

62-F-279M

BEAR CREEK

BRIDGE LOTS 64

CON 111 S.E.R

WARWICK

TWP

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Division,
(Foundation Section)

March 22, 1962.
REVIEW OF SOIL REPORT BY
E. M. PETO ASSOC., LTD. and
PRELIMINARY DWGS. BY MONTEITH
ASSOC., LTD. (Br. Ref. BA 1364)

Attention: Mr. K. L. Kleinsteinber,
Municipal Bridge Liaison Engr.

Re: Township of Warwick,
Bear Creek Bridge, Lambton County
Lots 6/7, Con. III, S.E.R., Dist. #1.

We have reviewed the Soil Report prepared by
E. M. Peto Associates, Ltd., and the bridge drawings prepared
by J. A. Monteith Associates, Ltd., and herewith submit our
comments for your consideration:-

If scour protection can be assured, economic
reasons should decide between spread footings or piles, both
sitting on bedrock. If spread footings are used and placed
half a foot into bedrock, the wedge as shown on the bridge
drawing, can be dispensed with.

We cannot agree with the reasoning put forward by
the Soil Consultant concerning the difficulties connected with
the bored piles or caissons. It seems that the Consultant is
not familiar with this kind of pile and his suggestion of stopping
the piles one or two feet above bedrock is most unusual.
Such piles, as he writes, could be loaded with only 10 to 12 tons/
pile, which makes the proposition very uneconomical.

AGS/MdeF
Encls.
cc: Foundations Office
Gen. Files.

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

P.S. -- Drawings being returned herewith,
as requested in your memo of Mar. 14/62

OFFICE LOCATION -
DOWNSVIEW AVE.,
KEELE ST. - HIGHWAY 401
TORONTO, ONTARIO.



ONTARIO
DEPARTMENT OF HIGHWAYS

POSTAL ADDRESS -
DEPARTMENT OF HIGHWAYS
PARLIAMENT BUILDINGS.
TORONTO 5, ONTARIO.

Bridge Division,
March 14, 1962.

MEMORANDUM TO:

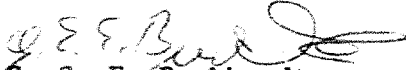
Mr. A. G. Stermac,
Principal Foundation Engineer,
Department of Highways,
Room 107, Lab. Building,
DOWNSVIEW, Ontario.

RE: Township of Warwick
Bear Creek Bridge
Lambton County
Lots 6/7, Con III
S.E.R. Our file
#BA 1364

Attached please find a copy of the
Foundation Report, by E.M. Peto Associates
Limited, and a copy of Preliminary Plans for
your comments.

We would like to approve the
Preliminary design within the next two weeks
and would appreciate it very much if we
could have your comments within two weeks.

Since we have only two copies of
the Preliminary Plans, we would like to have
the copy back which we are sending to you
to-day.


G. C. E. Burkhardt,
for K. L. Kleinsteinber,
Municipal Bridge Liaison Engr.

GCEB/z

BA 1364

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,
Toronto 19, Ontario.

Our Job Number 6226

Russell 9 - 1126.

February 23rd, 1962.

The Township of Warwick,
C/O J. A. Monteith Associates,
Consulting Engineers,
P. O. Box 579,
Petrolia, Ontario.

Gentlemen,

Re: Site Investigation
6/7 Sideroad Bridge
Township of Warwick.

We have pleasure in submitting four (4) copies of
our Report Number 6226 on the above site investigation.

The two test holes have disclosed that the black
shale bedrock, of good bearing capacity, commences between the
elevations 83.5 and 84.5, i.e. approximately at a depth of 10 ft.
below the ground surface in the area.

If the abutments are placed on the surface of the
shale, the allowable bearing capacity will be as high as 25 tons
per sq. ft. However, the excavations will encounter a considerable

quantity of groundwater in a layer of broken shale and silt resting on the surface of bedrock. The water was under head of the river.

For this reason, we have considered placing the footings at the elevation 89 or 88, in a clayey till stratum, where the allowable net bearing capacity would be 3 tons/sq. ft. Little water seepage would occur in excavations not reaching below the above elevations. However, it would be necessary to investigate a possible danger from scouring if the footings are placed at that depth.

Piles driven to refusal on the surface of the shale may prove most economical; the use of prebored caissons is also considered in the report.

Some settlement of approach embankments, particularly on the north side of the bridge, must be expected.

We consider the report to be fully comprehensive, but would be most pleased to provide further assistance should you wish to raise any points.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

P. L. Haman.

for E. M. Peto, P. Eng.

RKAp

THE TOWNSHIP OF WARWICK
c/o J. A. MONTEITH ASSOCIATES
CONSULTING ENGINEERS

SOILS REPORT

for

6/7 SIDEROAD BRIDGE

TOWNSHIP OF WARWICK, ONTARIO

E. M. PETO ASSOCIATES LTD.

1237 Caledonia Road,
Toronto, 19, Ontario.

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APPENDIX A - Standard Procedure

Borehole Logs

Site Plan and Profile

A. INTRODUCTION:

The site investigation described in this report was authorized on behalf of the Township of Warwick by Messrs. J. A. Monteith Associates, Consulting Engineers, by letter dated February 5th, 1962.

A new bridge is to be constructed over the Bear Creek to replace an existing structure, which is inadequate for present day traffic conditions. The scheme is referred to by the Client as No. 5/7 Sideroad Bridge.

No details of the proposed new structure were available at present, apart from the information that it is to be 60 ft. long.

Two test holes were required, 10 ft. off either end of the existing bridge.

The stream, crossed by the bridge, is approximately 50 ft. wide. The terrain to the north of the bridge is generally level, while the ground rises to the south.

B. GENERAL INFORMATION:

1. Two test holes were drilled at the site, in the positions indicated on the enclosed drawing. The positions were chosen and the test holes set out by the Consulting Engineer, who met our drilling crew at the site.
2. The ground elevations at the positions of the test holes are arbitrary, and were provided by the Consulting Engineer. We do not have information on the position of the temporary bench mark to which the elevations were referred.

The ground elevations are entered on the borehole logs and on the site plan and profile.

3. The depth of the test holes was specified as 40 ft. or to bedrock, whichever occurs first. Solid black shale bedrock was encountered at a depth of 10 ft. 0 in. in test hole 1, and 16 ft. 1 in. in test hole 2 (drilled at a higher elevation, on top of the existing embankment while test hole 1 was at the level of the ground adjacent to the bridge abutment). The test holes were carried down into the shale bedrock to prove its quality. Test hole 1 was diamond drilled from 9 ft. 8 in. to 20 ft. 2 in. and test hole 2 was diamond drilled from 16 ft. 1 in. to 21 ft. 2 in.

4. Test hole 1 was performed on February 13th, and test hole 2 on February 14th, 1962. Our standard drilling and sampling procedures were followed, as outlined in the enclosed Appendix A. The work was performed by our field crew using drill rig unit #3.

B. GENERAL INFORMATION: (Cont'd)

5. Details of the soil conditions found in the test holes are described on the enclosed borehole logs. A simplified soil profile, prepared from these logs, is shown on the enclosed drawing.

6. Laboratory testing of soil samples was limited to moisture content determinations and two unconfined compression tests on undisturbed samples of clay till. The results are entered on the borehole logs, together with standard penetration test results performed in the test holes.

C. SOIL CONDITIONS:

The outstanding feature of the subsoil disclosed by the test holes was the presence of the black shale bedrock surface at the elevation 84.0 in test hole 1, and 85.1 in test hole 2, i.e. at a depth of 9 ft. 8 in. and 16 ft. 1 in. respectively. The differences in the depths of overburden over the bedrock in the two borings are due to the different ground elevations; test hole 1 was drilled west of the north-west bridge abutment, near the stream bank, while test hole 2 was located on the west side of the embankment leading to the bridge from the south.

Details of the soil conditions found in the test holes are described on the borehole logs, while a simplified subsoil profile is plotted on the enclosed drawing.

C. SOIL CONDITIONS: (Cont'd)

The overburden in the two test holes was as follows:

In both borings a layer of glacial drift, in the form of broken, angular fragments of shale embedded in silt, rested immediately on the surface of solid bedrock. This layer was 3 ft. thick in test hole 1, and 1 ft. 3 in. thick in test hole 2. Although standard penetration test results (N-values) in this layer were 36 and 28 blows per foot, the relatively high resistance is caused by the presence of the shale fragments. The matrix of the material, consisting mainly of silt, with some sand and clay, was soft to firm. In test hole 2, the silt had a distinctly organic appearance and odour. Natural moisture content, determined on a sample from test hole 1, was 10.8%.

The layer of silt with broken shale was covered by a deposit of clayey silt or silty clay with pebbles, of till form. In test hole 1, this material began at a depth of 3 ft. 6 in. and was 3 ft. 2 in. thick; in test hole 2, it commenced at a depth of 5 ft. 8 in. and was 9 ft. 2 in. thick. In test hole 1, it had a more silty composition, while in test hole 2, it was more clayey and plastic. In both borings the till was of firm to stiff consistency, standard penetration tests giving values of 13 blows per foot in test hole 1, and 21 to 22 in test hole 2. The moisture contents varied, depending on the proportion of sand, silt or clay. In test hole 1 a moisture content of 14.3% was recorded while in test hole 2 the range was from 12.5% to 23.0%. The pebbles were up to one inch in diameter.

C. SOIL CONDITIONS: (Cont'd)

The silty clay or clayey silt fill was overlain in test hole 1 only by a layer of clayey and sandy silt with organic matter and grits, which was very soft (S. P. T. result 6 blows per foot).

In test hole 2, the organic clayey and sandy silt was not identified. Instead, the top 5 ft. 8 in. of subsoil consisted of a silty sand with gravel fill, of brown colour. The fill was quite loose, an S. P. T. at a depth of 4 ft. to 5 ft. 6 in. giving 7 blows per foot, while the moisture content was 5.8%. This was obviously an imported fill from which the existing embankments are constructed.

D. WATER CONDITIONS:

We do not know the exact elevation of water surface in the Bear Creek at the time of the site investigation; the creek was partly frozen and blocked with the ice, the surface of which was irregular. However, in test hole 1, ground water rising from the broken shale near elevation 85, reached an equilibrium level at the elevation 91.5, and in test hole 2, it rose to elevation 92.0. The ground water level in the stream, which probably controls the position of the water table at the site, thus appears to be between the elevations 91.5 and 92.0.

D. WATER CONDITIONS: (Cont'd)

In test hole 1, some ground water seepage occurred in the soft clayey and sandy silt layer between the depth of 2 ft. and 3 ft. 6 in. The seepage was slow and was cut off when test hole casing penetrated to a depth of 4 ft., and thus entered the firm grey clayey silt with pebbles (till). This material was sufficiently impermeable so that no noticeable seepage occurred during the drilling operations while the test hole passed through it. Ground water was again encountered when the boring penetrated into the soft, broken shale resting immediately on top of the solid black shale bedrock. Water gradually rose to the elevation 91.5 and the rate of percolation was sufficiently rapid not to allow bailing below the elevation 90.5.

In test hole 2, no free ground water was encountered until the hole reached the layer of soft shale and silt immediately on top of the shale bedrock. Ground water rose to the elevation 92, and the quantity was sufficiently large not to allow bailing below the elevation 91.

It may be concluded from the water observations that no free water-bearing layers are present, until the soft, broken shale or shale with silt deposit is reached. Considerable water seepage appeared to commence near the elevation 85.

D. WATER CONDITIONS: (Cont'd)

Ground water in the broken shale probably is in equilibrium with the water surface in the creek, as indicated by the rise of water level in both test holes from near the surface of shale bedrock to a level corresponding to the surface of the creek.

In addition, it must be concluded that the permeability of the water-bearing, broken shale is high, and that a considerable quantity of water would be encountered should excavations or preborings for cast-in-situ caissons be put down to the surface of the shale bedrock.

E. CONCLUSIONS AND RECOMMENDATIONS:

- i. The new bridge can be founded on:
 - a) The firm clayey till stratum, or
 - b) The black shale bedrock.

The relative merits of the two solutions will now be discussed.

E. CONCLUSIONS AND RECOMMENDATIONS: (Cont'd)

a) Spread Footing Foundations on Clayey Till

The layer of firm or stiff clay silt with pebbles or silty clay commenced near the elevation 90 in test hole 1 and 95.5 in test hole 2.

The depth at which the bridge footings will be constructed must be sufficient so that the foundations will be safe against any scouring by the stream. We do not have information on the depth of the stream or the maximum likely depth of scour, but the footings should be set below the zone of scouring.

Provided that the footings will be safe from scouring, they could be set at the elevation 88 to 89, at which the maximum allowable net bearing capacity was estimated as 3 tons/sq. ft. To this value an additional component equal to the pressure of the soil overburden above the footings may be added. The settlement under the maximum allowable pressure will not be excessive and will take place within a short interval after construction.

The stability of bottom of excavation carried down to the elevation 88 was considered, taking into account the presence of uplift water pressure, presumably controlled by the head of water in the stream, in the broken shale and silt immediately on top of the shale bedrock. For a trench of dimension 35 x 15 ft., which may be a likely size, there appeared to be no danger of a blowout.

E. CONCLUSIONS AND RECOMMENDATIONS:

a) Spread Footing Foundations on Clayey Till (Cont'd)

Excavations carried down to an elevation not lower than 87 should be practically free of water, assuming that any seepage from the stream will be cut out by sheeting or pumped out from the trench. Seepage occurred only in test hole 1 in the soft clay and sandy silt between the depth of 2 ft. and 3 ft. 6 in. No seepage was encountered in test hole 2.

Excavations carried to below the elevation 87 are likely to encounter the ground water present in the silt and broken shale on the top of the bedrock. The silt may become unstable and the quantity of water may be large. For this reason it may be preferable to place the foundations in the clay till rather than on the silt with broken shale fragments or on the shale bedrock itself.

Should the recommended footing elevation 88 to 89 be too high with respect to a danger from scouring, perhaps consideration could be given to increasing the span of the bridge and placing the abutments at a sufficient distance away from the stream, so that they would not be affected by scouring. Alternatively, sheeting could be driven on the stream side of the footings to a depth sufficient to protect them against scouring.

E. CONCLUSIONS AND RECOMMENDATIONS:

b) Spread Footing, Caisson or Pile Foundations on Surface of Shale Bedrock (Cont'd)

The major disadvantage of placing the foundation directly on the bedrock is the presence of water under the head in Bear Creek, in the broken shale and silt resting immediately on top of the bedrock. Water observations made in the test holes gave an indication that the quantity of water that would have to be dealt with in excavations carried to surface of the shale would be considerable. In order to keep the excavation dry, it may be necessary to pump from holes drilled outside the perimeter of the excavation, and penetrating to the surface of the solid shale. This method would probably be more reliable than trying to cut off seepage by sheeting driven to the top of the shale bedrock, since the sheeting would encounter considerable resistance or even virtual refusal in the permeable, broken shale; water seepage would continue under the bottom of the sheeting, so that the inflow would not be sealed off.

Consideration can be given to founding the bridge on short-bored piles or caissons. The piles could rest directly on the surface of the bedrock, in which case the maximum bearing capacity would be equal to the structural strength of the piles themselves. However, a similar water problem as in open excavations would arise, as soon as the pre-boring reaches the top of the water-carrying broken shale with silt. Provision of lining in which the piles could be cast may not solve the problem, as refusal could be reached in the broken shale, without sealing

E. CONCLUSIONS AND RECOMMENDATIONS:

b) Spread Footing, Caisson or Pile Foundations on Surface of Shale Bedrock (Cont'd)

off water seepage entirely. Water would continue to penetrate into the casing below the seating in the shale.

Because of the presence of water it may be easier to place the toes of bored cast-in-situ piles near the top of the layer of silt with broken shale, approximately near the elevation 86 to 87, in which case the boring would probably be dry. The maximum allowable bearing capacity of such piles could be estimated as 10 to 12 tons/sq. ft. Precompressed concrete pedestal type piles could be considered.

As an alternative to prebored piles, driven piles could be employed and steel H-piles appear to be well suited. Such piles could be driven to refusal at the surface of the black shale bedrock, provided that it is considered that sufficient anchoring against lateral pressures is available, particularly for tapered piles carrying any lateral thrusts.

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° 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 tubes are filled or prepared by cutting the soil in
succession at the top in the soil and on completion, subsequent water
level readings are taken for the duration of the field work. After necessary
readings are completed, with findings which conditions are encountered.
Moisture content samples are collected at convenient intervals in order to
the soil characteristics and the interpretation of water level readings.

Flowchart logs are prepared giving details of the flow chart
direction and condition as recorded in the field. These logs form the
basis of the soil profile, which indicates the general stratigraphic sequence
to exist between the layers as represented by the borehole logs.

The boreholes are normally set out by the client engineers, who
also records the ground elevations referred to a benchmark which may be
a fixed reference point. If the client has been responsible for setting out
the boreholes and recording their ground elevations this is stated in the
preliminary 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.P. if
where applicable.

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

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name 6/7 Sideroad Bridge

Job No. 6226

Borehole No. _____

Client Township of Warwick
c/o J. A. Monteith Assoc.

Casing 4" & BX

February 13th, 1962

Elevation Client's

Compiled By R. K.

Checked by S. B.

SAMPLE CONDITION



INDISTURBED



FAIR



DISTURBED



1051

SAMPLE TYPE

A.S. AUGER SAMPLE

C.S. CASING SAMPLE

5.5. 2" STANDARD SPLIT TUBE SAMPLES

5.1. SOLIT BARREL WITH LINERS

S.T. THIN-WALLED SHELLY TUBE SAMPLE

W. S. WASTE SAMPLE

W.S. WASTE SAMPLE
R.C. ROCK CORE

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST

C. SOIL SHEAR STRENGTH (LB/50.F.F.)

DATE	SOIL SHEAR STRENGTH
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W.1 GROUND WATER TABLE IN SOIL

W.T.P.F. WETTER THAN PLASTIC LIM

0.181. BELOW ELASTIC LIMIT

[illegible]





e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name	67 Sideroad Bridge	Job No.	6226	Borehole No.	2
Client	Township of Warwick	Casing	4" & BX	Boring Date	February 14th, 1962
Elevation	Client's	Completed By	PK	Checked By	S. B.

SAMPLE CONDITION

 UNDISTURBED
 FAIR
 DISTURBED
 LOST

SAMPLE TYPE

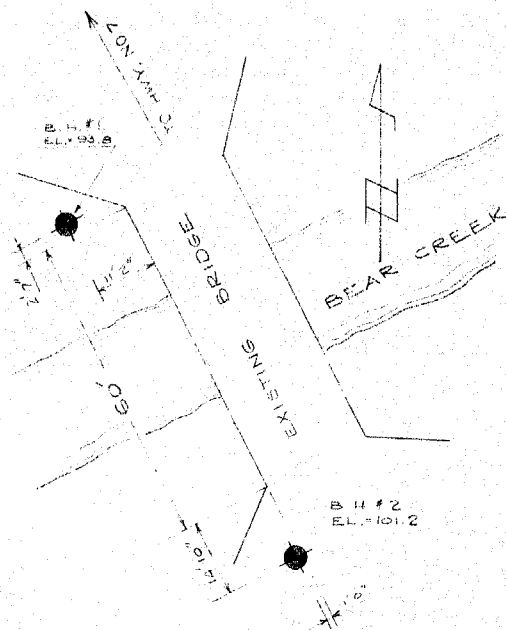
A.S. AUGER SAMPLE
 C.S. CASING SAMPLE
 S.S. 2" STANDARD SPLIT TUBE SAMPLE
 S.L. SPLIT BARREL WITH LINERS
 S.T. THIN-WALLED SHELBY TUBE SAMPLE
 W.S. WASH SAMPLE
 R.C. ROCK CORE

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST
 C. SOIL SHEAR STRENGTH LBS/SQ.FT.
 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

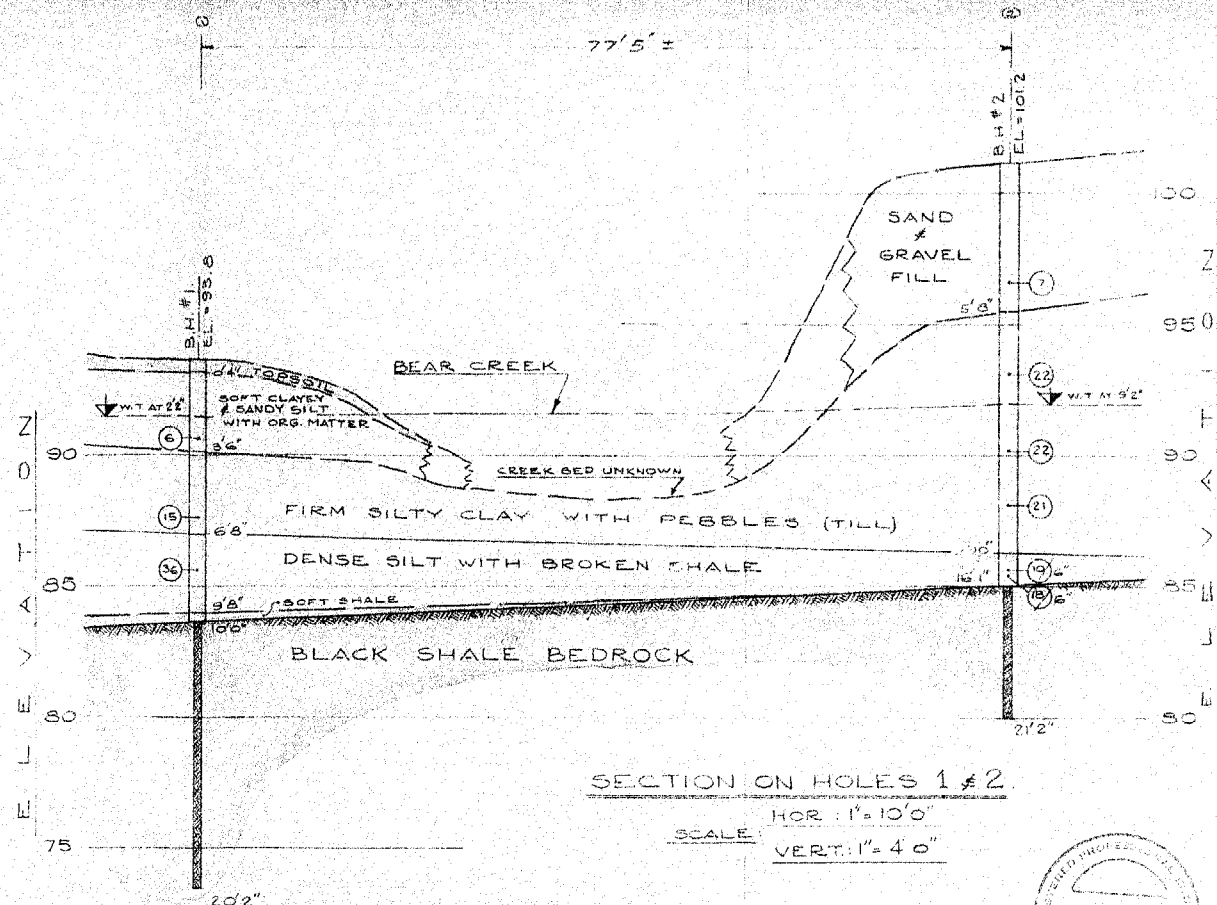
SOIL DESCRIPTION	COLOUR	Density or consistency	Depth Elevation	Legend	Sample No. and Contribution	Sample Type	No. of Blows per Ft.	Unit Weight	WATER LEVELS & REMARKS
			Elev. 101.2						
			0'0"						
Silty sand with gravel	Dark brown		3'8"		1	C.S.			
Fine to medium gravel with some silty sand (fill)	Brown	Loose	5'8"		3	S.S.	7	5.8	Softer at 3'8"
Fissured silty clay with pebbles (till)	Brown changing to grey	Stiff	10'0"		4	S.S.	22	17.3	
Silty clay with pebbles	Grey	Firm to Stiff	11'0"		5	S.S.	22	23.0	W.T. 9'2"
Ditto	Grey	Ditto	11'9"		6	S.S.	21	12.5	ulc shear strength: 2160 p.s.f. Bulk density: 133 p.c.f. More sandy and harder 11' - 11'9". Stones up to 1"
Broken shale in organic silt	Dark grey		14'10"		7	S.S.	10/6"		ulc shear strength: 4320 p.s.f.
Shale	Black with Grey bands	Solid	16'1"		8	R.C.	18/6"		Virtual refusal at 16'1"
			20'0"				2/1"		Diamond drilled 16'1" to 21'2"
			21'2"						Core recovery 5'1" (100%)

Test Hole Terminated at 21'2"



SKETCH SHOWING
LOCATION OF
TESTHOLES

(NOT TO SCALE)



SECTION ON HOLES 1 & 2

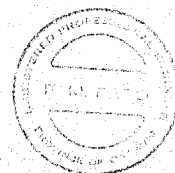
SCALE
HOR. 1" = 10' 0"
VERT. 1" = 4' 0"

LEGEND

- - BOREHOLE
- ⊙ - BLOWS/FOOT (SPT)

NOTES

- a) SEE BOREHOLE LOGS FOR COMPLETE SOIL DETAILS.
- b) POSITION OF T.B.M. AND CREEK BED UNKNOWN



JOB No 6226
empeto associates ltd.
FEBRUARY, 1962