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GEOCRES No. 40P4-40

W.P. No. _____

CONT. No. _____

W. O. No. _____

STR. SITE No. 14-291

HWY. No. _____

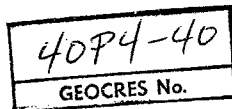
LOCATION TWP. BRIDGE # 41,
NEAR THEDFORD

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. NONE

REMARKS: _____

E. M. PETO ASSOCIATES LTD.

Job No. 62222

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

February 12th, 1963.

The County of Lambton,
c/o J. A. Monteith Associates Ltd.,
P. O. Box 579,
Petrolia, Ontario.Attention: Mr. G. Ingram, P. Eng.

Gentlemen:

Re: Soil Site Investigation,
Township Bridge No. 41,
Near Thedford, Ontario.

We have pleasure in submitting four copies of our
Report No. 62222 on the above site investigation.

The two test holes performed at the site have indicated that
the subsoil consists of very stiff silty clay and sandy silt till, followed by
a hard clay and then a hard silty clay, which may possibly be weathered
shale.

It is considered that the subsoil, from a depth of approximately
10 ft below the existing bridge deck downwards, (corresponding to elevation
13 and below), can safely support the new bridge foundations in the form of
spread footings, and that a contact pressure of up to 5 tons per sq. ft is
permissible without the risk of significant settlements. The actual foundation
depths will probably be greater and will be determined by the necessity to

protect the footings against scouring by the stream and against frost action.

The only difficulties with performance of the work can be caused by some water seepage in pervious seams in the upper layers of subsoil. However, the extent of such seams was limited, and the ground water problem is unlikely to be serious.

We consider the Report to be comprehensive within your terms of reference; we would, however, gladly provide additional assistance should you wish to raise any queries in connection with this work.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

RK:sb

THE COUNTY OF LAMBTON,
C/O J. A. MONTEITH ASSOCIATES LTD.

SOIL SITE INVESTIGATION
TOWNSHIP BRIDGE NO. 41,
NEAR THEDFORD, ONTARIO.

E. M. PETO ASSOCIATES LTD.
1287 Caledonia Road,
Toronto 19, Ontario.

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A. INTRODUCTION:

The work described in this report was authorized by Mr. G. Ingram, of J. A. Monteith Associates Limited, Consulting Engineers, by letter dated December 6th, 1962.

In connection with the proposed realignment of a minor road near Thedford, Ontario, a new bridge is to be provided to replace an existing structure where the road crosses the Decker Creek. A site investigation was required to determine the subsoil conditions for the design of foundations.

The existing bridge has a span of 16 ft. It will be replaced by 25 ft span, rigid frame structure of reinforced concrete, located close to the existing bridge.

Some 8 to 10 ft of fill will be placed above the existing road, making the new embankment about 12 to 14 ft above valley bottom.

No data concerning the pressures to be applied to the new bridge was available at this stage.

B. GENERAL INFORMATION:

1. Two test holes were performed at the site, one on either side of Decker Creek and along the south shoulder of the existing road. The approximate positions of the test holes were indicated by Mr. G. Ingram on drawing No. 194-6, on which the enclosed site plan is based. The test holes were 40 ft apart.

The test holes were set out in the field by our engineer, and the ground elevations were subsequently supplied by Mr. G. Ingram. The elevations are referred to a Consulting Engineers' temporary bench mark, the position of which is not known to us. The elevations are entered on the borehole logs and on the site plan and profile.

2. The test holes were terminated at a depth of 29.5 and 30.0 ft, in a stratum of hard, layered silty clay, or weathered shale.

3. The field work was performed by our drilling unit No. 6, between January 11th and 14th, 1963. Our standard drilling and sampling procedures were followed, as outlined in the enclosed Appendix A.

4. Laboratory testing of soil samples, apart from water content determinations, was confined to Atterberg Limit tests on typical samples of clayey materials, for soil classification purposes. The results of the tests are included in Appendix B.

C. SITE AND GEOLOGY:

The site of the proposed road is located on a minor road, running in an east-west direction on a line forming the continuation of the line of Highway 82, from a point where this highway makes a 90° turn towards the south, to Thedford; the bend is located approximately 1/2 mile north of the centre of Thedford, and the site of the bridge is approximately 600 yards to the east of this point. The minor road separates Lot 21, which is to the south, from Lot 22; both lots are in Concession III.

The Decker Creek, which flows in a northerly direction, meanders near the site, and has a flood plain approximately 350 ft wide. The bottom of the valley is some 15 to 20 ft below the level of the surrounding terrain.

The existing bridge, of 16 ft span, is approached by embankments which project roughly 6 to 7 ft above the creek level. To the north of the road, the terrain contains mud flats, while a wooded area extends to the south-west.

The creek at the site is approximately 15 ft wide, and only a few feet deep.

Geologically, the site is located in the St. Clair clay plain, where glacial processes have deposited a mantle of clayey till over a shale bedrock. The test holes have disclosed a silty sand till, followed by a clay till, which in turn rested on a highly fissured, hard silty clay with layered structure, which may in fact be highly weathered shale bedrock.

C. SITE AND GEOLOGY: (Cont'd)

Solid shale was not reached by the test holes.

D. SOIL CONDITIONS:

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

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

- a) Fill
- b) Very stiff silty clay till
- c) Very dense sandy silt till
- d) Stiff to hard clay
- e) Layered hard silty clay.

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

7) Embankment Fill

Both test holes were put down from the shoulder of the existing embankment, which was found to be built up of a silty clay mixed with some sand, which extended to a depth of 6.2 ft in test hole 1 and 7.9 in test hole 2. In the latter test hole, a gravelly fill extended from the ground surface to a depth of 2.2 ft.

The fill had a firm consistency in the upper layers, becoming soft to firm near the bottom of the embankments.

D. SOIL CONDITIONS:

a) Embankment Fill (Cont'd)

This material can be reused in the reconstructed embankments, but, due to the high silt content, the pavement should be protected with a cushion of granular material against possible frost heave.

b) Very stiff silty clay till

The fill was found to be resting on top of a layer of silty clay with pebbles and some sand, of till form and extending to a depth of 11.7 ft and 10 ft in test holes 1 and 2 respectively. This layer can be considered as the uppermost deposit suitable for the support of foundations.

The silty clay till was of grey colour with some brown mottling, due to partial oxidization, and it forms a desiccated, partly weathered crust of the subsoil. The material was very stiff, a standard penetration test at a depth of 7.5 ft in test hole 1 recording 43 blows per foot. The water content of two typical samples was 12.4% and 13.0%.

An Atterberg limit test performed on a sample of the stiff clay till gave the following results:

Liquid Limit 33%
Plastic Limit 17%
Plasticity Index 16

The material, in its natural state, thus has a water content well below the plastic limit.

D. SOIL CONDITIONS: (Cont'd)

c) Very dense sandy silt till

This deposit extended between the depths of 11 and 18.5 ft in test hole 1, and 10 and 18 ft in test hole 2. It consists of silty sand and sandy silt with pebbles, has a till form, and is of grey colour. The consistency of this stratum is very stiff, or dense, standard penetration test results recording 34 to 98 blows per foot, and the water content range was 10.8% to 17.2%; the higher water contents were in wet, sandy seams within the till.

Atterberg limit test on a typical sample of the till gave the following results:

Liquid Limit 24%
Plastic Limit 16%
Plasticity Index 8

Thus the water content is generally below the plastic limit.

d) Stiff to hard clay

A stratum of clay was encountered at a depth of 18.5 ft in test hole 1 and 18.0 ft in test hole 2. The clay was partly fissured, and was stiff in test hole 2 (29 blows per foot at a water content of 18.8%) and hard in test hole 1 (52 and 80 blows per foot, at water contents of 13.2% and 14.8%). In the latter test hole, the clay was distinctly fissured.

D. SOIL CONDITIONS:

d) Stiff to hard clay (Cont'd)

Two Atterberg limit tests gave the following results:

Test hole 1, Depth 20 ft:

Liquid Limit 42%
Plastic Limit 22%
Plasticity Index 20

Test hole 2, Depth 20 ft:

Liquid Limit 37%,
Plastic Limit 20%
Plasticity Index 17.

e) Layered hard silty clay

Both test holes were terminated in a deposit of layered hard silty clay, which was penetrated to a depth of 29.5 ft below the existing grade in test hole 1, and to 30 ft in test hole 2, which corresponds to 2.2 and 5.7 ft respectively below the surface of this stratum.

The deposit had a highly layered structure and was very fissured and dry. Standard penetration tests gave results of 76 to 170 blows for 6 inches of penetration and the water contents ranged from 6.9% to 8.7%.

An Atterberg limit test, performed on a typical sample from a depth of 29 ft in test hole 1, gave the following results:

Liquid Limit 35%
Plastic Limit 20%
Plasticity Index 15

The natural water content thus is very much below the plastic limit.

D. SOIL CONDITIONS:

1) Layered hard silty clay (Cont'd)

It is possible that this stratum in fact constitutes a highly weathered, upper crust of the shale bedrock; solid shale was not reached at this site.

E. WATER CONDITIONS:

It can be assumed that the position of the ground water table at this site is controlled by water level in the Decker Creek. The creek was frozen and covered by snow during the site investigation, but its surface was approximately 6.5 ft below the bridge deck.

Water level was established at a depth of 6.7 ft in test hole 1, and 9.0 ft in test hole 2, corresponding to the approximate elevation 14.

This probably corresponds with the normal water level in the creek.

Seepage seams were encountered near the bottom of the embankment fill, and in the sandy layers of very dense sandy silt till (deposit 2). No free wground water was struck below the bottom of this stratum.

Due to the limited extent of the pervious seams, seepage into excavations is likely to be of limited magnitude and will probably be easy to control during the operations.

F. CONCLUSIONS AND RECOMMENDATIONS:

1. Foundation level and bearing capacity

Foundations for the new bridge can be set at any suitable depth below elevation 13. However, a lower level may be necessary to protect the footings from frost heave and from scouring.

It is considered that the subsoil at any level below elevation 13 will safely support uniformly distributed contact pressure of 5 tons per sq. ft, with negligible settlements. However, should it be desired to design the structure for such high pressures, it is important to ensure that the bottom of excavations is not allowed to deteriorate before construction of footings due to absorption of free water or due to frost heave.

2. Excavation and backfilling considerations.

Slow seepage of water from the creek through pervious seams in subsoil is expected in excavations penetrating to a depth not greater than elevation 5; no seepage seams were evident below this level.

While the quantity of water originating from such pervious seams is not likely to be serious, every precaution should be taken to arrest such seepage and not allow it to collect in pools at the bottom of excavation, so as to prevent the deterioration of the formation grade below the footing.

F. CONCLUSIONS AND RECOMMENDATIONS:

2. Excavation and backfilling considerations (Cont'd)

Should it become unavoidable to retain the open excavations for any length of time before the construction of the footings, the excavated grade should be protected by an impervious seal, e. g. in the form of a thin layer of lean concrete. Alternatively, the last 6 to 12 inches of subsoil could be left unexcavated until the last possible moment before the construction of footings.

Should the quantity of seepage from any pervious layers prove considerable, the quantity of flow would probably be greatly decreased by diverting the creek. Alternatively, an attempt could be made to relieve the seepage by means of a trench, or well holes, located between the excavations and the creek channel.

Provided that it does not become excessively wet, the excavated material can be reused as compacted fill above footings, provided that a good standard of compaction can be achieved, so that settlements would not occur. Otherwise, the use of imported granular fill would be preferable and probably more practicable in confined space behind abutments.

F. CONCLUSIONS AND RECOMMENDATIONS:

3. New embankments.

It is believed that the new embankments will rise to between 12 and 14 ft above the valley bottom, which is roughly double the present embankment height.

On the basis of evidence obtained in the two test holes, the subsoil below the valley floor is very strong and of low compressibility, and provides a very sound basis for the support of the new embankments.

It is recommended to scrape off any loose fill or loam, or organic topsoil, in the areas which are to support the embankments. The new fill can then be placed on top of the scraped grade. Little, if any settlement of subsoil below the embankment is anticipated.

If the local soil is used for construction of the embankments, it is recommended to compact it well and to include a cushion of granular material at least 12 inches thick below the pavements to protect them against possible frost heave. The local subsoil contains a high proportion of silt, because of which it must be classified as susceptible to frost action.

F. CONCLUSIONS AND RECOMMENDATIONS:

3. New embankments (Cont'd)

As far as could be judged from the two testholes, the fill forming the existing embankments can be retained in the reconstructed bridge approaches, unless visual examination at the site during excavation indicates that it contains organic or otherwise inferior pockets, in which case it should be rejected.

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. 62222

February, 1963.

APPENDIX "A"
STANDARD PROCEDURE

The field investigation work is carried out by means of a skid mounted diamond drill rig.

Standard sampling procedures are followed. Casing is driven and cleaned, either by augers, tubes or by wash water.

Samples are recovered ahead of the casing at frequent intervals, with either a 2 inch or 3 inch O. D. split barrel sampling tube, Shelby tube, or split barrel sampling tube fitted with brass liners and special sharp cutting 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"

ATTERBERG LIMITS.

ATTERBERG LIMITS

B.H./Sa. No.	Depth Ft.	Liquid Limit, %	Plastic Limit, %	Plasticity Index	In situ water content, %
1 / 4	8	33.4	17.5	15.9	12.4
1 / 8	21	42.0	22.1	19.9	13.2
2 / 6	16	24.5	16.3	8.2	15.1
2 / 7	21	36.6	20.1	16.5	15.8
2 / 8	25	35.4	19.7	15.7	8.7

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Township Bridge #41
 Client County of Lambton
c/o J. A. Monteith Assoc.
 Elevation 23.2 (Client's)

Job No. 62222
 Casing 4" & BX
 Compiled By R. K.

Borehole No. 2
 Boring Date Jan. 14, 1963
 Checked By S. B.

SAMPLE CONDITION



UNDISTURBED



FAIR



DISTURBED



LOST

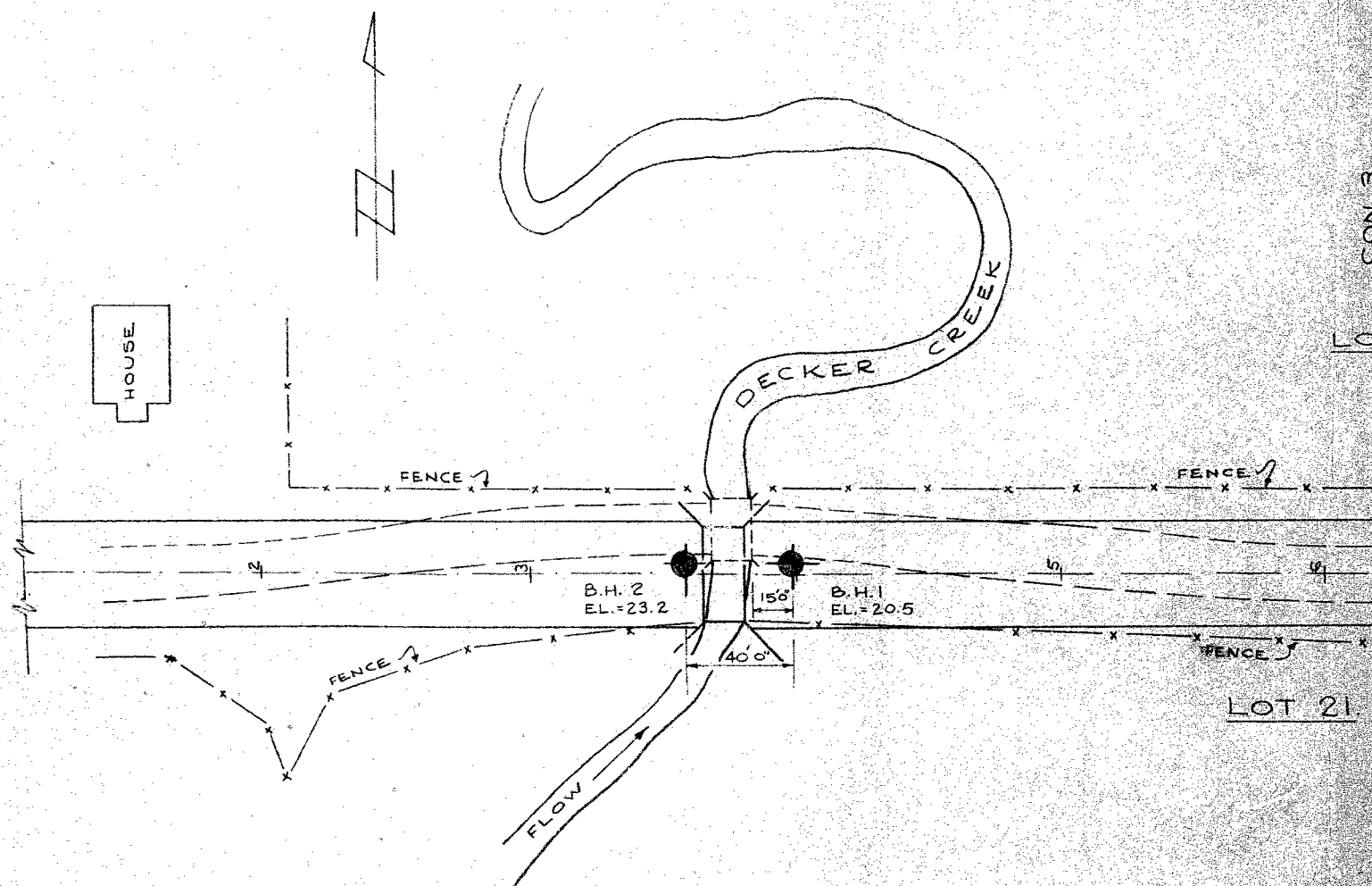
SAMPLE TYPE

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

ABBREVIATIONS

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

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth (Elevation)	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Water Level (ft.)	WATER LEVELS & REMARKS
			0'0"						
Sandy gravel fill									
Fill of silty clay, some fine sand	Brown	Firm	2'2"		1	S.S.	4/6" 11	14.9	
Fill of silty clay with some sand, with organic silt inclusion	Brown, some dark grey to black	Soft to Firm	6'3"		2	S.S.	3/6" 7	21.5	1" layer of wet coarse sand at 6'3"
Silty and sandy clay with pebbles (silty clay till)	Grey, some brown mottling	V. Stiff	7'10"		3	S.S.	4/6" 20	13.0	M. D. T. P. L. W. L. approx. 9' Wet sand layers.
Sandy and clayey silt with pebbles (sandy silt till)	Grey	Ditto	10'0"		4	S.S.	10/6" 13/6"	12.0	Water in hole after cleaning to 12'
Ditto	Ditto	Ditto			5	S.S.	21/6" 20/6" 28/6" 34/6"	9.3	
As above, less sandy; 1" hard limestone pebble	Ditto	Ditto	15'0"		6	S.S.	14/6" 18/6" 23/6"	15.1	M. D. T. P. L.
			18'0"						
Clay	Ditto	Stiff			7	S.S.	8/6" 29	18.8	D. T. P. L.
			24'3"						
Silty clay, layered, very fissured.	Ditto	Hard			8	S.S.	85/6"	8.7	
Ditto	Ditto	Ditto	30'0"		9	S.S.	76/6"	8.0	

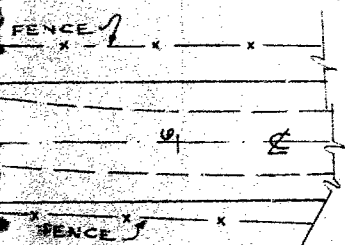


SITE PLAN

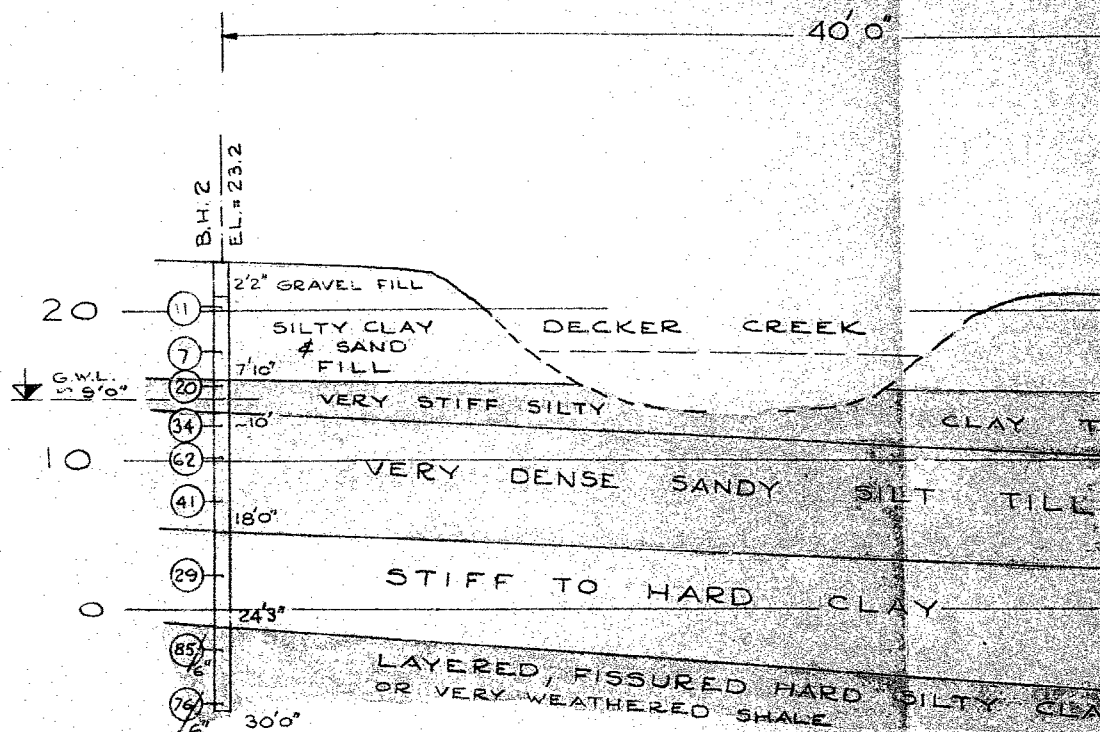
SCALE: 50' TO 1"

CON. 3

LOT 22



LOT 21.



SECTION ON HOLES 2 & 1

SCALES: HOR.: 5' TO 1"
VERT.: 10' TO 1"

LEGEND:

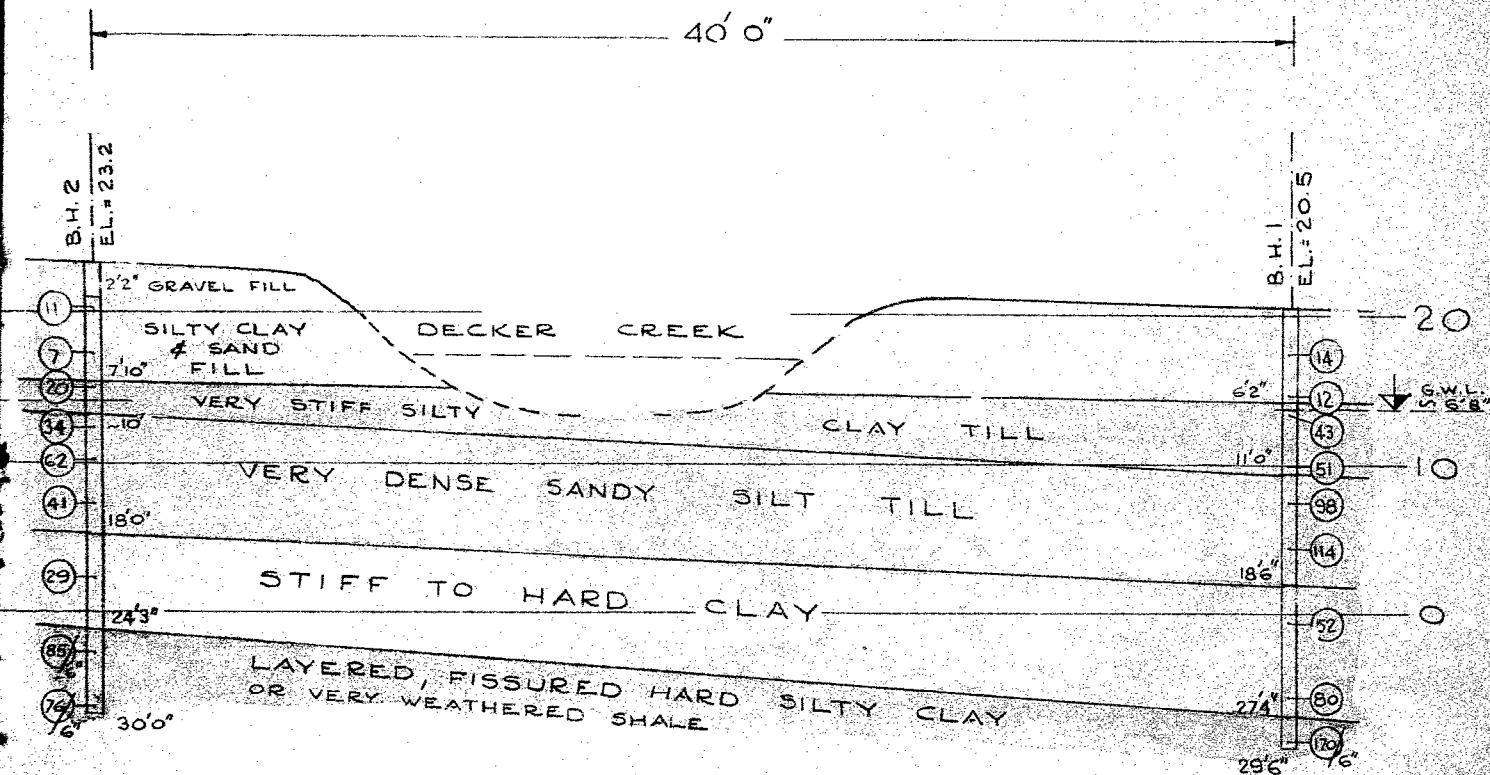
- BOREHOLE
- ② - BLOWS/FOOT (S.P.T.)
- ▼ - WATER LEVEL

NOTE:
SEE BOREHOLE LOGS FOR
COMPLETE SOIL DETAILS.

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



THE COUNTY OF J. A. MONTEITH CONSULTING ENGINEERS	
TOWNSHIP	
PREPARED BY E. M. PETO	
JOB No. 62222	JAN



SECTION ON HOLES 2 & 1

SCALES: HOR.: 5' TO 1"
VERT.: 10' TO 1"

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.

T (S.P.T.)

VEL

LOGS FOR
DETAILS.



THE COUNTY OF LAMBTON
c/o J. A. MONTEITH ASSOCIATES LTD.
CONSULTING ENGINEERS, PETROLIA

TOWNSHIP BRIDGE # 41.

PREPARED BY:

e.m. peto associates ltd

JOB No. 62222. JAN. 1963 DWN. BY: W.G. CHECKED: RK