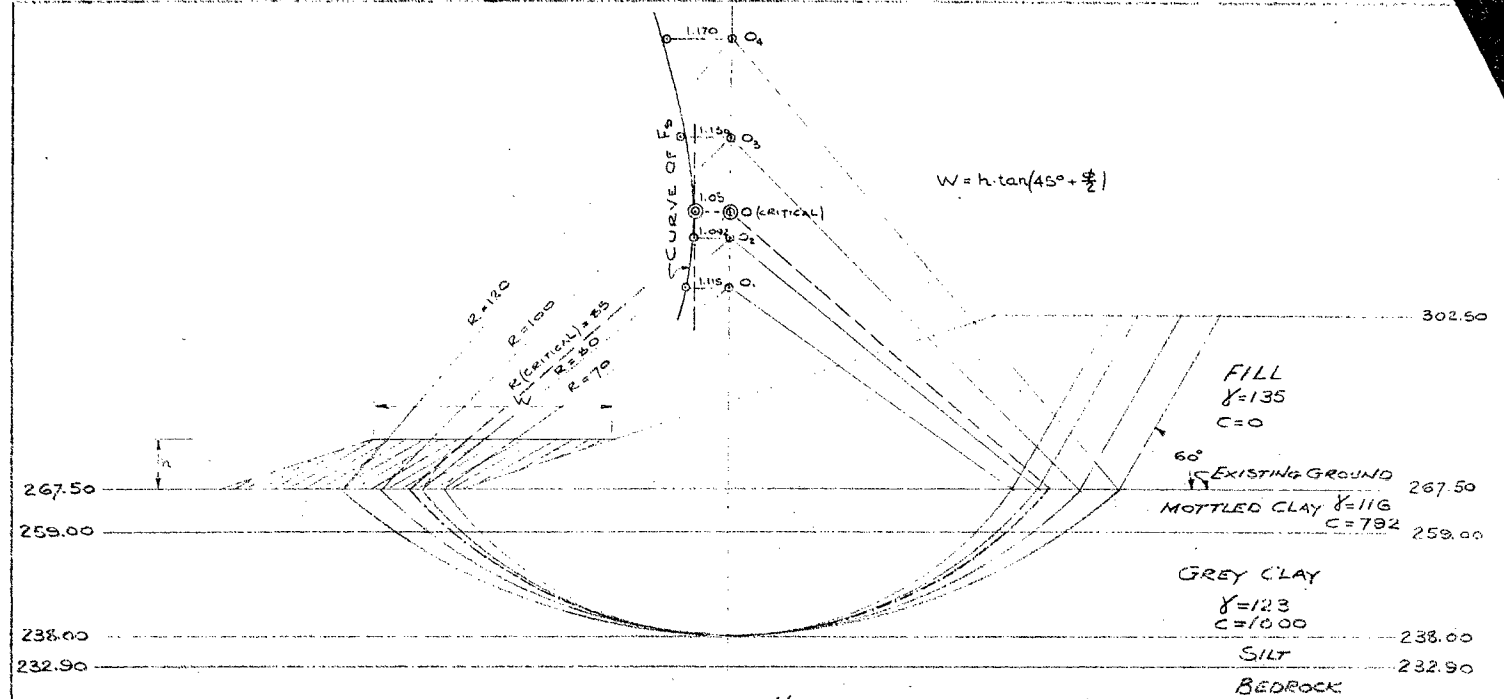


#59-F-208C

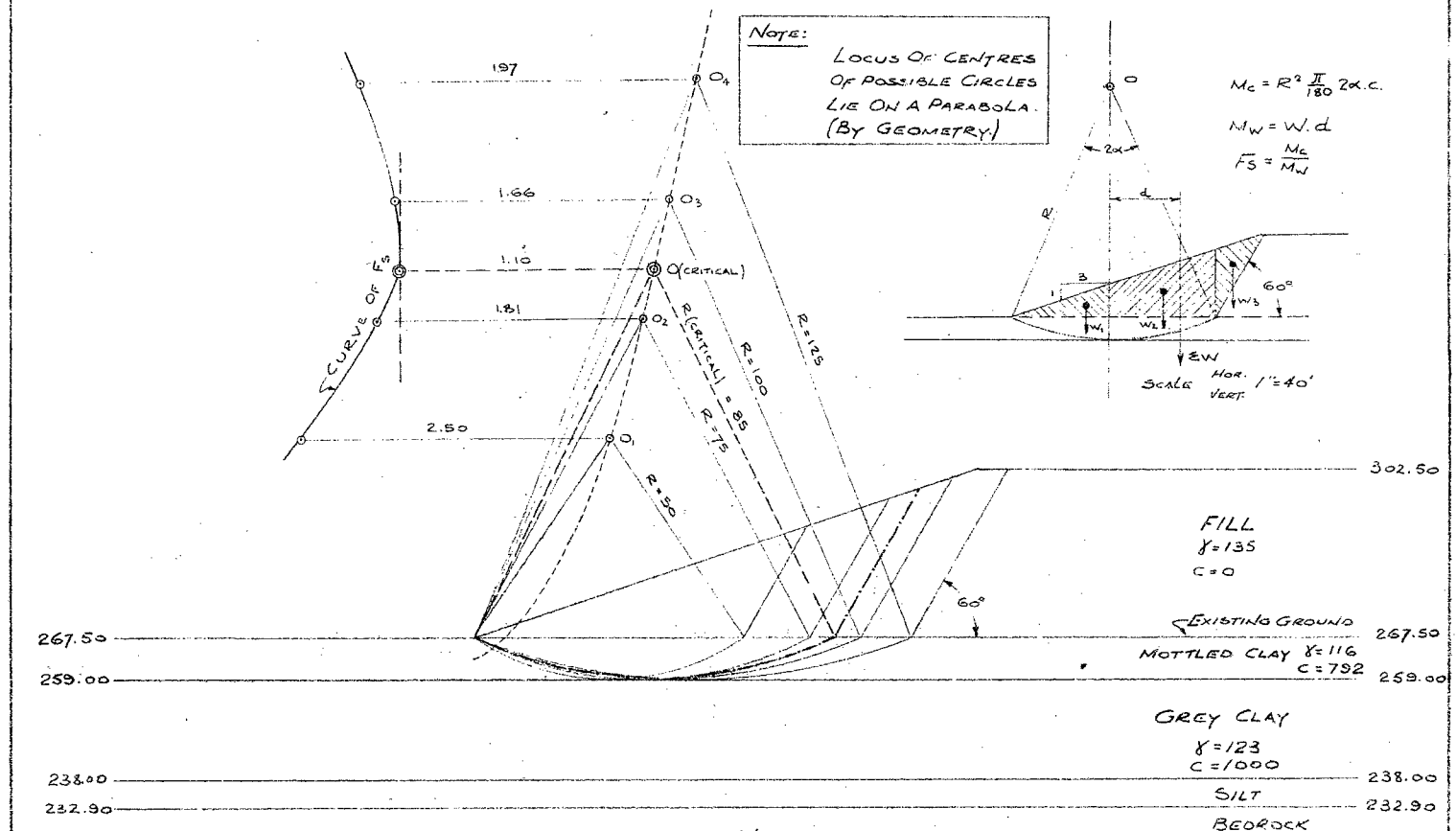
W.P.-217-58

HWY #16 &

C.N.R. CROSSING



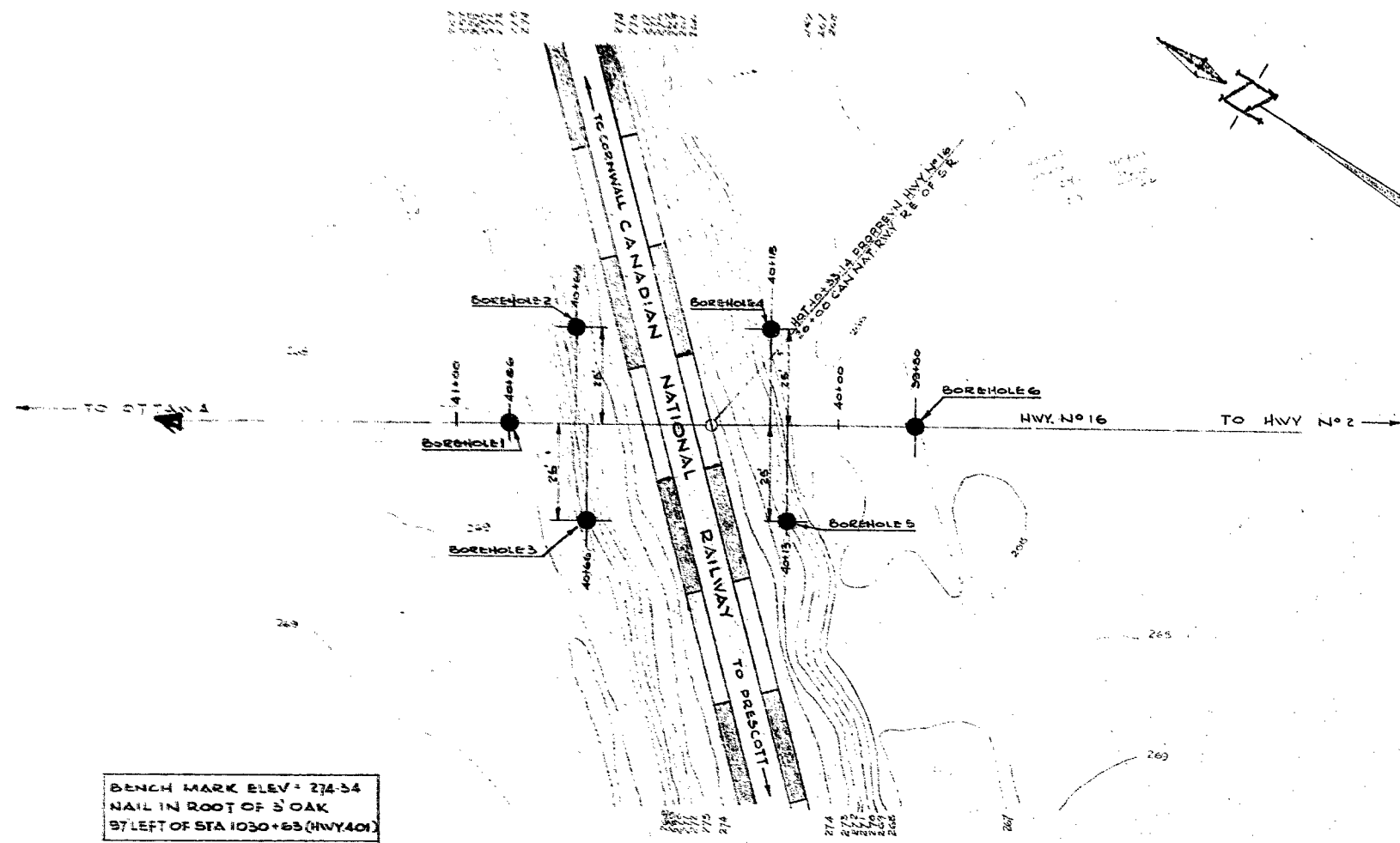
SCALE HOR. 1"=20'
VERT.



SCALE HOR. 1"=20'
VERT.

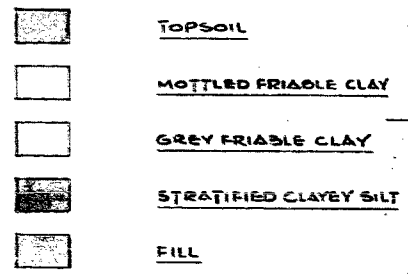
STABILITY ANALYSES.

JOB N° 5920
Empeto associates Ltd.
MAR. 19/1959 S.T.

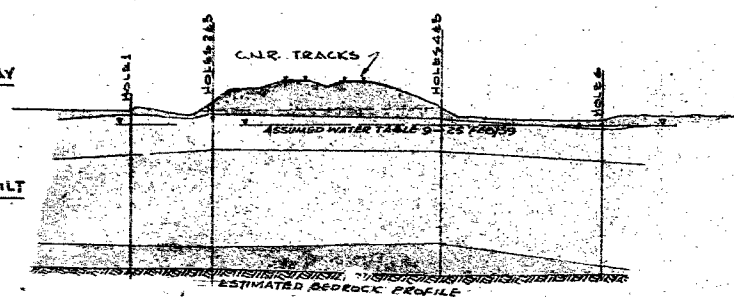


PLAN of SITE
SCALE 20:1

COLOUR KEY



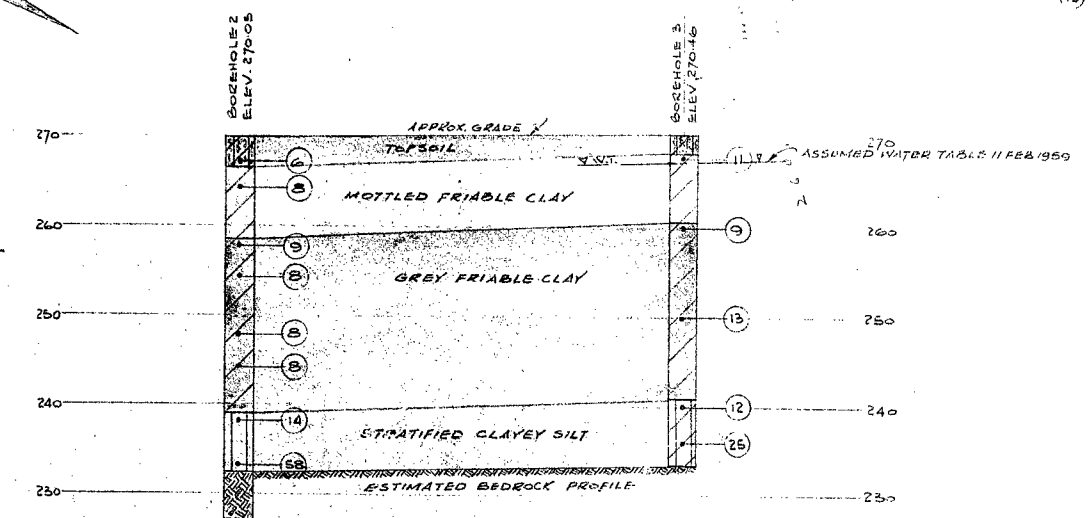
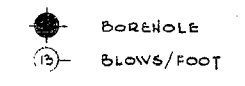
NOTE:
HOLES 2 & 3 AND 4 & 5 HAVE
BEEN INTERPOLATED TO
PRODUCE THESE RESULTS



SECTION ALONG LINE HWY 16 SHOWING STRATIFICATION

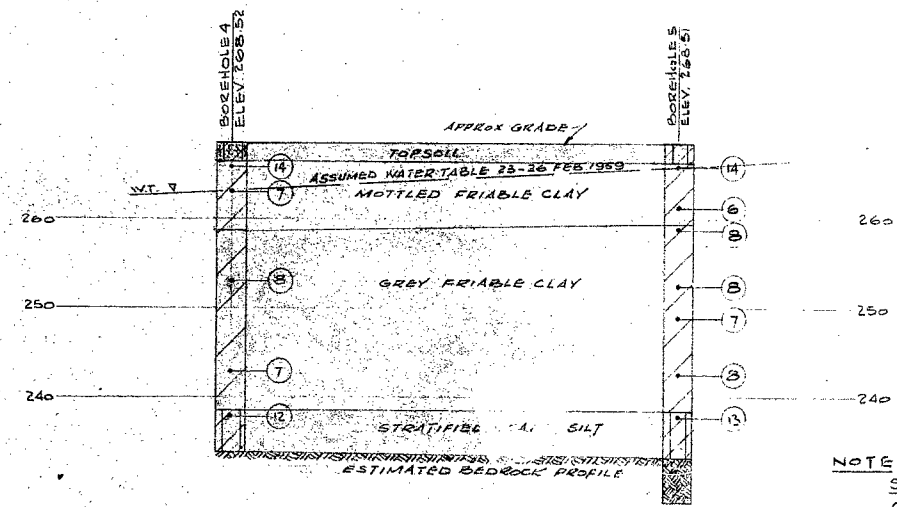
PROFILE SCALES
VERT 20:1
HOR 20:1

LEGEND



SECTION THROUGH HOLES 2 & 3

PROFILE SCALE
VERT 10:1
HOR 10:1



SECTION THROUGH HOLES 4 & 5

PROFILE SCALE
VERT 10:1
HOR 10:1

NOTE:
SEE BOREHOLE LOGS FOR
COMPLETE SOIL DATA



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 JOB NO. 5920 DATE 5 MAR 1959
CLIENTS PLAN NO. E3554-1 PER. C.J.W.

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 5920

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

March 20th, 1959.
59-F-2020

Department of Highways of Ontario,
Bridge Office,
280 Davenport Road,
Toronto, Ontario.

Attention: Mr. J. C. McAllister

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

Gentlemen:

We have pleasure in forwarding herewith four copies of our report covering the above investigation which was authorized by your letter of January 28th, 1959.

We have considered the site conditions in detail in the attached soil report. Here for your convenience we give a summary of our findings and recommendations. The soils encountered were:

- a) Black organic topsoil containing roots and some stone fragments. The thickness of this layer is 2 ft. It is generally very loose and saturated.

There is also a dark brown sandy silt loam containing a considerable amount of organic matter, some pebbles and roots with a thickness of 1'4" arising in hole 2 only.

- b) A stratum of grey and brown friable clay of high plasticity terminating at 9 to 11 ft. below surface. The natural moisture content of this soil is extremely high, generally much higher than the plastic limit. Standard penetration tests, ranging from 6 to 14, indicated a firm to stiff condition.

- c) Terminating at a variable depth from 29'6" to 32'8" below surface there is a layer of dark grey friable clay. A high natural moisture content, high plasticity, medium compressibility and low shear strength are the main characteristics of this soil. The conditions are variable, although they improve to some extent with depth. This stratum is preconsolidated.
- d) Beneath this clay for the remaining depth to bedrock there is a 1'7" to 7'6" thick stratum of stratified grey clayey silt. This soil is generally in a good condition, but the natural moisture content is somewhat above the plastic limit. Standard penetration test blows varied from 25 to 100 blows per foot. We believe, however, that the very high number of blows to be due to stone fragments.
- e) Underlying the entire site is a grey limestone bedrock with occasional pockets of embedded quartz and gypsum. No faults or water bearing seams were noted; the rock is in a good condition.

Ground water, near the surface, was noted in all the test holes at the time of the investigation. Seasonal ground water level variations are to be expected.

We consider that the laboratory shear strength results obtained from unconfined compression tests gave reliable values, which can be safely applied for design purposes.

We would prefer to apply a safety factor of 3 to the slope stability analyses, however, the results do not indicate that this would be either practical or economical. Accordingly, it would seem that the factor should be reduced to 1.5, which in turn, postulates the acceptance of some creep in the clay.

Approach embankments constructed by normal methods should be restricted to a maximum height of 28 ft. with side slopes not steeper than 3 horizontal to 1 vertical. Even resorting to staged loading we recommend precautions and strict supervision particularly during the placing of the fill. The soil stability conditions must be satisfied at each stage if this method of construction is used. Furthermore, practical considerations of the time involved lead us to recommend that the degree of consolidation to be attained in the subsoil should be restricted to 50% between each successive lift. We recommend the installation of a number of settlement test plates at suitable cross-sections, in order that information on settlement and creep is available at all times. In consequence, of the reduced factor of safety it is most advisable to collect observations during the process of filling in order to detect any impending slide, and prevent their occurrence by local modifications.

Consideration of the use of vertical sand drains on this site, showed that their installation would be of doubtful benefit in view of the probable low permeability of the materials. However, in view of the considerable depth of the clay could lead to some economy of time if sand drains were used. Horizontal sand drain trenches along the toes of the embankment should be considered.

The removal of the clay, (at least the upper mottled stratum) followed by replacement with an approved material, would provide an alternative solution, but the extent of the work involved indicates the adoption of such a method to be uneconomical.

Having in mind the existing soil conditions we do not recommend founding the bridge on footings on the claystrata.

One method would be to use caissons, placed by the "Chicago method" upon the hard silt layer at elevation 238.00. The allowable safe soil pressure in this case is 2.50 t.s.f. Settlement should be minor, having little effect on the design.

The second, and preferable alternative, is the use of a pile foundation. Piles would be driven down to bedrock where the allowable bearing value of the limestone is up to 40 t.s.f.

We trust that we have covered all the technical aspects of the soil on which you may need information. However, should you require any further additional information 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.

KP:sb

THE DEPARTMENT OF HIGHWAYS OF ONTARIO

SOILS REPORT

for

HIGHWAY #16 - C.N.R. CROSSING

W.P. 217 - 58 EDWARDSBURG TOWNSHIP

March, 1959

Job No. 5920

Client's Ref. No.

Date March 13th, 1959

Report on

SOIL SITE INVESTIGATION

at

HIGHWAY #16 - C.N.R. CROSSING

W.P. 217 - 58 EDWARDSBURG TOWNSHIP

for

DEPARTMENT OF HIGHWAYS OF ONTARIO

INTRODUCTION:

We were requested by letter from Mr. J. C. McAllister dated January 28th, 1959, to carry out a complete investigation for a railway crossing site for the proposed revision of the King's Highway #16.

The object of the investigation was:

- a) to determine existing soil and ground water conditions.
- b) to locate bedrock and determine its general condition.
- c) to determine any other facts affecting the design and construction of the proposed structure.

PROGRAMME OF WORK:

- February 6th, 1959: Crew and equipment moved to site from Toronto. Test holes located and ground levels determined by Field Engineer.
- February 7th, 1959: General site preparation; permission to enter C.N.R. property obtained.
- February 9th, 1959: Field work commenced at hole 1.
- February 19th, 1959: Visit to site by Field Engineer, assessed results, made some revisions to field procedures.
- February 27th, 1959: Field work completed.

Note: Field work was hampered throughout by very heavy snowfall, which tended to make the site inaccessible by truck. It was also necessary to haul water used in driving the test holes.

GENERAL INFORMATION:

1. Test holes were driven and sampled in accordance with our standard sampling procedures detailed in Appendix III.

2. Six test holes were driven, of which two were also cored to prove the soundness of bedrock.

Holes 1, 3, 4 and 6 were driven with BX (2-1/2" diameter) casing, and holes 2 and 5 were put down using 4 inch pipe casing in order to recover larger diameter samples for laboratory testing.

3. All elevations mentioned in the report are to Geodetic datum and were obtained by using a D. H. O. bench mark at this site. This was a nail in the root of a 3 ft. oak tree 97 ft. left of station 1030 + 63 of Highway #401. The elevation of this bench mark was taken to be 274.34.

4. All soil samples obtained at this site were carefully examined in our laboratory, and visually classified. Detailed individual borehole logs were drawn up, and these are included with the report.

5. Locations of the test holes are shown on a site plan prepared from your drawing (Plan E-3554-1). Profiles showing the soil stratification based on the test hole results are also included on our site plan, attached at the rear of the report.

6. Details of the laboratory tests and results are discussed under "Test Results" and shown either in tabular form or graphically in Appendix I, under "Laboratory Test Results".

SITE AND GEOLOGY:

The site is located in the region of Edwardsburg Township approximately 3/4 of a mile North of Johnstown, and about 200 ft. East of existing Highway #16, (H. C. T. 40 + 33.14 Proposed Revision Highway #16 - 26 + 00 Canadian National Railway R. E. of S. R.).

The topography at the site is fairly level, however, it is gently rolling at places.

SITE AND GEOLOGY: (Cont'd)

The site is located in the County of Grenville on the Northern shore of the St. Lawrence River in Eastern Ontario. The overall relief is small and the water table generally stands near the surface so much so that shallow muck has frequently developed in the area. Deposits of clay are found in close proximity to the rivers and streams in this locality. Occasionally they contain a small amount of grit, but for the most part they are stone-free. Most of the clay is covered by beds of sand, but such a cover was not noted on this site. Except for a small outcrop this County is underlain by Beekmantown limestone. This mostly consists of impure dolomite and magnesian limestones, and some calcareous sandstone.

SOIL CONDITIONS:

Soil conditions at this site are quite uniform. The soils encountered have been grouped into four main classes as follows:

a) Black Organic Topsoil

The site is covered by a layer of black organic topsoil containing roots and some stone fragments, with an average thickness of 2 ft. This soil is basically an organic sandy loam. It is generally very loose and saturated.

Beneath the topsoil at hole 2 only is a dark brown sandy silt soil with a thickness of 1'4". This soil contains pebbles, roots and a considerable amount of organic matter, and it is loose and saturated also.

b) Mottled Grey-Brown Silty Clay

Underlying the topsoil and terminating at a depth of 9 to 11'5" below surface, is a grey and brown, mottled clay of high plasticity. It has been subjected to some dessication or oxidation. The natural moisture content of this stratum is extremely high, generally much higher than the plastic limit; the soil is fully saturated (M. C. = 27.0 - 46.3%). It can be readily ruptured and crushed with moderate force, i.e. the soil has a friable structure. It is highly compressible, and has low sensitivity, indicating that it loses some of its original strength when remoulded.

The laboratory test results are as follows:

	<u>Average</u>	<u>Range</u>
Wet density, γ_w	113.3 p.c.f.	113.7 - 119.5 p.c.f.
Dry density, γ_d	82.9 p.c.f.	78.5 - 87.3 p.c.f.
Void ratio e	.846	.762 - .931
Degree of Saturation	100%	All samples
Liquid Limit	66.7%	
Plastic Limit	33.1%	

SOIL CONDITIONS:

b) Mottled Grey-Brown Silty Clay (Cont'd)

	<u>Average</u>	<u>Range</u>
Relative Consistency	0.77	0.61 - 0.92
Shear Strength, C	792 p.s.f.	576 - 1008 p.s.f.
Sensitivity	2.1	
Compressive Index	0.683	

c) Dark Grey Silty Clay

Beneath the upper dessicated clay is a stratum of dark grey friable clay with occasional grits and pebbles. This soil layer extends down to variable depths of 29'6" to 32'8" below surface. At a depth of 15 ft. some organic matter was noted in this stratum in hole 2 only. The natural moisture content is above the plastic limit, and in some specimens approached the liquid limit, and varies from 21.7% to 40.5%. The soil has a high plasticity and a medium compressibility. The upper boundaries of this layer are in a very poor condition, but soil conditions become somewhat better with depth. This clay has been preconsolidated in the past, with preconsolidation stress in the order of 5900 p.s.f. However, the shear strength is low due to a number of factors such as relief of previous stresses, followed by fissuring of the clay. The soil is insensitive, despite the friable structure.

The representative values for this soil based on laboratory tests are:

	<u>Average</u>	<u>Range</u>
Wet Density, γ_w	122.9 pc.f.	114.2 - 140.0 p.c.f.
Dry Density, γ_d	94.3 p.c.f.	82.5 - 117.5 p.c.f.
Void Ratio e	.806	.435 - 1.125
Degree of Saturation	100%	All samples
Liquid Limit	54.5%	50.5 - 65.6
Plastic Limit	22.9%	21.4 - 24.7
Relative Consistency	0.63	0.46 - 0.81
Shear Strength C	1000 p.s.f.	492 - 2130 p.s.f.
Sensitivity	1.35	1.03 - 1.73
Angle of internal friction ϕ	7°	
Compressive Index	0.200	

SOIL CONDITIONS: (Cont'd)

d) Grey Clayey Silt

Overlying the bedrock is a 1'7" to 7'6" thick stratum of stratified grey clayey silt. Although the natural moisture content is above the plastic limit, this soil is generally in a satisfactory condition. A few seams of fine sand were noted in some samples, and the soil is somewhat sandy at a depth of 35 ft. in hole 1.

Test results are as follows:

	<u>Average</u>	<u>Range</u>
Wet density, γ_w	137.7 p.c.f.	132.4 - 147.7 p.c.f.
Dry density, γ_d	113.9 p.c.f.	107.8 - 126.5 p.c.f.
Void Ratio e	.485	.335 - .564
Degree of Saturation	100%	All samples
Liquid Limit	27.3%	26.5 - 28.2%
Plastic Limit	17.5%	17.4 - 17.6%
Relative Consistency	0.36	0.73 - 0.99
Shear Strength C	3488 p.s.f.	1427 - 4605 p.s.f.
Sensitivity	1.07	

e) Bedrock

Underlying the entire site there is a grey limestone, containing occasional small pockets of gypsum, and quartz crystals, very thin sand seams are also interbedded at places. It can be scratched with a knife, and is not hard enough to scratch glass. No faults or water-bearing seams were noted, and the rock is generally in a very good condition. There is only slight variation in the top of bedrock level. This may be illustrated in the following table.

<u>Hole #</u>	<u>Ground Elevation</u>	<u>Depth to Bedrock</u>	<u>Bedrock Elevation</u>	<u>Depth of Hole</u>
1	268.14	35'9"	232.39	35'9"
2	270.05	37'9"	232.30	42'11"
3	270.46	37'0"	233.46	37'0"
4	268.52	34'3"	233.85	34'3"
5	268.51	35'2"	233.34	40'2"
6	266.32	34'3"	232.07	34'3"

Standard Penetration Tests:

The standard penetration test results have been plotted against elevation and is given under Appendix I. From this it is apparent that:

- a) In the mottled clay stratum the number of test blows ranged from 6 to 14, indicating a firm to stiff condition. It is possible that the higher values are due to some interference with the sampler, such as from stone fragments. Having in mind the laboratory test results we feel that a figure of 9 blows may be considered as representative for this soil.
- b) Standard penetration test results do not show variation, in the grey friable clay stratum, except in hole 3 at a depth of 20 ft. and hole 6 at a depth of 25 ft. where the number of the test blows increased to 13 and 12, respectively. It appears that a figure of 8 could be safely adopted for the "N" value of this stratum.
- c) The upper boundary of the grey clayey silt stratum is stiff with blows of from 12 to 14. At elevation 236 the density increased to very stiff (N= 25) after which the soil becomes extremely hard, with standard penetration test results in excess of 100 blows per foot. The unusually high figures for this soil are probably due to the resistance of some stone fragments, and to the build-up of pore water pressures in the saturated silt during driving.

WATER CONDITIONS:

In holes 2 and 5 wash water was used below a depth of 6 and 9 ft. respectively, while the other four holes were driven and sampled dry, without using wash water, the casing being cleaned out by means of auger.

Details of the water level readings in the holes during the course of our field work are as follows:

Hole #1 Elevation 268.14

On completion of the hole, water seepage was noted at a depth of 30 feet.

Hole bailed to 30' February 10th, 1959 5:00 p.m.

Hole caved in at 26', W. T. = 3'3" below surface, February 11th, 1959 4:00 a.m.

WATER CONDITIONS: (Cont'd)

Hole #2 Elevation 270.05

Depth 11'4", casing to 10', hole bailed February 13th, 4:30 p. m.

Depth 11'4", casing to 10', no water overnight February 14th.

Depth 37'9", casing to 25' Hole bailed. February 14th.

Depth 37'9", casing to 25' W. T. 3' February 16th, 9:00 a. m.

Depth 42'11", casing to 25' Hole bailed to 20', February 16th, 2:00 p. m.

Hole caved in at 28' after pulling casing W. T. = 3'. February 17th, 8:00 a. m.

Hole #3 Elevation 270.46

At a depth of 25 ft. water seepage was noted.

Depth 37', casing to 25' Hole bailed February 12th, 5:30 p. m.

Hole caved in at 27', W. T. = 3' February 13th 8:00 a. m.

Hole caved in at 27', W. T. = 3' February 14th, 9:00 a. m.

Hole #4 Elevation 268.52

Depth 34' casing to 30' No water in the hole February 26th, 8:30 a. m.

Hole caved in at 31' February 26th 9:00 a. m.

Depth = 31', casing to 20' W. T. = 5'6" February 26th, 10:00 a. m.

Hole #5 Elevation 268.51

In the topsoil some very slight water seepage was observed.

Depth = 31', casing to 24', hole bailed February 21st 4:30 p. m.

Depth = 31', casing to 24', W. T. 2'6" February 23rd, 8:30 p. m.

Depth = 35', casing to 19', W. T. 2'6" February 24th, 3:30 p. m.

Hole #6 Elevation 266.32

Slight water seepage noted from topsoil.

Hole to 34'3", casing to 30', no water February 18th, 2:00 p. m.

Casing withdrawn, hole caved in at 28 ft.

Hole to 23 feet, W. T. = 1'0" February 19th, 9:00 a. m.

To summarize the above observations:

- a) Some water seepage was encountered in holes 1 and 3 at a depth of 30 feet and 25 ft. respectively. In hole 1 the seepage water emanated from the interface between the silty clay and clayey silt, and in hole 2 it arose from a gravelly clay seam. Slight water seepage was observed in the topsoil at holes 5 and 6.

WATER CONDITIONS: (Cont'd)

- b) The holes were generally dry on completion of driving, and filled with water overnight.
- c) The water table, from numerous reliable observations, was very close to the ground surface during the field work. However, it is likely that the ground water level will drop slightly during the summer months, i.e. seasonal water level variations should occur.

TEST RESULTS:

1. Several Atterberg limit tests were performed to assist in classification of the materials. The results of these tests have been plotted and are given in Appendix I. The Atterberg limits corroborate the visual classification of the soil, but do not reflect the friable texture.
2. A number of unconfined compression tests were carried out on representative samples in order to determine the soil shear strength profile. However, no definite grouping of the shear values has been found on the plotted "Shear Strength vs. Elevation" graph. Hence a value of shear strength obtained by averaging has been used for each layer in our calculations, although obviously wild values have been excluded. There is some reason to believe the strength values for these clays are conservative.

From this graphical presentation it may be concluded that the shear strength is not uniformly variable through the profile, although there is a definite tendency of increasing shear strength with depth.

3. A quick undrained triaxial test on a specimen of the friable grey silty clay indicated an angle of internal friction of approximately 7° . This value is undoubtedly due to the nuggety texture of the material.

The angle of internal friction of the mottled grey-brown clay would be comparable, but these values should be neglected for design purposes.

4. Two consolidation tests on samples from different layers in different holes showed some interesting results:

- a) These clays have been subject to preconsolidation stresses in the order of 5900 p.s.f. (grey clay) and 4000 p.s.f. (mottled clay).
- b) The compressive index is low to medium, 0.200 in the grey clay stratum, and high, 0.683, in the mottled clay stratum. In the latter case the high degree of compressibility occurred at loads exceeding the preconsolidation load, when the original structure broke down.

TEST-RESULTS: (Cont'd)

- c) Although the consolidation tests show that settlement of the clay within the limits of safe design loadings is not a problem, this does not preclude the fact that soil creep due to disturbance of the soil structure could still occur at loadings less than the preconsolidation stress.

ENGINEERING CONSIDERATIONS:

I Embankment

We believe that it is at present proposed to place 35 ft. high embankments at both the North and South approaches to the bridge.

These embankments, we have assumed will be composed of selected granular material, fairly well graded, and compacted to a density approaching 95% of maximum obtainable for this type of material (Modified Proctor). The assumed characteristics of this fill are:

Wet Density	$\gamma = 135$ lb./cu.ft.
Angle of Internal Friction	$\phi = 40^\circ$
Cohesion	$C = 0$
Compressibility	$C_c = 0$

On reference to the attached stability analysis diagrams, it may be seen that in determining the average shearing resistance along a sliding surface, within the fill itself, we have assumed as an approximation, that the actual fill had been replaced by an ideal clay ($\phi = 0$) with a cohesion equal to zero. (After K. Terzaghi)

$$\text{i.e. } C_{fill} = 0 \text{ and } \phi_{fill} = 0$$

Soil conditions are somewhat critical at this site, and therefore the design of the embankment must be adapted to suit not only the character of the available fill material, but also the subsoil conditions.

The soil strata with which we have been most concerned during our studies, were the two soft clay layers (mottled clay and grey clay), because it is in these strata that any shear failure, settlement, creep, or related effects will occur in the future. Both of these two clay layers are weak, underlie the entire area, and together make up a thickness of approximately 30 ft. The upper mottled clay stratum is somewhat weaker than the deeper lying grey clay. We have considered failure occurring tangential to the interfaces between the mottled clay and the grey clay, and also between the grey clay and the dense silt. It is unlikely that the underlying dense silt, at considerable depth, will have any effect on the problem, because the deepest part of the slip circle will almost certainly occur entirely within the grey clay stratum.

ENGINEERING CONSIDERATIONS: (Cont'd)

I Embankment

Although the mass stability of the embankment in respect to the underlying strata must be such that there is a sufficient safety factor against failure, for economical reasons, in this instance, we feel it will be necessary to accept a lower factor of safety of 1.5, than normally used, for earth structures; although the maximum allowable settlement during the life-time of the embankment must, at the same time, be limited to an amount which will not cause detriment to the road surface.

The first step in a given slope stability problem is, of course, to assume a curve to represent the expected slide surface. The topography, stratigraphy and soil conditions of a site contribute largely to the approximate location of this surface, located at the position of the smallest resisting forces. It is therefore apparent that the length of slide located in the fill will be considerably shorter than one in the soft clay. For this reason we have assumed the surface of slide to be a composite one consisting of two sections, viz: an arc in the soft clay, and a straight line in the fill, meeting at an obtuse angle. Furthermore, the plane sloped at an angle of 60° to the horizontal, which is in accordance with the theoretical angle of

$$\phi = 45^{\circ} + \frac{\phi}{2}$$

Failure was expected to occur by plastic flow within the soft mass of clay.

In the stability analyses different sets of conditions have been investigated using the average stress method. In order to determine the factor of safety with respect to failure the diameter and the position of the critical circle that represents the surface along which sliding should occur has been located.

- a) Firstly, we assumed that the angle of internal friction for both clay layers is zero. This is the conventional analysis applied to saturated clays.
1. In the first case we considered the grey clay as a firm base, which was not penetrated by the slip circle, and a toe failure was likely to occur. When slopes of 3 horizontal to 1 vertical were used the factor of safety was in excess of 1.5. For the critical circle:

$$R = 85', \quad F_s = 1.56$$

ENGINEERING CONSIDERATIONS: (Cont'd)

I Embankment

2. Assuming the clayey silt stratum as a firm base, a failure along a circle passing at some distance below the toe of the slope is the most likely to occur. This is known as a base failure, and the circle along which the failure would be likely to occur is a mid-point circle.

With slopes of 2 horizontal to 1 vertical the factor of safety was less than 1.0 (for the critical circle $F_s = 0.862$). Using slopes of 3 horizontal to 1 vertical the factor of safety increased to 1.1.

In general, a factor of safety of 1.5 is obtained for a height of 27.4 ft. with slide slopes of 3:1.

- b) A second analysis assumed that for the upper layer $\phi = 0$, and for the lower layer of clay, based on a quick triaxial test result $\phi = 7^\circ$. In this computation the factor of safety amounted to a value of 2.36 (slopes 3:1), but we have discounted this analysis as being unrealistic.

Using Taylor's mathematical solution of slope stability as a check on the more rigorous method, the following results were obtained:

1. The underlying grey clay stratum being assumed stronger than the upper one to an extent that the slip circle was restrained by it. With a height of 35 ft. the results obtained were as follows:

<u>Depth Factor</u>	<u>Slope</u>	<u>Factor of Safety for H = 35'</u>	<u>Safe Height ($F_s = 1.5$)</u>
1.27	1:1	1.145	26.7
	2:1	1.315	30.5
	3:1	1.430	33.4

2. The slip circle passing through the underlying grey clay stratum, and being restrained by the firm stratum beneath. In this case:

<u>Depth Factor</u>	<u>Slope</u>	<u>Factor of Safety for H = 35'</u>	<u>Safe Height ($F_s = 1.5$)</u>
1.82	1:1	1.34	31.3
	2:1	1.46	34.1
	3:1	1.52	35.6

ENGINEERING CONSIDERATIONS: (Cont'd)

I Embankment

2 (Cont'd)

The apparent contradiction between the two sets of results can be explained by the fact that Taylor's solution is based on an ideal case which rarely, if ever, occurs in practice. The method can be very useful, however, in approximate calculations and comparisons.

Although it would be preferable to apply the conventional safety factor of 3, unfortunately as already stated above, this may not be economical, and it would appear to be more practical to use a factor of safety of only 1.5, and to expect in consequence some continuing creep.

In summing up our results we have drawn the following conclusions.

1. Every effort should be made to avoid embankment failure, and the normal rapid construction may have to be amended to reduce the risk to a minimum.
2. If the embankment is rapidly raised using normal construction methods the allowable height of the fill is 28 ft., and this with side slopes of 3:1.

However, although the clay strata are considerably preconsolidated, some settlement under the embankment due to dessication and fissuring of these clays should still be anticipated.

3. Stage construction, if possible, should be carefully considered, however, there are two principles which must be followed if the fullest benefit is to be derived from this method.
 - a) the maximum load to be placed on the clay layer for each lift, should be limited to the ultimate bearing capacity of the clay under the existing conditions at the time when the load is applied. In other words the conditions of soil stability must be satisfied for each layer placed.
 - b) the time interval between the placement of each successive layer should be long enough to allow some predetermined degree of consolidation of the layer. After the time required to consolidate the clay has been reached, then a new lift of material can be placed.

ENGINEERING CONSIDERATIONS:

I Embankment

It is evident that to wait for 100% consolidation is out of the question. For practical purposes the acceptable degree of consolidation would be 50% under each successive lift. A higher degree of consolidation would entail too long a waiting period, but a lower degree of consolidation would not have the effect desired.

In any event, it would be efficacious to install test plates at convenient cross-sections, in order that field settlement and creep data could be collected.

4. Berm construction has been quite successfully used in many such cases, and it may be a satisfactory solution to the problem in this case. However, if the method of rapid "build-up" is used, and the height of the embankment cannot be reduced, it is imperative to place toe berms, because the side slopes cannot be increased, because of lack of space. The required weight of the berm can be calculated from the following equation:

$$w = h \tan (45^\circ + \phi / 2) \quad \text{where}$$

w = width of the proposed berm

h = height of the proposed berm

ϕ = angle of internal friction of the material used.

After the toe berms have been placed, preferably concurrently with the main fill (the overall width of berms and embankment being identical with that of a fill constructed with side slopes of 3:1) to a height of 10 ft. the remainder of the embankment can be placed with side slopes of 2 horizontal to 1 vertical (See Appendix).

5. Horizontal sand drain trenches parallel to and at the toes of the embankment, should also be considered. Such drains have been effectively used and the method has the following advantages:
- a) It effectively drains the over saturated soil under pressure, and thereby reduces considerably the time of consolidation.
 - b) It increases the stability of the fill by acting as a retaining wall.
 - c) Berms at the embankment toes are not required.

ENGINEERING CONSIDERATIONS:

I Embankment

6. The removal of the soft clay beneath the proposed fill to a depth of 10 ft. into the weaker layer, and thereafter replacement with approved granular material would provide a satisfactory, but expensive, alternative solution.
7. The use of vertical sand drains to stabilize the clay does not appear to be either practical or economical, because of the extent of the area involved. The close spacing of the sand pipes required, combined with difficulty of installation, and the probable low rate of permeability of the clay, are the main disadvantages to this method in this instance.
8. Adequate surface drainage should be provided at all times in order to reduce the softening of the underlying clay to a minimum by water percolation through fissures from the surface.

II Overpass Structure

- 1 Supporting the structure on large footings founded upon the grey clay stratum has been considered. However, after serious consideration we have reached the conclusion that this type of foundation should, if possible, be avoided.

The allowable safe bearing capacity of this stratum at a depth of 10 ft. below surface is 0.9 tons per sq. ft., thus, the soil is already overloaded by the superimposed fill, and any additional loading would almost certainly cause failure in the soil.

If wide spread footings are used then we suggest the placement of the fill before the bridge is built, however, to avoid the undesirable consequences resulting from thrust (caused by the fill) being placed on the shallow footings.

It is possible for plastic deformation of the clay stratum to occur as a result of the superimposed embankment load. This in turn would produce some lateral thrust on the abutments and for this reason it would be desirable to place the foundation at a greater depth, and we do not consider the use of spread footings to be the best answer.

ENGINEERING CONSIDERATIONS:

II Overpass Structure

2. Existing soil conditions suggest the possible use of the founding depth being on the hard grey clayey silt stratum at elevation 238.00. The "Chicago method" could be successfully used.

The surface of the silt stratum is uneven, its depth below surface varies considerably, and this will require careful attention being paid during placement of the caissons. Each caisson must be keyed into the silt at least 4 inches.

The allowable safe soil pressure of the silt is 2.50 t.s.f. ($F_s = 3$).

Settlement would not be a problem since this layer appears to be virtually incompressible, although some minor settlements within tolerable limits may be expected with differential settlement up to 1/2 inch.

3. We feel, however, that piles would provide the best method of founding the proposed structure. The piles should be driven to bedrock.

Steel H, monotube, or concrete filled pipe pile, could all be satisfactorily used.

The safe bearing capacity of the bedrock is up to 40 t.s.f. Settlement would not occur.

If piles are used then the bridge can be built before any fill is placed which has definite advantages, and ground water, excavation and appurtenant problems connected with the use of spread footings or caissons would not arise.

E. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

KP:sb

BOREHOLE LOG

Checked By **E.M.P.**

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0' 0"					
BLACK SANDY SILT LOAM TOP SOIL WITH ROOTS AND ORGANIC MATTER			268.14 1' 0"		1	FROM CASING		SATURATED
MOTTLED, FRIABLE CLAY	BROWN & GREY	STIFF 3' 8" 264.47	267.14 5' 0"		2	S.S. H		M.C.=40.2% MUCH WETTER THAN PLASTIC LIMIT FEBRUARY 11, 1959 WATER FILLED HOLE OVERNIGHT
AS ABOVE	AS ABOVE	STIFF			3	S.S.	9	M.C.=36.0% W.T.P.L.
			10' 0"					
FRIABLE CLAY	DK. GREY		258.14		4	S.L. TAPPED		M.C.=35.5% ; S _r =100% C=984 TSF ; e=.964
			15' 0"					
AS ABOVE, OCCASIONAL PEBBLES	AS ABOVE	FIRM TO STIFF			5	S.S.	8	M.C.=39.4% M.W.T.P.L.
			20' 0"					
AS ABOVE, GRITS	AS ABOVE				6	S.L. TAPPED		M.C.=31.7% ; C=590 TSF e=.877
			25' 0"					
AS ABOVE	AS ABOVE	FIRM TO STIFF			7	S.S.	8	M.C.=32.0% M.W.T.P.L.
			30' 0"					
CLAYEY SILT WITH THIN SEAMS OF VERY FINE SAND (SLIGHTLY PLASTIC)	DK. GREY		238.14		8	S.L. TAPPED		WATER SEEPAGE AT 30' 0" M.C.=22.1% W.T.P.L.
AS ABOVE	AS ABOVE	HARD	35' 9" 232.39		9	S.S.	100	M.C.=15.1% DRIER THAN PLASTIC LIMIT
						REFUSAL		NOTE: HOLE SAMPLED DRY

BOREHOLE LOG

Datum Geodetic Compiled By K.P.

Checked By E.M.P.

ABBREVIATIONS

 UNDISTURBED

 FAIR

☒ DISTURBED

LOST

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

S. L. SPLIT BARREL WITH LINERS

5. T. THIN-WALLED SHELBY TUBE SAMPLE

W. S. WASH SAMPLE

R. C. ROCK CORE

V. T. IN SITU VANE SHEAR TEST
Q/u UNCONFINED COMPRESSIVE STRENGTH
W. L. WATER LEVEL IN CASING
W. T. GROUND WATER TABLE IN SOIL

V. T. IN SITU VANE SHEAR TEST

Q/u UNCONFINED COMPRESSIVE STRENGTH

W. L. WATER LEVEL IN CASING

W. T. GROUND WATER TABLE IN SOIL





SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0'0"					
ORGANIC TOPSOIL WITH ROOTS & STONES	BLACK		270.05	1	FROM CASING			SATURATED
SANDY SILT WITH PEBBLES, ROOTS, ORG. MATTER	DK. BROWN	FIRM	21'0" 268.05 3'4" 266.72 5'0"	2	S.S.	6		SATURATED
MOTTLED, FRIABLE CLAY	BROWN & GREY	FIRM TO STIFF		3	S.S.	8		M.C. = 44.4% MUCH WETTER THAN PLASTIC LIMIT
OCCASIONAL PEBBLES								
			10'0"					
FRIABLE CLAY	DK. GREY	STIFF	11'5" 258.53	4 5	S.L. TAPPED S.S.	9		SAMPLE LOST M.C. = 34.5% M.W.T.P.L. e = 1.042
			15'0"					
AS ABOVE, SOME ORGANIC MATTER	AS ABOVE	FIRM TO STIFF		6	S.S.	8		M.C. = 36.6% M.W.T.P.L.
			20'0"					
AS ABOVE, FEW SMALL STONES	AS ABOVE			7	S.L. TAPPED			
AS ABOVE	AS ABOVE	FIRM TO STIFF		8	S.S.	8		W.T.P.L.
			25'0"					
AS ABOVE	AS A	FIRM TO STIFF		9	S.S.	8		M.C. = 21.7% W.T.P.L.
AS ABOVE, GRITS STRATIFIED CLAYEY SILT	AS ABOVE DK. GREY	STIFF	31'0" 239.05	10 11	S.L. TAPPED S.S.	14		M.C. = 24.6% M.W.T.P.L.
			35'0"					
AS ABOVE	AS ABOVE	HARD		12	S.S.	58		M.C. = 17.5% W.T.P.L.
			37'9" 232.30					
			40'0"		R.C.			RECOVERY 98%
LIMESTONE	GREY		42'11" 227.13					NOTE: HOLE DRIVEN WITH THE USE OF WASH WATER
					HOLE TERMINATED			

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

Job Name Edwardsburg Twp.C.N.R. Overhead
Hwy. 16 Johnstown. Job No. 5920
Client Dep't. of Highways of Ontario Casing BX (2 1/2" Dia.)
Datum Geodetic. Compiled By K.P.R.

Borehole No. 3
Boring Date Feb. 12th. - 13th. 1959.
Checked By E.M.P.

SAMPLE CONDITION












 UNDISTURBED
 FAIR
 DISTURBED
 LOST

SAMPLE TYPE

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
Q/u UNCONFINED COMPRESSIVE STRENGTH
W.L. WATER LEVEL IN CASING
W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0'0"					
ORGANIC TOPSOIL WITH ROOTS	BLACK		270'46		1 	FROM CASING		VERY WET
MOTTLED, FRIABLE CLAY (FEW ROOTS)	BROWN & GREY	STIFF	3'0" 268'46		2 	S.S.	W.T. 11	M.C.=38.2% MUCH WETTER THAN PLASTIC LIMIT e=.931
AS ABOVE	AS ABOVE		5'0"		3 	S.L. TAPPED		FEBRUARY 13-14, 1959 WATER FILLED HOLE OVERNIGHT
FRIABLE CLAY	DK. GREY	STIFF	10'0" 260'46		4 	S.S.	9	M.C.=33.5% M.W.T.P.L.
AS ABOVE	AS ABOVE		15'0"		5 	S.L. TAPPED		M.C.=40.5% M.W.T.P.L. e=1.043
AS ABOVE	AS ABOVE	STIFF	20'0"		6 	S.S.	13	M.C.=27.1% W.T.P.L.
AS ABOVE, FEW SMALL STONES	AS ABOVE		25'0"		7 	S.L. TAPPED		WATER SEEPAGE AT 25' M.C.=37.7% M.W.T.P.L. e=.915
STRATIFIED CLAYEY SILT	DK. GREY	STIFF	29'6" 240'96		8 	S.S.	12	M.C.=24.4% M.W.T.P.L.
AS ABOVE	AS ABOVE	VERY STIFF	35'0"		9 	S.S.	25	M.C.=19.7% W.T.P.L.
			37'0" 233'46					NOTE: HOLE SAMPLED DRY

BOREHOLE LOG

Checked By **E.M.P.**

ABBREVIATIONS

V. T. IN SITU VANE SHEAR TEST
Q/u UNCONFINED COMPRESSIVE STRENGTH
W. L. WATER LEVEL IN CASING
W. T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0' 0"					
ORGANIC TOPSOIL & SOME COARSE SAND	BLACK		268'52"	1	FROM CASING			SATURATED
		STIFF	2' 0" 266'52"	2	S.S.	14		W.T. WITH CASING AT 5'
MOTTLED, FRIABLE CLAY	GREY & BROWN		5' 0" 266'02"	2/A	AUGER			
AS ABOVE	AS ABOVE	FIRM	5' 6" 263'02"	3	S.S.	7		FEBRUARY 26, 1959. W.T. WATER LEVEL WITH CASING AT 20' M.C. = 39.4% MUCH WETTER THAN PLASTIC LIMIT
			9' 8" 258'85"	4	S.L.	TAPPED		
FRIABLE CLAY	DK. GREY							
			15' 0"					
AS ABOVE	AS ABOVE	FIRM TO STIFF		5	S.S.	8		M.C. = 33.4% M.W.T. P.L.
			20' 0"					
AS ABOVE	AS ABOVE			6	S.L.	TAPPED		
			25' 0"					
AS ABOVE	AS ABOVE	FIRM		7	S.S.	7		M.C. = 29.1% W.T. P.L.
			30' 0"					
STRATIFIED CLAYEY SILT	DK. GREY	STIFF	238'52"	8	S.S.	12		M.C. = 19.2% W.T. P.L.
				9	S.L.	TAPPED		
			34' 8" 233'85"					
					REFUSAL			NOTE: HOLE SAMPLED DRY





e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Edwardsburg Twp. C.N.R. Overhead
 Job Name Hwy. 16. Johnstown. Job No. 5920 Borehole No. 5
 Client Dep't. of Highways of Ontario. Casing 4" Pipe Boring Date Feb. 19th. - 24th. 1959.
 Datum Geodetic. Compiled By K.P. Checked By E.M.P.

SAMPLE CONDITION



-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

SAMPLE TYPE

- 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
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0' 0"					
ORGANIC TOPSOIL, FEW STONES	BLACK		268'51"		1	FROM CASING		WATER SEEPAGE IN TOPSOIL
MOTTLED FRIABLE CLAY	BROWN & GREY	2 W.T. 2'-5" STIFF	2' 0" 266'51"		2	S.S.	14	M.C. = 43.1% WATER TABLE 23-24 FEB.
AS ABOVE	AS ABOVE		5' 0"		3	S.L. TAPPED		
AS ABOVE	AS ABOVE	FIRM			4	S.S.	6	
FRIABLE CLAY	DK. GREY	FIRM TO STIFF	9' 0" 259'51"		5	S.S.	8	M.C. = 30.9% MUCH WETTER THAN PLASTIC LIMIT
AS ABOVE	AS ABOVE		15' 0"		6	S.L. TAPPED		
AS ABOVE	AS ABOVE	FIRM TO STIFF			7	S.S.	8	
AS ABOVE, SEAMS OF SILT	AS ABOVE	FIRM	20' 0"		8	S.S.	7	M.C. = 26.2% W.T. P.L.
AS ABOVE	AS ABOVE		25' 0"		9	S.L. TAPPED		
AS ABOVE	AS ABOVE	FIRM TO STIFF			10	S.S.	8	
STRATIFIED CLAYEY SILT	DK. GREY	STIFF	30' 0" 238'51"		11	S.S.	13	M.C. = 21.3% W.T. P.L.
LIMESTONE	GREY		35' 2" 233'34"			R.C.		RECOVERY 98%
			40' 2" 228'34"					HOLE TERMINATED

BOREHOLE LOG

Borehole No. 9.
Boring Date Feb. 17th. - 18th. 1959.
Checked By E. M. P.

ABBREVIATIONS

V. T. IN SITU VANE SHEAR TEST
Q/u UNCONFINED COMPRESSIVE STRENGTH
W. L. WATER LEVEL IN CASING
W. T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND SURFACE			0' 0"					
SANDY SILT TOPSOIL, ROOTS, STONES, ORG. MATTER	BLACK	W.T. 1-0	266'32"	1	1	FROM CASING	7	19 FEB
MOTTLED, FRIABLE CLAY	BROWN & GREY	FIRM	264'65"	2	2	S.S.	7	M.C. = 46.3% MUCH WETTER THAN PLASTIC LIMIT
AS ABOVE	AS ABOVE	FIRM	5' 0"	3	3	S.S.	7	M.C. = 44.8% M.W.T.P.L.
FRIABLE CLAY	DK. GREY		9' 0"	4	4	S.L. TAPPED		
AS ABOVE	AS ABOVE	FIRM TO STIFF	15' 0"	5	5	S.S.	8	M.C. = 33.5% M.W.T.P.L.
AS ABOVE	AS ABOVE		20' 0"	6	6	S.L. TAPPED		M.C. = 30.6% M.W.T.P.L.
AS ABOVE	AS ABOVE	STIFF	25' 0"	7	7	S.S.	12	M.C. = 28.5% W.T.P.L.
AS ABOVE	AS ABOVE		30' 0"	8	8	S.L. TAPPED		
STRATIFIED CLAYEY SILT	DK. GREY	HARD	32' 8"	9	9	S.S.	60	M.C. = 19.2% W.T.P.L.
			34' 3"					
			232'07"					
					REFUSAL			

APPENDIX I

LABORATORY TEST RESULTS

SUMMARY OF UNCONFINED COMPRESSION TEST RESULTS

Job No. 5920

Bore Hole #	Sample	Depth	Soil Type	Nat. M. C. %	Wet Density	Void Ratio	Degree of Saturation	Strain at Failure	Shear Strength p. s. f.	Remoulded Shear Strength	Remarks
1	4	10' - 11'8"	Grey Clay	35.5	116.3	.964	99.5	20.0	984	624	
	6	20' - 21'8"	Grey Clay	31.7	118.0	.877	97.5	13.3	590	574	
	8	30' - 31'8"	Grey Clayey Silt	22.1	132.4	.554	100.0	20.0	1427		
2	5	11'6"-12'6"	Grey Clay	34.5	114.2	1.042	100.0	10.0	1368	792	
	9	25' - 26'	Grey Clay	21.7	140.0	.473	100.0	16.7	1670		
	12	36' - 37'	Grey Clayey Silt	17.5	138.7	.445	100.0	10.0	5040		
	3	5' - 6'8"	Mottled Clay	36.9	119.5	.931	100.0	3.3	1008		
	5	15' - 16'8"	Grey Clay	40.5	115.7	1.043	100.0	20.0	1198		
	6	20' - 21'	Grey Clay	27.1	131.2	.602	100.0	20.0	1943		
	7	25' - 26'8"	Grey Clay	37.7	121.1	.915	100.0	6.6	492	459	
	9	34' - 35'	Grey Clayey Silt	19.7	147.7	.335	100.0	15.0	4605		
4	3	10' - 11'8"	Grey Clay	37.5	117.2	.974	100.0	10.0	1640		
	5	20' - 21'8"	Grey Clay	45.0	115.0	1.125	100.0	10.0	1608		
	6	25' - 26'	Grey Clay	29.0	131.8	.653	100.0	20.0	2130		
	8	31' - 32'8"	Grey Clayey Silt	21.0	136.2	.496	100.0	13.3	4360		
5	5	14' - 15'4"	Grey Clay	34.6	125.0	.813	100.0	20.0	1425		
	9	25'6"-26'6"	Grey Clay	13.3	127.3	.435	100.0	20.0	1930		
	10	30' - 31'	Grey Clayey Silt	23.2	132.9	.564	100.0	16.7	3365		
6	3	5' - 6'	Mottled Clay	44.8	113.7	.762	100.0	13.3	576		
	4	10' - 11'8"	Grey Clay	30.7	120.0	.830	100.0	Low	648 ^x		
	6	21'2"-21'8"	Grey Clay	30.6	122.3	.799	100.0	15.0	720		
	8	30' - 31'8"	Grey Clay	23.7	129.0	.575	100.0				

Ø = 70 (Approx).

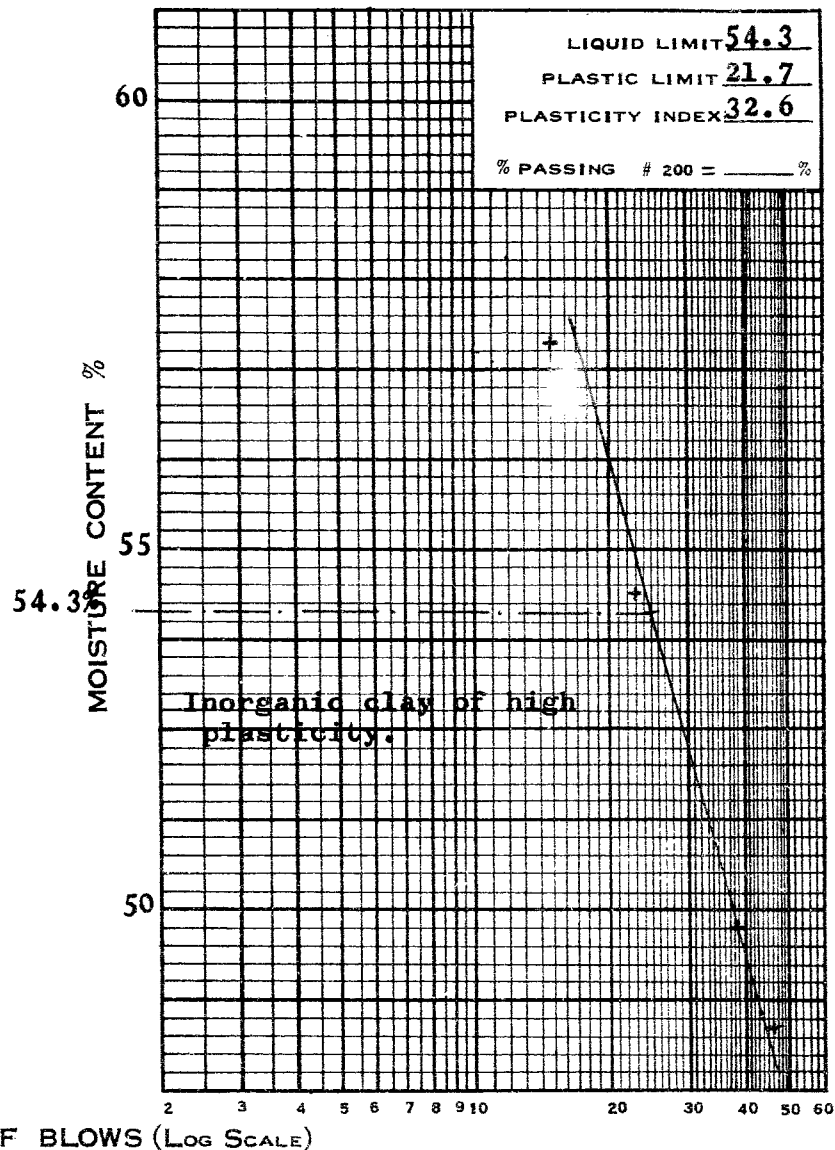
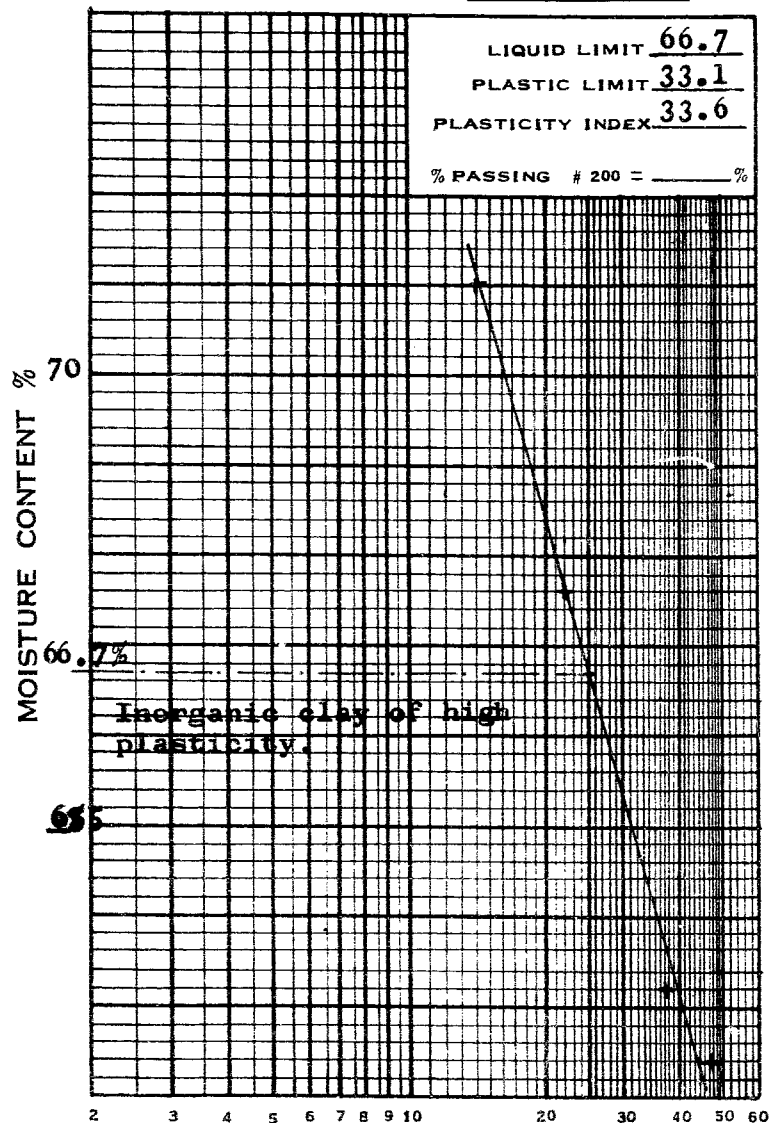
^xQuick Undrained Triaxial Test.

e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

JOB NO. 5920 PROJECT Edwardsburg Twp. C.N.R. Overhead Hwy. 16 Johnstown.
SAMPLE FROM B.H.# 1. Sample # 1. SAMPLE FROM B.H.# 1. Sample # 4.
DEPTH 2' - 3' DEPTH 15' - 16'

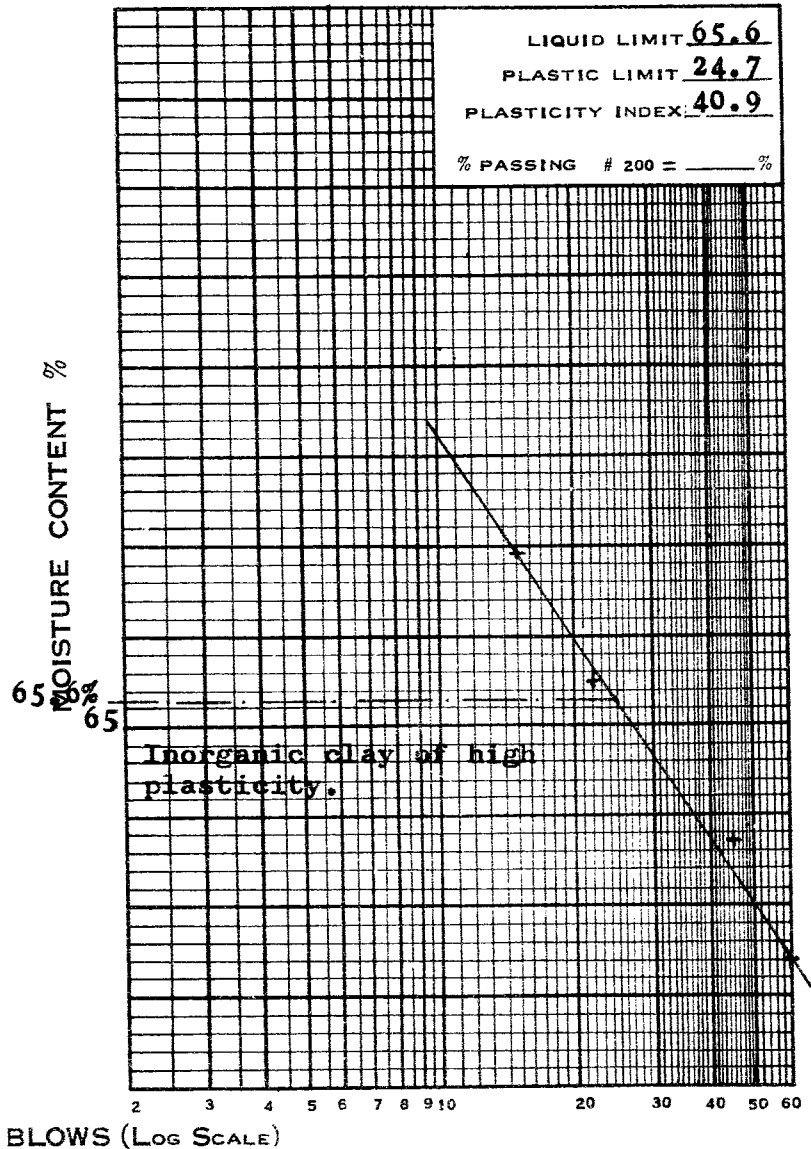
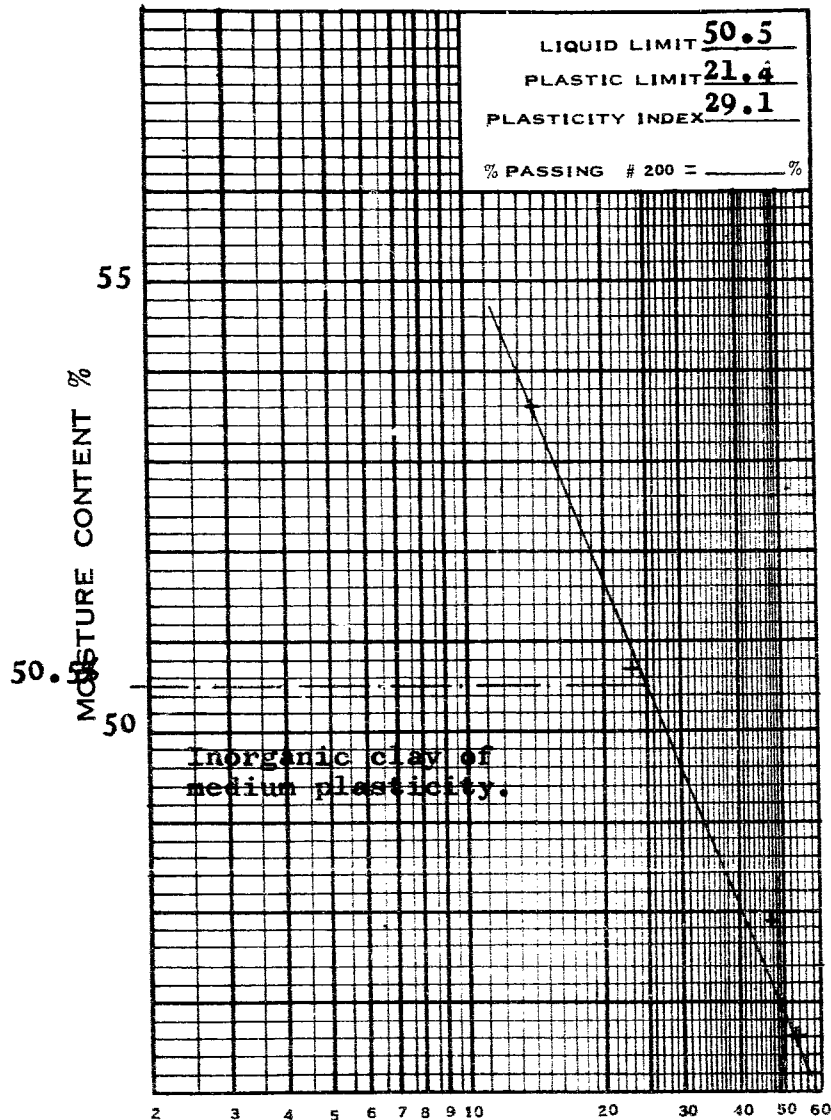


e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

JOB No. 5920 PROJECT Edwardsburg Twp. C.N.R. Overhead Hwy. 16 Johnstown.
SAMPLE FROM B.H. # 2. Sample # 4. SAMPLE FROM B.H. # 2. Sample # 6.
DEPTH 15' - 16' DEPTH 21'6" - 22'6"



e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

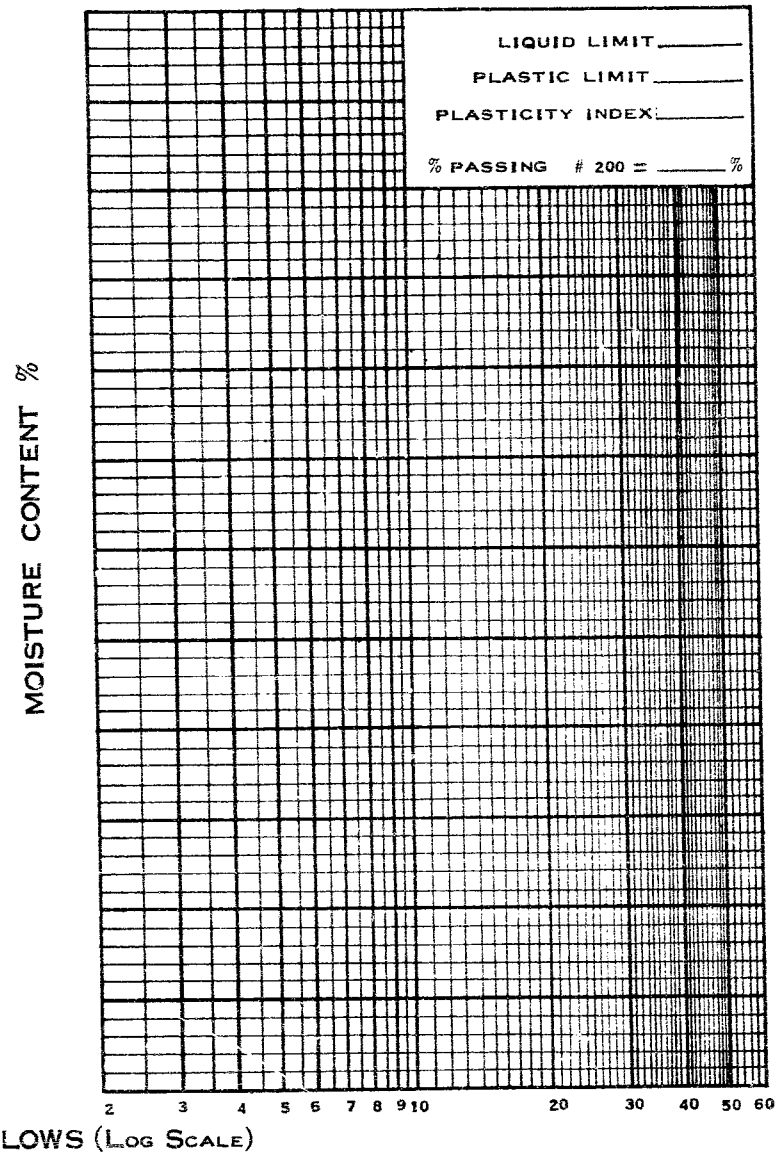
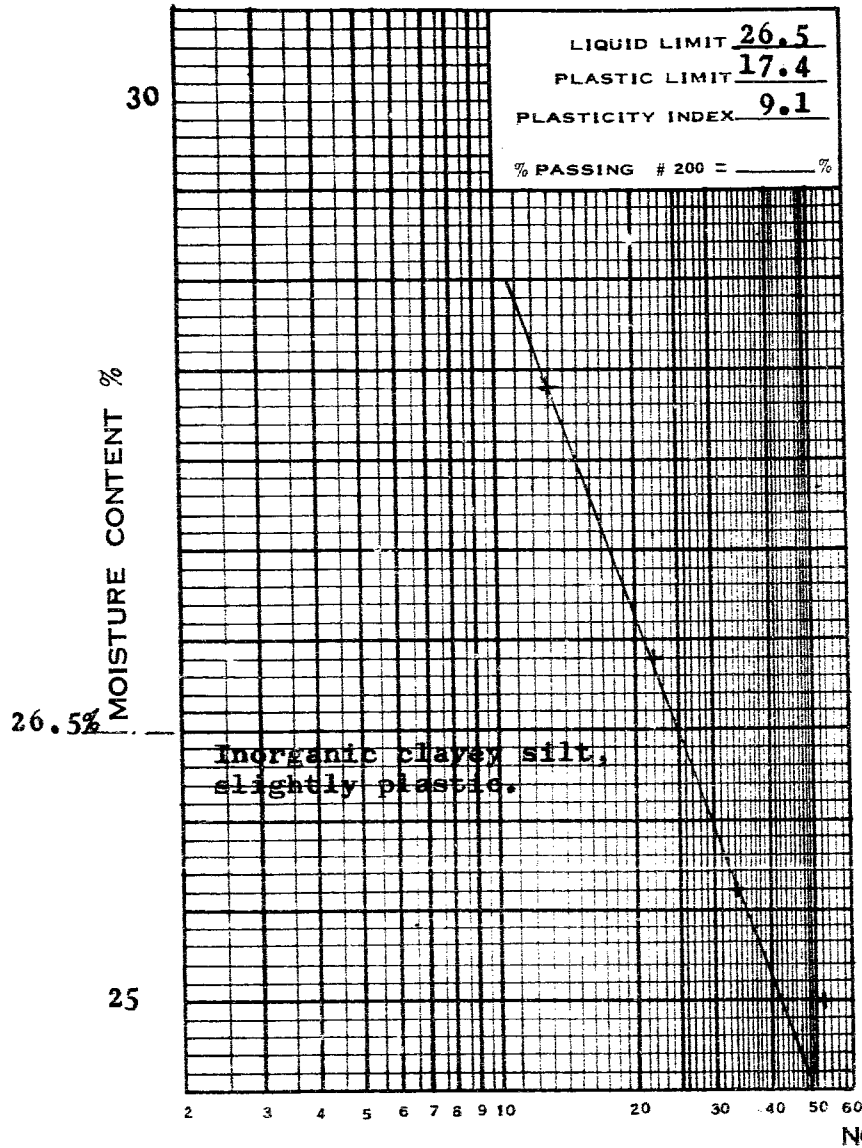
JOB No. 5920 PROJECT Edwardsburg Twp. C.N.R. Overhead Hwy. 16 Johnstown.

SAMPLE FROM B.H. # 2. Sample # 10.

SAMPLE FROM _____

DEPTH 36' - 37'

DEPTH _____

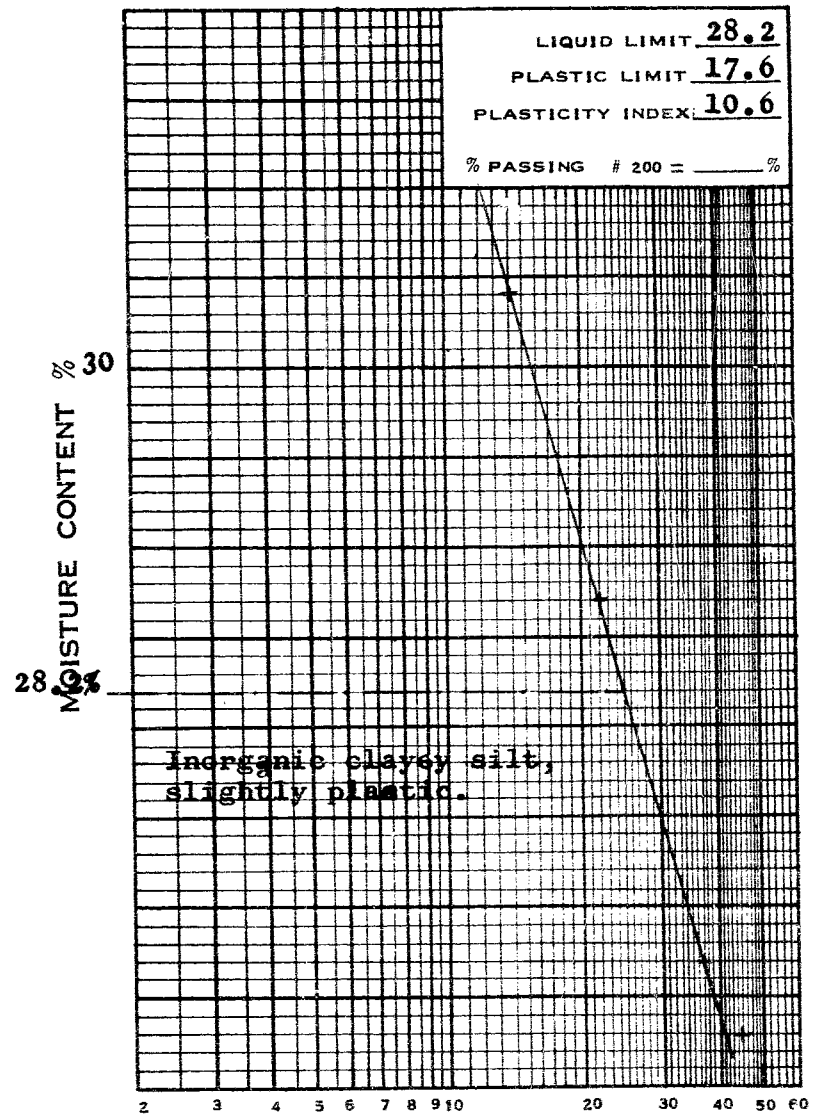
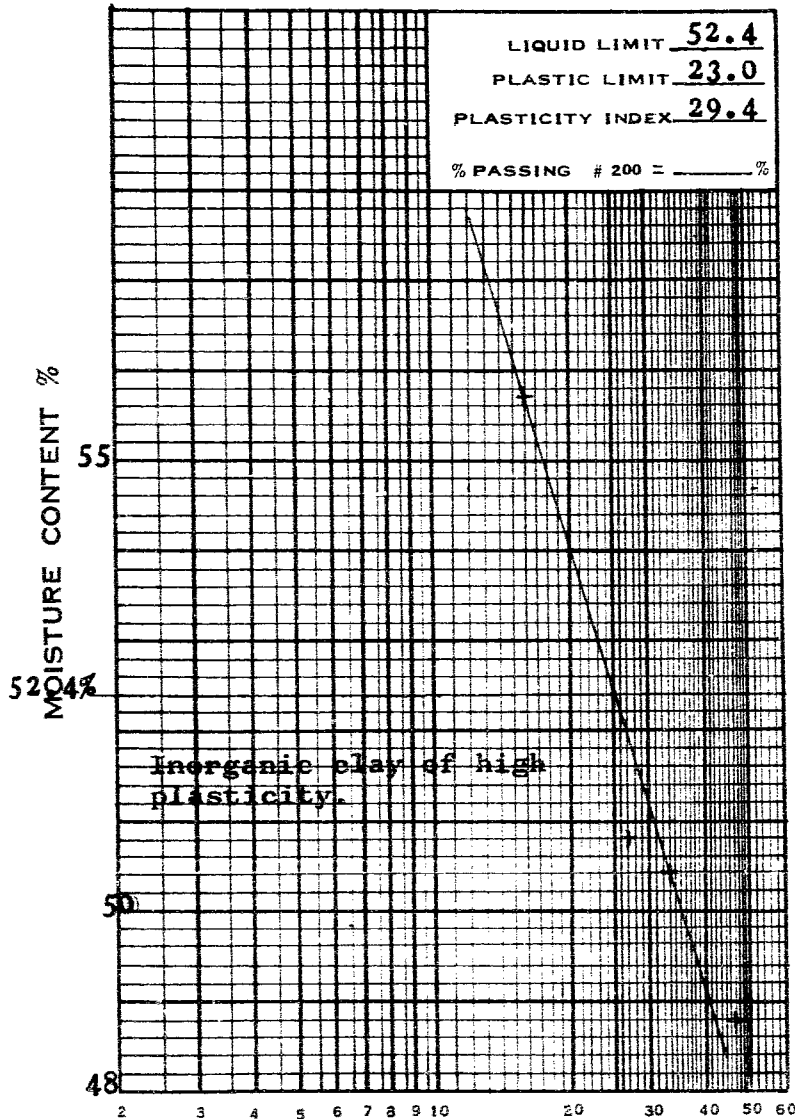


e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

JOB NO. 5920 PROJECT Edwardsburg Twp. C.N.R. Overhead Hwy. 16 Johnstown.
SAMPLE FROM B.H.# 3. Sample # 3. SAMPLE FROM B.H.# 3. Sample # 7.
DEPTH 10' - 11' DEPTH 30' - 31'

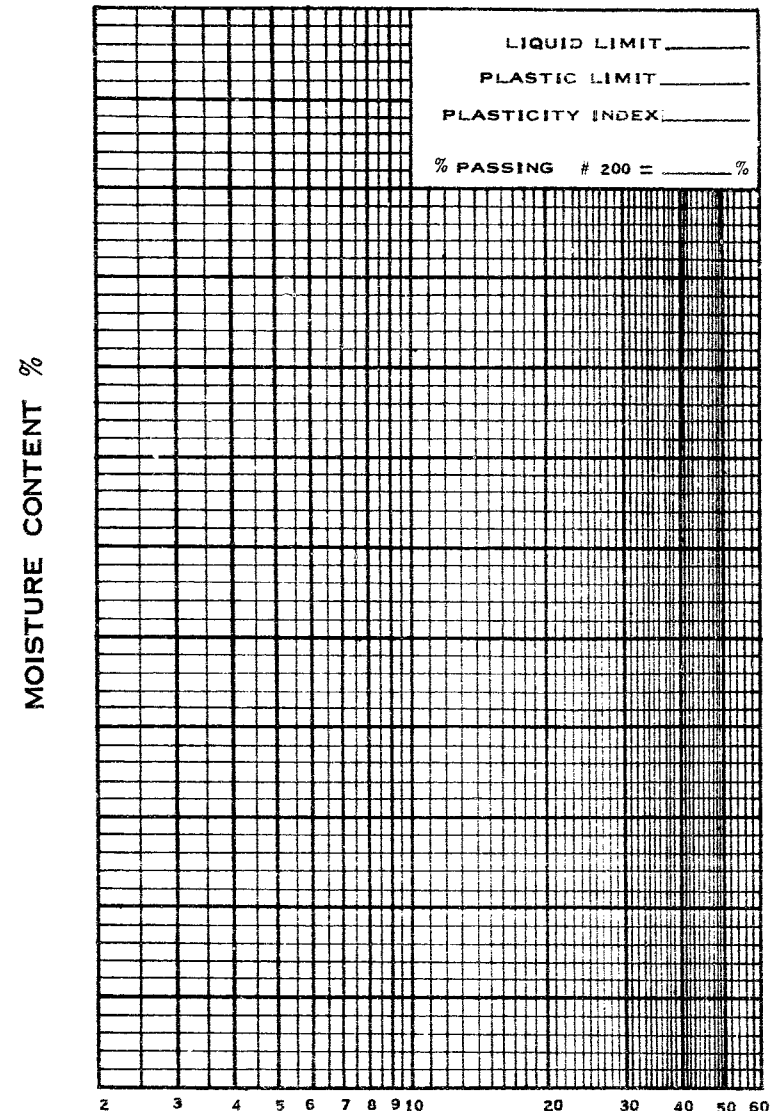
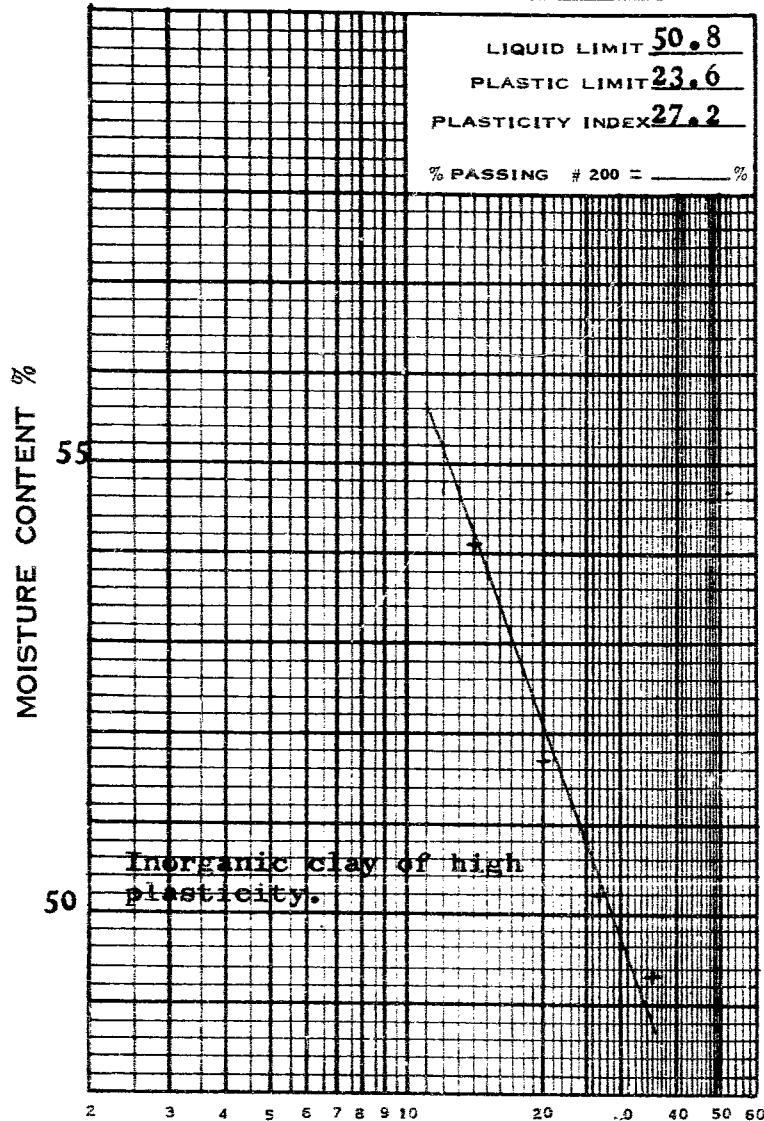


e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

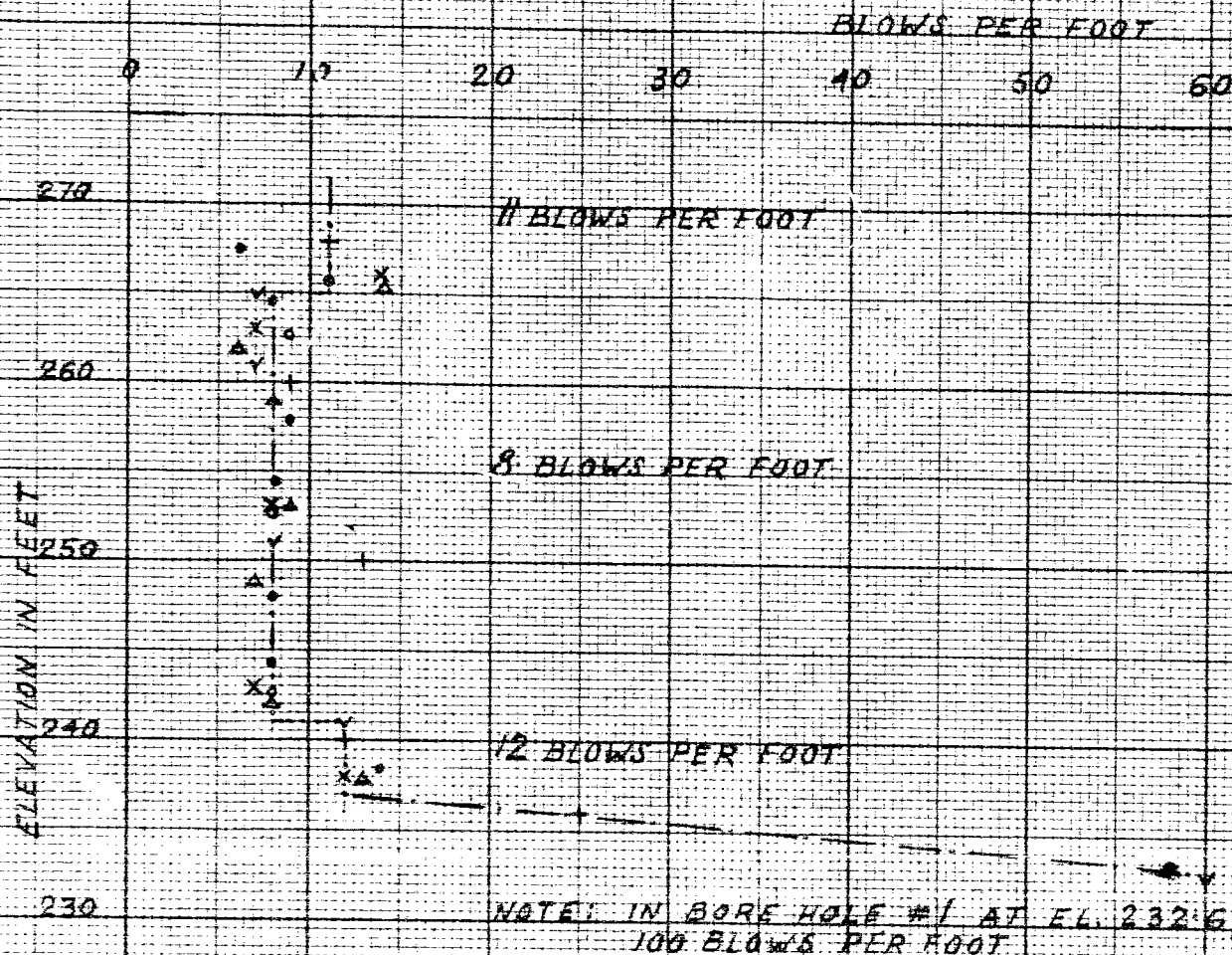
JOB No. 5920 PROJECT Edwardsburg Twp. C.N.R. Hwy. 16 Johnstown.
SAMPLE FROM B.H.# 6. Sample # 3. SAMPLE FROM _____
DEPTH 10' - 11'6" DEPTH _____



JAB NO. 5920.

EDWARDSBURG TWP. C.N.R. HWY # 16
JOHNSTOWN

STANDARD PENETRATION TESTS



BORE HOLE:

- | | |
|---|---|
| 1 | o |
| 2 | • |
| 3 | + |
| 4 | x |
| 5 | Δ |
| 6 | v |

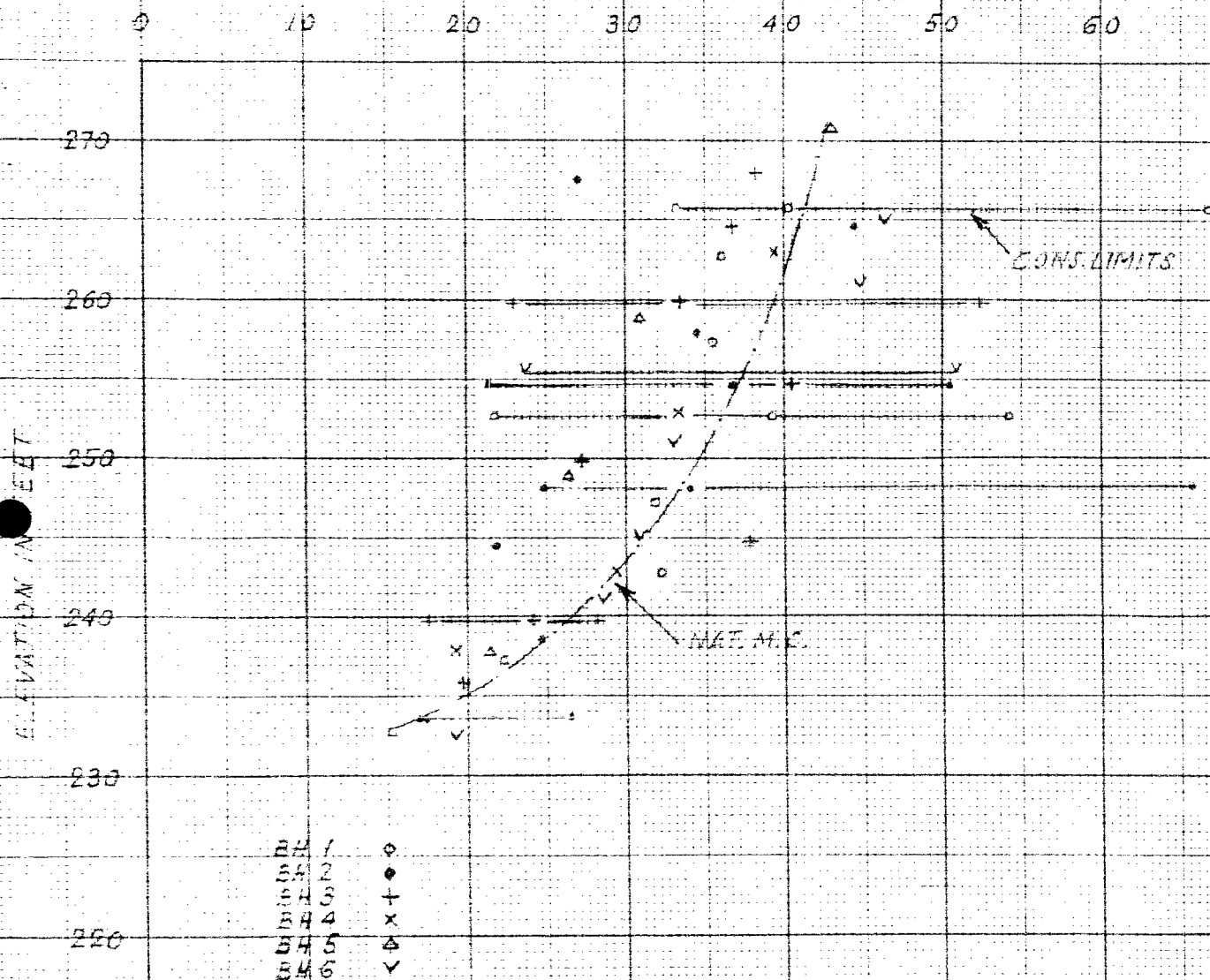
EM PETO ASSOCIATES LTD.

JOB NO. 5920.

EDWARDSBURG TWP. CNR. HWY #16
JOHNSTOWN.

MOISTURE CONTENT VERSUS ELEVATION

CONSISTENCY LIMITS: PL & LL %
NATURAL MOISTURE CONTENT: M.C. %



F. M. PETO ASSOCIATES LTD.

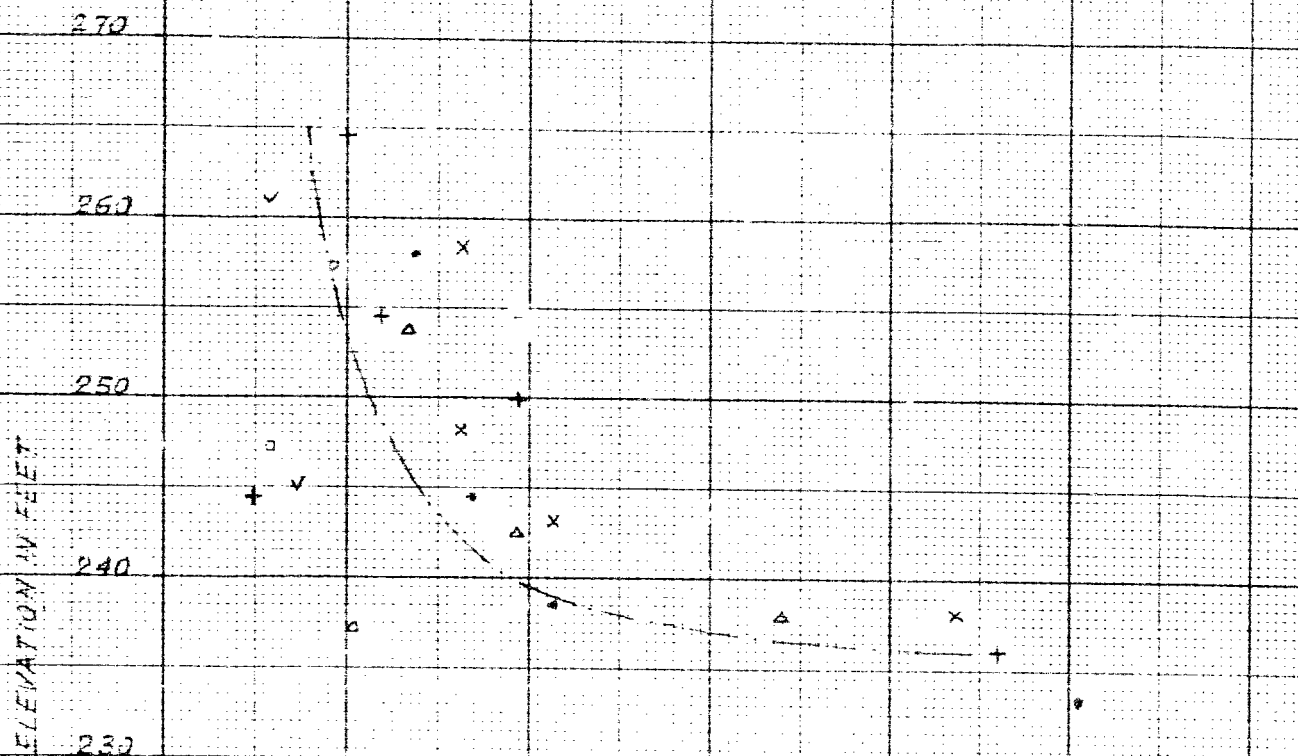
JOB NO. 5920.

EDWARDGEURGETOWN C.M.R. HWY # 16.
JOHNSTOWN.

UNCONFINED COMPRESSION TESTS.

SHEAR STRENGTH IN PSF

1000 2000 3000 4000 5000 6000



BH 1
BH 2
BH 3
BH 4
BH 5
BH 6

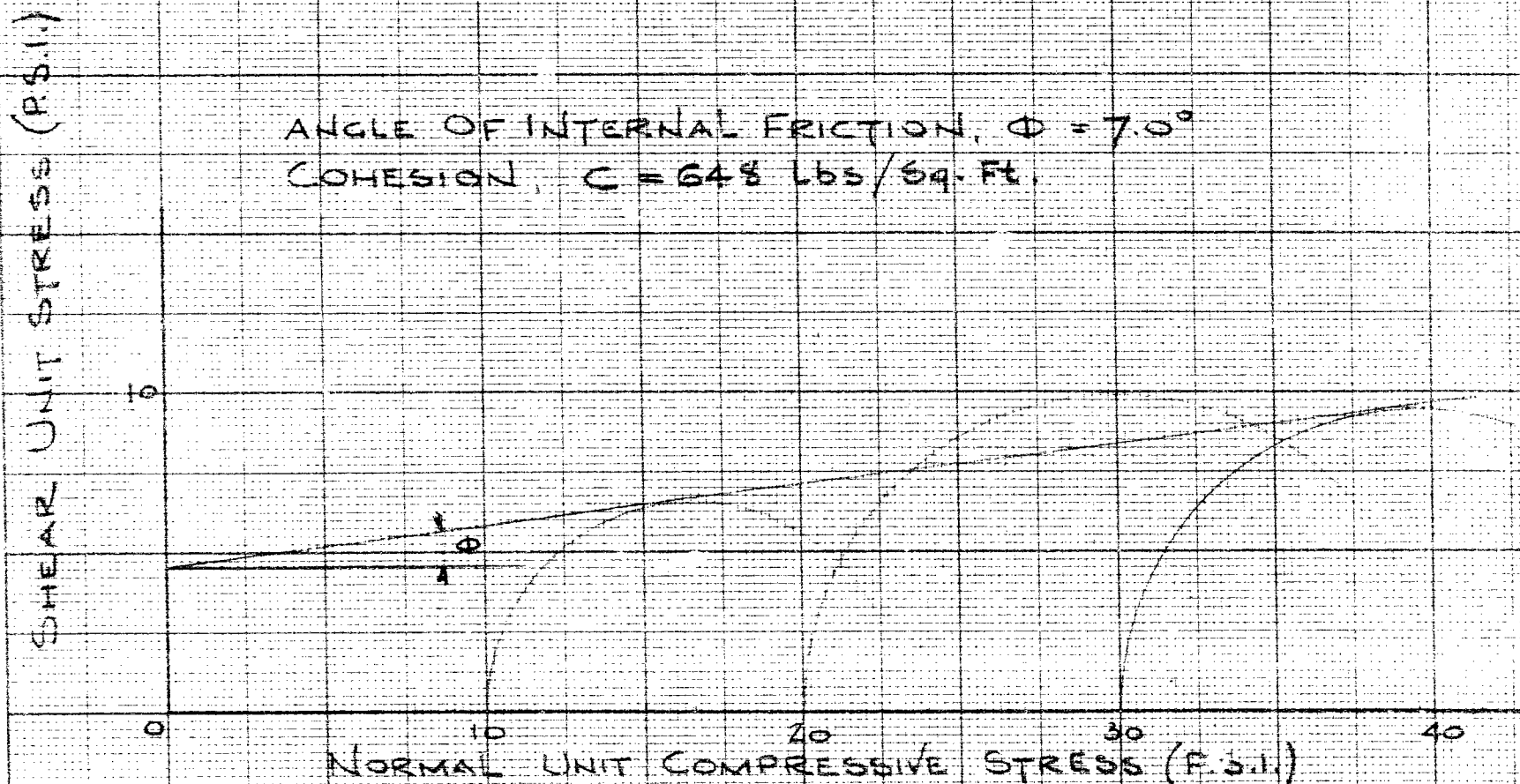
E.M. PETO ASSOCIATES LTD.

QUICK UNDRAINED TRIAXIAL COMPRESSION TEST.
MOHR'S CIRCLE DIAGRAM.

JOB No. 5920 HOLE No. 6
SAMPLE No. 8

DEPTHS: 30'0" - 30'6"
30'6" - 31'0"
31'0" - 31'6"

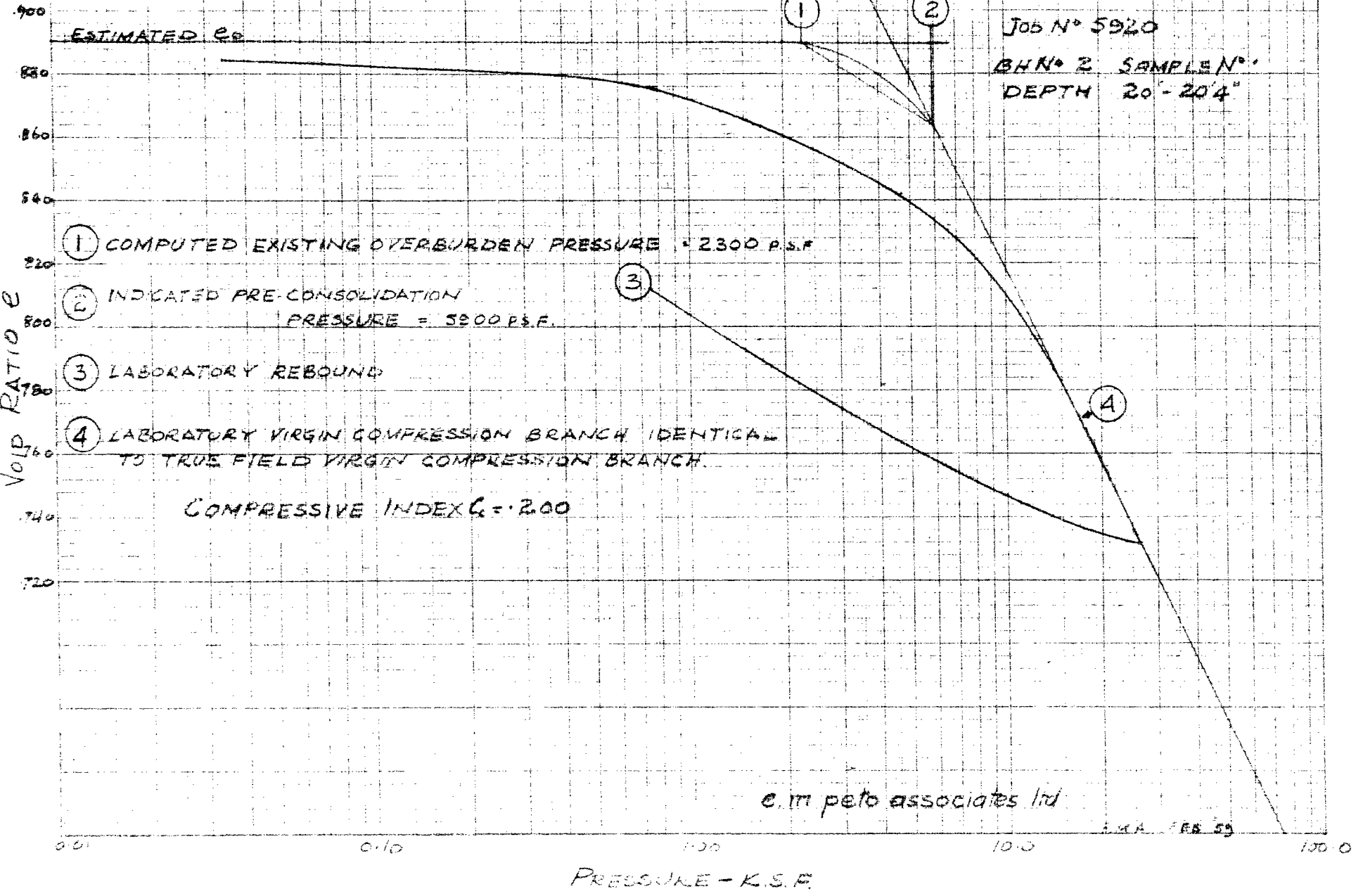
ANGLE OF INTERNAL FRICTION, $\phi = 7.0^\circ$
COHESION, $C = 648$ Lbs/Sq. Ft.



e.m. peto associates Ltd.
MAR. 4. 59 D.T.

CONSOLIDATION TEST

PRESSURE - VOID RATIO CURVE





ONTARIO
DEPARTMENT OF HIGHWAYS

Toronto 5,
March 24, 1959.

MEMORANDUM TO:

Mr. L. Loch,
Bridge Design Engineer,
Bridge Office.

Re: W.P. 217-58, Edwardsburg Twp. C.N.R.,
Hwy. #16, District #9.

Attached please find soil report BA 877 for
the design of the above structure.

A handwritten signature in cursive script, appearing to read "J. C. McAllister".

J. C. McAllister,
for S. McCombie,
Bridge Planning Engineer.

JCM:bh

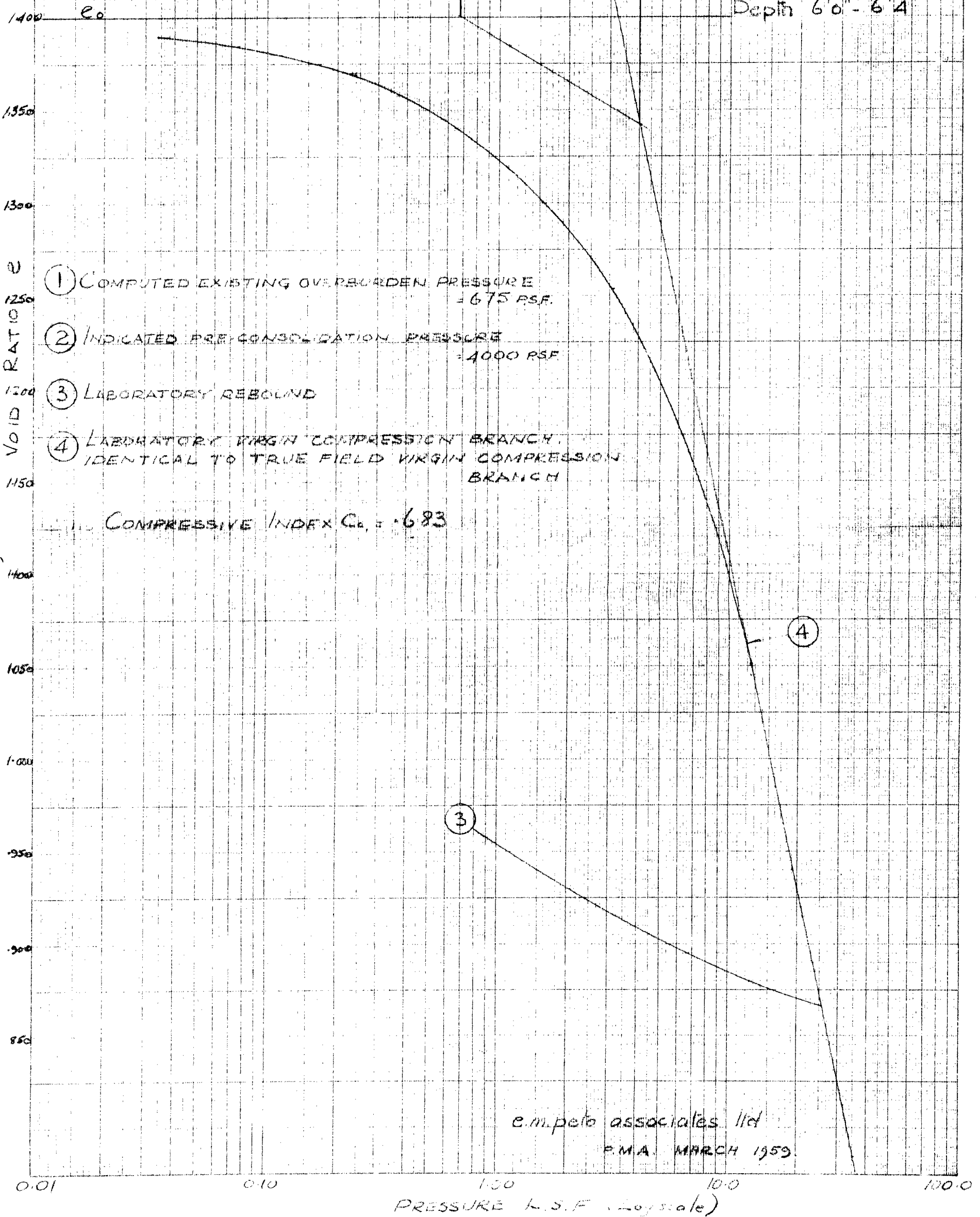
CONSOLIDATION TEST

PRESSURE-VOID RATIO CURVE

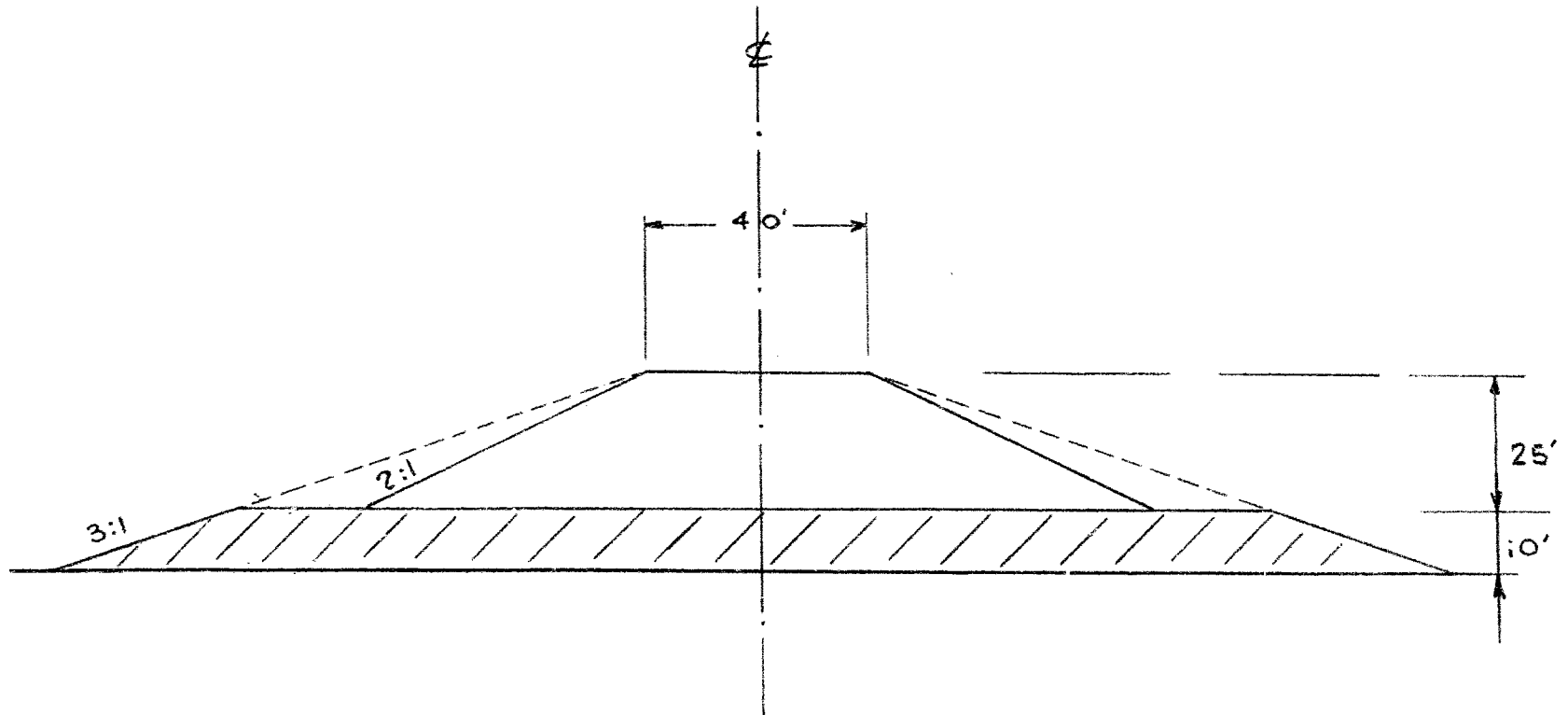
Job N° 5920

B.H.5 Sample 2

Depth 6'0" - 6'4"



APPENDIX II



PROPOSED CROSS - SECTION

FOR TOE BERM METHOD

e.m.peto associates limited.
March 20th.1959.G.T.

JOB NO. 5920

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" - 90° 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.