63-F-238M

BEAR CREEK

3 MILES N.E. OF PETROLIA

BA 1631

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,
Toronto 19, Ontario.

Our Job Number 62223

789 - 1126.

26th February 1963.

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

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

Gentlemen,

Soil Site Investigation,
Bear Creek Bridge
3 Miles N.E. of Petrolia, Ontario.

We have pleasure in submitting four copies of our Report Number 62223 on the above site investigation.

A full description of the various strata encountered in the two test holes performed at the site is given, but a brief summary of subsoil conditions is included on page 13 for your convenience.

This is followed by a discussion of suitable types of foundation as well as excavation and backfilling problems.

Spread footings resting on clay till at any depth below elevation 627 could be used and the allowable net pressure is estimated as 2.0 tons/sq.ft. Alternatively, pilesor caissons resting on the shale bedrock, which commences about 33 ft below the existing grade, may be considered.

We belive the report to be comprehensive; however, we would gladly discuss further any points which you may wish clarified. Also, we would welcome an opportunity to recalculate the theoretical settlement when the level, dimensions and load distribution of possible footing foundations have been decided.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

E. M. Peto, P. Eng.

RK/ap

THE TOWNSHIP OF ENNISKILLEN,

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

# SOIL SITE INVESTIGATION

## BEAR CREEK BRIDGE

## 3 MILES N.E. OF PETROLIA, ONTARIO.

E. M. PETO ASSOCIATES LTD.

1287 Caledonia Road,
Toronto 19, Ontario.

# TABLE OF CONTENTS

## A. INTRODUCTION:

The work described in this Report was authorized by Mr. G. W. Ingram, of J. A. Monteith and Associates Ltd., by letter dated December 6th, 1962.

It is proposed to provide a new diversion channel for Bear Creek at a point approximately three miles to the north-east of Petrolia, Ontario. A new bridge is to be provided at a point where the channel will be crossed by the 18th Side Road. The new channel is to be 50 ft wide and the invert is to be near elevation 629.

The bridge is to be built on dry land and the water channel cut later. The structure will have a 95 to 100 ft prestressed span, with normal type concrete abutments and wing walls.

This Company was authorized to perform a site investigation consisting of two test holes spaced 100 ft apart, in the approximate locations of the new bridge abutments.

## B. GENERAL INFORMATION:

1. Two test holes were performed in the positions indicated on the enclosed site plan, based on Consulting Engineer's drawing No. 1982. The test holes, spaced 100 ft apart, were set out in the field by Mr. G. Ingram, of J. A. Monteith Associates, Ltd., who also supplied the ground elevations.

2. Test hole 1, which was performed first, encountered sound shale at a depth of 33.2 ft, and proved the bedrock by 5.3 ft of diamond drilling. Test hole 2 encountered refusal at almost identical elevation and was terminated at that point, 32.7 ft below the existing grade.

3. The field work was performed by our drilling unit No. 6, between January 17th and 21st, 1963. Our standard drilling and sampling procedures were followed, as outlined in enclosed Appendix A.

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

Water content determinations,
Atterberg limit tests,
Particle size distribution tests,
Unconfined compression tests with volumetric analysis,
Triaxial compression test for determination of pore pressure parameter 'A'.
Consolidation test.

Results of the above tests are included in Appendix B.

## C. SITE AND GEOLOGY:

The site is located near to the present channel of Bear Creek, approximately three miles north-east of Petrolia, in the Township of Enniskillen, Ontario. The existing gravel sideroad 18, which runs in a north-south direction, separates Lot 18, which is to the west of the road, from Lot 19; the site is located on Concession 13, 530 ft north of the northern boundary of Concession 12.

The area is located within a left hand bend in the Bear Creek, in the flood plain of the creek. The terrain within several hundred feet of the site is flat, but ground rises on both sides of the flood plain.

Geologically, the area is located on the St. Clair clay plain, where glacial processes have deposited a mantle of clayey till over a shale bedrock. The shale was found to commence approximately 33 feet below the existing grade.

## D. SOIL CONDITIONS:

Details of the soil conditions encountered in the test holes are described on the enclosed borehole logs, while a simplified subsoil profile, in the form of a section through the test holes, is included on the drawing.

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Details of the soil conditions encountered in the test holes are described on the enclosed borehole logs, while a simplified subsoil profile, in the form of a section through the test holes, is included on the drawing.

D. SOIL CONDITIONS: (Cont'd)

Variation of consistency, penetration resistance and shear strength of the subsoil with depth is illustrated on Fig. 1, Appendix B.

The subsoil can be subdivided into the following strata, in the order of occurrence:

a) Silty clay and gravel fill
b) Firm brown clayey silt
c) Soft clayey sand and gravel
d) Stiff clay with pebbles (clay till)
e) Firm clayey sand with gravel (sandy and gravelly till)
f) Shale bedrock.

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

a) Silty clay and gravel fill

A surficial layer, 4 to 8 inches thick, of silty clay mixed with gravel was found immediately below the existing grade, and forms the present road surface.

b) Firm brown clayey silt

Below the surficial fill, the subsoil was found to consist of a brown clayey silt with some fine sand and occasional pebbles, which extended to a depth of 5.7 ft in test hole 1 and 6.3 ft in test hole 2. The material had a generally firm consistency, standard penetration tests resulting in 5 and 14 blows per foot in the two test holes respectively. The moisture contents were 22.6% and 25.4%.

## D. SOIL CONDITIONS:

### b) Firm brown clayey silt (Cont'd)

The material was partly fissured due to desiccation, and included some plant roots. It became wet at a depth of 5.5 ft in test hole 2.

Due to the high silt content, this portion of subsoil must be considered as an inferior backfilling material, difficult to compact under controlled water content conditions, and susceptible to frost heave.

### c) Clayey sand and gravel

Variable and generally soft layers, of alluvial character, were encountered in test hole 1 between the approximate depths of 5.7 and 7.8 ft, and in test hole 2 between 6.3 and 9.2 ft. They consisted mostly of clayey sand with seams of sandy clay and with gravel. Ground water was present in these layers, the level of water corresponding roughly to the water level in Bear Creek.

A standard penetration test performed in this material at a depth of 6 to 7.5 ft in test hole 2, gave a result of 11 blows per foot, corresponding to a loose to compact density. The moisture content was between 21.2% and 25.8%.

Considerable seepage of ground water must be expected in excavations penetrating through this layer.

## D. SOIL CONDITIONS: (Cont'd)

### d) Stiff clay with pebbles (clay till)

A stratum of generally stiff clay, of till form, was encountered in test hole 1 at a depth of 7.8 ft and in test hole 2 at a depth of 9.2 ft below the existing grade, corresponding to elevations 632.0 and 629.3 respectively. It extended to a depth of approximately 29 ft in test hole 1 and 28 ft in test hole 2, where it changed to a more sandy and gravelly till.

Since the stiff clay till may be required to support the proposed bridge foundations, its geotechnical properties were studied in some detail and can be summarized as follows. Reference should also be made to Figs. 1, 2 and 3 of Appendix B.

1. Appearance and composition.

The material consisted mainly of silt and clay with some sand and pebbles; a grain size distribution curve of a typical sample is presented in Appendix B, Fig. 3a.

In test hole 1, upper layers of this stratum had a brown colour due to desiccation, with some yellow-brown patches, the material became progressively more grey with depth. In test hole 2, the brown portion was not recorded.

## D. SOIL CONDITIONS:

### d) Stiff clay with pebbles (Clay till)

### 1. Appearance and composition (Cont'd)

The plasticity of the material was investigated by means of three Atterberg limit tests, which gave the following results.

| | |
|---|---|
| Liquid Limit | 36 to 41% |
| Plastic Limit | 19 to 20% |
| Plasticity Index | 17 to 21 |

The above results are typical of the clayey till which forms the main overburden over bedrock in the St. Clair clay plain.

A considerably more plastic layer was present in test hole 1 between the depths 7.8 and 11.3 ft. This material, which was partly fissured and contained some grits, had the following Atterberg limits.

Liquid Limit 56%, Plastic Limit 25%, Plasticity Index 31.

Similar material of high plasticity was not encountered at a corresponding depth in test hole 2; however, in the latter test hole the alluvial soft clayey sand and gravel extended to a lower level.

## D. SOIL CONDITIONS:

d) Stiff clay with pebbles (Clay till) (Cont'd)

2. Consistency and shear strength

Variation with depth of standard penetration resistance, water content and undrained shear strength is illustrated on Fig. [illegible], Appendix B. Standard penetration test results (N-values) in test hole 1 were in the range of 22 to 30 blows per foot, and in test hole 2 varied from 18 to 23 blows per foot. The slight[illegible] [illegible]ffer consistency indicated for the case of test hole 1 was further con[illegible] [illegible] lower moisture contents in this test hole, which ranged f[illegible] [illegible]8.2% [illegible] [illegible]0.4%, whi[illegible] the moisture contents in test hole 2 were in the r[illegible] of 20.6% to 23.3%. The clay till in test hole 1 was more desiccated, as indicated by the presence of some brown colour, down to a depth of 20 to 25 ft; the material changed to pure grey only below this depth, while in test hole 2 it was grey from the very top.

Compared to Atterberg limits, the in situ water contents were found to be generally near or a little above the plastic limit, corresponding to a stiff consistency.

The undrained shear strength was measured by means of unconfined compression tests on "undisturbed" samples. The results are tabulated in Appendix B, together with densities and void ratios. The undrained shear strength, taken as one half of unconfined compressive strength, is plotted against elevation in Fig. 1 and against water content in Fig. 2.

## D. SOIL CONDITIONS:

### d) Stiff clay with pebbles (Clay till)

#### 2. Consistency and shear strength (Cont'd)

Tests on two samples from the upper portions of the stratum in test hole 1 confirmed the higher strength of the clay till in this test hole, indicated by the N-values and water contents.

At a greater depth, the undrained shear strengths of samples from both test holes was similar.

In Fig. 2, the undrained shear strength is plotted against water content, and a curve is included relating the average relationship from our amassed results for a similar clay till, from 12 site investigations in the area.

The average water content below the probable foundation level (elevation 622) is between 19.5 and 21.5%, corresponding to a range of shear strength of 2100 to 1,800 lb/sq.ft. A triaxial compression test was performed on a typical sample of the clay till from elevation 617 in test hole 1. The sample was previously consolidated at a pressure slightly below the estimated effective overburden pressure at the depth from which the sample was taken. The pore pressures were measured during compression, and hence Skempton's pore pressure parameter 'A' was deduced; the results are plotted in Appendix B in Fig. 5. The results are applicable to the settlement analysis, by the method suggested by Skempton and Bjerrum, Geotechnique 1957, "A Contribution to Settlement Analysis of Foundations on Clays".

## D. SOIL CONDITIONS:

d) Stiff clay with pebbles (Clay till)-(Cont'd)

3. Compressibility

An oedometer consolidation test was performed on an undisturbed sample of the clay till from near elevation 617 in test hole 1. The results, in the form of void ratio versus log pressure curve are presented on Fig. 4, Appendix B. Coefficients of compressibility and consolidation for the various load stages of the test are included on the above figure in tabular form.

Young's modulus, or modulus of linear deformation, E, was not measured on samples from this site. However, on the basis of our amassed results for similar material, it is considered that a value of E equal to 250 times the average undrained shear strength can be assumed; that is 250 tons/sq.ft. This value is used in calculation of "elastic", or immediate settlement of footings.

D. SOIL CONDITIONS: (Cont'd)

e) Clayey sand with gravel (sandy till)

The clayey till changed at a depth of 28 to 29 ft to a considerably less plastic and more sandy material, containing a high proportion of gravel. Many of the stones had the form of angular fragments of shale. This deposit, which can be considered as a sandy and gravelly till with a clay and silt binder, extended to a depth of 33.2 ft in test hole 1 and 32.7 ft in test hole 2, where an abrupt boundary with the underlying shale bedrock occurred.

A typical grain size distribution of this material is included on Fig. 3b.

N-values in this stratum were 30 in test hole 1 and 19 in test hole 2. A moisture content of 11.8% was recorded in test hole 1 at a depth of 31.5 ft.

On the basis of the above results, this material can be considered at least as strong as the overlying clayey till, while its compressibility can be assumed as negligible.

## D. SOIL CONDITIONS: (Cont'd)

### f) Shale Bedrock

A black shale bedrock was encountered at a depth of 33.2 ft in test hole 1, (elevation 606.6); the shale was proved by diamond drilling to a depth of 38.4 ft andthe core recovery was excellent. The material was found to be very sound, and can be considered as capable of supporting high foundation pressures, transmitted through end-bearing piles or caissons.

It should be noted that the boundary between the shale bedrock and the overlying sandy till was very well defined.

In test hole 2, refusal was obtained at a depth of 32.7 ft, corresponding to elevation 605.8. Diamond drilling was not performed in this test hole, but because of the sharp change from the till to the shale observed in test hole 1, it can be concluded that the refusal in test hole 2 was caused by the surface of shale bedrock, and it is considered that the presence of shale bedrock at the site at an average elevation 605.8 to 606.6 has been reliably established.

## E. WATER CONDITIONS:

Free water was encountered at a depth of about 5.5 ft in the soft clayey sand and gravel layers, resting on top of the clay till. The rate of inflow was moderate, but all seepage was cut off when test hole casing penetrated into the clay till at a depth of 8 to 10 ft below the existing grade.

The level of phreatic surface at the site can be assumed as roughly corresponding to the mean level of the Bear Creek.

## F. CONCLUSIONS AND RECOMMENDATIONS:

### 1. Summary of Subsoil Conditions

A simplified subsoil profile is presented on the enclosed drawing.

The two test holes spaced 100 ft apart encountered the black shale bedrock at a depth of 32.7 and 33.2 ft below the existing grade, corresponding to elevations 605.8 to 606.6. The shale is sound and capable of supporting high loads.

The bedrock is covered in turn by some 4 ft of a sandy and gravelly till, and by 19 to 21 ft of a stiff clay till. The latter deposit, which commences at a depth of 7.8 to 9.2 ft below the existing grade, can support a spread footing type of foundation, although piles to rock mayprove more economic.

# F. CONCLUSIONS AND RECOMMENDATIONS:

## 1. Summary of Subsoil Conditions (Cont'd)

The only water seepage was encountered in a 2 to 3 ft thick layer of soft clayey sand and gravel, resting on top of the clay till stratum. This pervious layer was covered by 5.7 to 6.3 ft of firm brown clayey silt, encountered immediately below the gravel fill layer forming the surface of the road from which the test holes were put down.

## 2. Foundation Level and Bearing Capacity

It is considered that the new bridge can be supported on

a) Spread footings located in clay till; or

b) end-bearing piles, or caissons, supported on shale.

Both possibilities will now be discussed in turn.

### a) Spread footings

Spread footings could be placed in the clay till stratum at any level from elevation 627 downwards. The depth of foundation will depend on the level of creek diversion invert, and the need to place the footings at a sufficient depth to protect them against possible scouring or frost heave.

## F. CONCLUSIONS AND RECOMMENDATIONS:

### 2. Foundation Level and Bearing Capacity

Spread footings (Cont'd)

The upper part of the claytill was somewhat stiffer in the proposed location of the northern abutment (test hole 1), as was discussed on page 9, and will locally have a higher safe bearing capacity. However, according to information obtained from the Consulting Engineer, the effective width of the footings will be about 15 ft; consequently, the lower portions of the stratum will be affected by a major proportion of the stresses transmitted by the footings, particularly if the footings are located deeper than elevation 627. Since in these lower portions the shear strength in the two test holes was similar, it is considered that an average undrained shear strength of 1,900 lb/sq.ft can be used for calculation of allowable bearing capacity of footings set at or below elevation 627 for both abutments. However, the settlement of the northern abutment, founded on the stiffer clay will be somewhat smaller.

An adjustment of foundation sizes to ensure equal settlement of both abutments would require additional field information, from several test holes which would determine the extent of the stiffer material in the area of test hole 1. Also, an additional consolidation testing programme would be needed for a more accurate settlement prediction.

# F. CONCLUSIONS AND RECOMMENDATIONS:

## 2. Foundation Level and Bearing Capacity

### a) Spread footings (Cont'd)

While it is felt that an attempt at specifying different foundation sizes for uniform settlement on the basis of the presently available information would merely have an academic value, from the limited magnitude of the calculated settlement based on the simplified assumptions it appears that the more detailed investigation is not economically justified.

The allowable net bearing capacity can be taken as 2.0 tons/sq.ft at any depth in the clay till stratum below elevation 627. This figure includes a factor of safety of at least three with respect to shear failure. An overburden component, equal to the least weight of final overburden over the footing (or the depth below the maximum anticipated scouring depth) can be added.

The amount of settlement caused by a uniformly distributed net pressure of 2.0 tons/sq.ft will depend on the level of footings and the size and shape of foundation. An estimate of settlement was made, assuming a foundation having an effective width of 15 ft and a length of 30 ft, founded at elevation 622.

# F. CONCLUSIONS AND RECOMMENDATIONS:

## 2. Foundation Level and Bearing Capacity

### a) Spread footings (Cont'd)

The theoretical settlement was found to be as follows:

| | | |
|---|---|---|
| Immediate ("elastic") settlement: | 0.40 | inches. |
| Long-term, consolidation settlement: | 1.06 | inches. |
| Total final settlement: | 1.46 | inches. |

The above figures are for a net stress increase of 2.0 tons/sq. ft. at foundation level. If the actual pressure is less than 2.0 tons/sq. ft the settlement will be proportionately smaller.

If the actual foundation size will differ appreciably from the assumed dimensions, the settlements should be recalculated.

### b) Pile foundation

The bridge could be supported on pile foundations, resting on the shale bedrock. At the positions of the two test holes, the shale commenced at elevations 605.8 and 606.6. Only very small variations in the level of shale are anticipated at the site.

If driven piles are employed, the bearing capacity on top of the bedrock will be very high, but care should be taken that the shale is not shattered by excessive hammering after refusal is reached.

# F. CONCLUSIONS AND RECOMMENDATIONS:

## 2. Foundation Level and Bearing Capacity

### b) Pile Foundations (Cont'd)

The driving through the overburden is unlikely to be excessively hard, and it is anticipated that the moment when a pile reaches the shale will be easily noticeable in most instances, although the possibility of existence of boulders in the last few feet of till over the bedrock cannot be excluded.

Prebored caissons could be considered at this site, the recommended safe bearing pressure being 25 tons/sq. ft of caisson toe. To cut off water seepage, it would probably suffice to provide lining to elevation 628, or higher, i.e. to penetrate a short distance below the surface of the stiff clay.

## 3. Excavation considerations.

No serious difficulty with excavations is anticipated. The only significant water seepage may originate from the soft clayey sand and gravel seam, encountered between the depths 5.7 and 9.2 ft below the existing grade. However, it is considered that the quantity of water will not be large and the seepage will be controllable. It may be possible to collect the water in troughs suspended below the pervious seam or, if the seepage is only slight, to collect it in drains provided along the perimeter of excavation, and to remove it from the excavation by occasional u pumping from a sump.

F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

Should the quantity of flow prove excessive for the above methods, it may be possible to arrest much of the flow by pumping from wells spaced outside the perimeter of the excavation and penetrating to the pervious layers.

The most effective method of cutting - off all seepage would be by means of sheeting driven to a depth of about 10 ft below the existing grade. However, sheeting may not be essential for supporting sides of excavation, and it s use merely to cut off water seepage may not be economically justified.

Theoretically, the subsoil is strong enough to stand unsupported in near vertical cut during a reasonable period of construction. However, normal safety measures should be adopted.

The excavated formation grade below the footings should be protected against free water, which must not be allowed to collect in pools at the bottom of excavation. If it should prove unavoidable to retain an open excavation for any length of time, the grade below the footings should be protected by an impervious seal, for instance in the form of a thin layer of lean concrete or bituminous coating; or, alternatively, the last six to twelve inches of subsoil should be left unexcavated until the last possible moment before construction of footings.

## F. CONCLUSIONS and RECOMMENDATIONS - Cont'

If prebored piles or caissons are employed, lining should be driven a short distance into the top of the clay till stratum. No water should be tolerated in the boring, as the shale easily softens on contact with free water.

### 4. Backfilling and embankments.

The materials occurring above the clay till stratum are unsuitable for use as backfill behind abutments, and a good granular fill should be imported for this purpose. The clay till could be used for backfilling, but in areas of restricted working space behind abutments its proper compaction, which would ensure no road settlement, may be difficult and the use of a granular fill would probably prove more practical.

If the samples recovered in the two test holes can be considered as representative of the subsoil that will be recovered in the diversion channel excavation, the spoil will be an inferior material for the construction of road embankments. On account of the predominant

## F. CONCLUSIONS and RECOMMENDATIONS - Cont'd

silt content, wet state and presence of some organic matter, it may be difficult to comp act to a high standard required for a satisfactory performance of the new road surface. If it is used in embankments, a well compacted granular layer, at least 12 inches thick, should be provided below the pavement to protect it against frost heave.

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/sb/ap

Our Job Number 62223

February, 1963.

## 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 bose.

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

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

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

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

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

# LABORATORY TEST RESULTS

# ATTERBERG LIMIT TESTS

| B.H./Sa. No. | Depth, ft. | Liquid Limit, % | Plastic Limit, % | Plasticity Index | In Situ Water content % |
|---|---|---|---|---|---|
| 1 / 5 | 11 | 56.4 | 25.6 | 30.8 | 22.7 |
| 1 / 8 | 21 | 40.5 | 20.1 | 20.4 | 20.4 |
| 2 / 6 | 16 | 37.8 | 19.3 | 18.5 | 21.7 |
| 2 / 9 | 26 | 36.3 | 19.3 | 17.0 | 23.3 |

# UNCONFINED COMPRESSION TEST RESULTS

| Hole No. | Sample No. | Depth. | Nat. M.C. % | Densities, p.c.f. Wet | Dry | Void Ratio e | % Strain at Failure | u/c Shear Strength p.s.f. |
|---|---|---|---|---|---|---|---|---|
| 1 | 7 | 15'0"-16'6" | 21.6 | 136.8 | 112.5 | 0.58 | 20.0 | 2335 |
| 1 | 8 | 20'0" - 21'6" | 20.3 | 131.9 | 109.0 | 0.56 | 20.0 | 5770 |
| 1 | 11 | 27'0" - 27'6" | 22.5 | 130.3 | 106.4 | 0.58 | 20.0 | 1800 |
| 1 | 11 | 27'6" - 28'0" | 19.2 | 131.1 | 110.0 | 0.53 | 20.0 | 1330 |
| 2 | 7 | 17'0" - 17'6" | 21.1 | 134.3 | 111.0 | 0.52 | 20.0 | 1600 |
| 2 | 7 | 17'6" - 18'0" | 19.8 | 136.9 | 114.1 | 0.48 | 20.0 | 1960 |
| 2 | 10 | 27'0" - 27'6" | 21.5 | 133.7 | 110.0 | 0.53 | 20.0 | 1590 |
| 2 | 10 | 27'6" - 28'0" | 19.3 | 134.3 | 112.5 | 0.50 | 20.0 | 2270 |

e. m. peto associates ltd.

# GEOTECHNICAL SOIL PROPERTIES

JOB NO. 62223
APPENDIX B.
FIG. 1.

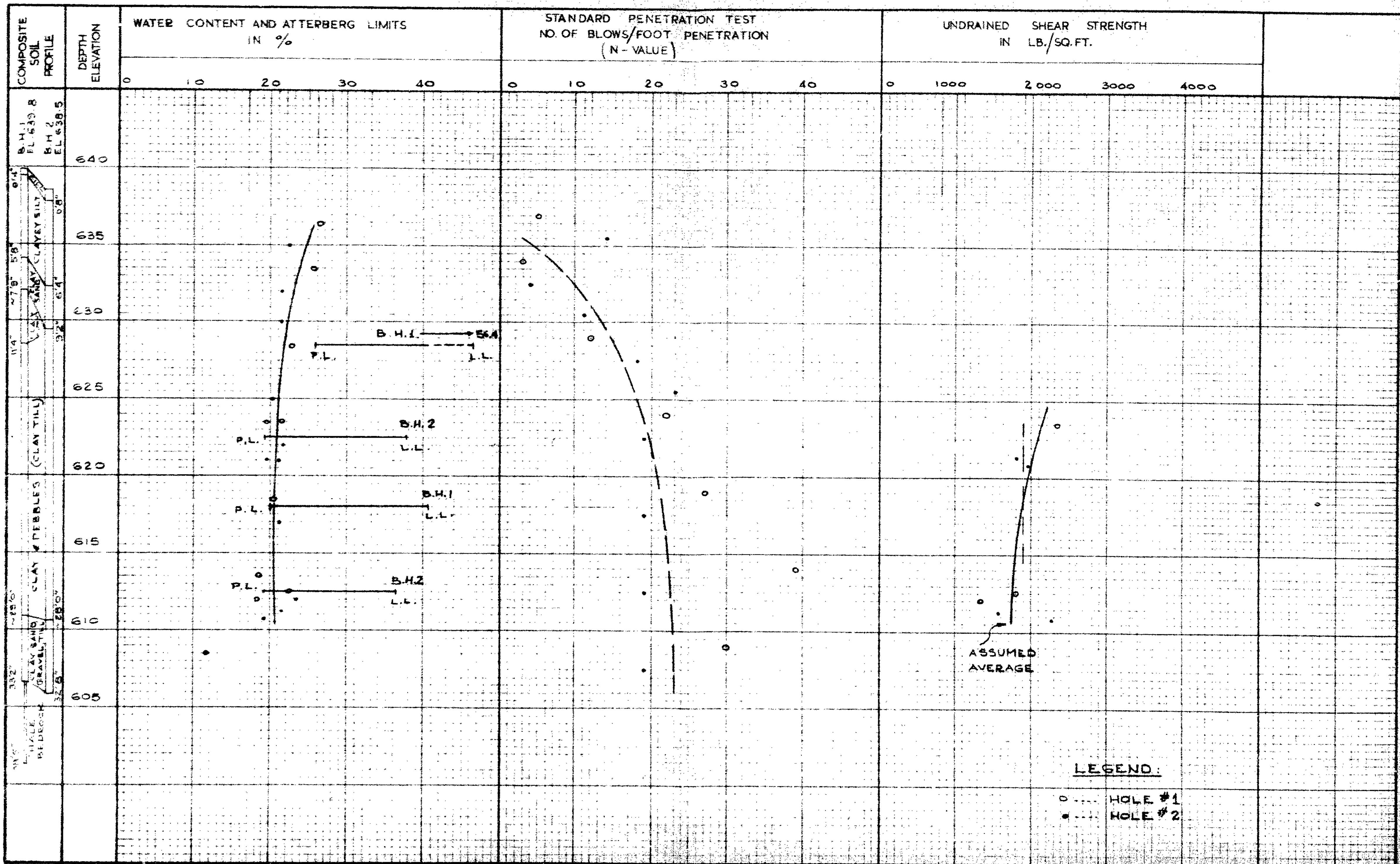
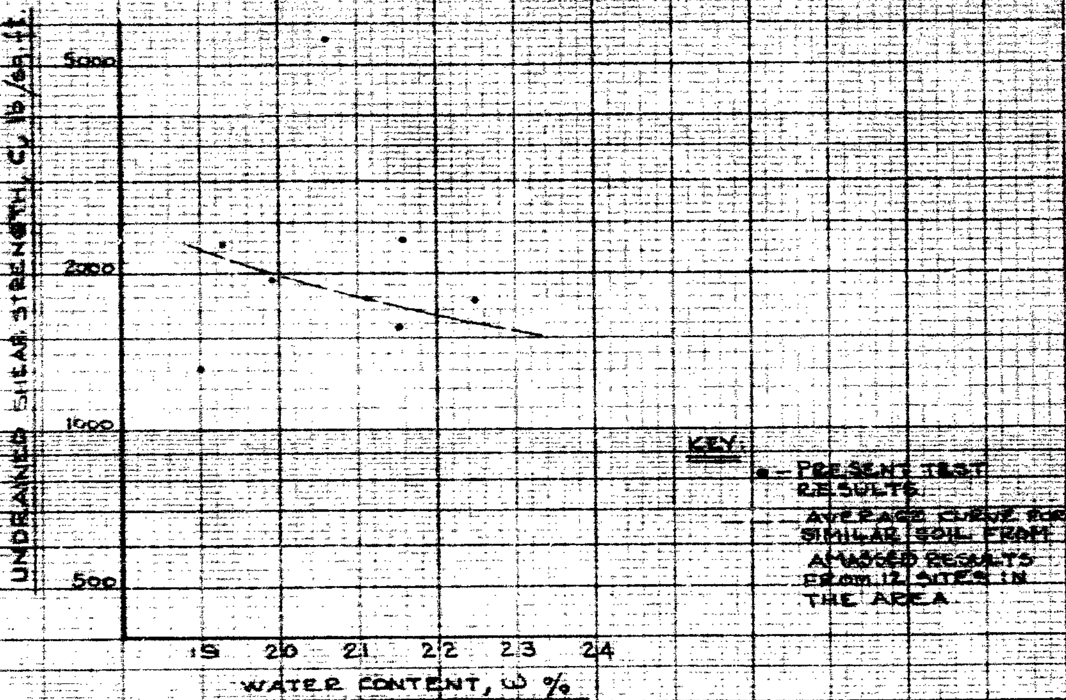


FIG. 2




RELATIONSHIP BETWEEN WATER CONTENT AND UNDRAINED SHEAR STRENGTH

CLAY TILL $\omega_L = 36 - 41\ \%$

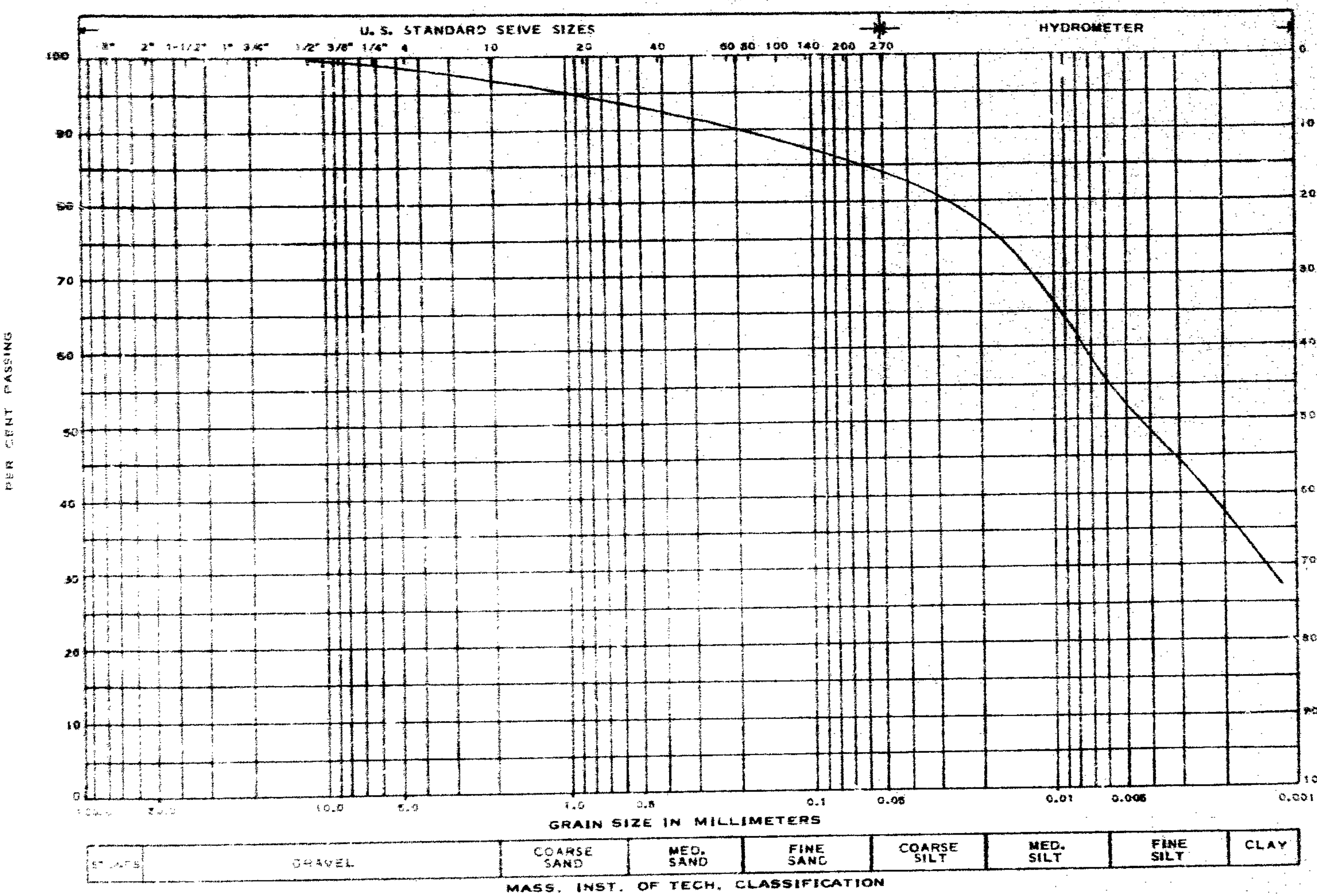
$\omega_p = 19 - 20\ \%$

$I_p = 17 - 21$

JOB No. 62223

**e. m. peto associates ltd.**

**Toronto 19, Ontario**




JOB NAME Bear Creek Brioge JOB NO. 62223 HOLE NO. 2 SAMPLE NO. 8

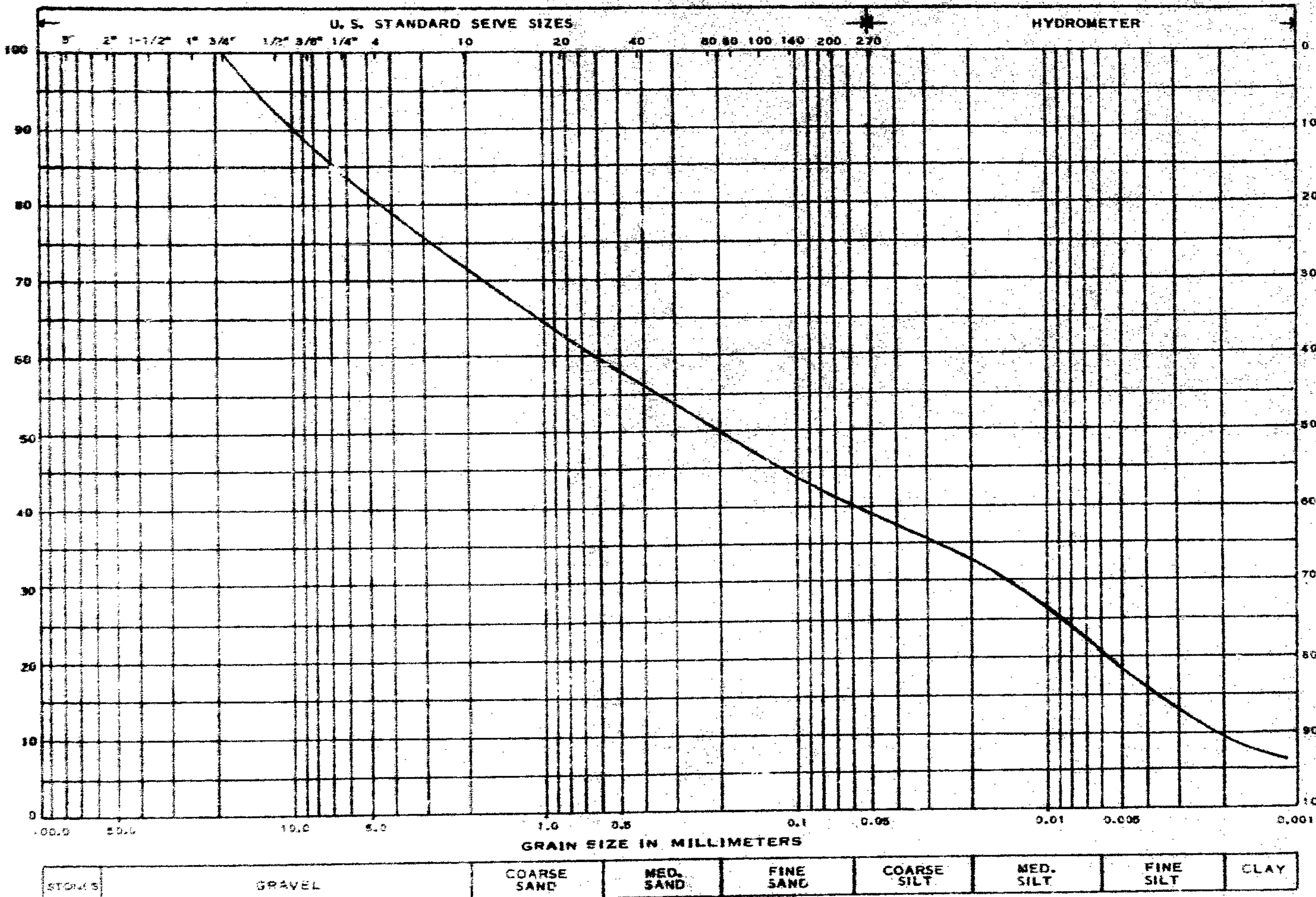
DEPTH 20' -21'6" ELEVATION 618 REMARKS Silty clay with pebbles (clay till)

GRAIN SIZE DISTRIBUTION

Fig 3a

e. m. peto associates ltd.

Toronto 15, Ontario



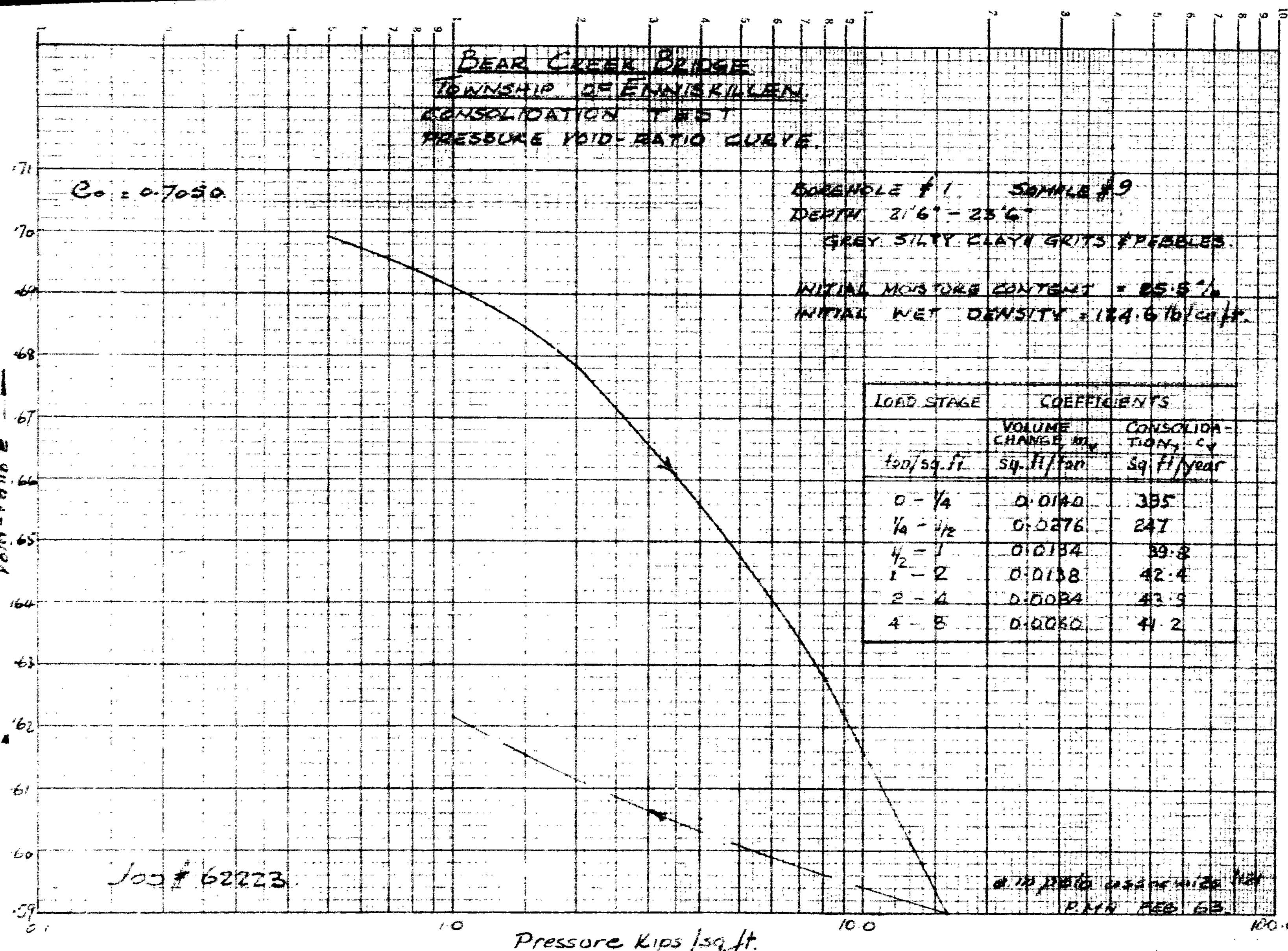

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Bear Creek Bridge JOB NO. 62223 HOLE NO. 1 SAMPLE NO. 12

DEPTH 30'-31'6" ELEVATION 609 REMARKS Sandy and clayey till, high gravel content

GRAIN SIZE DISTRIBUTION

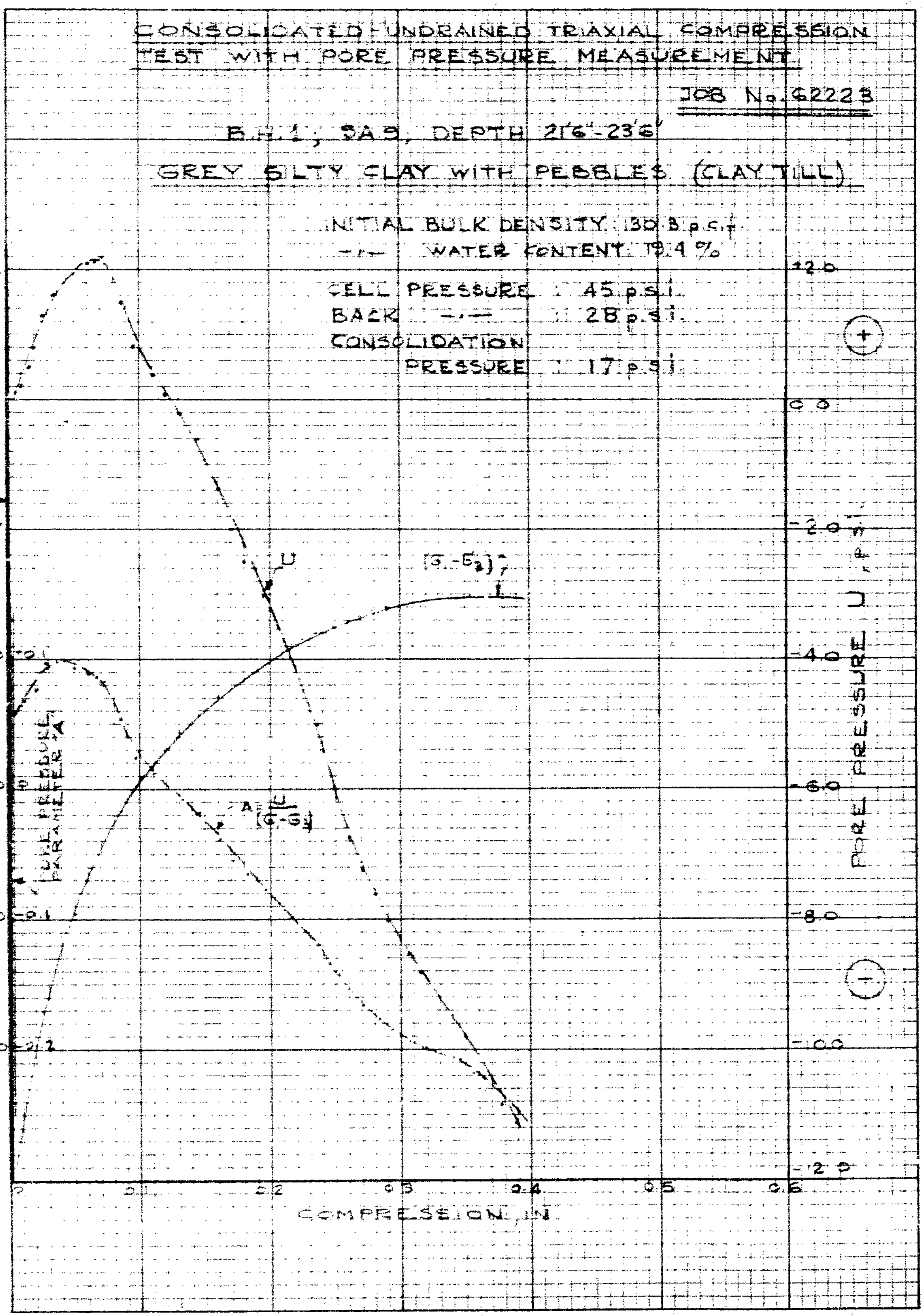
Fig 3b




| LOAD STAGE | COEFFICIENTS | |
|---|---|---|
| | VOLUME CHANGE $m_v$ | CONSOLIDATION, $c_v$ |
| ton/sq.ft. | sq. ft/ton | sq. ft/year |
| 0 - 1/4 | 0.0140 | 395 |
| 1/4 - 1/2 | 0.0276 | 247 |
| 1/2 - 1 | 0.0184 | 39.8 |
| 1 - 2 | 0.0138 | 42.4 |
| 2 - 4 | 0.0084 | 43.3 |
| 4 - 8 | 0.0060 | 41.2 |

FIG. 4

FIG. 5.



CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST WITH PORE PRESSURE MEASUREMENT
JOB No. 62223
B.H.1; SA9, DEPTH 21'6"-23'6"
GREY SILTY CLAY WITH PEBBLES (CLAY TILL)
INITIAL BULK DENSITY 130.3 p.c.f.
WATER CONTENT 19.4 %
CELL PRESSURE : 45 p.s.i.
BACK : 28 p.s.i.
CONSOLIDATION PRESSURE : 17 p.s.i.
DEV STRESS ($\sigma_1$-$\sigma_3$) p.s.i.
PORE PRESSURE PARAMETER 'A'
PORE PRESSURE U, p.s.i.
U
($\sigma_1$-$\sigma_3$)
A = U/($\sigma_1$-$\sigma_3$)
COMPRESSION, IN.
+
−

# e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

## BOREHOLE LOG

| | | | | | |
|---|---|---|---|---|---|
| Job Name | Bear Creek - Bridge Enniskillen Twp. | Job No. | 62223 | Borehole No. | 1 |
| Client | c/o J. A. Monteith Associates | Casing | 4" to 15', BX to 30' | Boring Date | January 17th & 18th, 196 |
| Elevation | 639.8 (Client's) | Compiled By | R. K. | Checked By | S. B. |

**SAMPLE CONDITION**

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

**SAMPLE TYPE**

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

**ABBREVIATIONS**

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

| SOIL DESCRIPTION | COLOUR | Density or Consistency | Depth Elevation | Legend | Sample No. and Condition | Sample Type | No. of Blows per Ft | Natural Water Content | WATER LEVELS & REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| | | | 0'0" | | | | | | |
| Gravelly fill | | | 0'4" | | 1 | C. S. | | | |
| Clayey silt with fine sand some pebbles, and plant roots, fissured (fill) | Brown | Firm | | | 2 | S. S. | 3/6" 5 | 26.4 | |
| | | | 5'8" | | 3 | S. S. | 2/6" 3+ | 25.8 | |
| Very clayey sand and sandy clay | Brown | V. Soft | ~7'9" | | 4 | 2"S. L. Tapped | | | Water in hole after pulling out S. L. |
| Clay, partly fissured, some grits | Brown with yellow-brown patches | Stiff | | | 5 | S. S. | 4/6" 12 | 22.7 | W. T. P. L. |
| Silty clay, some sand and grits (clay till) | | | 11'4" | | 6 | 2"S. L. Tapped | | | |
| | | | 15'0" | | | | | | |
| Ditto | Grey-brown, some yellow-brown patches | Ditto | | | 7 | S. S. | 5/6" 22 | 19.8 | Near P. L. |
| | | | 20'0" | | | | | | |
| Silty clay, some grits | Grey, some yellow-brown patches | V. Stiff | | | 8 | S. S. | 7/6" 27 | 20.4 | D. T. P. L. |
| | | | | | 9 | S. T. | | | |
| | | | 25'0" | | | | | | |
| Ditto | Grey | Ditto | | | 10 | S. S. | 10/6" 39 | 18.9 | |
| | | | | | 11 | 2"S. L. Tapped | | | |
| | | | ~29'0" | | | | | | |
| Clay to sand, with medium gravel (till) | Dark grey | Ditto | | | 12 | S. S. | 8/6" 30 | 11.8 | Semi-angular gravel, mostly black shale |
| | | | 33'2" | | | | | | |
| Shale bedrock | Black | Solid Hard | | | | R. C. | | | Diamond drilled from 33 to 38'5". Recovery 5'3" (100%) |
| | | | 38'5" | | | | | | |

Test Hole Terminated at 38'5".

# e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

## BOREHOLE LOG

Job Name: Bear Creek Bridge — Job No.: 62223 — Borehole No.: 2

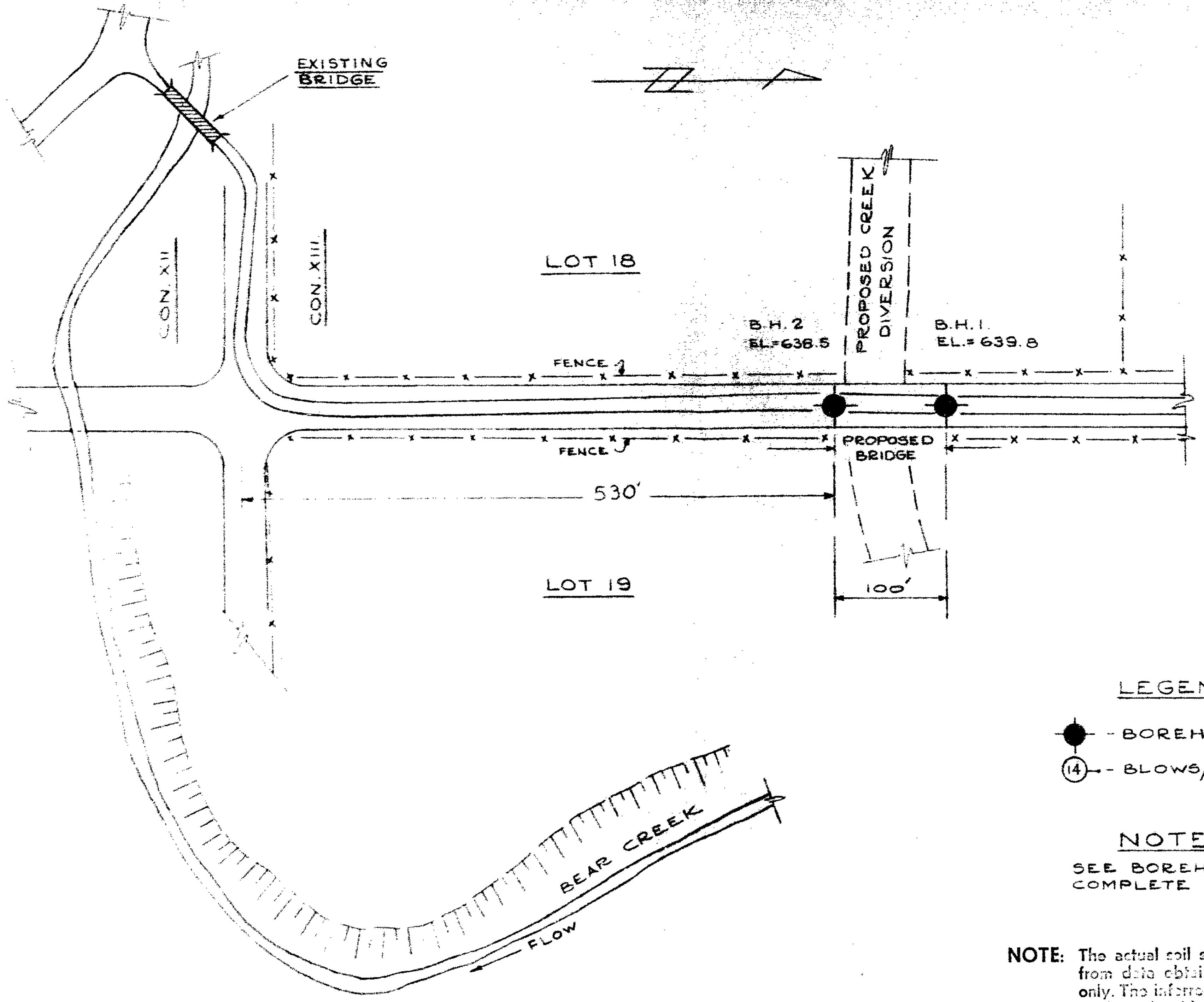
Client: Enniskillen Twp. c/o J. A. Monteith Associates — Casing: 4" BX — Boring Date: January 19th, 1963

Elevation: 638.5 (Client's) — Compiled By: R. K. — Checked By: S. B.

| SAMPLE CONDITION | SAMPLE TYPE | ABBREVIATIONS |
|---|---|---|
| UNDISTURBED | A.S. AUGER SAMPLE | V.T. IN SITU VANE SHEAR TEST |
| FAIR | C.S. CASING SAMPLE | M. MOIST |
| DISTURBED | S.S. 2" STANDARD SPLIT TUBE SAMPLE | W.L. WATER LEVEL IN CASING |
| LOST | S.L. SPLIT BARREL WITH LINERS | W.T. GROUND WATER TABLE IN SOIL |
| | S.T. THIN-WALLED SHELBY TUBE SAMPLE | W.T.P.L. WETTER THAN PLASTIC LIMIT |
| | W.S. WASH SAMPLE | D.T.P.L. DRIER THAN PLASTIC LIMIT |
| | R.C. ROCK CORE | A.P.L. ABOUT PLASTIC LIMIT |

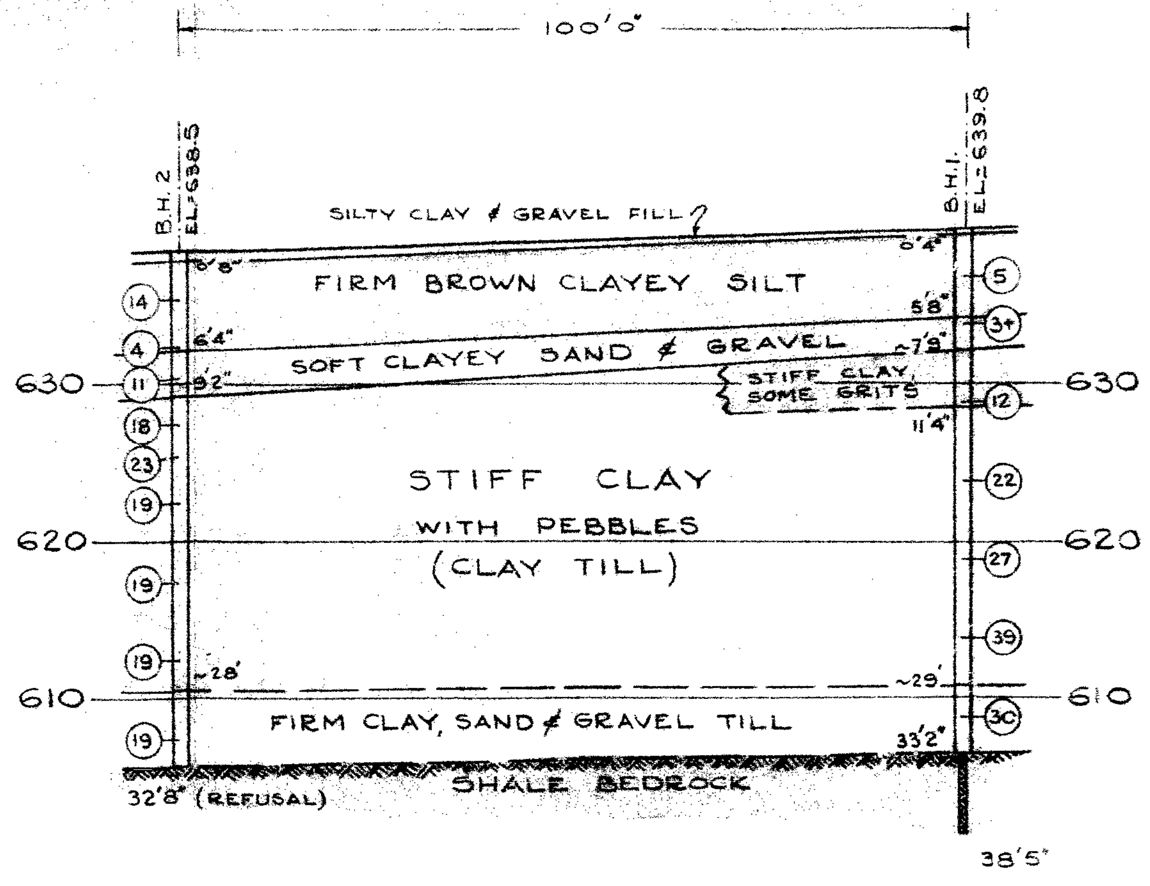
| SOIL DESCRIPTION | COLOUR | Density or Consistency | Depth Elevation | Legend | Sample No. and Condition | Sample Type | No. of Blows per Ft. | Natural Moisture Content | WATER LEVELS & REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| | | | 0'0" | | | | | | |
| Fill of silty clay & gravel | | | 0'8" | | | | | | |
| Clayey silt, fissured | Brown | Firm | | | 1 | S.S. | 6/6" 14 | 22.6 | |
| Silty clay, silt & sand lenses | Ditto | Soft | 6'4" | | 2 | S.S. | 1/6" 4 | 21.2 | Becoming wet at 5'6" |
| Fine to coarse gravel in clayey and sandy silt | Ditto | Loose to Compact | 9'2" | | 3 | S.S. | 5/6" 11 | 21.3 | |
| Silty clay with some grits and sand (clay till) | Grey | Stiff | | | 4 | S.S. | 6/6" 18 | | A little D. T. P. L. |
| Ditto | Ditto | Firm to Stiff | | | 5 | S.S. | 6/6" 23 | 20.6 | |
| | | | 15'0" | | | | | | |
| Ditto | Ditto | Ditto | | | 6 | S.S. | 5/6" 19 | 21.7 | W. T. P. L. |
| | | | | | 7 | 2"S.L. Tapped | | | |
| | | | 20'0" | | | | | | |
| Ditto | Ditto | Ditto | | | 8 | S.S. | 5/6" 19 | 21.4 | |
| | | | | | | S.T. | | | |
| | | | 25'0" | | | | | | |
| Ditto | Ditto | Ditto | | | 9 | S.S. | 7/6" 19 | 23.3 | |
| | | | | | 10 | 2"S.L. Tapped | | | |
| | | | 30'0" | | | | | | |
| As above, more grits and more sandy | Dark Grey | Firm | | | 11 | S.S. | 6/6" 19 | | |
| | | | 32'8" | | | | | | Refusal at 32'8" |
| Shale bedrock | | | | | | | | | (Presumably bedrock) |

Test Hole Terminated at 32'8"




SITE PLAN

SCALE: 100' TO 1"




SECTION ON HOLES 2 & 1

SCALES: HOR.: 20' TO 1"
VERT.: 10' TO 1"

## LEGEND

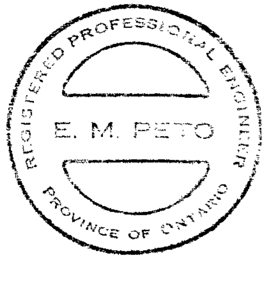
● - BOREHOLE

(14) - BLOWS/FOOT (S.P.T.)

## NOTE:

SEE BOREHOLE LOGS FOR COMPLETE SOIL DETAILS.

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




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

BEAR CREEK BRIDGE

PREPARED BY:
e.m.peto associates ltd.

| JOB No. 62223 | FEB. 1963 | DWN. BY: W.G. | CHECKED BY: RK |
|---|---|---|---|