

62-F-280 m

BEAR CREEK

BRIDGE LOTS 9/10

WARWICK

TWP.

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

March 13, 1962.

REVIEW OF SOILS REPORT BY  
E. M. PETO & ASSOC., LTD. and  
DWG. BY J. A. MONTEITH ASSOC., LTD.  
(Bridge Office Ref. BA 1365)

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

Re: Twp. of Warwick, Lots 9/10  
County of Lambton, Can. IS.R.F.  
Bear Creek Bridge. District #1.

We have reviewed the above-mentioned report submitted by E. M. Peto & Associates, Ltd., and also, the bridge foundations as presented on Dwg. No. B-1893-1, prepared by J. A. Monteith Associates, Ltd., Consulting Engineers. Below, we are submitting our comments for your consideration:-

The designed footings are at elevation 77.0 and 78.5, respectively, where 2.5 T/sq.ft. net allowable pressure is recommended, while only 1.5 and 3.0 ft. deeper, respectively - (elevation 75.5) bedrock was found and for which an allowable pressure of 2.5 T/sq.ft. is given.

We would suggest that an analysis be requested in which the costs of different foundation types as well as the costs of foundations at different elevations are compared. Only on the basis of such a comparison can the choice between technically correct solutions be made.

It also seems to us that the key at the middle of the footing could be dispensed with, and we would therefore suggest that this question be discussed with the bridge designer.

AGS/MdeF

cc: Foundations Office  
Gen. Files.

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

BA 1365

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,  
Toronto 19, Ontario.

Our Job Number 6227

Russell 9 - 1126.

February 26th, 1962.

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

Gentlemen,

Re: Site Investigation,  
9/10 Sideroad Bridge  
Township of Warwick.

We have pleasure in submitting four (4) copies of  
our Report Number 6227 on the above site investigation.

The black shale bedrock, with a very high bearing  
capacity, was found to commence between the elevations 75.4 and  
75.8. Bridge foundations, in the form of footings, caissons or  
piles, can be constructed on the shale, although the placement of  
footings on a compact sand or a stiff clayey till, which overlies  
the bedrock, has been considered in the report.

PAGE TWO

Groundwater inflow into excavation between elevations 91 and 80 can be expected, so that the use of piles may be more economical.

We consider the report to be comprehensive, but would be most pleased to answer any queries, which you may wish to raise.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

A handwritten signature in dark ink, appearing to read 'E. M. Peto', written in a cursive style.

E. M. Peto, P. Eng.

RK/ap

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

SOILS REPORT  
for  
9/10 SIDEROAD BRIDGE  
TOWNSHIP OF WARWICK, ONTARIO

E. M. PETO ASSOCIATES LTD.,

1287 Caledonia Road,  
Toronto 19, Ontario.

## CONTENTS OF REPORT

Page No.

Covering Letter

Report

A. INTRODUCTION	1
B. GENERAL INFORMATION	2
C. SOIL CONDITIONS	3
D. WATER CONDITIONS	6
E. CONCLUSIONS AND RECOMMENDATIONS	8

APPENDIX A - Standard Procedure

Borehole Logs

Site Plan

#### 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 near Warwick, Ontario, to replace an existing structure, which is inadequate for present day traffic conditions. The new bridge is referred to by the Client as 9/10 Sideroad Bridge. A site investigation was required to determine the subsoil conditions for the design of the foundations. Two test holes were required 10 ft. off either end of the existing bridge. No details of the new structure were available at the moment, apart from the information that it will be 60 ft. long.

The bridge is to span the Bear Creek, which is approximately 50 ft. wide. The terrain in the locality of the bridge is generally flat, and the existing bridge is approached on either side by embankments roughly 5 ft. high.

## B. GENERAL INFORMATION:

1. Two test holes were drilled, in the positions chosen and set out by the Consulting Engineer, who met our drilling crew at the site. Both borings were located on the existing embankments; No. 1 was 10 ft. east of the north-east corner of the existing bridge, and borehole 2 was 10 ft. 10 in. west of the south-west corner of the bridge. The positions of the test holes are sketched on the enclosed drawing.

2. The elevations at the positions of the test holes were provided by the Consultants, and are referred to an arbitrary bench mark, the position of which is known to the Client. The elevations are included on the borehole logs and on the enclosed site plan and profile.

3. The depth of the test holes was specified as either 40 ft. or bedrock, whichever occurs first. It was required to prove the bedrock by diamond drilling.

In test hole 1, the black shale bedrock was reached at a depth of 23 ft. 1 in. (elevation 75.7) and was proved by 10 ft. 2 in. of diamond drilling. In test hole 2, the bedrock was reached at a depth of 24 ft. 1 in. (elevation 75.4) and was diamond drilled for 5 ft. 2 in.

4. The field work was carried out by our crew using drill rig unit No. 3. Test hole 1 was performed on February 15th, and 16th, while test hole 2 was carried out on February 16th and 17th, 1962. Our standard drilling and sampling procedures were followed, as outlined in the enclosed Appendix A.



B. GENERAL INFORMATION: ( Cont'd)

5. Details of the soil conditions found in the test holes are entered on the enclosed borehole logs, which contain also results of standard penetration tests and in situ moisture content determinations. A simplified subsoil profile was prepared and is plotted on the enclosed drawing.

6. No laboratory soils testing, apart from the moisture content determinations, was considered necessary.

C. SOIL CONDITIONS:

The two test holes drilled at the site have established that the black shale bedrock commences between the elevation 75.8 and 75.4, which corresponds to a depth of 23 ft. 11 in. in test hole 1 and 24 ft. 1 in. in test hole 2. Both holes were located on the existing embankments. The shale was found to be solid, with the exception of a 6 inch thick layer of sand and decomposed shale at the elevation 72, in test hole 1. However, the bedrock can undoubtedly be considered as an excellent foundation stratum.

The overburden on top of the bedrock was very variable and the enclosed borehole logs should be consulted for detailed description of individual soil samples. Generally, the material was soft or loose down to a depth of about 15 ft.

C. SOIL CONDITIONS: (Cont'd)

The subsoil can be tentatively subdivided into the following layers, possessing similar characteristics, in the order of occurrence.

- a) Topsoil and silty sand and gravel fill. (embankment materials),
- b) Organic clayey silt,
- c) Sand and gravel layers,
- d) Clay and sandy silt with broken shale, (till),
- e) Black shale bedrock.

The existing embankments were found to consist of silty sand and gravel fill, or of sandy and clayey silt with pebbles, as described on the borehole logs. The materials were quite variable, and the consistency of the fill was soft or loose, standard penetration tests (S. P. T.) giving results of 4 and 7 blows per foot at a depth of 5 ft. in the two test holes. Moisture contents were 23.3% and 25.8% at this depth. The embankment fill appeared to extend to a depth of 6 ft. 3 in. in test hole 1 and to 6 ft. 10 in. in test hole 2.

The embankments were found to rest on top of a clayey silt or silty clay with a high but variable organic content. A large fragment of decomposed wood was encountered at a depth of 10 ft. in test hole 2. The clayey silt or silty clay was very soft, as indicated by the results of six standard penetration tests, which ranged from 2 to 7 blows per foot. Moisture contents were variable, depending on the organic content, and ranged from 24.8% to 92.5%.

C. SOIL CONDITIONS: (Cont'd)

The soft organic layer was followed by a predominantly sandy deposit, which commenced at a depth of 8 ft. 4 in. in test hole 1, and 11 ft. 11 in. in test hole 2. It extended to a depth of 10 ft. 9 in. in test hole 1, and 18 ft. in test hole 2, at which depth the sandy layers were followed by a clayey till. The sandy layers were very loose to a depth of about 15 ft. (S. P. T. results in the range of 5 to 7 blows/ft.) with the exception of a compact silt layer at a depth of 15 ft. 2 in. to 15 ft. 9 in. in borehole 1. The sand was mainly fine with some medium, and contained variable proportions of silt and gravel. In test hole 2, some samples of the sand contained organic silt or fragments of decomposed wood. Moisture contents in the sandy layer ranged from 15.0% to 34.0%, again depending on the organic content.

The silty clay with pebbles or clayey silt, of till form, which was found underneath the sandy stratum commenced approximately at the elevation 80 to 81.5. It contained a considerable quantity of angular black shale fragments and was obviously deposited by glacial action. Standard penetration test results indicated that the stratum was generally firm or dense, S. P. T. results ranging from 14 to 40 blows/ft. However, the clayey matrix of the material had a soft to firm consistency.

C. SOIL CONDITIONS: (Cont'd)

Moisture contents were in the range of 13.1% to 17.8%. The proportion of shale fragments was found to increase with depth, and thin layers of broken shale were encountered, as shown on the borehole logs.

The solid black shale bedrock, which commenced at the elevation 75.8 in test hole 1, and 75.4 in test hole 2, was overlain by a few inches of soft broken shale.

D. WATER CONDITIONS:

In test hole 1, ground water was encountered near the top of the sand and gravel stratum, and the equilibrium depth was 8 ft. 7 in. (elevation 91.0).

In test hole 2, an organic clayey silt, of relatively low permeability, extended down to a depth of 11 ft. 11 in., where it was followed by the sand. Free ground water was reached only when the test hole entered the sandy stratum, but the level rose to a depth of 8 ft. 6 in., where it remained steady. This depth also corresponds to the elevation 91.0 which thus can be taken as the surface of the ground water table at the time of the site investigation.

We do not have the elevation of surface of the Bear Creek, but since the ground water table is likely to be controlled by the creek level, it would appear that the free water surface is near the elevation 91.

D. WATER CONDITIONS: (Cont'd)

In test hole 2, the rate of percolation of water into the test hole, as measured by the rate of rise of water level after bailing, was moderate. In test hole 1, the sand was backing up in the casing driven to a depth of 15 ft. It can be concluded that a considerable quantity of water would be encountered in any open excavation taken down to below the depth of the ground water table, and that any such excavation would have to be protected by sheeting, though the use of well-points could be considered.

The layer of clay and sandy silt or silty clay with pebbles, resting immediately above the black shale bedrock, was of low permeability, as indicated by the fact that when the borehole casing penetrated into this material, the rate of inflow of water into the test hole was very slow. In test hole 1, with the casing at 23 ft. 11 in. the water level rose by 8 inches in a period of 40 minutes, after bailing to a depth of 23 ft. 8 in. After further bailing to a depth of 34 ft., the ground water level rose to 32 ft. 10 in. in 9 minutes and remained steady at this level during the next 11 minutes.

In test hole 2, with the casing at a depth of 24 ft., water level rose after bailing from a depth of 25 ft. 5 in. to 24 ft. 6 in. in a period of 50 minutes.

## E. CONCLUSIONS AND RECOMMENDATIONS:

1. The test holes have disclosed that a stratum of organic clayey silt of soft consistency, followed by loose sand and gravel, extends to a depth of about 15 ft., or to the elevation 84. Below this depth, the sand and gravel becomes compact to dense, and is followed by a firm to stiff clayey till commencing near the elevation 80 to 81.

The clayey till rests on top of the black shale bedrock which commenced at the elevation 75.4 and 75.8 in test holes 2 and 1 respectively.

2. Ground water table began near the elevation 91, and probably is controlled by the water level in the creek. The ground water is carried by the variable stratum of sand and gravel, and the quantity of seepage into an open excavation would be excessive.

If an open excavation is required, driving sheeting around the perimeter of the excavation and into the clayey and sandy silt layer resting over the bedrock will be partially effective in cutting off the water seepage. The remainder of water would probably be removed during the progress of work by pumping from the bottom of the excavation. In order to limit the quantity of water seeping into the excavation, the sheeting should be driven to as near the surface of the shale bedrock as possible.

E. CONCLUSIONS AND RECOMMENDATIONS: (Cont'd)

3. The following types of foundations could be considered for the bridge:

a) Spread footings, placed on compact sand and gravel, near the elevation 83, or in the clayey till, at the elevation 79 or below.

b) Footings placed on the black shale bedrock.

c) Pile foundations with piles driven to the top of the shale bedrock.

Both solutions (a) and (b) would require an open excavation, which would have to be protected from ground water seepage from the creek through the sand and gravel stratum.

The allowable bearing capacity at the elevation 83 would be approximately 1.5 tons/sq. ft., to which figure a component equal to the pressure of the soil overburden above the footings could be added.

The bearing capacity in the clayey till, at the elevation 79 or less could be taken as 2.5 tons/sq. ft., plus the overburden component.

The bearing capacity<sup>of</sup> footings placed directly on the black shale bedrock would be as high as 25 tons/sq. ft.

E. CONCLUSIONS AND RECOMMENDATIONS:

3. (Cont'd)

The use of bored piles or caissons could be considered and these could be founded in the dense clayey till at the elevation 79 or on the surface of the black shale bedrock. If the piles rest on the shale, the allowable bearing capacity would be determined by the structural strength of the piles themselves.

If driven piles are chosen, steel H-piles appear to be well suited. Such piles should be driven to refusal on the surface of the shale bedrock near the elevation 75.5, provided that it is considered that sufficient resistance to lateral pressures is available, particularly for tapered piles carrying any horizontal thrust components.

4. Because of the presence of the organic clayey layer between the elevations 91 and 93 in test hole 1 and 87 and 93 in test hole 2, some settlement of the embankments leading to the bridge must be expected. It would be advisable to construct the embankments before completion of the bridge, thus allowing settlements to take place before the road surface is placed.



E. CONCLUSIONS AND RECOMMENDATIONS: (Cont'd)

4. (Cont'd)

Also, because of the likely settlement of the fill, a step can form between the bridge deck and the backfill behind the abutment, particularly if the bridge, founded on the shale bedrock, would not be subject to any settlements. The fill behind the abutments should therefore be well compacted. Also, it would be advisable to remove the organic layers immediately behind the abutments, and replace them by well compacted granular materials. The thickness of the excavated material could be tapered off to zero some distance away from the bridge.

5. The fill from which the present embankments are constructed is variable and of moderate quantity, but can be retained in the reconstructed embankments. However, because of its variability and a silt content, it is recommended that a layer of clean, granular material at least 12 inches thick and well compacted, should be placed immediately below the pavement as a precaution against damage due to frost heave.

E. M. PETO ASSOCIATES LTD.,

*C. F. Freeman*

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

RK:sh

Job No. 6227

February, 1962.

## 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 out 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 edge.

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 Auer 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 drilled 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 descriptions 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, and also records the ground elevations referred to a known or 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 as far as possible in the report.

A plan is drawn up from drawings supplied by the Client or its representatives, showing the locations of the boreholes and the T.P.s 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 results. Details, ground water readings and other soil characteristics which can be best illustrated in graphical form.

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

# e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

## BOREHOLE LOG

Job Name 9/10 Sideroad Bridge

Job No. 6227

Borehole No. 1

Client Township of Warwick

Casing 4" and BX





Boring Date Feb. 15th, and 16th, 1962

Elevation Client's

Compiled By R. K.

Checked By A. P.

### SAMPLE CONDITION

-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

### SAMPLE TYPE

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

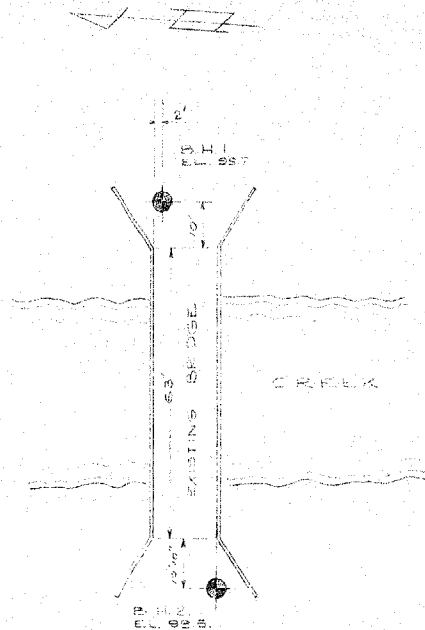
### ABBREVIATIONS

- 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.P.L. WETTER THAN PLASTIC LIMIT
- D.T.P.L. DRIER THAN PLASTIC LIMIT

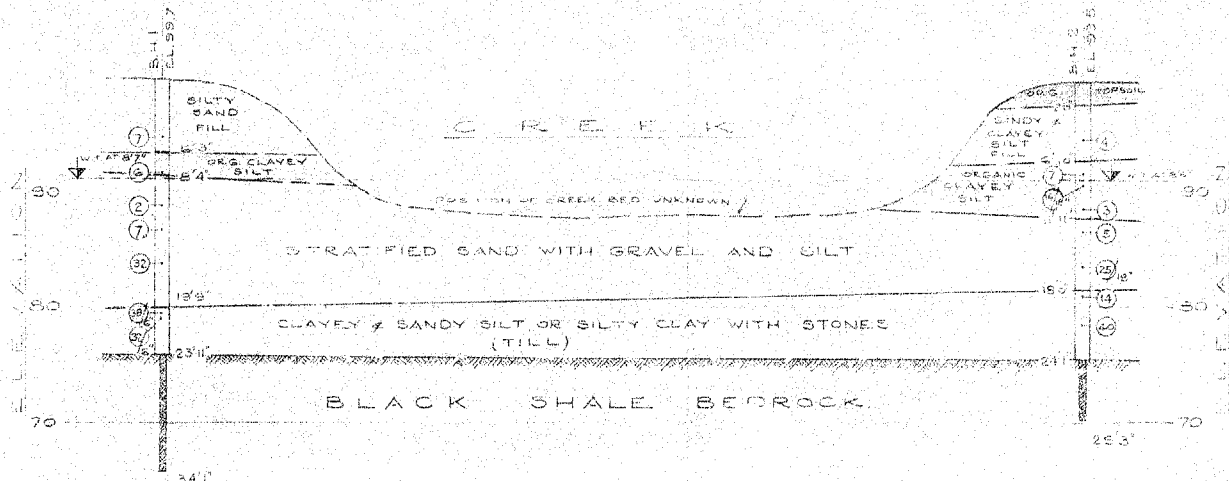
SOIL DESCRIPTION	GROUP	Details as Observed	Depth (Feet)	Sample No. and Location	Sample Type	Blow Count (2' S.T.)	WATER LEVEL & REMARKS
			99.7 0'-0"				
Silty sand & gravel fill	Dk. brown			1	CS		Frost penetration 3'-3"
Silty sand fill	Dk. brown		2'-0"	2	CS		
Silty sand fill	Dk. brown			3	CS		
Silty sand fill	Rusty brown	Loose	5'-0"	4	SS	7	29.3
Organic clayey silt	Grey and brown mottled	V. soft	6'-3"				
			8'-0"	5	SS	6	37.4
Fine silty sand with organic silt & layers of decayed vegetation	Dk. grey	V. soft	8'-4"				W.T. at approx. 8'-7"
			10'-0"	6			
			10'-0.5"	7	2" S.L. Pushed		24.8
Fine to coarse sand and fine gravel	Grey	V. loose	11'-4"	8	SS	2	
			12'-0"	9	SS	7	30.2
							29.8
Fissured, sandy silt	Dk. grey	Compact to dense	15'-0"				
			15'-9"	10	SS	32	15.0
Fine to medium silty sand			17'-0"				
Fine gravel with silty sand			18'-0"				
			19'-9"	11	WS		
Clayey & sandy silt layer of angular shale fragments	Dk. grey	Dense	21'-1"	12	SS	28/6"	17.8
Clayey and sandy silt soft shale			21'-6"			32/6"	18.1"
Shale bedrock	Black with grey bands	Elev. 75.8 Solid	23'-5"				
			23'-11"				
Sand layer		Softer	27'-6"	13	RC		Diamond drilled 23'-11" to 29'
			28'-0"				Core recovery 4'-4"
			30'-0"				Bituminous shale layer between 30' and 32' 5"
Shale bedrock		Solid	32'-5"	14	RC		Diamond Drilled 29' to 34'-1"
			34'-1"				Core recovery 5'-1" (100%)

TEST HOLE TERMINATED AT 34 FT. 1 IN.





SKETCH SHOWING LOCATIONS  
OF TESTHOLES  
(NOT TO SCALE)



SECTION ON HOLES 1 & 2

SCALE: 1"=50'  
(NATURAL)

# LEGEND

- - BOREHOLE
- ⑦ - BLOWS/FOOT (SPT)

# NOTES

- A) SPT BOREHOLE LOGS FOR COMPLETE SOIL DETAILS
- B) POSITION OF T.B.M. AND CREEK BED UNKNOWN.



JOB No. 6227  
empeto associates ltd  
FEB. 1962 W.S.