

#59-F-201C

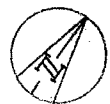
W.P. #56-59

Hwy #4018

COUNTY RD.

HARWICH8

RALEIGH E.



T.B.M. = 597.99
CUT CROSS ON WALL OF
CONCRETE CULVERT 20' LEFT
OF STATION 5+75
COUNTY ROAD CHAINAGE

PAVED COUNTY ROAD

PAVED COUNTY ROAD

TO KENT CENTRE

DUTCH CONE 4
ELEV. 596.7

BOREHOLE 5
ELEV. 595.6

BOREHOLE 3
ELEV. 596.4

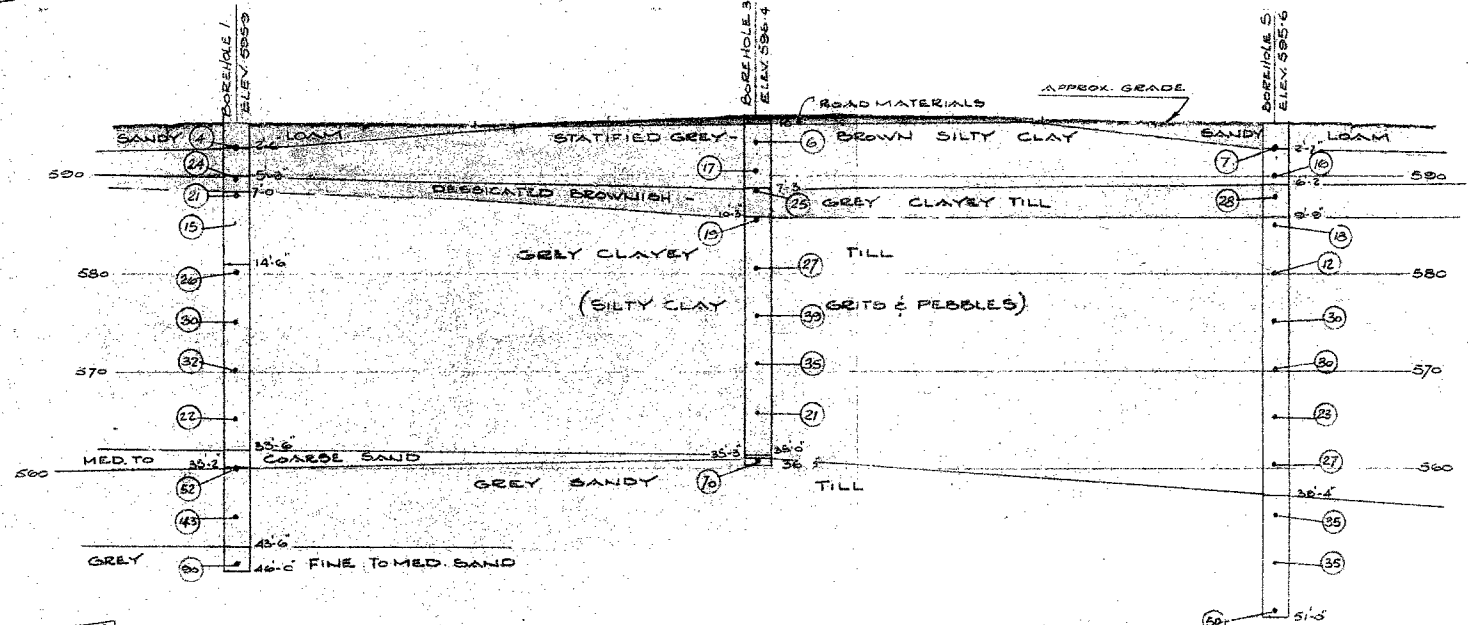
BOREHOLE 1
ELEV. 595.9

DUTCH CONE 2
ELEV. 595.6

CON. CULV.

SITE PLAN

SCALE 100' = 1"



LEGEND

- BOREHOLE
- BLOWS / FOOT

NOTE: SEE ATTACHED BOREHOLE LOGS FOR COMPLETE DATA ON SOILS



e.m. peto & associates ltd.	
SOIL SITE INVESTIGATION AT HIGHWAY 401 UNDERPASS TOWNSHIP OF HARWICH & RALEIGH FOR DEPT. OF HIGHWAYS OF ONTARIO	
OUR JOB No. 59252	DATE 17 DEC/50
CLIENTS PLAN No. P-3533-1	PER. G.J.V.

Mr. A. M. Toye,
Bridge Engineer.

May 3, 1960

REVIEW OF SUBSOIL CONDITIONS

Materials & Research Section.

Attention: Mr. Bruce Davis.

Re: W.P. 81-59: Raleigh Twp. Bridge No. 13
W.P. 86-59: County Road Crossing near Ridgetown.
W.P. 56-59: County Road Crossing
-- Differential Settlements --

As requested, we have reviewed the subsoil conditions at the above noted structure sites with respect to differential settlements between the piers and the abutments. Our findings are as follows:-

1. Raleigh Twp. Bridge No. 13, Hwy. 401 - W.P. 81-59:

At this site, according to the subsoil data reported by Dominion Soil Investigation, Ltd., a layer of medium compressible silty clay, approximately 18 ft. in thickness, was encountered at a depth of approx. 32 ft. below the existing ground surface. The consolidation characteristics of this clay layer have not been investigated by the soil consultants but, according to our experience in the area, the coefficient of volume compressibility of 0.007 ft.²/ton has been found to be a reasonable value for computations of consolidation settlements in the clay. Results of settlement calculations are as follows:-

Settlement of abutment due to 3 t.s.f. footing pressure + fill = 1.7".

Settlement of pier adjacent to abutment due to 3 t.s.f. footing pressure + fill = 1".

Settlement of centre pier due to 3 t.s.f. footing pressure = 0.5".

We would like to point out that the above computed settlements are on a long-term basis and we conclude that for design purposes, the maximum differential settlement will be of the order of 1/2" within the lifetime of the structure.

cont'd. /2 ...

2. County Road Crossing near Ridgetown - W.P. 86-59:

The foundation investigation at this structure site was carried out by William A. Trow & Associates. The site, in general, is underlain by a deep deposit of over-consolidated silty clay followed by bedrock. If a single-span design is used, in view of the fact that the subsoil conditions are relatively uniform, little differential settlement of any consequence, need be anticipated.

If it is proposed to use a design incorporating a centre pier, some long-term differential settlement between the abutments and the pier can be expected. An ultimate movement of the order of 2 inches has been estimated by the Consultants. It appears that for an over-consolidated clay, this magnitude of differential settlement has been over-estimated. In view of the slow rate of consolidation, as expected of clays, we are of the opinion that for practical purposes, a differential settlement of the order of 1 inch can be used for design within the lifetime of the structure.

3. County Road Crossing - W.P. 56-59:

The foundation investigation at this structure site was carried out by E. M. Peto Associates. The subsoil consists of a deep deposit of heavily over-consolidated clay. Little differential settlements of any consequence need be anticipated of a single-span or a multi-span structure.

If we can be of further assistance in connection with these projects, please contact our Office.

L. G. Soderma,
PRINCIPAL SOILS & FOUNDATIONS ENGR.
Per:

AKL/MdeF
cc: S. McCombie
Foundations Office
Gen. Files.

AKGh
(A. K. Loh,
PROJECT FOUNDATION ENGR.)

Re: Proposed Crossing of Gravel Road
Revised Line between Lots 6 & 7,
Con. VI and Hwy. 401 - Raleigh Twp.,
Kent County - District 1
W.P. 87-59 - 1

CHECKING OF PRELIMINARY PLANS

Preliminary plans seem to be in agreement with the suggestions contained in the covering letter of the Soil Investigation Report (Dominion Soil Investigation, Ltd.).

Foundation elevation is 579.0'. It is not visible from the plan what bearing capacity has been chosen. In the covering letter, it was suggested to use 3 T/sq.ft.

Remark:

The foundation overburden will be approx. 9'.
On account of this, the allowable bearing capacity could probably be raised to 4 T/sq.ft.

AS/MdeF

A. Stermac
A. Stermac,
FOUNDATIONS OFFICE ENGINEER

April 11, 1960.

Mr. A. M. Towe,
Bridge Engineer.
Materials & Research Section.

March 3, 1960.

FOUNDATION REPORT - by
Dominion Soil Investigation,
Limited.

Attention: Mr. S. McCombie.

Re: Proposed Crossing of Gravel Road
Revised Line between Lots 6 & 7,
Con. VI and Hwy. 401 - Raleigh Twp.,
Kent County - Dist. 1 - W.P. 81-59.

The detailed foundation report prepared by Dominion Soil Investigation, Ltd., for the proposed structure at the above location, has been reviewed by the Foundation Section. Comments arising from the review of this report, are as follows:-

1. The subsoil at the above site is generally a clay till.
2. The proposed structure may be supported by spread footings founded at elevation 579.0' or lower. Between elevations 579.0' and 565.0', spread footings may be designed for an allowable bearing pressure of 3 T/ft.². The recommended allowable bearing pressure has been reduced from 4 T/ft.² to 3 T/ft.², after considering total settlement, and possible softening of the bearing surface of the till before placing the concrete.
3. If the footings are not placed immediately after the completion of excavation, consideration should be given to the placing of a 6" concrete working mat at the bottom of the excavation. This working mat will prevent softening of the bearing material.

cont'd. /2 ...

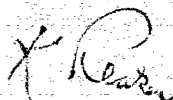
Comments: (cont'd.) ...

4. No problems associated with seepage water, or embankment stability, are anticipated.

If further queries arise regarding this report, please contact the Foundation Section.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGR.

per:



(K. Peaker,
FOUNDATION FIELD SUPERVISING ENGR.)

KP/MdeP
Attach.

cc: Messrs. A. M. Foye (2)
H. A. Tregaskes
D. G. Ramsay
A. Gater
G. U. Howell
J. Roy
A. Watt

Foundations Office ✓
Gen. Files.

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section.

January 8, 1960.

FOUNDATION INVESTIGATION - by
E.M. Peto & Associates, Ltd.

Attention: Mr. S. McCombie.

Re: Proposed Underpass - Hwy. 401 & County Rd.,
Twp. of Harwich and Saleigh East,
W.P. 56-59 - District #1, Chatham, Ontario.

The detailed foundation investigation carried out at the above site by E.M. Peto & Associates, and accompanying this memo, has been reviewed by the Foundation Section. We are in general agreement with the conclusions reached in this report and these conclusions are summarized below for your convenience:-

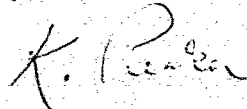
1. The main soil types encountered at this site consist of a stiff to hard upper desiccated layer of silty clay underlain by a stiff layer of silty clay containing some sand and gravel.
2. Spread footings may be used to support the proposed structure. These footings may be placed at elevation 590' or lower, and may be designed for a bearing pressure of 3 tons/ft.² provided the footing width ranges between 5 - 10 feet.
3. No problems associated with approach fill stability are expected, and ordinary design procedures using a 2:1 side slope may be used.
4. Water entering the excavations for footings will be small, and will be mainly due to surface run-off. All seepage water should be easily controlled with low-capacity pumps.

cont'd. /2 ...

If further queries should arise regarding the conclusions of this report, please contact the Foundation Section.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGR.

per:



(K. Peaker,
FOUNDATION FIELD SUPERVISING ENGR.)

KP/Maef
Encl.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Ramsay
A. Cater
G. U. Howell
J. Roy
A. Watt

Foundation Section ✓
Gen. Files.

23-62-43-L

E. M. PETO ASSOCIATES LTD.,

Job No. 59252

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

December 18th, 1959.

59-F-201C

Supplementary Copy

Department of Highways of Ontario,
Soil and Foundation Engineering Branch,
Parliament Buildings,
Toronto 2, Ontario.

Attention: Mr. L. Soderman, P. Eng.

Re: Soil Site Investigation,
Proposed Underpass,
County Road, Chatham to Charing Cross,
Highway 401 - W. P. 56-59 - District #1

Dear Sirs:

We have pleasure in submitting herewith ten (10) copies of our report for this project.

In the following report we have described the soil conditions encountered in some detail and we have discussed the engineering aspects arising from the soil conditions.

For your convenience we summarize our findings and recommendations.

1. The soil conditions as encountered at the above site are uniform with nearly horizontal stratification planes between individual soil types.
2. The proposed structure may be founded on the desiccated brownish-grey clayey fill layer. The allowable bearing values are given on page 11 of the report.

3. No difficulty is foreseen in placing the embankment, provided the soils containing organic matter are replaced. The subsoil conditions permit the construction of an embankment with side slopes of 1 in 1. The estimated Factor of Safety of an embankment of 30 ft. in height and side slopes of 1 in 1 is in excess of 4.0.
4. No difficulty will be encountered in excavations to the required foundation depth.
5. There is no water problem connected with the excavation and the placing of structure at the above site, except possibly from small amounts of seepage water.

We believe this report to be complete, and to contain all the information you require. However, should you require some additional information in connection with this investigation we shall be pleased to be of further service.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,

BL:sb

E. M. Peto, P. Eng.

THE DEPARTMENT OF HIGHWAYS OF ONTARIO

SOILS REPORT

for

HIGHWAY 401 UNDERPASS

W. P. 56 - 59

TOWNSHIP HARWICH AND RALEIGH EAST

E. M. PETO ASSOCIATES LTD.

1287 Caledonia Road,
Toronto 19, Ontario.

CONTENTS OF REPORT

Page No.

Covering Letter

REPORT

A. INTRODUCTION	1
B. PROGRAMME OF WORK	1
C. GENERAL INFORMATION	2
D. SITE AND GEOLOGY	3
E. SOIL CONDITIONS	4
F. CONCLUSIONS AND RECOMMENDATIONS	11

APPENDIX I - FIELD TEST RESULTS

APPENDIX II - LABORATORY TEST RESULTS

APPENDIX III - METHOD OF OPERATION

BOREHOLE LOGS

SITE PLAN

A. INTRODUCTION:

We were authorized by a letter from the Department of Highways of Ontario dated December 8th, 1959 and signed by Mr. A. Rutka, Materials and Research Engineer, to carry out a soil investigation at the proposed location of the underpass of Highway #401 and the county road (Highway #89) in the Township of Harwich and Raleigh East. At the same time we were issued with the D.H.O. plan No. 3533 and profile F-3533-1 and 3533-2, showing the location of the proposed underpass and the profile.

We were required to:

- a) determine the existing soil conditions and the soil stratigraphy at this site,
- b) determine any pertinent ground water information.
- c) obtain the shear strength and the allowable bearing values of the subsoil which would provide a basis for the design computations.
- d) advise of any possible construction problems at this site.

B. PROGRAMME OF WORK:

- December 10th, 1959 - Test holes located and the elevations at the existing grade obtained by our Field Supervisor. Field crew moved onto site. Test hole 1 commenced.
- December 11th, 1959 - Test hole 1 completed.
- December 12th, 1959 - Test hole 3 completed.
- December 13th, 1959 - Test hole 5 completed.

B. PROGRAMME OF WORK (Cont'd)

December 14th, 1959 - Dynamic cone penetration test carried out at holes 2 and 4. The field crew moved off the site.

C. GENERAL INFORMATION:

a) The test holes were put down in accordance with our standard procedure as outlined in Appendix III under "Method of Operation".

b) The details of the test holes (i. e. depth, diameter of casing and the ground elevations) were as follows:

<u>Test Hole No.</u>	<u>Elevation at the existing surface</u>	<u>Depth</u>	<u>Diameter of Casing</u>
1	595.90	46'0"	BX (2-1/2")
3	596.36	36'0"	BX (2-1/2")
5	595.62	51'0"	BX (2-1/2")
<u>Probe Holes</u>			
2	595.61	20'0"	
4	596.05	22'0"	

c) The elevations as given above and in the following report were obtained using the D. H. O. Bench Mark which was a cut cross on the wall of a concrete culvert 20 ft. left of sta. 5 + 48 County Road Chainage having elevation 597.99.

C. GENERAL INFORMATION: (Cont'd)

- d) The detailed soil description, the results of standard penetration tests and the natural moisture contents are given in the individual borehole logs.
- e) The location of the test holes (as suggested by the client) and the assumed soil profile is shown on the attached site plan.
- f) The graphical representation of the results of the standard penetration tests, the dynamic cone penetration tests, (test hole and probe holes 2 and 4) and the natural moisture contents, versus elevation are given in Appendix I under "Field Test Results".
- g) The results of the laboratory shear strength tests (Unconfined compressive tests) are shown in tabular form in Appendix II under "Laboratory Test Results".

D. SITE AND GEOLOGY:

The site proposed for the Highway #401 and the county road underpass is situated some 3.4 miles South-East of Chatham. The topography is very flat with cultivated fields adjacent at both sides to the county road.

Geologically, the drift deposits at the above site are underlain by soft blue and grey shale and limestone of Hamilton formation of the Devonian System. Glacial Lake Whittlesey deeply covered the present Essex, Kent and Lambton counties; and technically, most of the soil at depth is a clay till or alluvial clay, i.e. the water laid sediments.

E. SOIL CONDITIONS:

During the present investigation the following main soil types were encountered.

- a) Topsoil and sandy loam
- b) Stratified brown and grey silty clay
- c) Clayey till (silty clay, grits and pebbles).
- d) Grey fine to medium sand
- e) Sandy till.

The following is a detailed description of each type of soil.

a) Topsoil and Sandy Loam

This uppermost layer was encountered in test holes 1 and 5. In test hole 3 underlying the asphalt surfacing and the concrete base of the county road, a 3 inch thick layer of organic topsoil was found.

In test hole 1 the sandy loam layer with traces of organic matter and mixed brown in colour was 2'6" deep. In test hole 5 it was encountered to a depth of 2'7" below the existing grade. Generally the sandy loam layer was found to be moist to quite moist and of loose density.

b) Stratified Brown and Grey Silty Clay

Underlying the sandy loam, a stratum of stratified brown and grey silty clay was found. A certain amount of desiccation of this layer was evident, as the colour in some instances was rusty-brown. The lower boundary of this stratum was encountered as follows:

E. SOIL CONDITIONS:

b) Stratified brown and Grey Silty Clay (Cont'd)

Test Hole No.

- 1 - at 5'2" below grade
- 3 - at 7'3" below grade
- 5 - at 6'2" below grade

As may be seen, the lower boundary is nearly horizontal.

In test hole 3 traces of organic matter were met in the layer of stratified brown and grey silty clay. Pockets of wet sandy gravel were found in test hole 5. The natural moisture content of brown and grey silty clay was, in every instance, wetter than its Plastic Limit, and varied between 22.6% (test hole 3) and 28.6% (test hole 5). As an average moisture content a value of 24.7% may be assumed.

The results of the standard penetration test indicated that the stratified silty clay layer was firm to very stiff with the number of blows per foot penetration varying between 4 (test hole 1, depth 2 - 3 ft.) and 17 (test hole 3, depth 5-6 ft.). It is evident that the density of the stratum increased with depth.

The mechanical analysis conducted on two samples (test holes 3 and 5 - sample 2) gave the following results.

<u>Test Hole No.</u>	<u>Depth Ft.</u>	<u>Elevation</u>	<u>Gravel %</u>	<u>Sand %</u>	<u>Silt %</u>	<u>Clay %</u>
3	5'6"	590.9	-	14	51	35
5	5'6"	590.1	-	5	35	60

According to these results the stratified grey and brown silty clay is a clay to silty clay soil, with the percentage of clay particles varying between 35 and 60%.

E. SOIL CONDITIONS: (Cont'd)

c) Clayey Till

A transition zone between the overlying stratified silty clay and the lower-lying grey clayey till consisted of a stratum of brownish-grey clayey till. Although geologically it belongs to the grey clayey till layer the difference in colour is mainly due to the desiccation process of the upper portions of grey clayey till layer. The gradual change in colour from brownish-grey to grey did not permit a clear picture to be established of the lower boundary of this desiccated layer, although, as may be seen from the attached soil profile, an attempt was made to establish this lower boundary and it was estimated that the upper limit of grey clayey till stratum was at the following depths below the existing grade:

Test hole 1 - at about 7'0"

Test hole 3 - at 10'3"

Test hole 5 - at 9'9"

The desiccated brownish-grey till layer was found to be very stiff with "N" values ranging from 24 (test hole 1) to 28 (test hole 5) with the natural moisture content around 17%, i.e. more or less at the plastic limit of the material.

Below the desiccated layer there was a grey clayey till layer. The lower boundary of the grey clayey till was encountered at the following depths:

Test hole 1 - at 34'6" below grade

Test hole 3 - at 35'0" below grade

Test hole 5 - at 36'4" below grade

E. SOIL CONDITIONS:

c) Clayey Till (Cont'd)

According to these results a slight dip of the lower boundary of grey clayey till layer exists from the area of test hole 1 towards test hole 5.

The results of the standard penetration tests indicated that the density of grey clayey till layer increases with depth to an elevation of about 574 to 575 then decreases and at an elevation of about 565 nearly identical results were obtained in all the test holes. The "N" value at this point was 21 in test hole 3, 22 in test hole 1 and 23 in test hole 5.

From this depth there is a marked increase in density again, particularly in test holes 1 and 3. In test hole 5 the rate of increase in density with depth decreases.

The results of the Atterberg Limits indicated a decrease in the values of Liquid Limit, Plastic Limit and the Plasticity Index with depth. The following table gives the results of the Atterberg Limit tests.

<u>Test Hole</u> <u>No.</u>	<u>Depth</u>	<u>Elevation</u>	<u>In per cent</u>		
			<u>L. L.</u>	<u>P. L.</u>	<u>P. I.</u>
1	5 - 6	580.4	33.1	20.3	12.8
3	15 - 16	580.9	31.5	18.2	13.3
1	25 - 26	570.4	29.6	17.0	12.6
5	35 - 36	560.1	27.8	18.5	9.3

E. SOIL CONDITIONS:

c) Clayey Till (Cont'd)

According to Casagrande's Classification System the clayey till is a C L. - C. I soil changing with depth to C. L. - soil. Therefore it is an inorganic silty clay to clayey silt soil with plasticities changing with depth from medium to slight.

The mechanical analysis gave the following results:

<u>Test Hole</u> <u>No.</u>	<u>Depth</u>	<u>Elevation</u>	<u>Per cent of</u>			
			<u>Gravel</u>	<u>Sand</u>	<u>Silt</u>	<u>Clay</u>
3	20-21	575.9	3	17	45	35
3	30-31	565.9	7	23	40	30

Analysing the data of the Atterberg Limits tests and the mechanical analysis it may be seen that the percent age of silt and clay particles decreases with depth, and this corresponds with the decreased plasticity of the soil as observed from the Atterberg Limit tests.

The textural classification of the grey clayey till is thus:

silty to sandy clay.

It was observed during the field work that the density of the clayey till was somewhat lower at a depth of 12 - 13 ft. below grade. In order to be able to obtain a minimum shear strength of the clayey till, which was thought to exist at this depth a number of unconfined compressive tests were conducted. The detailed results of these tests are given in Appendix II. It was found that the following minimum average values for the layer of grey clayey till may be assumed.

E. SOIL CONDITIONS:

c) Clayey Till (Cont'd)

Shear strength: 3920 p.s.f.

Wet density : 135.8 p.c.f.

Dry density : 116.1 p.c.f.

The average void ratio $e = 0.446$

Degree of saturation = 100%

d) Grey fine to medium sand

Located between the stratum of grey clayey till and grey sandy till in test holes 1 and 3 a layer of grey fine to medium sand was encountered. In test hole 1 it was located between 34'6" and 35'2" below grade and in test hole 3 between 35'0" and 35'3". No trace of this layer was found at this depth in test hole 5, thus it may be assumed that it terminated between test holes 3 and 5. This sand layer was found to be quite moist and is probably a water bearing seam.

Another grey fine to medium sand layer was established at 43'6" below grade in test hole 1. The presence of this layer was not observed in the remainder of the test holes, possibly due to the insufficient depth of the test holes. This lower fine to medium sand layer contained natural gas in small quantities, since, following a two-day period, no trace of gas was found to be present. Both layers of grey sand (the upper and the lower seam) were extremely dense.

E. SOIL CONDITIONS: (Cont'd)

e) Grey sandy till

The material underlying the grey sand seam at test holes 1 and 3 from a depth of 35'2" and 35'3" respectively, below grade, and in test hole 5 from a depth of 38'4" was classified as sandy till.

The results of the mechanical analysis confirmed the above description as texturally the material was found to be silty sand. The mechanical analysis gave thus the following result.

Test Hole No.	Depth	Elevation	PER CENT OF			
			Gravel	Sand	Silt	Clay
1	40 - 41	555.4	15	39	32	14
5	45 - 46	550.4	15	35	36	14

The Atterberg Limits were as follows: (test hole 5, depth 40 - 41 ft: elevation 555.1)

Liquid Limit 21.6%

Plastic Limit 14.2%

Plasticity Index 7.4%

According to Casagrande's Classification System it is an SF-CL soil, i.e. clayey sand or clayey silt with low plasticity.

The results of the standard penetration tests indicated that the sandy till is dense to extremely dense with the number of blows per foot of penetration increasing with depth.

The natural moisture content was slightly wetter than the Plastic Limit of the material.

F. CONCLUSIONS AND RECOMMENDATIONS:

1. The soil conditions as encountered at this site are uniform with the stratification planes nearly horizontal.
2. The proposed structure may be founded on a desiccated layer of brownish-grey clayey till. Placing the foundations on the stratified grey and brown silty clay is not recommended because the plasticity characteristics of this soil may lead to settlements in excess of permissible amounts.
3. The recommended allowable bearing values are as follows:

Foundation depth below grade	Foundation Width	Allowable bearing value in lbs./sq. ft.	
		Strip	Isolated
8	5	6400	7650
	10	5300	6700
	15	5300	6400
	20	5200	6200
10	5	6200	8100
	10	5800	7000
	15	5400	6600
	20	5300	6400
12	5	7150	8500
	10	6000	7150
	15	5600	6700
	20	5400	6600
14	5	7250	8700
	10	6200	7350
	15	5700	6900
	20	5500	6600

F. CONCLUSIONS AND RECOMMENDATIONS: (Cont'd)

4. The above allowable bearing values were calculated assuming that the shear strength of the soil is constant with depth and is equal to 2900 p.s.f. which as may be seen represents the assumed minimum shear strength for the grey clayey till layer, and that the allowable bearing value is given by:

$$q_a = \frac{N_c \times S_u}{3} \quad (\text{Assuming Factor of Safety of 3})$$

Where N_c is the parameter depending on the geometry of foundation and obtained according to Skempton's empirical equation

S_u = the undrained shear strength.

It may be noted that the effect of the overburden was neglected.

5. As the underlying soil is of stiff to very stiff density, the foundation design depends primarily on the consideration of the shear strength as the settlements will be restricted to very small amounts. The settlements will be less than the permissible amounts for the type of structure proposed (the limit of maximum total settlement assumed to be 1 inch).
6. Removal of all sandy loam and part of the stratified grey and brown silty clay containing organic matter is recommended before placing the embankment. Subsoil conditions are sufficiently good to allow an embankment with a side slope of 1 : 1 to be constructed. The calculated factor of safety of embankment of 30 feet in height and with slopes of 1 in 1 is in excess of 4.0. Some settlement of the embankment should be expected but this will have no serious influence on the proposed underpass.

F. CONCLUSIONS AND RECOMMENDATIONS: (Cont'd)

7. During the present investigation apart from a negligible amount of seepage water encountered in the sandy loam layer, there were indications of water-bearing characteristics in the grey sand layers. The probable ground water table may be situated some 30 - 35 feet below existing grade. The exact location of the ground water table was not established because of the short term observations.

Thus it may be concluded that the possible ground-water table will have no effect on the proposed structure.

8. No difficulties will be encountered in excavation to the required depths and placing of foundations. However, provision should be made for a limited amount of pumping, due to some seepage water.

E. M. PETO ASSOCIATES LTD.,

BL:sb

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

Job No. 59252

December 18th, 1959.

APPENDIX I

FIELD TEST RESULTS

DYNAMIC CONE PENETRATION TEST

Test Hole #2

Depth		No. of Driving Blows
From	To	
0	1	2
1	2	2
2	3	2
3	4	5
4	5	5
5	6	11
6	7	13
7	8	14
8	9	20
9	10	23
10	11	30
11	12	40
12	13	46
13	14	65
14	15	63
15	16	103
16	17	163
17	18	167
18	19	237
19	20	240

Test Hole #4

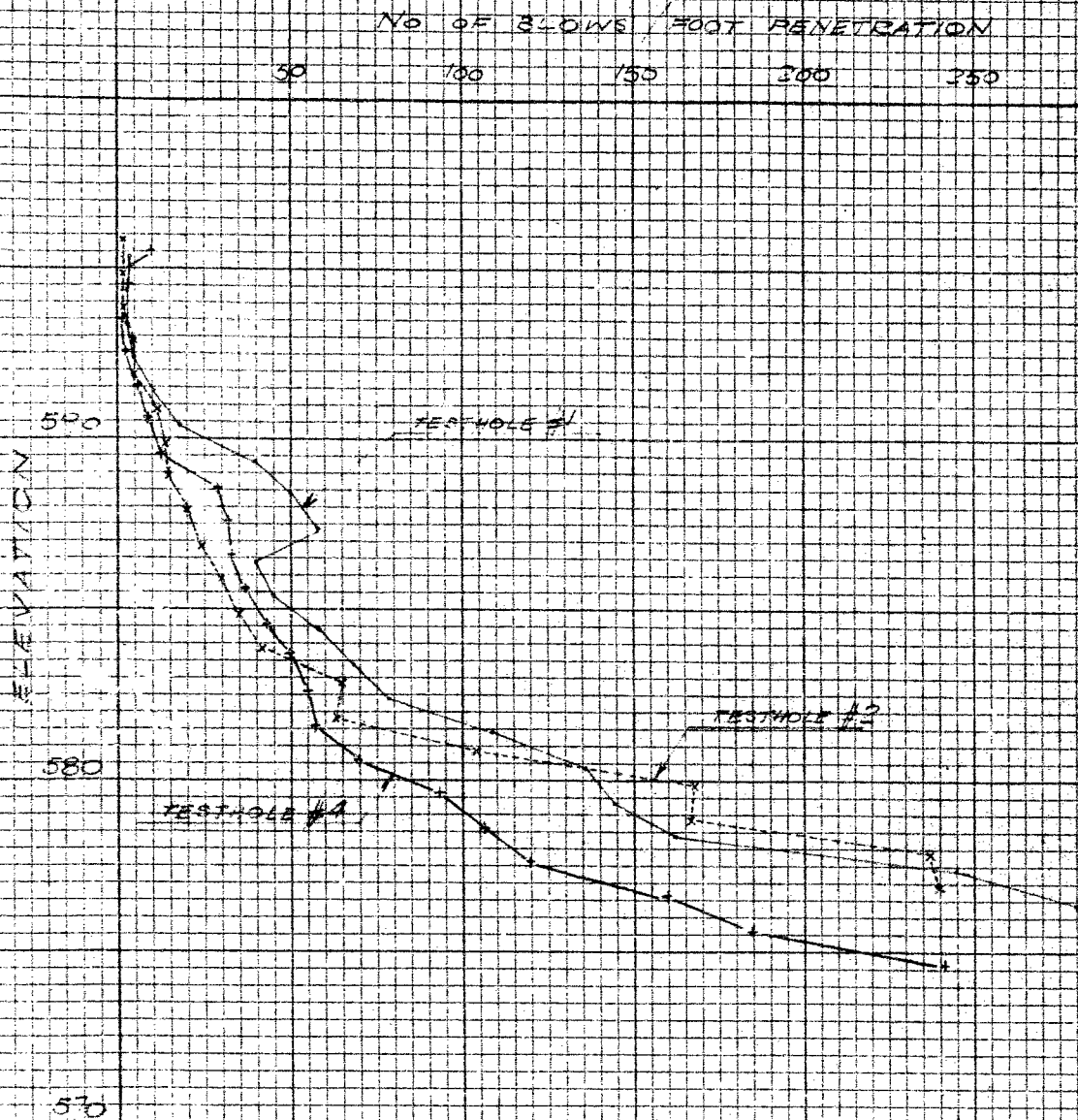
	Depth		No. of Driving Blows
	From	To	
0	1	10	
1	2	4	
2	3	2	
3	4	3	
4	5	6	
5	6	8	
6	7	13	
7	8	28	
8	9	32	
9	10	33	
10	11	37	
11	12	43	
12	13	50	
13	14	55	
	14	15	57
	15	16	70
	16	17	93
	17	18	107
	18	19	120
	19	20	160
	20	21	185
	21	22	242

DYNAMIC CONE PENETRATION TEST

Test Hole #1

Depth		No. of Driving Blows
From	To	
0	1	4
1	2	3
2	3	3
3	4	5
4	5	11
5	6	18
6	7	40
7	8	50
8	9	58
9	10	40
10	11	45
11	12	58
12	13	68
13	14	78
14	15	109
15	16	136
16	17	144
17	18	167
18	19	245
19	20	230
20	21	Further cone penetration test
21	22	Abandoned
22	23	

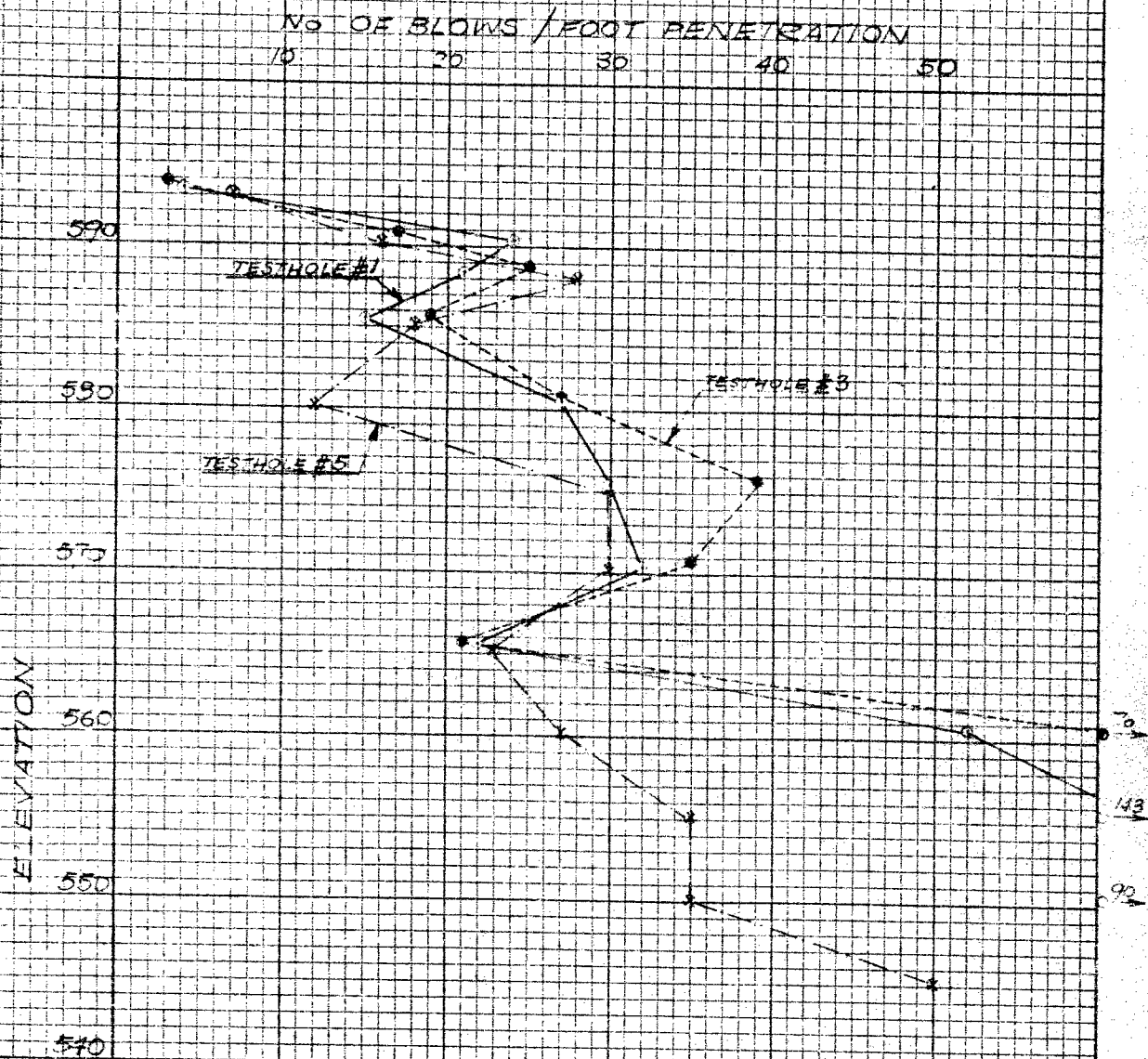
Results of the Dynamic Penetration Test versus Elevation



DB No 59253

E. M. PETO ASSOCIATES LTD

Results of Standard Penetration Tests versus Elevation.



JOB No 59052

E.M. PETO ASSOCIATES LTD

APPENDIX II

LABORATORY TEST RESULTS

Hole No.	Depth in Feet	Ground Level Feet	Elevation Feet.	UNCONFINED COMPRESSION TEST RESULTS							U/C Shear Strength p.s.f.	Description of Sample
				Nat. M.C.	Wet density p.c.f.	Dry density p.c.f.	Degree of Saturation	Void ratio e	% Strain at Failure			
1	13'0" - 13'6"	595.9	582.6	15.4	135.5	117.5	100	.43	20.0	3360	Silty clay grits & pebbles	
3	12'6" - 13'0"	596.4	583.7	18.0	136.0	115.0	100	.46	20.0	4100		
3	13'0" - 13'6"	596.4	583.2	16.8	134.0	115.0	100	.46	20.0	4600		
6	12'0" - 12'6"	595.6	583.3								As above sand layer	
5	12'6" - 13'0"	595.6	582.8	15.4	138.0	119.5	100	.41	16.5	3940		
5	13'0" - 13'6"	595.6	582.3	17.0	136.0	116.0	100	.45	20.0	3600		
5	18'0" - 18'6"	595.6	577.3	18.9	135.5	114.0	100	.47	20.0	2610		

APPENDIX III
METHOD OF OPERATION

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 tubes or by wash water.

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

The standard penetration test results are recorded when sampling with the regular 2 inch O.D. split barrel sampler, these being the number of blows of a 140 pound hammer falling 30 inches, required to drive the sampling tube a distance of one foot into undisturbed soil.

The Dutch cone probe test is made by driving the drill rods into the ground with a 2-1/4" 6000^{lb} 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 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.

APPENDIX III

METHOD OF OPERATION

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 tubes or by wash water.

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

The standard penetration test results are recorded when sampling with the regular 2 inch O.D. split barrel sampler, these being the number of blows of a 140 pound hammer falling 30 inches, required to drive the sampling tube a distance of one foot into undisturbed soil.

The Dutch cone probe test is made by driving the drill rods into the ground with a 2'-1/4" - 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 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.

E. M. PETO ASSOCIATES LTD.

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section.

January 8, 1960.
FOUNDATION INVESTIGATION - by
E.M. Peto & Associates, Ltd.

Attention: Mr. S. McCombie.

Re: Proposed Underpass - Hwy. 401 & County Rd.,
Twp. of Harwich and Raleigh East,
W.P. 56-59 - District #1, Chatham, Ontario.

The detailed foundation investigation carried out at the above site by E.M. Peto & Associates, and accompanying this memo, has been reviewed by the Foundation Section. We are in general agreement with the conclusions reached in this report and these conclusions are summarized below for your convenience:-

1. The main soil types encountered at this site consist of a stiff to hard upper desiccated layer of silty clay underlain by a stiff layer of silty clay containing some sand and gravel.
2. Spread footings may be used to support the proposed structure. These footings may be placed at elevation 590' or lower, and may be designed for a bearing pressure of 3 tons/ft.² provided the footing width ranges between 5 - 10 feet.
3. No problems associated with approach fill stability are expected, and ordinary design procedures using a 2:1 side slope may be used.
4. Water entering the excavations for footings will be small, and will be mainly due to surface run-off. All seepage water should be easily controlled with low-capacity pumps.

cont'd. /2 ...

If further queries should arise regarding the conclusions of this report, please contact the Foundation Section.

L. G. Soderman,
PRINCIPAL SOILS & FOUNDATIONS ENGR.

per:

K. Peaker

(K. Peaker,
FOUNDATION FIELD SUPERVISING ENGR.)

KP/MdeF
Encl.

cc: Messrs. A. M. Teye (2) ✓
H. A. Tregaskes
D. G. Ramsay
A. Gater
G. U. Howell
J. Roy
A. Watt

Foundation Section
Gen. Files.

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 59252

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

December 18th, 1959.

59-F-203C

59-F-201C

Department of Highways of Ontario,
Soil and Foundation Engineering Branch,
Parliament Buildings,
Toronto 2, Ontario.

Attention: Mr. L. Soderman, P. Eng.

Re: Soil Site Investigation,
Proposed Underpass,
County Road, Chatham to Charing Cross,
Highway 401 - W.P. 56-59 - District #1.

Dear Sirs:

We have pleasure in submitting herewith ten (10) copies of our report for this project.

In the following report we have described the soil conditions encountered in some detail and we have discussed the engineering aspects arising from the soil conditions;

For your convenience we summarize our findings and recommendations:

1. The soil conditions as encountered at the above site are uniform with nearly horizontal stratification planes between individual soil types.
2. The proposed structure may be founded on the dessicated brownish-grey clayey till layer. The allowable bearing values are given on page 9 of the report.

3. No difficulty is foreseen in placing the embankment, provided the soils containing organic matter are replaced. The subsoil conditions permit the construction of an embankment with side slopes of 1 in 1. The estimated Factor of Safety of an embankment of 30 ft. in height and side slopes of 1 in 1 is in excess of 4.0.

4. No difficulty will be encountered in excavations to the required foundation depth.

5. There is no water problem connected with the excavation and the placing of structure at the above site, except possibly from small amounts of seepage water.

We believe this report to be complete, and to contain all the information you require. However, should you require some additional information in connection with this investigation we shall be pleased to be of further service.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

BL:sb

THE DEPARTMENT OF HIGHWAYS OF ONTARIO

SOIL SITE INVESTIGATION

HIGHWAY 401 UNDERPASS

W. P. 56 - 59

TOWNSHIP HARWICH AND RALEIGH EAST

December, 1959

e. m. peto associates ltd.,

NOTE NEW ADDRESS

~~831 Rosedale Avenue, Toronto 19, Ontario~~

1287 CALEDONIA ROAD

TORONTO 19, ONTARIO

Job No.

59252

Client's Ref. No.

Date

December 18th 1959

Report on

SOIL SITE INVESTIGATION

at

HIGHWAY # 401 UNDERPASS

W. P. 56 - 59

TOWNSHIP HARWICH and RALEIGH EAST

for

THE DEPARTMENT OF HIGHWAYS OF ONTARIO

1. INTRODUCTION

We were authorized by a letter from the Department of Highways of Ontario dated December 8th 1959 and signed by Mr. A. Rutka, Materials and Research Engineer, to carry out a soil investigation at the proposed location of the underpass of Highway # 401 and the county road (Highway # 89) in the Township of Harwich and Raleigh East. At the same time we were issued with the D.H.O plan No. 3533 and profile F-3533-1 and 3533-2, showing the location of the proposed underpass and the profile.

We were required to:

- (a) determine the existing soil conditions and the soil stratigraphy at this site,
- (b) determine any pertinent ground-water information
- (c) obtain the shear strength and the allowable bearing values of the subsoil which would provide a basis for the design computations.
- (d) advise of any possible construction problems at this site.

2. PROGRAMME OF WORK

December 10th 1959 - Test holes located and the elevations at the existing grade obtained by our Field Supervisor. Field crew moved onto site
Test hole 1 commenced.

2. PROGRAMME OF WORK(cont'd.)

- December 11th, 1959 - Test hole 1 completed.
- December 12th, 1959 - Test hole 3 completed
- December 13th, 1959 - Test hole 5 completed
- December 14th, 1959 - Dynamic cone penetration test carried out at holes 2 and 4. The field crew moved off the site.

3. GENERAL INFORMATION

- (a) The test holes were put down in accordance with our standard procedure as outlined in Appendix III under "Method of Operation".
- (b) The details of the test holes (i. e. depth, diameter of casing and the ground elevations) were as follows:

<u>Test hole no.</u>	<u>Elevation at the existing surface</u>	<u>Depth</u>	<u>Diameter of casing</u>
1	595.90	46'0"	BX (2-1/2")
3	596.36	36'0"	BX(2-1/2")
5	595.62	51'0"	BX (2-1/2")
<u>Probe holes</u>			
2	595.61	20'0"	
4	596.05	22'0"	

- (c) The elevations as given above and in the following report were obtained using the D. H. O. Bench Mark which was a cut across on the wall of a concrete culvert 20 ft. left of Sta 5 + 48 County Road Chaniage having elevation 597.99.
- (d) The detailed soil description, the results of standard penetration tests and the natural moisture contents are given in the individual bore hole logs.

3. GENERAL INFORMATION (cont'd.)

- (e) The location of the test holes (as suggested by the client) and the assumed soil profile is shown on the attached site plan.
- (f) The graphical representation of the results of the standard penetration tests, the dynamic cone penetration tests, (test hole and probe holes 2 and 4) and the natural moisture contents, versus elevation are given in Appendix I under "Field Test Results".
- (g) The results of the laboratory shear strength tests (Unconfined compressive tests) are shown in tabular form in Appendix I 1 under "Laboratory Test Results".

4. SITE AND GEOLOGY

The site proposed for the Highway # 401 and the county road underpass is situated some 3.4 miles south-east of Chatham. The topography is very flat with cultivated fields adjacent at both sides to the county road.

Geologically, the drift deposits at the above site are underlain by soft, blue and grey shale and limestone of Hamilton formation of the Devonian System. Glacial lake Whittesley deeply covered the present Essex, Kent and Lambton counties; and technically, most of the soil at depth is a clay till or alluvial clay i. e. the water-laid sediments.

5. SOIL CONDITIONS

During the present investigation the following main soil types were encountered:

- (a) Top soil and sandy loam
- (b) Stratified brown and grey silty clay
- (c) Clayey till (silty clay, grits and pebbles)
- (d) Grey fine-to-medium sand
- (e) Sandy till

The following is a detailed description of each type of soil.

5. SOIL CONDITIONS (cont'd.)

(a) Top soil and sandy loam

This uppermost layer was encountered in test holes 1 and 5. In test hole 3 underlying the asphalt surfacing and the concrete base of the county road, a 3-inch thick layer of organic top soil was found.

In test hole 1 the sandy loam layer with traces of organic matter and mixed brown in colour was 2'6" deep. In test hole 5 it was encountered to a depth of 2'7" below the existing grade. Generally the sandy loam layer was found to be moist to quite moist and of loose density.

(b) Stratified brown and grey silty clay

Underlying the sandy loam, a stratum of stratified brown and grey silty clay was found. A certain amount of dessication of this layer was evident, as the colour in some instances was rusty-brown. The lower boundary of this stratum was encountered as follows:

<u>Test hole</u> <u>no.</u>	
--------------------------------	--

1	- at 5'2" below grade
3	- at 7'3" below grade
5	- at 6'2" below grade

As may be seen, the lower boundary is nearly horizontal.

In test hole 3 traces of organic matter were met in the layer of stratified brown and grey silty clay. Pockets of wet sandy gravel were found in test hole 5. The natural moisture content of brown and grey silty clay was, in every instance, wetter than its Plastic Limit, and varied between 22.6% (test hole 3) and 28.6% (test hole 5). As an average moisture content a value of 24.7% may be assumed.

The results of the standard penetration test indicated that the stratified silty clay layer was firm to very stiff with the number of blows per foot penetration varying between 4 (test hole 1), depth 2-3 ft.) and 17 (test hole 3, depth 5-6 ft.). It is evident that the density of the stratum increased with depth.

5. SOIL CONDITIONS (cont'd.)

The mechanical analysis conducted on two samples (test holes 3 and 5 - sample no. 2) gave the following results:

Test hole no.	Depth ft.	Elevation	Gravel %	Sand %	Silt %	Clay %
3	5'6"	590.9	-	14	51	35
5	5'6"	590.1	-	5	35	60

According to these results the stratified grey and brown silty clay is a clay to silty clay soil, with the percentage of clay particles varying between 35 and 60%.

(c) Clayey till

A transition zone between the overlying stratified silty clay and the lower - lying grey clayey till consisted of a stratum of brownish-grey clayey till. Although geologically it belongs to the grey clayey till layer the difference in colour is mainly due to the dessication process of the upper portions of grey clayey till layer. The gradual change in colour from brownish-grey to grey did not permit a clear picture to be established of the lower boundary of this dessicated layer, although, as may be seen from the attached soil profile, an attempt was made to establish this lower boundary and it was estimated that the upper limit of grey clayey till stratum was at the following depths below the existing grade:

Test hole 1 - at about 7'0"
 " " 3 - at 10'3"
 " " 5 - at 9'9"

The dessicated brownish-grey till layer was found to be very stiff with "N" values ranging from 24 (test hole 1) to 28 (test hole 5) with the natural moisture content around 17% i. e. more or less at the plastic limit of the material.

Below the dessicated layer there was a grey clayey till layer. The lower boundary of the grey clayey till was encountered at the following depths:

Test hole 1 - at 34'6" below grade
 " " 3 - at 35'0" " "
 " " 5 - at 33'4" " "

5. SOIL CONDITIONS (cont'd.)

According to these results a slight dip of the lower boundary of grey clayey till layer exists from the area of test hole 1 towards test hole 5.

The results of the standard penetration tests indicated that the density of grey clayey till layer increased with depth to an elevation of about 574 to 575 then decreases and at an elevation of about 565 nearly identical results were obtained in all the test holes. The "N" value at this point was 21 in test hole 3, 22 in test hole 1 and 23 in test hole 5.

From this depth there is a marked increase in density again, particularly in test holes 1 and 3. In test hole 5 the rate of increase in density with depth decreases.

The results of the Atterberg Limits indicated a decrease in the values of Liquid Limit, Plastic Limit and the Plasticity Index with depth. The following table gives the results of the Atterberg Limit tests.

Test hole no.	Depth	Elevation	In per cent.		
			L. L.	P. L.	P. I.
1	5 - 6	590.4	33.1	20.3	12.8
3	15 - 16	580.9	31.5	18.2	13.3
1	25 - 26	570.4	29.6	17.0	12.6
5	35 - 36	560.1	27.8	18.5	9.3

According to Casagrande's Classification System the clayey till is an CL-CI soil changing with depth to CL-soil. Therefore it is an inorganic silty clay to clayey silt soil with plasticities changing with depth from medium to slight.

The mechanical analysis gave the following results:

Test hole no.	Depth	Elevation	Per cent of			
			Gravel	sand	Silt	Clay
3	20-21	575.9	3	17	45	35
3	30-31	565.9	7	23	40	30

5. SOIL CONDITIONS (cont'd.)

Analysing the data of the Atterberg Limits tests and the mechanical analysis it may be seen that the percentage of silt and clay particles decreases with depth, and this corresponds with the decreased plasticity of the soil as observed from the Atterberg Limits tests.

The textural classification of the grey clayey till is thus:

silty to sandy clay.

It was observed during the field work that the density of the clayey till was somewhat lower at a depth of 12-13 ft. below grade. In order to be able to obtain a minimum shear strength of the clayey till, which was thought to exist at this depth a number of unconfined compressive tests were conducted. The detailed results of these tests are given in Appendix II. It was found that the following minimum average values for the layer of grey clayey till may be assumed:

Shear strength:	3920 p.s.f.
Wet density:	135.8 p.c.f.
Dry density:	116.1 p.c.f.

The average void ratio $e = 0.446$

Degree of saturation = 100%

(d) Grey fine to medium sand

Located between the stratum of grey clayey till and grey sandy till in test holes 1 and 3 a layer of grey fine to medium sand was encountered. In test hole 1 it was located between 34'6" and 35'2" below grade and in test hole 3 between 35'0" and 35'3". No trace of this layer was found at this depth in test hole 5, thus it may be assumed that it terminated between test holes 3 and 5. This sand layer was found to be quite moist and is probably a water-bearing seam.

Another grey fine to medium sand layer was established at 43'6" below grade in test hole 1. The presence of this layer was not observed in the remainder of the test holes, possibly due to the insufficient depth of the test holes. This lower fine to medium sand layer contained natural gas in small quantities, since,

5. SOIL CONDITIONS (cont'd.)

following a two-day period, no trace of gas was found to be present. Both layers of grey sand (the upper and the lower seam) were extremely dense.

(e) Grey sandy till

The material underlying the grey sand seam at test holes 1 and 3 from a depth of 35'2" and 35'3" respectively, below grade, and in test hole 5 from a depth of 38'4" was classified as sandy till.

The results of the mechanical analysis confirmed the above description as texturally the material was found to be silty sand. The mechanical analysis gave thus the following result:

<u>Test hole no.</u>	<u>Depth</u>	<u>Elevation</u>	<u>Per cent of</u>			
			<u>gravel</u>	<u>sand</u>	<u>silt</u>	<u>clay</u>
1	40-41	555.4	15	39	32	14
5	45-46	550.4	15	35	36	14

The Atterberg Limits were as follows: (Test hole 5, depth 40-41 ft.: elevation 555.1)

Liquid Limit - 21.6%
Plastic Limit - 14.2%
Plasticity Index - 7.4%

According to Casagrande's Classification System it is an SF-CL soil i. e. clayey sand or clayey silt with low plasticity.

The results of the standard penetration tests indicated that the sandy till is dense to extremely dense with the number of blows per foot of penetration increasing with depth.

The natural moisture content was slightly wetter than the Plastic Limit of the material.

6. CONCLUSIONS AND RECOMMENDATIONS

1. The soil conditions as encountered at this site are uniform with the stratification planes nearly horizontal.
2. The proposed structure may be founded on a dessicated layer of brownish-grey clayey till. Placing the foundations on the stratified grey and brown silty clay is not recommended because the plasticity characteristics of this soil may lead to settlements in excess of permissible amounts.
3. The recommended allowable bearing values are as follows:

Foundation depth below grade	Foundation width	Allowable bearing value in lbs. / sq. ft.	
		Strip	Isolated
8	5	6400	7650
	10	5600	6700
	15	5300	6400
	20	5200	6200
10	5	6800	8100
	10	5800	7000
	15	5400	6600
	20	5300	6400
12	5	7150	8500
	10	6000	7150
	15	5600	6700
	20	5400	6600
14	5	7250	8700
	10	6200	7350
	15	5700	6900
	20	5500	6600

4. The above allowable bearing values were calculated assuming that the shear strength of the soil is constant with depth and is equal to 2900 p.s.f. which as may be seen represents the assumed minimum shear strength for the grey clayey till layer and that the allowable bearing value is given by:

$$q_a = \frac{N_c \times Su}{3} \quad (\text{Assuming Factor of Safety of 3})$$

where N_c - is the parameter depending on the geometry of foundation and obtained according to Skempton's empirical equation

Su - the undrained shear strength.

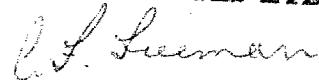
It may be noted that the effect of overburden was neglected.

6. CONCLUSIONS AND RECOMMENDATIONS (cont'd.)

5. As the underlying soil is of stiff to very stiff density, the foundation design depends primarily on the consideration of the shear strength, as the settlements will be restricted to very small amounts. The settlements will be less than the permissible amounts for the type of structure proposed (the limit of maximum total settlement assumed to be 1 inch).
6. Removal of all sandy loam and part of the stratified grey and brown silty clay containing organic matter is recommended before placing the embankment. Subsoil conditions are sufficiently good to allow an embankment with a side slope of 1 : 1 to be constructed. The calculated factor of safety of embankment of 30 feet in height and with slopes of 1 in 1 is in excess of 4.0. Some settlement of the embankment should be expected but this will have no serious influence on the proposed underpass.
7. During the present investigation, apart from a negligible amount of seepage water encountered in the sandy loam layer, there were indications of water-bearing characteristics in the grey sand layers. The probable ground water table may be situated some 30 - 35 feet below existing grade. The exact location of the groundwater table was not established because of the short term observations.
- Thus it may be concluded that the possible ground-water table will have no effect on the proposed structure.
8. No difficulties will be encountered in excavation to the required depths and placing of foundations. However, provision should be made for a limited amount of pumping, due to some seepage water.

Yours very truly,

E. M. PETO ASSOCIATES LTD .



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

CFF/jn

BL

APPENDIX I
FIELD TEST RESULTS

DYNAMIC CONE PENETRATION TEST

Test Hole #2

Depth		No. of Driving Blows
From	To	
0	1	2
1	2	2
2	3	2
3	4	5
4	5	5
5	6	11
6	7	13
7	8	14
8	9	20
9	10	23
10	11	30
11	12	40
12	13	46
13	14	65
14	15	63
15	16	103
16	17	168
17	18	167
18	19	237
19	20	240

Test Hole # 4

	Depth		No. of Driving Blows.
	From	To	
0	1	10	
1	2	4	
2	3	2	
3	4	3	
4	5	6	
5	6	8	
6	7	13	
7	8	28	
8	9	32	
9	10	33	
10	11	37	
11	12	43	
12	13	50	
13	14	55	
	14	15	57
	15	16	70
	16	17	93
	17	18	107
	18	19	120
	19	20	160
	20	21	185
	21	22	242

DYNAMIC CONE PENETRATION TEST

Test Hole #1

Depth		No. of Driving Blows
From	To	
0	1	4
1	2	3
2	3	3
3	4	5
4	5	11
5	6	18
6	7	40
7	8	50
8	9	58
9	10	40
10	11	45
11	12	58
12	13	68
13	14	78
14	15	109
15	16	136
16	17	144
17	13	167
18	19	245
19	20	280
20	21	Further cone penetration test. Abandoned
21	22	
22	23	

APPENDIX II

LABORATORY TEST RESULTS

UNCONFINED COMPRESSION TEST

Job No. 59252

Borehole Number	Depth Feet	Ground Level Feet	Elevation Feet	Natural Moisture Content.	Wet Density p. c. f.	Dry Density p. c. f.	Degree of Saturation %	Void Rat., e	% Strain at Failure	u/c Shear Strength p.s.f.	Description of Sample
1	13' - 13'6"	595.9	582.6	15.4	135.5	117.5	100	.43	20.0	3360	Silty clay grits & pebbles.
3	12'6"-13'0"	596.4	583.7	18.0	136.0	115.0	100	.46	20.0	4100	
3	13' - 13'6"	596.4	583.2	16.8	134.0	115.0	100	.46	20.0	4600	
5	12' - 12'6"	595.6	583.3								As above sand layer
5	12'6"- 13'0"	595.6	582.8	15.4	138.0	119.5	100	.41	16.5	3940	2240x
5	13' - 13'6"	595.6	582.3	17.0	136.0	116.0	100	.45	20.0	3600	
5	18' - 18'6"	595.6	577.3	18.9	135.5	114.0	100	.47	20.0	2610	

e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

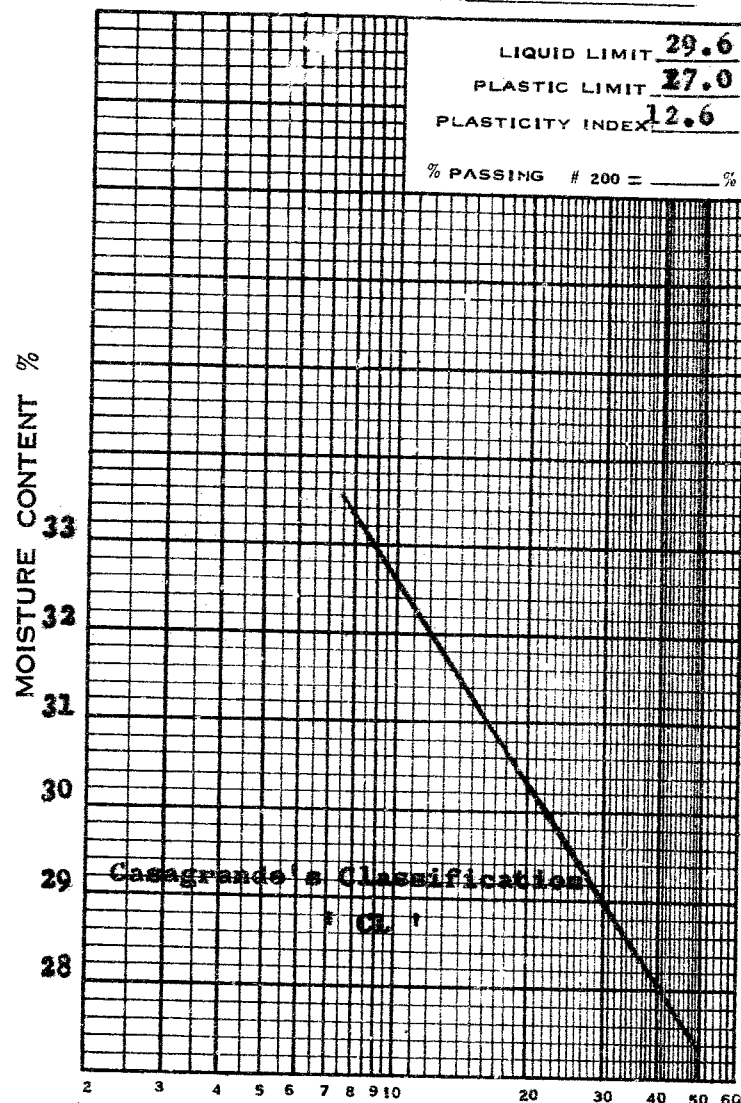
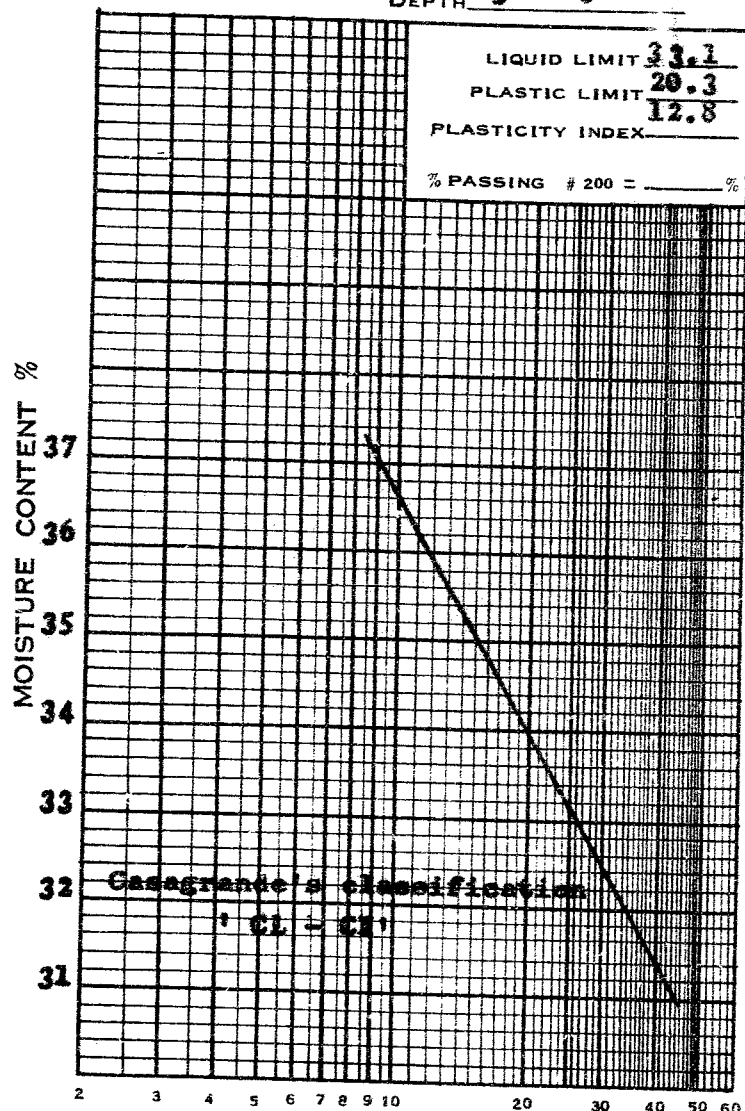
FLOW LINE CHARTS

JOB No. 59252 PROJECT Hwy. 401 Underpass, Twp. Harwich.
SAMPLE FROM B.H. # 1. Sample # 3.

DEPTH 5' - 6'

SAMPLE FROM B.H. # 1. Sample # 7.

DEPTH 25' - 26'



NO. OF BLOWS (LOG SCALE)

e. m. peto associates ltd.

Toronto 19, Ontario

LIQUID LIMIT TEST

JOB No. 59252 PROJECT Hwy. 401 Underpass, Twp. Harwich.

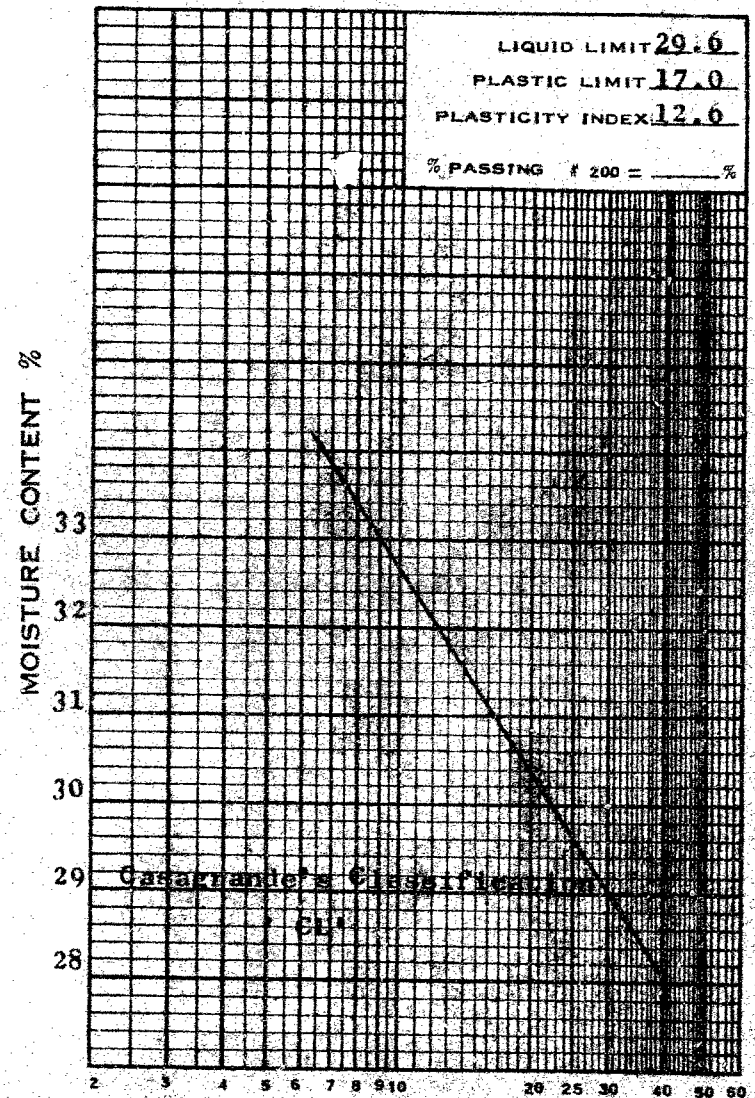
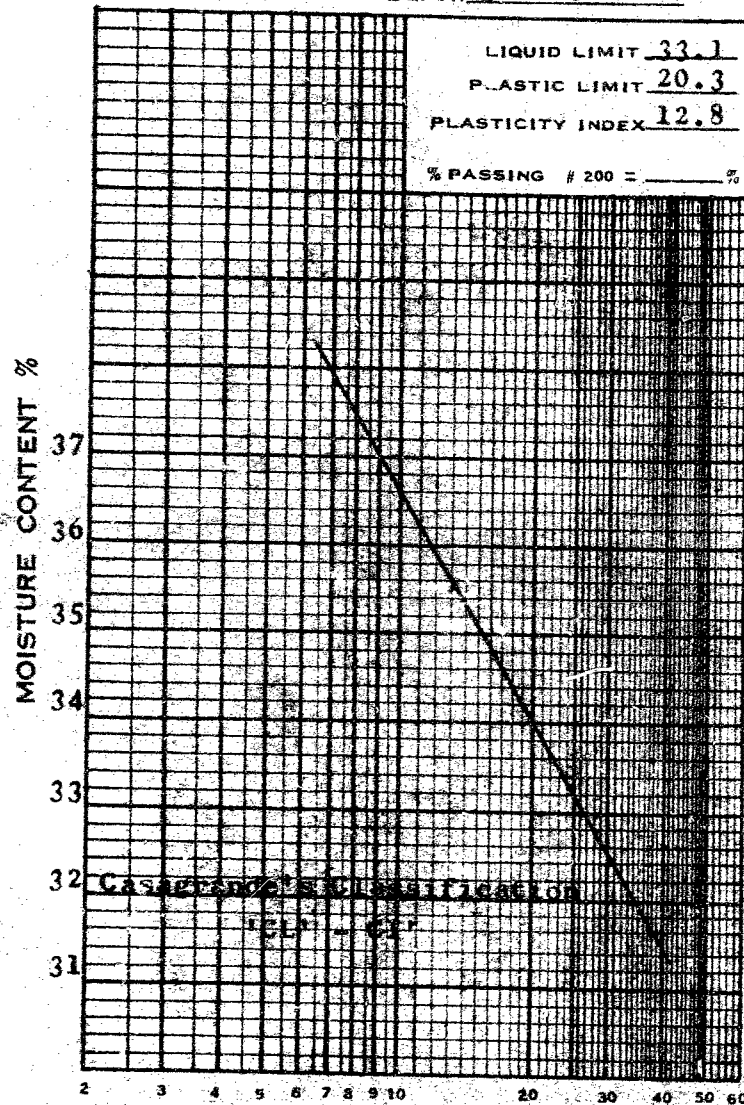
SAMPLE FROM B.H. #1 Sample #3

DEPTH 5' - 6'

FLOW LINE CHARTS

SAMPLE FROM B.H. #1. Sample #7.

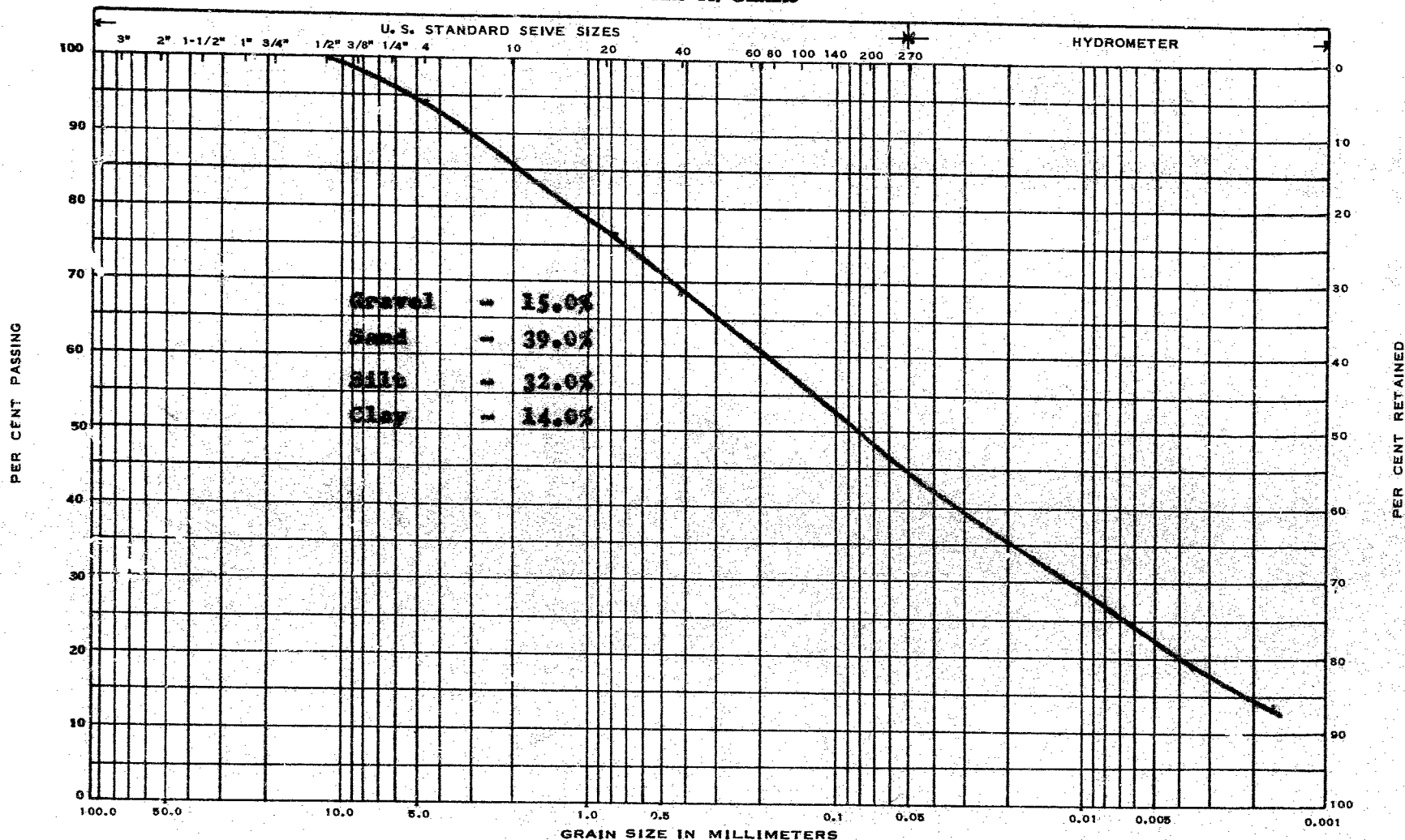
DEPTH 35' - 26'



NO. OF BLOWS (LOG SCALE)

e. m. peto associates ltd.

Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

MASS. INST. OF TECH. CLASSIFICATION

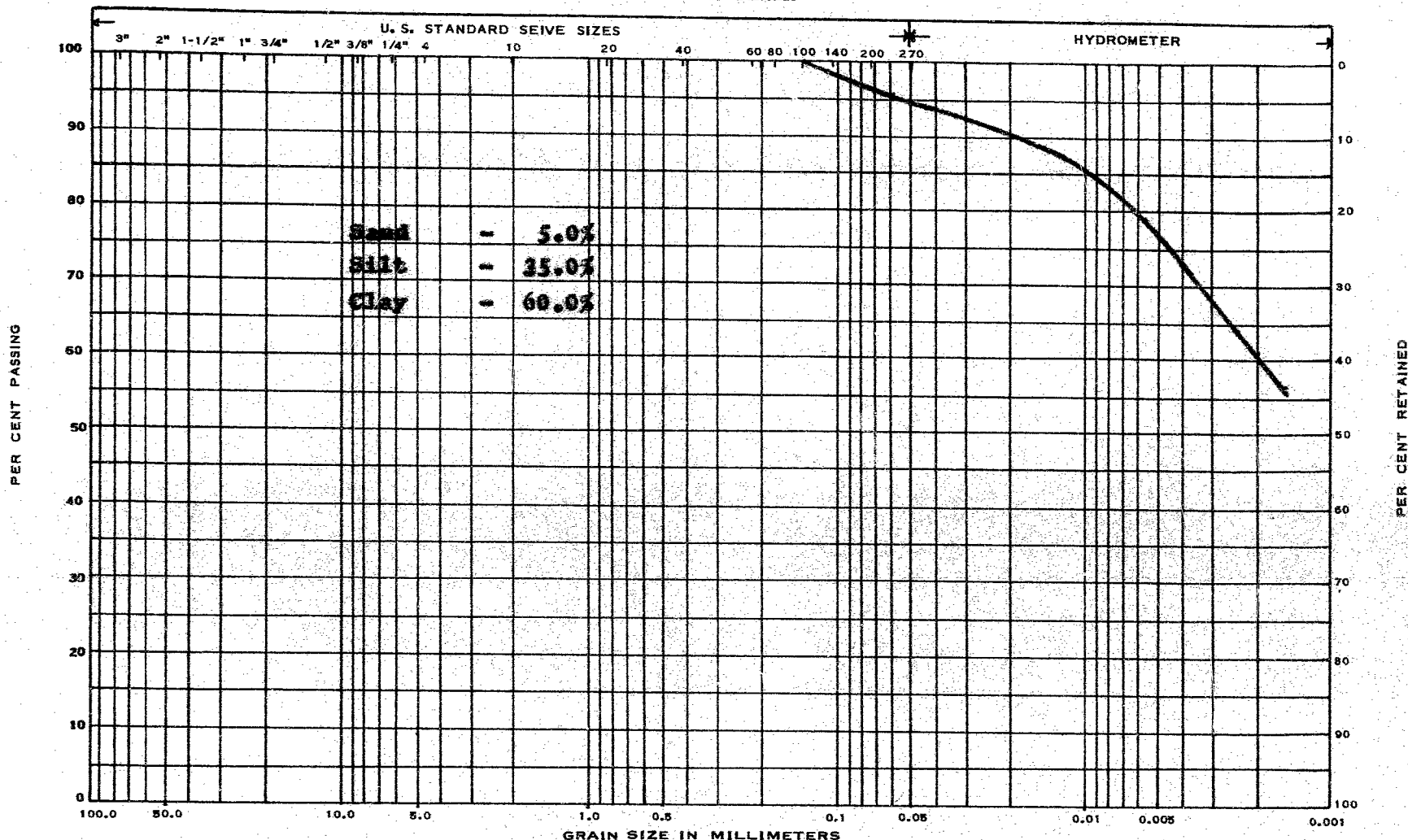
JOB NAME Box 401 Underpass JOB NO. 59252 HOLE NO. 1 SAMPLE NO. 10

DEPTH 155.4 ELEVATION 155.4 REMARKS Textural classification: Silty Sand (Till)

GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.

Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Hy. 401 Underpass.

JOB NO. 59252

HOLE NO. 5

SAMPLE NO. 2

DEPTH 5'-6'

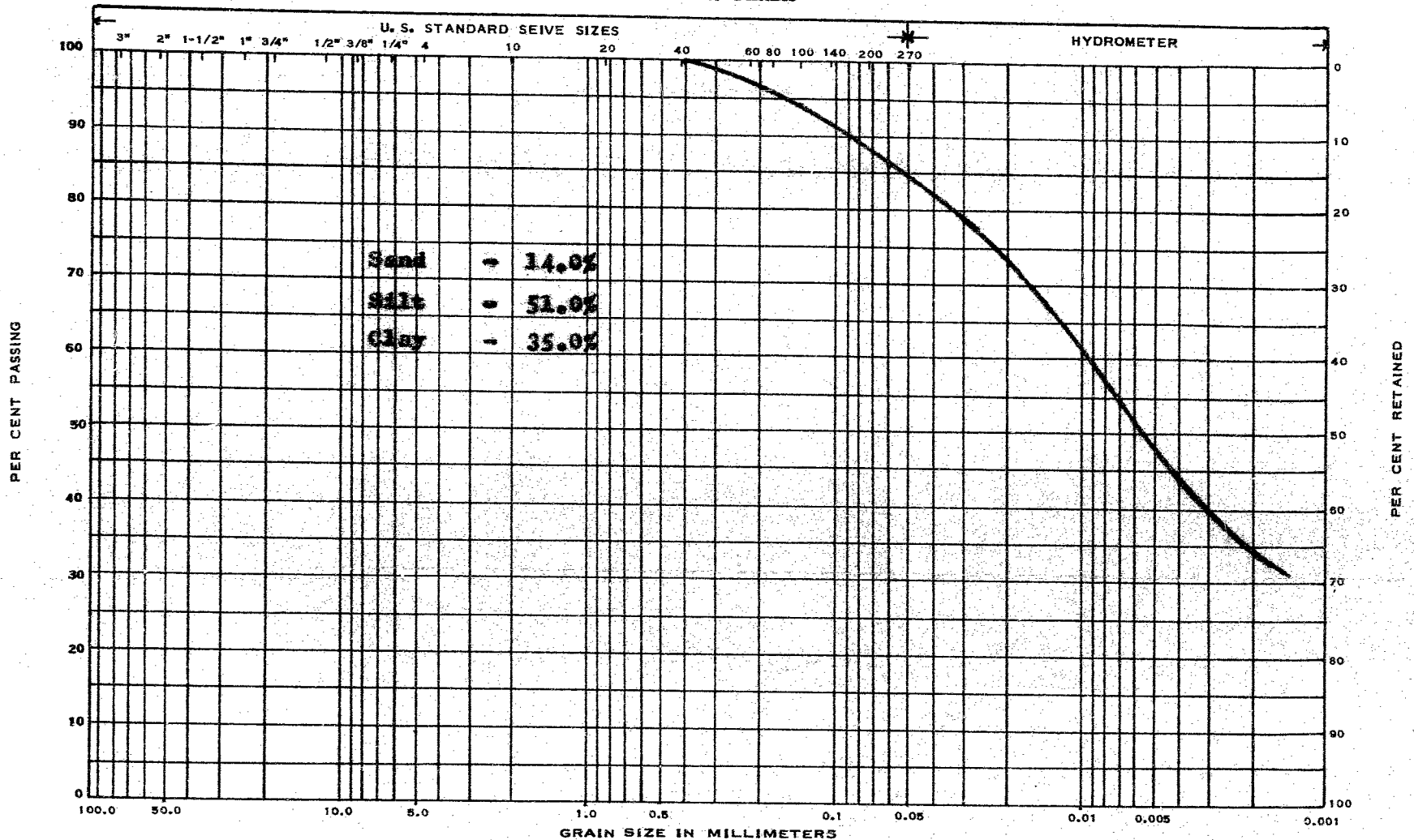
ELEVATION 590.1

REMARKS Textural Classification - 1 Clay

GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.

Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Hwy. 401 Underpass.

JOB NO. 59252

HOLE NO. 3

SAMPLE NO. 2

DEPTH 5'-6"

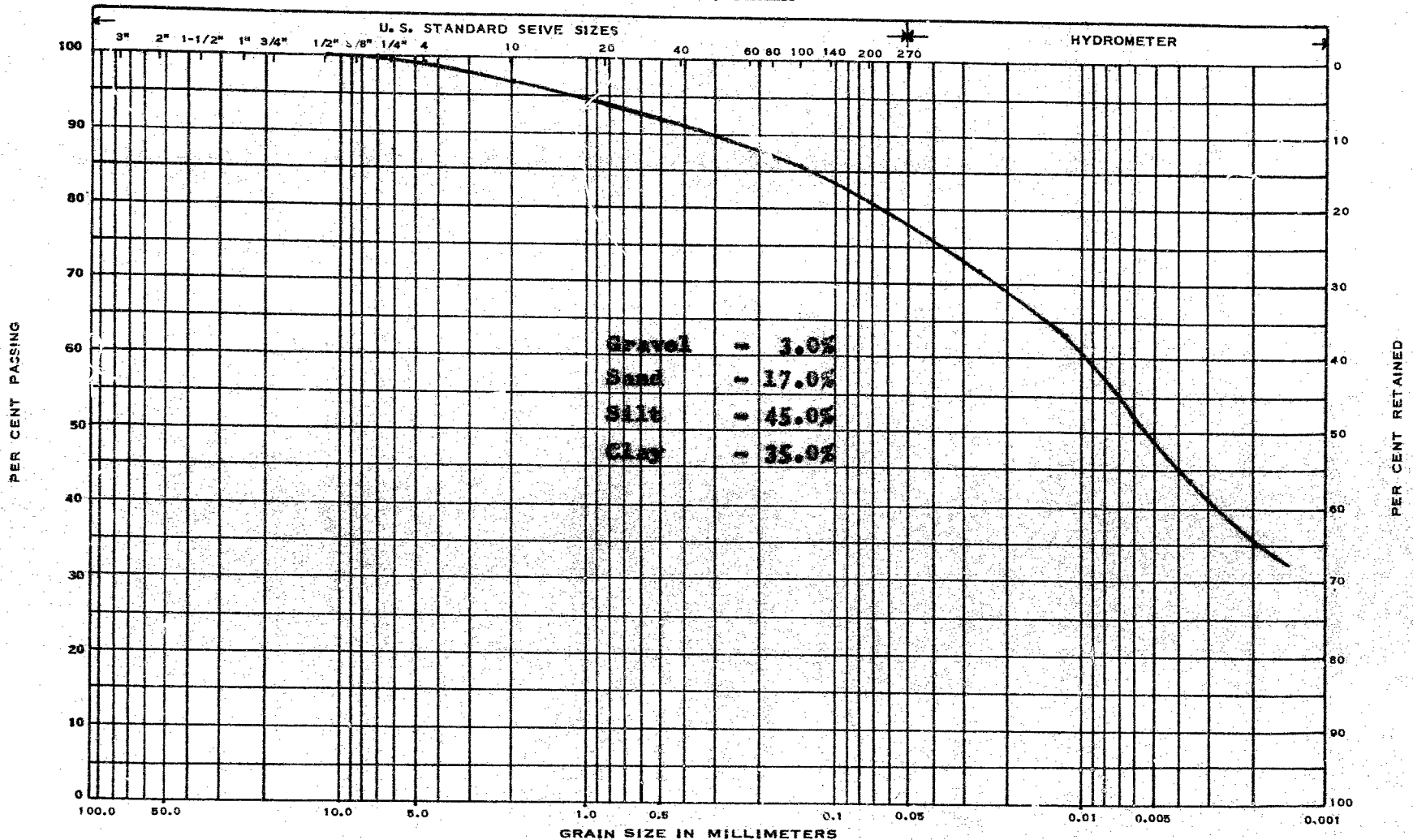
ELEVATION 599.9

REMARKS Textural Classification = ! Silty Clay !

GRAIN SIZE DISTRIBUTION

e. m. peto associates ltd.

Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

JOB NAME Reg. 401 Underpass

JOB NO. 51

LOCATION

HOLE NO. 3

SAMPLE NO. 6

DEPTH 20'-21' ELEVATION 575.9

REMARKS Textur

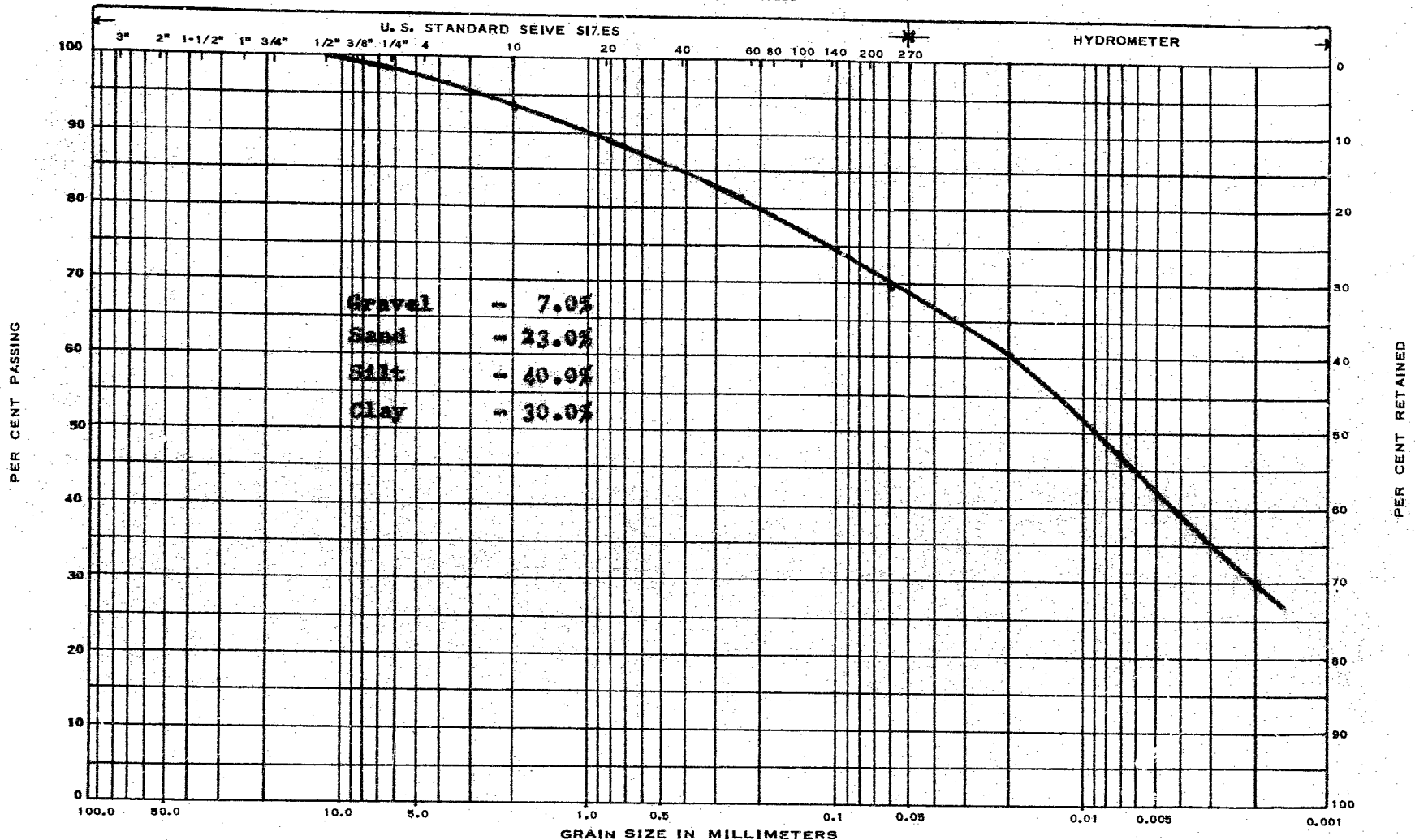
ation 'Silty clay'

GRAIN S

N

e. m. peto associates ltd.

Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Hwy. 401 Underpass

JOB NO. 59252

HOLE NO. 3

SAMPLE NO. 8

DEPTH 30'-31' ELEVATION 565.9

REMARKS Textural Classification - 1 Sandy Clay 1

GRAIN SIZE DISTRIBUTION

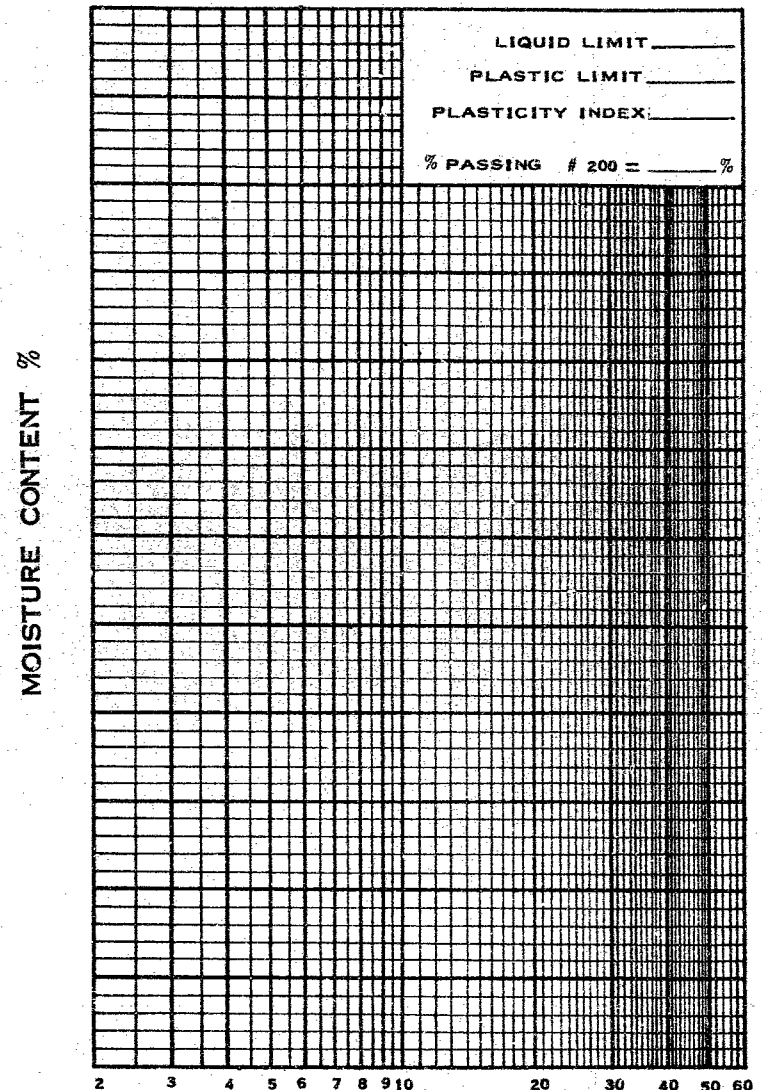
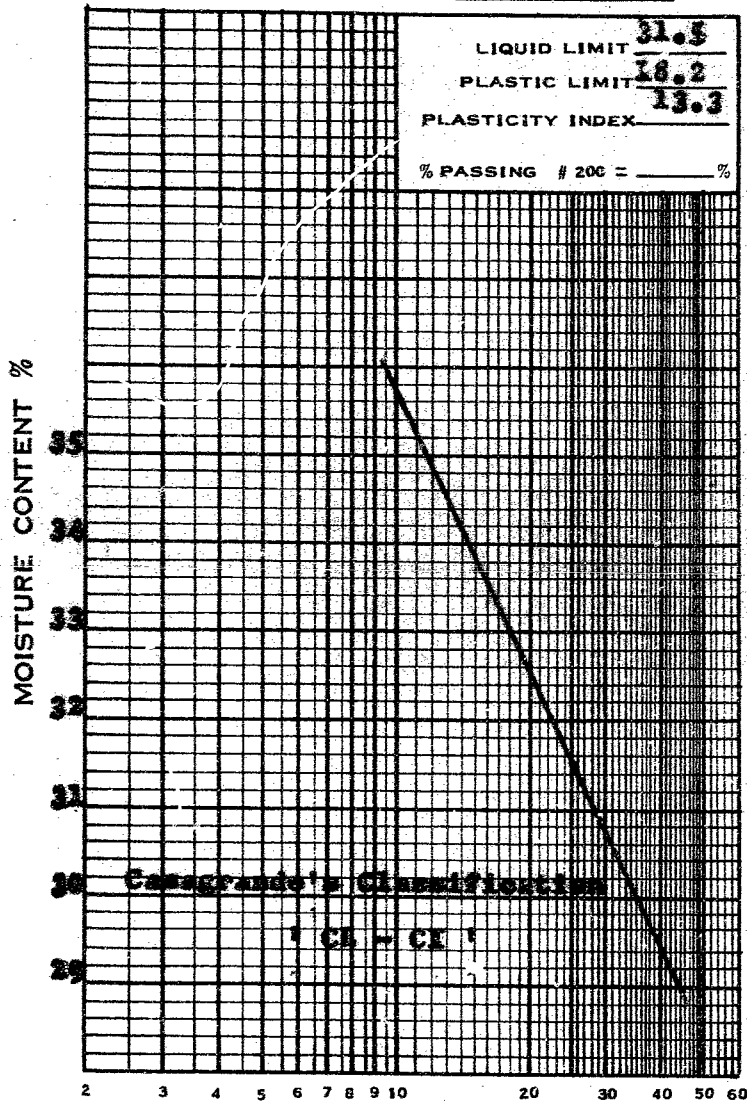
e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

JOB No. 59232 PROJECT Hwy. 401 Underpass
SAMPLE FROM B.M. # 8. Sample # 5.
DEPTH 15' - 16'

SAMPLE FROM _____
DEPTH _____



e. m. peto associates ltd.

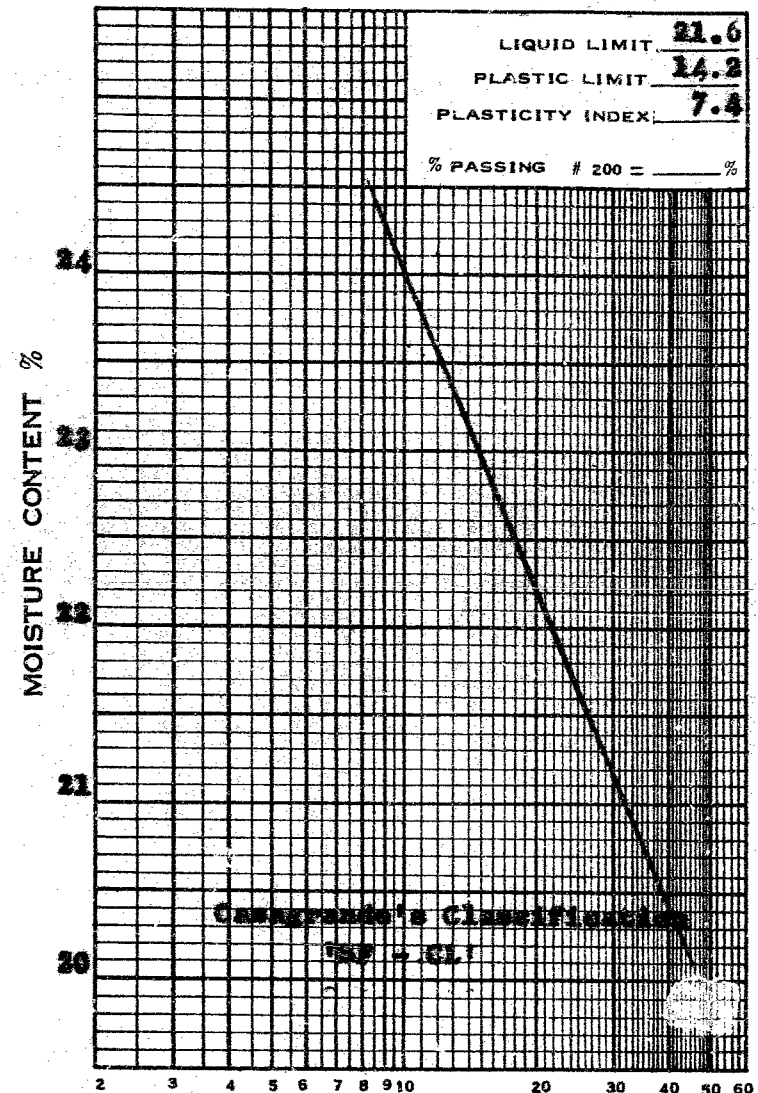
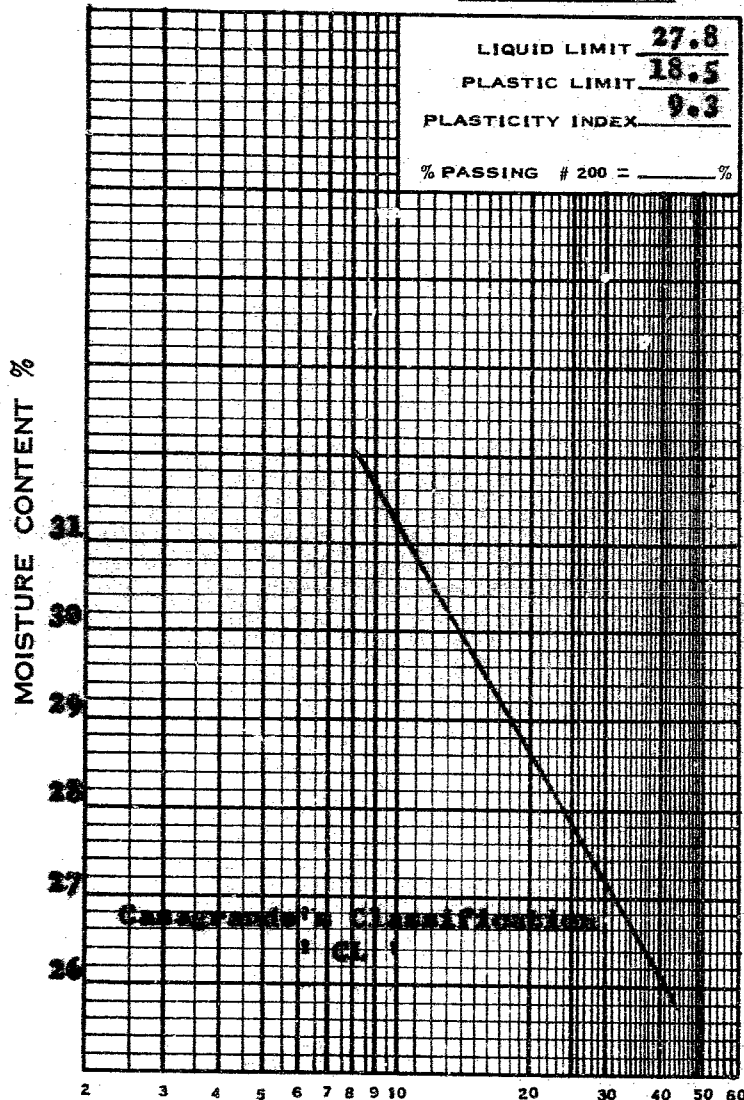
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

JOB No. 59252 PROJECT Hwy. 401 Underpass
 SAMPLE FROM B.H. # 5. Sample # 9.
 DEPTH 35' - 36'

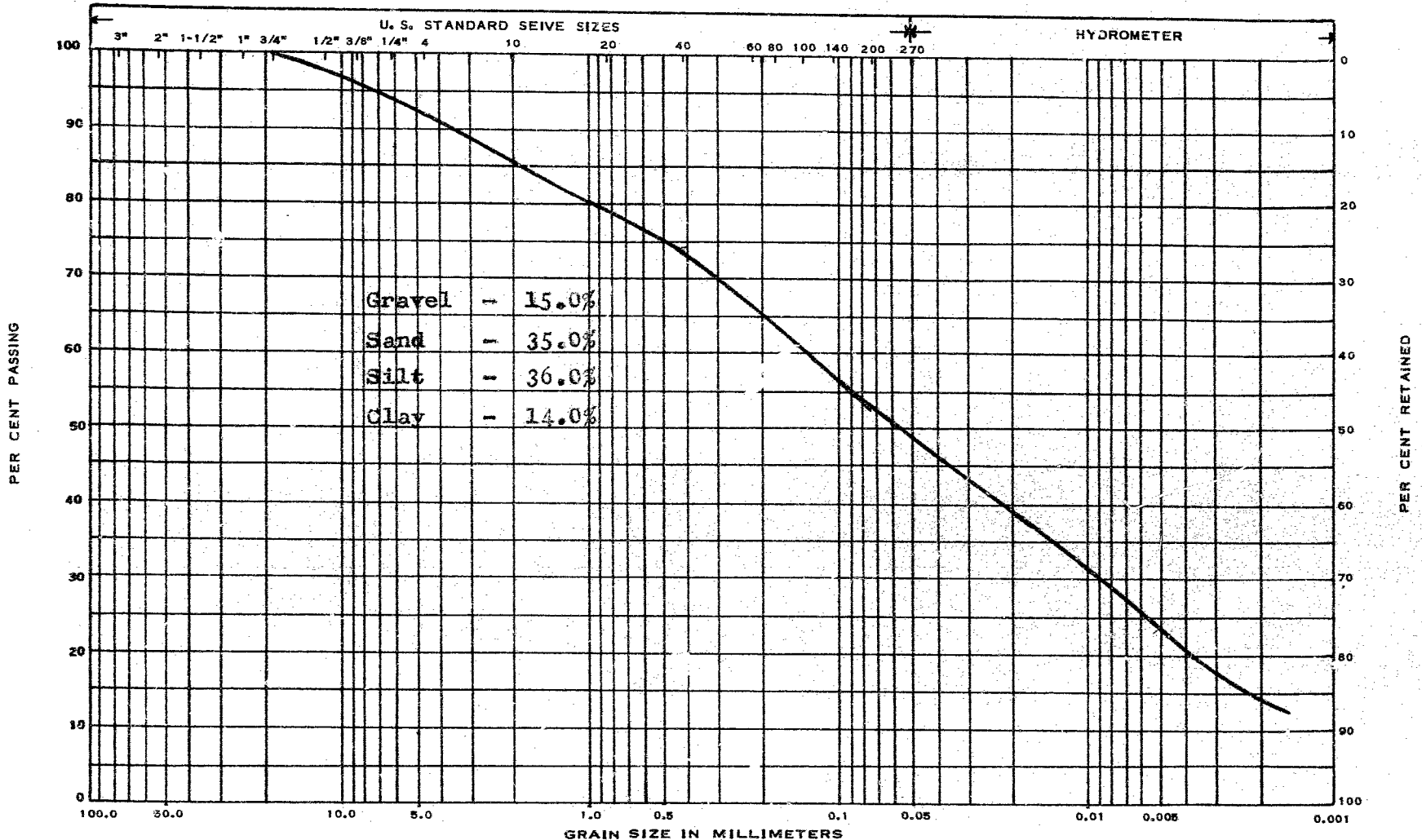
SAMPLE FROM B.H. # 5. Sample # 10.
 DEPTH 40' - 41'



NO. OF BLOWS (LOG SCALE)

e. m. peto associates ltd.

Toronto 19, Ontario



STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT	FINE SILT	CLAY
--------	--------	-------------	-----------	-----------	-------------	-----------	-----------	------

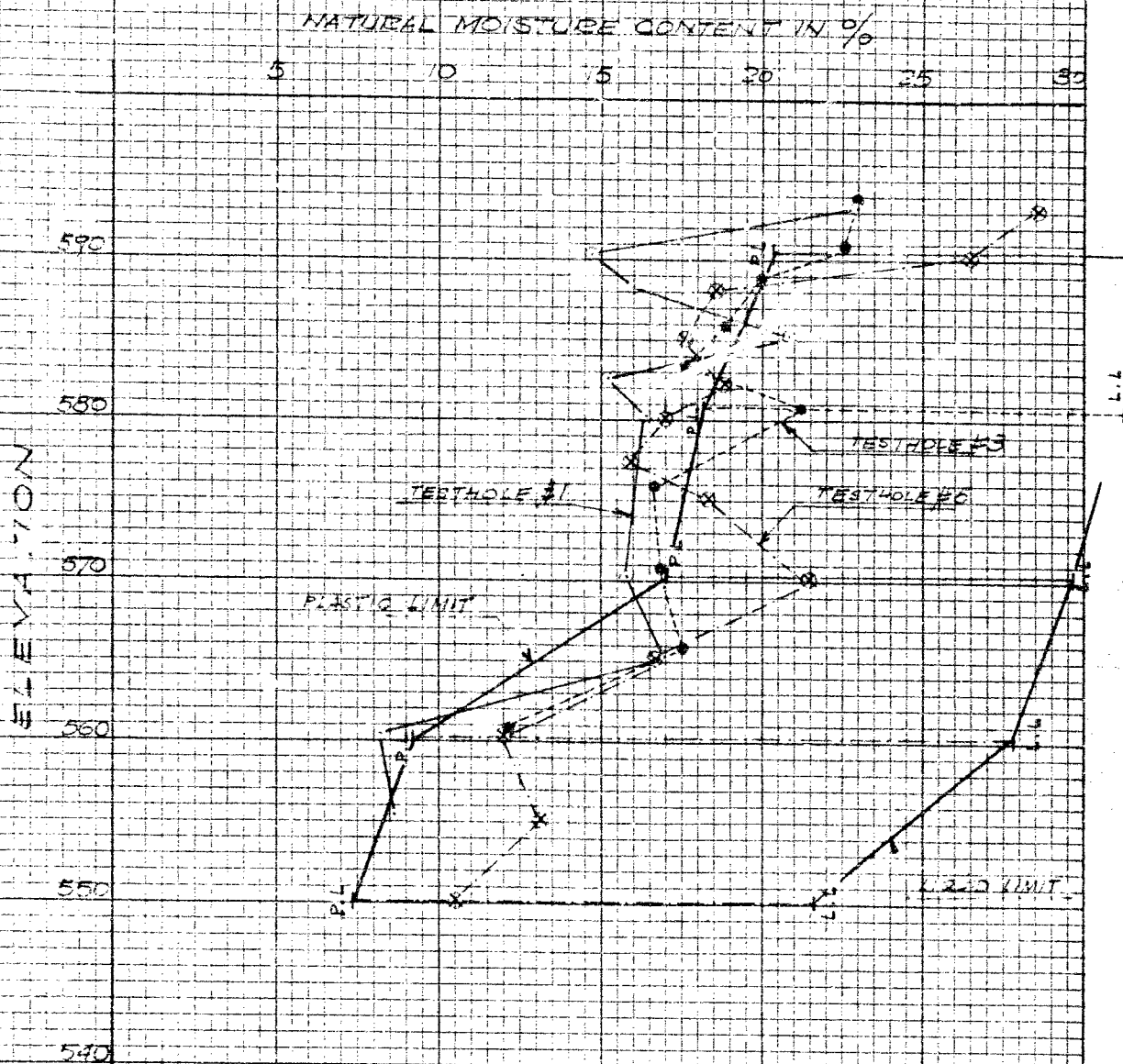
MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Hwy. 401 Underpass JOB NO. 59252 HOLE NO. 5 SAMPLE NO. 11

DEPTH 4-46 ELEVATION 550.4 REMARKS Textural classification, Silty sand (Till)

GRAIN SIZE DISTRIBUTION

Natural Moisture Content & Atterberg Limits versus Elevation



JOB No. 59359

E.M. PETO ASSOCIATES LTD.





BOREHOLE LOG

Hwy #401 Underpass
 Job Name Twp. Harwich & Rte 10 E. Job No. 50252
 Client Dept. of Highways, Ont. Casing NK
 Datum Meadetic Compiled By B.J.

Borehole No. #1
Boring Date Dec. 10 - 11/59
Checked By E.M.P.

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 SHELBY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

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

[illegible]

Checked By E.M.P.

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 SHELBY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

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

[illegible]





e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. # 401 Underpass Job No. 59252 Borehole No. # 3.
 Client Twp. Harwich & Raleigh East, Dep't. of Highways of Ontario Casing BX (2.1/2" Dia.) Boring Date Dec. 12th. 1959.
 Datum Geodetic Compiled By B.L. Checked By F.H.

SAMPLE CONDITION

 UNDISTURBED
 FAIR
 DISTURBED
 LOST

SAMPLE TYPE

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

ABBREVIATIONS





V.T. IN SITU VANE SHEAR TEST
 C. SOIL SHEAR STRENGTH LBS/SQ.FT.
 W.L. WATER LEVEL IN CASING
 W.T. GROUND WATER TABLE IN SOIL
 W.T.P.L. WETTER THAN PLASTIC LIMIT
 D.T.P.L. DRIER THAN PLASTIC LIMIT

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Natural Moisture Content	WATER LEVELS & REMARKS
Ground surface			0' 0"						
Asphalt surface & concrete base, organic topsoil.	DK. Brown to black	596.4	0' 7" 10"		1	C.S.			Moist.
Stratified silty clay, traces of organic matter.	Layers of grey & brown	Firm	5' 0"		2	S.S.	6	23.0%	W.T.P.L.
As above	Layers of grey & brown	Stiff to very stiff	7' 3"		3	S.S.	17	22.6%	W.T.P.L.
Clayey till	Brownish-grey	Very stiff			4	S.S.	25	20.0%	D.T.P.L.
Clayey till (Silty clay, some sand content, grits & pebbles)	Grey	Very stiff	10' 3" 10' 8"		5	S.S.	19	18.8%	W.T.P.L.
As above	Grey		15' 0"		6	2" S.L. Tapped		17.6%	
As above	Grey	Very stiff			7	S.S.	27	21.3%	W.T.P.L.
			20' 0"						Washwater used after 16' 0" depth.
As above.	Grey	Hard			8	S.S.	39	16.7%	W.T.P.L.
			25' 8"						
As above	Grey	Hard			9	S.S.	35	16.8%	W.T.P.L.
			30' 0"						
As above	Grey	Very stiff			10	S.S.	21	17.5%	W.T.P.L.
			35' 0"						Stiffens at 32' 6"
Sandy till	Dark grey	Very dense	35' 0" 36' 0"		11	S.S.	70	12.2%	3" thick layer of mixed grey fine sand at 35' 0" to 35' 1" 1/2"
									Hole terminated at 36' 0"

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Hwy. # 401 Underpass Job No. 59252 Borehole No. # 5
 Client Twp. Harwich & Raleigh East. Casing BX (2.1/2" Dia.) Boring Date Dec. 13th. 1959.
 Datum Geodetic Compiled By F.H. Checked By B.L.

SAMPLE CONDITION

 UNDISTURBED
 FAIR
 DISTURBED
 LOST

SAMPLE TYPE

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

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST
 C. SOIL SHEAR STRENGTH LBS/SQ.FT.
 W.L. WATER LEVEL IN CASING
 W.T. GROUND WATER TABLE IN SOIL
 W.T.P.L. WETTER THAN PLASTIC LIMIT
 D.T.P.L. DRIER THAN PLASTIC LIMIT

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	Natural Moisture Content	WATER LEVELS & REMARKS
Ground surface			0' 0"						
Topsoil	Dark brown to black	595.0	0' 8"		1	C.S.			Moist.
Sandy loam	Brown		2' 7"		2	S.S.	7	28.6%	Moist.
Silty clay some organic matter, pockets of wet sandy gravel.	Layers of grey & brown	Firm	5' 0"						W.T.P.L.
As above	Layers of grey & brown	Stiff to very stiff	6' 2"		3	S.S.	16	26.5%	W.T.P.L.
Clayey till	Brownish-grey	Very stiff to hard			4	S.S.	28	18.6%	D.T.P.L.
			9' 9"						
Clayey till, some stones	Grey	Very stiff			5	S.S.	18	17.6%	W.T.P.L.
As above	Grey				6	2" S.L. Tapped	18.8%		At 13' 6" seam of grey silty fine sand
			15' 0"						
As above	Grey	Stiff			7	S.S.	12	17.0%	W.T.P.L.
As above	Grey				8	2" S.L. Tapped	15.5%		
			20' 0"						
As above, pebbles.	Grey	Very stiff to hard			9	S.S.	30	18.3%	D.T.P.L.
									Washwater used after 21' 0" depth.
			25' 0"						
As above	Grey	Very stiff to hard			10	S.S.	30	21.5%	D.T.P.L.
			30' 0"						
As above	Grey	Very stiff			11	S.S.	23	16.7%	W.T.P.L.
			35' 0"						
As above	Grey	Very stiff			12	S.S.	27	12.1%	Seam of sandy silt.
			38' 4"						
			40' 0"						
Sandy silt	Grey	Dense			13	S.S.	35	13.1%	Just moist.
			45' 0"						
Sandy till (Sandy silt some clay content)	Grey	Dense			14	S.S.	35	10.5%	layers of grey fine sand.
			50' 0"						Stiffened at 50' 9"
As above	Grey	Dense to very dense	51' 0"		15	S.S.	50		
									Hole terminated at 51' 0"