

# 63 - F - 237 M

HARDY CREEK BRIDGE

LOT 24, CON. X / X1

BROOKE TWP.

## MEMORANDUM

To: Mr. A. Stermac,  
Principal Foundation Engineer,  
Room 107, Lab. Bldg.

FROM: G. C. E. Burkhardt

DATE: November 22, 1963.

OUR FILE REF.

IN REPLY TO


SUBJECT: Township of Brooke  
Hardy Creek Bridge  
Lot 24, Con. X/XI  
County of Lambton  
Structure Site No. 15-160  
Our File No. BA 1712

Attached please find one copy of the  
Foundation Report, by E. M. Peto Associates Ltd.,  
for above mentioned structure.

The designer proposed a single 65 foot  
simply supported steel beam superstructure,  
supported on concrete abutments with spread foot-  
ings, founded at El. 683.0.

We would appreciate it very much, if we  
could have your comments at your earliest con-  
venience.

GCEB/es

  
G. C. E. Burkhardt,  
for K. L. Kleinsteinber,  
Mun. Bridge Liaison Engineer.

SEDROCK EL. 673.0

67.0  
No comments by phone to G.C.E.B.  
Nov. 25, 1963.

E. M. PETO ASSOCIATES LTD.

Job No. 6394

1287 Caledonia Road,  
Toronto 19, Ontario.  
RUssell 9-1126-7

May 29th, 1963.

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

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

Gentlemen:

Re: Subsoil Investigation,  
Hardy Creek Bridge,  
13 miles east of Petrolia, Ontario.

We have pleasure in submitting five copies of our Report  
No. 6394 on the above investigation.

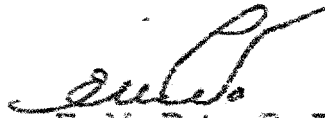
Several possibilities of foundation design are considered,  
of which the most practicable appear to be footing foundations, designed  
to a maximum net pressure of 3.0 ton/sq. ft and placed on a silt till  
commencing near elevation 77.0; or steel H-piles, driven to refusal on  
the shale bedrock near elevation 67.3.

The first of these two solutions suffers from the drawback that the silt till contains ground water seams, so that measures for lowering the deep-seated water table during excavation and construction would be necessary. This is the chief reason why we are inclined to regard steel H-piles, driven to the shale bedrock, as the preferable solution.

We consider the report to be comprehensive, but would be very glad to provide additional assistance, should you wish to discuss further any points connected with this work.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

RK:rb

THE TOWNSHIP OF BROOKE,

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

SUBSOIL INVESTIGATION

HARDY CREEK BRIDGE

13 MILES EAST OF PETROLIA, ONTARIO.

E. M. PETO ASSOCIATES LTD.

1287 Caledonia Road,  
Toronto 19, Ontario.

## TABLE OF CONTENTS

	<u>Page Number</u>
A. INTRODUCTION	1
B. GENERAL INFORMATION	2
C. SOIL CONDITIONS	3
D. WATER CONDITIONS	7
E. CONCLUSIONS and RECOMMENDATIONS	7

BOREHOLE LOGS

SITE PLAN and PROFILES

## A. INTRODUCTION:

The work described in this report was authorized verbally by Mr. G. Ingram, P. Eng. of J. A. Montalith Associates Ltd., Consulting Engineers.

A new bridge is to be provided on a gravel road, having an east-west orientation and separating Concessions XI and X in the Township of Brooke, Ontario. The gravel road continues west directly to Petrolia, and the site is located 2.0 miles east of its intersection with Highway 79.

Hardy Creek is approximately 25 ft wide at the site, where its average orientation of flow is from the north-east to the south west. A new channel is to be provided to the east of the present bed, the proposed centre line of the diversion being located about 30 ft east of the centre of the existing bridge.

Two test holes were required, at the diagonally opposite corners of the proposed new bridge.

B. GENERAL INFORMATION:

1. Locations of the two test holes are indicated on the drawing, which is based on a site sketch supplied by the Consulting Engineers. Test hole 1 was located 35 ft east of the eastern end of the existing bridge deck and on the north side of the gravel road; test hole 2 was located 25 ft east of the existing bridge and on the south of the gravel road.

2. Ground elevations were supplied by the Consulting Engineers, and are referred to an arbitrary datum, the position and elevation of which is not known to us. The elevations are entered on the borehole logs and drawing.

3. The field work was performed by our drilling rig unit No. 7, between May 23rd and 23th, 1953. Our standard sampling and drilling procedures were followed.

4. Test hole 1 encountered a black shale bedrock at a depth of 28.6 ft; the shale was proved by 5 ft of diamond drilling, and 100% core recovery was obtained.

The test hole 2 encountered refusal at almost identical elevation as the level of shale surface in test hole 1; consequently, it was assumed that the refusal occurred on the shale surface, which is known to be almost horizontal in the area, and diamond drilling was not performed in the second test hole.

5. No laboratory testing of soil samples, apart from water content determinations, was considered necessary.



### C. SOIL CONDITIONS:

Details of the subsoil conditions encountered in the test holes are described on the enclosed borehole logs, which include also in situ water contents and results of standard penetration tests. A simplified subsoil profile, in the form of a section through the two test holes, is presented on the drawing.

The following soil types were established in the test holes; the inferred contacts between the various strata are indicated on the subsoil profile.

- a) Sand and gravel fill
- b) Clayey loam
- c) Layered clay and silt
- d) Dense silty till
- e) Black shale bedrock.

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

#### a) Sand and gravel fill

A layer of sand and gravel fill, of brown colour and loose to compact density, was encountered immediately below the existing grade, and forms the eastern approach embankment to the existing bridge. This material can probably be retained in the reconstructed embankments.

C. SOIL CONDITIONS: (Cont'd)

b) Clay loam

DEFECTS IN REPRODUCTION DUE TO  
CONDITION OF ORIGINAL DOCUMENT

A silty clay with some sand and a few pebbles, possessing an appearance characteristic of surficial loam, was encountered immediately below the sand and gravel fill in both test holes, and extended to an approximate depth of 9 ft below the existing grade. This material was of brown-grey colour and firm to stiff consistency in the upper layers, becoming softer lower down. In test hole 1, an organic silt layer was observed between the depths of 8 and 9 ft, and some water seepage originated at that depth.

The loamy clay is unsuitable for the support of foundations and also, on account of its high water and silt contents, must be regarded as an inferior backfilling material, classified as highly susceptible to frost heave.

c) Layered clay and silt

A deposit of layered clay and silt extended below the loamy clay and down to the approximate elevation 78.0 in test hole 1 and 76.6 in test hole 2. The upper portions of this deposit in both test holes consisted of a plastic, brittle clay of firm to stiff consistency, but with a water content on the wet side of the plastic limit. The material became softer with depth, and in test hole 1 remained mainly a clay, while in test hole 2 below a depth of 12 ft it consisted of interlayered bands of clay and clayey silt.

Shear strength and compressibility of this deposit were not investigated in the laboratory, since the bearing capacity is likely to be inadequate for the proposed structure, which should be founded at a greater

C. SOIL CONDITIONS:

c) Layered clay and silt (Cont'd)

depth instead.

d) Dense silt till

This deposit, commencing at the approximate elevation 78.0 in test hole 1 and 76.6 in test hole 2, can be considered as the uppermost stratum suitable for the support of foundations, if the allowable net pressure is to exceed 3.0 ton/sq. ft.

This stratum, resting directly on the shale bedrock, has a typical form of a glacial till of residual character, consisting mainly of fragments of the underlying black shale bedrock churned up by the glacier and re-deposited locally, mixed with a minor content of imported material. The silt-size particles predominated, but fine to coarse gravel-size, <sup>of</sup> angular shape, shale fragments were very numerous, so that the material was very gravelly. The colour was dark gray to black.

Standard penetration test results in this deposit ranged from 31 to 42 blows per foot, and the water contents were in the 7.3% to 8.8% range, except for a higher value in a seepage seam. The clay or clayey silt matrix was softer than the plastic limit, so that the material can be regarded as compressible. For this reason, it is considered that the foundation pressure for footings resting on this deposit should be limited to 3.0 ton/sq. ft.

C. SOIL CONDITIONS:

d) Dense silt till (Cont'd)

Water seepage seams were encountered in the till, the head of water approximately corresponding to the water level in the Hardy Creek.

e) Shale Bedrock

Shale bedrock was encountered in test hole 1 at a depth of 28.6 ft (elevation 67.4). It was proved by 5 ft of diamond drilling and 100% core recovery was obtained. The core consisted of solid, laminated black shale, which is typical of the Ordovician shale, forming the bedrock over most of Lambton County.

In test hole 2, refusal was reached at elevation 67.2, which is very similar as the level of surface of bedrock in test hole 1; it was assumed therefore that the refusal was obtained on the surface of solid shale, which can be regarded as practically horizontal throughout the site.

The shale is capable of supporting very high foundation pressures.

#### D. WATER CONDITIONS:

Details of ground water observations are entered on the borehole logs. In test hole 1, water seepage seams were encountered near the 8 ft depth at the bottom of the alluvial surficial layers and again in the silty till at the depths 20 and 23 ft. The water rose in the test holes to a depth of 3.7 to 4.4 ft below the existing grade, which approximately corresponds to the water level in Hardy Creek.

Excavations penetrating to the dense silt till below elevation 73, or to the shale bedrock, will encounter seepage seams at the above depths and the quantity of water seepage, particularly below the 13 ft depth, can be considerable. Special precautions may be necessary to combat this feature of the subsoil conditions during excavation and construction operations.

#### E. CONCLUSIONS AND RECOMMENDATIONS:

##### 1. General Considerations.

Theoretically, foundations for the proposed bridge could have the following alternative forms:

Spread footings on firm clay between elevations 37 and 77,

Spread footings or short piles on dense till below elevation 77,

Piles resting on shale bedrock near elevation 67.3.

## E. CONCLUSIONS AND RECOMMENDATIONS:

### 1. General Considerations. (Cont'd)

In the first case, the layered clay probably is not strong enough to allow a foundation pressure in excess of 1.0 ton/sq. ft. and some settlement would have to be accepted. Since considerably superior foundation media are located at only slightly greater depth, the first solution appears impractical and the shear strength and compressibility of the clay deposit existing between elevations 87 and 77 were not investigated. Only the latter two foundation types will be considered in detail.

### 2. Foundations supported on dense till

Footings foundations, placed near the top of the dense silt till at the approximate elevation 77.0, appear to be a practicable solution, and would necessitate an excavation approximately 15 ft deep. However, because the material consists of broken shale embedded in a silt and clay matrix, of consistency softer than plastic limit, some settlement could materialize in this deposit and the net footing pressure is recommended not to exceed 3.0 ton/sq. ft.

This method of foundation suffers from the drawback that the silt till contains ground water seams, under a head roughly corresponding to the water level in Hardy Creek.

E. CONCLUSIONS AND RECOMMENDATIONS:

2. Foundations supported on dense till (Cont'd)

Ground water seepage from the bottom of the alluvial layers occurred only in test hole 1 between elevations 86 and 87 and was absent in test hole 2, which is located further away from the creek. This seepage source therefore may not be a major problem in excavations.

On the other hand, ground water from the silty till stratum, if uncontrolled, could cause quicking in the subgrade when excavation reaches the footing level near elevation 77, which would result in a loosening of the till and in a loss of bearing support. For this reason, it is advisable to temporarily reduce the water level in this deposits below the setting level of footings. The water table could be lowered by pumping from well points placed along the perimeter of the excavation. The quantity of water to be disposed of is not anticipated to be large.

Consideration could be given to excavating only down to elevation 80, and to placing the footings at this level. In this way, the ground water in the underlying till would not be reached. To ensure against a blow-out of bottom of excavation, relief wells could be provided, in the form of pipes penetrating into the till to elevation 76. The last two feet of excavation should preferably be performed by hand, to avoid disturbance of the clay below the footing.

## E. CONCLUSIONS AND RECOMMENDATIONS:

### 2. Foundations supported on dense till (Cont'd)

Although some settlement of the clay remaining below elevation 80 would occur, it is probable that most of this settlement would develop during the period of construction of the abutments and without detrimental effects on the bridge superstructure.

### 3. Pile foundation

Piles could be supported, theoretically, either on the dense till below elevation 77, or on the shale bedrock near elevation 67.3.

The end-bearing capacity of piles resting on the surface of the bedrock will be considerably higher than in the overlying till below elevation 77 and it would appear much more economical to set the piles on the bedrock.

Because of the presence of ground water in the silt till below elevation 77, driven, displacement piles are preferable to prebored piles, which would encounter ground water problems. Steel H-piles appear to be well suited for this site, and the end-bearing capacity of such piles, driven to refusal on the surface of the shale near elevation 67.3, would be very high and determined by the design structural strength of the piles themselves, with little, if any, settlement.



## E. CONCLUSIONS AND RECOMMENDATIONS:

### 3. Pile foundation (Cont'd)

The driving of H-piles would be very easy down to elevation 77, but considerably more difficult below this level in the silt till, where dense accumulation of shale fragments will offer considerable resistance. However, judging from information available to us with regard to the driving of steel H-piles in similar material, it will be possible to drive the piles successfully to refusal on the surface of shale bedrock near elevation 67.3. However, it is recommended to reinforce the pile tips to prevent damage during the driving operations.

The level of the shale surface can be assumed to lie within inches of elevation 67.3 throughout the site, and thus the length of the piles and the setting depth can be confidently specified. It is recommended not to overdrive below the elevation given above, so as to avoid shattering the shale bedrock and to prevent distortion of the piles.

#### 4. Excavation and backfilling:

Excavations for footing foundations have been discussed in  
item 2.

Excavations for the new creek channel will pass mostly through the loamy clay and possibly through the upper layers of the layered clay and clayey silt. Relatively easy excavation is anticipated, and no major difficulty with ground water will be experienced during the performance of the work.

E. CONCLUSIONS AND RECOMMENDATIONS:

4. Excavation and backfilling (Cont'd)

The excavated material will consist mostly of clay and silt of soft to firm consistency, which must be regarded as a difficult backfilling material, due to its relatively high plasticity and water content. It is also highly susceptible to frost heave.

Imported, granular fill, placed to a high standard of compaction must be provided behind the new bridge abutment.

If the locally excavated subsoil must be used for the reconstructed embankments, some settlement within the embankments must be expected, and it is recommended not to place the final pavement surface until the embankments have settled. A layer of clean granular material should be included immediately below the pavements as a protective measure against frost action.

E. M. PETO ASSOCIATES LTD.,

*C. I. Freeman*

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

RK:eb

Report Prepared By :

*R. Kulesza*

R. Kulesza, P. Eng.

Job No. 6394

May, 1963.

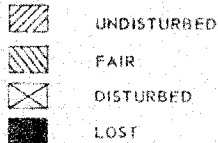
# e. m. peto associates ltd.

## SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

### BOREHOLE LOG

Job Name Hardy Creek Bridge Job No. 5244 Borehole No. 1  
 Client The Township of Brooke Casing 4" x 10' x 1/2" V.P. 10' x 1/2" Boring Date May 23rd & 24th 1963  
 Elevation 260.0 (G.T.M.) Compiled By R.P. Checked By R.P.

#### SAMPLE CONDITION



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

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth (feet)	Legend	Sample Type	Blows per ft.	Moisture %	WATER LEVELS - REMARKS
Sand and gravel fill	Brown		0'0"					
Silty clay with some sand, loam form, could be fill	Brown grey	Soft	2'4"		SS	11	23.9	D.T.P.L.
Ditto	Dk. brown-grey	Firm			SS	7	20.4	Grits and pebbles from 2.5 ft. in.
As above, layered	Mottled	Firm	8'0"		SS	6	25.6	Water seepage between 8 and 9 feet
As above, with organic silt	Grey-brown		9'0"					
Clay, layered	Grey-brown	Soft	87'0"		SS	12	27.5	Brittle, plastic, W.T.P.L.
	Grey	Firm			SS	8	21.6	
Ditto		Soft			2" SL	7	24.2	
			16'0"					
			78'0"					
			20'0"					W.T.P.L. V. moist to wet
Very sandy clay with dense fine gravel & some larger shale fragments	Dk. grey to black	Dense but clay soil			SS	32	13.6	Gravel consists of angular fragments of black shale. Water seepage at 20 ft. and 26 ft.
Very sandy silt with numerous shale fragments	Ditto	Dense			SS	42	8.1	
Shale bedrock	Black	Solid	67.4		SS	50/1"	28'6"	Reins. at 26'-7"
			28'7"					Diamond drilled from 28'7" to 31'7"
			22'7"					Recovery 60% (100%)

TEST HOLE TERMINATED AT 33 FT 7 IN.

#### Ground water data:

23rd May 1963

Casing at 5 ft. hole to 10 ft.

Water rose from 9'5" to 4'6" in 7 minutes and to 4'5" in further 5 minutes.

24th May 1963

Casing at 20 ft. hole to 21' ft. bottom 5' - slightly wet at bottom next morning.

Casing to 20 ft. hole to 28' - 7" water table to 2' - 8" in 5 minutes and to 3' - 7" in further 5 minutes.

DEFECTS IN NEGATIVE DUE TO  
 CONDITION OF ORIGINAL DOCUMENT

# e. m. peto associates ltd.

## SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

### BOREHOLE LOG

Job Name Hardy Creek BridgeJob No. 6394Borehole No. 2Client The Township of ProctorCasing 4" I.D. S.P. X 10' 20'Boring Date May 24th & 25th 1963Elevation 96.0 (Client's)Compiled By RRChecked By AP

## SAMPLE CONDITION

## SAMPLE TYPE

## ABBREVIATIONS



UNDISTURBED



FAIR



DISTURBED



LOST

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 SPLIT TUBE SAMPLE

W.S. WASH SAMPLE

R.C. ROCK CORE

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

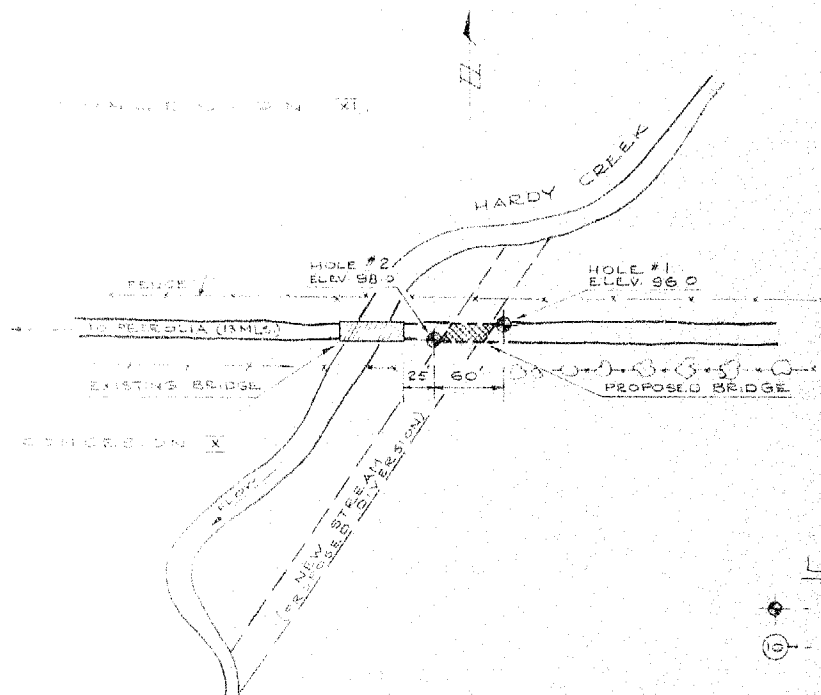
SOIL DESCRIPTION	COLOR	Density or moisture	Depth (feet)	Legend	Sample No. and Condition	Depth (feet)	W.T. (feet)	W.T.P.L. (feet)	WATER LEVELS & REMARKS
Sand and gravel fill	Brown		0'0"						
Silty clay with some sand lean form, possibly fill.	Mottled brown & grey		2'10"		1 SS	7	19.6		
As above, with some organic inclusions	Dk. brown	Firm			2 SS	7	27.3		
Ditto	Ditto	Soft	9'0"		3 SS	6	28.7		
	Mottled brown & grey		10'10"		4 SS	9	26.0		
Layered clay	Grey	Firm to soft	87.0		5 SS	10	20.9		Plastic, brittle
Silty clay and clayey silt layers	Grey	Firm							W.T.P.L.
Ditto	Ditto	Ditto			6 SS	7	22.3		
As above, with organic silt	Ditto	Ditto	24'5"		7 SS	10	23.0		
Clayey & sandy silt with some gravel	Dk. grey to black		24.6"		8 SS				Clay W.T.P.L.
Ditto	Ditto	Dense, but clay soft			9 SS	31	7.8		Gravel in the form of angular fragments of shale.
Ditto	Ditto	Ditto			10 SS	40	8.8		
			30'10"		11 SS	14'6"	8.2		Remains at 30'-10"
			67.2				50.4		(Presumably solid shale)

TEST HOLE TERMINATED AT 30' FT 10" B.

## Ground water data

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

Little noticeable seepage to 20 ft.  
Free water started between 23 and 25 ft. rose rapidly  
to 3 ft 2 in. and continued to rise slowly, but rate  
not measured.



SKETCH SHOWING BOREHOLE  
LOCATIONS

SCALE: 100' TO 1"

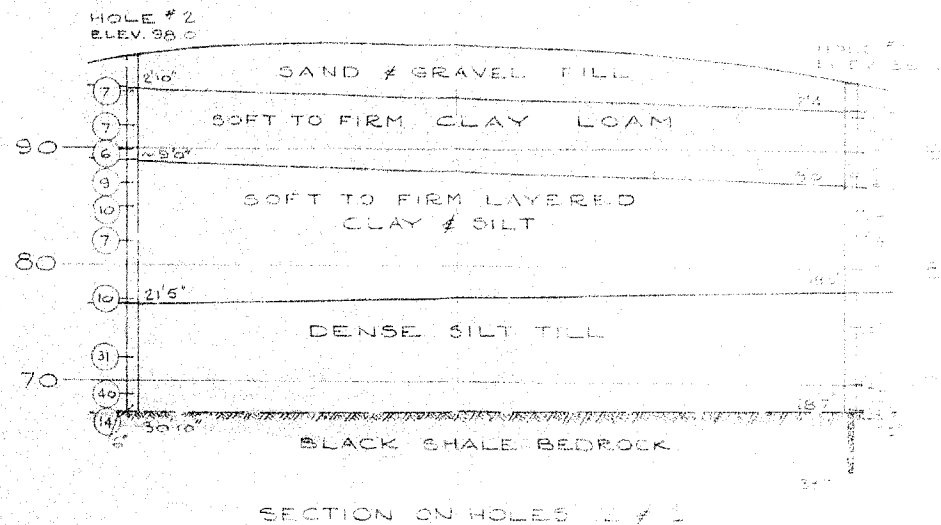
# LEGEND:

- - BOREHOLE
- ⑩ - BLOWS/FOOT (S.P.T.)

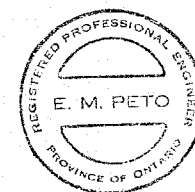
## NOTE 5:

- 1) SEE BOREHOLE LOGS FOR COMPLETE SOIL DETAILS.
- 2) ELEVATIONS HAVE BEEN SUPPLIED BY CLIENT

**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.



SCALE: 10' TO 1" (NATURAL)



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

HARDY CREEK BRIDGE  
PETROLIA

PREPARED BY:  
e.m.peto associates inc

JOB No. 6394 MAY, 1963 DWN BYWMS CHECKED BY: R/P

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT