

#63-F-283 m
BRIDGE OVER
CONESTOGO RIVER
LOT. 1 CONXII
WELLESLEY
TWP.

Materials and Research Division

June 7, 1963

E. M. Peto Associates, Ltd.,
Consulting Engineers,
1287 Caledonia Road,
Toronto 19, Ontario.

Attn: Mr. C. F. Freeman, Chief Engr.

Re: Township of Wellesley,
Prop. Bridge over the Conestogo River,
Lot 1, Con. XII, East Section,
County of Waterloo,
Structure Site No. 34-39,
(Bridge Office File Ref. BA 1646)

Dear Sirs:

In a memorandum to Mr. K. L. Kleinsteinber, Municipal Bridge Liaison Engineer, dated May 31, 1963, we indicated our intention to comment, at a later date, on part of the discussion and conclusions contained in the above-mentioned report, because we felt that some clarification on a number of items is needed.

In the following paragraphs, we are putting forth a number of questions on which we would appreciate having your comments:

In considering spread footings (pages 17 through 19), a computation of the allowable bearing capacity, in terms of effective stresses, is presented. It is stated that the ultimate net bearing capacity is obtained from the familiar Terzaghi's equation, but expressed in effective stress form.

The Terzaghi's equation, being an extension of the basic work of Prandtl and Reissner, is based on the assumption of incompressibility of the material - i.e., no volume change. All stresses in such a case, are total stresses. It is, therefore, difficult to see how a simple substitution of shear strength parameters expressed in terms of effective stresses into this equation, can be made.

cont'd. /2 ...

E. M. Peto Associates,
Attn: Mr. C. F. Freeman

- 2 -

June 7, 1963

On pages 21 through 24, the stability of the bottom of the excavation is discussed. Here, again, it is stated that the analyses were carried out in terms of total and effective stresses. On page 23, the following statement is made: "The effective stress analysis would have to be applied in the event that the silt deposit was disturbed and lost its cohesive strength."

We believe, that under "cohesive strength" the undrained shear strength of the material is meant. If this is correct, then the stability calculation, in terms of total stresses, shows the bottom of the excavation to be unstable. We find it difficult to understand what purpose a stability analysis, in terms of effective stresses, would serve in this particular case.

If you require any additional clarification in connection with the above, please feel free to call on our Office.

Yours very truly,

A. G. Sternac

A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF

cc: Mr. K. L. Kleinsteinber

Foundations Office
Gen. Files

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

Attn: Mr. G.C.E. Burkhardt

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

May 31, 1963

Township of Wellesley,
Proposed Bridge over the Conestogo River,
Lot 1, Con. XII, East Section,
County of Waterloo,
Structure Site No. 34-39,
Bridge Office File No. BA 1646.

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

Because of the scour danger and difficulties due to artesian pressures that would be encountered during construction of spread footings, it is recommended that the footings be founded on piles. The Soil Consultant also gives preference to this solution.

Steel 'H' piles driven to approx. elevation 60.0, are recommended. For design purposes, a safe load of 50 T/pile is suggested. The bearing capacity should be checked on the basis of the Hiley Formula.

The Consultant has, in his report, carried out and discussed different analyses concerning the spread footing design and excavation stability. Since these do not affect the pile design, we are of the opinion that presently, no comments are necessary. However, we will review these at another time, because we feel that a number of items require some clarification.

AGS/MdEF
Encl. (2)

cc: Foundations Office
Gen. Files

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

P.S. -- We are returning to you the drawings,
as requested.

BA 1646

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,
Toronto 19, Ontario.

Our Job Number 6349

769 - 1126.

13th May 1963.

The Township of Wellesley,
c/o McCargar Filer and Hachborn,
Consulting Engineers,
30 Francis Street South,
Kitchener, Ontario.

Attention: Mr. E. G. Hachborn.

63-F-283 M

Gentlemen,

Re: Subsoil Investigation
Conestogo River Bridge,
Hawkesville, Ontario.

We have pleasure in submitting five copies of our
Report Number 6349 on the above investigation.

The subsoil essentially consists of two gravel strata,
separated by a layer of sandy silt occurring between average elevations
71 and 76. Ground water under artesian pressure is present in the
lower gravel and is restrained by the silt deposit.

We have investigated the aspect of possible scouring at this site and have established an opinion that footings placed on top of the silt deposit, approximately at elevation 76, and supporting piers and abutments designed with an object of minimizing the scour depth, would be safe against scour.

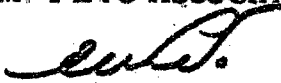
However, the problem is complicated by the presence of the artesian pressure below the relatively thin layer of silt, necessitating special precautions during construction, and introducing an element of doubt into the performance of the foundations in time of flood.

Both the solutions of spread footings, and the alternative pile foundation, penetrating into the lower gravel stratum, are fully treated in the Report. However, in view of the probable difficulties and uncertainties involved in a footing foundation, we are inclined to consider driven piles as the preferable choice.

Although in our opinion the Report is fully comprehensive, we would be very pleased to provide additional assistance, should you wish to discuss further any of the points.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,


E. M. Peto, P. Eng.

RK:sb

THE TOWNSHIP OF WELLESLEY

c/o McCARGAR FILER AND HACHBORN

CONSULTING ENGINEERS

SOILS REPORT

for

CONESTOGO RIVER BRIDGE

HAWKESVILLE, ONTARIO

E. M. PETO ASSOCIATES LTD.,

1287 Caledonia Road,
Toronto 19, Ontario.

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BOREHOLE LOGS (6)

DRAWING - SITE PLAN AND SUBSOIL PROFILES

A. INTRODUCTION:

The work described in this Report was authorized on behalf of the Township of Wellesley by Mr. E. G. Hachborn, of McCargar Filer and Hachborn, Consulting Engineers, on March 28th, 1963.

A new bridge is to replace an existing structure spanning the Conestogo River, one quarter mile east of Hawkesville, Ontario. A site investigation was required for the design of foundations.

The new bridge will be centred on the present structure and its width is to be 38 ft. Approach spans of 50 ft length each and a 70 ft long centre span are contemplated. The Consulting Engineer's propose to design the new bridge as a continuous girder reinforced concrete structure, possibly with open abutments.

The road between the bridge and Hawkesville will be kept low for spillway purposes, if flood conditions are abnormal. However, the Consulting Engineers were very concerned about possible erosion and scour to piers and abutments, following floods.

B. GENERAL INFORMATION:

1. Positions and depths of test holes.

Six test holes were performed at the site, in the positions indicated on the enclosed site plan. In addition, standard cone probe was driven within a few feet of each test hole.

Originally, four test holes were put down, designated Nos. 1 to 4. Of these, No. 1 was on the south side of the proposed eastern abutment, No. 2 (drilled from the existing bridge deck) near the probable location of the eastern pier, and Nos. 3 and 4 in the approximate location of the western abutment.

On the basis of the original four borings, it was established that the subsoil is characterized by a layer of dense silt with pebbles, occurring between approximate elevations 71 and 76, which restrains ground water under an artesian head exceeding the water level in the river, and contained in a sandy gravel stratum which underlies the silt.

It was decided to perform two additional test holes, Nos. 5 and 6, in order to obtain further information about the thickness of the silt stratum and about the artesian pressures; accurate knowledge of these factors was considered to be necessary for the design of the proposed bridge foundations. Test hole 5 was performed on the north side of the proposed location of the eastern abutment and test hole 6, performed from the bridge deck, was located in the probable location of the western pier.

B. GENERAL INFORMATION:

1. Positions and depths of test hole (Cont'd)

The original test holes, Nos. 1 to 4, were put down to a depth ranging from 40.0 to 41.5 ft below the existing grade or bridge deck level; the additional test hole No. 5 was terminated at a depth of 31.5 ft, while test hole 6 was terminated at 41.5 ft. below the bridge deck.

2. Ground elevations.

The ground elevations at the positions of the test holes, measured by our field engineer who also set out the test holes, were referred to a temporary bench mark of assumed elevation 100.0, indicated by the Consulting Engineers. The T.B.M. was a red chalk mark on the south-west corner of the eastern masonry abutment of the existing bridge. The elevations are entered on the borehole logs, site plan and subsoil profiles.

3. Field Work.

Test holes 1 to 4 were performed by our field unit No. 3, between April 5th and 15th, 1963.

Test holes 5 and 6 were performed by our field unit No. 7 between April 24th and 26th, 1963.

Our standard drilling and sampling procedures were followed, as outlined in Appendix A.

B. GENERAL INFORMATION:

3. Field Work (Cont'd)

In addition to the test holes, standard cone probes were driven within 2 to 3 ft of each test hole in the locations indicated on the site plan. The probes were terminated in dense silt or gravel when virtual refusal was encountered.

In all the six test holes, the artesian water pressure in the gravel stratum underlying the dense silt deposit was carefully measured in BX casing projecting above ground level and, where possible, left overnight.

During withdrawal of test hole casing, in every borehole the artesian pressure in the lower gravel was sealed off with cement, in and below the silt stratum.

After completion of the test holes performed from the bridge deck, the holes in the deck were covered with steel plates screwed to the deck.

4. Laboratory Work.

The following soil mechanics tests were performed in our laboratory.

Water content determinations,
Grain size distribution of gravel and sandy silt,
Undrained triaxial compression tests,
Consolidated-undrained triaxial compression tests with
pore water pressure measurements.

B. GENERAL INFORMATION:

4. Laboratory Work. (Cont'd)

The water contents are entered on the borehole logs, while the remaining tests are included in Appendix B.

The results of standard penetration tests performed in-situ are entered on the borehole logs and on the subsoil profiles. The borehole logs include also in the "Remarks" column results of the standard cone probes driven near the relevant test holes. The cone probe results are expressed as the number of blows per successive foot of penetration.

C. SITE AND GEOLOGY:

The site of the bridge is located on a gravel road which runs east from Hawkesville, Ontario and the Conestogo River crossing is approximately 300 yards from Hawkesville.

The existing bridge deck has a single span of 126 ft at deck level and is of steel truss construction with a wooden deck. It is supported on masonry abutments, the stonework of which shows cracks through the cement joints. It is believed that this bridge is about 70 years old.

C. SITE AND GEOLOGY: (Cont'd)

The grade of the bridge deck is 8 to 9 ft above the level of the flood plain near the river, and the bridge is approached by gravel embankments, which rise several feet towards the bridge from a lower level of the gravel road further away from the river. The north side of the western embankment near the bridge abutment is partly collapsed, and a wooden barrier is broken at this point.

The flood plain of the Conestogo River is approximately 500 yards wide and the river at the bridge has a width of approximately 110 ft, flowing towards the south-south-east. It divides into two branches a short distance downstream from the bridge, and encompassing a small island.

At the time of the site investigation, the water surface was at elevation 88.4 to 88.7. In the positions of test holes 2 and 6 the depth of water was 4.1 and 4.5 ft respectively.

The flood plain near the river is at the approximate elevation 95 and consists of grass land with a few large trees. The slopes of the valley rise to Geodetic elevation 1175 within 300 yards to the east and to the west of the site, while a hill located to the south-east reaches elevation 1200 within 550 yards of the site, and 1300 within 1,300 yards.

C. SITE AND GEOLOGY : (Cont'd)

Geodetic elevation of the ground at the site has not been determined but from a topographic map of the area it is estimated to be approximately 1125.

A fairly uniform stratification of subsoil was established, consisting of two sandy gravel deposits, separated by a layer approximately five feet thick, of cohesive sandy silt. The stratification suggests that the site formed a spillway during the glacial periods. The neighbouring ground-moraines, (Waterloo Hills on the south and the western part of the Orangeville Moraine on the north) were in glacial times and are presently drained through this valley. Some of the hills are kames while others are built of a sandy till. Sandy outwash prevails in the hollows.

The gentle topography and the presence of much sandy and gravelly till in the area suggest that these deposits were left by an earlier glaciation and were overridden and rearranged by the Wisconsin glacier.

The lower sandy gravel stratum probably originated after an earlier glaciation, while the silt deposit and the upper gravel were laid down when the last ice sheet began to retreat. However, at least a part of the upper gravel and sand could have been transported by the present Conestogo River.

D. SOIL CONDITIONS:

Details of the subsoil conditions encountered in the test holes are described on the enclosed borehole logs, while simplified subsoil profiles, in the form of sections through the test holes, are presented on the appended drawing.

The subsoil can be subdivided into the following main types, in the order of occurrence,

Silty organic topsoil,
Gravel and sand of variable density (Upper gravel),
Very dense silt with sand and pebbles,
Very dense sandy gravel with sand seams (Lower gravel).

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

a) Topsoil

An organic topsoil, consisting mostly of silt with decayed organic matter, and of brown to black colour, was encountered in test holes 3, 4 and 5 and extended to a depth ranging from 2.5 to 2.7 feet below the existing grade. This material was absent in test hole 1.

b) Upper gravel stratum

A sandy gravel was encountered either below the organic topsoil or immediately below the existing grade (in test hole 3), and below the river bottom in test holes 2 and 6. The gravel extended down to an elevation ranging from 76.1 in test hole 3 to 82.7 in test hole 5; below these depths the gravel was followed by the silt layer.

D. SOIL CONDITIONS:

b) Upper gravel stratum (Cont'd)

The gravel is highly pervious and contains ground water. the head of which corresponds to the water level in the river.

Grain size distribution curves of two typical samples of the sandy gravel are presented on Fig. 1a, Appendix B. The gravel content of the two samples was 57% and 67% and the silt content was 7% and 12%, the remainder being sand.

The gravel was of variable character and derived from various rocks; the majority of the stones were angular or semi-angular in shape. In test hole 1, boulders were found to extend to a depth of 10 ft.

Standard penetration tests performed in the test holes indicated a variable density of the stratum, the results (N-values) showing a spread from as low as 3 to as high as 57 blows per foot, but most of the results were in the range of 12 to 50 blows per foot. The loosest layers were near the top of the stratum.

Standard cone probe results, entered on the borehole logs, are consistent with penetration resistance determinations in the open test holes

D. SOIL CONDITIONS:

c) Sandy silt deposit

The silt deposit, characterized by the presence of sand and pebbles and by a till-form appearance, commenced in the six test holes between elevations 76.1 and 78.7. In addition, in test hole 6 a layer of almost pure silt intervened between the upper gravel and the sandy silt between elevations 80.9 and 78.7, while in test hole 5 the gravel was first followed by a silty clay layer between elevations 82.7 and 81.4, and then by a layered, pure silt between elevations 81.4 and 77.7.

The bottom of the sandy silt with pebbles varied between elevations 69.7 and 73.4.

Grain size distribution curves performed on typical samples of the sandy silt are presented on Fig. 1b, and indicate a variable sand content. The clay fraction in the samples tested varied from 5% to 13%.

Natural water contents in the sandy silt were mainly in the range of 12% to 16%. Some higher values were recorded in the more clayey and less sandy samples.

Standard penetration test results in the silt were high, ranging from 35 to 148 blows per foot, and indicating a stiff, or dense to extremely dense, consistency.

D. SOIL CONDITIONS:

c) Sandy silt deposit (Contd)

Two undrained triaxial compression tests were performed on samples of the silt. The undrained shear strength, assumed equal to one half of the compressive strength, was 3160 and 4090 lb/sq. ft. respectively.

Two consolidated undrained triaxial compression tests were performed on undisturbed samples from this stratum, for the determination of the effective stress parameters. One of the samples was more clayey and the other more sandy. Mohr's envelopes are included in Appendix B. The more clayey sample gave a value of the angle of shearing resistance in terms of effective stresses, ϕ' of 29° , while the more sandy sample gave: $\phi' = 38^\circ$. The effective cohesion intercept, c' was zero in both cases.

On the basis of the above results, an average value of $\phi' = 35^\circ$ was assumed for the silt stratum, as it is mostly sandy.

d) Lower sandy gravel

All six test holes were terminated in a sandy gravel stratum, which followed the sandy silt layer. The greatest thickness of gravel proved was 20.5 ft in test hole 1.

D. SOIL CONDITIONS:

d) Lower sandy gravel (Cont'd)

The gravel was quite variable in composition, including more sandy and silty layers as well as layers of almost clean fine to medium gravel. A characteristic observed feature of the gravel was a more rounded shape of the majority of the stones, in contrast to the more angular shape of most stones in the upper gravel stratum.

The lower gravel stratum contains ground water under artesian head, which in all six test holes consistently corresponded to elevation 97.8 to 98.6. The artesian ground water was encountered soon after penetrating through the silt layer, but the boundary between the silt and the underlying gravel is not very distinct. The lower layers of the silt are very sandy and gravelly and considerably wetter than the greater part of the deposit, while the upper layers of the gravel contain silt and clay seams. The artesian head was fully developed after penetrating about 2 ft into the gravel. However, the gravel stratum contains silty layers of lower permeability, occurring at various depths. The flow of artesian water was slowed down when test hole casing reached such a less pervious layer.

A typical grain size distribution curve of a sample of the less silty gravel is included on Fig. 1c.

D. SOIL CONDITIONS:

d) Lower sandy gravel (Cont'd)

Standard penetration test results in the lower gravel varied from 30 to 135 blows per foot, but most results ranged between 50 and 70 blows per foot, corresponding to a very dense state of packing. The lower values may be affected by the artesian pressure. Standard cone probes soon reached virtual refusal near the top of this stratum.

On the basis of these results, the gravel can be considered as a suitable medium for the support of a pile foundation. It is expected that driven piles would reach refusal a short distance below the top of this stratum.

E. WATER CONDITIONS:

The following measurements of water level in the Conestoga River were made during the site investigation.

<u>Date</u>	<u>River Level</u>
10/4/63	88.4
24/4/63	88.7

The river was 4.5 and 4.1 ft deep in the locations of test holes 2 and 6 respectively. The sand and gravel stratum, resting above the silt layer, contained ground water at a level which corresponded to the water level in the river.

E. WATER CONDITIONS: (Cont'd)

When test hole casing entered the layer of sandy silt, free inflow of ground water was cut off.

When the test holes reached the sandy gravel stratum which underlies the silt deposit, ground water under artesian head was encountered, at an average elevation of 71. The pressure head of this water was carefully measured in all six test holes by means of BX test hole casing projecting above the ground or river level, and observations were made until the water level became constant. Between the 5th and 26th April, 1963, the artesian head was in all six test holes consistently between elevations 97.8 and 98.6.

During the withdrawal of casing, in every test hole the artesian pressure was carefully sealed off with cement, in and below the silt layer.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

1. General Considerations.

The subsoil consists essentially of two sandy gravel strata, separated by a deposit of sandy silt with pebbles, occurring between average elevations 71 and 76. The lower gravel stratum contains ground water under artesian head corresponding to an average elevation 98.5 at the time of the site investigation. The artesian water is constrained by the silt layer.

The proposed bridge foundations, in the form of spread footings, could be placed on top of the silt deposit near elevation 76. According to the information which we were able to obtain concerning the likely depth of scour in the Conestogo River, the footings, placed at this depth, should be safe against scour, though some additional measures for their protection, in the form of sheeting driven another two feet into the silt, and of rip-rap on the river bottom and banks in the vicinity of the bridge are recommended.

However, the presence of artesian water under the silt layer, as well as the possibly variable thickness of the silt, necessitate considerable care in the construction of the foundations, so that the silt should not be pierced and the artesian pressure released. This may suggest the inclusion of temporary relief wells outside the excavations.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

1. General Considerations (Cont'd)

Also, the artesian pressure may vary seasonally and major changes in it could conceivably cause some minor vertical movement of foundations, which cannot be reliably predicted on theoretical grounds.

For the above reasons, a pile foundation may be simpler and safer. Steel H-piles may be most appropriate and should be driven into the lower gravel stratum, containing artesian water. The piles may reach refusal a short distance below elevation 70.

In the following sections, the bearing capacity and stability of footings placed on the silt layers, as well as the use of the alternative pile foundation, are considered.

2. Footing foundations

The bearing capacity of footings placed on the silt stratum near recommended elevation 76.0 was considered both in terms of total stresses and in terms of effective stresses.

In the total stress analysis, the allowable bearing capacity was calculated on the basis of the undrained shear strength of the silt, together with the standard penetration test results and the cone probe resistance data. The allowable bearing capacity of foundations, for a factor of safety against shear failure of three, was estimated as 4.0 ton/sq. ft.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

2. Footing foundation (Cont'd)

The computations of the allowable bearing capacity in terms of effective stresses were made on the basis of results of the effective stress parameters, determined on two undisturbed samples of the silt. The angle of internal friction in terms of effective stresses was assumed to have an average value of 35° . The effective cohesion was zero.

The critical condition of effective stress was assumed to correspond to pore water pressure in the lower gravel below the foundations equal to the artesian head of elevation 100.0.

The ultimate net bearing capacity, q_u in terms of effective stresses was obtained from the familiar Terzaghi's equation, but expressed in effective stress form:

$$q_u = c'.N_c + p'.(N_q - 1) + \frac{\gamma'.B.N_{\gamma}}{2}$$

Where N_c , N_q , N_{γ} are bearing capacity factors,

dependent on angle of friction, ϕ' ,

p' is effective overburden pressure

c' is effective cohesion,

γ' is submerged density of subsoil.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS

2. Footing foundation (Cont'd)

The first term of the above equation is zero due to c' being zero, as determined by the tests. The second term was found to be ineffective under critical conditions of artesian pressure and scour. Thus, for a factor of safety of three, the allowable bearing capacity equation is reduced to the form:

$$q_a = \frac{\gamma' N_\gamma}{6} \cdot B ;$$

For $\gamma' = 70$ p.c.f.

$$N_\gamma = 42 \text{ for } \phi' = 35^\circ$$

$$q_a = \frac{B}{4} \text{ ton/sq. ft.}, \text{ where } B \text{ is the width of footing in feet.}$$

This equation should be used to obtain the bearing capacity, but the upper limit of allowable pressure determined by the total stress analysis is 4.0 ton/sq. ft.

The possible effect of a reduction of the angle of friction ϕ' due to disturbance of subsoil by the artesian pressure, (possibly during construction or as could occur following the removal of the upper gravel by scouring) was considered. For a reduction of ϕ' from the assumed value of 35° to 30° , the allowable bearing capacity would be halved ($q_a = \frac{B}{8}$), while for $\phi' = 25^\circ$, $q_a = \frac{B}{10}$ ton/sq. ft. was obtained. Such possible reduction of ϕ' is partly covered by the factor of safety of three included in the value of the

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

2. Footing foundation. (Cont'd)

allowable pressure; however, the analysis showed that scour and artesian pressure considerations can be critical in this project. If the solution of footings on silt is adopted, provisions for the permanent relief of the artesian pressure could be considered.

3. Excavations.

a) Performance of excavations.

Should a solution calling for the construction of footing foundations placed on the silt be adopted, it was assumed that the excavation would be performed inside a cofferdam consisting of sheet piling, driven a short distance below the top of the silt stratum, or to the approximate elevation 75, in order to cut off the inflow of water from the upper gravel stratum. Water contained within the cofferdam would then be pumped out and excavation and construction performed in the dry.

Driving the sheeting to below elevation 75 should be avoided in order not to disturb the silt, which could result in a blow-out of the bottom of the excavation by the uplift water pressure in the lower gravel stratum. However, the elevation of the surface of the relatively impervious silt may vary to some extent; should piling driven to elevation 75 not be effective in cutting-off the ground water, it should be driven deeper, a few inches at a time, until it has achieved its purpose.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

3. Excavations

a) Performance of excavations (Cont'd)

It must be pointed out, however, that the driving of sheeting at this site may be a relatively difficult operation. The upper gravel contains dense layers and is largely of angular shape. In addition, boulders were present in it, particularly numerous down to a depth of 10 ft in test hole 1. Resistance to driving in the dense silt would be particularly great. The sheeting would thus have to be strong, though the necessary length of the piling will probably be only 15 to 18 ft.

Alternative solutions to piling for performing the excavations were considered, but none appears very practicable. Construction of earth cofferdams around the excavations would not be effective because of underseepage through the upper gravel. The gravel would have to be dredged out to the top of the silt deposit and replaced with impervious material. An alternative solution, of diverting the flow and lowering the water table in the upper gravel would also probably be impractical due to the considerable volume of flow in the river.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

3. Excavations (Cont'd)

b) Stability of sheet piling

The stability of the walls of a cofferdam consisting of sheet piling driven to the top of the silt stratum would be achieved by means of horizontal struts, spanning the excavation, and designed by a method applicable to cohesionless soil.

The lateral pressure diagram and proposed design distribution of the pressure on the struts are shown graphically in Fig. 3, Appendix C.

c) Stability of bottom of excavation

There exists a distinct danger of a blow-out of the bottom of excavation, due to the uplift pressure in the lower gravel stratum.

The stability of the bottom of excavation was analysed in terms of both total and effective stresses, assuming that the excavation, performed dry within a sheet-pile cofferdam, will reach the silt stratum near elevation 76.

Further, the critical subsoil conditions were assumed to be as established in test hole 3, where the silt layer was apparently only 3 ft thick.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

3. Excavations

c) Stability of bottom of excavation (Cont'd)

(i) The total stress analysis assumes that the silt is cohesive, will not be disturbed by the excavation and possesses an average undrained shear strength of 3500 lb/sq. ft, which will remain effective when the excavation has reached this deposit.

The results of the analysis are presented graphically on Fig. 4, Appendix C, in the form of curves of maximum allowable elevation of the artesian water pressure head in the lower gravel stratum, for a range of widths of excavation and ratio of length to width. Curves for factors of safety of 1.0 and 2.0 are provided.

Because of the uncertainty of the exact effective thickness of the cohesive silt and very limited data on the undrained shear strength, it is recommended to work to a factor of safety of 2.0. Examining the curves in Fig. 4, it will be observed that, for the condition of artesian pressure head extending to elevation 98 as obtained during this investigation, no lowering of the artesian pressure is required for widths of excavation up to 11 ft, ($L/B = 3$). An excavation 20 ft wide ($L/B = 2$) would require that the artesian pressure head in the lower gravel stratum be reduced to elevation 91.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

3. Excavations.

c) Stability of bottom of excavation

(i) (Cont'd)

Reduction of the artesian pressure head to an acceptable level could probably be achieved by providing relief wells outside the excavation, which would freely discharge into the river. Pumping would be needed only for a lowering of the pressure head to below the river level.

(ii) The effective stress analysis would have to be applied in the event that the silt deposit was disturbed and lost its cohesive strength. The ultimate result of such occurrence would be the establishment of "quicking" conditions, and the analysis of stability then would be analogous to piping criteria in cohesionless soils.

Assuming that the quicking conditions would prevail over an area of the excavation which is large compared to the thickness of the silt, stability of bottom of excavation could be achieved only by lowering the pressure head in the lower gravel deposit to a level above the top of the silt equal approximately to the thickness of the silt layer. For the assumed critical case of the silt extending between elevations 76 and 73, as in test hole 3, the artesian pressure would have to be lowered to elevation 79 for stability, and pumping would be necessary.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

3. Excavations

c) Stability of bottom of excavation

(ii) (Cont'd)

Reduction of the quickening potential by driving the sheeting sufficiently deep into the lower gravel stratum would probably be impractical, due to a great resistance to driving in this very dense deposit.

A comparison of the requirements for stability of bottom of excavation in terms of total and effective stresses underlines the great importance of preserving the cohesive properties of the silt deposit and care necessary in the performance of excavations.

4. Pile foundations.

The earlier discussion has indicated that founding the bridge on footings, resting on the silt stratum, involves numerous difficulties and uncertainties.

An alternative solution would be to support the bridge on piles. Driven, steel H-piles appear appropriate and should penetrate into the lower gravel stratum.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

4. Pile foundations (Cont'd)

The depth to which such piles should be driven may depend on the resistance encountered, but, for the foundation to be safe from effects of possible "quicking" in the silt and gravel, they should preferably be set not higher than elevation 60. However, hard driving conditions are anticipated and the pile tips should be reinforced.

The allowable bearing capacity of such piles would be very high and may be determined by the pile design specifications. However, a pile test is recommended, and should be carried out on a pile preferably after several piles in a group have been driven.

Short, prebored piles, resting on top of the silt stratum would be analogous to footing foundations but probably easier to execute, (cast in lining driven to top of silt). However, they may be unsafe after critical scouring conditions, when they could lose their lateral support, and it is doubtful whether steel sheet piling placed outside the pile group would provide adequate scour protection unless driven well into, or through the silt stratum.

F. ENGINEERING CONSIDERATIONS AND CONCLUSIONS:

5. Embankments

The new embankments will not be high and will rest mostly on the upper gravel stratum, so that little settlement is anticipated. However, the organic silty topsoil, which was up to 2.7 ft thick in the test holes, should be scraped off before placement of the embankment fill.

Abundant gravel is present at the site and in quarries in the vicinity of the site, providing excellent material for the construction of the embankments.

E. M. PETO ASSOCIATES LTD.,

C. F. Freeman

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

RK:sb

Report Prepared By:

R. Kulesza

R. Kulesza, P. Eng.

Job No. 6349

May, 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 hose.

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.

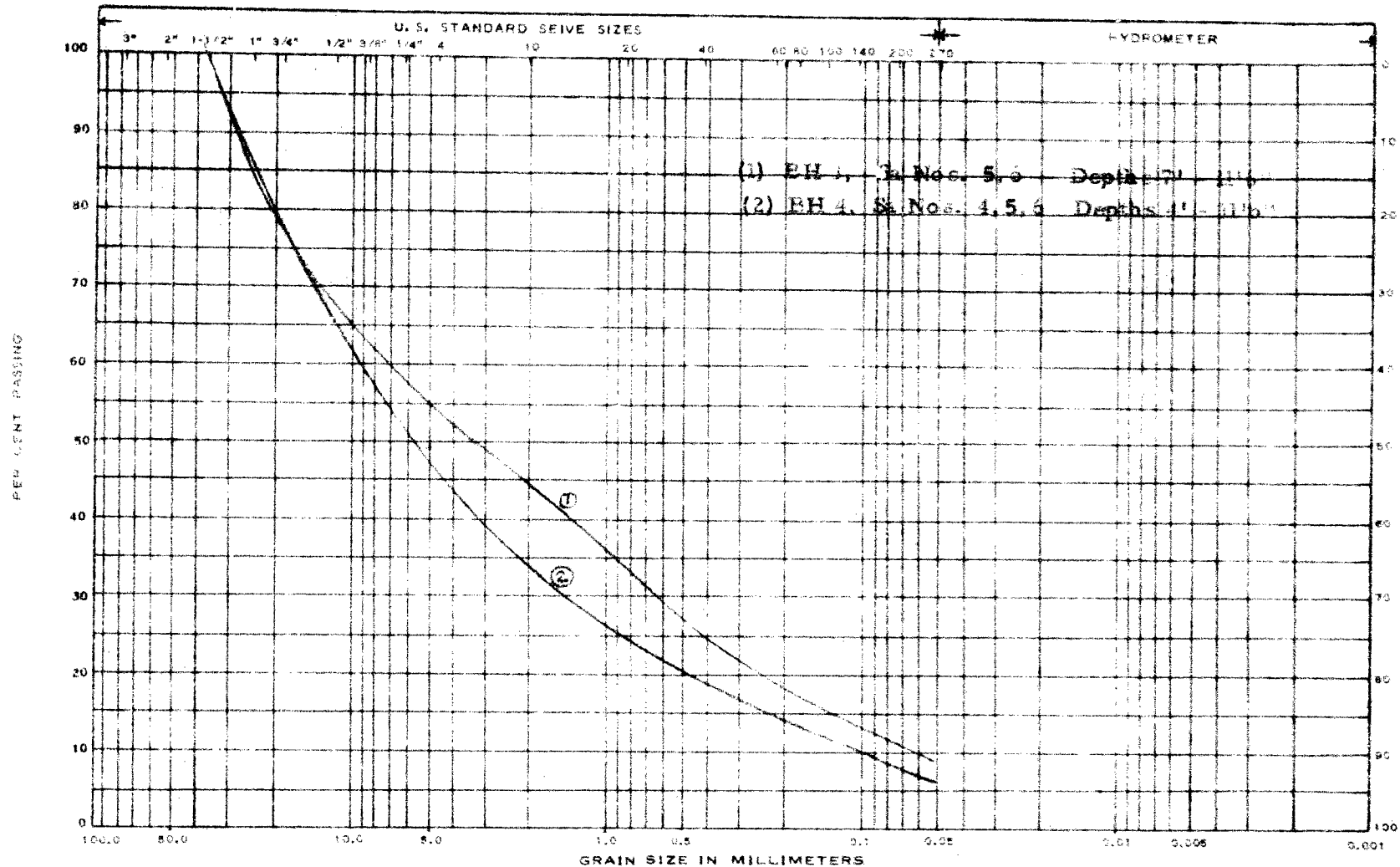
Normally, the standard penetration blows and the natural moisture contents are plotted against elevation as a graph, and these graphs form part of the appendices, together with laboratory test result details, ground water readings and other soil characteristics which can be best illustrated in graphical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

APPENDIX "B"

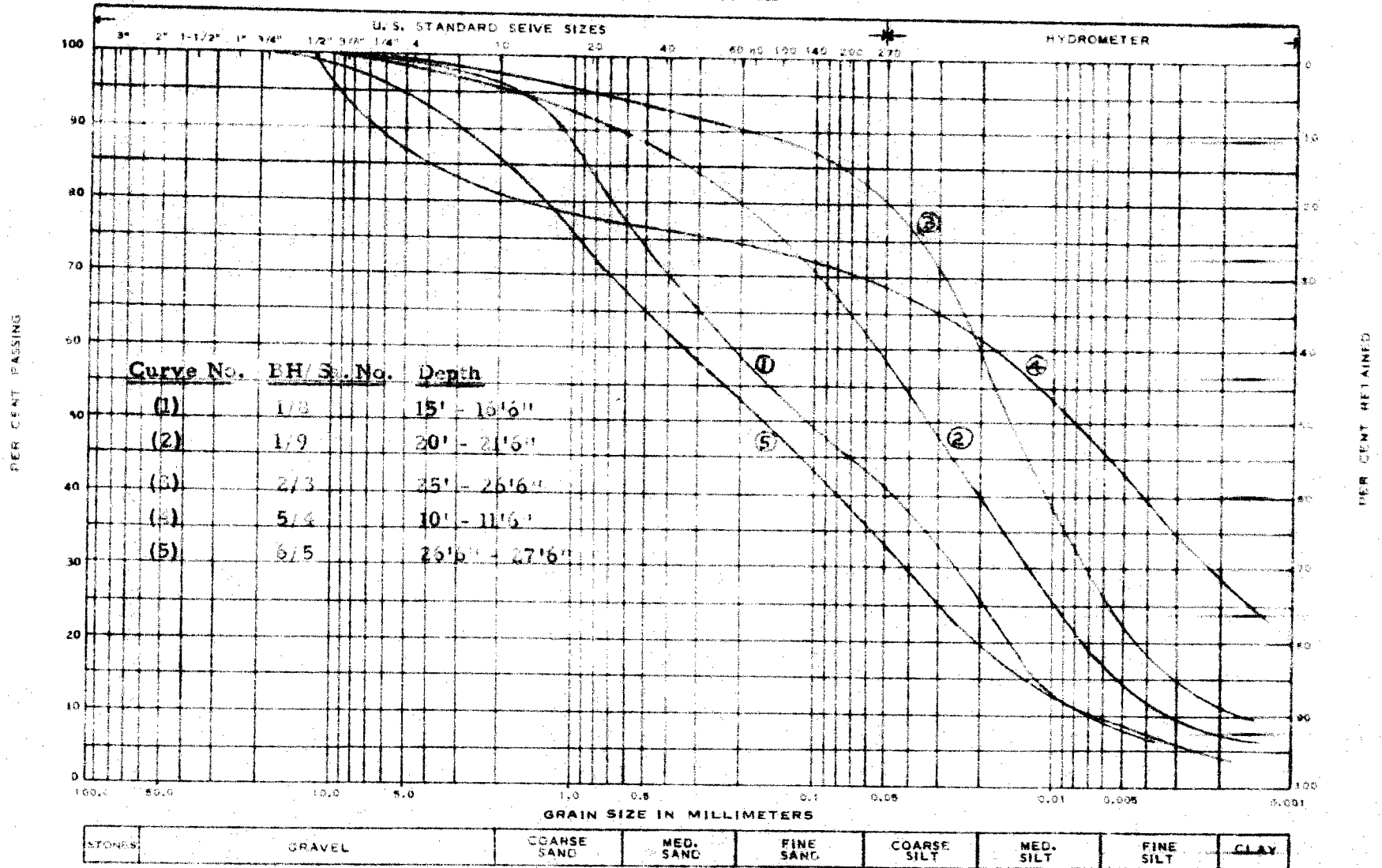
LABORATORY TEST RESULTS

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Toronto 19, Ontario



e. m. peto associates ltd.
Toronto 19, Ontario



MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Conestogo River Bridge JOB NO. 6347 HOLE NO. As shown SAMPLE NO. As shown

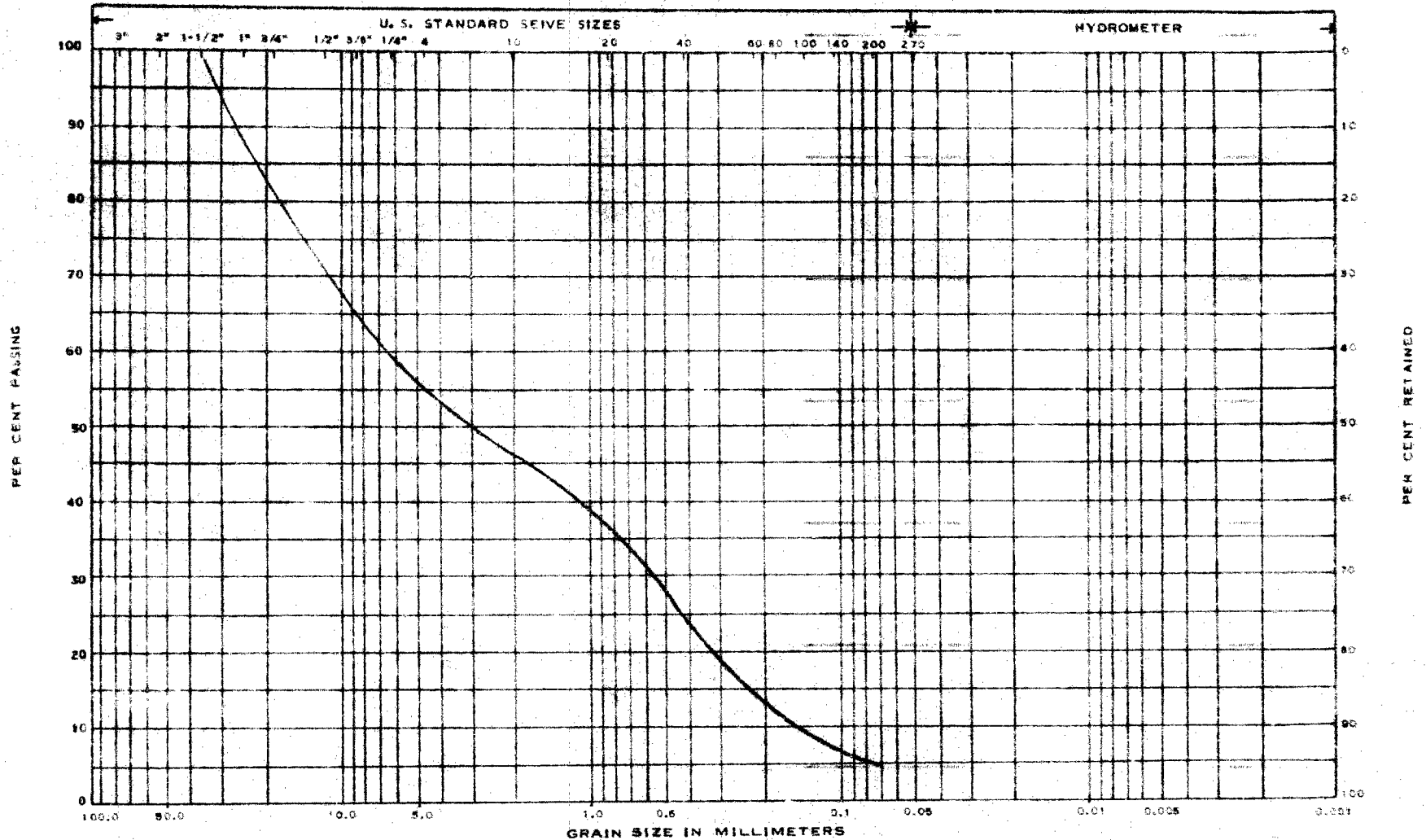
DEPTH As shown ELEVATION As shown REMARKS Sandy silt with gravel and clay

GRAIN SIZE DISTRIBUTION DEFECTS IN NEGATIVE COPY
CONDITION OF ORIGINAL DOCUMENT

Fig. 16

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Toronto 19, Ontario



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

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Conestogo River Bridge JOB NO. 6349 HOLE NO. 3 SAMPLE NO. 10
 DEPTH 25'-26'6" ELEVATION _____ REMARKS Lower, sandy gravel

GRAIN SIZE DISTRIBUTION

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

FIG. 10

UNDRAINED TRIAXIAL COMPRESSION TESTS

<u>BH/SA No.</u>	<u>Depth</u> <u>ft.</u>	<u>Bulk</u> <u>Density</u> <u>p. c. f.</u>	<u>Dry</u> <u>Density</u> <u>p. c. f.</u>	<u>Water</u> <u>Content</u> <u>%</u>	<u>Void</u> <u>Ratio</u>	<u>Strain at</u> <u>failure</u> <u>%</u>	<u>Undrained</u> <u>shear</u> <u>strength</u> <u>p. s. i.</u>
2/3	26	147.5	131.0	12.7	0.35	20	4090
4.8	16	150.0	136.0	10.1	0.23	20	3160

CONSOLIDATED - UNDRAINED TRIAXIAL TESTS WITH PORE WATER PRESSURE MEASUREMENT

Multi-stage testing of single specimen

Maximum effective stress ratio failure criterion

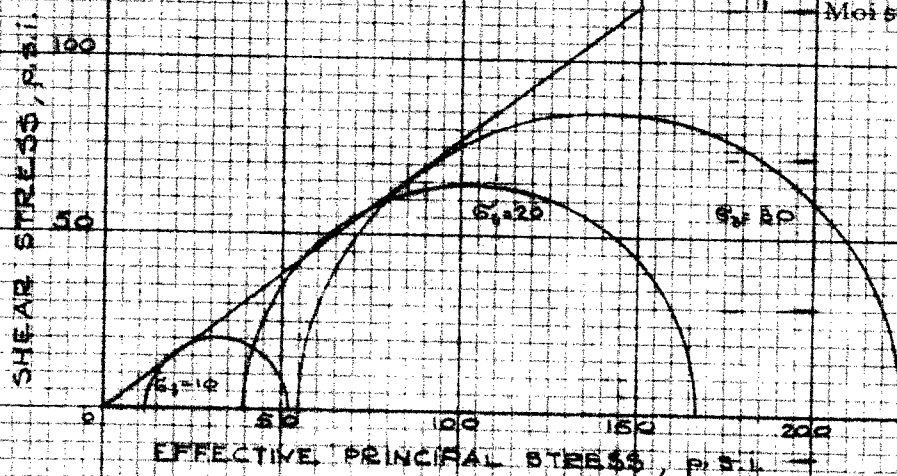
Borehole #6 Sample #5

Depth 26'6" - 27'6"

Sandy & gravelly silt

Initial wet density 141.0 p.c.f.

Moisture content 14.6%



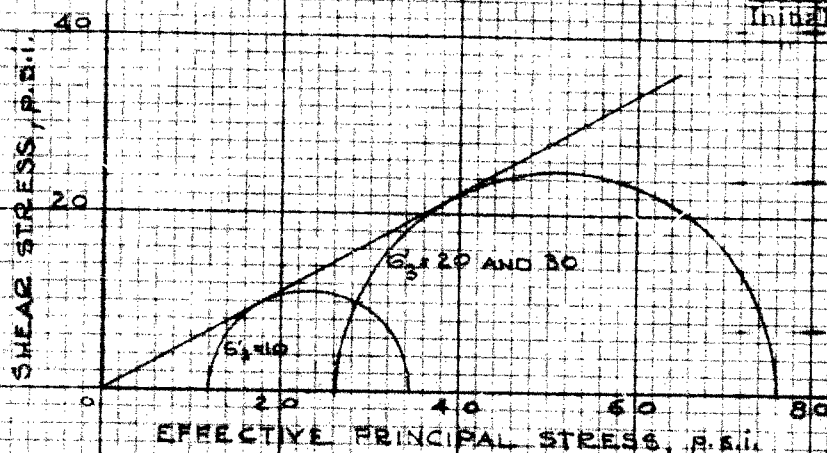
Borehole #5 Sample #4

Depth 10' - 11'6"

Clayey silt

Initial Net density 138.0 p.c.f.

Moisture content 15.3%



Lab #6349

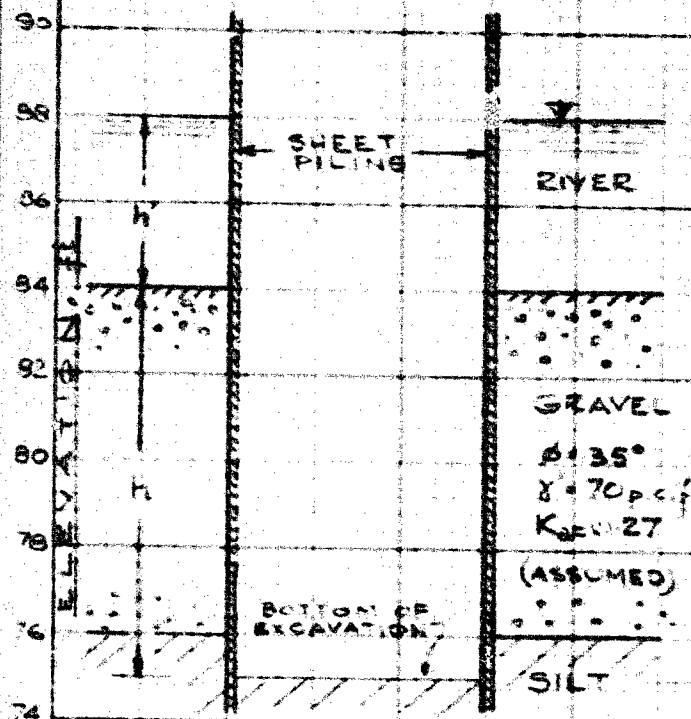
Fig. 2

APPENDIX "C"

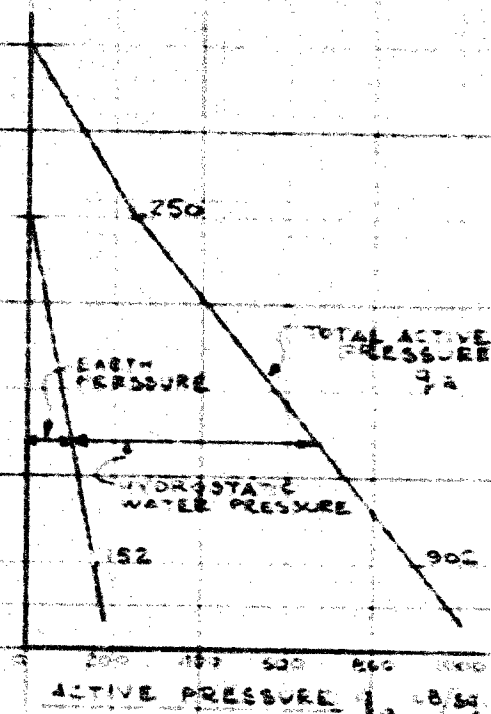
**STABILITY OF EXCAVATION and SHEET
PILING DIAGRAM.**

LATERAL PRESSURE ON SHEET PILING

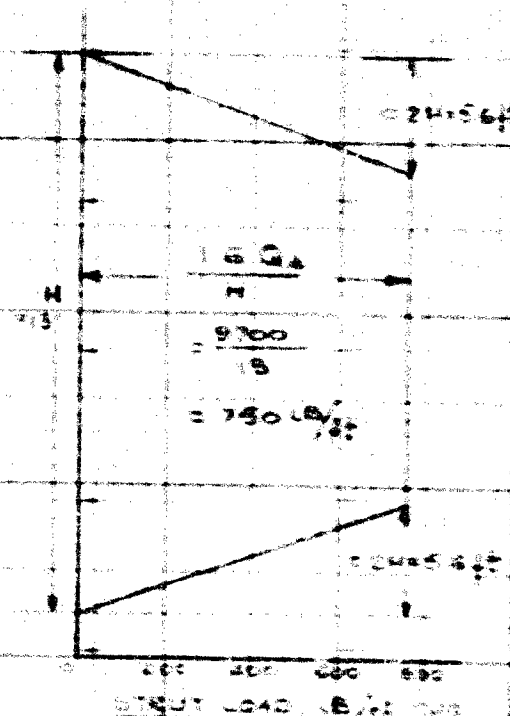
ASSUMED CONFIGURATION



ACTIVE PRESSURE DIAGRAM



STRUT LOAD DISTRIBUTION DIAGRAM



ACTIVE PRESSURE INTENSITY $q_a = K_a \cdot \gamma \cdot h + \gamma_w (h - h_w)$

TOTAL ACTIVE PRESSURE $Q_a = \frac{K_a \gamma^2 h^2}{2} + \frac{\gamma_w (h - h_w)^2}{2}$

NOTE:

FOR DIFFERENT CONFIGURATIONS, THE PRESSURE / LOAD DIAGRAMS MUST BE ALTERED, USING THE GIVEN EQUATIONS FOR q_a / Q_a

Fig. 10-10-10

Soil: Undrained shear strength $C_u = 1500$ p.s.f.
 Bulk density $\gamma = 145$ p.c.f.
 Governing equation of stability:

$$F = \frac{1}{1 + \frac{1}{2} \frac{C_u}{\gamma H} \left(1 + \frac{1}{2} \frac{C_u}{\gamma H} \right)}$$

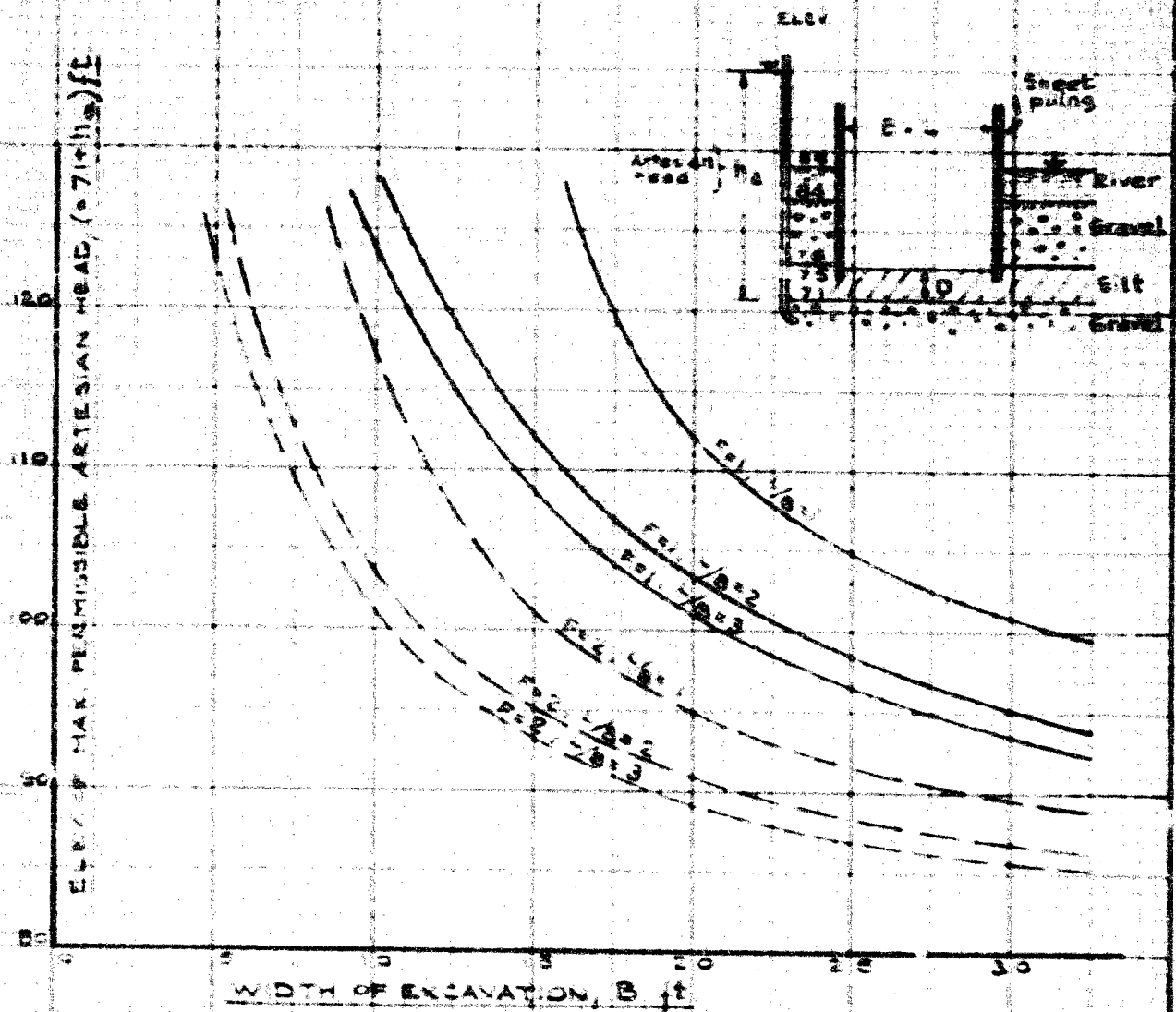


FIG. 4. STABILITY OF BOTTOM OF EXCAVATION

Total stresses
 B = Width of excavation
 L = Length of excavation
 F = Factor of safety
 (on shear strength of silt only)

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SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Conestogo River Bridge Job No. 6349 Borehole No. 1
Twp. of Wellesley, 4" to 15'
Client c/o McCargar Titer & Hachborn, Casing BX to 40" Boring Date April 5 & 6, 1963
Elevation 92.4 (Arbitrary) Compiled By RK Checked By JD

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	COLOR	Density & Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Penetration (inches)	WATER LEVELS & REMARKS
			0' 0"						Std. cone blows /ft
Coarse, sandy gravel	Brown				1	CS			Stones up to 4"
Coarse gravel in clay mixed with sand	ditto	v. loose			2	CS			
ditto					3	CS			Could not drive SS because of boulders
		Compact			4	CS			ditto
			7' 0"						ditto
Fine to coarse gravel in v. silty & clayey sand	Light grey-brown	Dense			5	CS			
		ditto							Cone probe 3 ft
		ditto	10' 0"						North of B.H. 1
Fine to coarse gravel, quite clean	Brown	Compact			6	SS	18"		Softer at 10'
Gravel in clayey sand	Light grey	V. dense			7	SS	50/6"	14.2	Wash water used to advance testhole below 10 ft
Sandy silt and pebbles			14' 0"						
			14' 6"						
Silt with layers of v. silty sand	Light grey-brown	ditto	77.9		8	SS	61	13.2	
ditto	ditto	ditto	21' 0"		9	SS	61	19.3	Artesian water struck at 21 ft (El. 71.4)
Mainly medium, silty sand, some grits, Silty gravel		ditto	71.4						
			24' 0"						
Fine to medium sand	Light brown	Compact to dense			10	SS	30	20.3	With casing at 25', water level overhight at 5' 9" above ground level.
Gravel, denser			27' 6"						(El. 98.2)
			29' 9"						Water overflowing ground
Fine to coarse gravel with fine to coarse silty sand	Light grey-brown	V. dense			11	SS	70		- level at rate 1 gal. in 6 sec.
As above	ditto	ditto			12	SS	57	8.9	Casing to 35', water level 6' 3" above ground level
(Fine gravel)					13	WS			
			39' 0"						Softer
Fine to coarse gravel with fine silty sand	Yellow-brown	Dense			14	SS/	37		
			41' 6"		15				Water samples (2 jars)
					15A				
									Testhole terminated 41 ft. 6 in.

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"						Std. cone probe blow/ft
Bridge deck			101.5"						
									Casing at 40', hole to 41' 6", artesian water at 3' 3" overnight (el. 98.2)
Water surface			13' 0"						
			88.5						
Water									
River bottom	Brown	Dense	17' 6"						17' 7"
Fine to coarse, sandy gravel, some silt			84.0	1	X	SS	8/6"		
Layers of silt with pebbles and gravelly sand	Ditto	ditto		2	X	SS	32.6"	9.5	
							14/6"		
							43	7.6	
			24' 0"						
			77.5						
Silt with some grits & sandy lenses	Grey-brown	ditto		3	X	SS	35	12.3	
Ditto			30' 3"						
Fine gravel with fine to coarse, silty sand	Light brown	V. dense	71.2	4	X	SS	97	12.6	
									Artesian water struck at 30' 3"
			34' 6"						Casing at 35', sand backed up to 31 ft.
Mainly medium sand with fine to medium gravel	Brown	Dense		5	X	SS	44	18.7	
			40' 6"						
Fine to coarse gravel with coarse to medium silty sand.	ditto	V. dense	41' 6"	6	X	SS	98	11.4	
			60.0						
Testhole terminated 41 ft 6 in.									

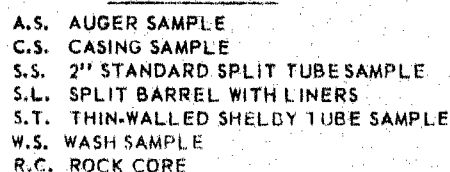
SAMPLE CONDITION		SAMPLE TYPE		ABBREVIATIONS	
	UNDISTURBED	A.S. AUGER SAMPLE	V.T. IN SITU VANE SHEAR TEST		
	FAIR	C.S. CASING SAMPLE	M. MOIST		
	DISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	W.L. WATER LEVEL IN CASING		
	LOST	S.L. SPLIT BARREL WITH LINERS	W.T. GROUND WATER TABLE IN SOIL		
		S.T. THIN-WALLED 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"						Std. cone probe blows/ft
Silty organic topsoil, layers of sand	Dark brown to black	V. soft			1 	CS			4
Layers of organic silt and gritty sand, some gravel and wood fragments	Dark brown	ditto	2' 6"		2 	CS			13
					3 	SS	3	25.6	5
	Various	Loose to compact	5' 0"		4 	SS	1 7/6"		2
							4 7/6"		9
							10 7/6"		35
Fine to coarse, angular gravel with silty sand	Light brown	ditto			5 	SS	12		20
									15
As above, more sandy	ditto	Compact			6 	SS	22	17.7	11
Mainly fine gravel & coarse sand, quite clean	Dark Brown & Grey	Dense			7 	WS	47		10
									12
As above, with coarse gravel	ditto	ditto			8 	SS	3.2		17
									35
			17' 6"						60
			76.1						47
Layers of sandy silt with grits									20
Medium to coarse, silty sand with mainly fine gravel.	Dark grey-brown	ditto	20' 6"		9 	SS	18 7/6"		47
			73.1		9A 		32 7/6"	14.9	70
			22' 0"				33 7/6"		70
									80
									325
									
Fine to coarse, sandy and silty gravel	Light brown	ditto			10 	SS	53	13.6	
									
			28' 0"						
									
Fine to coarse gravel with medium to coarse sand, quite clean	Brown	ditto			11 	SS	52		
									
									
As above, more silty	Light brown	ditto			12 	SS	63 7/6"		
							30 7/6"		
							18 7/6"		
Mainly coarse sand and fine gravel	Brown	Extr. dense	40' 0"		13 	SS	75 7/6"		
			53.6				48 7/6"		

SAMPLE CONDITION

SAMPLE TYPE

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

[illegible]

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Conestogo River Bridge,		
Job Name	Twp. of Wellesley,	Job No. 6349
Client	c/o McCagar Filer & Hachborn,	Borehole No. 5
Elevation	92.2 (Arbitrary)	Casing 4" to 5', BX to 30'
	Compiled By RK	Boring Date 24 & 25 April 1963.
		Checked By JD

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

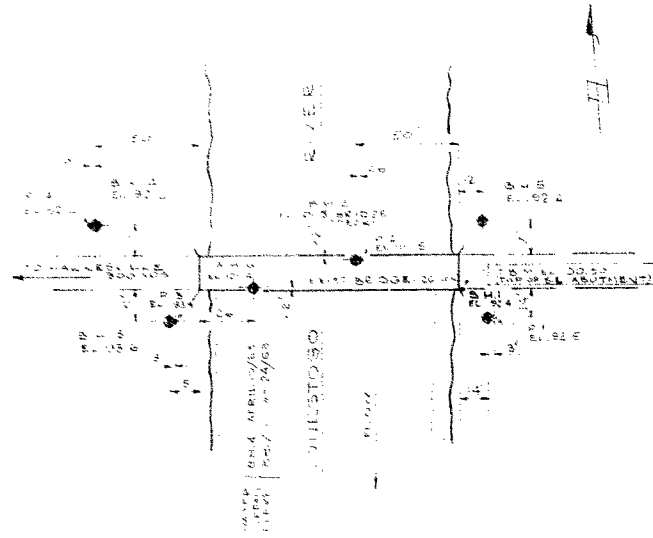
[illegible]

Checked By JD

ABBREVIATIONS

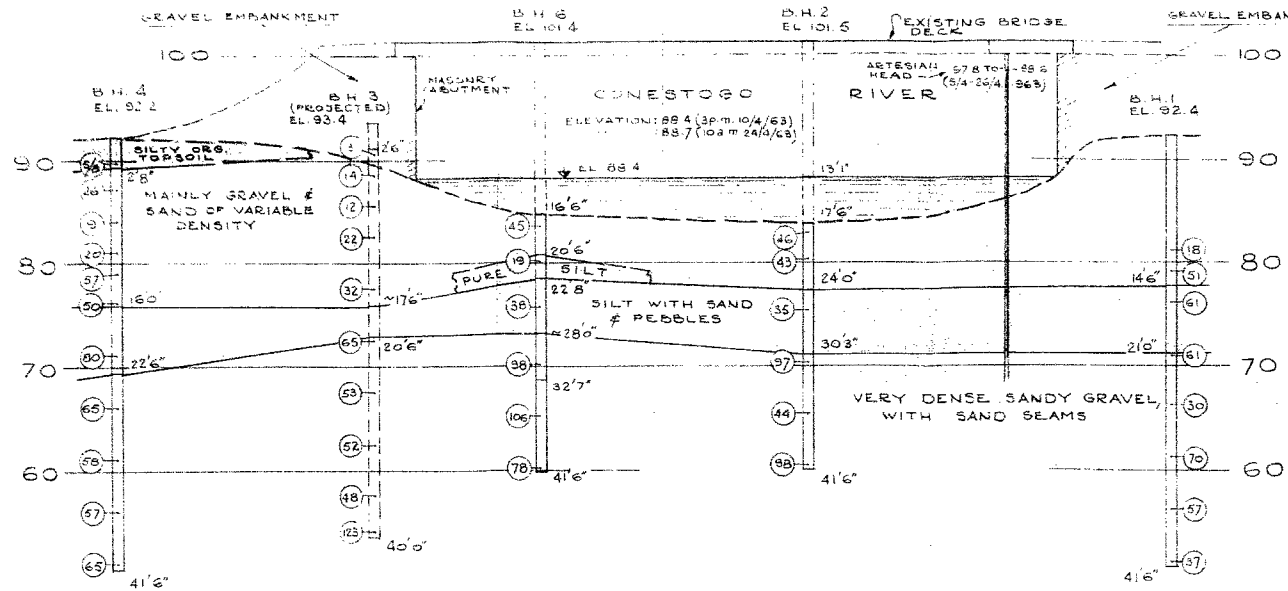
V.T.	IN SITU VANE SHEAR TEST
M.	MOIST
W.L.	WATER LEVEL IN CASING
W.T.	GROUND WATER TABLE IN SOIL
W.T.P.L.	WETTER THAN PLASTIC LIMIT
D.T.P.L.	DRIER THAN PLASTIC LIMIT
A.P.L.	ABOUT PLASTIC LIMIT

[illegible]



SKETCH SHOWING BOREHOLE AND PROBE-HOLE LOCATIONS

SCALE: 50' TO 1"



SECTION ON HOLES 4, 3 (PROJECTED), 6, 2 & 1

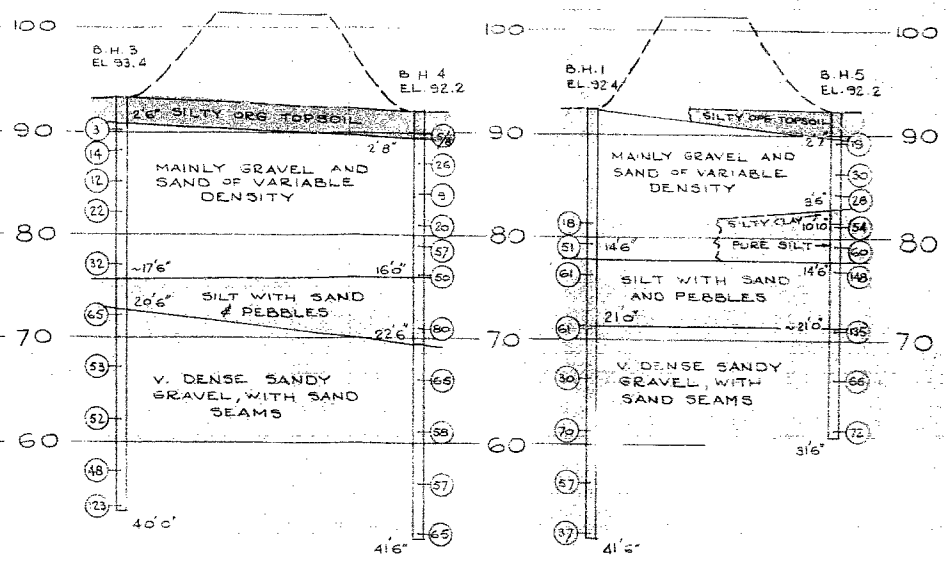
HOR: 20' TO 1"
VERT: 10' TO 1"

LEGEND:

- - BOREHOLE
- + - PROBEHOLE
- (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.



SECTION 3-4

SECTION 1-5



THE TOWNSHIP OF WELLESLEY
c/o McCARGAR FILIP & HACHBORN
CONESTOGO RIVER BRIDGE
HAWKESVILLE, ONT.
PREPARED BY:
e.m. peto associates ltd.
JOB #6349 MAY 1963 DWN BY W.G. CHECKED: RM