would recommend that additional test holes be performed at the actual positions of thepiers in order to:

i) determine the depth of river-deposited materials, indicating the depth of scouring,

ii) compare the strength of the clay under the river to that measured in the two present test holes performed from the banks,

iii) obtain samples for consolidation tests for the prediction of settlement of the piers.

The additional test holes would need to penetrate only to elevation+30, approximately.

If the abutments are placed sufficiently far from the slope, they could probably be designed for a pressure of approximately 1.5 ton/sq.ft, for footing elevations between 70 and 75, i.e. in the desiccated, brownlayers of silty clay. To this value, an overburden component equal to the least weight of final overburden above the footing level could be added.

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

3. End-bearing Piles

The problem of settlement of the bridge would be practically eliminated if the foundations were supported on piles resting on the stratum of extremely dense sandy silt with broken shale, which commenced in test hole 1 at a depth of 115.2 ft (elevation - 32.8); or if thepiles were driven to the shale bedrock, which is likely to commence not deeper than elevation - 40 or thereabouts. However, because of the extremely dense nature of the sandy silt with broken shale stratum, it is possible that refusal would be obtained in this material, and it would not be necessary to drive the piles to bedrock.

The allowable bearing capacity of piles set at or below the elevation-35 would be determined by the structural strength of the piles themselves.

Because of hard resistance to driving ofpiles through the considerable thickness of the silty clay stratum, preboring could be considered as an alternative to driving the piles through the full thickness of the clay.

If drivenpiles are adopted, steel H-piles would probably be most appropriate, as they would encounter relatively low resistance to driving.

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

4. Friction Piles

(a) Level of Bottom of Piles

From the examination of variation of the geotechnical properties of the clay till stratum with depth, as illustrated on the enclosed Drawing, the most favourable level to which friction piles should be driven appears to be the zone between elevations + 20 and 0. In this zone, the average undrained shear strength of the clay was about 1000 lb/sq.ft and the sensitivity was near unity. Also, the standard penetration tests reached the highest values, while the water contents were lowest, thus providing further evidence that this layer of clay is the strongest.

The strength appeared to increase further between elevations 0 and - 15; however, below - 15 the clay rapidly becomes considerably softer and more compressible. It is therefore considered advisable to place the pile toes at a sufficient distance above the softer layers, so that the pressure bulb below a group of piles partly dissipates before reaching the more compressible material.

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

Above the elevation + 20 the clay was softer, more sensitive, considerably more compressible, and also very variable, as shown by the scatter of test results on the Drawing. A conservative value of shear strength would have to be adopted for the point-bearing component of total bearing capacity of piles driven to above elevation + 20, while between elevations + 20 and 0 the undrained shear strength is more consistent and a greater reliance can be placed on the point resistance.

Driving the piles to a level between elevations + 20 and 0 also appears to offer a satisfactory compromise between an adequate value of allowable load per pile and a reasonable length of pile.

(b) Bearing Capacity of Friction Piles

The total bearing capacity of the piles can be taken as the sum of the point-resistance below pile toes and the skin friction, mobilized along the shafts of the piles.

The point bearing capacity is normally calculated as the product of the undisturbed shear strength of the clay, Cu, at and a short distance below the level of pile toes; bearing capacity factor, N_c (normally taken as 9 for piles); and cross sectional area of pile toe, Ap.

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

Because of the uncertainty as to what is the effective cross-sectional area of steel H-piles, and because of the variability of glacial till materials, in which local soft pockets may be more critical below an H-pile if high stresses are transmitted through a small cross-sectional area, we consider it safer to neglect the point-bearing resistance for steel H-piles in this type of material, and to rely wholly on the skin friction. In the case ofpiles with a large cross-sectional area, however, we consider that the end-bearing component can be included.

The skin friction component is calculated as the product of the surface area of pile effective in mobilizing skin friction and the adhesive strength of clay.

For the case of the piles supporting the piers, we consider that the effective length of pile in mobilizing friction should be assumed to commence at elevation 55, as a precaution against possible scouring below the river bed,which is to be at the elevation 61.

In the case of the abutments, full skin friction may not become mobilized in the desiccated,upper portion of the clay till stratum above the water table and in the fill materials, due to possible tension cracks. We consider it reasonable to assume full mobilization of skin friction below the elevation 65, and to neglect it completely above this level.

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS CONT'D

The effective area of shaft in mobilizing skin friction is taken as the product of pile perimeter, P and the length of pile effective in mobilizing skin friction, D_e, for circular and square piles. For H-piles, we employ the outside perimeter of the pile, i.e. 4 ft for a 12" x 12" H-pile.

The value of adhesion of clay on piles, C_a, was discussed in Chapter D, and it isproposed to use an average C_a of 650 lb/sq.ft.

The allowable load per pile, for a factor of safety F, will thus be:

$$Qa = \frac{1}{F} \times \left[(C_u \times N_c \times A_p + C_a \times P \times D_e) \right] \quad \text{for circular or square piles}$$

$$= \frac{1}{F} \times (C_a \times P \times D_e) \quad \text{for Steel H-piles,}$$

Where C_o = 650 lb/sq.ft,

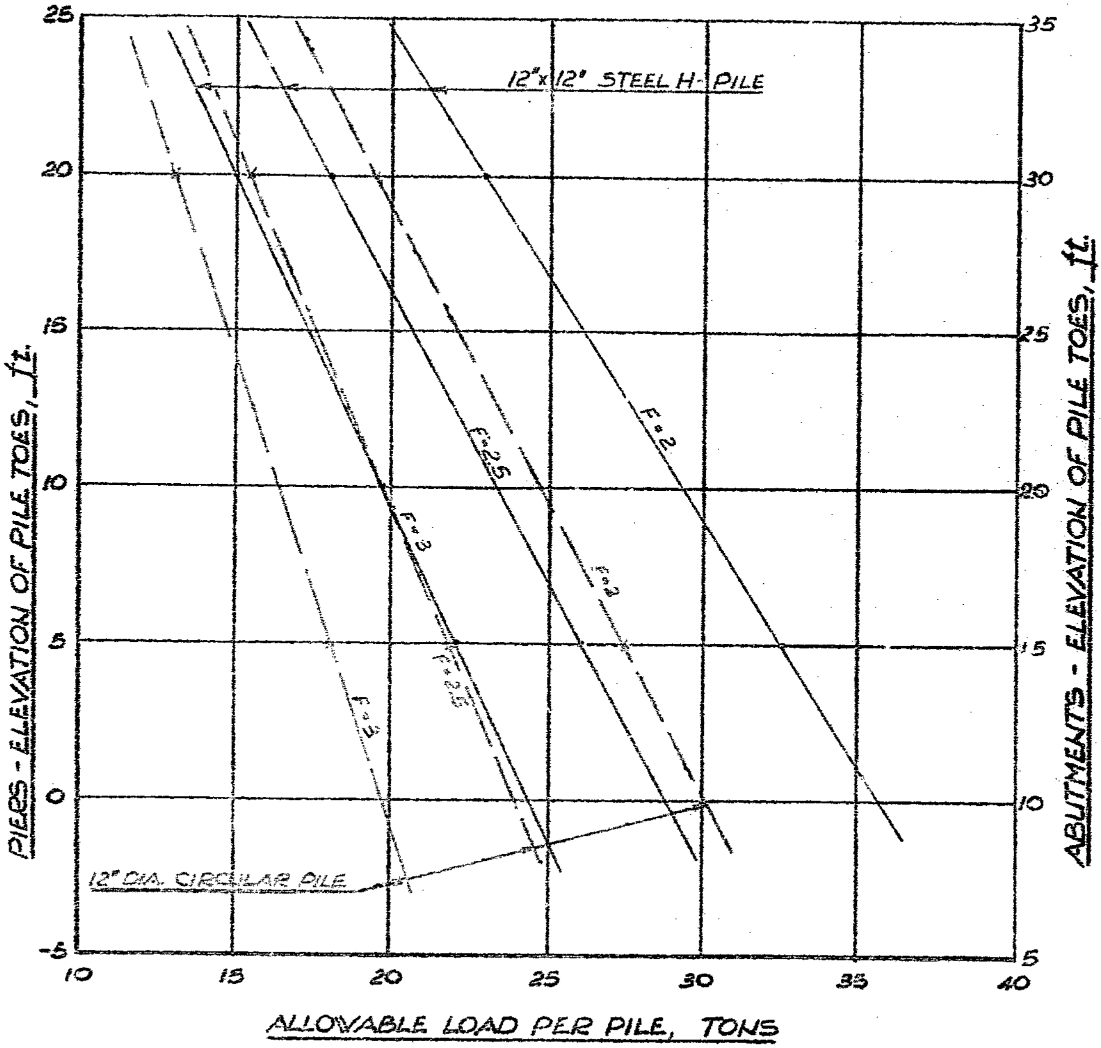
C_u = 1000 lb/sq.ft. between elevations 0 and + 20,

and other terms are as defined above.

Using the above relationships, the allowable load per pile was calculated, for a factor of safety of 2, 2.5 and 3, for the case of:

(i) 12" x 12" steel H-pile

(ii) 12" dia. circular pile



RELATIONSHIP BETWEEN ELEVATION OF BOTTOM OF DRIVEN PILES AND ALLOWABLE LOAD PER PILE, FOR VARIOUS FACTORS OF SAFETY

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

The results arepresented on the diagram on page 28 as a plot of allowable load per pile, against the elevation of bottom ofpiles.

Similar relationships can be obtained for other sizes and shapes of piles, by substituting the actual values of A_p and P in the above equations.

The bearing capacity of a typical pile group, consisting of 12 piles with a load of 30 tons per pile (as for a pier), driven to elevation 0 and having overall dimenstions 20 x 5 ft was examined, and was found to be less critical than for individual piles (a factor of safety of over three was obtained), so that the bearing capacity of individual piles will be the design criterion. However, should the overall dimensions of the pile group be considerably smaller than assumed above, the safety of a group of piles should be re-examined.

We consider that the above method of calculating the theoretical bearing capacity of frictionpiles in the most satisfactory within the present state of development of the science of Soil Mechanics. However, opinions vary considerably on the adhesive strength of clay onpiles, which is known to depend on such factors as stiffness, sensitivity and plasticity of clay, and its pore pressures coefficients, as well as on type ofpile, method and rate of driving, time after driving, spacing

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

of piles and drainage conditions. Even if the shear strength profile of the subsoil has been adequately established, as we consider is the case at the present site, an adequate factor of safety should be included to allow for the various unknown factors, some of which have been listed above.

If no in-situ pile loading test is performed, we consider that a factor of safety of 2.5 should be adopted in pile design. If satisfactory evidence is obtained from a pile loading test, it may be permissible to reduce the factor of safety to 2.

We consider that it would be very useful to perform an in-situ pile loading test at this site. In view of the similarity of the subsoil conditions throughout this part of Lambton County, results of a comprehensive test could be applied to possible other bridge sites in the area, and the expense of the test may well repay in the long run by allowing a more economical pile design.

It is considered that, for the clay till material exhibiting plastic failure characteristics and in which the failure point is difficult to define, a constant rate ofpenetration (CRP) type of test would provide most useful information. This method of testing was recently developed by the British Building Research Station and was described by T. Whitaker and R. W. Cooke in the Proceedings of 5th International Conference on Soil Mechanics and Foundation Engineering, Paris 1961 ("A New

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

Approach to Pile Testing"). In this type of test, the pile is made to penetrate at a constant speed, the force necessary to maintain the constant rate of penetration being continuously measured.

(c) Settlement of Pile Group

The settlement of abutments and piers supported by friction piles, the toes of which are located in the clayey till stratum, will depend on the distribution of stress along the length of the pile, on the spacing of piles and the area and shape of a pile group, as well as on the level of pile toes and soil compressibility. A reasonable estimate of theoretical settlement can be made only after the length and arrangement of piles has been decided.

However, to obtain a rough indication of the order of magnitude of settlement, a computation was made for the case of a 20 x 5 ft group of piles, driven to the elevation 0 and supporting a total dead load of 290 tons.

Assuming that the stress distribution along the length of pile effective in mobilizing skin friction results in a horizontal spread of loading in the ratio of 1:4, horizontal to vertical, the stress increase at the level of pile toes would be 0.16 ton/sq.ft, acting on an area 35 x 50 ft. Assuming the coefficient of volume change, m_v to be 0.0075 sq.ft/ton between elevations 0 and -15, and 0.015 sq.ft/ton between the

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

elevation -15 and -35, and assuming the subsoil to be incompressible below the elevation -35, the total settlement of a pile group was estimated to be 0.3 inches.

As was stated earlier, we would gladly revise this value when the lay-out and depth of piles is decided.

(d) Type and Installation of Friction Piles

Steel H-piles would probably be easiest to drive to the required elevation and, together with their economic advantages, may appear more favourable. However, it is not certain whether a steel pile with an H-section is the most appropriate one to use if its bearing capacity must rely on the mobilization of skin friction in a clayey till. There is a danger of gaps, or voids, forming adjacent to the pile along its embedded length, which can be subsequently filled with water, causing progressive softening of the surrounding clay and a consequent loss of adhesive strength. Such voids may be formed by the transverse vibration of piles during the hard driving operations, by the displacement of stones by the pile toe, and due to air (and water, if under river) being drawn in during the driving. The danger of formation of such gaps appears to be particularly acute in the case of piles with an H-section, due to the large surface area and the presence of sharp corners between the flanges and web.

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

Piles with a circular, or square section, may therefore bepreferable, and there would appear less doubt as to what proportion of surface area is effective in mobilizing friction.

Pile driving conditions are anticipated to be fairly hard, and it may become necessary to interrupt the driving to allow dissipation of pore-water pressures, which may build up during the driving operations and result in increased resistance to driving. The driving would be continued after a few days. 2

It may be worthwhile to prebore the abutment piles through the brown, stiffer and desiccated crust of the subsoil; this would allow easier subsequent driving. The preboring preferably should not be taken down deeper than elevation 60, in order not to reduce the adhesive strength of clay onpiles, which will be relied uponbelow this elevation to support the piles in skin friction.

5. Embankments

Judging from theprofile on Consulting Engineers' Drawing No. B-1907, the new embankment is unlikely to exceed 15 to 20 ft in height to the east of the bridge, while some cutting is anticipated on the west side.

The embankments will be supported on the desiccated, stiff crust of the subsoil and, as far as can be judged from results of

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

the two test holes, there is no apparent danger of instability.

Some minor settlement of the embankment may take place, and it is advisable to remove all organic topsoil and also not to reuse the existing fill, if visual examination indicates that it contains a considerable quantity of organic matter.

If the brige is founded on piles, little settlement is anticipated (particularly in the case of end-bearing piles resting over the bedrock), so that a step could form between the bridge deck and the embankment subjected to some settlement. To avoid this, the embankment fill should be placed before completion of the bridge and allowed to settle. A temporary surface could be placed, and final surface only constructed after most of the settlement has ceased.

Good compaction should be ensured in the fill behind the abutments, to avoid settlement adjacent to the bridge deck.

If, as is likely, the clayey till material, abundant in the area, is used for construction of the new embankments, a granular layer at least 6 inches thick should beprovided under the pavements, to

F. ENGINEERING CONSIDERATIONS & CONCLUSIONS Cont'd

prevent possible damage of road surface by frost heave. The clay till material, on account of its high content of silt, must be regarded as susceptible to frost action.

Report prepared by:

R. Kulesza

R. Kulesza, P. Eng.

E. M. PETO ASSOCIATES LTD.

C. F. Freeman

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

RK/ap

APPENDIX "A"

STANDARD PROCEDURE

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

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

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

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

The Dutch Cone probe test is made by driving the drill rods into the ground with a 2 inch dia. x 60^{0} cone tip. The number of 4200 inch pound blows per foot of penetration are recorded, as in the standard penetration test.

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

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

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

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

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

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

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

APPENDIX "B"

SOIL TEST RESULTS

ATTERBERG LIMIT TEST RESULTS

Job No. 62136

B.H./Sample No.	Depth ft	Liquid Limit %	Plastic Limit %	Plasticity Index	Natural Water Content %
1 / 6	13	50.7	22.4	28.3	28.1
1 / 12	31	40.6	19.9	20.7	26.3
1 / 17	41	41.1	18.5	22.6	28.0
1 / 25	57	43.3	20.9	22.4	25.6
1 / 33	76	37.8	17.0	20.8	20.6
1 / 39	96	34.9	17.1	17.8	19.7
1 / 42	111	41.1	20.1	21.0	34.0
2 / 11	26	42.5	19.4	23.1	26.5
2 / 16	41	39.8	18.8	21.0	26.0
2 / 22	56	37.9	19.5	18.4	22.3

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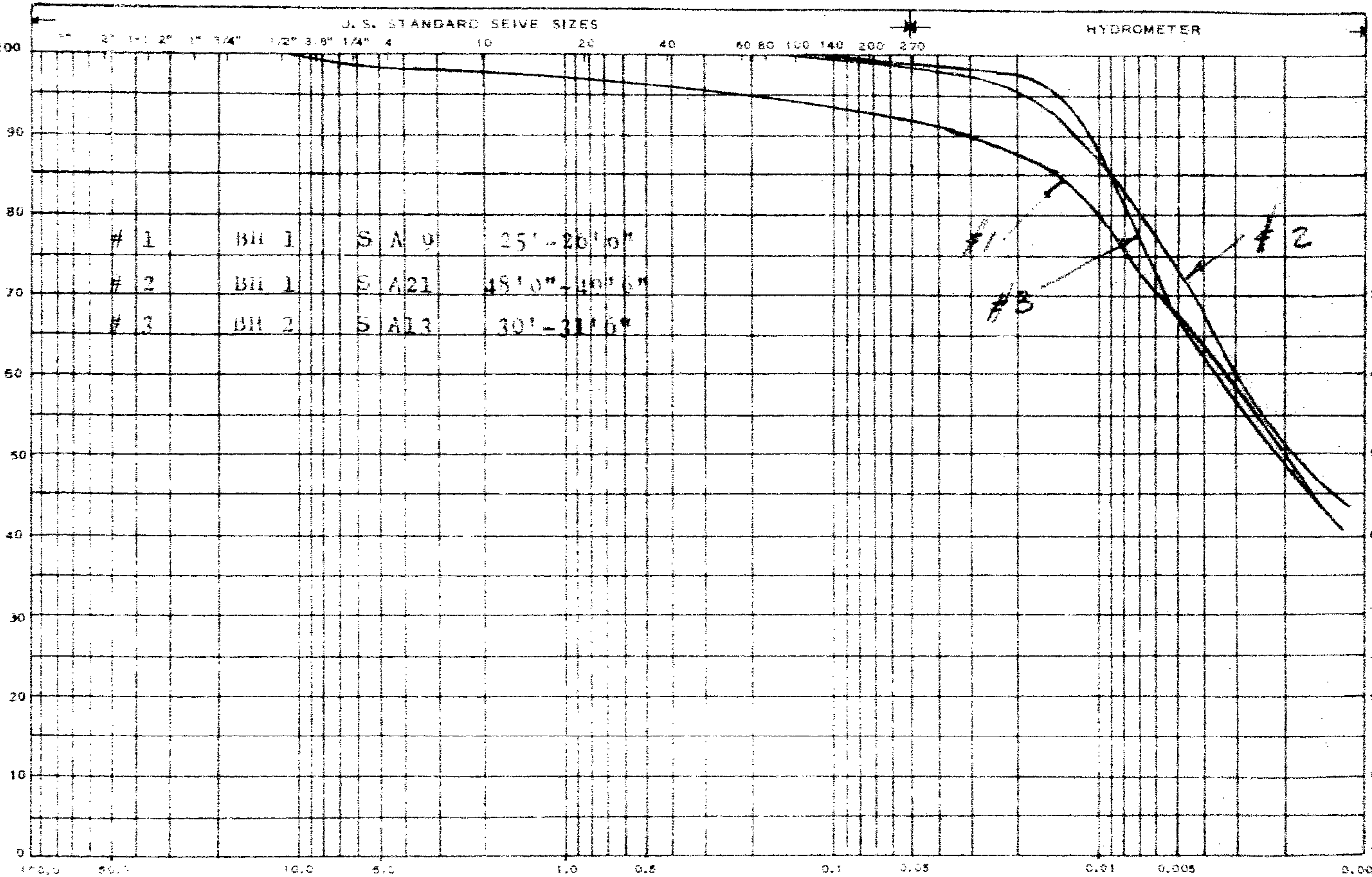

GRAIN SIZE IN MILLIMETERS

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

MASS. INST. OF TECH. CLASSIFICATION

As shown

JOB NAME Township Bridge #10 Bear Creek JOB NO. 02136 HOLE NO. ______ SAMPLE NO. ______

DEPTH ______ ELEVATION ______ REMARKS Soft t[illegible] firm silty [illegible] layers between elevations 55 and 25 [illegible]

GRAIN SIZE DISTRIBUTION

Fig 1a

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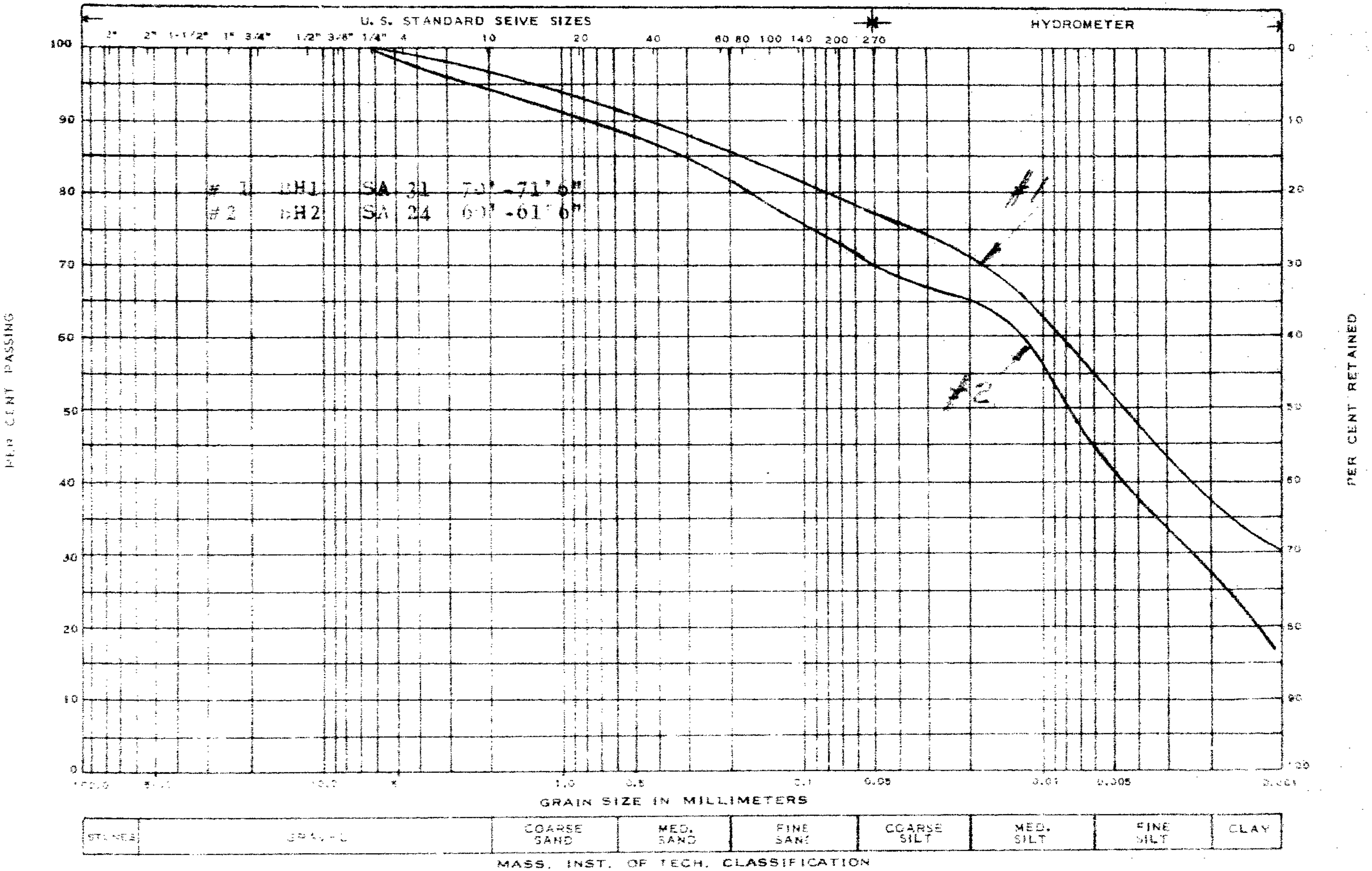


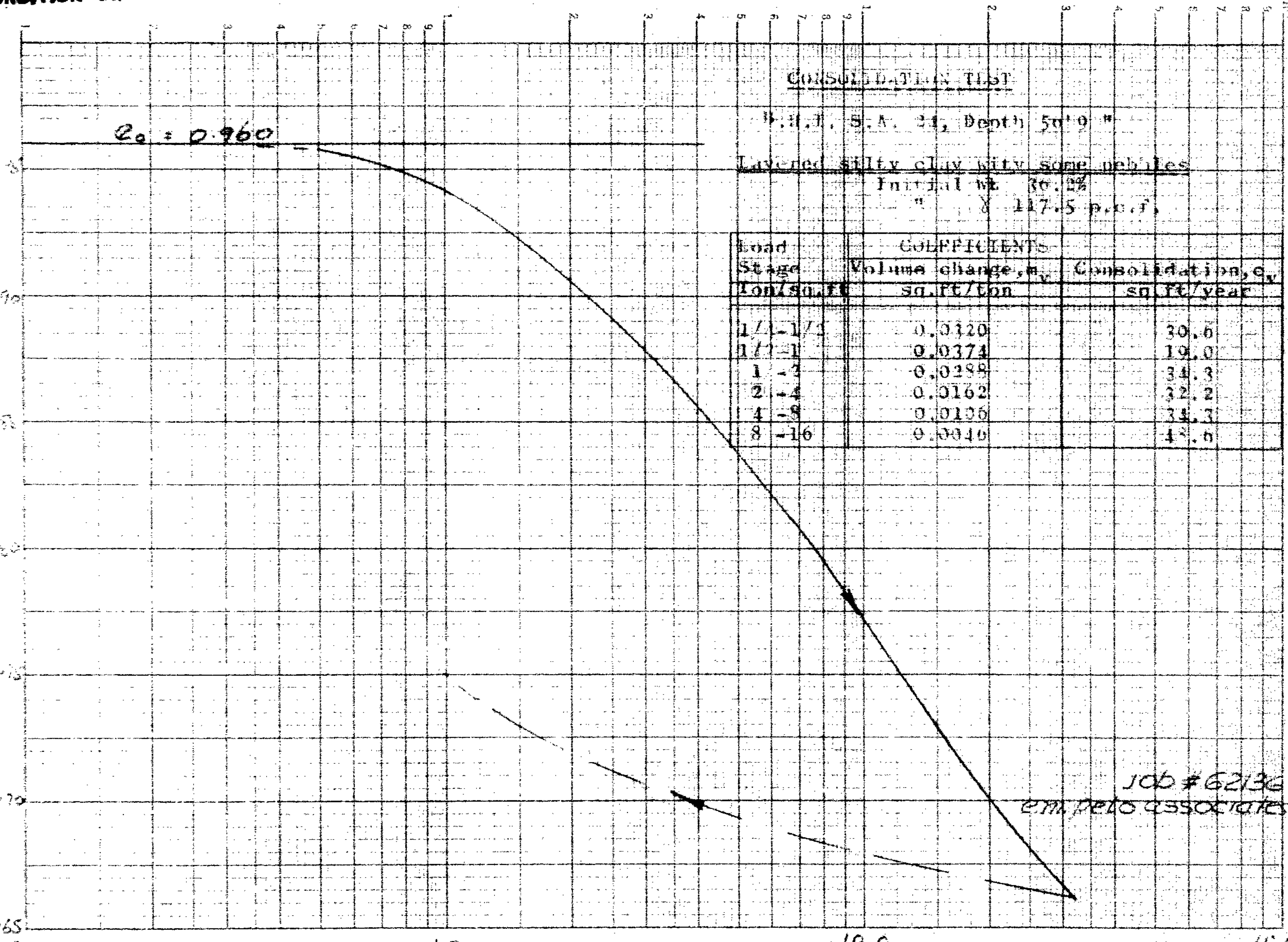
JOB NAME Township Bridge #10 Bear Creek JOB NO. 62130 HOLE NO. ______ SAMPLE NO. AS SHOWN

DEP ______ ION ______ REMARKS Silty and sandy clay with pebbles (Layers between elevations 20 and 0).

GRAIN SIZE DISTRIBUTION

Fig. 16

Void Ratio



Load Stage Ton/sq.ft	COEFFICIENTS Volume change, m_v sq.ft/ton	Consolidation, c_v sq.ft/year
1/4-1/2	0.0320	30.6
1/2-1	0.0374	19.0
1 -2	0.0288	34.3
2 -4	0.0162	32.2
4 -8	0.0106	34.3
8 -16	0.0046	4[illegible].6

Pressure Kips/sq ft

Fig. 2a

SEMI-LOGARITHMIC 359-71
KEUFFEL & ESSER CO.

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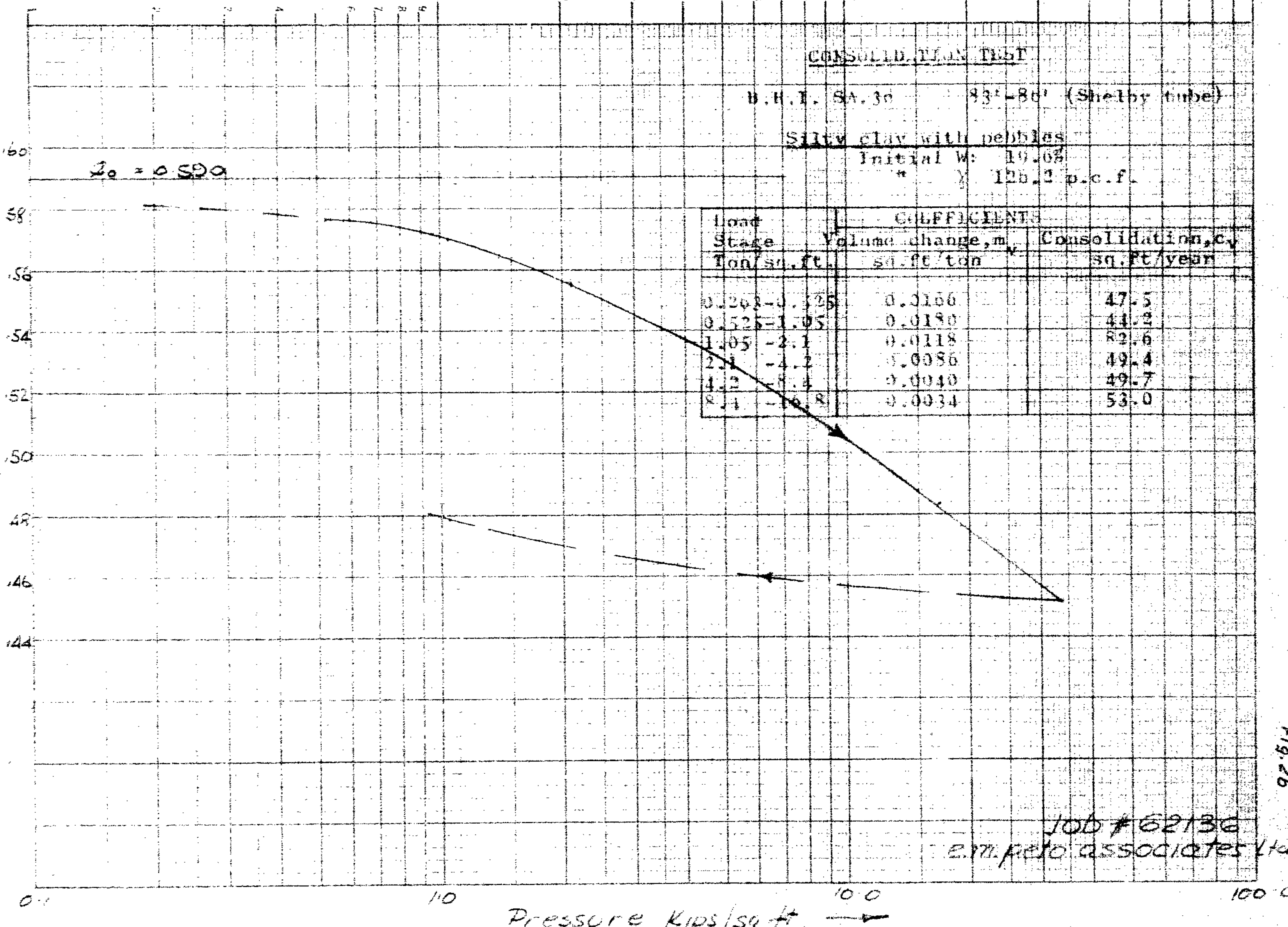

Load Stage Ton/sq.ft.	COEFFICIENTS Volume change, m_v sq.ft/ton	Consolidation, c_v sq.ft/year
0.263-0.525	0.0166	47.5
0.525-1.05	0.0180	44.2
1.05 -2.1	0.0118	82.6
2.1 -4.2	0.0086	49.4
4.2 -8.4	0.0040	49.7
8.4 -16.8	0.0034	53.0

Fig. 26

Job # 6213d
e.m. peto associates Ltd.

E. M. PETO ASSOCIATES LTD.

UNCONFINED COMPRESSION TEST DATA SHEET

Job No. 62136

Borehole Number	Sample Number	Depth feet	Nat. M. C.	Wet Density p.c.f.	Dry Density p.c.f.	Void Ratio, e	u/c Shear Strength Undisturbed, p.s.f.	u/c Shear Strength Remoulded, p.s.f.	% Adhesion (Acc. to M.J. Tom--linson.)	Adhesion on piles p.s.f.
1	11	29'-29'6"	25.1	128.4	101.0	0.67	1215	794	62	493
1	13	32'6"-33'	27.2	127.2	100.0	0.69	1052	810	69	558
1	16	38'6"-39'	24.8	127.2	101.9	0.66	1230		61	752
1	18	39'-39'6"	24.1	126.8	102.0	0.65	1118		68	759
1	20	47'-47'6"	25.1	126.4	101.0	0.67	1280	665	59	756
1	20	47'6"-48'	24.2	125.5	101.0	0.67	810	648	82	665
1	23	53'-53'6"	31.2	117.4	89.5	0.88	665	583	89	595
1	26	62'6"-63'	25.2	125.0	99.8	0.69	647		90	582
1	35	80'-81'6"	24.3	124.0	99.8	0.69	865		79	685
1	23	53'6"-54'	34.4	118.8	88.3	0.91	1150	713	65	748
1	28	62'-63'6"	22.0	124.0	101.8	0.66	1297	972	58	564
1	32	72'-72'6"	21.9	126.3	103.8	0.62	972	1020	73	710
1	32	72'6"-73'	21.3	124.9	102.9	0.64	1297	1392	58	751
1	34	77'-77'6"	21.3	124.9	102.9	0.64	875	810	78	708
1	34	77'6"-78'	24.4	128.0	103.0	0.64	1086	1100	68	738

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UNCONFINED COMPRESSION TEST DATA SHEET

Job No. 62136

Borehole Number	Sample Number	Depth feet	Nat. M. C.	Wet Density p.c.f.	Dry Density p.c.f.	Void Ratio, e	% Strain at Failure	u/c Shear Strength undisturbed p.s.f.	u/c Shear Strength remoulded p.s.f.	% Adhesion (Acc. to M.J. Tomlinson)	Adhesion on piles p.s.f.
2	8	17'6"-18'	27.0	105.8	83.2	1.04	10.0	437		101	441
2	12	27'-27'6"	26.8	124.0	98.0	0.72	20	615		92	565
2	12	27'6"-28'	26.3	126.3	100.0	0.69	20	955	745	75	715
2	15	37'-37'6"	27.8	123.2	96.5	0.75	20	566		95	537
2	15	37'6"-38'	27.9	126.3	99.0	0.70	20	908	648	77	699
2	17	41'6"-42'	28.0	126.3	98.9	0.70	20	550		96	528
2	17	42'-42'6"	27.7	124.8	97.8	0.73	20	826	405	81	670
2	23	57'-57'6"	22.2	127.2	104.0	0.62	20	1150		65	748
2	23	57'6"-58'	28.1	121.8	95.0	0.77	20	350	350	106	371
2	19	47'-47'6"	34.1	116.9	87.0	0.94	20	502		98	492
2	19	47'6"-48'	37.4	116.9	85.0	0.98	15.0	843	518	80	675
2	20	50'-51'6"	31.8	123.1	93.5	0.80	18.3	1122	690	66	740

UNDRAINED TRIAXIAL COMPRESSION TESTS

FOR DETERMINATION OF MODULUS OF

LINEAR DEFORMATION, E (YOUNG'S MODULUS).

H/SA. NO.	DEPTH FT	WATER CONTENT %	BULK DENSITY PCF	DRY DENSITY PCF	VOID RATIO	UNDRAINED SHEAR STRENGTH C_u PCF	YOUNG'S MODULUS E TON/SQ.FT	$\frac{E}{C_u}$	CELL PRESSU PSI
1/24	55.5	39.5	121.5	87.0	1.07	550	69.5	253	50
1/36	85	20.0	130.5	109.0	0.54	1700	110	130	75

NOTE: E was obtainedfrom average slope of stress - strain curve loops in repeated loading cycles.

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FIELD VANE TEST RESULTS

B. H. #	DEPTH	SHEAR STRENGTH	
		UNDISTURBED	REMOULDED
	FT	PSF	PSF
1	50.5 - 51	4650	1540
2	38 - 39	3000	1200

NOTE: Because of the large discrepancy between the above results and the shear strength measured by unconfined compression tests, the field vane tests were disregarded.

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e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

ge # 10 Job No. 9?139 Borehole No. 1

bton Casing 4" to 10', 65 to 70' Boring Date July 30th to August 3rd 1962

Compiled By R. K. Checked By

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

	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Natural Moisture Content	WATER LEVELS & REMARKS
			0'						
b les					1	C.S.			
1") nred	Mottled brown & grey	Stiff			2	S.S.	20	17.4	Dry
	"		6'4"		3	S.S.	18	21.1	
e pebbles, ore	Brown with some grey fissures	"	10'		4	S.S.	18	23.5	W.T.P.L.
	Brown some grey	"			5	S.S.	14	20.0	
					6	S.S.	14	28.1	
			15'						
les	Grey	Soft to Firm	16'3"		7	S.S.	9	26.8	
			17'6"						
			20'						
ts ganic s	Brown with grey fissures	"	22'8"		8	S.S.	15	21.4	
			25'						
its	Grey	Firm			9	S.S.	11	27.3	
					10	3"SL	Tapped		
					11	2"SL	"		
	"	"	30'		12	S.S.	14	26.3	
					13	2"SL	Tapped		
			35'						
	"	"			14	S.S.	15	28.6	
					15	3"SL	Tapped		
					16	2"SL	Tapped		
			40'						No grits and pebbles below 39'6"
	"	"			17	S.S.	14	25.0	
					18	2"SL	Tapped		
			45'						
					19	3"SL	"		
	"	"			20	2"SL	"		
					21	S.S.	18	28.4	
			50'						
					⊗				Vane Test at 50"

	"	"		20	2"SI	"		
				21	S.S.	18	8.4	
			50'					
								Vane Test at 50'
	"	"		22	S.S.	13	34.2	
				23	2"SI	Tapped		
			55'					
with some				24	3"SI	"	30.2	
cture	"	"		25				
				26	S.S.	17	25.0	3" stone
			60'					
	"	"		27	S.S.	19	25.8	
				28	2"SI	Tapped		
			65'					
	"	"		29	S.S.	19	21.0	Distinctly flaky structure
				30	2"SI	Tapped		
			70'					
	"	"		31	S.S.	27	21.2	
				32	2"SI	Tapped		
			75'					
	"	"		33	S.S.	17	20.6	
				34	2"SI	Tapped		
			80'					Started to use wash water at 79'
	"	"		35	S.S.	16	29.7	C_w = 305
					2"SI	Tapped		
				36	S.T.			Blows on S. T. for successive 6" 4,8,12,18.
			85'10"					
	"	"		37	S.S.	31	13.2	
			88'3"					
			90'					
with	"	"		38	S.S.	22	23.8	
			95'					
	"	"		39	S.S.	42	19.7	
			100'					
	"	"		40	S.S.	38		2" stone
			105'					
merous	"	"	~~106'4"~~	41	S.S.	27	32.9	
			110'					
ay	"	"		42	S.S.	22	34.0	
			115'2"					
d and of	Dark grey to black	Extremely dense	116'6" 116'8"	43	S.S.	122	10.1	

Testhole terminated at 116'8"

this margin reserved for binding

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

...ge # 10 Job No. 92130 Borehole No. 2

...ton Casing 1" Boring Date August 7 & 8/1962

Compiled By R.K. Checked By

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

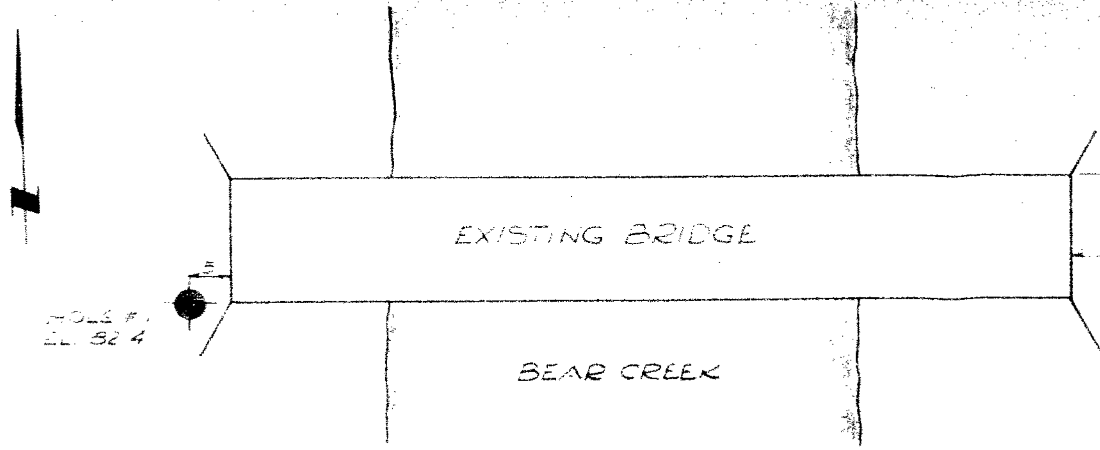
ABBREVIATIONS

- 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

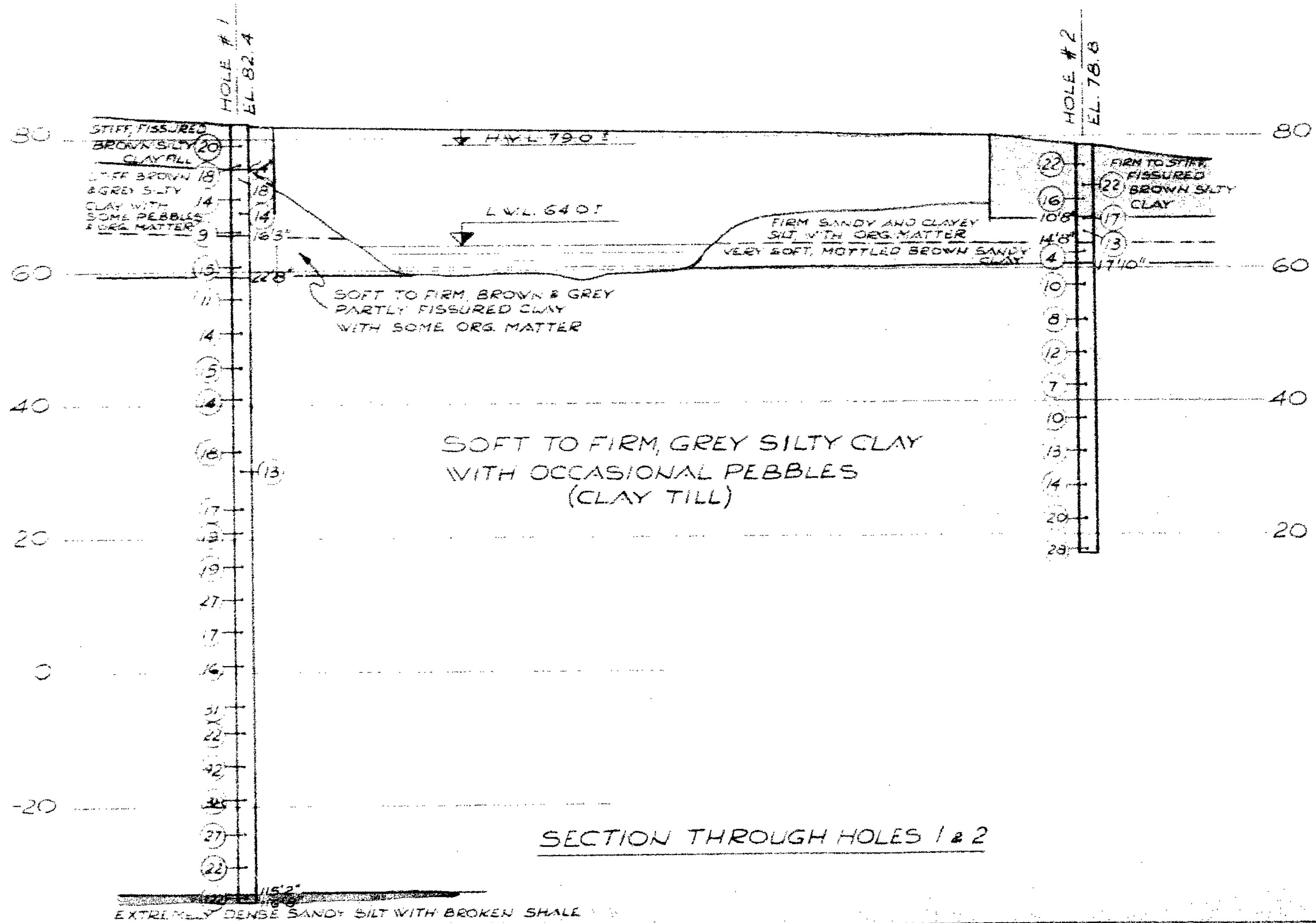
	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	Natural Moisture Content	WATER LEVELS & REMARKS
			0'						
...th	Grey brown				1	C.S.			
...issured	Brown	Firm			2	S.S.	22	10.4	
			5'						
	"	V. Stiff			3	S.S.	22	17.[illegible]	
	"				4	S.S.	16	18.0	
...ilt with	"	Firm	10'6"		5	S.S.	17	20.6	
					6	S.S.	13	21.6	
...rganic ...ragments	"		14'8"						Much softer below 14'8"
	Mottled brown and grey	V. soft			7	S.S.	4	25.0	
			17'10"		8	2"SL	Tapped		River level
...bbles	Grey	Soft to firm			9	S.S.	10	20.5	Stones up to 1½"
					10	3"SL	Tapped		
			25'						
...casional	"	"			11	S.S.	8	20.5	Fewer pebbles
					12	2"SL	Tapped		
			30'						
	"	"	31'4"		13	S.S.	12	24.2	
					13A	3"SL	Tapped		
			35'						
	"	"			14	S.S.	7	20.5	
					15	2"SL	Tapped		
					⊗				Vane test
			40'						
	"	"			16	S.S.	10	20.0	Few pebbles
					17	3"SL	Tapped		
			44'						
	"	Firm			18	S.S.	13	31.3	
					19	2"SL	Tapped		
			50'						
	"	"			20	S.S.	14	30.1	
					21	3"SL	Tapped		
			55'						
...e	Dark grey	Soft to firm			22	S.S.	20	22.3	Less plastic and more pebbles
					23	2"SL	Tapped		
			60'						
	"	"	61'6"		24	S.S.	[illegible]	[illegible]	

...DUE TO ...DOCUMENT

Testhole terminated at 61'6"



SITE PLAN
SCALE 20'TO1'



SECTION THROUGH HOLES 1 & 2

LEGEND

BOREHOLE

(16)— BLOWS/FOOT S.P.T.

PROFILE SCALE

20' TO 1' (NATURAL)

NOTE:
SEE BOREHOLE LOGS FOR COMPLETE SOIL DATA.

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

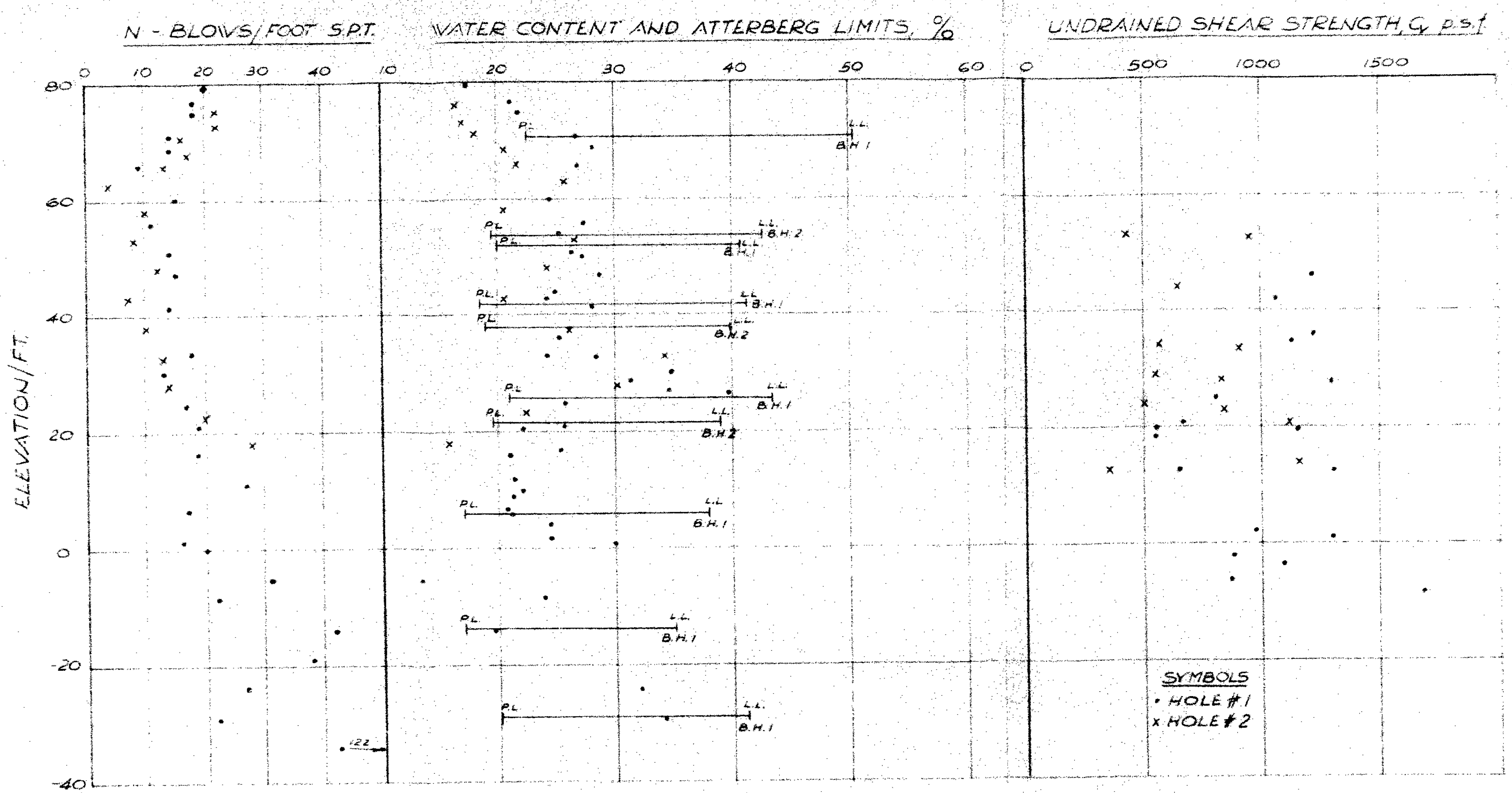
BOREHOLE

BLOWS/FOOT S.P.T.

PROFILE SCALE

20' TO 1" (NATURAL)

NOTE:
SEE BOREHOLE LOGS FOR COMPLETE SOIL DATA.



VARIATIONS OF GEOTECHNICAL PROPERTIES WITH DEPTH

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.

REGISTERED PROFESSIONAL ENGINEER
E. M. PETO
PROVINCE OF ONTARIO

THE COUNTY OF LAMBTON % J.A. MONTEITH & ASSOCIATES LTD., CONSULTING ENGINEERS		
TOWNSHIP BRIDGE # 10, BEAR CREEK		
PREPARED BY e.m. peto associates ltd.		
JOB NO. 62136	DATE: SEPT. 1962	DRAWN BY: K.K.