

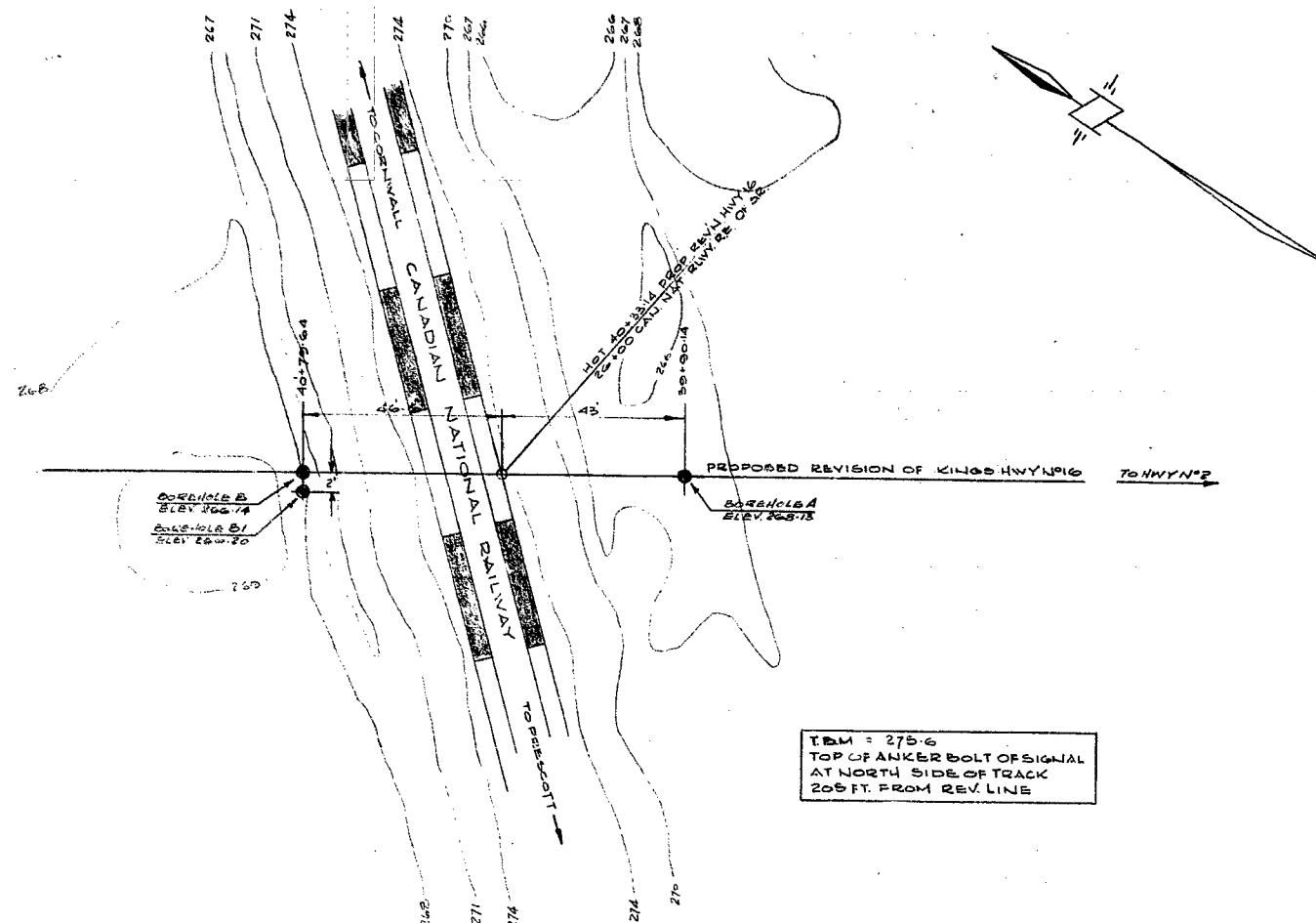
#59-F-209C

WP #217-58

HWY #16 &

C.N.R. CROSSING

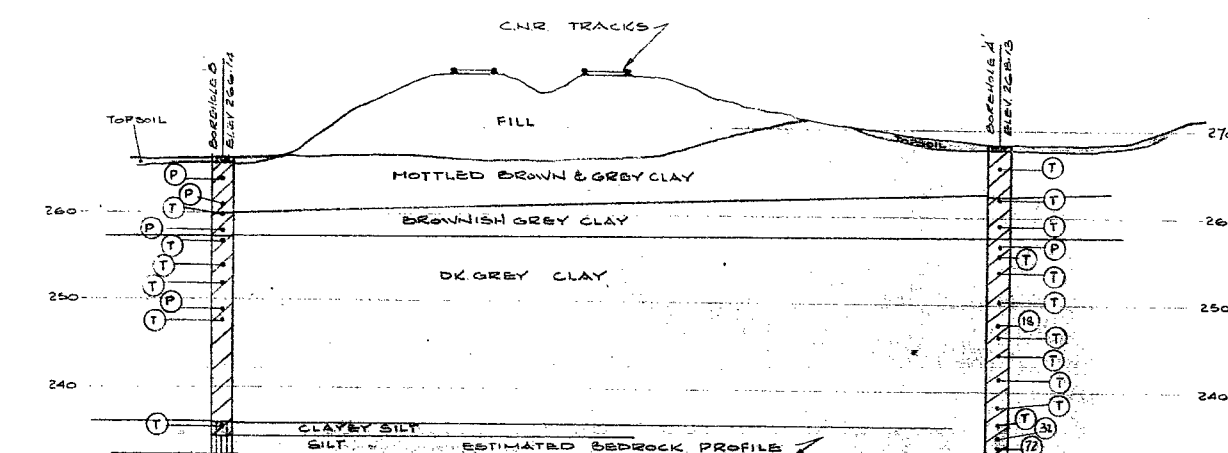
TO OTTAWA



- LEGEND**
- BOREHOLE
 - (B) BLOWG/FOOT
 - (T) TAPPED OR PUSHED

TBM = 275.6
TOP OF ANKER BOLT OF SIGNAL
AT NORTH SIDE OF TRACK
205 FT. FROM REV. LINE

SITE PLAN
SCALE 20:1



SECTION ALONG C OF PROPOSED HWY.
SECTION SCALES VERT. 10:1
HOR. 10:1

NOTE:
SEE BOREHOLE LOGS FOR COMPLETE
SOIL DETAILS
STRATIFICATION OF SOIL BETWEEN
BOREHOLES HAS BEEN ESTIMATED
AND MAY DIFFER FROM THAT SHOWN



e.m. peto & associates Ltd.
SOIL SITE INVESTIGATION
AT
C.N.R. OVERHEAD HWY 16
EDWARDSBURG TWP. JOHNSTOWN
FOR
DEPT OF HIGHWAYS OF ONTARIO
OUR FILE NO. 59129 DATE 14 JULY/59
CLIENTS PLAN No. E3554-1 PLOT C J.V.

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 59129

850 roselawn avenue,
TORONTO 19, ONTARIO.
RUssell 1-4955.

August 8th, 1959.
51-F-2690

The Department of Highways of Ontario.
Soil & Foundation Engineering Branch,
Downsview, Ontario.

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

Re: Soil Site Investigation
Highway # 16 - CNR Crossing
Edwardsburg Twp. W.P. 217-58

Dear Sirs:

We have pleasure in submitting herewith ten copies of our report. This report covers the additional soil investigation at the above mentioned site, as requested and authorized by Mr. Soderman's letter of July 2nd, 1959.

Apart from the report on the shear strength characteristics of the sub-soil, a slope stability analysis is included, carried out in accordance with discussions and conclusions reached at a meeting in the offices of the Soil and Foundation Engineering Branch, between your Mr. L. G. Soderman and our Mr. B. Lewicki.

If any additional information is required in connection with this report, we shall be pleased to be of further service.

Yours very truly,

E. M. PETO ASSOCIATES LTD.



E. M. Peto, P. Eng.

BL/sam

THE DEPARTMENT OF HIGHWAYS OF ONTARIO

SOILS REPORT

for

HIGHWAY # 16 - C.N.R. CROSSING
EDWARDSBURG TWP. - W.P. 217-58

August 1959.

Job No. 59129

Client's Ref. No.

Date August 6th. 1959.

Report on

SOIL SITE INVESTIGATION

HIGHWAY # 16 - C.N.R. CROSSING

EDWARDSBURG TWP. - W.P. 217-58

for

THE DEPARTMENT OF HIGHWAYS, ONTARIO.

1. INTRODUCTION:

We were requested by a letter from Mr. L. G. Soderman, dated July 2nd, 1959 - to conduct an additional investigation at the above site.

The previous investigation (Report No. 5920, March 1959) did not include the in-situ shear strength measurements, and due to the character of the encountered cohesive deposits, there was no evident conformity in the results of the laboratory shear strength data. Thus, in order to define more exactly the strength - depth characteristics of the sub-soil, this supplementary investigation was called for.

Two additional boreholes were put down at the locations shown on the attached site plan. The sampling, and in-situ shear strength measurements, were nearly continuous.

2. THEORETICAL CONSIDERATIONS:

From the results of investigations in a similar material (marine clays) (see: "Geotechnical Properties of Leda Clay in the Ottawa Area" by W. J. Eden and C. B. Crawford - Soil Mechanics Section, Division of Building Research, Ottawa in Proceedings 4th. Int. Conf. on Soil Mechanics and Foundation Engineering, London 1957, I, 1a/6) it may be seen that the most consistent results of shear strength were obtained from field vane tests. The laboratory shear strength results, due to the sensitivity and the structure of the clay, as well as the present method of sampling, were found to be on the low side. Possibly, by more refined sampling methods - the laboratory results might be improved, but it is felt that even withdrawing the sample from the virgin environment results in changes in the stresses which affect to a considerable degree the results obtained in later laboratory tests.

THEORETICAL CONSIDERATIONS: Cont'd.

The findings of this report detailed in outline below greatly substantiate the conclusions reached by Messrs: W. J. Eden and C. B. Crawford, i. e.:

1. The marine clays are of a "fissured" nature, perhaps better referred to as being of a "friable" character in the case of this investigation. A hypothesis put forward by A. Casagrande of "mixed structure" (A. Casagrande: "The structure of clay and its importance in Foundation Engineering" - J. Boston Soc. C. Eng. - April 1932), i. e. a mixture of individual silt particles and the flocculated colloidal particles in various degrees of consolidation, which might have been caused by some electrolytic process, seems to be the best explanation for the structure of the clay encountered on this site.
2. The clay was found to be slightly precompressed.
3. The in-situ shear strength tests (vane tests) and to some degree, laboratory vane tests, gave the most consistent shear strength results.
4. The lowest values of shear strength were obtained from unconfined compression tests, and the maxima values from in-situ vane tests, with the results of the triaxial tests lying between these two.
5. There is no apparent general relationship between the shear strength and the depth.

3. PROGRAMME OF WORK:

- July 6th. 1959 - Field crew and equipment travelled to the site from Toronto.
- July 7th. 1959 - Testholes and elevations located by Field Engineer; Testhole # B commenced.
- July 8th. 1959 - Testhole # B completed.
- July 9th. 1959 - Testhole # B-1 completed, equipment moved to testhole # A.
- July 10th. 1959 - Testhole # A completed
- July 11th. 1959 - Field crew and Engineer moved off the site.

4. GENERAL INFORMATION:

1. The testholes were driven using 4" pipe casing without the use of wash water.
2. The sampling and testing of the testholes was performed in accordance with the attached sketch of "Scheme of Sampling and Shear Vane Testing."

4. GENERAL INFORMATION: Cont'd.

3. The details of sampling, field vane testing, the natural moisture contents and the water level readings are given in the attached borehole logs.
4. Due to the disturbance of the soil caused by a broken vane at a depth of 19'11" in testhole # B - an additional testhole was driven (Testhole # B-1), and samples taken to supplement the missing samples of testhole # B.
5. The location of the testholes is shown on the attached site plan, together with the assumed soil profile.
6. The vane used to determine in-situ shear strength is of 2" O.D. with an Area Ratio of 16.4%.
7. All elevations referred to in the report are in respect to D. H. O. Bench Mark located to the left of station 1030 + 63 of Highway # 401 (Elevation 274.34).
8. The results of the in-situ tests and the laboratory tests are given in tabular and graphical form in Appendix I.
9. The table of the average shear - Strength values, the proposed and the assumed values used in the slope stability analysis are shown in tabular form in Appendix I.
10. The natural moisture contents versus elevation (tabular and graphical form) and other properties (void ratio, bulk density) versus elevation are shown in Appendix I.
11. The slope stability analysis is given in Appendix II.

5. SOIL CONDITIONS:

This supplementary investigation confirms for all practical purposes the soil conditions found in the earlier investigation at this site (Report No. 5920).

Underlying a dark brown to black organic top-soil, there is a stratum of mottled grey-brown silty clay, extended from 6'2" to 6'3" below the existing grade.

From present knowledge of marine clay deposits it is known that generally the upper layers of these clays have been subjected to some desiccation. This condition was found to exist at the above site.

SOIL CONDITIONS: Cont'd.

The following are the average values for the mottled grey-brown clay layer:

Wet Density $\gamma_w = 116.4$
Void ratio $e = 0.987$
Degree of saturation $S = 100\%$
Shear Strength $c = 1340$ p.s.f.
Sensitivity 6.5

Beneath the upper, desiccated brown-grey clay an intermediate layer, brownish-grey in colour, was encountered. It represents a transition zone between the desiccated stratum and the dark grey marine clay which has its upper boundary at an average elevation of 257.0.

The average values of this intermediate layer are as follows:

Wet Density γ_w = 117.5
Void ratio e = 0.950
Sensitivity 6.5 - 5.0
Degree of Saturation S = 100%
Shear Strength c = 1425 p.s.f.

From elevation 257 to 238, a dark grey coloured clay was penetrated. As mentioned previously in the report it is a marine clay, having a friable or "blocky" structure.

During the laboratory shear strength testing no definite pattern in failure of specimens could be observed. This characteristic was also observed during the investigation of marine clays at the Grass River Lock (see "Physical Properties of Marine Clays and their effect on the Grass River Lock Excavation" - by R. H. Burke and W. L. Davies, proceedings 4th. Int. Conf. on Soil Mechanics and Foundation Engineering, London 1957, II, 6/9).

The average values for the dark grey clay were found to be as follows:

Wet Density 121.0 lb/cu. ft.
Void ratio 0.855
Degree of Saturation S = 100%
Shear Strength: from elevation 257 to 250 c = 1675 p.s.f.
from 250 to 245 c = 2000 p.s.f.
from 245 to 238 c = 1800 p.s.f.

Just before bedrock was reached a 5 feet deep layer of grey clayey silt and silt was found.

SOIL CONDITIONS: Cont'd.

The average values for this stratum may be assumed to be as follows:

| | | | |
|----------------|------------|---|------------------|
| Wet Density | γ_w | = | 134.4 lb/cu. ft. |
| Void ratio | e | = | 0.507 |
| Shear Strength | c | = | 2450 p.s.f. |

6. SHEAR STRENGTH - DEPTH RELATION:

To determine the shear strength - depth relationship, apart from the field vane tests, numerous laboratory shear tests were conducted. The results of the shear strength testing is given in tabular and graphical form in Appendix I.

a) Unconfined Compressive Tests:

From a number of tests (9 tests) - five tests were selected as being representative, as it was felt that the results obtained from the other 4 tests were affected by some sample disturbance, and thus were not characteristic.

From the attached graph it may be seen that the shear strength does increase with depth in the stratum of dark grey clay. Nearly constant values were observed for the layer of mottled grey-brown clay and the intermediate layer of brownish grey clay. The values for the upper two layers were between 1165 and 1410 p.s.f. with an average value of 1275 p.s.f. for the mottled grey-brown clay, and 1300 p.s.f. for the intermediate layer. An increase, from about 1400 p.s.f. at elevation 257, to 2000 p.s.f. at elevation 240 was recorded according to the results of the unconfined compressive tests conducted on samples of dark grey clay.

b) Triaxial Shear Tests:

The triaxial shear tests were of the immediate (undrained) type, which are usually carried out for conditions where critical stresses develop in a saturated soil mass too rapidly for the moisture content to change appreciably. The results of these tests are applied in the case of " $\phi = 0$ " analysis of slope stability.

The details of triaxial shear tests are given in Appendix I, together with the graphical representation of the obtained values versus elevation.

A definite increase of shear strength with depth could be observed here. The relationship was nearly linear. The minimum value of 1310 p.s.f. was obtained at elevation 262.38 increasing to 3160 p.s.f. at elevation 235.70.

SHEAR STRENGTH - DEPTH RELATION: Cont'd.

The assumed constant average values were as follows:

| | | |
|---|-------------|-------------|
| Mottled brown and grey clay: | 1400 p.s.f. | |
| Brownish-grey clay: | 1550 p.s.f. | |
| Dark grey clay: from elevation 257 to 250 | | 1950 p.s.f. |
| from elevation 250 to 236 | | 2400 p.s.f. |
| Clayey silt and silt: | 2850 p.s.f. | |

c) Laboratory Shear Vane Tests:

A great number of laboratory shear vane tests were performed, and the details are given in tabular form in Appendix I.

The average values of each test group were then plotted in graphical form versus elevation.

From the graph (Appendix I) of the results of shear vane tests versus elevation it may be seen that there is no definite strength-depth relation of the soil. As mentioned earlier in this report, this finding agrees with the conclusions reached by other investigators. A wide scatter of the results, (even after the elimination of "wild results", due to sample disturbance, or other reasons), was observed. The general trend was towards a constant value of shear strength, throughout the depth of the investigated soil, with possibly a small increase with depth.

The values were found to range between 1500 and 2700 p.s.f. More consistent results were observed in the upper strata, where the scatter was less pronounced. One outstanding result at elevation 243.63, with a value of 1520 p.s.f., is specifically noted. This result was obtained at testhole A and comparing the natural moisture contents at the same depth, it may be seen that there is a definite increase of moisture content, with values over 41%, compared with 29% as obtained in the other borehole. Due to this increased moisture content, resulting from water in the seam of gravel and stones encountered around this depth, the shear strength decreased considerably.

This decreased shear strength, of course, influenced the recommended shear strength values as given in the table in Appendix I, where, for the layer between elevations 245 and 236, an average value of 1800 p.s.f. is given.

d) Field Vane Tests:

The most reliable results, it is felt, were produced by field vane shear tests. A number of vane tests at lower elevations could not be taken since considerable difficulty was experienced in pushing the vane into the soil, and on occasions when it was felt that such a procedure might disturb the soil, the test was abandoned. Nevertheless, the results of the in-situ vane tests do complement the results of the laboratory shear vane tests, as may be clearly seen from the attached graph.

SHEAR STRENGTH - DEPTH RELATION: Cont'd.

Because of the number of readings obtained from remoulded samples, a plot of sensitivity versus elevation was considered feasible. This plot is given together with the results of tests on undisturbed field and laboratory shear vane tests in Appendix I. It may be seen that the sensitivity varies with the depth and has a value of between 3 and 7.

e) Average Shear Strength from Various Tests:

It may be easily seen that, as mentioned previously, the unconfined compressive tests give the lowest shear strength values, next followed by the results of the triaxial shear tests, with the results from the field and laboratory shear vane tests giving the maximum values. This confirms the findings reached by many other investigators of marine clays.

In order to arrive at recommended values for each individual stratum, an average value between the unconfined compressive and triaxial shear test results was calculated. (see Appendix I). It may be seen that the values arrived at in this manner are still lower than the average values obtained from in-situ and laboratory vane tests. The actual, true values may therefore lie between the values as obtained from triaxial shear tests, and vane shear tests. To carry out the slope stability analysis, the following average shear values (constant throughout the depth of individual layers) were taken:

| | |
|-----------------------------|-------------|
| Mottled brown and grey clay | |
| Brownish-grey clay | 1800 p.s.f. |
| (Elevation 267 - 257) | |
| Dark grey clay | 2000 p.s.f. |
| (Elevation 257 - 238) | |

7. SLOPE STABILITY ANALYSIS:

The following slope stability analysis was made according to the " $\phi = 0$ case". This method although producing good results has the following limitations:

1. The angle of shearing resistance for fully saturated clays is zero only when there is no water content change under the applied stresses, i.e. no drainage will take place under the structure.
2. When fully saturated clays are tested with a constant moisture content, the angle of internal friction is not, in fact, zero, as the true friction angle of soil is never zero.
(see A. W. Skempton: "The $\phi = 0$ Analysis of Stability and its Theoretical Basis" - Proc. Second Int. Conf. on Soil Mechanics and Foundation Engineering, Rotterdam (1948) 1, 72;

A. W. Skempton and H. Q. Golder: "Practical examples of the $\phi = 0$ Analysis of Stability of Clays" - Proc. Second Int. Conf. on Soil Mechanics and Foundation Engineering, Rotterdam (1948) 2, 307.

SLOPE STABILITY ANALYSIS: Cont'd.

D. W. Taylor - "Fundamentals of Soil Mechanics" Page 473 - 474.

3. By the $\phi = 0$ analysis, the applied stresses at failure are given correctly but as the shear surface is controlled by the true angle of internal friction - the analysis will not lead, in general, to a correct prediction of the failure shape.

For the following slope stability analysis, the following assumptions have been made:

1. The potential sliding surface is of cylindrical shape.
2. The analysis is two - dimensional.
3. The shear strength is constant within each layer.
4. At the moment of failure the shear strength is completely mobilized at every point along the sliding surface. However the shear strength in fill is neglected ($c = 0$ $\phi = 0$).
5. The factor of safety is defined as the ratio between the available shear strength and the average shear stress necessary for the equilibrium along the critical surface of sliding.
6. Following are the densities and shear strength values which were used in the stability computation.
 - a) Fill (from elevation 302 to 267)

$\gamma = 135$ lb/cu. ft.
 $c = 0$
 - b) Mottled grey and brown clay and (from elevation 267 to 257) brownish-grey clay

$\gamma = 117$ lb/cu. ft.
 $c = 1800$ p.s.f.
 - c) Dark grey clay (from elevation 257 to 238)

$\gamma = 121$ lb/cu. ft.
 $c = 2000$ p.s.f.
7. The typical cross section is shown on the attached drawing in Appendix II.
8. Failure is expected to occur between the dark grey clay and clayey silt. (Base-failure).
9. The slope stability investigation was limited to an embankment having a slope of 2 horizontal to 1 vertical.

SLOPE STABILITY ANALYSIS: Cont'd.

For all 9 different positions of sliding, surfaces were assumed with the centers of rotation marked 0₁ to 0₉. Then with the values of the factor of safety for each assumed sliding surface, contourlines of Factor of Safety were drawn (lines of equal values of the factor of safety) (see Appendix II). It was found that the center of rotation of the critical center is represented by 0₅. This graphical solution of the position of the critical center of rotation agreed with the calculated one according to the data presented by N. Janbu - "Stability Analysis of Slopes with Dimensionless Parameters".

For the critical circle:

Factor of Safety: $F_s = 1.942$

Radius of Slip Circle $R = 92$ feet.

8. CONCLUSIONS:a) Embankment:

1. The approach embankments may be constructed with side slopes of 2 horizontal to 1 vertical, although the factor of safety in this instance then becomes slightly less than 2. In our previous report (No. 5920) we recommended using side slopes of 3 horizontal to 1 vertical. We feel that the reasons for this disparity may be summarized as follows:-

- a) At the time of the first investigation we were concerned with the wide scatter of shear strength results obtained. We had had similar experiences of non uniformity of results on previous investigations in marine clays in the same general area for the Department of Highways, and others.

On those occasions it was agreed by all interested parties that the field shear vane tests always gave higher results than laboratory strength tests, and for reasons of prudence were, therefore, largely discounted. For this reason field shear vane tests were not taken on this site on the first occasion in mid winter. In addition, we naturally tended to use a conservative average for the shear strength obtained from the laboratory tests, because of the scattering of the results, particularly as the variation of results had been viewed by the Department, others, and ourselves with some concern, in the past.

- b) In the interim period, since the first investigation, we have carried out some experiments with laboratory and field shear vane equipment and found that by applying similar much reduced rates of strain, the vane results, (both field and laboratory) more closely coincided with laboratory strength tests. Accordingly, on our second visit to the site we applied our additional knowledge, and employed the field shear vane as much as conditions allowed.

CONCLUSIONS: Cont'd.

- c) The results of the second investigation were virtually as widely scattered as before, and again the field and laboratory shear vane tests were higher than the unconfined and triaxial tests, but on this occasion in spite of the great similarity to previous results we can justify the acceptance of the less conservative results without hesitation for the following reasons.
- 1.) We accept the results of both the field and laboratory shear vane tests, because of our conviction that the true shear strengths lie somewhere between the laboratory strength tests and field shear vane results at greatly reduced rates of strain.
 - 2.) When almost identical results were obtained on this occasion both in field and laboratory we felt that there must be some reason for the lack of uniformity. A considerable amount of research by various members of our staff elicited the fact that such scattering in this type of material was not unusual, and that a number of learned papers had been written on the subject by well known authorities. These papers have been touched upon in the report. On the basis of our additional findings, and the technical evidence that we have found to support the adoption of higher average strength we can now recommend the adoption of the steeper side slope. Some settlement due to the weight of the embankment should be anticipated, thus a stage construction of the embankment as recommended in the previous report (No. 5920) should be followed.
 - 3.) The recommendations of previous report (No. 5920) pertaining to drainage should be carefully considered.

b) Overpass Structure

1. If a pile foundation is used, then we would recommend the use of steel H piles driven to refusal depths in order to avoid excessive disturbance to the clay. Even with the use of steel H piles, we would recommend
 - a) That the structure be built before the approach embankment.
 - b) That a reasonable period of time should be allowed to elapse (commensurate with the Department's past experience with this problem) before proceeding with the placing of any embankment fill adjacent to the bridge abutments, in order to allow the soil to regain most of its original strength.

CONCLUSIONS: Cont'd.

2. Should the Highway programme for this structure not permit any appreciable delay period, then consideration may be given to the use of a caisson foundation to bedrock with caissons excavated perhaps by California auger. However, it will be necessary to stabilize or freeze the water bearing seam specifically noted at hole A. The test holes did not stand open without casing for any appreciable time, and tended to collapse at the water bearing or seepage layers.

E. M. PETO ASSOCIATES LTD.



E. M. Peto P, Eng.

BL/sam

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Hwy.# 16 & C.N.R. Crossing
 Job Name Edwardsburg Twp. Job No. 59129 Borehole No. "A"
 Client Dep't. of Highways of Ontario. Casing 4" Pipe Boring Date July 10th. 1959.
 Datum Geodetic Compiled By B.L. Checked By

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 | Dk. Brown to Black | | 0' 6" | | | | | | |
| Clay | Mottled brown & grey | | 5' 0" | | | 3"S.L. Tapped | | | |
| | | | 6' 2" | | | V.T. V.T. | | | |
| Clay | Brownish-grey | | 10' 0" | | | 2"S.L. Tapped | 31.1% | | |
| | | | 10' 4" | | | V.T. 3"S.L. Tapped | 38.4% | | |
| Clay | Dk. Grey | | 15' 0" | | | V.T. 2"S.L. Pushed Tapped | 35.9% | | |
| At 12'6" seam (2" thick) of grey fine silty sand. | | | | | | | | | |
| At 13'6"-2" thick seam of grey silt. | | | | | | | | | |
| Clay | Dk. Grey | | 20' 0" | | | 2"S.L. Tapped | 29.2% | | |
| At 19'10" seam of gravel & large stones. (Boulder?) | | | | | | | | | |
| Clay | Dark grey | | 25' 0" | | | 3"S.T. 18 2"S.L. Tapped | 24.4% | | |
| | | | 30' 0" | | | 3"S.L. Tapped | 42.7% | | |
| | | | 35' 0" | | | 2"S.L. Tapped | 24.1% | | |
| | | | 36' 0" | | | 3"S.L. Tapped | 20.2% | | |
| | | | | | | 2"S.L. Tapped | | | |
| Refusal (Bedrock) | | | | | | | 32 | | From 33'0" Dutch Cone penetration test. |
| | | | | | | | 72 | | |
| | | | | | | | 112 | | |





Hole terminated at 36'0"

WATER LEVEL READINGS.

| July 10th. 1959. | Time | Depth of Casing | Depth of Hole | Depth of Water | |
|------------------|-----------|-----------------|---------------|----------------|--|
| July 10th. 1959. | 9.30 am. | 2'3" | 5'0" | 4'10" | |
| | 11.20 am. | 5'0" | 10'9" | 3'11" | |
| | 12.10 pm. | 10'0" | 13'6" | None | After Bailing Out. |
| | 12.30 pm. | 10'0" | 15'7" | 15'2" | |
| | 1.15 pm. | 10'0" | 15'7" | 12'4" | |
| | 4.50 pm. | 20'0" | 26'0" | 8'7" | Note: Water came up very fast from a depth of 20'0" where the seam of gravel and stones was encountered/ |
| | 4.55 pm. | 20'0" | 26'0" | 7'10" | |
| | 5.00 pm. | 20'0" | 26'0" | 7'10" | |
| | 6.45 pm. | 29'0" | 32'10" | 24'4" | After bailing out. |
| July 11th. 1959. | 6.45 am. | 29'0" | 36'0" | 2'2" | |
| | 8.05 am. | None | 36'0" | 4'1" | |
| | 10.35 am. | None | 36'0" | 2'5" | |

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

Hwy.# 16 & C.N.R.Crossing
Job Name Edwardsburg Twp. Job No. 59129 Borehole No. "B"
Client Dep't.of Highways of Ontario. Casing 4" Pipe Boring Date July 7th.& 8th. 1959.
Datum Geodetic. Compiled By B.L. Checked By

| SAMPLE CONDITION | | SAMPLE TYPE | | ABBREVIATIONS | |
|--|-------------|-------------|--------------------------------|---------------|--------------------------------|
|  | UNDISTURBED | A.S. | AUGER SAMPLE | V.T. | IN SITU VANE SHEAR TEST |
|  | FAIR | C.S. | CASING SAMPLE | C. | SOIL SHEAR STRENGTH LBS/SQ.FT. |
|  | DISTURBED | S.S. | 2" STANDARD SPLIT TUBE SAMPLE | W.L. | WATER LEVEL IN CASING |
|  | LOST | S.L. | SPLIT BARREL WITH LINERS | W.T. | GROUND WATER TABLE IN SOIL |
| | | S.T. | THIN-WALLED SHELBY TUBE SAMPLE | W.T.P.L. | WETTER THAN PLASTIC LIMIT |
| | | W.S. | WASH SAMPLE | D.T.P.L. | DRIER THAN PLASTIC LIMIT |
| | | R.C. | ROCK CORE | | |

| 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 | Dr. Brown to Black | 266.14 | 0'0" | | | | | | |
| Clay | Mottled brown & grey | | 5'0" | | | 3"S.L. Pushed | 18.4% | | |
| | | | 6' 3" | | | V.T. | | | |
| Clay | Brownish-grey | | 10'0" | | | 2"S.L. Tapped | 38.9% | | |
| | | | 15'0" | | | V.T. | | | |
| | | | 20'0" | | | 3"S.L. Pushed | 38.1% | | |
| | | | 25'0" | | | V.T. | | | |
| Clay | Dk. Grey | | 30'3" | | | 2"S.L. Tapped | 34.8% | | |
| | | | 31'10" | | | V.T. | | | |
| | | | 34'1" | | | 3"S.L. Tapped | 33.1% | | |
| | | | | | | V.T. | | | |
| | | | | | | Pushed | 32.3% | | |
| | | | | | | Tapped | | | |
| | | | | | | | | | Note: No sampling was performed from 19'11" to 30'0" depth as the sub-soil was disturbed by vane which broke down at 19'11" |
| Clayey silt | Grey | | 30'3" | | | 2"S.L. Tapped | 18.9% | | |
| Silt | Grey | | 31'10" | | | 3"S.L. | 19.3% | | Seepage observed at 32'0" depth. |
| Refusal (Bedrock) | | | 34'1" | | | | | | |

| Hole terminated at 34'1" | | | | |
|--------------------------|-----------|-----------------|---------------|---------------------------|
| WATER LEVEL READINGS. | | | | |
| DATE | TIME | DEPTH OF CASING | DEPTH OF HOLE | DEPTH OF WATER |
| July 7th. 1959. | 1.10 pm. | 5'0" | 8'3" | 6'2" |
| | 1.13 pm. | 5'0" | 8'3" | 5'4" |
| | 1.40 pm. | 5'0" | 8'3" | 0'10" |
| | 1.45 pm. | 5'0" | 8'3" | 0' 7" |
| | 1.50 pm. | 5'0" | 8'3" | 0' 5" |
| | 3.35 pm. | 5'0" | 10'10" | 0' 4" |
| | 3.45 pm. | 10'0" | 11' 0" | None |
| | 3.55 pm. | 10'0" | 13'11" | None |
| | 4.05 pm. | 10'0" | 15'4" | None |
| | 4.50 pm. | 10'0" | 15'4½" | 14'11½" |
| | 5.10 pm. | 10'0" | 17' 0" | None |
| | 7.55 am. | 15' 0" | 17'0" | 16' 9" |
| | 11.05 am. | 15'0" | 25' 0" | 13' 5" |
| | 11.10 am. | 15'0" | 25'0" | 12'0½" |
| | 11.15 am. | 15'0" | 25'0" | 11'1½" |
| | 11.19 am. | 15'0" | 25'0" | 10' 6" |
| | 11.31 am. | 15'0" | 25'0" | 8' 8" |
| | 1.15 pm. | 30'0" | 27'0" | 26'11" |
| | | | | 3.40pm. 30'0" 32'9" 31'0" |
| | | | | 5.00pm. None 32'9" 27'4" |

BOREHOLE LOG

Checked By

[illegible]

APPENDIX I

LABORATORY AND FIELD TEST RESULTS.

E. M. PETO ASSOCIATES LTD.

UNCONFINED COMPRESSION TEST DATA SHEET

File separated into 3

| Borehole Number | Depth Feet | Elevation Feet | M.C. Tin No. Nat. M. C. | Wet Density p. c. f. | Dry Density p. c. f. | Degree of Saturation % | Void Ratio, e | % Strain at Failure | u/c Shear Strength p. s. f. |
|--------------------|---------------|-------------------|----------------------------|-------------------------|-------------------------|---------------------------|---------------|------------------------|--------------------------------|
| A | 2'3"-3'7" | 265.05 | 40.4 | 114.2 | 81.5 | 100 | 1.076 | 20.0 | 1410 |
| A | 14'4"-15'7" | 253.30 | 36.4 | 120.8 | 88.4 | 100 | 0.907 | 20.0 | 1165 |
| A | 29'4"-30'8" | 238.30 | 26.3 | 133.5 | 105.8 | 100 | 0.596 | 13.3 | >1712 |
| B | 14'0"-15'4" | 250.97 | 33.2 | 125.0 | 93.7 | 100 | 0.800 | 8.3 | 1440 |
| B-1 | 20'4"-21'8" | 244.70 | 30.6 | 125.3 | 96.1 | 100 | 0.754 | 11.7 | 2130 |

E. M. PETO ASSOCIATES LTD.

TRIAxIAL SHEAR TEST DATA SHEET:

| Borehole Number | Depth Feet | Elevation Feet | M. C. Tin No. | | Wet Density p. c. f. | Dry Density p. c. f. | Degree of Saturation % | | Void Ratio, e | Triaxial Shear Strength p. s. f. | Angle of Internal Friction ϕ |
|--------------------|---------------|-------------------|---------------|---------|-------------------------|-------------------------|---------------------------|-------|---------------|-------------------------------------|--------------------------------------|
| | | | M. C. | Tin No. | | | | | | | |
| A | 5'0"-6'10" | 262.38 | 33.5 | | 118.6 | 89.0 | 100 | 0.890 | | 1310 | 0 |
| | | | 34.4 | | 118.0 | 87.8 | 99.5 | 0.920 | | | |
| A | 11'0"-12'10" | 256.88 | - | | 115.6 | - | - | - | | 1370 | 0 |
| A | | 255.88 | 37.7 | | 117.6 | 85.5 | 100 | 0.970 | | 2055 | 0 |
| | | | 38.2 | | 116.8 | 84.5 | 100 | 0.990 | | | |
| A | 17'0"-18'10" | 249.88 | 31.8 | | 122.0 | 92.7 | 100 | 0.820 | | 2520 | 0 |
| A | 21'2"-23'2" | 246.21 | 25.8 | | 127.5 | 101.5 | 100 | 0.660 | | 2305 | 0 |
| | | | 26.8 | | 125.2 | 97.4 | 100 | 0.730 | | | |
| A | | 245.71 | 23.6 | | 120.5 | 97.7 | 100 | 0.725 | | 1990 | 0 |
| A | 31'-32'10" | 235.88 | 20.7 | | 136.0 | 112.8 | 100 | 0.492 | | 2455 | 0 |
| | | | 19.8 | | 134.5 | 112.4 | 100 | 0.498 | | | |
| B | 5'0"-6'8" | 259.89 | 34.7 | | 121.5 | 90.5 | 100 | 0.860 | | 1740 | 0 |
| | | | 34.4 | | 120.0 | 89.4 | 100 | 0.878 | | | |
| B | 11'0"-12'10" | 254.14 | 38.0 | | 116.4 | 84.5 | 100 | 1.010 | | 1505 | 0 |
| | | | 37.5 | | 115.3 | 84.0 | 99.8 | 1.010 | | | |

E. M. PETO ASSOCIATES LTD.

TRIAXIAL SHEAR TEST DATA SHEET:

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| Borehole Number | Depth Feet | Elevation Feet | M. C. Tin No. Nat. M. C. | Wet Density p. c. f. | Dry Density p. c. f. | Degree of Saturation % | Void Ratio, e | Triaxial Shear Strength p. s. f. | Angle of Internal Friction ϕ |
|--------------------|---------------|-------------------|-----------------------------|-------------------------|-------------------------|---------------------------|---------------|-------------------------------------|--------------------------------------|
| B | 17'0"-18'8" | 248.14 | 33.5 | 121.2 | 90.8 | 100 | 0.858 | 2465 | 0 |
| | | | 31.8 | 121.0 | 91.8 | 100 | 0.835 | | |
| B-1 | 23'0"-24'8" | 241.78 | 26.4 | 126.0 | 99.7 | 100 | 0.690 | 2620 | 0 |
| | | | 26.3 | 125.0 | 99.0 | 100 | 0.700 | | |
| B-1 | 29'0"-31'0" | 235.70 | 19.3 | 135.5 | 113.5 | 100 | 0.483 | 3160 | 6 |
| | | | 21.2 | 135.6 | 112.0 | 100 | 0.510 | | |

IN SITU SHEAR VANE TEST:

| Test Hole | Depth | Elevation | Test Cond. | Tongue Reading | Shear Strength p. s. f. | Sensitivity |
|-----------|--------------|-----------|------------|----------------|----------------------------|-------------|
| A | 9'10"-10'3" | 258.01 | UND | 347 | 1770 | 3.85 |
| | | | REM | 90 | 460 | |
| | 10'4"-10'9" | 257.59 | UND | 413 | 2110 | 6.26 |
| | | | REM | 66 | 337 | |
| | 13'0"-13'5" | 254.92 | UND | 533 | 2720 | - |
| | | | REM | - | - | |
| B | 3'6"-4'1" | 252.43 | UND | 480 | 2460 | 7.20 |
| | | | REM | 67 | 342 | |
| | 4'4"-4'9" | 261.60 | UND | 493 | 2520 | 7.04 |
| | | | REM | 70 | 358 | |
| | 9'7"-10'0" | 256.35 | UND | 348 | 1780 | 3.54 |
| | | | REM | 96 | 490 | |
| | 10'0"-10'5" | 255.93 | UND | 387 | 1980 | 4.17 |
| | | | REM | 93 | 475 | |
| | 10'5"-10'10" | 255.51 | UND | 355 | 1820 | 3.39 |
| | | | REM | 105 | 537 | |
| | 13'0"-13'5" | 252.93 | UND | 633 | 3230 | 3.65 |
| | | | REM | 173 | 885 | |
| | 13'6"-13'11" | 252.43 | UND | 547 | 2800 | 3.17 |
| | | | REM | 173 | 885 | |
| | 16'0"-16'5" | 249.93 | UND | 480 | 2460 | 4.50 |
| | | | REM | 107 | 546 | |
| | 16'6"-16'11" | 249.43 | UND | 467 | 2390 | 3.64 |
| | | | REM | 129 | 659 | |
| | 19'0"-19'5" | 246.93 | UND | 647 | 3300 | 2.24 |
| | | | REM | 290 | 1480 | |
| | 19'6"-19'11" | 246.43 | UND | 787 | 4020 | |

LABORATORY SHEAR VANE TEST RESULTS

| Test Hole | Depth in ft. | Elevation | Shear Strength in p. s. f. | | Sensitivity | Average Values | | Sensitivity |
|-----------|--------------|-----------|----------------------------|-----------|-------------|----------------------------|-----------|-------------|
| | | | Undisturbed | Remoulded | | Shear Strength Undisturbed | Remoulded | |
| A | 2'3"-3'7" | 265.54 | 1520 | - | | | | |
| | | | 1890 | 278 | 6.85 | | | |
| | | | 2140 | 220 | 9.72 | | | |
| | | | 1870 | - | - | | | |
| | | | 2220 | - | - | 2350 | 415 | 5.65 |
| | | | 2200 | - | - | | | |
| | | | 2575 | - | - | | | |
| | | | 2770 | - | - | | | |
| | | | 2950 | 550 | 5.4 | | | |
| | | | 2670 | 490 | 5.5 | | | |
| | 8'4"-9'7" | 259.63 | 3025 | 540 | 5.6 | | | |
| | | | 1940 | - | - | | | |
| | | | 2030 | - | - | 2090 | - | - |
| | | | 2310 | - | - | | | |
| | 14'4"-15'7" | 252.42 | 1780 | - | - | | | |
| | | | 1720 | - | - | | | |
| | | | 2400 | 800 | 3.0 | | | |
| | | | 2570 | - | - | | | |
| | | | 2550 | - | - | | | |
| | | | 2200 | 700 | 3.15 | 2160 | 650 | 3.32 |
| | | | 1850 | 625 | 2.96 | | | |
| | | | 2120 | 730 | 2.90 | | | |
| | | | 2120 | 540 | 3.93 | | | |
| | | | 2400 | 620 | 3.87 | | | |
| | 23'4"-24'8" | 243.63 | 2000 | 540 | 3.71 | | | |
| | | | 1450 | 250 | 5.80 | | | |
| | | | 1425 | 200 | 7.13 | 1520 | 240 | 6.35 |
| | | | 1700 | 270 | 6.30 | | | |
| | 29'4"-30'8" | 237.23 | 1800 | - | - | | | |
| | | | 1770 | 325 | 5.45 | | | |
| | | | 2320 | 400 | 5.80 | | | |
| | | | 1700 | 320 | 5.32 | 1965 | 369 | 5.32 |
| | | | 2200 | 400 | 5.50 | | | |
| | | | 2000 | - | - | | | |
| | | | 2000 | - | - | | | |
| | | | 2000 | - | - | | | |

LABORATORY SHEAR VANE TEST RESULTS:

| | | | | | | | | |
|-----|-------------|--------|------|-----|------|------|-----|------|
| B | 2'3"-3'6" | 263.72 | 2050 | 400 | 5.13 | | | |
| | | | 1700 | 300 | 5.68 | | | |
| | | | 1800 | 280 | 6.44 | 2010 | 325 | 6.20 |
| | | | 2470 | 320 | 7.75 | | | |
| | | | 2420 | 500 | 4.84 | | | |
| | 8'3"-9'7" | 256.72 | 2900 | 400 | 7.25 | | | |
| | | | 2750 | 500 | 5.50 | 2320 | 420 | 5.53 |
| | | | 1750 | 350 | 5.00 | | | |
| | | | 1780 | 350 | 5.10 | | | |
| | | | 2800 | 600 | 4.67 | | | |
| B-1 | 31'6"-32'8" | 234.14 | 3850 | 430 | 6.63 | 2670 | 516 | 5.17 |
| | | | 3000 | - | | | | |
| | | | 2030 | - | | | | |
| | | | 3070 | 920 | 3.34 | | | |
| | | | 2750 | - | | | | |
| | 20'4"-21'8" | 245.53 | 3250 | - | | | | |
| | | | 2600 | 720 | 3.61 | | | |
| | | | 2670 | - | | 2660 | 720 | 3.41 |
| | | | 2325 | - | | | | |
| | | | 2470 | 700 | 3.53 | | | |
| | | | 2400 | | | | | |
| | | | 2350 | | | | | |

AVERAGE SHEAR STRENGTH VALUES FROM VARIOUS TEST:

| Soil Stratum | Elevation | Average Shear Strength in p. s. f. | | | Recommended Shear Strength in p. s. f. | Assumed Shear Strength in Stability Analysis in p. s. f. |
|------------------------------|-----------|------------------------------------|----------|-------------------|--|---|
| | | Unconfined | Tetaxial | Field & Lab. Vane | | |
| Mottled brown & grey clay | 267.0 | | | | | |
| | 261.0 | 1275 | 1400 | 2250 | 1340 | 1800 |
| Brownish- grey clay | 261.0 | | | | | |
| | 257.0 | 1300 | 1550 | 2100 | 1425 | |
| Dark grey clay | 257.0 | | | | | |
| | | 1400 | 1950 | 2450 | 1675 | |
| | 250.0 | | | | | |
| | | 1800 | 2400 | 2500 | 2000 | 2000 |
| | 245.0 | | | | | |
| | | 1900 | 2400 | 1800 | 1800 | |
| | 238.0 | | | | | |
| | | | | | | |
| Clayey silt and silt | 238.0 | | | | | |
| | | - | 2850 | 2450 | 2450 | |
| | 233.0 | | | | | |

TABLE OF THE NATURAL MOISTURE CONTENTS

Test Hole # A

| Elevation: | M/C | Note: |
|------------|------|-----------|
| 265.05 | 40.4 | uncon. T. |
| 264.90 | 45.8 | |
| 262.38 | 33.5 | Tr. T. |
| | 34.4 | Tr. T. |
| 261.30 | 31.1 | |
| 258.71 | 39.2 | Uncon. T. |
| 258.69 | 38.4 | |
| 256.88 | 27.4 | Tr. T. |
| 256.38 | 39.6 | Tr. T. |
| | 40.2 | Tr. T. |
| 255.88 | 37.7 | Tr. T. |
| | 38.2 | Tr. T. |
| 255.30 | 35.0 | |
| 253.30 | 36.4 | Uncon. T. |
| 252.55 | 43.1 | |
| 250.38 | 39.8 | Tr. T. |
| 249.88 | 31.8 | |
| 249.30 | 29.2 | |
| 246.71 | 28.5 | Tr. T. |
| 246.21 | 25.8 | Tr. T. |
| | 28.8 | Tr. T. |
| 245.71 | 23.6 | Tr. T. |
| 244.90 | 24.4 | |
| 244.30 | 21.0 | Uncon. T. |
| 243.46 | 42.7 | |
| 241.38 | 41.2 | Tr. T. |
| 240.55 | 33.9 | Tr. T. |
| | 38.3 | Tr. T. |
| 240.30 | 24.1 | |
| 238.30 | 26.3 | Uncon. T. |
| 237.46 | 20.2 | |
| 236.88 | 22.6 | Tr. T. |
| 236.38 | 19.4 | Tr. T. |

Test Hole # B

| Elevation: | M/C | Note: |
|------------|------|-----------|
| | 19.1 | Tr. T. |
| 235.38 | 20.7 | Tr. T. |
| 235.46 | 19.8 | Tr. T. |
| | | |
| 263.34 | 47.9 | Uncon. T. |
| 260.39 | 41.2 | Tr. T. |
| | 34.7 | Tr. T. |
| | 34.4 | Tr. T. |
| 259.47 | 38.9 | |
| 256.56 | 38.1 | |
| 254.39 | 38.0 | Tr. T. |
| | 40.3 | Tr. T. |
| 253.89 | 37.5 | Tr. T. |
| 253.31 | 34.8 | |
| 250.97 | 33.2 | Uncon. T. |
| 250.97 | 33.1 | |
| 248.41 | 33.5 | Tr. T. |
| 247.91 | 31.8 | Tr. T. |
| 247.41 | 32.3 | |
| 244.70 | 30.0 | Uncon. T. |
| 244.31 | 27.5 | |
| 243.31 | 28.5 | |
| 241.78 | 26.4 | Tr. T. |
| | 26.3 | Tr. T. |
| 241.31 | 28.1 | |
| 238.20 | 28.8 | Uncon. T. |
| 237.20 | 23.1 | |
| 236.48 | 19.3 | Tr. T. |
| 235.95 | 21.2 | Tr. T. |
| 234.89 | 19.3 | Tr. T. |
| | 19.4 | Tr. T. |
| 234.64 | 18.9 | |
| 233.47 | 19.3 | |

Test Hole # B

TABLE OF THE NATURAL MOISTURE CONTENTS Cont'd.

NOTE: Uncon. T. - Moisture contents as obtained from undisturbed samples used for unconfined compressive tests.

Tr. T. - Moisture contents as obtained from undisturbed samples used for triaxial shear tests.

ELEVATION

UNCONFINED COMPRESSIVE TEST RESULTS

267

260

247

SYMBOL

- TESTHOLE #A
- + TESTHOLE #B
- o TESTHOLE #B

1000

2000

3000

UNCONFINED SHEAR STRENGTH IN P.S.F.

Job No 54129

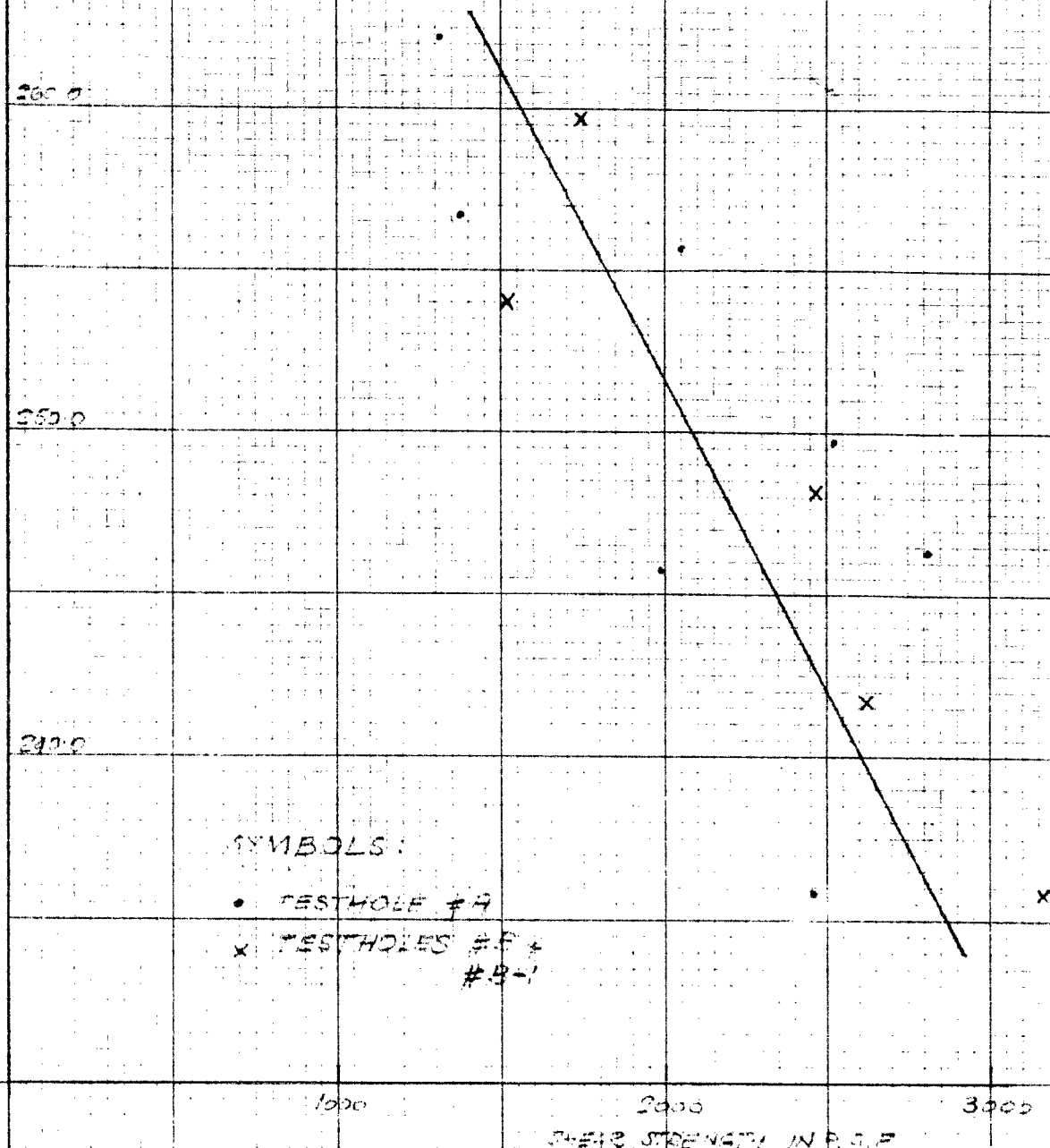
E.M. PETO ASSOCIATES

K&E
1/2 X 10 TO THE INCH 359-5
KEUFFEL & ESSER CO. MADE IN U.S.A.

10 X 10 TO THE INCH
KEUFFEL & ESSER CO.



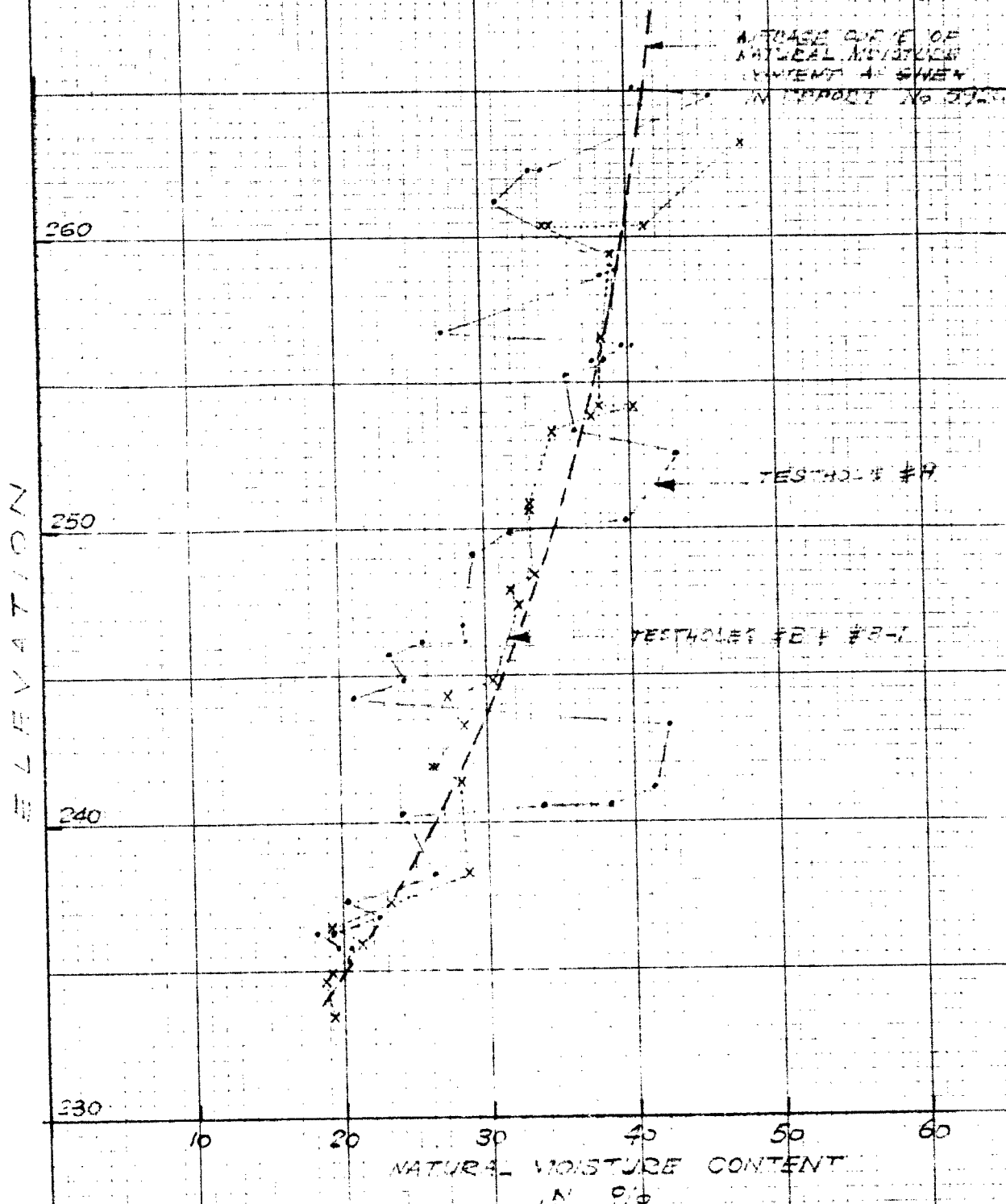
2000



103 No 59/20

E. M. PETO ASSOCIATES

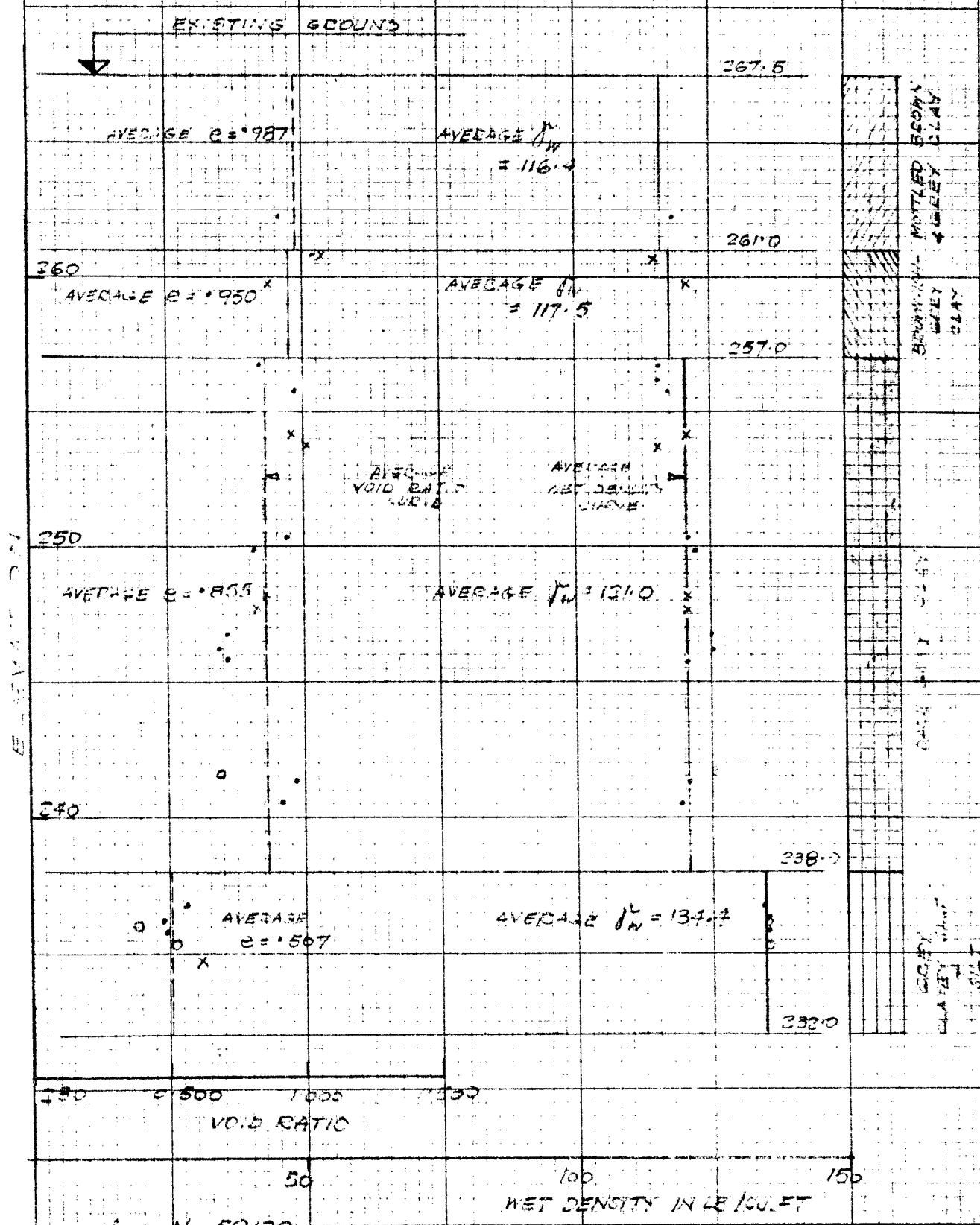
NATURAL MOISTURE CONTENT VS. ELEVATION



JOE No. 59129

E. M. PETO ASSOCIATES

VOID RATIO AND WET DENSITY VERSUS ELEVATION



JOB No 59129

E.M. PETS ASSOCIATES

APPENDIX II
SLOPE STABILITY ANALYSIS

Mr. D. G. Ramsay,

August 20, 1959.

Rd. Design Engineer.

SUPPLEMENTAL FOUNDATION REPORT

Materials & Research Section.

by: E. M. Peto & Associates.

Attention: Mr. H. D. McMillan

Re: Highway #16 - C.N.R. Crossing,
Edwardsburgh Twp. - W.P. 217-58.

Enclosed herewith is the supplemental foundation report prepared by E. M. Peto & Associates, dealing with the subsoil conditions and embankment slope design at the above site.

You will recall that the initial report prepared by E. M. Peto, recommended 3:1 side slopes with berms -- (Ref. my letter of July 2, 1959, to Mr. D. G. Ramsay). Our review of this initial report led us to conclude that the soil conditions reported were correct in calculations, but grossly in error with respect to engineering properties. Peto was authorized, under our supervision, to carry out additional borings and to accurately determine the strength characteristics to be used in embankment analysis.

Reference to the contents of the attached report shows that the strength of the subsoil is such that 2:1 slopes can be used for the approach fills and with these slopes the safety factor is not less than 1.9. A conventional safety factor of 1.3 based on total stress analysis, is considered adequate.

It is interesting to read the contents of this report in detail. The Consultants have indicated that the conclusions presented in their initial report are the result of a naturally occurring wide scatter in the strength characteristics of the subsoil -- that this scatter of results and low strength values used in their initial analysis is the result of damn poor sampling techniques, does not appear possible to them. In addition, it is interesting to note that the initial investigation cost the Department approximately \$6,000.00. If we had followed their recommendations and designed the embankment as they suggested, we could have spent another \$20,000.00 in quantities. We have taken this matter up in detail with the Consultants and expressed our general dissatisfaction with the quality of work that they are turning out.

cont'd. /2 ...

Our system of control over consultants doing foundation work in future, for us, should eliminate serious discrepancies as found at this site during the initial investigation stages.

If you have any further queries with regard to the design of this crossing, we would be pleased to assist in any way possible.

L. G. Soderman

LCG/xdg
Encl.

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

cc: Messrs.

A. M. Toye
J. McCoable ✓
H. A. Tregaskes
C. Booth
T. A. Sharpe
J. E. Craspler
W. J. Kovich

Foundation Section
Gen. Files