

62-F. 278m
WILKESPORT
BRIDGE OVER
SYDENHAM R.
SOMBRA
TWP.

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

May 9, 1962.

REVIEW OF FOUNDATION REPORT BY
E. M. PETO ASSOCIATES, LTD.
(Bridge Office Ref. BA 1354)

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

Re: County of Lambton, Wilkesport Bridge
over the Sydenham River, Twp. of Sombra,
Lots 15/16, Con. III., District #1.

We have reviewed the above-mentioned report and
the following comments may be made:


(a) The design recommendations made in a letter dated
Jan. 17, 1962, are relevant since the soil conditions at the
new location do not vary significantly from the original
location.

(b) From the soil data presented by the Consultant,
friction pile foundations should be adequate. If, for any
economical reasons, that end-bearing piles are used, there
appears to be no difficulty of driving steel H-piles to bedrock
without the aid of jetting. We shall appreciate your substantiation
of the advantages that can be gained by jetting.

If further information is required in connection with
this project, please do not hesitate to contact our office.

KYL/MdeF

cc: Foundations Office
Gen. Files.

For: 
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

P.S. -- We are returning herewith,
report, plans, etc., per-
taining to this project.



ONTARIO

DEPARTMENT OF HIGHWAYS

Bridge Division

Memo to Mr. A. Stermac
Principal Foundation Eng.
Materials & Research Section
Lab. Bldg.
From G.C.E. Burkhardt

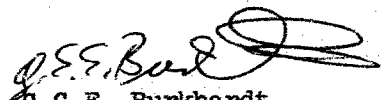
Date May 1 1962
Subject County of Lambton,
Wilkesport Br. over the
Sydenham Riv. Twp. of Sombra
Lots 15/16, Con. III,
Our File BA 1354

We are enclosing herewith a copy of the Foundation Report on the additional borings, which were taken at the site of this project by E. M. Peto Associates Limited, for your comments.

The designer would like to jet the "H" Piles if feasible. Do you have any objections or comments?

Late last year you reviewed for us a soils report for the same structure, but it was at a slightly different location, approximately 40 feet east of the present one. For this first report we received your Design Recommendations in a letter dated January 17, 1962.

GCEB/m


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

-3492

Mr. A. M. Toye,

January 17, 1961.

Bridge Engineer.

DESIGN RECOMMENDATIONS

Materials & Research Division,

(Foundation Section)

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

Re: Proposed Wilkesport Bridge,
County of Lambton (County Bridge C-7-4)
District #1.

In response to the request by Mr. K. L. Kleinsteinber, we have reviewed the above-mentioned report submitted by the consultants, E. M. Peto & Associates, (Job No. 6140).

The consultants' report shows the proposed bridge to be located approximately 40 ft. West of the existing one and their subsoil investigation has been confined to this location. The Preliminary Plan prepared by the bridge consultants - (Todgham and Case, Ltd., Chatham, Ontario, Dwg. No. 61161-3), shows the centre line of the proposed bridge to coincide with the centre line of the existing one. No subsoil information is available at the location of the existing bridge; therefore, if this line is selected, further field work will be necessary.

A meeting was held in this Office on January 9, 1962, to clarify and discuss the various procedures and recommendations made by the consultants in their report. On most of the questions, agreement was reached.

Following, we are submitting to you, our recommendations pertaining to the design and construction of the footings of this structure:-

Spread footing support can be obtained in the medium-stiff silty clay till. A safe design load of 3000 p.s.f. can be used for footing design typically 8 ft. to 10 ft. wide.

cont'd. /2 ...

Recommendations - (cont'd.) ...

North Abutment	562.0	or below
South Abutment	570.0	or below
Pier	557.0	or below

Note:- Footing elevations are based on the assumption that the proposed structure will have the same span lengths as the present structure.

The prediction of 3 inches of settlement, as reported by the consultants, seems to be reasonable and most likely the greatest part will occur within 6 months.

A dewatering scheme will be necessary as excavations will be carried out below the river bottom. On the North bank the subsoil is relatively impermeable; hence, no major problems are anticipated. On the East bank and centre pier location, however, the subsoil is much more permeable because of the presence of organic silt and sandy material. Protection against scour will be necessary for the abutment and pier footings. If sheeting is used for this, it may be incorporated into the dewatering scheme.

As an alternative, the structure can be founded on large displacement piles. For steel tube piles, a design load of 30 Tons per pile can be used, if the piles are driven to an estimated elev. 515.0. For timber piles driven to approx. elevation 535.0, a design load of 15 Tons per pile may be used. Piles should be treated if not completely below the lowest established water table.

We believe that the above-mentioned will prove to be adequate for the future design work. If we can be of any further assistance, please feel free to call on our Office.

P.S. -- Returned herewith,
is your copy of Peto's
report and Todgham &
Case's Dwg. 61161-3.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.
Per:

MD/MdeF

M. Devata
(M. Devata,
SR. PROJECT FOUNDATION ENGR.)

cc: Foundations Office ✓
Gen. Files.

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 6140

1287 caledonia road
TORONTO 19, ONTARIO.
RUssell 9-1126

January 15th, 1962.

The Department of Highways
of Ontario,
Materials and Research Division,
(Foundation Section),
Parliament Buildings,
Toronto 5, Ontario.

Attention: Mr. M. Devata, P. Eng.

Dear Sir,

In reference to our conversation of January 9th, 1962, please find attached a copy of our letter dated November 2nd, 1961, re Sydenham Bridge, County of Lambton, County Bridge C-7-4 which was sent to Todgham and Case, Consulting Engineers,

Yours very truly,

E. M. PETO ASSOCIATES LIMITED



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

CFF/ap
Attachment.

November 2nd, 1961.

Todgham and Case,
Consulting Engineers,
P. O. Box 386,
151 Thames Street,
Chatham, Ontario.

Attention: Mr. H. H. Todgham

Dear Sir,

Sydenham Bridge,
County of Lambton,
County Bridge C-7-4

Further to our telephone conversation last week in connection with (a) the settlement at the piers for this bridge (b) the proposed elevation for the south abutment, elevation 564, bearing in mind the possible widening of the channel up to the abutment face.

The details of the proposed pier are as follows:-

Elevation	447.0
Gross Load	1, 882, 000 lbs.
Footing Mat	52' X 12'

The theoretical settlement has been re-calculated using this latest data with the result that the total final settlement has been computed as being 3.2 inches. However, bearing in mind that this is caused mainly by the dead load, by virtue of the condition that the live loads do not act for any extended period of

time, it is considered that the total settlement is unlikely to exceed a maximum figure of 3 inches. This is a little more than given in our report ref. 6140, and this increase can be attributed to the larger footing size being used as compared with the footing size assumed for the original calculation.

Turning now, to the proposed elevation of the south abutment at elevation 864.

There are two aspects to consider here, the first being the bearing value of 2 tons/sq. ft. given in the report, and the second the possibility that the channel may be widened right back to the abutment.

In the report the bearing value was based on three assumptions which are now not valid. These were

- (i) the abutment would be set back clear of the river at or near the position of boring #4.
- (ii) Because of (i) an overburden effect was applicable and could be counted on to increase the bearing value.
- (iii) The conditions were superior in so far that the abutment would not be affected by water and therefore an undrained shear strength of 1200 p.s.f. was used as opposed to 1000 p.s.f. for the piers.

Now it is considered that the overburden may be removed or at least reduced to a minimum of 4 feet and, in addition, the river could have access to the abutment. Thus the conditions at the abutment approach those applicable to the pier.

Therefore applying these altered conditions to the abutment including reducing the undrained shear strength from 1200 p.s.f. to 1000 p.s.f., the bearing value for an abutment 10 ft. wide x 25 ft. long (assumed in the report) is reduced to 1.5 t.s.f. In order to obtain a bearing value of 2.0 t.s.f. near the water edge such an abutment would have to be placed at elevation 860.

In view of this it may be more economical to increase the span of the bridge slightly, with the object of obtaining additional protection to the abutment, reducing the depth of excavation and still maintaining a bearing value of 2.0 t.s.f. at elevation 564.0. As a final check we would like to have the opportunity to review the stability of the abutment when the size and location has been decided; such a check would also include a review of the horizontal stability of the south abutment as an earth retaining structure.

The question of erosion is difficult to advise on without a knowledge of local conditions in respect of the regime of the creek. From a soils aspect the silty clay till is not a soil which is easily eroded, but an examination of the creek profile at the line of the borings would appear to suggest that, in the central area of the river, erosion has taken place to elevation 561.0, decreasing to 564.7 and less as the south bank is approached. Furthermore the creek bed is protected by a mantle of sand, stones and organic debris. This latter layer is the one that is likely to be washed away depending on the volume of water and velocity of flow under existing conditions.

Following the construction of the piers the conditions will be changed, nevertheless it would appear reasonable to deduce that if the south abutment is to be placed at the water edge (by the widening of the channel) it would be a wise precaution to drop the footing elevation from 564.0 to 560. On the other hand if the creek is not widened or the abutment is moved clear of the water edge then elevation 564.0 could be regarded as safe from erosion.

We trust you will find these observations helpful.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

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

CFF/ap

Report by E. M. Peto & Associates

Re: Sydenham Bridge, County of Lambton (County Bridge C-7-4)

General Information.

Page 2 a) Is this clay till or not - The consistency of this material is soft to v. soft.

b) What is meant by soft shale - is it weathered shale?

c) Depth of each layer is recorded to ^{nearest} inch - Is this true?

Page 3. In general information no mention of spread footings

Page 6: Why it is not economical to perform in-situ vane tests.

Page 7:- Difficulties of construction of 2 structures in the vicinity of the above mentioned structure is described - The report doesn't indicate the type failure, such as embankment failure, movement of piles etc.

SITE & GEOLOGY

Page 8:- Is sandy till is not till?

Page 14:- Last Paragraph - ~~The~~ ~~stat~~ On what basis it is assumed normally consolidated.

Page 2 - 1st Paragraph - ... the ...
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Page 2 - 2nd Paragraph - ...
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Page 2 - 3rd Paragraph - ...
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Page 2 - 4th Paragraph - ...
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Page 25 - Paragraph 1 - ...
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DEFECTS IN NEGATIVE DUE TO
COLLISION OF ORIGINAL DOCUMENT

For 3. The consistency should be noted. Also
and, therefore, should be noted how Φ supports
2 types of foundation.

1) Near Alignment of 8 Trenches at elevation 560.0

Refer to #1 & 6

Mean strength value from 4m to 1000 (Elev 570 to 580)
is 4.4 kN/cm²
Below 1000 is not measurable

Testing depth \rightarrow 2000 to 5000

Other values are very few. Mean strength measured to 21

at elevation 560.0 (Refer to #6)

2) Joint Rock at elevation 571.0 at elevation 570

Other values are not available for this joint

Refer to #1 & 6

No shear strength measurements at Elev 571 and above
Below Elev 570 \rightarrow 1300 to 2000 to 5000

~~above Elev 570~~ \rightarrow Above 570

Major N values were obtained above Elev 570

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

~~See also to line 5750~~

At elev 5800 water table is below 10.

At elev 5800 or below the shear strength values
in the order of 100 to 200 and the bearing capacity
is in the order of 100 to 200 and is recommended by
the designer.

Reference Recommended by some one at elev 5800

Ref. B.H. 2 4 4 2

At elev 5800	N. 4 to 8 kN/m ²	Elev 5800	72,500 kN/m ²
N. 6 to 8 kN/m ²	Elev 5800	700 kN/m ²	

Reference should be corrected at elev 5800

And a bearing capacity of $1\frac{1}{2}$ to 2.

Note: The designer recommended higher bearing pressure
since they observed lower shear strength values in the basement
location. ~~whereas~~ whereas at the floor location the measured
shear strength is higher but the recommended bearing pressure is lower.

The results of the experiment.

[illegible][illegible]

Foundations

① → Why do we need state key system
if we have plain → Why not a lockable file

② → regarding the other question above that 200 is
 a small amount to be charged to the company. It is not a
 significant amount. 3/4 of the 200 is extended for 1 or 2 yrs
 to start.

$\textcircled{5} \rightarrow \textcircled{1}$ ~~the~~ ~~same~~ ~~idea~~ ~~for~~ ~~the~~ ~~moment~~ - We should
 have had some ~~idea~~ ~~about~~ ~~the~~ ~~size~~ ~~of~~ ~~the~~ ~~small~~
 Is it

De "Globe" 30. 1884. - Abt. 5. 1.

FEB. 1961

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

Page 22 - Last Paragraph

Page

Page 23 - Paragraph 1

... the ...
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 ... the ... etc

Page 24 - Paragraph 1

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Paragraph 2 - Paragraph 3

Page 25 - Paragraph 1

Paragraph 2

Paragraph 3

Paragraph 4 ...
 ... the ...
 ... the ...
 ... the ...

Page 29 - Paragraph 1

She conceals himself better than ...
 ... the ...

End bearing Pile

What is the reason for end bearing pile

Page 2 The reason is that the pile extends to the end bearing
of the pile and is not in the soil.

ENC

$$9 \times \frac{\pi}{4} \times 2000 + 28 \times 750 \times \pi$$

$$9 \times \frac{5 \times 4}{4} \times \frac{5 \times 11}{2000} + \frac{\frac{30}{20} \times 15 \times 11}{4}$$

$$\frac{4x}{5} = 12$$

100

大光明

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

FEB. 1961

Re Proposed Wakesport Road
Survey of Canada (by E.M. Gato Vol No 645)

Footings for North Section

Footings Elev 560 C

SIZE OF FOOTING 10' x 40'

N = 6 Blows/ft

C = 1000 lbs/ft

Net bearing pressure (tons) = $q_n = C/N$

$$q_n = \frac{C}{N} = \frac{1000}{6} = 167$$

(where q_n = net of foundation)

q_n = net of foundation

Note: q_n

$$D = 577 - 560 = 17$$

$$q_n = \frac{C/N}{\frac{1000 \times 7}{2000 \times 3}} = \frac{7}{6} = 1.17 = \text{say } \underline{1.2 \text{ tons/ft}^2}$$

Footings for South Section

Footings Elev 570 C or below

N = 7 Blows/ft

C = 1500 lbs/ft

Net bearing pressure (tons) = $q_n = \frac{C}{N}$

$$D = 590 - 570 = 20$$

Note: q_n

$$q_n = \frac{C/N}{\frac{1500 \times 7}{2000 \times 3}} = \frac{21}{12} = \underline{1.75 \text{ T s.f.}}$$

Footings for Pier

Footings Elev. 557.0

Nails = 8 bowls

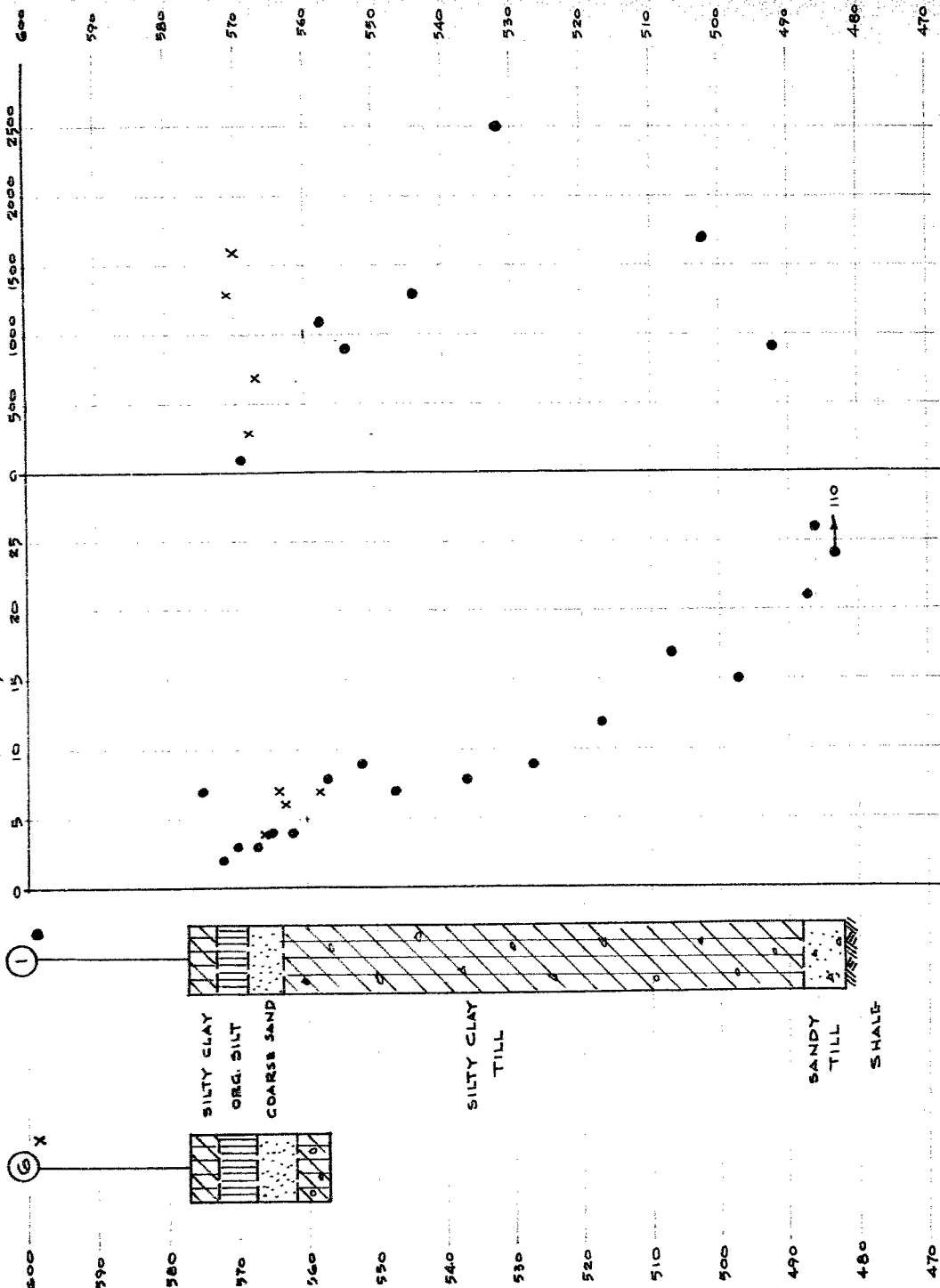
Covers = 1200 P-34

$$P_a(\text{net}) = \frac{C_u}{F} = \frac{200 \times 7}{200 \times 3} = 1.5 + 3.1$$

SHEAR STRENGTH
P. S. F.

STANDARD PENETRATION
BLOWS/FOOT

BOREHOLES



ORIGINATED

DRAWN

CHECKED

APPROVED

DATE

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & RESEARCH SECTION

Project: Highway Bridge

County of Waterloo (By A.M. S.H.)

SCALE

W. P. NO.

JOB NO.

DWG. NO.

SHEAR STRENGTH

P. S.F.

2500 2000 1500 1000 500 0

600

590

580

570

560

550

540

530

520

510

500

490

480

470

3750 P.S.F.

STANDARD PENETRATION

BLOWS/FOOT

25

20

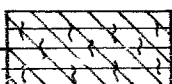
15

10

5

0

(5) X



BOREHOLES

(4)

SILTY CLAY
ORG. MAT'L.

SILTY CLAY
TILL

SANDY TILL
HARD SHALE

600

590

580

570

560

550

540

530

520

510

500

490

480

470

ORIGINATED
DRAWN
CHECKED
APPROVED
DATE

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

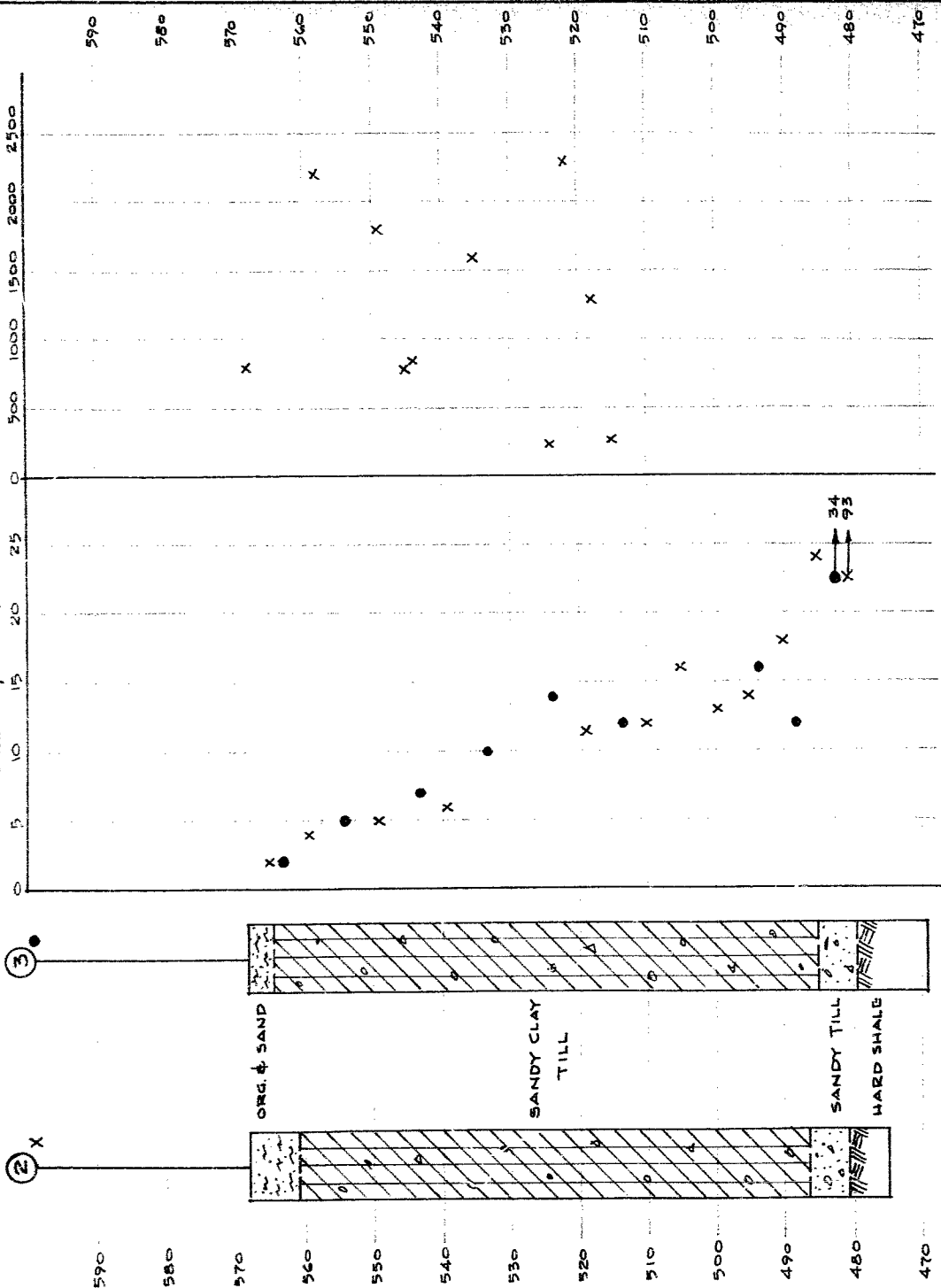
SCALE
W. P. NO.
JOB NO.
DWG. NO.

Entered into file by [illegible]
Copy of Lab. Work (by E.M. Perry)

SHEAR STRENGTH
P.S.F.

STANDARD PENETRATION
BLOWS/FOOT

BOREHOLES



ORIGINATED

DRAWN

CHECKED

APPROVED

DATE

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

PROPOSED WILFESPORT BRIDGE
COUNTY OF LAMBERTON (W.E.M.R.)

SCALE

W. P. NO.

JOB NO.

DWG. NO.

BA B54A

E. M. PETO ASSOCIATES LIMITED

1287 Caledonia Road,
Toronto 19, Ontario.

Our Job Number 6216

Russell 9 - 1126.

March 6th, 1962.

Messrs. Todgham & Case,
Consulting Engineers,
P. O. Box 386,
151 Thames Street,
Chatham, Ontario.

62-F-278M

Attention: Mr. H. H. Todgham.

Dear Sir,

Sydenham Bridge,
County of Lambton
County Bridge C-7-4
Additional Testholes

We have pleasure in submitting three copies of our Report Number 6216, dealing with the soil conditions at the final positions of the new bridge abutments. One additional copy has been forwarded directly to Mr. O. Van Deurs, County Engineer, County of Lambton.

The design pressure of 3000 lb /sq. ft., which, according to your letter of February 9th, 1962, has now been adopted, can be considered as permissible, provided that the footings will be covered with a sufficient height of overburden, as indicated in the report.

PAGE TWO

However, the south abutment will have to be set below the elevation 566 which was considered in the above letter, since the clayey till stratum at this abutment was found to commence only at the elevation 566.5.

We trust that the additional information, contained in the report, will be sufficient for the completion of the design of the bridge. However, we would be very pleased to provide further assistance should you wish to raise any points.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

C. J. Heiman

for E. M. Peto, P. Eng.

HK/ap

THE COUNTY OF LAMBTON

c/o TODGHAM AND CASE, CONSULTING ENGINEERS

SOILS REPORT

for

SYDENHAM BRIDGE (COUNTY BRIDGE C-7-4)

ADDITIONAL TEST HOLES

E. M. PETO ASSOCIATES LTD.,

1287 Caledonia Road,
Toronto 19, Ontario.

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Covering Letter

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Appendix "A" Standard Cone Probe Results

Appendix "B" Laboratory Test Results

Borehole Logs

Drawings - Site Plan and Profiles

A. INTRODUCTION:

The work described in this report was authorized verbally by Mr. H. H. Todgham, of Messrs. Todgham and Case, Consulting Engineers, on behalf of the County of Lambton.

A new bridge is to replace an existing structure, carrying a road over the north branch of the Sydenham River near Wilkesport, in the Township of Sombra, County of Lambton.

Between April 27th and May 4th, 1961, this company has carried out a preliminary site investigation, consisting of six test holes. The results of this investigation were presented in our Report No. 6140 of June 15th, 1961.

At the time of the first site investigation, the exact alignment of the new bridge was not known. During a conversation held at the County Engineer's office on Mar. 29, 1961, Mr. O. VanDeurs, the County Engineer, County of Lambton, informed our engineer, Mr. R. Kulesza that the re-aligned highway may run near the eastern side of a cemetery, which is located a few hundred feet west from the northern end of the existing bridge. Mr. VanDeurs advised us to carry out the test holes on a line running about 50 ft west of the northern abutment of the existing bridge, and converging on the present alignment of the road at the cutting south of the existing bridge.

A. INTRODUCTION: (Cont'd)

On the same day, Mr. R. Kulesza visited the office of the Consulting Engineers to inform them of the start of the site investigation and discussed with Mr. H. H. Todgham the proposed location of the test holes.

After the site investigation was completed, a decision was made that the highway would be realigned so that the new bridge would be just east of the existing structure. Additional test holes were therefore required to confirm whether the subsoil conditions at the new location are similar as established during the preliminary site investigation. It was decided to drill two test holes, one on either side of the river, and near the likely location of the new bridge abutments.

At the time when the additional test holes were drilled, embankment fill for the realigned highway was already placed up to the probable position of abutments.

B. GENERAL INFORMATION:

1. The two additional test holes were drilled near the probable positions of the new bridge abutments. Testhole 7 was drilled on the south side of the river and test hole 8 on the north side. In addition, a standard probe was driven four feet away from test hole 8.

The positions of the new test holes are shown on the enclosed Drawing No. 1, on which the positions of test holes 1 to 6, drilled previously, are also indicated.

Test hole number 7 was drilled 106 ft south of the centre of the bridge and 16 ft east of the eastern wall of the southern abutment.

Test hole 8 was located 102 ft north of the centre of the bridge and 5 ft east of the northern abutment. The positions and plan of the abutments, plotted on Drawing No. 1, were obtained from Client's Plan - No. 61161-1.

2. Ground elevations at the positions of the test holes were measured by our engineer and referred to a temporary bench mark, taken as the middle of the bridge deck at mid-point of the bridge. According to the Client's survey plan number 61161-1, the elevation at the described point is 591.7. The elevations are shown on the borehole logs and on the soil profiles.

B. GENERAL INFORMATION: (Cont'd)

3. The depth of the additional test holes was only sufficient to confirm whether the subsoil conditions, as established during the original site investigation, can be extended to the new site of the bridge. Both test holes were terminated at a depth of 36 ft 6 in.

4. Details of the soil conditions found in the test holes 7 and 8 are described on the enclosed borehole logs. Simplified soil profiles were prepared, using also results of the test holes drilled during the original site investigation, as presented in our Report No. 6140. The profiles are plotted on the enclosed Drawing No. 1 and 2.

5. A standard cone probe was driven 3 ft 9 in. south of test hole 8, to a depth of 35 ft. The results, expressed as the number of blows for each successive foot of penetration, are tabulated in Appendix A.

6. The field work was carried out by our field crew using drilling rig No. 4, between January 30th and February 1st, 1962. Our standard drilling and sampling procedures were followed, as outlined in Appendix A of our Report No. 6140.

7. Laboratory soil testing programme consisted of moisture content determinations, measurements of undrained shear strength of clay by unconfined compression tests, wet and dry density determinations, and Atterberg Limit tests.

B. GENERAL INFORMATION:

7. (Cont'd)

The moisture contents are entered on the borehole logs and plotted on Fig. 1, against the elevation.

Results of the unconfined compression tests are tabulated in Appendix B, together with the wet and dry densities and void ratios of the various samples. The undrained shear strength has been plotted against depth on Fig. 1.

The results of two Atterberg Limit tests are included in Appendix B, and are also plotted against elevation in Fig. 1.

The results of the corresponding soil tests, carried out during our original site investigation, are also included in Fig. 1 for the purpose of comparison of the results throughout the area investigated.

C. SOIL CONDITIONS:

During the site investigation presented in Report No. 6140, it was established that the main overburden over bedrock at the site has the form of a silty clay with pebbles, (clay till) of glacial origin. This stratum was found to commence at an elevation varying from 561.0 under the river bed (test hole 2) to 563.4 in test hole 1 north of the river, and to 575.8 in test hole 4 south of the river. The clayey till rested on dense silty sand with stones, commencing at an elevation varying from 485 to 487.9; the dense sand was followed by the black shale bedrock, the surface of which was encountered between the elevations 479.6 and 482.0.

The silty clay till was covered under the river by a layer of sand and gravel with organic debris and some mud. The granular stratum was found to extend north of the river, where it was covered also by some organic silt or very soft silty clay with peat, and by a weathered clayey crust with plant roots.

South of the river, test hole 4 disclosed that the till was overlain mainly by a compact clayey backfill with some sand and organic matter.

Bridge foundations in the form of spread footings would have to be supported in the clayey till stratum, and the suitable elevations and bearing capacity were given in Report No. 6140.

C. SOIL CONDITIONS: (Cont'd)

Because of the altered position of the new bridge, the two additional test holes were to indicate whether the data on footing elevations and the allowable bearing capacity, was still applicable to the altered position of the bridge.

The results of the new borings were as follows:

Test hole 7 (south of the river)

In this test hole, a 1 ft 1 in. layer of organic topsoil was followed by a silty and sandy clay with organic matter, of very soft consistency (standard penetration test results were in the range of 1 to 5 blows/ft). The soft clay extended to a depth of 8 ft 9 in. (elevation 567.3), where it was followed by a 9 inch thick layer of water-bearing sand and gravel. The clayey till stratum commenced below the sand and gravel layer, i. e. at the elevation 566.5.

The top five feet of the clayey till were found to be soft or soft to firm, and the material became firm near the elevation 560. Beginning at the elevation 556 standard penetration test results were 12 to 13 blows/ft. At the elevation 540 the N-value rose to 16 blows/ft.

The undrained shear strength of the clayey till, measured by unconfined compression tests on relatively undisturbed samples recovered between the elevations 540 and 560, was in the range of 1230 to 1620 lb/sq. ft, with one result of 865 lb/sq. ft. This single low result may have been caused by sample disturbance or the presence of a stone, since the moisture content

C. SOIL CONDITIONS:

Test hole 7 (south of the river) (Cont'd)

of the sample was low and the density high.

The undrained shear strength between the elevations 540 and 560 in test hole 7 was found to conform well with the average of tests performed on samples from test holes 1 to 6, which were replotted against the elevation on the enclosed Fig. 1. The curve shows that the average value of shear strength of 1200 lb/sq. ft, which was assumed for the bearing capacity calculations for the south pier, is correct. Consequently, the footing elevation and the allowable bearing capacity, as indicated in our report No. 6140 for the south abutment, are applicable to the new position of this abutment.

The moisture contents plotted on Fig. 1 have lower values than at corresponding elevations in the original set of test holes. In addition, the standard penetration test results below the elevation 560 range from 12 to 16 blows/ft and thus are considerably higher than the average of the original holes. This apparent stiffer consistency of the material may be due to an overconsolidation caused by the weight of the abutment of the existing bridge and of the embankment fill behind this abutment, which are in the vicinity of test hole 7.

C. SOIL CONDITIONS: (Cont'd)

Test hole 8, (north of the river)

In test hole 8 a 5 ft 6 in. thick layer of desiccated silty clay with organic matter, of brown colour and firm consistency, extended to the elevation 572.8. It was followed by saturated, grey silty clay with considerable organic content and a softer consistency. A decomposed wood log was present near the depth of 8 ft.

A layer of sand and fine gravel with clay and organic matter followed at the elevation 568.1, and was 2 ft 4 in thick. It was very soft, with a moisture content of 127%. A 1 ft 9 in. thick layer of very soft silty clay with organic matter followed, and at the elevation 564.0 the test hole encountered an 11 in. thick layer of sand with gravel, which was water-bearing.

The silty clay till stratam began at the depth of 15 ft 2 in. (elevation 563.1). In contrast to test hole 7, where a shear strength of the clay between the elevation 560 and 540 was found to conform with the results of the previous test holes, in test hole 8 the clay till was considerably softer. Unconfined compression tests carried out on six samples, supposedly undisturbed, from between the elevations 555 and 544 gave results of undrained shear strength in the range of 520 to 970lb/sq. ft.

C. SOIL CONDITIONS:

Test hole 8 (north of the river) - Cont'd

However, the moisture contents in the corresponding range of elevations either coincided with the average of the previous test holes or were lower than this average, as shown on Fig. 1. Also, the results of three standard penetration tests performed between the elevations of 554 and 544 ranged from 12 to 15 blows/ft, thus considerably exceeding the average of 5 to 7 blows in the corresponding range of elevations in the previous test holes. Also, the liquid limit of a sample from borehole 8 from the elevation 542 was even lower than the previously established average.

It is difficult to explain this contrast between the standard penetration test results, which are considerably higher than in the previous test holes, and the seriously reduced undrained shear strength of the material between the elevations 540 and 560 in test hole 8. One tentative explanation which may be put forward is the possibility of existence of excess pore water pressure in the clay till in the area of testhole 8 at the time when this boring was performed. The excess pore water pressure may have been caused by the placement of fill for the new embankment just north of this test hole. The fill has been placed very recently; it extended to very near the position of the test hole, and was some 10 to 12 or more feet thick. The excess pore pressures set up by the placement of the fill could have reduced the shear strength of the clayey till, while causing the observed increased resistance in standard penetration tests.

C. SOIL CONDITIONS:

Test hole 8, (north of the river) - Cont'd

Should the above supposition be correct, the excess pore pressures will dissipate with time, and the shear strength of the material will increase. However, it would not be possible to predict as to what extent the shear strength will increase by the time the abutment is constructed, without field measurements of ^{pore} pressures. For this reason, in view of the observed low undrained shear strength between the elevations 540 and 560 in test hole 8, and in view of the fact that two unconfined compression tests performed on samples from test hole 2, located also north of the river, from the elevation 544 gave an undrained shear strength of about 800 lb/sq. ft, we consider it safer to use a reduced shear strength for the bearing capacity calculation for the north abutment. The revised bearing capacity, based on a shear strength of 750 lb/sq. ft (as compared to the 1000 lb/sq. ft used in the original calculations) is given in Chapter E.

D. WATER CONDITIONS:

In test hole 7, at the south abutment, free ground water was struck in a sand and gravel layer, located between the elevations 567.3 and 566.5; water rose in the test hole to the elevation 573.3 in one minute, and became steady at this level.

In test hole 8, at the north abutment, some seepage occurred in the layer of sand and fine gravel with clay and organic matter, present between the elevations 568.1 and 565.8. A more rapid inflow of ground water was observed when the test hole reached the layer of sand and gravel resting between the elevations 564.0 and 563.1. Water level rose in the test hole to the elevation 574.1.

The river at the existing bridge was frozen during this site investigation, and the surface of the ice was at the elevation 573.5 on January 25th, 1962. The test holes were performed on January 30th and 31st, 1962.

The water level observations prove the earlier conclusion that the sand and gravel layer, located below the upper layers of silty clay with organic matter and above the clayey till stratum, has a relatively high permeability and carries water from the river. The equilibrium position of the ground water table at the position of the abutments approximates therefore to the water level in the river. The pressure of water in the sand and gravel layer will fluctuate with the water level in the river, and will be greatest during the spring floods.

In order to isolate the excavations from water seepage, sheeting should be driven into the clayey till stratum, to the elevation 562 or below.

E. CONCLUSIONS AND RECOMMENDATIONS:

1. South Abutment

According to the information contained in a letter from the Consulting Engineers dated February 9th, 1962, the face of the south abutment at the centre line of the roadway will be 25 ft. south of the face of the existing abutment. Test hole 7 was located 27 ft. south of the face of the existing abutment and in the line of the new roadway, so that the soil conditions, determined in this test hole, can be taken as strictly applicable to the foundation problems of the south abutment.

It was found in test hole 7 that the stratum of clayey till begins at the elevation 568.8. The till is of firm consistency, and between the elevations 560 and 540 the average undrained shear strength gradually increased from 1200 to 1400 lb/sq.ft. This is in agreement with the average of results presented in our Report No. 6140.

No shear strength results are available above the elevation 560 in test hole 7, but two standard penetration tests indicated that above the elevation 562 the clay till is softer than at the greater depth. Consequently, we consider it advisable to use a reduced value of shear strength of 800 lb/sq.ft for the calculation of the bearing capacity above the elevation 563, 1000 lb/sq.ft between elevations 560 and 563, and 1200 lb/sq.ft below the elevation 560.

E. CONCLUSIONS AND RECOMMENDATIONS:

1. South Abutment (Cont'd)

The net allowable bearing capacity of a typically 17 ft wide and 26 ft long footing will then be:

South Abutment

<u>Elevation</u>	<u>Net allowable bearing capacity.</u>
563 and above	1600 lb/sq. ft
560 to 563	2100 lb/sq. ft
Below 560	2500 lb/sq. ft

To the above values, an additional component, equal to the pressure of the final overburden over the footing may be added. Using a minimum value of unit weight of backfill of 100 lb/cu. ft, 14 feet of overburden would be required to raise the safe bearing capacity to 3000 lb/sq. ft for footings set at the elevation 563 or above. At lower elevations, the total bearing capacity would be correspondingly greater.

The proposed design pressures, as outlined in the letter of February 9th, 1962 from the Consulting Engineers, are therefore permissible for the new location of the south abutment.

E. CONCLUSIONS AND RECOMMENDATIONS:

1. South Abutment (Cont'd)

However, in view of the fact that the clay till stratum in test hole 7 was found to commence only at the elevation 566.5, we consider that the footing will have to be placed at a depth exceeding the elevation 566, considered in the above-mentioned letter. In order to limit the settlement to a tolerable amount, within the previously indicated range, the footings should be placed not higher than at the elevation 564.

2. North Abutment

According to the information included in the letter from the Consulting Engineers of February 9th, 1962, the north abutment at the centreline of the roadway will be approximately 25 ft north of the present north abutment. Results of test hole 8, located 22 ft north of the existing abutment face and in the line of the new roadway, can therefore be considered as representative of the soil conditions at the new north abutment.

The stratum of clayey till, on which the abutment will be founded, commenced at the elevation 563.1. No undisturbed samples of the clay were recovered above the elevation 555, but the undrained shear strength between the elevations 555 and 544 was found to be lower than expected, and ranged from 520 to 970 lb /sq.ft. However, in the same range of depth, standard penetration test results were high^{er} than would have been expected from results of previous test holes, and ranged from 12 to 15 blows/ft.

E. CONCLUSIONS AND RECOMMENDATIONS:

2. North Abutment - Cont'd

As was discussed in Chapter D, the phenomenon of increased penetration resistance and decreased shear strength may be caused by an excess pore water pressure, set up by the embankment fill, recently placed just north of the test hole. However, we consider it prudent to estimate the allowable bearing capacity of the north abutment on the basis of a reduced shear strength of 750 lb /sq.ft.

For a 17 ft wide and 26 ft long footing, the net allowable bearing capacity is:-

<u>North Abutment</u>	
<u>Elevation</u>	<u>Net allowable bearing capacity</u>
562 or below	1500 lb /sq.ft

To the above value, an additional component, equal to the pressure of the final overburden over the footing may be added, as indicated for the case of the south abutment.

In order to obtain a total bearing capacity of 3000 lb /sq.ft, and assuming unit weight of fill as 100 lb /cu.ft, the footing, placed at the elevation 562 would thus have to be covered with fill up to the elevation 577. Alternatively, the level of the footing could be lowered.

E. CONCLUSIONS AND RECOMMENDATIONS:

3. Bridge Piers

No additional test holes were carried out on the water in the final positions of the piers, and no new information on the shear strength of clay till under the piers is therefore available.

A safe design load of 3000 lb /sq.ft at the elevation 557.0 or below was recommended in the memorandum from the Department of Highways, which is mentioned in the letter from the Consulting Engineers to this company of February 9th, 1962. This design load would require a shear strength of clay till of 600 to 800 lb /sq.ft, depending on the overburden effect, which in turn depends on the assumptions made regarding the possible depth of scour.

As the average shear strength of the clay was in no case lower than the above requirement, we consider the design pressure of 3000 lb /sq. ft at the elevation of piers of 557 entirely safe, and possibly somewhat conservative.

E. M. Peto Associates Ltd.,

C. F. Freeman

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

RK/sb/ap

Job Number 6216

March, 1962.

APPENDIX "A"

STANDARD CONE PROBE RESULTS

STANDARD CONE PROBE RESULTS

Job Number 6216

<u>Depth</u> <u>from to</u>	<u>No. of blows</u>
0 - 1	15
1 - 2	7
2 - 3	6
3 - 4	4
4 - 5	3
5 - 6	6
6 - 7	6
7 - 8	9
8 - 9	9
9 - 10	10
10 - 11	12
11 - 12	14
12 - 13	13
13 - 14	16
14 - 15	17
15 - 16	17
16 - 17	19
17 - 18	21
18 - 19	26
19 - 20	31
20 - 21	31
21 - 22	40
22 - 23	46
23 - 24	45
24 - 25	50
25 - 26	54
26 - 27	55
27 - 28	61
28 - 29	60
29 - 30	65
30 - 31	61
31 - 32	66
32 - 33	74
33 - 34	85
34 - 35	97

APPENDIX "B"

LABORATORY TEST RESULTS

E. M. PETO ASSOCIATES LTD.
UNCONFINED COMPRESSION TEST DATA SHEET

Job Number 6216

Corehole Number	Sample Number	Depth Feet	Nat. M. C.	Wet Density p. c. f.	Dry Density p. c. f.	Void Ratio, e	u/c shear strength p. s. f.
7	8	20'-21'6"	23.4	130	105	0.62	1150
7	10	25'6"-26'6"	23.7	130	105	0.63	1370
7	12	30'6"-31'6"	21.6	131	107	0.60	865
7	14	35'6"-36'6"	21.6	132	109	0.58	1270
7	6	16'-16'6"	21.9	129	104	0.62	1260
7	7	19'6"-20'0"	22.0	127	104	0.64	1230
7	9A	23'6"-24'0"	21.9	128	105	0.63	1620
7	9B	24'-24'6"	24.3	128	103	0.66	1390
7	11	29'-29'6"	22.9	126	102	0.67	1620
7	13A	33'6"-34'0"	22.0	127	104	0.65	1540
7	13B	34'-34'6"	18.8	126	106	0.61	1330
8	8	24'6"-25'6"	24.0	129	104	0.65	575
8	7B	23'-23'6"	23.3	127	103	0.62	780
8	9A	28'6"-29'0"	24.3	123	99	0.73	520
8	9B	29'-29'6"	22.3	127	104	0.65	860

Toronto 19, Ontario

LIQUID LIMIT TEST

FLOW LINE CHARTS

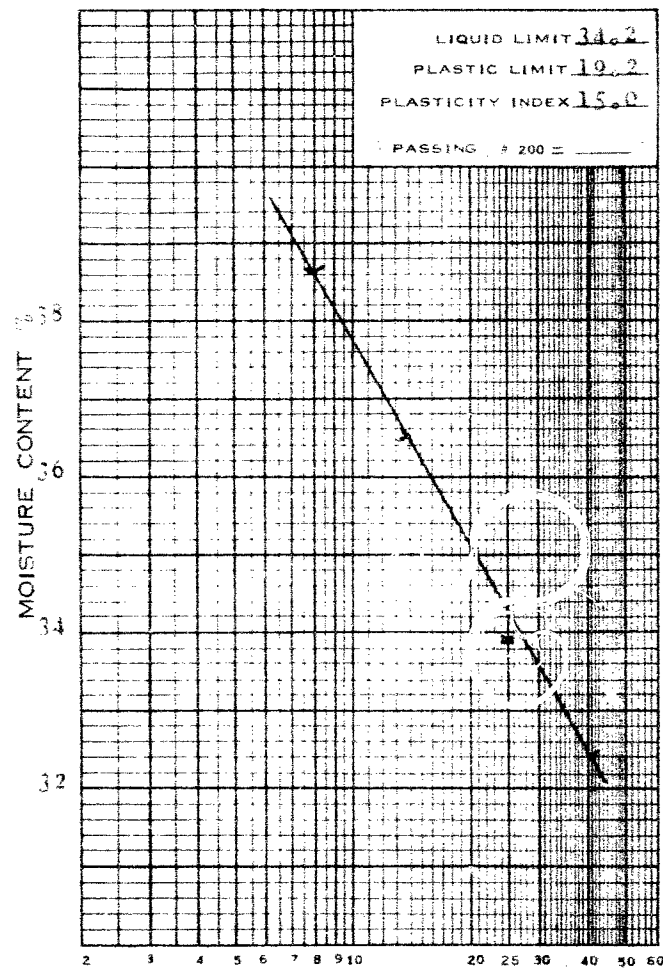
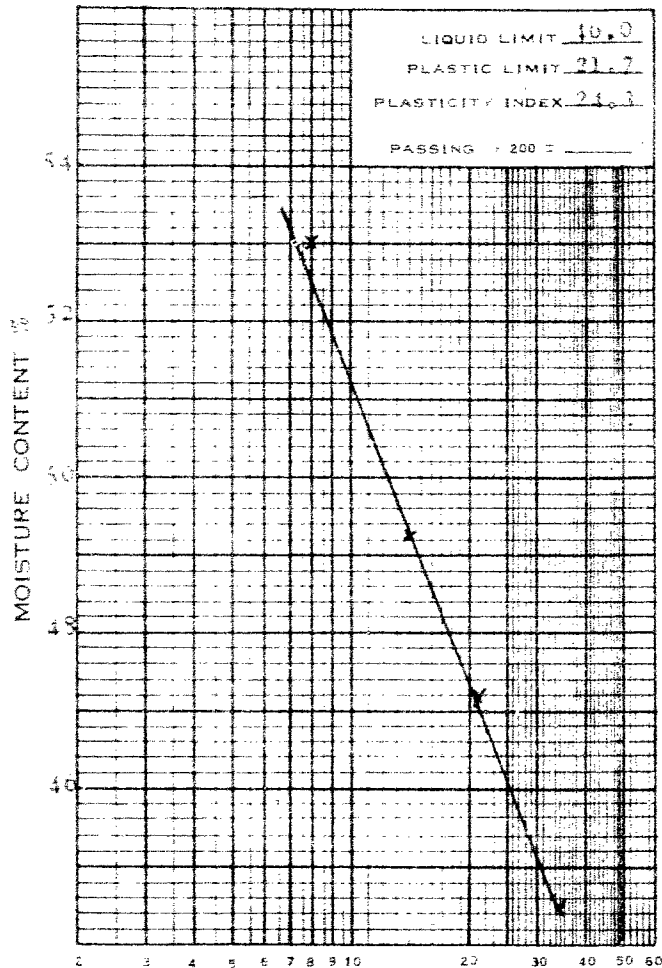
Job No. 0216 Project Sydenham Bridge Additional Testholes

SAMPLE FROM Bit #7 Sample #1

SAMPLE FROM Bit #8 Sample #12

DEPTH 10'6" - 11'0"

DEPTH 35'0" - 36'0"



NO. OF BLOWS (LOG SCALE)

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO





BOREHOLE LOG

Job Name Sydenham Bridge (C-7-4)
Additional Test Holes
 Client County of Lambton
 Elevation Geodetic

Job No. 6216
 Casing 4" & BX
 Compiled By R. K.

Borehole No. 7
 Boring Date January 30th, 1962
 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	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Value (unadjusted)	WATER LEVELS & REMARKS
			575.02						
			0'0"						
Organic topsoil	Brown	Frozen	1'1"						
Silty and sandy clay with organic matter	Grey	Very soft	5'0"		1	S.S.	5	106.5	
As above, with pebbles	Grey	Very soft	5'0"		2	S.S.	1	32.0	
As above, less organic matter	Grey	Very soft	8'9"		3	S.S.	2	27.4	
Sand with gravel			9'6"						Water from sand rose to 2'8" in 1 minute.
Silty clay a few small pebbles	Brown grey	Soft to Firm			4	S.S.	7	27.2	
Layered silty clay	Grey	Soft	12'8"		5	S.S.	5	25.5	
			15'0"						
As above, with pebbles	Grey	Soft to firm			6	2"S. L. tapped		24.1	Pebbles up to 1/2"
			20'0"		7	2"S. L. tapped		24.7	
Silty clay with pebbles	Grey	Firm			8	S.S.	13	24.5	
			25'0"		9A	2"S. L. tapped		22.0	
As above	Grey	Firm			10	S.S.	12	25.7	
			30'0"		11	2"S. L. tapped		21.4	
As above	Grey	Firm			12	S.S.	12	20.0	
			35'0"		13A	2"S. L. tapped		21.0	
As above	Grey	Firm			13B	S.S.	16	12.1	
			36'6"						Test hole Terminated at 36'6".





e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Sydenham Bridge (C-7-4) Job No. 6216 Borehole No. 8
 Client County of Lambton Casing 4" & BX Boring Date January 31st, 1962
 Elevation Geodetic Compiled By R. K. Checked By S. B.

SAMPLE CONDITION
















-  UNDISTURBED
 FAIR
 DISTURBED
 LOST

SAMPLE TYPE

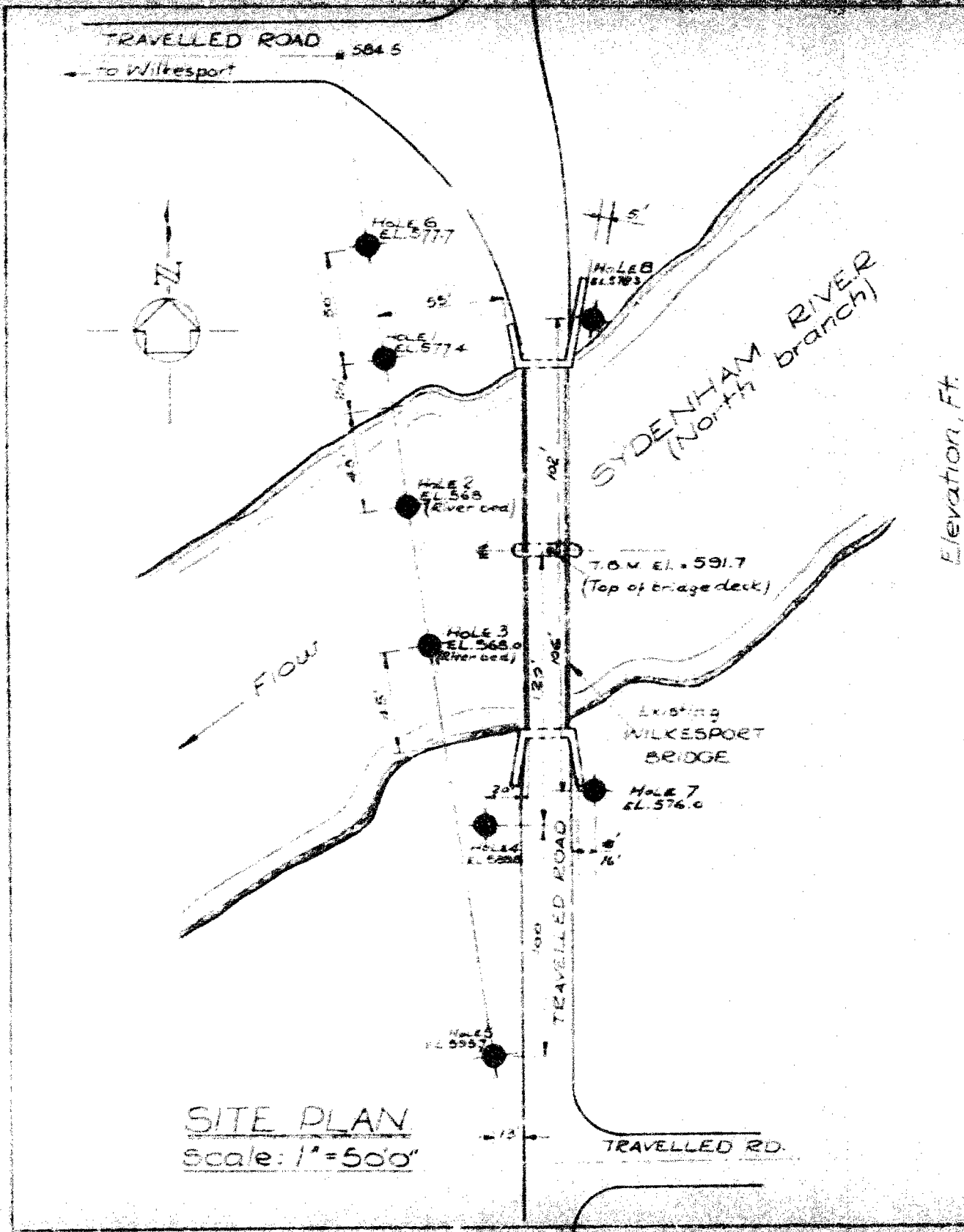
- A.S. AUGER SAMPLE
 C.S. CASING SAMPLE
 S.S. 2" STANDARD SPLIT TUBE SAMPLE
 S.L. SPLIT BARREL WITH LINERS
 S.T. THIN-WALLED SHELBY TUBE SAMPLE
 W.S. WASH SAMPLE
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ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
 C. SOIL SHEAR STRENGTH LBS/SQ.FT.
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 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 Condition	Sample Type	No. of Blows per Ft.	Time and Method	WATER LEVELS & REMARKS
			573.30'						
			0'0"						
Silty clay with organic matter	Brown	Firm		1		S.S.	9	33.8	
As above	Grey	Firm	5'8"	2		S.S.	6	46.0	
(decomposed wood log)	Black			3		S.S.	4	66.5	
			10'2"						
Sand and fine gravel with clay and organic matter	Dark grey	Very soft		4		S.S.	3	127.0	
			12'6"						
			12'6"	5		S.S.	2	31.0	
Silty clay with organic matter	Dark grey	Very soft	14'3"						
Sand with Gravel			15'2"						Water bearing seam
Silty clay with pebbles.	Grey	Soft		6		S.S.	5	26.6	Water came up to 4'2" Pebbles up to 1/2"
						2" S. L.			
			20'0"						
As above	Grey	Soft to firm		7A		2" S. L. tapped	25.7		
				7B					
			25'0"	8		S.S.	13	22.9	
As above.	Grey	Soft to firm		9A		2" S. L. tapped	25.5		
			30'0"	9B					
Silty clay with pebbles	Grey	Soft to Firm		10		S.S.	12	24.1	
			35'0"	11A		2" S. L. tapped	16.6		
				11B					
As above	Grey	Firm	36'6"	12		S.S.	15	21.8	

Test Hole Terminated at 36 ft. 6 ins.



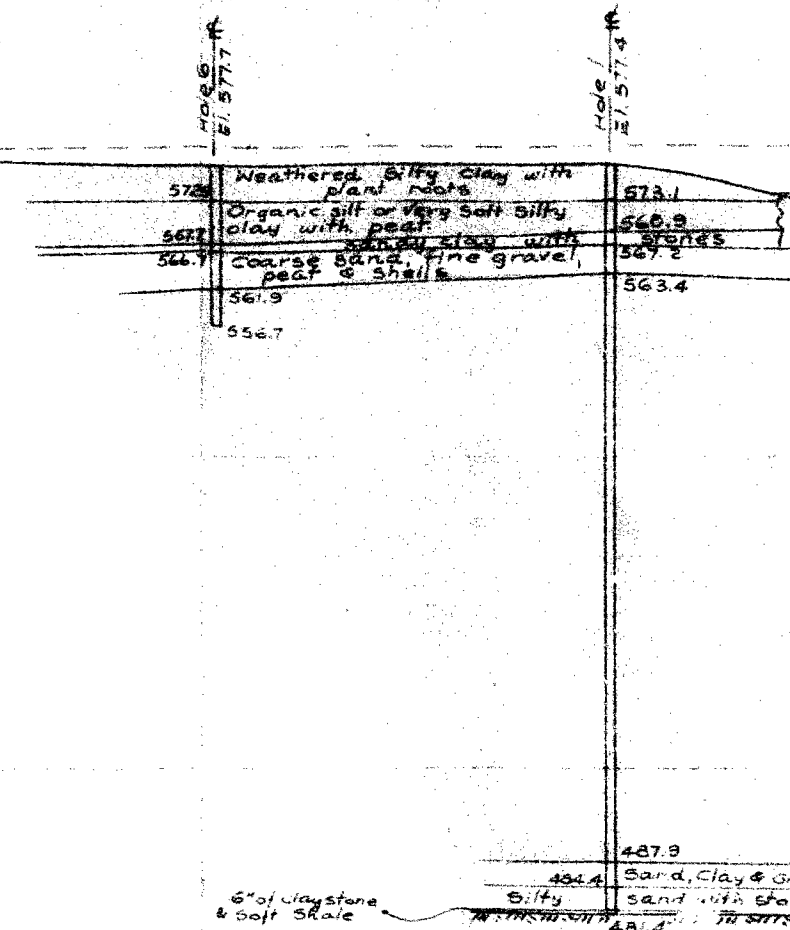
580

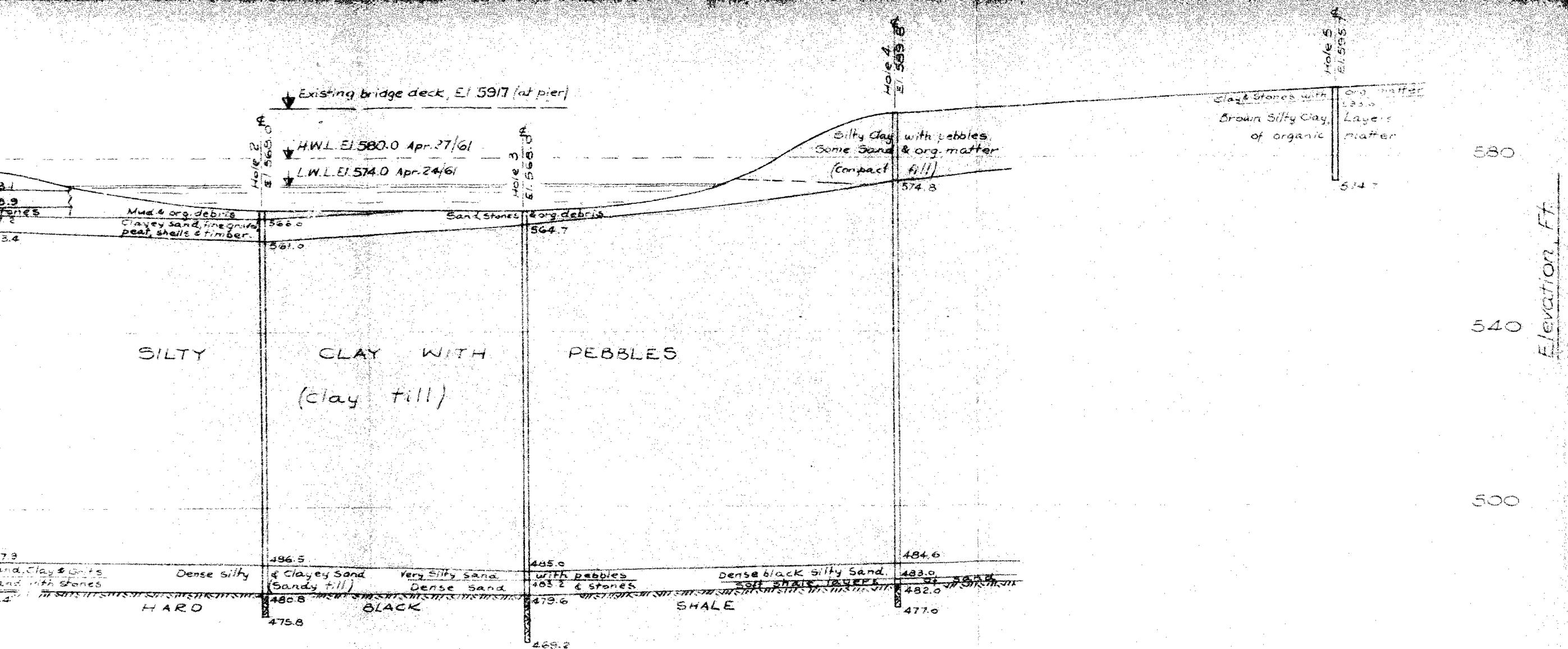
540

500

NOTES:-

- 1) See borehole logs for detailed description of soil and bedrock.
- 2) See appendix "B" for std. penetration test results plotted against elevation.
- 3) The soil stratigraphy between boreholes has been estimated and may differ from that shown.



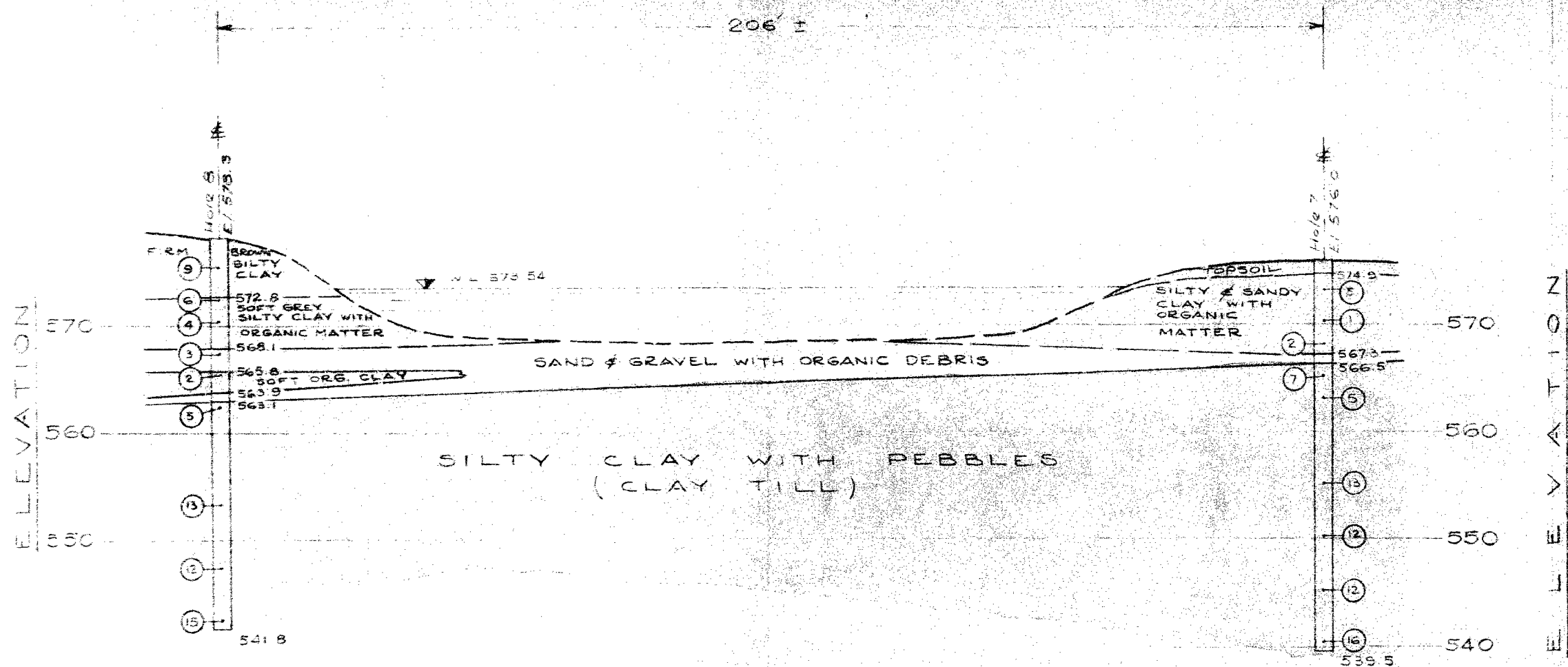


SOIL PROFILE THROUGH HOLES 6, 1, 2, 3, 4 & 5.

Scale: 1" = 20' 0" (Natural)



Job # 6140
empeto associates ltd
June 1961 g.t



SECTION ON HOLES 8 & 7

PROFILES:

SCALE: HOR.: 1"=20' 0"
VERT.: 1"=10' 0"

LEGEND:

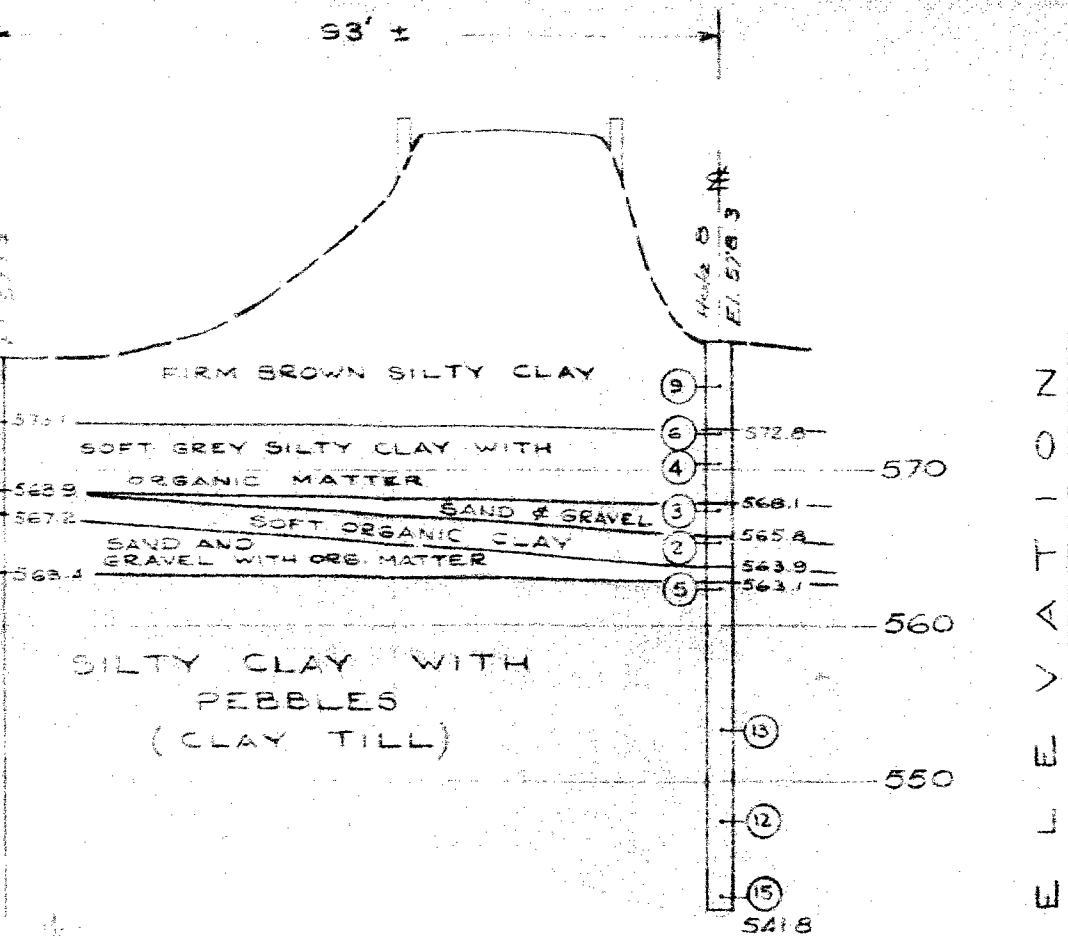
⑨ — BLOWS/FOOT (S.P.T.)

NOTE:

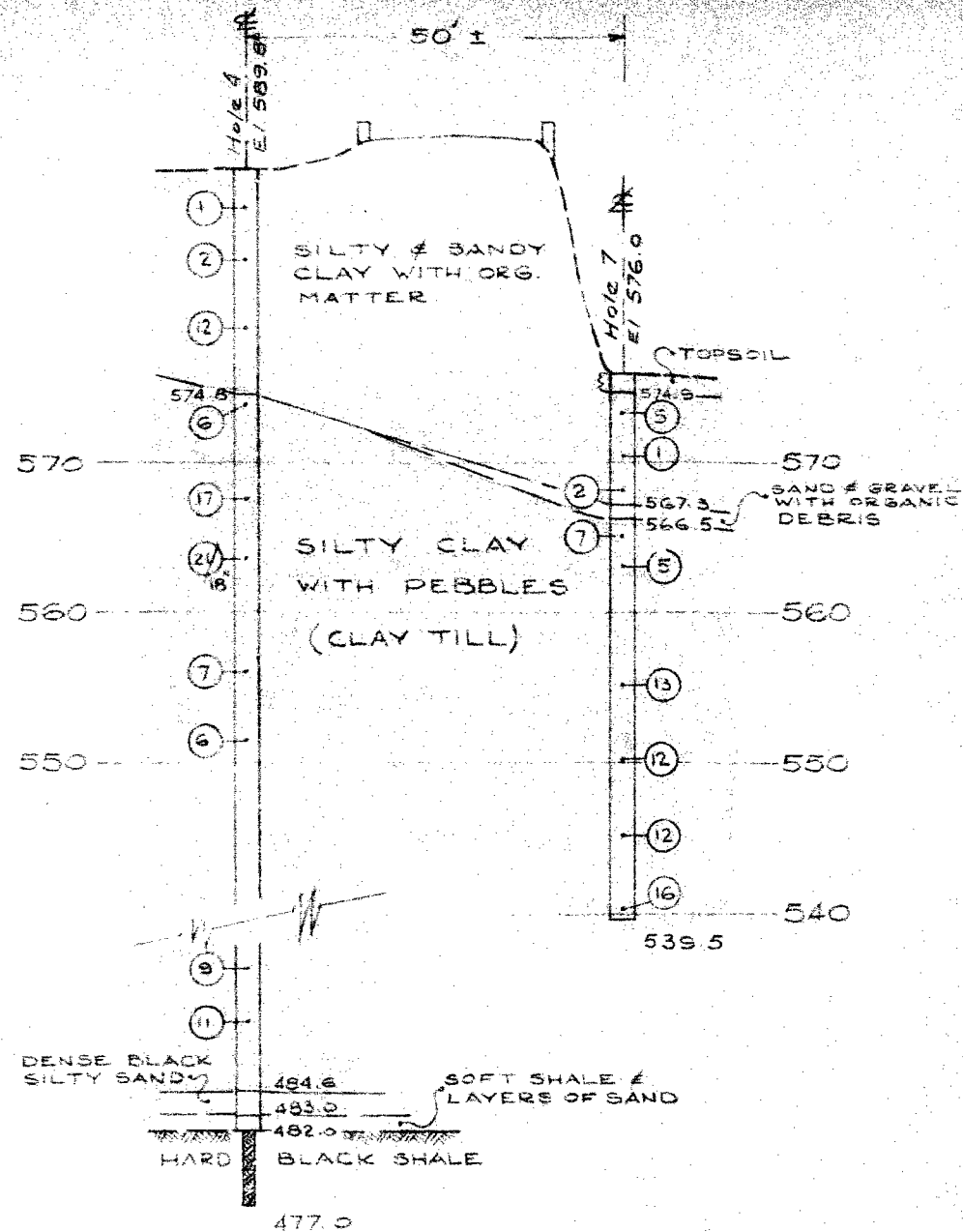
SEE BOREHOLE LOGS FOR COMPLETE SOIL DETAILS.

6" OF CLAYSTONE
& SOFT SHALE

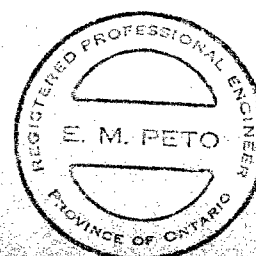
487.9
484.4
481.4
HARD BLACK SHALE



SECTION ON HOLES 4 & 8



SECTION ON HOLES 4 & 7



JOB No. 6216
e.m.peto associates ltd.
FEBRUARY, 1962 W.S.