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e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 57149

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

February 18th, 1958.

57-F-241C

1/2 Mr. J. C. McAllister
From CONSULTANTS
E. L. Peto

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

Attention: Mr. J. C. McAllister

Re: Hwy. 401 - C.N.R. Crossing
W.P. 69-57, Twp. of Charlottensburg

Dear Sirs:

1. We refer to your letter dated December 9th, 1958, in which you requested us to carry out a soils investigation at the proposed site of the above mentioned bridge.
2. Our terms of reference required that we should carry out a complete soils investigation at the site and make a report on our findings together with such recommendations and conclusions as we considered pertinent.
3. We have submitted one copy of this report to your Consultants, Messrs. Laughlin, Wylie and Ufnal, in order to allow them to expedite their design of this bridge.
4. The soil conditions and factors leading to our conclusions are considered in detail in the report attached hereto, together with an Appendix of supporting test results. Here for your convenience is a summary of the conclusions we have reached as a result of these studies:

- a) Only three main classes of material were encountered,
- i) Soft silty clay in the upper stratum in the central portion of the site.
 - ii) Dense glacial till overlying bedrock.
 - iii) Limestone bedrock.
- b) The silty clay is an unsuitable material on which to place foundations, due to its low bearing capacity and high compressibility.
- c) The most suitable method of supporting the bridge is on a steel H-pile foundation.
- d) The ground water table on this site occurs virtually at the present ground surface, and this will present a problem during construction, particularly with regard to any excavations.
- e) The soft silty clay stratum will, in all likelihood, fail under the load imposed by the proposed 33 foot high approach embankments. There are two solutions to this problem which seem to be the most practical:
- i) Lengthen the West approach span by 20 feet, and add a further span of 70 feet to the East approach.
 - ii) Drive a long line of continuous steel sheet piling parallel to the railway tracks on the East side, and in addition, lengthen the two approach spans by 20 feet and 30 feet on the West and East sides respectively.
- f) "Staged" construction of the approach embankments will not be of any great benefit on this particular site.
5. We shall be pleased to supply any further information that the Department or your Consultants may require, wherever possible.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

EM:ab

Job No. 57149

Client's Ref. No.

Date February 13th/58

Report on
SOIL CONDITIONS

at

HWY. 401 - C. N. R. CROSSING

W. P. 69-57 TOWNSHIP OF CHARLOTTENBURG

for

DEPARTMENT OF HIGHWAYS OF ONTARIO

TERMS OF REFERENCE:

On December 9th, 1957, we received a letter from Mr. J. C. McAllister, acting for the Chief Bridge Engineer, instructing us to proceed with the soil investigation for the above project. The Client's plan No. F-3165-9 and profile F-3165-14 were enclosed with the letter, and six suggested borehole locations were shown on the plan. However, we were advised verbally that in view of the proposed relatively high approach embankments, the soil boring programme should be expanded to include a number of boreholes further back from the railway tracks, at or near potential abutment locations, if soil conditions warranted it.

The field work was to be carried out in accordance with our standard practice, but with more frequent sampling in the poorer soil strata.

METHOD OF OPERATIONS:

The site investigation was performed by our number 1 field crew, using a skid-mounted Sullivan "12" diamond drill rig, which was trucked to the site from our office in Toronto on January 10th, 1958. The test holes were staked out on this date, and work commenced the following day. The work was completed on January 29th, 1958, on which date a final check of the ground water levels was made, and the equipment was moved off the site on January 30th, 1958.

METHOD OF OPERATIONS: (Cont'd)

The test holes were sunk by driving and cleaning BX (2-1/2" diameter) drill casing, and sampling ahead of the casing at frequent intervals with either a 2" standard split barrel sampler or an adapted 2" split barrel sampler with brass liners and an extremely sharp, thin cutting edge. Standard penetration test results were recorded whenever the standard split spoon sampler was used, but the split barrel with liners was pushed in every case, to minimize the sample disturbance.

Two of the soil test holes, one on each side of the railway tracks, were diamond drilled using an AXT core barrel and bit in order to prove the reliability and continuity of bedrock. All the other test holes extended as deep as was possible without actual core drilling.

Careful note of the ground water data was kept throughout the performance of the field work.

All soil samples obtained on the site were carefully examined in our laboratory. Additional tests were carried out, as deemed necessary, on selected samples. Detailed borehole logs showing the soils encountered, their physical properties, and the position of the ground water table at each hole are included at the rear of this report, along with some of the graphs of the laboratory test results under cover of Appendix I.

A site plan showing the locations of the soil test holes, together with a longitudinal section along the highway centre-line is attached at the rear.

All elevations in this report are to Geodetic datum, and are referred to a D.S.I. bench mark, which is a nail in the South East root of a 1.5 ft. elm 35 ft. right of Sta. 643 + 50. The elevation of this bench mark was taken to be 186.35.

SITE AND GEOLOGY:

The site lies in the generally physiographic region known as the Glengarry till plain. The topography is strongly undulating to rolling. Many drumlins (elongated rounded ridges) occur in the region, and these roughly parallel the St. Lawrence River valley, indicating that the glaciers which formed these drumlins advanced locally from the Northeasterly direction.

SITE AND GEOLOGY: (Cont'd)

It is of interest to note that although this part of North America was subject to at least three glaciations, only the soils of the last (Wisconsin) glaciation influence the conditions at this site.

Following the recession of the Wisconsin ice, an arm of the sea, named Gilbert Gulf, entered this area. Much sediment was formed in the depths of this post-glacial water. The ridge to the West of the site is strewn with many boulders and stones, a result of the removal of fine material from the crests to the valleys during the submergence.

Over much of the area the till is less than 25 feet in depth, and directly overlies bedrock, which is a Chazy or Black River Limestone.

SOIL CONDITIONS:

Soil conditions at the centrally located holes close to the railway tracks were almost identical, but results at some of the holes for the approaches at first appeared to be both inconsistent and peculiar. However, careful consideration of the geological history and topography of this site have provided the necessary explanation. The fine-grained water-laid silty clay was encountered only at the central portion of the proposed bridge site; this was due to an old arm of the Gilbert Gulf or, more likely, a post-glacial stream bed following a course between the drumlin ridges on either side. The railway lines apparently also follow such a course in order to meet grade requirements.

All the soils encountered above bedrock at this site fall into only two main classifications:

1. Glacial till with a heavy loamy texture and a high proportion of limestone.
2. Very fine-textured, relatively stone-free, calcareous sedimentary silty clay, less alkaline than the adjacent glacial till, and generally inadequately drained.

SOIL CONDITIONS:Silty Clay

The central portion of the site investigated, which unfortunately encompasses the probable pier and abutment locations of the bridge, is overlain by from 6-2/3 feet of silty clay at borehole 10 on the West side of the tracks to 26-1/3 feet at borehole 8 on the East side.

The upper horizons of this silty clay stratum appear to be somewhat weathered or leached, and are a mixed grey and brown to light grayish-brown in colour. At depth, however, this material is grey. It is of friable consistency throughout.

In order to better classify the material in this stratum three Atterberg limit tests and one hydrometer grain size test were performed. The liquid limit, plastic limit and plasticity index respectively were in the ranges 61.0% - 76.6%, 25.5% - 27.4%, and 35.5 - 49.2, indicating that this is an inorganic clay of high plasticity and high compressibility. The hydrometer grain size analysis on this soil showed that the clay size particles are by far the most predominant, the soil being composed of 69% clay, 29% silt and only 2% fine sand.

Natural moisture contents in the silty clay stratum ranged from a low of 34.3% in the grey-brown soil near ground surface to a high of 73.1% in the grey clay at depth. Generally the natural moisture contents were much in excess of the plastic limit, and in some cases were actually higher than the liquid limit.

A large number of shear strength tests were performed on undisturbed samples from the silty clay stratum. Although the results of these tests appear to be highly variable, they are actually quite consistent if we divide them into two groups; a) the grey-brown silty clay near surface with lower moisture contents, and b) the grey silty clay at depth with higher moisture contents. The average unconfined compressive strength of the grey-brown clay was found to be 1229 lbs. per sq. ft., and that of the grey clay 578 lbs. per sq. ft. This latter value was corroborated by the quick undrained triaxial compression test, which indicated a negligible angle of internal friction (to be expected for a saturated clay), and a cohesion of 302 p.s.f. corresponding to an unconfined compressive strength of 604 p.s.f. The ratios of undisturbed to remoulded shear strengths, from three tests, were found to be 1.92, 2.26 and 3.49, indicating that the silty clay has low sensitivity.

SOIL CONDITIONS:Silty Clay (Cont'd)

The wet density of this soil is in the order of 120 p.c.f., and the dry density approximately 75 p.c.f. However, since the water table at the central portion of this site is virtually at surface, only the submerged unit weight of 55 p.c.f. should be used for design purposes.

Glacial Till

Underlying the silty clay stratum, and occurring at surface on both approaches to the bridge site is a grey to dark grey glacial till, containing a high proportion of grits and angular limestone fragments, and considerable clay and silt binder. The one exception to this is on the ridge at the West approach, where the finer-grained materials have been washed away, leaving only sandy till.

The till is heterogeneous in character, at some points even becoming predominately silty. It is loose or compact near its upper boundary, increasing to extremely high density at depth. Standard penetration test results in the glacial till varied from a low of only 6 blows to a high of well over 150 blows. Natural moisture contents of the till are between 7.8% and 16.2%, and the Modified Proctor optimum moisture content would be approximately 10%.

The unit wet weight of the material in situ is in the order of 140 p.c.f., and its high silt content would definitely make it a frost susceptible soil.

Limestone Bedrock

Directly beneath the glacial till is the parent material on this site, a slightly crystalline limestone, grey-black in colour, with occasional fossil-bearing seams of black shale.

Although the quality of the limestone is generally excellent at depth, the upper 3 or 4 feet are either slightly fractured and faulted, or else consist of alternate strata of rock and dense sandy till. No major water-bearing seams or large faults were noted in the two holes drilled to prove bedrock.

SOIL CONDITIONS:Limestone Bedrock (Cont'd)

In the central portion of the site, along the highway 401 centre-line, the top boundary elevation is almost constant at from 141.7 to 144.6. However, at both approaches the bedrock appears to rise sharply and to be at a much shallower depth. None of the test holes for the approach road were cored to prove bedrock, since these holes do not affect the foundations of the bridge itself, and provide sufficient information for analyzing the approach embankment problem. The primary consideration in these test holes was to establish the type and consistency of the soil above bedrock.

WATER CONDITIONS:

The water table on this site at the time of the investigation was virtually at surface, as shown on the profile. At two of the holes, namely boreholes 1 and 6, ground water was actually observed to be flowing very slowly from the hole. This water emanated from the stratum immediately above the bedrock surface, but the flow was of small magnitude, and this does not constitute a true quickening condition.

Because of the poor drainage and the high water table, any unsheeted excavation on this site would almost certainly fill with water, necessitating practically continuous pumping. Owing to the configuration of the ground, which forms almost a natural water basin at the site, it would be difficult to lower the ground water table.

ENGINEERING CONSIDERATIONS:

There are a number of alternatives regarding the construction of this bridge, which will affect the design, and which must be considered before the final conclusions can be drawn. These are:

1. The railway line to be crossed is a main line, and the traffic cannot be disrupted. It may be possible, however, to re-route the railway traffic to adjacent lines in the area, excavate all of the soft silty clay below the railway tracks and the proposed approach embankments, replace with suitable fill, and then rebuild the railway grade to original line and level. The excavation and replacement of fill would be made exceedingly difficult by the high water table. The feasibility of such a suggestion is outside the scope of this report.

ENGINEERING CONSIDERATIONS: (Cont'd)

2. It is presently proposed to construct approach embankments to the railway crossing which would be up to 33 feet high, and with a base width of approximately 200 feet. Such an embankment would apply shearing stresses to the soft clay subsoil under the roadway which are dangerously close to or exceed the ultimate shear strength of the grey silty clay. Calculations by a number of recognized methods were made, among them the Prandtl and the Terzaghi theories. The former indicated a very nearly critical condition and the latter method predicted definite failure.

It is apparent that the actual shear strength (as opposed to ultimate strength under a strip load) will be exceeded. One would normally expect, therefore, that plastic flow would occur in the clay under the new embankment and considerable settlement would take place, with a heave of the existing soil beyond the toe of the embankment. However, the clay material tested invariably did not exhibit plastic failure, but failed in a brittle manner, due to its friable, anggetty structure and its high moisture contents. This leads us to the conclusion that if the full embankment shown on our profile was constructed as presently proposed, the subsoil would in all likelihood undergo a sudden major shear failure, probably disturbing the railway grade.

Unfortunately, this is not a case where gradual loading in a series of stages would be of any great value, because of the difficulty of draining the clay layer sufficiently for consolidation to take place.

3. Even though the possibility of excavation of all the clay material may be ruled out, (because of the 20 ft. thickness under the railway tracks) we have considered the removal of the clay material under the approach embankment only, keeping the excavated slope angle below the critical value.

- complete slope stability analysis was made to determine the feasibility of this solution, but the results indicated that this method would be impractical.

4. Extend the West approach span, and add a further span to the East approach, so that any shear failure involving soil movement, will not affect the railway tracks.
5. Increase the length of both approach spans by relatively small amounts, and drive steel sheet piling along a line parallel to the railway tracks, at approximately the toe of the West side of the railway embankment, to prevent slip circle failure.

ENGINEERING CONSIDERATIONS: (Cont'd)

6. Construct a cofferdam of steel sheet piling enclosing the poor material underlying the proposed East approach embankment, followed by placement of the fill. This method will be quite expensive, since the sheet piling will have to be heavily reinforced to prevent buckling under the high embankment loads.
7. Seek an alternative railway crossing for Highway 401, involving re-alignment of a considerable portion of the road, with the accruing problems of acquisition of right-of-way, etc.

RECOMMENDATIONS AND CONCLUSIONS:

After due consideration of the alternatives listed above, only two of them appear to be practical. The final choice depends on the relative economies of the two schemes.

1. Increase the length of the West approach span by 20 feet, and add a second approach span on the East side of the 70 feet. The actual apportioning of the span lengths would naturally rest with the Consultants. This in effect involves the terminal toes of the approach embankments being moved 20 feet and 70 feet respectively further away from the railway tracks.
2. Drive approximately 120 lineal feet of fairly heavy gauge, interlocking, steel sheet piling to refusal along a line parallel to the present railway embankment, and adjacent to its toe on the East side. In conjunction with this, lengthen the West and East approach spans by 20 and 30 feet respectively.

The sheet piling will become a permanent part of the bridge works, and cannot be removed. It serves to prevent the development of a full slip circle in the silty clay.

Regardless of which of the two proposals is finally selected, the approach embankments must be placed before the construction of the bridge.

The bridge piers and abutments should be founded on steel H-piles driven into the top surface of the bedrock. The piles for the spill-through type abutments will have to be driven through the fill.

RECOMMENDATIONS AND CONCLUSIONS: (Cont'd)

We believe that the soils on this site are mildly alkaline in reaction, and will not prove detrimental to steel piling.

Timber piles are not recommended for this site, because a) of the likelihood of damage during driving through the glacial till, and b) because of the difficulty of achieving the desired penetration, resulting in insufficient lateral support of the pile tips.

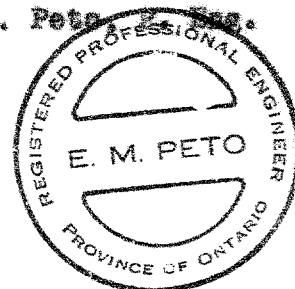
Monotube, or similar displacement piles, are not recommended for the foundations for generally similar reasons to those given above.

E. M. PETO ASSOCIATES LTD.,



MM:sb

E. M. Peto, P. Eng.



E. M. Peto Associates Ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name: Hwy. 401 - C.N.R. Crossing Job No. 57149 Borehole No. 1
Client: Dept. of Highways of Ontario Casing: BX (2-1/2" diam.) Boring Date: Jan. 23rd - 25th, 1958.
Datum: Geodetic Compiled By: M. Mindess Checked By: E.M. Peto

SAMPLE CONDITION		SAMPLE TYPE		ABBREVIATIONS	
	UNDISTURBED	S.S. 2" STANDARD SP. TUBE SAMPLE		V.T. IN SITU VANE SHEAR TEST	
	FAIR	S.L. SPLIT BARREL WITH LINERS		Q. UNCONFINED COMPRESSIVE STRENGTH	
	DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE		W.L. WATER LEVEL IN CASING	
	LOST	W.S. WASH SAMPLE		W.T. GROUND WATER TABLE IN SOIL	
		R.C. ROCK CORE			
SOIL DESCRIPTION	COLOR	Consistency	Depth (Feet)	Sample Type	Remarks
ICE			0' 0" - 1' 0"	ICE	7' Hole flowing over very slowly upon completion. Water table at surface.
SILTY CLAY, NUGGETTY. GREY & BROWN	MIXED	FIRM.	1' 0" - 5' 0"	S.S. 8	ALMOST AT PLASTIC LIMIT. $Q_u = 1440$ P.S.F.
AS ABOVE.	GREYISH-BROWN	"	5' 0" - 10' 0"	S.S. 8	MOIST. SLIGHTLY WETTER THAN PLASTIC LIMIT. $Q_u 5' - 6' = 1599$ P.S.F.
SILTY CLAY, ODD PEBBLES.	"	SOFT	10' 0" - 16' 6"	S.L. 8 PUSHED. S.S. 8	$Q_u 8' - 9' = 805$ P.S.F. $Q_u 9' - 9\frac{1}{2}' = 1035$ P.S.F. NAT. MC = 49.4%
CLAYEY AND SILTY VERY FINE SAND, GRITS & PEBBLES.	GREY	LOOSE	16' 6" - 15' 0"	S.S. 8	3/4" QUITE MOIST.
SILTY FINE TO VERY COARSE SAND, GRITS & ROCK FRAGMENTS.	"	COMPACT	15' 0" - 20' 0"	S.S. 23	WET.
SILTY FINE SAND, GRITS AND PEBBLES. MINOR CLAY CONTENT.	"	"	20' 0" - 25' 0"	S.S. 13	MOIST.
AS ABOVE, NUMEROUS GRITS AND PEBBLES.	"	"	25' 0" - 30' 0"	W.S. -	
FINE SANDY AND CLAYEY SILT, GRITS AND ROCK FRAGMENTS.	"	"	30' 0" - 33' 0"	S.S. 14	MUCH WETTER THAN PLASTIC LIMIT.
AS ABOVE	DARK GREY	DENSE	33' 0" - 35' 0"	S.S. 58	MOIST. WETTER THAN PLASTIC LIMIT.
CLAYEY SILT, FEW GRITS AND ROCK FRAGMENTS.	"	"	35' 0" - 38' 0"	S.S. 43	SLIGHTLY DRIER THAN PLASTIC LIMIT.
ALTERNATE STRATA OF ROCK AND SAND.	"	VERY HARD	38' 0" - 40' 0"	CHOIPPED WITH BX CROSS CHOIPPING BIT.	
SLIGHTLY CRYSTALLINE LIMESTONE, FEW FOSSILS AND 3' SEAMS OF BLACK SHALE, SOME SAND STRATA.	GREYISH-BLACK	HARD	40' 0" - 45' 0"	AXT R.C.	GOOD REACTION WITH DILUTE HYDROCHLORIC ACID. 31.7% RECOVERY.
DOLOMITIC LIMESTONE WITH SMALL MARINE FOSSILS, OCCASIONAL THIN BANDS OF BLACK, FINE-GRAINED SHALE.	GREY-BLACK	VERY HARD	45' 0" - 50' 0"	AXT R.C.	GENERALLY OF EXCELLENT QUALITY, EXCEPT FOR SAND SEAMS. 100% RECOVERY.
AS ABOVE.	"	"	50' 0" - 55' 0"	AXT R.C.	92.7% RECOVERY.

HOLE TERMINATED.

BOREHOLE LOG

Compiled By

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

[illegible]

BOREHOLE LOG

Borehole No. 3
Boring Date Jan. 28th, 1958
Checked By E. M. Peto

ABBREVIATIONS

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

REFUSAL PROBABLY BEDROCK





E. M. Peto Associates Ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Hwy. 401 - C.N.R. crossing
Client Dept. of Highways of Ontario
Datum Geodetic

Job No. 57149
Casing BX (2-1/2" diam.)
Compiled By M. Mindess

Borehole No. 4
Boring Date Jan. 11th - 14th, 1958.
Checked By E.M. Peto

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	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Combination	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
ORGANIC SILTY LOAM	BLACK		0' 0" 176.9					W.T. = 0' 10" JAN. 22, 1958
Clayey silt, nuggety texture.	Greyish-Brown	Firm	1' 6" 175.4		1	S.S.	5	NAT. M.C. = 43.3% Moist.
As above	Light Greyish-Brown	Firm	5' 0"		2	S.T. PUSHED (15)		$Q_u = 1166$ P.S.F. NAT. M.C. = 56.3% SENSITIVITY = 1.92
As above, grits	Greyish Brown	Firm			3	S.S.	5	$Q_u = 648$ P.S.F. NAT. M.C. = 50.9%
As above	Gray	Soft	10' 0" 166.9		4	S.L. PUSHED		4A: $Q_u = 592$ P.S.F. M.C. = 67.0% 4B: $Q_u = 328$ P.S.F. M.C. = 66.8% 4C: $Q_u = 1448$ P.S.F. M.C. = 65.6% SENSITIVITY SAMPLE 48 = 2.26
As above	Grey	Very soft			5	S.S.	2	Much wetter than Plastic Limit.
As above	Grey	Very soft	15' 0"		6	S.L. PUSHED		NAT. M.C. = 50.1%
Silty clay, thin lenses of gray fine sand.	Dark Grey	Very soft			7	S.S.	1	M.C. CLAY = 53.8% M.C. SAND = 11.6% $Q_u \approx 400$ P.S.F. Much wetter than Plastic Limit.
Clay and silty coarse sand, grits and pebbles.	Grey	Compact	19' 6" 157.4		8	S.S.	29	Saturated.
			25' 0"					
Sandy and clayey silt with numerous angular rock fragments.	Dark Grey	Compact	28' 0" 148.9		9	S.S.	14	Moist. Right at Plastic Limit. STIFFENED AT 28'
			30' 0"					
As above, but non-Plastic	Dark Grey	Very dense			10	S.S.	106	Moist
								CHOPPED WITH BX CROSS CHOPPING BIT
Silty fine to coarse sand	Dark Grey	Very dense	35' 2" 141.6		11	S.S.	150/11"	
					12	S.S.	300/11"	SAMPLE 13: WASH SAMPLE FROM 34' 9" TO 35' 3"

VIRTUAL REFUSAL. HOLE TERMINATED.

TRIAxIAL TEST ON SAMPLES GA, GB, GC:
C = 302 P.S.F.
 $\phi = 1^\circ$
NAT. M.C.'s: 73.1%, 69.1%, 70.1%

WET DENSITY SA.4C = 116.1 P.C.F.
DRY DENSITY SA.4C = 70.2 P.C.F.

BOREHOLE LOG

Job Name Hwy. 401 C.N.R. Crossing

Job No. 57149

Borehole No. 6

Client Dept. of Highways of Ontario

Casing BX (2-1/2" diam.)





Boring Date Jan. 16th - 19th, 1958.

Datum Geodetic

Compiled By M. Mindess

Checked By E. M. Peto

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
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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	Log	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
ORGANIC SILTY AND SANDY LOAM	BROWNISH-BLACK		0' 0" 177.5					▽ Hole flowing over very slowly upon completion. Water table at surface.
CLAYEY SILT, WITH FINE TO VERY COARSE SAND & GRITS	GREYISH-BROWN	VERY LOOSE	4' 0" 173.5		1	S.S.	1 1/2	SATURATED
SILTY CLAY	GREY	VERY SOFT			2	S.S.	1	NAT. M.C. = 78.1%. CLOSE TO LIQUID LIMIT Q _u ≈ 450 P.S.F.
"	"	"	10' 0" 167.5		3	S.S.	1 1/2	NAT. M.C. = 94.0% WETTER THAN LIQUID LIMIT
"	"	"	16' 0" 161.5		4	S.S.	1 1/2	NAT. M.C. = 52.3%
"	"	"	15' 0"		5	S.S.	2	NAT. M.C. = 61.3%. Q _u = 382 P.S.F.
"	"	"			6	2" S.T.	PUSHED	
AS ABOVE, ODD GRITS	"	"	21' 0" 156.5		7	2" S.T.	PUSHED	NAT. M.C. 20'-20 1/2' = 63.3%
CLAYEY AND SILTY FINE SAND. MANY GRITS & PEBBLES.	GREY	LOOSE	25' 0"		8	S.S.	10	NAT. M.C. 20 1/2' - 21' = 42.3% (DUE TO SAND SEAMS) NAT. M.C. = 10.5%
MATRIX OF CLAYEY SILT, WITH MANY ANGULAR GRITS & PEBBLES.	"	"	30' 0"		9	S.S.	6	NAT. M.C. = 16.2%
AS ABOVE	GREY	VERY DENSE	36' 0"		10	S.S.	140	
SILTY VERY FINE SAND, MANY GRITS AND PEBBLES, MINOR CLAY CONTENT.	GREY	VERY DENSE	42' 0" 142.5		11	W.S.		
SHALE, DENSE SAND LAYERS.	GREY-BLACK	EXTREMELY DENSE	38' 0"		12	W.S.		USED CHOPPING BIT. PROGRESS VERY DIFFICULT.
FINE-GRAINED LIMESTONE, SMALL MARINE FOSSILS AND SOME IRON PYRITES	GREYISH-BLACK	SLIGHTLY HARDER THAN GLASS	40' 0" 137.5		13	AXT R.C.		REACTS FREELY WITH DILUTE HCl
COARSE-GRAINED DOLOMITIC LIMESTONE, WITH VERY THIN BANDS OF BLACK MATERIAL AS ABOVE, FEW FOSSILS, SOMEWHAT CRYSTALLINE FROM 43' - 45'	DARK GREY	VERY HARD	43' 0" 134.5		14	AXT R.C.		CORE RECOVERY 33' - 45' = 91.1%
			45' 0"		15	AXT R.C.		GENERALLY VERY GOOD QUALITY THROUGHOUT
			55' 0" 127.5		16	AXT R.C.		

HOLE TERMINATED





e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401 - C.N.R. Crossing Job No. 57149 Borehole No. 7
 Client Dept. of Highways of Ontario Casing BX (2-1/2" diam.) Boring Date Jan. 20th, 1958.
 Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

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
ORGANIC SILTY LOAM	BROWNISH-BLACK		0' 0" 177.3					W.T. = 0' 10", JAN. 22, 1958.
SILTY CLAY, NUGGETTY TEXTURE	MIXED GREY & BROWN	FIRM	2' 0"		1  S.S.	8		MOIST SLIGHTLY DRIER THAN PLASTIC LIMIT
AS ABOVE, VERY NUGGETTY	AS ABOVE	FIRM	5' 0"		2  S.S.	8		MOIST AT PLASTIC LIMIT
AS ABOVE	GREY-BROWN	SOFT			3  S.S.	3 1/2		NAT. MC = 64.8% MUCH WETTER THAN PLASTIC LIMIT
" " SOME GRITS	BROWNISH-GREY	VERY SOFT	10' 0" 167.3		4  S.L.	PUSHED		NAT. MC = 64.0%
" "	"	"			5  S.S.	1 1/2		Q _u 10' 10 1/2" = 559 P.S.F. MC = 54.1%
" "	GREY	"	15' 0"		6  S.S.	1		NAT. MC = 55.0%
" "	GREY	"	20' 0"		7  S.L.	PUSHED		NAT. MC = 91.0%
" "	"	SOFT			8  S.S.	4		
SILTY CLAY, WITH COARSE SAND AND GRITS	"		24' 0" 153.3		9  W.S.	2 1/2		
CLAYEY AND SILTY VERY FINE SAND, MANY GRITS AND PEBBLES	GREY	DENSE			10  S.S.	48		MOIST
SANDY AND CLAYEY SILT, MANY GRITS AND ANGULAR ROCK FRAGMENTS	"	VERY DENSE	30' 0"		11  S.S.	54		SLIGHTLY MOIST
		EXTREMELY DENSE	34' 0" 142.6					CHOPPED WITH BX. CROSS CHOPPING BIT.
								REFUSAL, PROBABLY BEDROCK.

C. M. Peto Associates Ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Hwy. 401 - C. N. R. Crossing Job No. 57149 Borehole No. 8
Client Dept. of Highways of Ontario Casing BX (2-1/2" diam.) Boring Date Jan. 21st, 1958.
Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

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
ORGANIC SILTY LOAM.			0' 0" 177.9					DEPTH OF FROST = 1'0" W.T. = 1'6" JAN. 22, 1958.
Clayey silt, nuggety. Organic content.	Mixed Grey-Brown	Firm	5' 0"	1	 S.S.	5		Moist
Clayey silt, nuggety. odd pebbles.	Mixed Grey-Brown	Firm	5' 0"	2	 S.S.	9		Slightly wetter than Plastic Limit.
As above			10' 0" 167.9	3	 S.L.	PUSHED		$Q_u \frac{8\frac{1}{2}-9}{8} = 1299$ P.S.F. NAT. M.C. = 54% $Q_u \frac{9-9\frac{1}{2}}{8} = 608$ P.S.F. NAT. M.C. = 54%
As above	Brownish Grey	Soft	15' 0"	4	 S.S.	4		$Q_u \approx 950$ P.S.F. Much wetter than Plastic Limit.
As above.			15' 0"	5	 S.L.	PUSHED		NAT. M.C. = 61.3% $Q_u \approx 330$ P.S.F. M.C. $\frac{14-14\frac{1}{2}}{14} = 42.0\%$
As above	Grey	Soft	20' 0" 157.9	6	 S.S.	3		Much wetter than Plastic Limit.
As above	Grey	Very soft	25' 0"	7	 S.S.	PUSHED		$Q_u \approx 400$ P.S.F. NAT. M.C. = 78.5% Wetter than Liquid Limit.
As above	Grey	Very soft	27' 3" 150.7	8	 S.S.	1 1/2		
Silty very fine to fine sand, grits and pebbles.	Grey	Very dense	30' 0"	9	 S.S.	103		Moist.
			34' 1" 143.8					CHOPPED WITH BX CROSS CHOPPING BIT

REFUSAL PROBABLY BEDROCK

BOREHOLE LOG

Checked By E. M. Peto

W. T. GROUND WATER TABLE IN SOIL

[illegible]

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

Job Name Hwy. 401 - C.N.R. crossing

Job No. 57149

Borehole No. 10

Client: Dept. of Highways of Ontario

Casing BX (2-1/2" diam.)

Boring Date Jan. 29th, 1958.

Geodetic

Compiled By M. Mindess

Checked By E.M. Peto.

ABBREVIATIONS

 UNDISTURBED

S.S. 2" STANDARD SPLIT TUBE SAMPLE

V. T. IN SITU VANE SHEAR TEST

 FAIR

S. L. SPLIT BARREL WITH LINERS

Q/4 UNCONFINED COMPRESSIVE STRENGTH

☒ DISTURBED

S. T. THIN-WALLED SHELBY TUBE SAMPLE

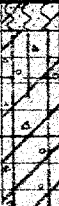
W.L. WATER LEVEL IN CASING

LOST

W.S. WASH SAMPLE

W. T. GROUND WATER TABLE IN SOIL

R. C. ROCK CORE

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Coring	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
ORGANIC SILTY LOAM.			0' 8" 178.2					
CLAYEY SILT, GRITS AND PEBBLES.	MIXED GREY AND BROWN	FIRM	5' 0"		1	S.S.	7	NAT. M.C. AT PLASTIC LIMIT WT. = 2' 10" JAN. 29, 1958.
SILTY CLAY, NUGGETTY TEXTURE GRITS AND PEBBLES.	MIXED GREY AND BROWN	FIRM	7' 8" 170.8		2	S.S.	7	QUITE MOIST NAT. M.C. = 34.3 $Q_u = 403 \text{ p.s.f.}$
SILTY VERY FINE SAND, NUMEROUS GRITS AND ROCK FRAGMENTS.	GREY	DENSE	8' 10" 168.4		3	S.S.	32/8'	MOIST
VIRTUAL REFUSAL. PROBABLY BEDROCK.								





BOREHOLE LOG



Checked By E. M. Peto.

SOIL DESCRIPTION	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Location	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
FINE SANDY SILT, GRITS.	MIXED BROWN	VERY LOOSE	0' 0" 186.3	1	X S.S.	3		SATURATED. W.L. = 3' 10" JAN. 27, 1958 HOLE DRIVEN DRY.
CLAYEY AND SILTY VERY FINE SAND. NUMEROUS GRITS AND PEBBLES.	BROWN	COMPACT	5' 0" 172.3	2	X S.S.	23		QUITE MOIST.
			8' 0" 172.3					BOULDERS OR BEDROCK.

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

Job Name Hwy. 401 - C.N.R. Crossing Job No. 57149 Borehole No. 12, 12A, 12B, 12C
Client Dept. of Highways of Ontario Casing BX (2-1/2" diam.) Boring Date Jan. 27th, 1958.
Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

SAMPLE CONDITION		SAMPLE TYPE		ABBREVIATIONS	
	UNDISTURBED	S. S.	2" STANDARD SPLIT TUBE SAMPLE	V. T.	IN SITU VANE SHEAR TEST
	FAIR	S. L.	SPLIT BARREL WITH LINERS	Q/u	UNCONFINED COMPRESSIVE STRENGTH
	DISTURBED	S. T.	THIN-WALLED SHELBY TUBE SAMPLE	W. L.	WATER LEVEL IN CASING
	LOST	W. S.	WASH SAMPLE	W. T.	GROUND WATER TABLE IN SOIL
		R. C.	ROCK CORE		

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
			B.H. 12	1				
			0' 0"					
			187.05					
VERY FINE SAND, GRITS AND PALE PEBBLES SEAMS OF MED. SAND	BROWN	DENSE			1 	S.S.	34	MOIST.
			5' 0"					HOLE DRY UPON COMPLETION
			182.0					
								VIRTUAL REFUSAL BOULDER OR BEDROCK.
			B.H. 12A	1' SOUTH OF E				
			0' 0"					
			187.1					
PROBABLY BROWN SAND WITH GRITS.			3' 3"					DROVE CASING TO 3'3" HIT SOMETHING VERY HARD.
			183.8					
								VIRTUAL REFUSAL POSSIBLY LARGE BOULDER.
			B.H. 12B	3' EAST OF B.H. 12, 1' SOUTH OF E				
			0' 0"					
			186.8					
PROBABLY SANDY SOIL WITH GRITS			3' 3"					DROVE CASING TO 3'3"
			183.5					
								VIRTUAL REFUSAL BOULDER OR BEDROCK.
			B.H. 12C	18' EAST OF B.H. 12, 2'6" SOUTH OF E				
			0' 0"					
			185.5					
PROBABLY SANDY SOIL, GRITS.			5' 0"					DROVE AND CLEANED EX CASING.
VERY FINE TO FINE SAND BROWN GRITS AND ROCK FRAGMENTS UP TO 1/4" SIZE.	BROWN	DENSE			2 	S.S.	38	MOIST.
			7' 8"					
			177.8					
								REFUSAL. PROBABLY BEDROCK.

APPENDIX I

Laboratory Test Data

e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

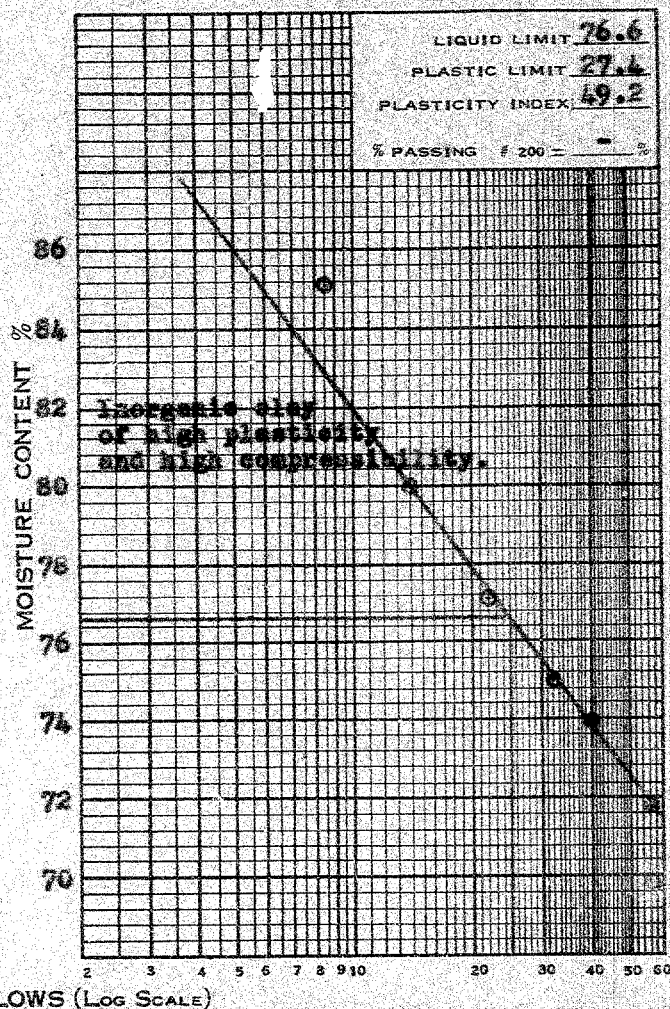
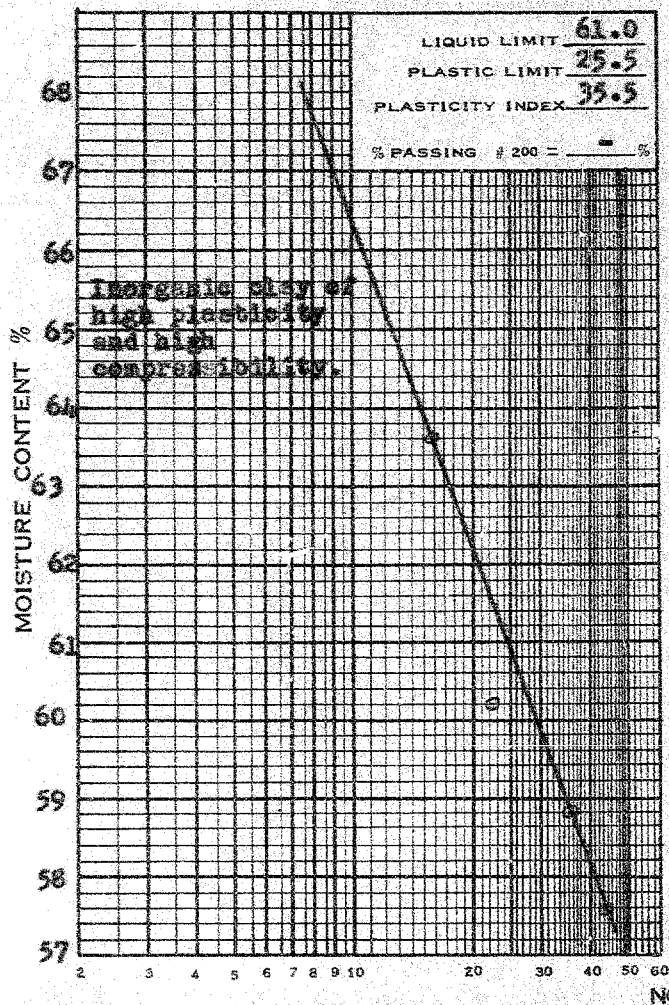
JOB No. 57149 PROJECT Rwy. 401 - C.N.R. Crossing.

SAMPLE FROM B.H. 2 Sample 4 and 6

SAMPLE FROM B.H. 4, Sample 2

DEPTH 9-1/2' - 16'

DEPTH 6 1/2' - 7'

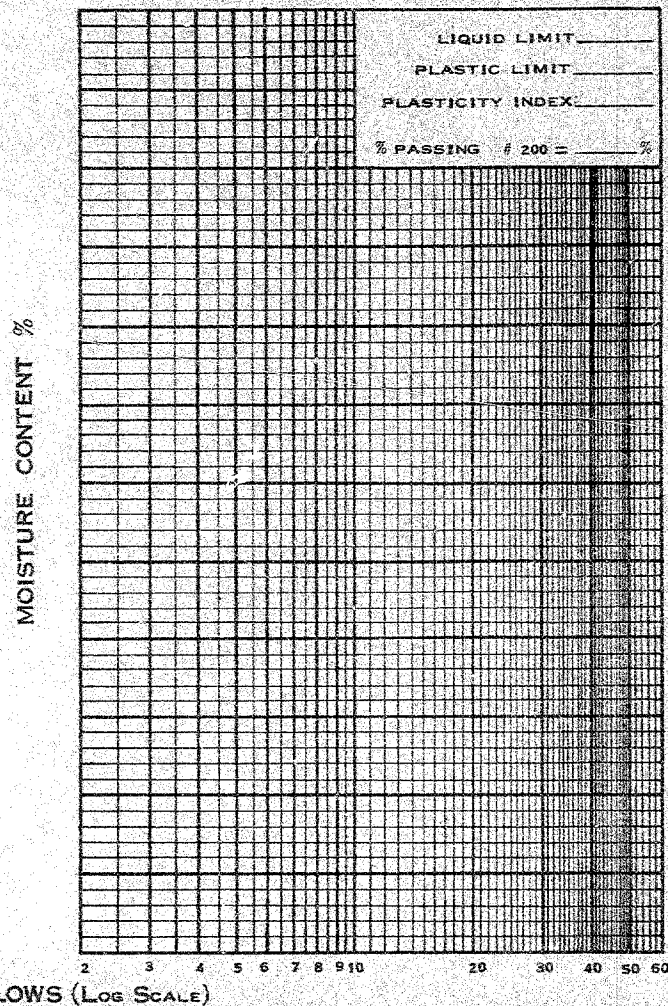
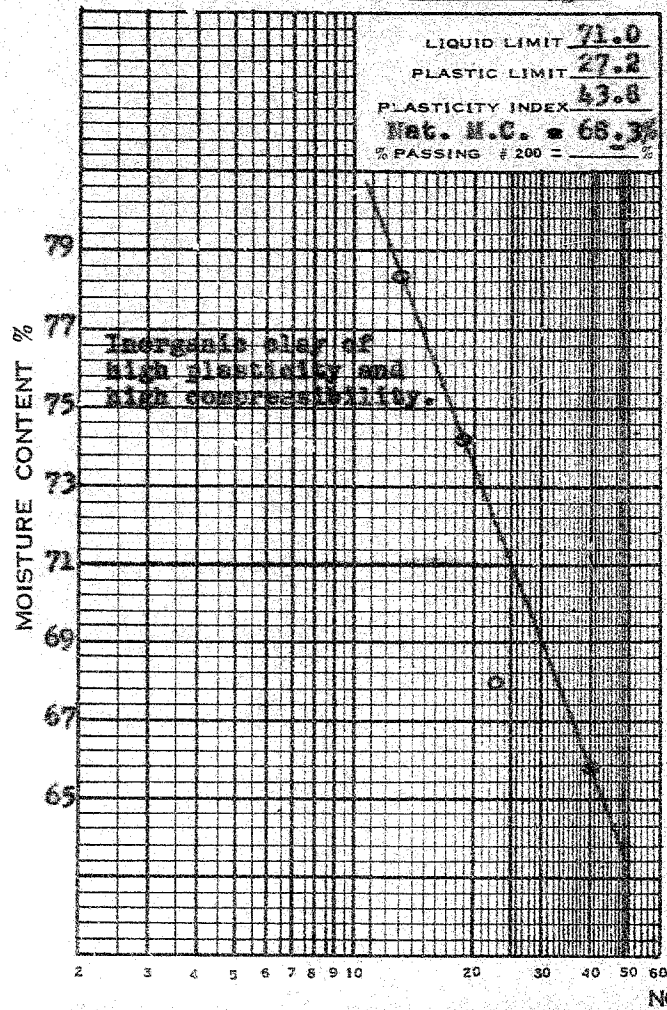


e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

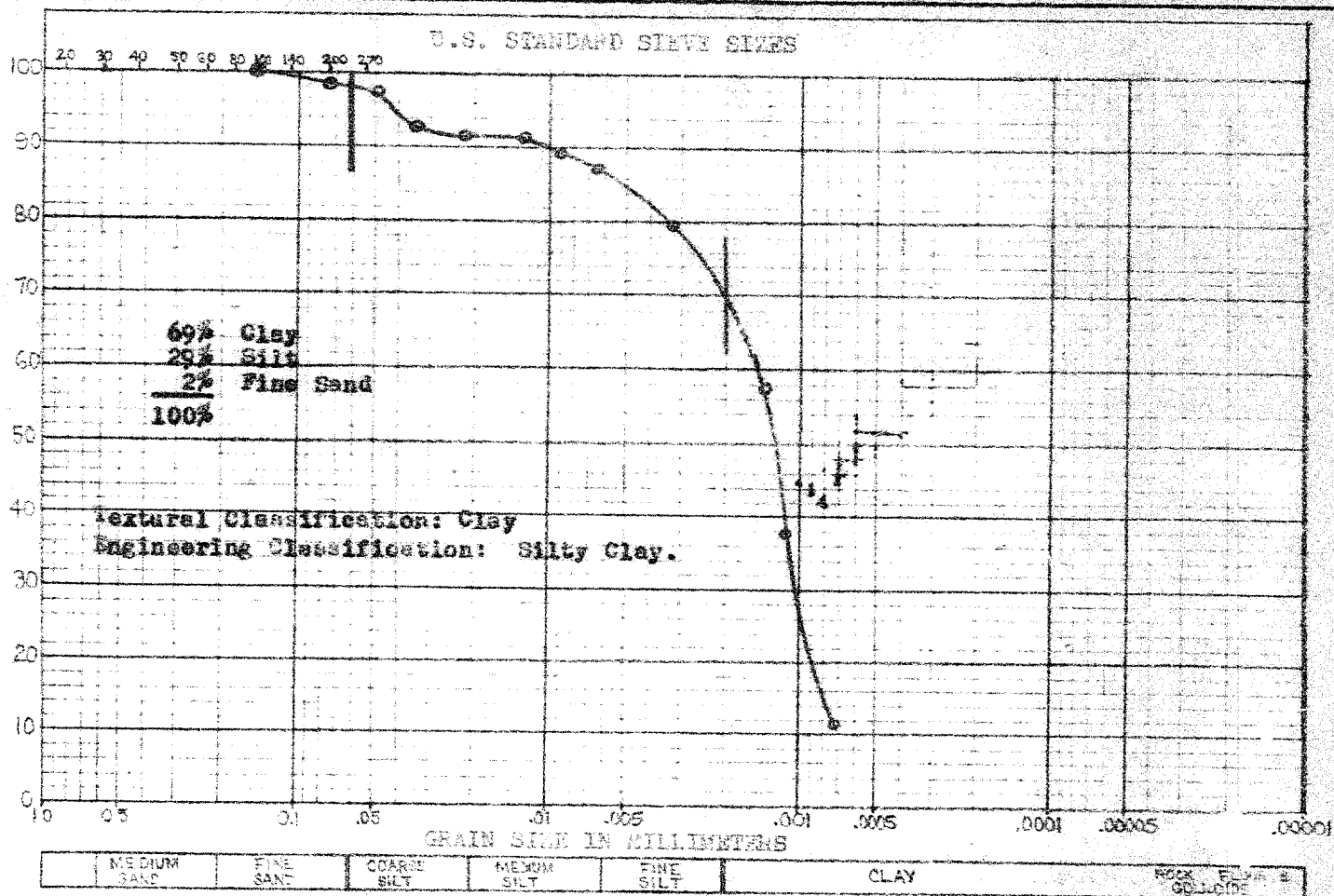
FLOW LINE CHARTS

JOB No. 57149 PROJECT Hwy. 401 - C.N.R. Crossing
SAMPLE FROM S.H. 4, Sample 5 SAMPLE FROM _____
DEPTH 12' - 13 1/2' DEPTH _____



E. M. PETO ASSOCIATES LTD.
HYDROMETER GRAIN SIZE DISTRIBUTION DIAGRAM

PER CENT FINER BY WEIGHT



M.I.T. CLASSIFICATION

Job Name Hwy. 401 - C.N.R. Crossing Job No. 57149 Borehole No. 5 Sample No. 7
Depth 17-1/2' - 18' Elevation 159.4 Remarks Specific Gravity of this sample = 2.73

57149

SUMMARY OF UNCONFINED COMPRESSION STRENGTH TESTS

<u>Sample from B.M.</u>	<u>Sample Number</u>	<u>Depth</u>	<u>Type of Failure</u>	<u>Q/u p.s.f.</u>	<u>Nat. E.C.</u>	<u>Remarks</u>
1	1	2½'-3'	Sudden shear at low unit strain	1440		
1	2	5½'-6'	Splitting at end of cylindrical sample	1559		Slightly wetter than Plastic 1
1	3A	8'-8½"	Gradual shear along a number of well-defined planes @ 46° to horizontal.	805		
1	3C	9'-9½'	As above, but planes @ 54½° to horizontal.	1035	49.4%	
2	2	5½'-6'	Gradual shear and local crippling at ends.	1252		
2	5B	13½'-14'	Gradual shear along well-defined planes.	181	62.0%	
2	5C	14'-14½'	Gradual shear @ Plane of 55½° to horizontal.	435	69.9%	
3	1	2½'-3'	Sudden shear @ low unit strain.	965	48.7%	
3	3A	8½'-9'	Definite shear failure at one end only @ 46° to horizontal.	345	55.3%	
3	3B	9'-9½'	Shear failure at one end only.	633	54.5%	
3	3A & 3B	8½'-9½'	Gradual, very plastic deformation little bulging.	140	59.5%	Remould. Sensiti = 3.49 (approx)
4	2	6½'-7'	Rapid shear failure @ angle of 56° to horizontal	1166	56.3%	
4	2	6½'-7'	Gradual plastic bulging.	608	53.1%	Remould. Sensiti = 1.92
4	3	8'-9'	Gradual shear	648	50.9%	
4	4A	10'-10½'	Rapid shear @ low unit strain.	592	67.0%	
4	4B	10½'-11'	Plastic, then shear	328		

<u>Sample from B.R.</u>	<u>Sample Number</u>	<u>Depth</u>	<u>Type of Failure</u>	<u>Q/u p.s.f.</u>	<u>Est. M.C.</u>	<u>Remarks</u>
4	4B	10½'-11'	Plastic.	145	65.5%	Very difficult to handle when removed. Remould. Sensitivity = 2.26.
4	4C	11'-11½'	Rapid shear @ low unit strain @ angle of 47° to horizontal.	1448	65.6%	Texture more nuggety than samples above.
5	2	6'-6½'	Very sudden, brittle.	1120	45.2%	
5	4A	10'-10½'	Gradual shear @ 53½° from horizontal.	550	71.0%	
5	4B	10½'-11'	Rapid shear @ low unit strain.	805	72.4%	
5	7	17'-17½'	Sudden shear @ plane of 53° to horizontal.	532	70% (approx.)	Better than Plastic Limit.
6	5	13½'-14'	Bulging and gradual shear.	382	61.3%	
7	4A	10'-10½'	Gradual shear @ 58° from horizontal.	559	54.1%	Very nuggety, but better than Plastic Limit when remoulded.
8	3B	8½'-9'	Sudden shear and crumbling at low unit strain.	1299	54.5%	Very nuggety, but better than Plastic Limit when remoulded.
8	3C	9'-9½'	Sudden shear and crumbling at low unit strain.	608	54.4%	As above.
10	2	5½'-6'	Sudden shear.	403	34.3%	

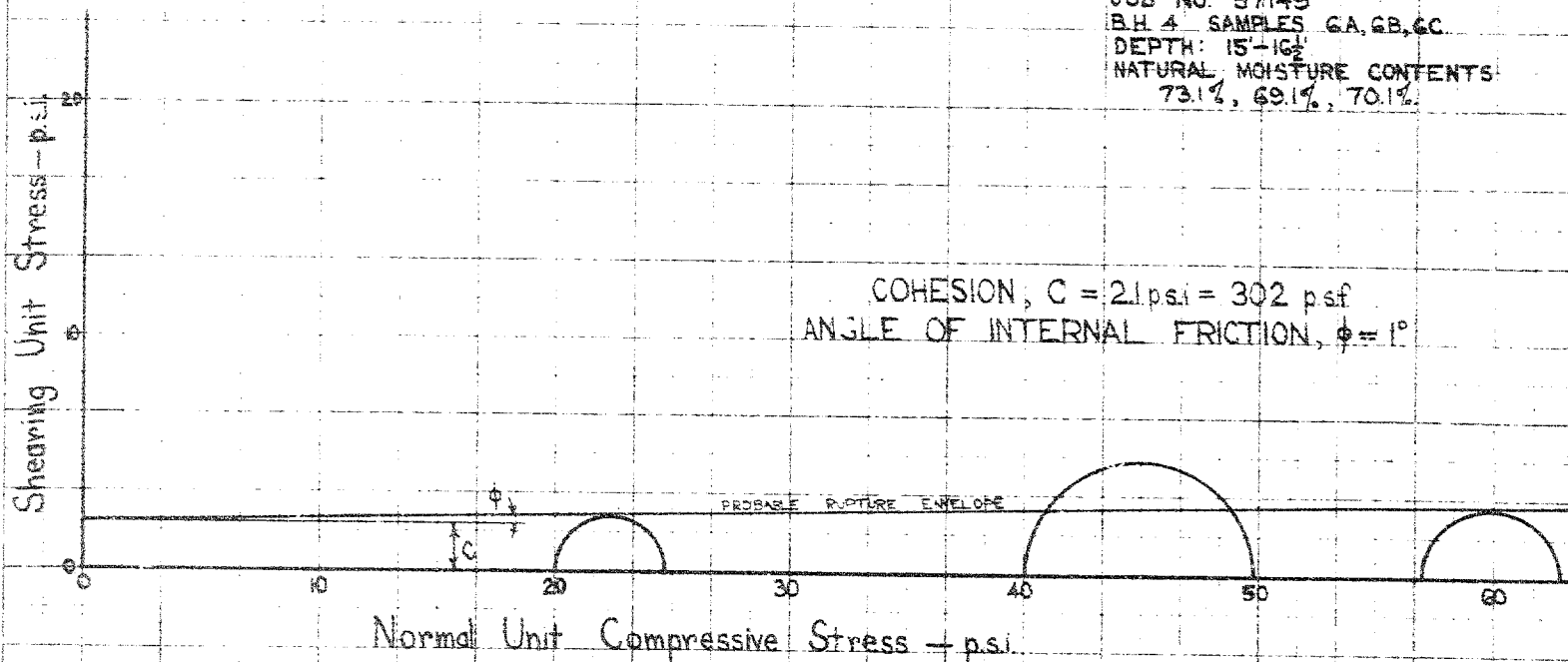
MOHR'S CIRCLE DIAGRAM

TRIAXIAL COMPRESSION TEST

ON VERY SOFT, GREY, SILTY CLAY

JOB NO. 57149
B.H. 4 SAMPLES GA, GB, GC
DEPTH: 15'-16"
NATURAL MOISTURE CONTENTS:
73.1%, 69.1%, 70.1%

COHESION, $C = 2.1 \text{ psi} = 302 \text{ psf}$
ANGLE OF INTERNAL FRICTION, $\phi = 1^\circ$



e. m. peto associates ltd.

YOUR REFERENCE:- **W. P. 00 - 57**

OUR REFERENCE:- **57149A**

850 Roselawn Avenue,

TORONTO, ONTARIO.

RUssell 1 - 4955.

8th August, 1958.

**Bridge Office,
The Department of Highways of Ontario,
280 Davenport Road,
TORONTO,
Ontario.**

Dear Sirs,

**Supplementary Report on Original Proposed Highway 401
C.N.R. Crossing - Township of Charltonberg.**

We refer to the original report on this site submitted under our reference 57149 and to the Department's request for a more detailed study of the conditions and soil types arising at this site.

We have pleasure in submitting herewith in the form of a supplementary report four (4) copies of our further studies of the soil conditions at this site.

The additional testing to obtain the information required to complete the work has been somewhat protracted, and in view of the detail called for, the report has tended to be complex and lengthy. The studies involved in making this report have given due consideration to the work done on similar soils by other investigators. Here for your convenience we are summarizing our conclusions and recommendations:-

1. The rates of strain applied during the laboratory strength tests on the Cornwall marine clay should be considerably slower than the customary accepted rates in order to obtain consistent and logical results. Accordingly, we are of the opinion that the field vane tests should be performed at similar rates of strain in order to provide a common basis for relating the results.

The field vane tests have given results very much higher, of the order of 300 to 500% greater, than laboratory unconfined compression and triaxial tests; furthermore they could not be related to the laboratory test results. In view of different rates of strain applied during the field tests we believe this has had a considerable bearing on the discrepancy between the results.

2. The clay displays a sensitivity ranging from 3.0 to 5.0 and generally becomes more sensitive with depth.

It has been subject to preconsolidation stresses in the past and has an overconsolidation ratio of approximately 4. However, the shear strength is low, due to a number of factors such as relief of previous stresses, followed by fissuring of the clay and possibly due to the leaching out of pore salt water.

3. We consider that laboratory shear strength tests, either unconfined compression or triaxial give reliable values which can be safely applied for design purposes, subject to these tests being performed on the lines referred to in paragraph 1.

4. We would prefer to apply a safety factor of three to the design strength values for the Cornwall marine clay; however our results do not indicate that this is either practical or economical and accordingly it appears that this must be reduced to 1.5 which in turn postulates the acceptance of some continuing creep in the clay.

5. On the basis of results obtained from slow drained triaxial tests and quick triaxial tests it is possible to arrive at a relationship between critical pore water pressure at failure and applied load.

Turning now to the application of these conclusions to the practical aspect of highway construction over Cornwall marine clay, we recommend:-

1. Highway embankments constructed by normal methods should be restricted to a maximum height of 34 feet with side slopes not steeper than 4 horizontal on 1 vertical.

2. Even with staged loading we do not recommend the total height of embankment should exceed 32 feet; furthermore the soil stability conditions must be satisfied for each stage where this method of construction is adopted.

3. 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 between 50% and 60% for each successive lift. This latter form of construction will involve some difficulty in installing piezometers in the field in order to give reliable results, having in mind the sensitivity of the clay. Accordingly we recommend the installation of a number of settlement test plates at suitable cross sections in order that information on settlement and creep could be collected.

4. Consideration of the use of sand drains on this site has led us to the conclusion that their installation here would be of doubtful benefit in view of the sensitivity and low permeability of the material and the relatively thin layer of the clay. Doubtless where the depth of the clay is considerably greater than on this site they could be of some benefit and could lead to some economy of time.

5. It is evident that the removal of the clay and replacement with an approved material will provide an alternative solution. However, the extent and application of this form of construction will depend on economic factors.

We trust that this report will provide the Department with some useful and practical information. Should there be any points arising from this report requiring further clarification, we shall be most pleased to discuss the matter with you.

Yours very truly,

E. M. PETO ASSOCIATES LTD.



E. M. Peto, F. Eng.

MMH:pl

DEPARTMENT OF HIGHWAYS OF ONTARIO

ABANDONED LINE "G" CROSSING
HIGHWAY 401 - C.N.R. TRACKS
CORNWALL, ONTARIO.

SUPPLEMENTARY SOILS REPORT

by

e. m. peto associates ltd.

Toronto, Ontario.

August, 1958.

SUPERIMPOSED DOCUMENT MAY
APPEAR AS MULTI-FEED ON FILM.

TABLE OF CONTENTS

		<u>Page</u>
1.	Introduction and General Background	1
2.	Procedure	2
3.	Vane Test Procedure	3
4.	Soil Conditions	4
5.	Observations.	
	Vane Tests	5
	Rate of Loading	6
	Sensitivity	6
	Laboratory Shear Strength Tests	7
	Consolidation Characteristics	11
6.	Engineering Considerations	11
7.	Conclusions	16

Borehole Logs.

Appendix I - Laboratory Test Results.

Site Plan.

Job No. 57149A

Client's Ref. No.

Date 5th August, 1958.

Supplementary Report on

PROPOSED ORIGINAL HIGHWAY 401 - C. N. R. CROSSING

W. P. 55 - 57 TOWNSHIP OF CHARLOTTENBURG

for

DEPARTMENT OF HIGHWAYS OF ONTARIO.

INTRODUCTION

Subsequent to the submission of our original soils report for the above mentioned project, certain facts came to light which necessitated additional field work. A careful review of our recommendations in the original report was also to be made, to provide the Department with additional technological background on the relatively sensitive marine clays in the Corawall area.

1. Further field investigation of the possibilities of shifting the highway alignment to a more favourable site was required. The drumlinoid topography, combined with depressions containing very poor sedimentary marine soils, indicated that much better soil conditions existed no more than 600 feet from the original line "G". A new line for the highway some 550 feet North of line "G" has been investigated, and our report covering this work has already been issued under separate cover as Job No. 57149B. We believe that the revised line to the North of line "G" has been finally adopted.

2. When placing high fills on marine clay soils, initial failure of the approach embankments with subsequent rebuilding to grade is to be avoided because of the danger that embankment failures will be progressive. Construction procedures to try and eliminate embankment failures entirely must be followed.

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BE REPRODUCED ON FILM.

3. The berm construction has been used quite successfully in the south-eastern part of Ontario adjacent to the St. Lawrence River, and the Department would not be averse to using such a method of construction in the Cornwall area.

4. The time of construction is not a critical factor, within reasonable limits.

5. There is some reason to believe the strength values for the Cornwall marine clays, which although not hyper-sensitive like the adjoining Leda marine clays, when derived from unconfined compression and quick triaxial tests, are conservative. Accordingly, vane shear tests to determine the "in situ" soil shear strength and sensitivity were to be performed adjacent to boreholes for which the laboratory shear strengths had been previously determined. Slow triaxial tests with measurement of pore pressure were also to be attempted.

PROCEDURE

The equipment was trucked from Toronto to Cornwall on March 31st, 1958. The only access road to the site, a farm trail, was very muddy and snow covered, and considerable difficulty was experienced in moving the equipment to the site. This was finally accomplished with the aid of a large bulldozer. The actual soil sampling work commenced on April 2nd, 1958, under the direction of our Field Engineer, and was halted on April 5th, 1958, after three holes had been completed. Work on the potential new site 300 feet to the North was started on April 7th, 1958. On May 2nd, 1958, a fourth test hole was driven to obtain more vane test data at the original site centred on line "G". Three of these holes, which were primarily for "in situ" vane shear testing, were driven within 6 feet of test holes from our original soil investigations, so that a comparison of laboratory strength tests and "in situ" vane tests could be made. In addition, one test hole was driven a short distance south of the original site to see if soil conditions improved in this direction.

A number of 3" diameter undisturbed samples were obtained at borehole 5A so that consolidation tests could be carried out to determine the compression index and amount of preconsolidation, if any, of the marine clay stratum.

Our original site plan was revised to show the additional soil test hole locations.

PROCEDURE (contd.)

Complete borehole logs, graphical presentation of data, and supporting laboratory data are included.

VANE TEST PROCEDURE

The vane tests performed on this site were done with our own equipment, a modification and improvement upon the Acker vane test kit. Depending upon the relative soil stiffness, either one of two different vanes is employed.

Our nominal 2" diameter vane has the constants:
vane diameter $D = 1.86"$, vane length $H = 2.46"$, $\frac{H}{D} = 1.57$, vane area ratio $= 15.55\%$

Our nominal 3-1/2" diameter vane has the constants:
vane diameter $D = 3.31"$, vane length $H = 5.13"$, $\frac{H}{D} = 1.55$, vane area ratio $= 7.70\%$

Investigations of vane testing equipment by Cedling and Odensestad have indicated that the ratio of vane length to diameter $\frac{H}{D}$ should be two. The $\frac{H}{D}$ ratios for our vanes, particularly the small vane, are close enough to two to ensure the accuracy of the results is not jeopardized.

The vane area ratio is defined as the ratio of the average cross-sectional area of the vane to the cross-sectional area of the cylinder sheared. Sample disturbance is directly related to the area ratio of the sampler; the lower the area ratio the lower the degree of sample disturbance. The area ratios of our two vanes are equal to or less than the accepted allowable area ratio values.

In our vane tests we have deviated from standard procedure by never doing more than three successive tests (covering a vertical distance of only 2 feet) with any one set-up of the equipment. In this way the A-drill rod to which the vane is attached never extends more than a few inches below the tip of the casing, and rod friction is not a factor in our vane tests. Extensive data which we have collected with our equipment on different sites indicates that rod friction is not nearly so minor a factor as many investigators believe.

VANE TEST PROCEDURE (contd.)

Another factor to be considered is that the clay often sticks to the point of fracture of the vanes and the central shaft, thereby greatly increasing the area ratio.

Another feature of our vane tests on this particular site is that the remoulded tests were all done exactly ten minutes after remoulding. This is at variance with the procedures followed by National Research Council investigators on Leda marine clays. Their tests have been conducted at an interval of exactly one minute after remoulding, since they have found that the Leda clays regain strength rapidly with time.

SOIL CONDITIONS

The soil conditions encountered during the course of this additional investigation are of course identical to those found previously. A description of the soil properties has been given in detail in our original report, and accordingly is not repeated here.

The basic soil strata at this site are:- A layer up to 8 feet thick of grey-brown silty clay, a layer up to 16 feet thick of grey, soft silty clay, dense sandy till. The till is underlain by limestone bedrock.

However, it should be noted that at borehole # 13, 75 feet South along the railway tracks from borehole # 3, the thickness of the silty clay layer decreases, while that of the compact to dense glacial till increases. The top of bedrock profile also was found to dip Southward.

OBSERVATIONS

The soil stratum with which we were most concerned during our additional studies of this site was the soft grey-brown to grey silty clay, since this was the stratum in which any shear failure, settlement, creep or other related effects would occur. We have been able to make the following observations:-

OBSERVATIONS (contd.)1. Vane Tests

The "in situ" vane tests on this particular marine clay, having been performed with all due care, appear to give results which are consistently three to five times higher than the laboratory unconfined compression and quick triaxial tests. These results are basically in agreement with data published by the National Research Council on vane tests at Beauharnois on a similar, although thicker, marine clay deposit with almost identical moisture contents and Atterberg limits. However, the discrepancy between the laboratory tests and the vane tests determined by the National Research Council was not quite as large as that indicated by our vane tests.

Part of the difference between the laboratory tests and the field vane tests is attributed to the disturbance of the laboratory samples, which tends to give results on the low side. Unfortunately, the structure of every test sample is partly altered as a result of the stress changes that occur when the sample is removed from the ground, and the distortions that are caused by the sampling operation and subsequent laboratory handling. In Leda clay and similar marine clays, the most serious disturbance arises from the relief of stresses in the clay when the soil is removed from its natural environment.

When a torque is applied to the vane test apparatus, this mobilizes resistance in the soil made up of two components, which are cohesion and internal friction. It is common practice, when interpreting vane test results, to assume that the internal friction is equal to zero. In the particular case under investigation, based on three sets of quick untrained triaxial tests at a rate of strain of 0.002 inches per minute made on a total of nine test specimens, a definite ϕ value was indicated. The shearing resistance due to cohesion determined using this ϕ value in the interpretation of the vane test results, would be more comparable to the cohesion values obtained from the unconfined compression tests. It is our considered opinion that the soil shear strength value which should be used in all engineering analyses on the Cornwall marine clay lies somewhere between the field vane test results and the laboratory unconfined compression test results.

OBSERVATIONS (contd.)2. Rate of Loading

A factor which has considerable effect on the ultimate shear strength of this clay is the applied rate of strain during testing. Our experience has shown that rates of strain lower than those conventionally used (0.01 - 0.05 inches per minute) must be applied for proper results. For a good comparison of field vane and laboratory results, the unit strain in the field and unit strain in the laboratory should be equal. Unfortunately this was not always the case in our tests, since a number of various procedures had to be tried until the best one could be selected.

The vane tests were performed at a higher rate of strain than all of the laboratory tests. This is one of the major reasons for the discrepancy between the field and laboratory tests.

It has been determined during the course of our experiments on the Cornwall clay, that a rate of strain of 0.05 inches per minute, which gives 30% strain of a 5 inches sample in ten minutes, is too quick a rate of loading, and gives strength values which are on the low side. The best explanation for this is that this rate of strain disturbs the sensitive clay and does not permit any thixotropy or regain in strength to take place. On the other hand, tests at 0.005 inches per minute apparently did permit some regain in strength, and somewhat higher shear strength values were obtained.

3. Sensitivity

The true sensitivity, i. e. ratio of undisturbed to remoulded shear strength, is a most difficult quantity to assess, different methods giving different results. It is dependent, among other things, upon the pore water salt concentration and the liquidity index. The published National Research Council paper makes the following statements on the subject:

"The authors do not wish to discredit the value of the remoulded field vane tests but wish to suggest that the field sensitivity is no more than an indication of the sensitivity of the clay."

OBSERVATIONS (contd.)

"The field sensitivity cannot be related to laboratory sensitivity determinations with any confidence. A better measure of sensitivity appears to be in the relation between sensitivity and the liquidity index."

However, the correlation of sensitivity and liquidity index which is given in the same National Research Council publication, is not a particularly good one, having a wide scatter of points.

Actually our field vane test sensitivities ranged from 1.34 to 10.60, with 4.0 being a good average value. When we computed the liquidity indices, using data from the Atterberg limit tests we had done, and applied them to Bjerrum's correlation chart of sensitivity to liquidity index, we found the sensitivities ranging from 3.8 to 7.0 to 9.0. These are higher than values obtained from laboratory remoulded tests.

The conclusion to be drawn from this is that the Cornwall marine clay is a sensitive one, and can be taken to have sensitivity values increasing with depth from 3.0 to 8.0 for all practical purposes.

4. Laboratory Shear Strengths based on Unconfined Tests

The shear strength of the soil profile has been determined in the laboratory by quick unconfined compression tests, and "in situ" by the vane shear test. Plotting the test results on a Shear Strength vs Depth Chart, which appears in Appendix I under the title "Average Shear Profile Determination", a definite grouping of the shear values has been found.

From this graphical presentation the following conclusion can be drawn. The shearing strength of the soft silty clay is not uniform through the profile, but decreases with depth. In the upper five feet of the material three unconfined compression tests were made and the cohesion of the material was found to be between 450 and 750 p.s.f.

OBSERVATIONS (contd.)

Between the five and ten foot depths the cohesion values determined by the unconfined compression test converged fairly well around the numerical average of 415 p.s.f. On the other hand, cohesion values based on "in situ" vane shear tests were scattered over such a wide range that no attempt was made to find a representative average value.

Below the depth of ten feet, and down to the layer of dense glacial till, the average value of cohesion from the unconfined compression test was found to be 225 p.s.f., and from the vane shear test 1,000 p.s.f.

In evaluating design values based on field and laboratory test results, due consideration should also be given to the experience gained from other investigations conducted on similar materials which possess a common geological history and have similar drainage conditions.

A comparable major construction site is the Grass River Lock on the St. Lawrence Seaway, which is located several miles south-west of the site of the present investigation. The critical soil condition of this site was extensively investigated by the U.S. Army Corps of Engineers, and the results of their investigations were reported by Mr. H. E. Burke, Chief, Foundation and Materials Branch, Buffalo District, in the proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London 1957. Vol. II, page 301. At the proposed lock site a design shear value of 550 p.s.f. was found for the upper part of the post-glacial marine silty clay (ranging in colour from brown to grey, blue-grey), based on 427 triaxial compression tests. For the bottom ten feet of the same material a design value of 400 p.s.f. was found.

In order to check the value obtained from the triaxial tests a trench excavation was made to a depth greater than the critical one; consequently four major slides occurred. Computing back the cohesion values at the critical condition for two slides, a value of $C = 300$ p.s.f. was obtained, and for the other two 400 p.s.f. was found.

Analyzing the test result obtained in our laboratory in this perspective we feel as it was pointed out in our report of February 15th, 1958, that our shear values obtained from the unconfined compression test are on the conservative side and the "in situ" strength of the material is actually greater than that reflected by the laboratory tests.

OBSERVATIONS (contd.)

Since the "in situ" vane test strengths are from 300% to 500% higher than the shear strength "C" obtained from unconfined compression tests, we would be reluctant to recommend vane test shear values for design purposes.

Based on these considerations we would recommend using the design shear value of 300 p.s.f. between the depth of ten feet below present ground surface and the underlying till. This value is the minimum obtained by the Corps of Engineers from their excavation procedure, and compares favourably with the range of results from our laboratory investigation.

Increasing the shear value at the depth of five to ten feet in proportion to the above, the proposed shear value for this layer will be 500 p.s.f.

For temporary condition the minimum safety factor recommended for the Cornwall marine clay is 1.5 and for permanent condition 3.0.

5. Triaxial Tests

As part of the extensive investigation on Cornwall marine clay, to discover facts that would be of aid when constructing Highway 401 through this area, a number of triaxial tests with pore water pressure measurements, and slow drained triaxial tests were performed in our laboratory.

These tests have given the following results:-

1. At the site investigated, adjacent to the revised Highway 401 - C.N.R. Crossing, the clay may be divided into two distinct layers: An upper stratum up to ten feet thick of grey-brown, very fissured, partly desiccated or leached silty clay. A stratum up to fourteen or fifteen feet thick of grey, soft, saturated silty clay.

OBSERVATIONS (contd.)

2. The quick undrained triaxial tests gave values of cohesion which are in practical conformity with the range of values from the unconfined tests, but slightly higher. The small difference is partly due to the different rates of loading between the two types of tests.

Below is a comparison of the cohesion values obtained by the different tests:-

	<u>Quick Unconfined</u>	<u>Quick Triaxial</u>	<u>Slow Triaxial</u>	<u>Recommended Design Value</u>
Partly desiccated, fissured, grey- brown clay.	415	484	582	550 p.s.f.
Saturated, slightly fissured, grey clay.	225	256	-	300 p.s.f.

3. A definite ϕ value of approximately 16° was indicated for the grey-brown clay, which can be explained by its fissured texture, anisotropy, and not quite fully saturated condition.

4. A very small ϕ value of approximately $2\frac{1}{2}^\circ$ was obtained for the grey soft clay, but this should be neglected for all practical purposes.

5. The fully drained triaxial test on the grey-brown clay gave an effective stress cohesion value which is roughly equal to our recommended design value based on the unconfined tests. In addition, a high effective angle of internal friction was obtained, showing that the intergranular pressures had been fully developed by allowing all excess pore pressures to dissipate.

For over-consolidated clays the effective stress parameters C' and ϕ' should be higher than C and ϕ from conventional quick tests, and this is exactly what has occurred in our tests.

The fully drained test enables the Engineer to quantitatively evaluate the increase of soil shearing strength as a result of soil consolidation (drainage of excess pore water).

OBSERVATIONS (contd.)

6. Consolidation Characteristics

Two consolidation tests on samples from different depths in the same hole showed a number of interesting results:-

- (a) Despite the apparent sensitivity of the clays, the specimens tested showed but little disturbance due to handling and sampling.
- (b) The marine silty clay has been subject to preconsolidation stresses in the order of 3,600 to 4,830 lbs. per sq. ft. These values are in complete agreement with most probable preconsolidation stress values arrived at by National Research Council investigators for a similar marine clay. The over-consolidation ratio of this clay is approximately 4.0.
- (c) The compressive index (slope of the virgin compression branch) is extremely high, in the order of 1.0.
- (d) Although the consolidation tests show that settlement of the Cornwall clay in the normal sense is not a problem within the limits of safe design loadings, 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

There are two conditions to satisfy in embankment design:-

- (a) The mass stability of the embankment in respect of the underlying strata should be such that there is a sufficient safety factor against toe or base failure.
- (b) The maximum allowable settlement (which is made up of both plastic deformation and consolidation) during the life-time of the structure should be limited to a value which would not be detrimental to the road surface.

ENGINEERING CONSIDERATIONS (contd.)

In order to determine the maximum height of the embankment and the necessary side slopes, the strength characteristics of the underlying soil were measured in the laboratory.

The strength of a soil element is a function of the cohesion between the individual soil particles, and the internal stability, which is measured by the angle of the plane of failure (ϕ) and the angle of internal friction (ϕ) in the soil mass.

For the determination of these physical properties, unconfined compression test, quick undrained and slow drained triaxial compression tests were performed on representative samples from the two clay layers to simulate different field conditions. The average design values deduced from these tests are given below in tabular form:-

	Depth ft.	Ultimate Shearing Strength due to cohesion	Angle of Internal Friction	
			quick undrained	slow drained
Partly desiccated, fissured grey brown clay.	0-10	550 p.s.f.	16° 0'	26° 20'
Saturated, slightly fissured grey clay.	10-24	300 p.s.f.	2° 40' use 0° 0'	assumed 5° 0'

Design values obtained from unconfined compression tests and quick undrained triaxial tests give a measure of the stability of the embankment under general construction procedure. In this case the individual soil particles are separated by the pore water, drawn to the particles by molecular attraction.

The values obtained from the slow, drained triaxial compression tests represent the condition when the applied load on the saturated clay is of such a magnitude that it partially overcomes the forces of molecular attraction and squeezes out some of the pore water, reducing the pore water pressure to zero and producing a more intimate contact between the particles; as a consequence, the ultimate shearing strength of the soil is increased, due to the greater internal friction.

ENGINEERING CONSIDERATIONS (contd.)

It should be realised that the soil condition is critical at the site under investigation. Accordingly, in order to arrive at a reasonable and economical design it will be necessary to take a calculated risk and to accept a lower factor of safety than is generally recommended for earth structures of this type.

It is the practice to use a safety factor of 3 for permanent structures and 1.5 for temporary construction.

In the stability analyses discussed in a following part of this report, three different sets of conditions were investigated, and a safety factor of 1.5 was used, but the following clay characteristic should be considered when applying this value:-

If the shearing stress acting on a sample of undisturbed clay is less than a value known as the creep strength or static yield value, the clay deforms almost instantaneously upon application of the shearing stress, and thereafter experiences no progressive deformation. On the other hand, when the creep strength is exceeded, the clay deforms continuously under constant shearing stress. When this shearing stress is increased, the rate of creep increases also. Stresses in excess of the creep strength or static yield value have been an important cause of progressive lateral movement of some structures.

It should be borne in mind that for many clays the static yield value is approximately one quarter of the unconfined compressive strength. The value obtained by applying a safety factor of 1.5 to unconfined compressive strength is probably higher than the static yield value, and therefore creep of the underlying soil with such a safety factor is almost certain to occur.

Stability analyses, with the results reported graphically in the form of required side slope for any height of embankment, were made using the following three different assumptions:-

- (a) The angle of internal friction for both layers is zero. This is the conventional analysis applied to saturated or near-saturated clays.

ENGINEERING CONSIDERATIONS (contd.)

- (b) For the upper clay layer the angle of internal friction is 16° , based on the quick triaxial tests, and for the lower soft layer, it is for all practical purposes zero.
- (c) Both layers are consolidated. The angle of internal friction for the upper layer is 26° $20'$ based on slow, drained triaxial compression test and for the lower layer an assumed conservative ϕ value of 5° will be developed. This is the ideal case, when stage loading with full consolidation between the different lifts, is carried out.

If stage constructive of the embankment is used in the field, a condition somewhere intermediate between (b) and (c) will actually be obtained.

The element method of stability analysis was used, and the slope of the fill for various heights and for the constant shoulder to shoulder width of 110 feet was determined within the limitations imposed by the selected safety factor of 1.5. Only the worst case was considered, when the fill is underlain by 10 feet of grey-brown clay and 14 feet of grey soft clay. The condition is obviously improved where the clay stratum is thinner. Further it was assumed that the fill will be constructed of selected granular material, fairly well graded and compacted to 95% of the maximum density.

It is clear from the accompanying "Embankment Stability" curves that if the conventional $\phi = 0$ assumption is made for both clay layers, construction of a fill higher than 15 feet is not an economical proposition. At the points where the test holes were driven, there is a definite indication that the upper stratum of partly desiccated clay possesses some internal stability which is reflected in the internal friction value obtained from the quick triaxial tests.

From practical experience the value of $\phi = 16^\circ$ is higher than expected and this value should be substantiated by further laboratory evidence before it is used for extensive areas outside the limit of our field investigation. Our investigation at the Highway 401 - C.P.R. crossing site proved that the partly desiccated grey-brown clay does not even exist in some areas.

ENGINEERING CONSIDERATIONS (contd.)

Accepting the value $\phi = 10^\circ$, the stability analyses show that the maximum practical embankment height is 24 feet when slopes of 4 horizontal on 1 vertical are used.

The slow, drained triaxial tests show that the internal stability characteristics of the soil are improved considerably by full consolidation, as shown on the appropriate "Embankment Stability" curve for the fully drained case when $\phi_A = 20^\circ$ and $\phi_B = 5^\circ$.

Excessive slopes can be replaced by properly designed toe berms, where the combined length of the berm corrected slope should be equal to the slope as designed and the volume of the material shall remain the same.

There are two basic principles which must be obeyed in order to benefit from stage construction:-

First, the maximum load to be placed on the clay layer at each successive time, should be limited to the ultimate bearing capacity of the clay under the existing conditions when the load is applied.

Second, the time interval between each successive load application should be such that will allow some predetermined degree of consolidation of the layer and corresponding dissipation of the pore pressure.

To stress this point and to give factual information the relationship between the time required to consolidate the soft clay layer and the applied load is given in graphical form, based on our consolidation test results, and using the equation:

$$T_{\%}^2 = t_{\%}^2 \left(\frac{H}{h} \right)^2 \quad \text{where}$$

$T_{\%}^2$ time required to consolidate the soft clay layer to a predetermined percent of consolidation.

$t_{\%}^2$ time required to consolidate the laboratory sample to the predetermined percent of consolidation.

H thickness of the soft clay layer.

h thickness of the laboratory sample.

ENGINEERING CONSIDERATIONS (contd.)

It can be seen from the shape of the curves that to obtain 100% consolidation corresponding to the consolidated drained triaxial test the waiting time required is very large and totally impractical.

CONCLUSIONS

As a result of our extensive studies on Cornwall marine clays, we have been able to draw quite a number of interesting conclusions, which will be of use in dealing with Cornwall marine clays and in general highway construction in the Cornwall area.

A. With regard to the clay itself:-

1. Cornwall marine clay should be tested in the laboratory at rates of strain of 0.005 inches per minute or less, and any vane shear tests should be done at comparable low rates of strain. Field loading of the clay could rarely ever exceed the above rate of strain.
2. The clay is a sensitive one, with sensitivities ranging from 2.0 to 8.0, and generally becomes more sensitive with depth. Because of the sensitivity, failure and subsequent rebuilding should definitely not be permitted.
3. The clay has been subject to preconsolidation stresses in the past and has an overconsolidation ratio of approximately 4. Despite this the shear strength is very low, due in part to the relief of the previous stresses and subsequent fissuring of the clay, and in part to the leaching out of pore salt water. This latter effect has not yet been fully investigated, but some existing theories maintain that it has considerable bearing on both the sensitivity and shear strength of marine clays.
4. Laboratory shear strength tests, either unconfined or triaxial, give what we consider to be good values which can be applied directly in engineering design, providing that the rates of strain are as suggested and that there are sufficient test results.

CONCLUSIONS (contd.)

5. "In situ" vane shear strength tests can give reasonably reliable results, provided that the greatest care is used in the testing procedures, and that the torque is applied to the drill rods very slowly by mechanical means. Otherwise the results are erroneously high.

6. Although it is preferable to apply the conventional safety factor of 3 in designs on Cornwall marine clay, this may not be practical or economical, and it may be necessary to use a safety factor of only 1.5, and to expect some continuing creep as a consequence.

7. It is possible, on the basis of slow drained triaxial tests (pore water pressure = 0) performed in conjunction with quick triaxial tests on similar material, to arrive at a relationship between critical pore pressure at failure and applied load.

8. With regard to highway construction over Cornwall marine clay:-

1. With rapid build-up of the highway embankment under normal construction methods the maximum practical embankment height is 24 feet, and this involves side slopes of 4:1.

2. Staged loading permits considerable improvement in the subsoil condition, but even so highway embankments higher than 22 feet are not recommended. If stage construction is used, the soil stability conditions must be satisfied for each layer.

3. The maximum practical degree of consolidation which should be aimed at in stage construction is 50% to 60% consolidation under each successive lift. A higher degree of consolidation would require too much waiting time, and a lower degree would not have much beneficial effect.

4. It would be difficult in this case to install piezometers in the field so that the developed pore pressures could be checked. The piezometers would have to be located at the point of maximum subsoil stress, and it is likely that the surrounding sensitive soil would be disturbed during the installation, giving erroneous results.

CONCLUSIONS (contd.)

However, we recommend that a number of settlement test plates be installed at various cross-sections so that field settlement and creep data could be collected.

5. The use of vertical sand drains to stabilize the Cornwall marine clay does not appear to be practical or economical, because of the extent of the area requiring stabilization, the close spacing of the sand pipes required, difficulty of installation, and the very low permeability of the clay which means that considerable time would elapse before drainage of the clay could occur.

6. It is apparent that removal of the sensitive clay and replacement with approved material will provide an alternative solution. However the extent and application of such excavation and replacement will depend on economic factors.

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SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name HWY. 401 - C.N.R. Crossing Job No. 57149ABorehole No. 5AClient Dept. of Highways of Ontario Casing 4" pipeBoring Date April 3, 1958.Datum Geodetic Compiled By M. MindessChecked By C.F.F.

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
Organic silty loam	Black		0' 0" 177.1					
Silty clay, some sand.	Grey-brown	Soft			1	S.S.	4	NAT. M.C. = 53.6%. Much wetter than Plastic Limit.
Silty clay, some fine to coarse sandy clay.	Grey-brown	Soft	5' 0" 172.1		2	S.S.	2 1/2	NAT. M.C. = 33.8%.
Silty clay, Nuggetty texture.	Grey	Very soft.	6' 6" 170.6		3	S.S.	1 1/2	NAT. M.C. = 73.5%. Wetter than Liquid Limit.
As above.	Grey	Very soft.	10' 0"		4	S.L.	PUSHED	TRIAXIAL TEST ON SA 4B: C=216
					5	2" V.T.	"	NAT. M.C. = 67.7% CONSOLIDATION TEST ON SAMPLE 4C.
					6	2" V.T.	"	
					7	2" V.T.	"	
As above.	"	"	15' 0"		8	3" V.T.	PUSHED	POSSIBLY SOME STONES AT 15'
As above.	"	"			9	3" V.T.	"	
As above.	"	"			10	3" V.T.	"	
Silty clay, nuggetty, thin seams of fine sand.	Grey	Very soft.	20' 0" 156.1		11	S.L.	PUSHED	NAT. M.C. = 66.9%
					12	S.S.	PUSHED	NAT. M.C. = 35.4%, BUT SAMPLE INCLUDED SAND SEAM.
Probably sandy and silty clay, grits.		Firm	25' 0"		13	S.S.	13	
Sandy and silty clay, grits and rock fragments.	Grey	Loose.	27' 0" 150.1		14	S.S.	4	Saturated.
HOLE TERMINATED.								

Note: See accompanying laboratory data sheets for vane and consolidation test results.

BOREHOLE LOG

Checked By C. F. F.

ABBREVIATIONS

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
mic silty loam	Brownish-black		0' 0" 177.3					
Clay, very nuggetty texture.	Mixed Grey-brown	Firm	2' 0"		1	S.S.	5	SLIGHTLY WETTER THAN PLASTIC LIMIT NAT. M.C.=49%
above	"	"	5' 0"		2	VANE TEST ATTEMPTED, BUT SOIL TOO STIFF		
clay, nuggetty	Grey	Soft			3	2" V.T. PUSHED		
					4	2" V.T. "		
above	Grey	Very soft	10' 0"		5	3" V.T. PUSHED		
					6	3" V.T. "		
above	"	" "			7	3" V.T. PUSHED		
					8	3" V.T. "		
			15' 0"		9	3" V.T. "		
above	Grey	Very soft			10	3" V.T. PUSHED		
and silty clay, y grits and pebbles.	Grey	Firm	20' 0" 156.5		11	S.S.	5	SATURATED
fine sandy and yey silt, grits and bles.	Grey	Loose	23' 0" 153.3		12	S.S.	9	QUITE MOIST
HOLE TERMINATED								Note: See accompanying laboratory data sheets for vane test results.

C. III. PETU ASSOCIATES III.

Borehole No. 8A

Boring Date April 2, 1958.

Checked By C.F.F.

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

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

F415

DISTURBED

LOST

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
ic silty loam, many roots. Black-sandy and silty clay. Brownish-clay, nuggety texture. Grey. roots.		Firm.	0' 0" 178.0		A B S.L.	PUSHED		Wet. Wetter than plastic limit. NAT. M.C. = 47.5% - 44.7%
above, some fine sand and grits.	Brownish-grey.	Firm.	5' 0" 171.0		2 S.S.	7		Too stiff for vane test at 5' NAT. M.C. = 35.5% $C_u = 1728 \text{ PS}$ $\gamma_{\text{wet}} = 108.2 - 113.1 \text{ PCF}$ $\gamma_{\text{dry}} = 73.5 - 78.3 \text{ P}$
etty silty clay.	Grey	Stiff.	10' 0"		3 4 5	2" V.T. 2" V.T. 2" V.T.	PUSHED " "	Much wetter than plastic lim NAT. M.C. = 44.3%
above,	Grey				6 7 8	2" V.T. 2" V.T. 2" V.T.	" " "	Close to liquid limit when remoulded.
above, minor sand content.	Bluish-grey.	Firm.	15' 0"		9 10 11	2" V.T. 2" V.T. 2" V.T.	" " "	
etty silty clay.	Bluish-grey.	Firm.	20' 0"		12 13	2" V.T. 2" V.T.	" "	
above Odd grits.					14	S.S.	4 10/c	Quite moist.
ver of sand, grits, dy till.	limestone fragments. Dark grey.	Dense.	26' 10" 151.2		HOLE TERMINATED.			Note: See accompanying laboratory data sheets for vane and triaxial test result

BOREHOLE LOG

Job Name HWY. 401 - C.N.R. Crossing Job No. 57149A

Borehole No. 13





Client Dept. of Highways of Ontario Casing BX (2 1/2" diam.)

Boring Date April 5, 1958.

Datum Geodetic Compiled By M. Mindess

Checked By C. F. Freeman

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	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
Organic silty loam	Black		0' 0" APPROX 173.0					
Silty clay, nuggetty texture.	Grey-brown	Firm	5' 0"	1	 S.S.	6		NATURAL MOISTURE CONTENT = 46.4%.
Silty clay, nuggetty, some sand and grits.	" "	Firm		2	 S.S.	7		NAT. M.C. = 31.3% $Q_u = 2350$ P.S.F.
Silty clay, nuggetty texture, some grits.	Grey-brown	Firm		3	 S.S.	7		NAT. M.C. = 56.7% MUCH WETTER THAN PLASTIC LIMIT $Q_u = 1493$ P.S.F.
Silty clay, thin layers of fine sand to medium sand.	Grey	Soft.	9' 6" 169.6	4	 S.S.	4		NAT. M.C. = 54.4% CLAY NEAR LIQUID LIMIT.
SANDY TILL: Clayey and silty fine sand. Grits and rock fragments.	Grey	Compact.	11' 3" 167.2	5	 S.S.	17		NAT. M.C. = 8.6%.
As above.	"	"	15' 0"	6	 S.S.	32		WASH SAMPLE RETAINED.
Sandy till as above. Many limestone fragments.	"	"	20' 0"	7	 S.S.	17		
Sandy till.	Dark grey	Compact	25' 0"	8	 S.S.	28		
Sandy till.	" "	Dense	30' 0"	9	 S.S.	36		NAT. M.C. = 8.1%
Sandy till.	" "	Dense	35' 0"	10	 S.S.	38		
Sandy till.	" "	Very dense	41' 0" 138.0	11	 S.S.	75		NAT. M.C. = 8.0%

HOLE TERMINATED.

APPENDIX I

Laboratory Test Results and Graphs

- I. Typical Atterberg Limit Results - Cornwall Marine Clay.
- ii - iv. Charts for Comparison of Vane Tests and Laboratory Compression Tests.
- v - ix. Results of Various Triaxial Compression Tests on Cornwall Marine Clay.
- x. Typical Stress - Strain Curve. Obtained in Laboratory Compression Tests.
- xi. Plot of Minimum versus Maximum Principal Stresses at Failure in Triaxial Compression Tests: Summary of all Triaxial Test Results without applying Mohr's Circle Graphical Analysis.
- xii - xiv. Results of Consolidation Tests on Cornwall Marine Clay.
- xv. Average Shear Profile Determination showing Recommended Design Values.
- xvi. Time Required to Consolidate the Soft Clay Layer under Various Applied Loads.
- xvii. Graphical Presentation of Computations for Embankment Stability, considering Three Different Cases.

e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

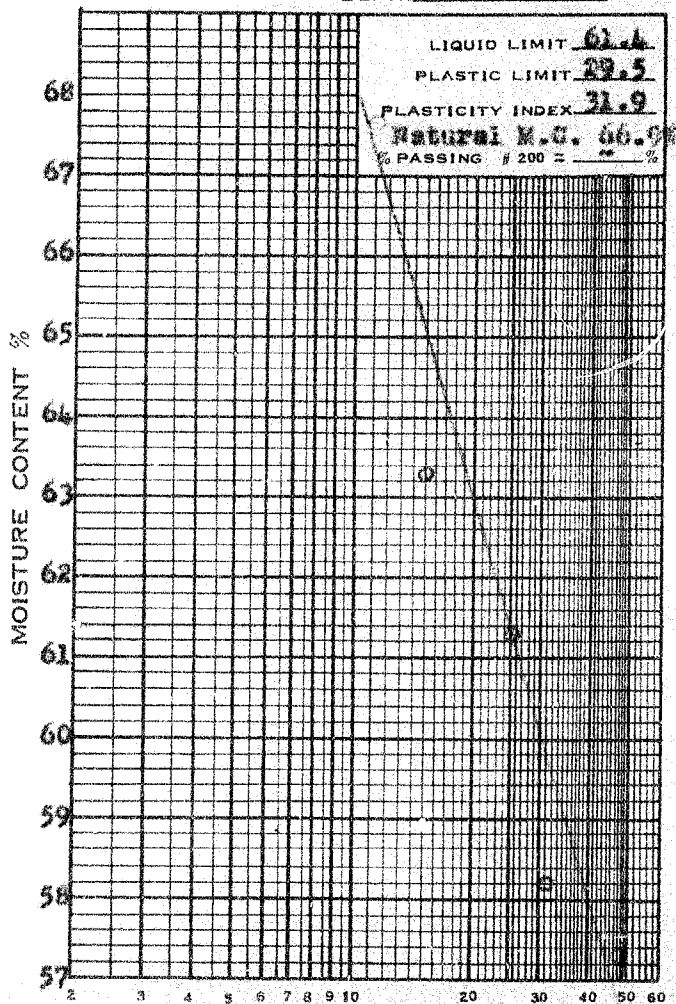
JOB No. 571424 PROJECT Hwy. 401 - C.N.R. Crossing, Cornwall.

SAMPLE FROM Borehole 5A, Sta. 11B

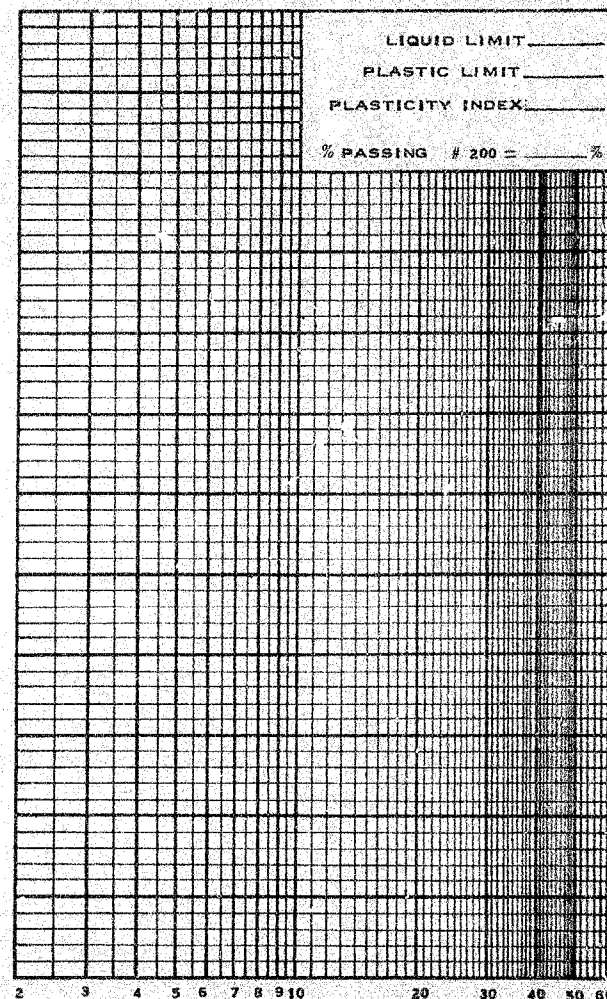
SAMPLE FROM _____

DEPTH 17'4" - 17'8"

DEPTH _____



MOISTURE CONTENT %



NO. OF BLOWS (LOG SCALE)

CHART FOR COMPARISON OF IN SITU VANE TESTS AND LABORATORY COMPRESSION TESTS

PLOT OF SOIL SHEAR STRENGTH VERSUS DEPTH

Job No. 57149A

B.H. 5 and 5A

B.H. 8 and 8A

Soil Shear Strength - p.s.f.

Soil Shear Strength - p.s.f.

Depth below surface - feet

Depth below surface - feet

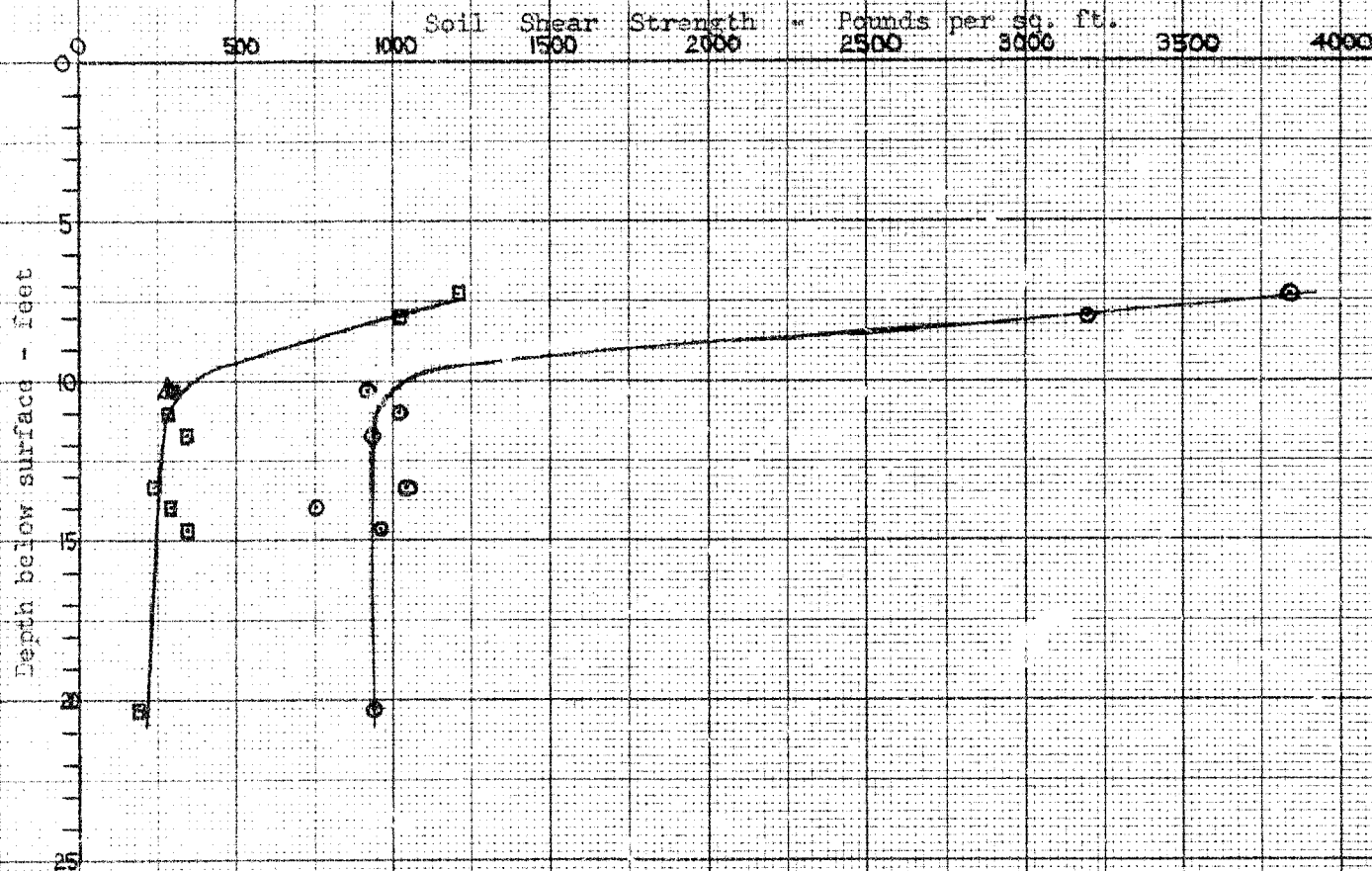
- △ - 'C' from unconfined compression or quick triaxial test
- - Shear strength from in situ vane test
- - Remoulded vane test 10 minutes after original test

CHART FOR COMPARISON OF IN SITU VANE TESTS AND LABORATORY COMPRESSION TESTS

PLOT OF SOIL SHEAR STRENGTH VERSUS DEPTH

Job No. 57149A

B.E. 7 and 7A



- △ - 'C' from unconfined compression or triaxial test
- - Shear strength from in situ vane test
- - Remoulded vane test 10 minutes after original test

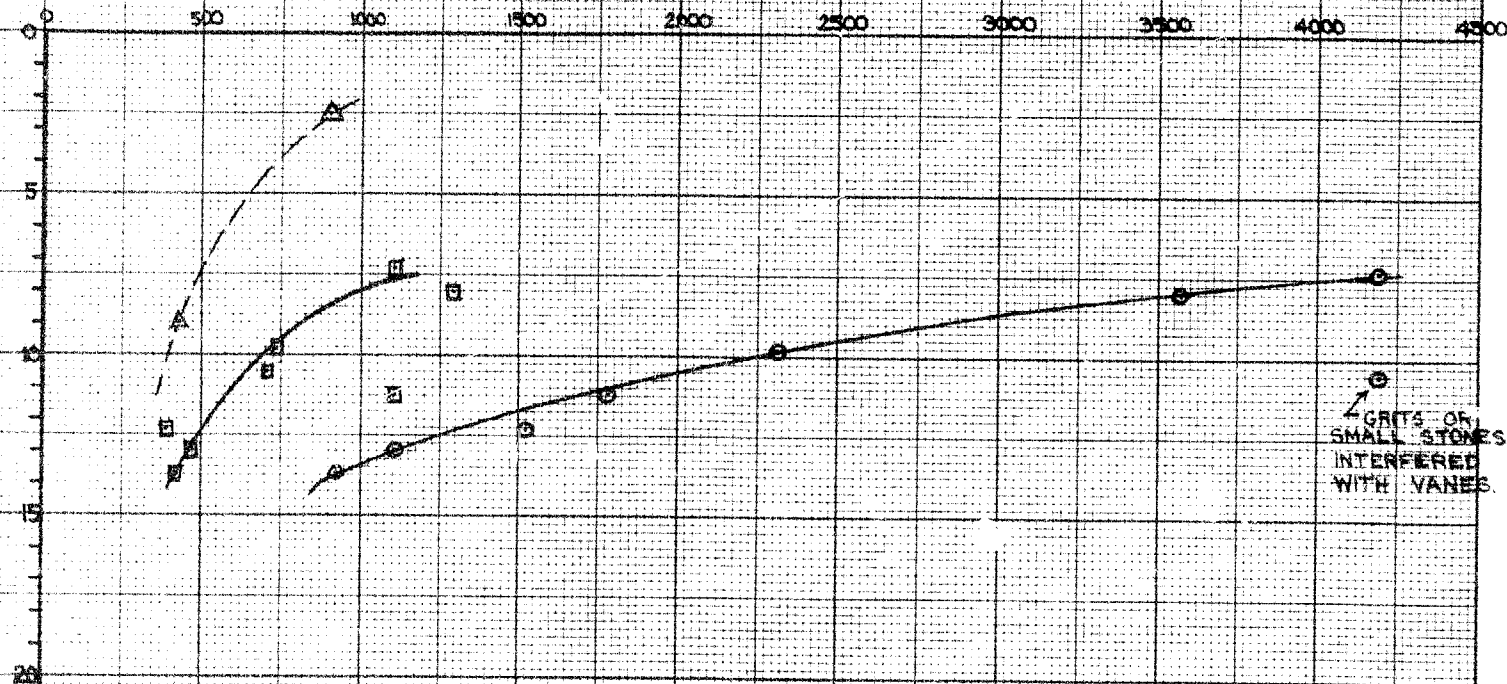
CHART FOR COMPARISON OF IN SITU VANE TESTS AND LABORATORY COMPRESSION TESTS

PLOT OF SOIL SHEAR STRENGTH VERSUS DEPTH

JOB NO. 57149B
B.H. II AND B.H. IIA

SOIL SHEAR STRENGTH - P.S.F.

DEPTH BELOW SURFACE - FT



GRITS OR SMALL STONES INTERFERED WITH VANES.

- △ - 'C' FROM UNCONFINED COMPRESSION OR QUICK TRIAXIAL TEST
- - SHEAR STRENGTH FROM IN SITU VANE TEST
- - REMOULDED VANE TEST 10 MINUTES AFTER ORIGINAL TEST

QUICK TRIAXIAL COMPRESSION TEST

ON SAMPLE OF GREY-BROWN, NUGGETTY
TEXTURED, SILTY CLAY

JOB NO. 57149A

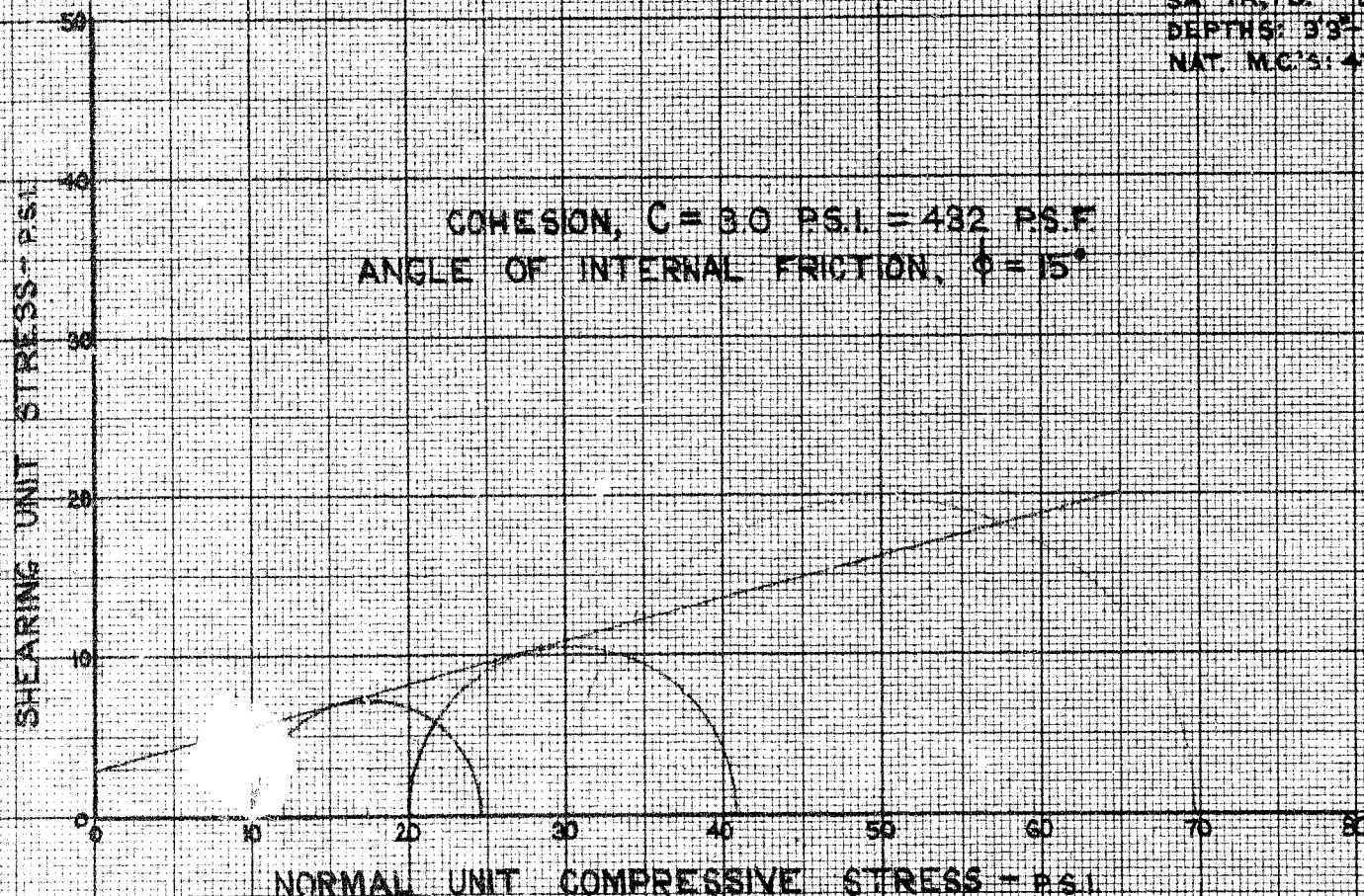
SA 1A, 1B. B.H. 8A

DEPTHS: 33'-39", 39'-43"

NAT. MC'S: 475%, 45.0%, 44%

COHESION, $C = 3.0 \text{ PSI} = 432 \text{ PSF}$

ANGLE OF INTERNAL FRICTION, $\phi = 15^\circ$



QUICK, UNDRAINED TRIAXIAL TEST

ON SAMPLE OF VERY SOFT, NUGGETTY, GREY
SILTY CLAY

JOB NO. 57148

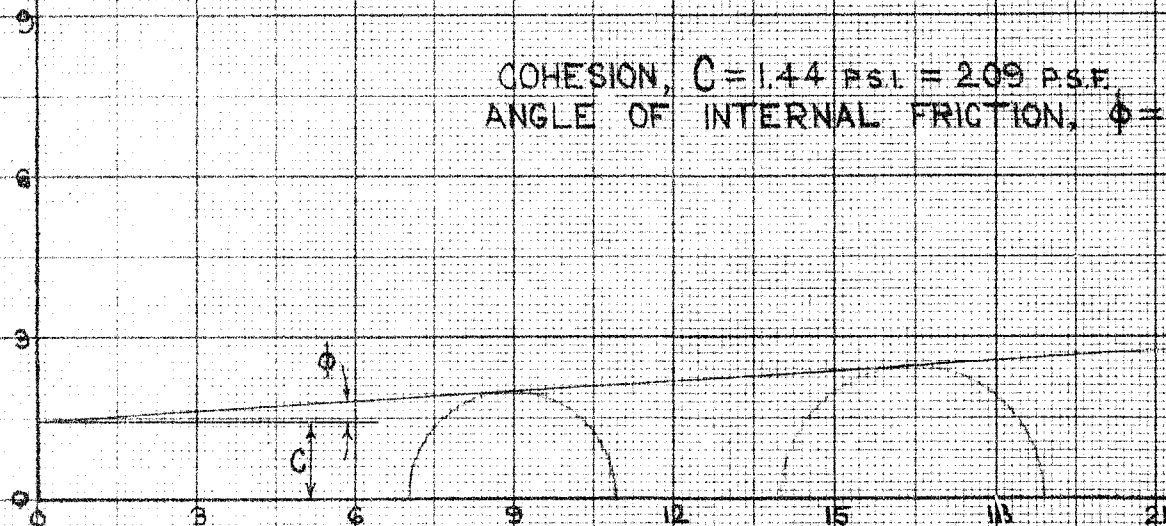
B.H. 9 SAMPLES: 4A, 4B

DEPTH: 8'-9"

NATURAL M.C.'s: 88.6%, 40.6%

COHESION, $C = 1.44 \text{ PSI} = 209 \text{ P.S.F.}$
ANGLE OF INTERNAL FRICTION, $\phi = 3\frac{1}{2}^\circ$

SHEARING UNIT STRESS - P.S.I.



NORMAL UNIT COMPRESSIVE STRESS - P.S.I.

QUICK UNDRAINED TRIAXIAL COMPRESSION TESTS
with
PORE WATER PRESSURE MEASUREMENTS

Job No. 57149B
Borehole 11
Samples 3B, 3C

Specimen No.	Depth	Degree of Saturation %	Wet Density p.o.f.	Natural M. C.
①	8'6"-8'9"	100.0	105.0	59.4%
②	8'9"-9'0"	100.0	105.0	54.6%
③	9'0"-9'3"	98.6	102.2	61.2%
④	9'3"-9'6"	98.6	102.2	60.3%

Shearing Unit Stress - p.s.i.

$$C_{TOTAL} = 480 \text{ P.S.F.}$$

$$\phi_{TOTAL} = 15^{\circ}0'$$

EFFECTIVE STRESS CIRCLES
MOST OF PRESSURE CARRIED
BY PORE WATER

MOST PROBABLE RUPTURE
ENVELOPE - TOTAL STRESSES

Normal Unit Compressive Stress - p.s.i.

0 5 10 15 20 25 30 35 40 45 50 55 60

0 5 10 15 20 25 30 35 40 45 50 55 60

QUICK UNDRAINED TRIAXIAL COMPRESSION TESTS
with
PORE WATER PRESSURE MEASUREMENTS

Job No. 57149B
Borehole 7
Sample 3.

Specimen No.	Depth	Deg. of Saturation %	Wet Density p.c.f.	Natural M. C.
①	9'0"-9'3"	100.0	110.0	49.5 %
②	9'3"-9'6"	92.2	110.0	44.7 %

Shearing Unit Stress - p.s.i.

$$C_{TOTAL} = 3.75 \text{ P.S.I.} \approx 540 \text{ P.S.F.}$$

$$\phi_{TOTAL} \approx 8^{\circ} 30'$$

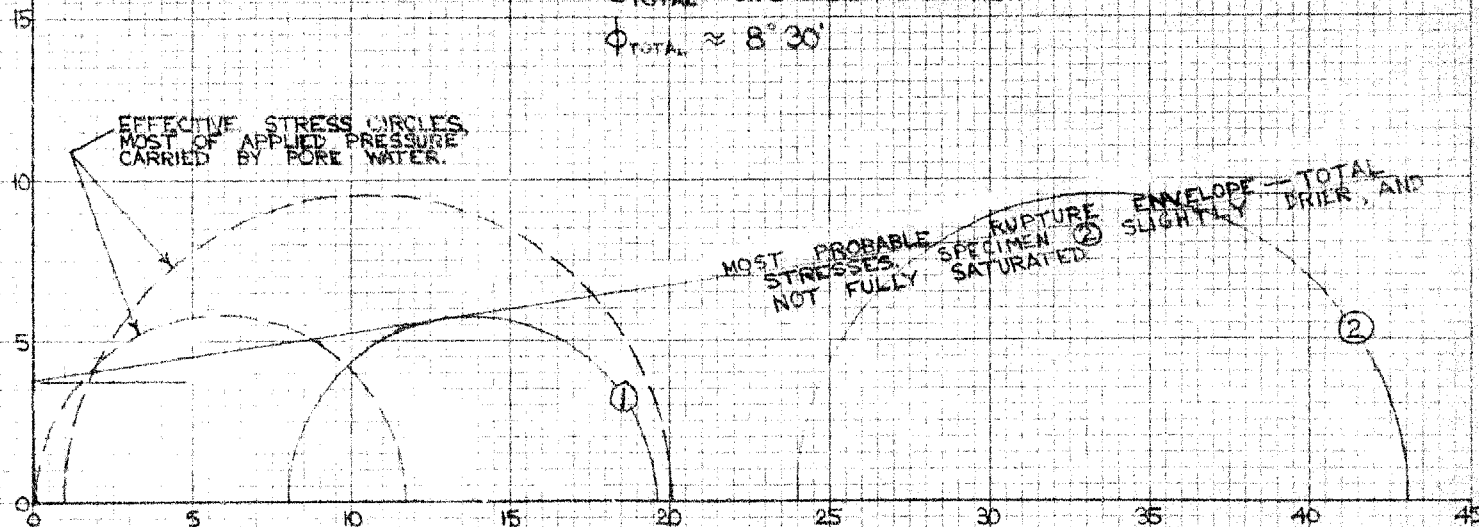
EFFECTIVE STRESS CIRCLES
MOST OF APPLIED PRESSURE
CARRIED BY PORE WATER.

MOST PROBABLE
STRESSES
NOT FULLY

RUPTURE
SPECIMEN ②
SATURATED

ENVELOPE - TOTAL
DRIER, AND
SLIGHTLY

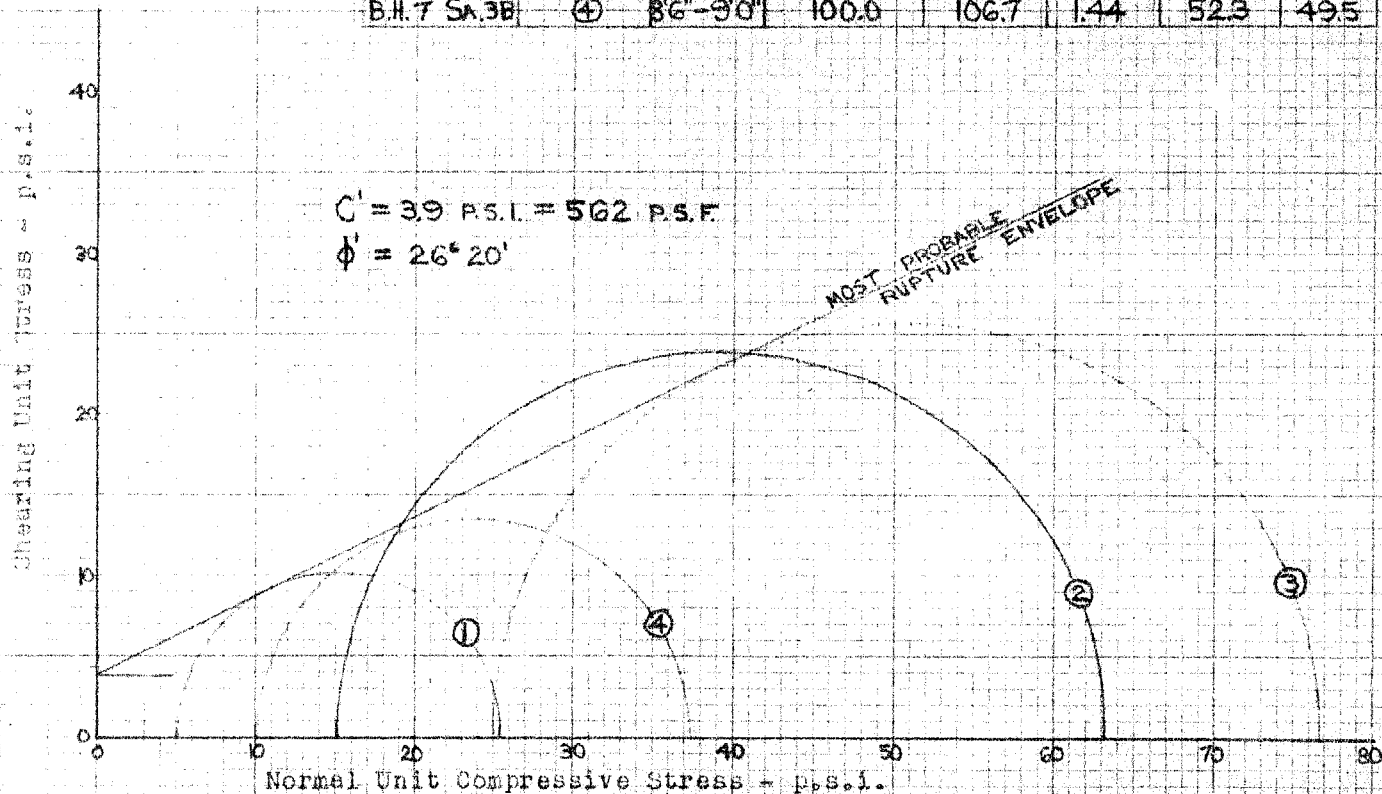
Normal Unit Compressive Stress - p.s.i.



FULLY DRAINED, SLOW TRIAXIAL COMPRESSION TEST
on samples of
CORNWALL MARINE CLAY

Job No. 571149B

	Specimen No.	Depth	Degree of Saturation %	Wet Density p.p.s.f.	Void Ratio	Initial M. C. %	Final M. C. %
B.H. 11A SA. 1B	①	56"-58"	100.0	111.0	1.23	45.0	43.6
B.H. 11A SA. 1B	②	58"-60"	100.0	111.0	1.23	45.0	44.6
B.H. 11A SA. 1C	③	60"-62"	100.0	108.5	1.37	50.4	49.3
B.H. 7 SA. 3B	④	86"-90"	100.0	106.7	1.44	52.3	49.5

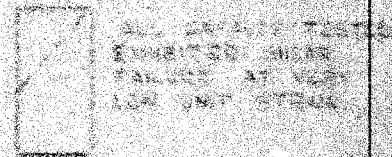


TYPICAL
SLOW TRIAXIAL
STRESS-STRAIN RELATIONSHIP
UNCONFINED COMPRESSION TEST

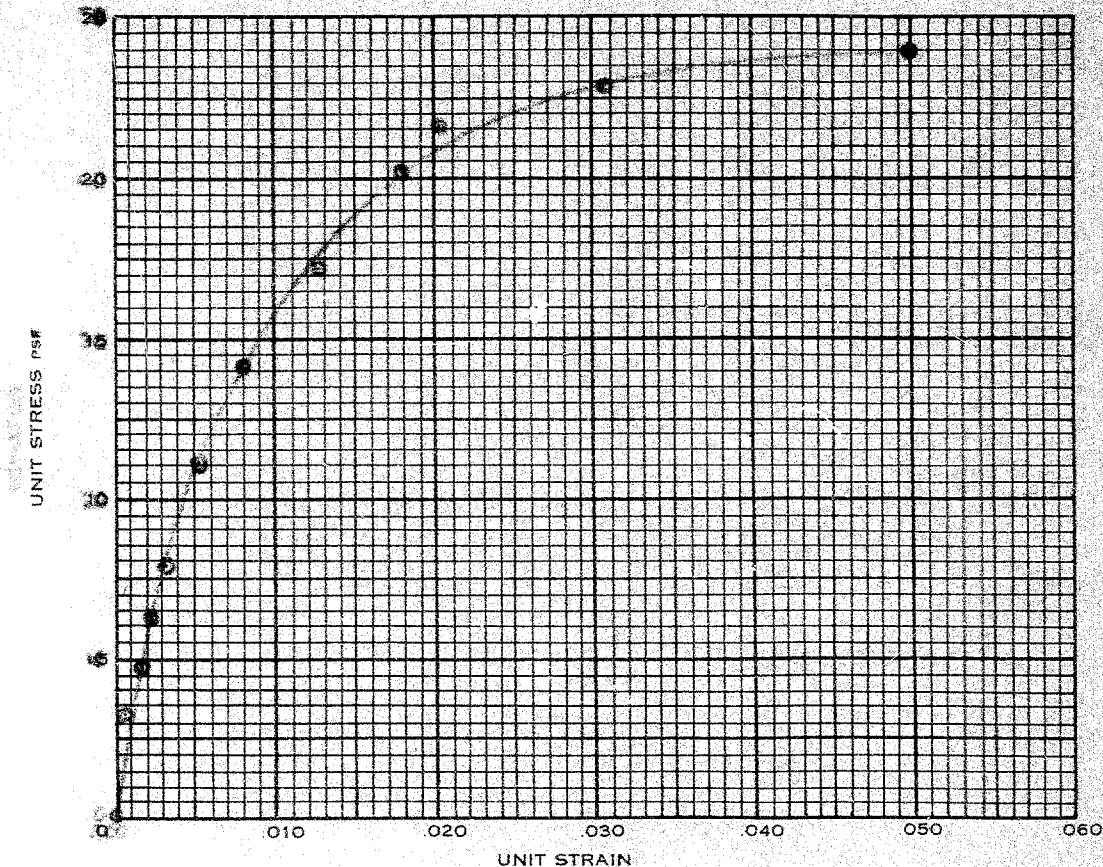
JOB No. 571428 NAME OF JOB HIGHWAY 401-CNR CONCRETE
HOLE No. 11A SAMPLE No. 13 DEPTH 12'-00"
TYPE OF SAMPLE: ☒ UNDISTURBED ☐ MODIFIED PROCTOR MAXIMUM DENSITY
☐ PROCTOR MAXIMUM DENSITY ☐ REMOULDED AT NATURAL MOISTURE CONTENT
☐ OTHERS DESCRIPTION: _____

MOISTURE CONTENT 50%
AVERAGE DIA 1.25 AVERAGE HEIGHT 3.00
SPECIFIC GRAVITY OF SOLIDS 2.74
DENSITY 111.0 LBS./CU. FT. 17.7
DEGREE OF SATURATION 100.0 %
UNCONFINED COMPRESSIVE STRENGTH 4.8

SKETCH AT FAILURE



STRESS - STRAIN DIAGRAM

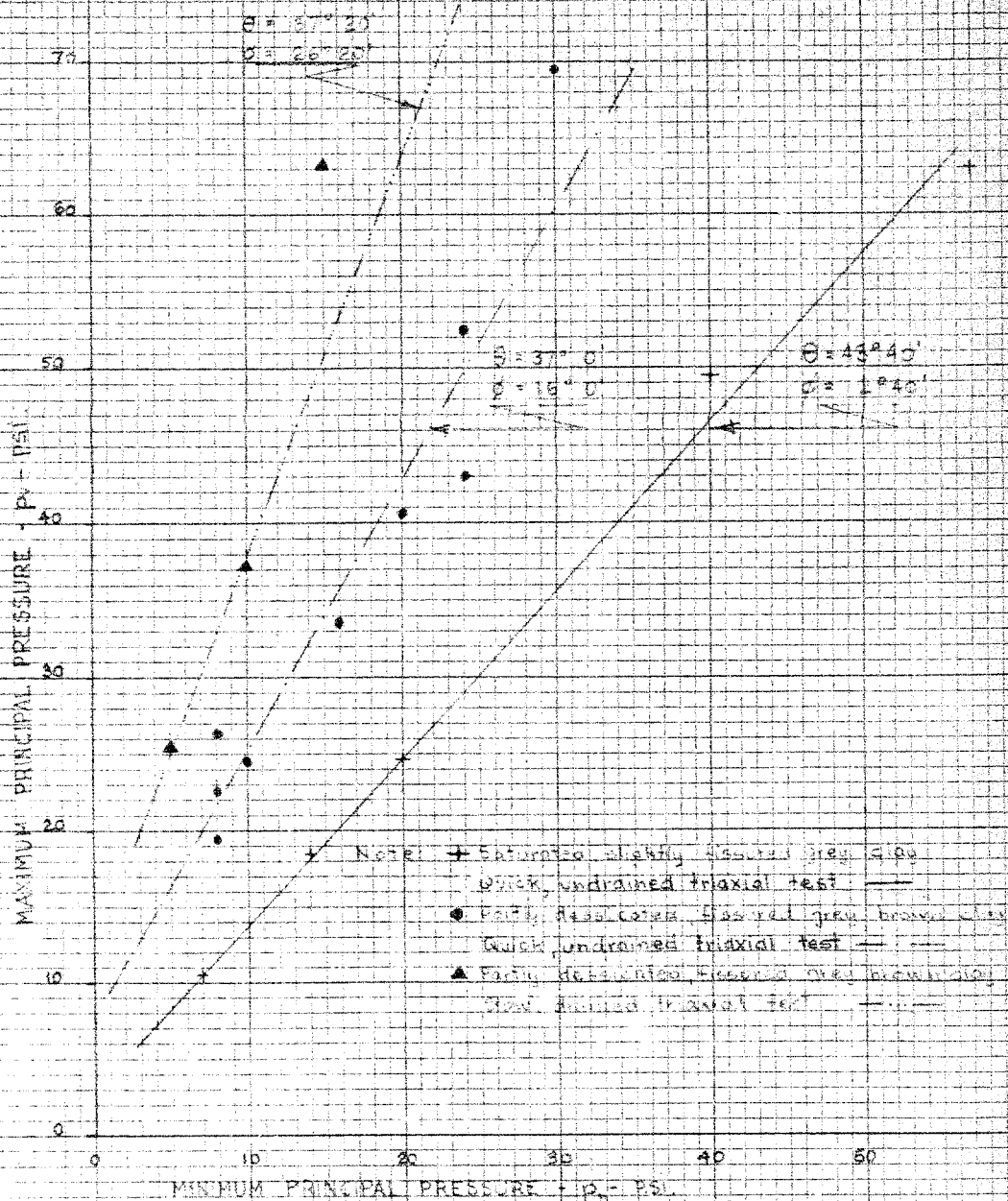


MODULUS OF DEFORMATION =

REMARKS:

TESTED: PM 3 DATE: May 2-1958
PLOTTED: PM 3 DATE: May 2-1958
COMPUTED: PM 3 DATE: May 2-1958

e. m. peto associates ltd.
850 ROSELAWN AVENUE
TORONTO - ONTARIO



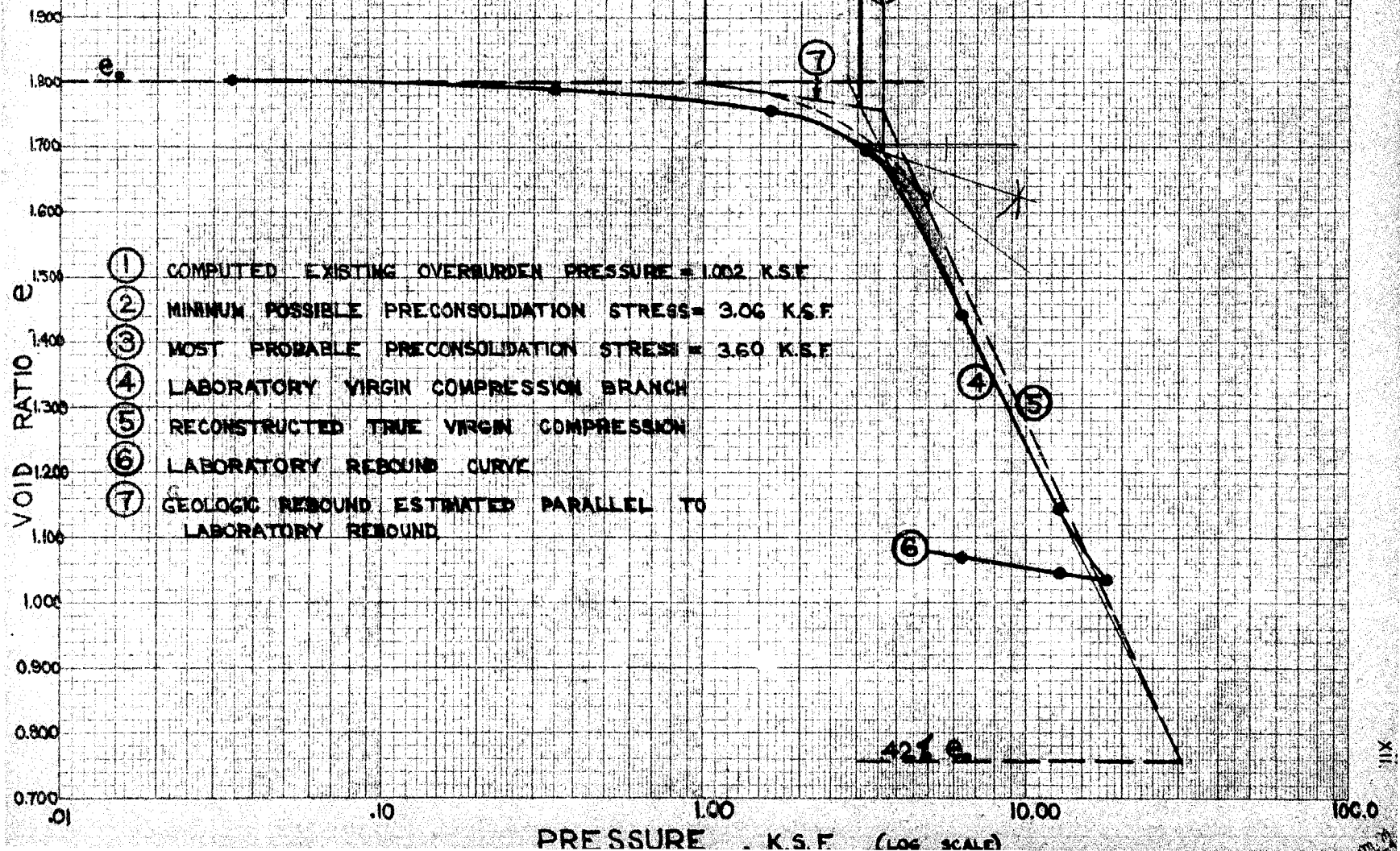
MINIMUM VS. MAXIMUM PRINCIPAL STRESSES AT
 FAILURE IN TRIAXIAL COMPRESSION TESTS

CONSOLIDATION TEST

JOB NO. 5749A BOREHOLE NO. 5A

SAMPLE NO. 4C DEPTH: 11'0" - 11'4"

PRESSURE-VOID RATIO CURVES



Soil Testing Laboratory
E. M. PETO ASSOCIATES LTD.

CONSOLIDATION TEST TIME CURVE

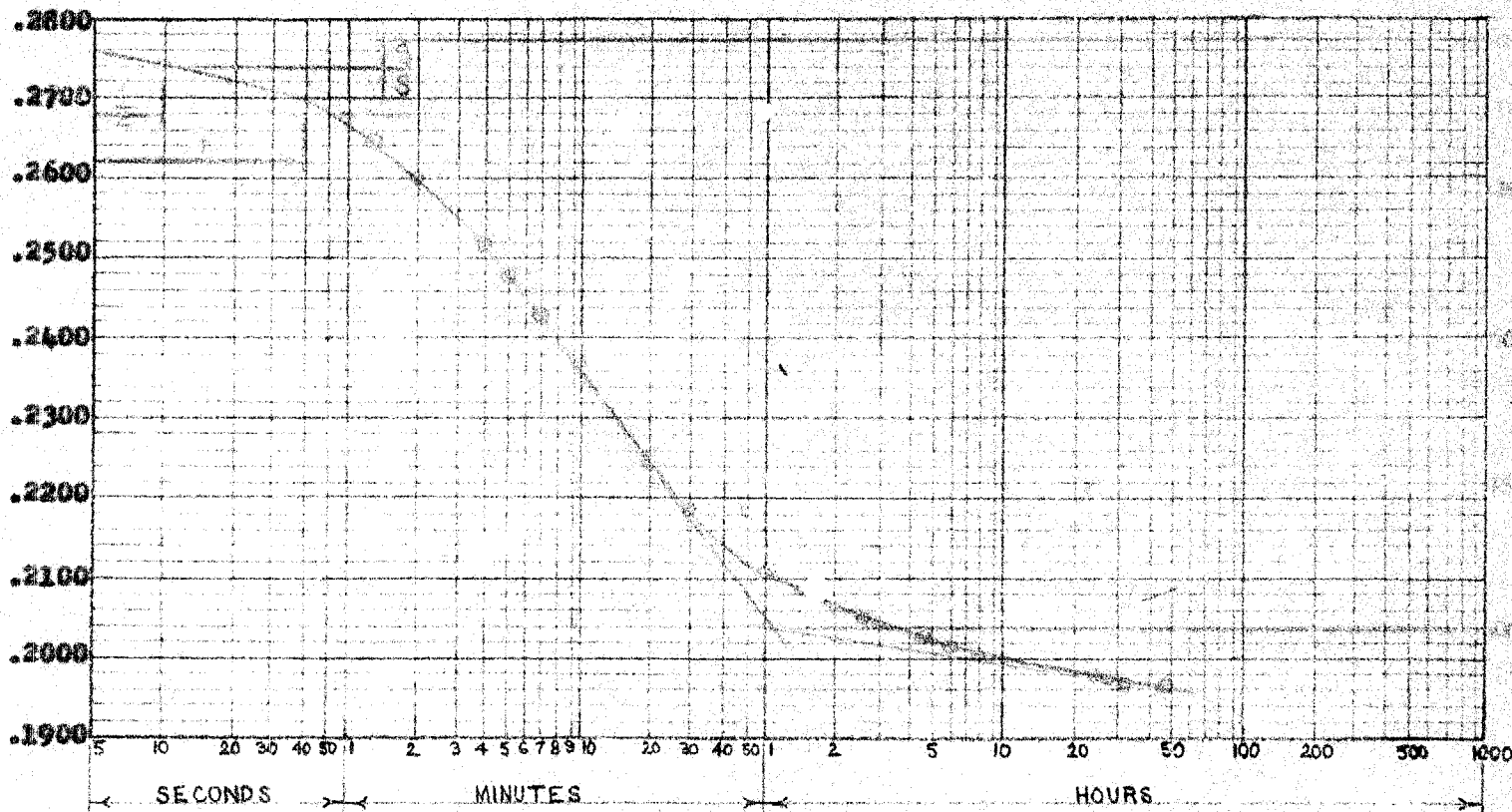
Job No. **57149A.**

Borehole **..5A...**

Sample No. **..40....**

Depth from **..11'0" to 11'4".**

DIAL READING



PERCENT CONSOLIDATION

ELAPSED TIME (LOG SCALE)

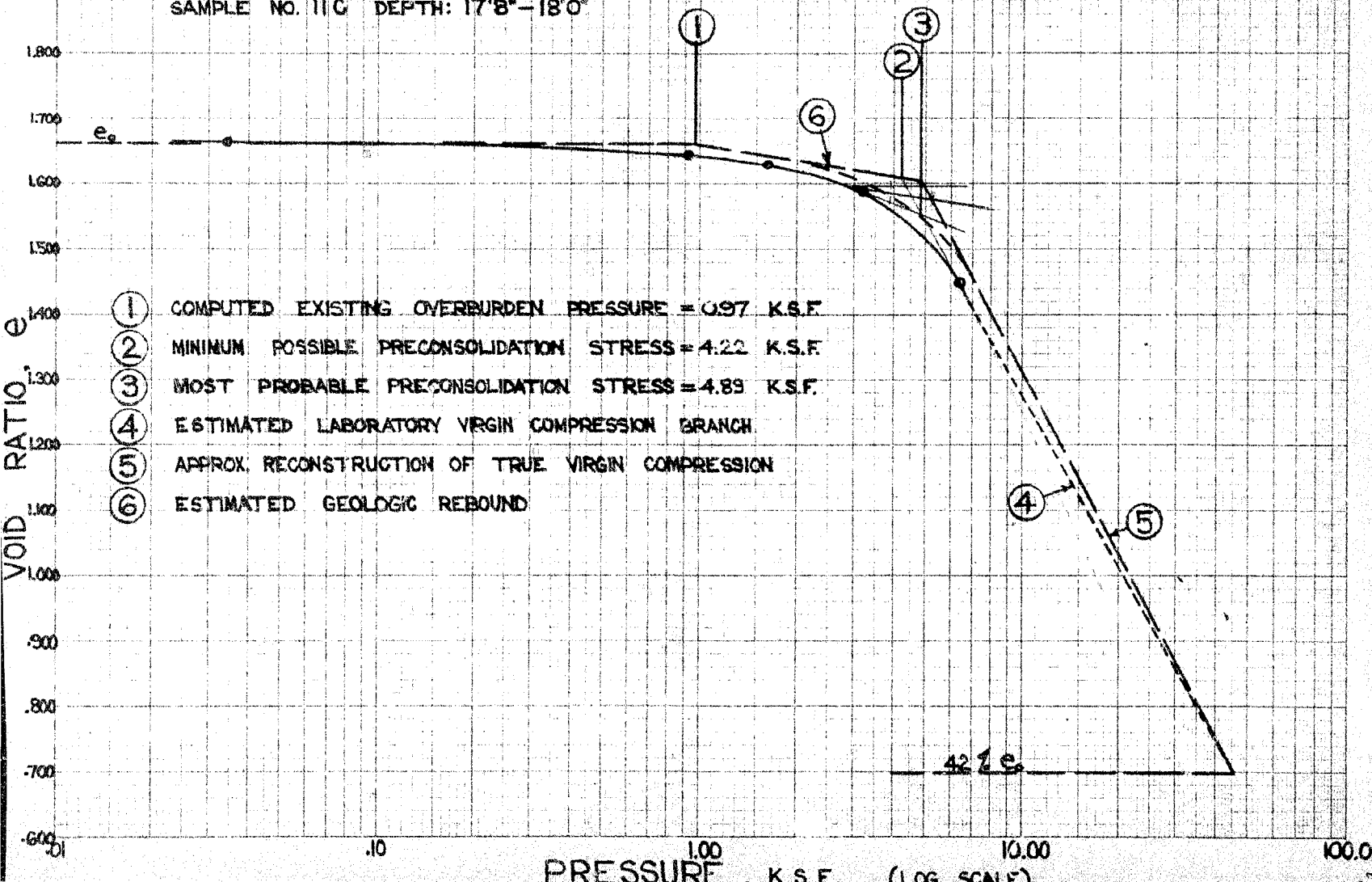
Load on Sample **441.6** lbs.
Previous Load **221.2** lbs.
Load Increment **220.4** lbs.

Date of Test **Apr. 21-23, 1958.**
Tested by **E. M. Peto**
Checked by

CONSOLIDATION TEST

JOB NO. 57149A BOREHOLE NO. 5A
 SAMPLE NO. 11C DEPTH: 17'8"-18'0"

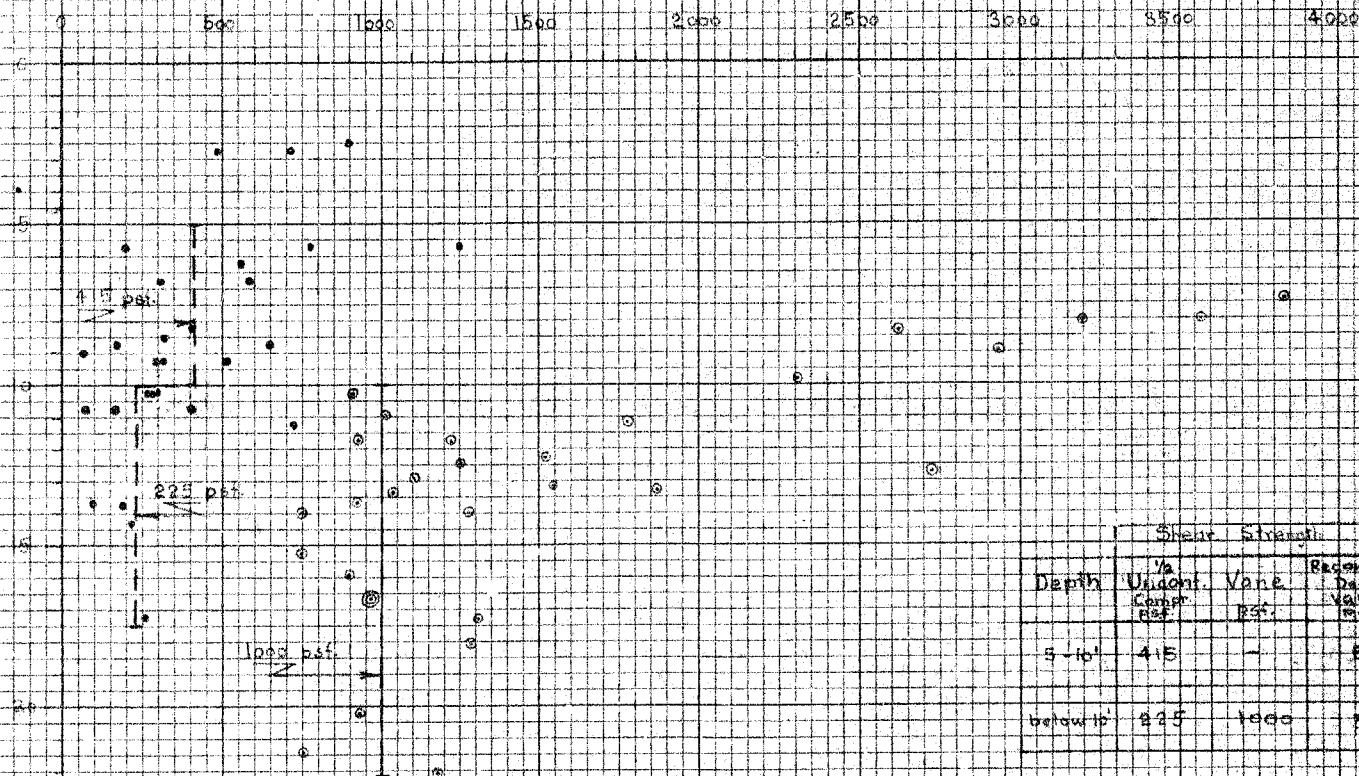
PRESSURE-VOID RATIO CURVE



AVERAGE SHEAR PROFILE DETERMINATION

SHEAR STRENGTH in PSF.

DEPTH BELOW THE GROUND SURFACE IN FT.



Depth	Shear Strength		Recommended Design Value in PSF
	1/2 Unconf. Comp. Test	Vane Test	
5-10'	415	-	550
below 10'	825	1000	500

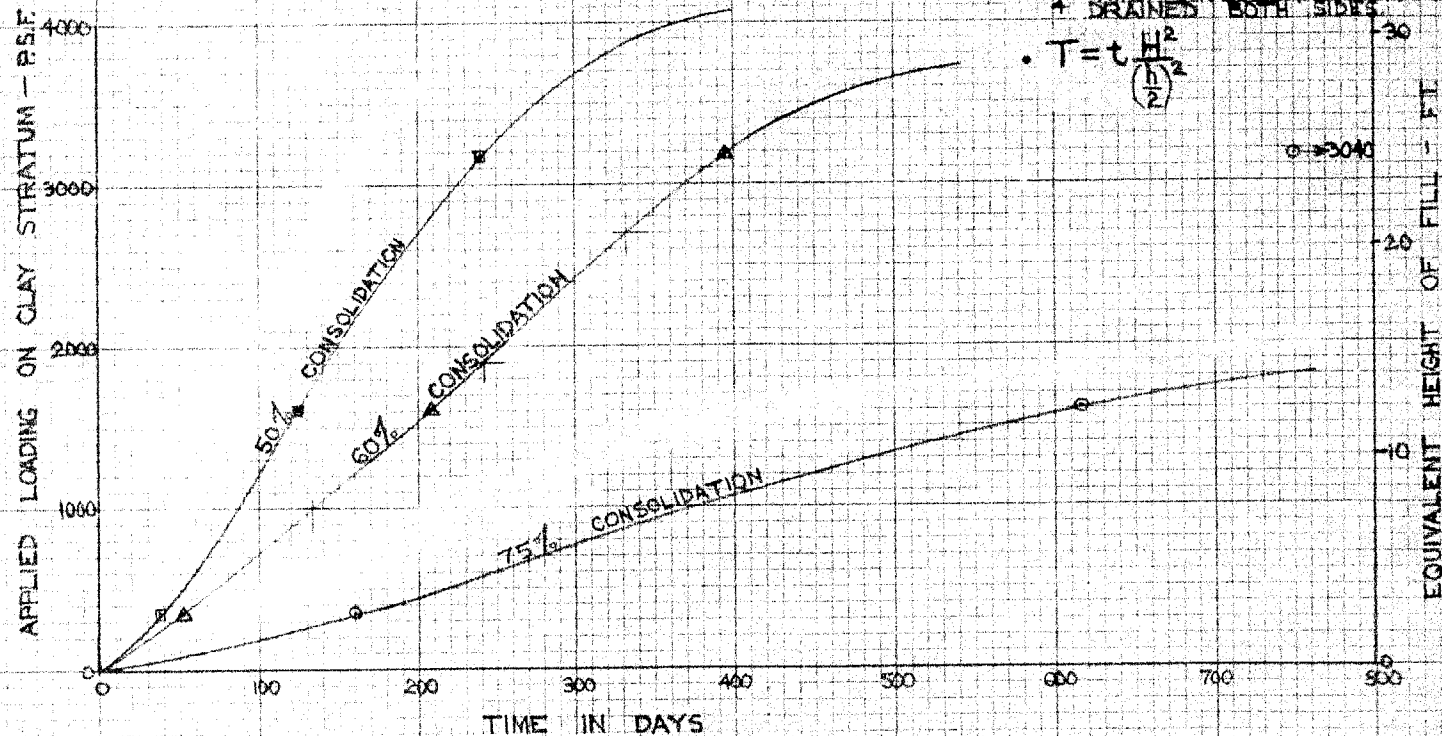
- Shearing Strength determined by unconfined compression test.
- Shearing strength determined by in situ Vane shear test.

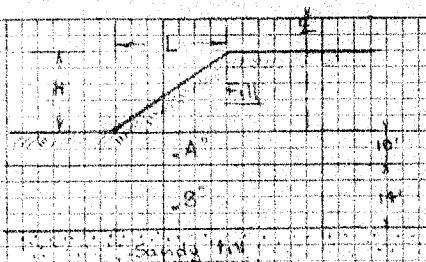
TIME REQUIRED TO CONSOLIDATE THE SOFT CLAY LAYER UNDER VARIOUS APPLIED LOADS

ASSUMPTIONS:

- CLAY LAYER 15 FT THICK.
- OVERLAIN BY PARTLY DESICCATED CLAY UNDERLAIN BY SANDY FILL.
- THE CLAY LAYER DRAINED ONE SIDE ONLY.
- 2" THICK CONSOLIDATION SAMPLE.
- DRAINED BOTH SIDES.

$$T = t \frac{H^2}{\left(\frac{H}{2}\right)^2}$$





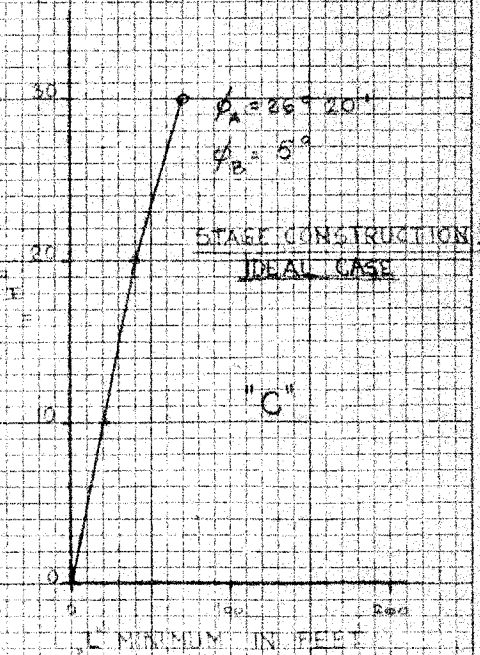
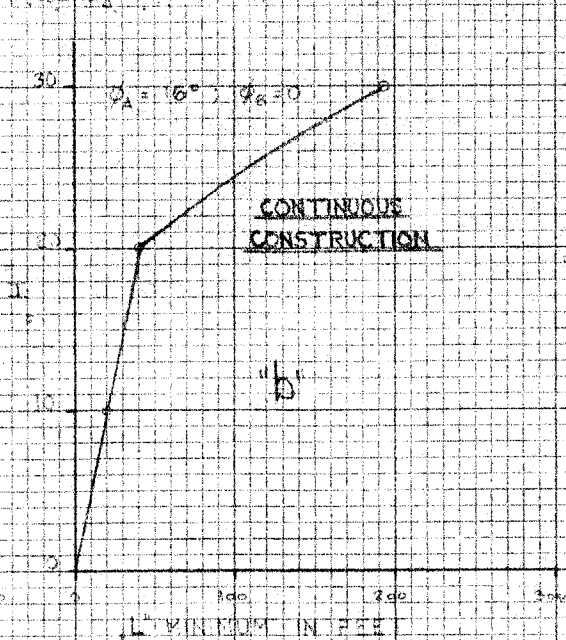
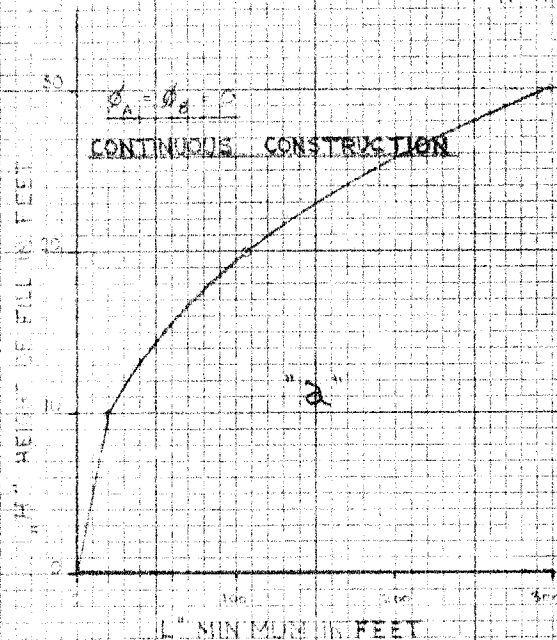
$$S_A = 0$$

$$\phi_A = 45^\circ$$

$$S_B = \frac{550}{F}$$

$$\phi_B = \frac{300}{F}$$

FACTOR OF SAFETY = 1.5



EMBANKMENT STABILITY

Note: Minimum slope 2 horizontal and 1 vertical has been used based upon practical considerations.

DEPARTMENT OF HIGHWAYS OF ONTARIO

HIGHWAY 401 - C. N. R. CROSSING

W. P. 93 - 37 TOWNSHIP OF CHARLOTTENBURG

SOILS REPORT

E. M. PETO ASSOCIATES LTD.

TORONTO, ONTARIO

MAY, 1958

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 57149B

850 roselawn avenue,
TORONTO, ONTARIO.

RUSSELL 1 - 4955.

May 30th, 1958.

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

Attention: Mr. J. C. McAllister

Re: Hwy. 401 - C.N.R. Crossing
W.P. 68 - 57 Township of Charlottenburg

Dear Sir:

We are pleased to submit herewith four (4) copies of our soils report for this project. Our terms of reference for the work at this site required that we should carry out a complete soils investigation and make a report on our findings together with such recommendations and conclusions as we considered pertinent.

The soil conditions and factors leading to our conclusions are considered in detail in the report attached hereto, together with an appendix containing laboratory test results. Here for your convenience is a summary of our findings and recommendations.

1. The soil conditions on this site consist of:
 - a) A thin surface layer covering the entire site of organic topsoil derived from the material below.
 - b) A stratum from 5-1/2 to 15-1/2 feet thick of weak, fissured silty clay, which does not occur over the entire site, but only on the West side of the railway tracks South of the Highway 401 revised centre-line. For this reason the clay stratum at this site will only affect the Eastbound traffic lane on one side of the tracks.

SUPER IMPOSED DOCUMENT MAY
APPEAR AS MULTI-FEED ON FILM.

- c) A dense glacial till which overlies bedrock.
- d) A limestone bedrock of generally good quality, the top of which lies between elevations 151.0 and 145.0.
- 2. The revised Highway 401 centre-line on the east side of the tracks roughly follows the longitudinal centre-line of a drumlin.
- 3. The ground water table, except on the drumlin ridge itself, is virtually at ground surface. Provision must therefore be made for some pumping in any excavations.
- 4. There appear to be no soil problems connected with the future Westbound traffic lane, since the underlying soil consists mainly of dense glacial till.
- 5. It is possible to found the Eastbound bridge piers and abutments on large spread footings placed directly on the glacial till, with fairly high safe allowable loadings, which are given in the body of the report. The recommended loadings are based on limiting total settlements to one inch.
- 6. The thickness of clay under the West pier and abutment of the Eastbound bridge is limited; accordingly no undue stability problems should be encountered in the excavations for these footings, and the railway embankment will not be affected.
- 7. The thirty foot high approach embankments at both the East and West abutment locations will be stable, although some settlement of the West approach fill can be expected, where it is located over the marine silty clay stratum.
- 8. We have not discussed the use of piles for this site, since we feel that soil conditions do not warrant their use, and in addition considerable difficulties in driving them would be involved.

We shall be pleased to supply any further information that the Department or your Consultants may require.

Yours very truly,

E. M. PETO ASSOCIATES LTD.,



E. M. Peto, P. Eng.

MM:ab

Job No. 57149B

Client's Ref. No.

Date May 20th, 1958.

Report on
SOIL SITE INVESTIGATION
at
REVISED LOCATION
HIGHWAY 401 - C. N. R. CROSSING
W. P. 69 - 57 TOWNSHIP OF CHARLOTTENBURG
for
DEPARTMENT OF HIGHWAYS OF ONTARIO.

INTRODUCTION:

We were requested, by verbal order on April 3rd, 1958, from Mr. J. C. McAllister, acting for the Chief Bridge Engineer, to carry out a complete investigation of a proposed crossing site some 550 feet Northerly from proposed Highway 401 centre-line "G". Our boring programme and subsequent laboratory testing was to be of such an extent that a complete appreciation of the soil conditions at this site could be made.

PROGRAMME OF WORK:

- April 3rd, 1958: Investigation at line "G" crossing indicates poor soil conditions. Meeting with Mr. McAllister, and decision to investigate apparent drumlin ridge 550 feet to the North.
- April 7th, 1958: Equipment moved from line "G" to new site and work commenced.
- April 8th, 1958: First test hole completed. Results good.
- April 9th - 12th, 1958: Equipment removed from site pending further discussion, and decision as to whether or not to carry on with this investigation.

PROGRAMME OF WORK: (Cont'd)

April 14th, 1958: Equipment moved back to site. Further soil borings commenced.

April 18th, 1958: Field elevations taken by Engineer.

April 23rd, 1958: Work continuing on site but slight revision to line made by Mr. S. Markiewicz of the Department of Highways.

May 2nd, 1958: Field work completed.

May 3rd, 1958: Final check of ground water levels, equipment moved from site with help of bulldozer, and trucked to Belleville.

GENERAL INFORMATION:

a) Standard soil sampling procedures were followed. These are described in Appendix II.

b) Rock core was obtained only at one borehole on this site, since a number of test holes at the adjacent site at line "G" had been diamond drilled to prove the reliability of the bedrock, and since the bedrock appeared to be quite consistent.

c) The ground water observations on this site were made over an extended period of time, are quite reliable, and are consistent with results obtained at the line "G" site.

d) Laboratory test results and detailed borehole logs are attached, together with a site plan showing both the original and the revised lines, and two soil cross-sections. All elevations are to Geodetic datum, and are referred to a bench mark in the form of a bolt set in the East face of a concrete box culvert under the railway tracks, approximately 210 feet South of the revised line. The elevation of this bench mark was taken to be 1'8.00.

SITE AND GEOLOGY:

The site lies in the physiographic region known as the Glengarry till plain. The topography in the area is strongly undulating to rolling. Many drumlins, or elongated rounded ridges of glacial material, roughly parallel to the St. Lawrence River valley, occur in the region.

The final revision line investigated, on the East side of the railway tracks, follows the approximate longitudinal centre-line of one of these drumlins. On the West side of the railway tracks, which are built roughly at the end of this drumlin, the Southerly part of the revised Highway 401 right-of-way crosses the low ground between the drumlins where the soils consist of post-glacial marine clays.

Over much of the area the till is less than 25 feet in depth, and directly overlies bedrock, which is a Chazy or Black River limestone.

SOIL CONDITIONS:

The soil types encountered on this site are:

- i) Poorly developed organic topsoil of very limited thickness, derived from the material below.
- ii) Very fine-textured, relatively stone-free, marine clay.
- iii) Glacial till with a heavy loamy texture and a high proportion of limestone.

i) Organic Topsoil

The entire proposed bridge site is covered by one to two feet of organic black to brownish-black topsoil. Where this material overlies glacial till it is generally only one foot or less in thickness, brownish-black in colour, and is basically an organic sandy loam. Where the topsoil overlies the clay at the lower parts of the site, it is one to two feet in thickness, black in colour, more highly organic, and consists of a heavy organic silty loam.

SOIL CONDITIONS: (Cont'd)ii) Marine Silty Clay

The marine silty clay stratum occurs only to the South of the revised Highway 401 centre-line, and on the West side of the C. N. R. railway tracks. It possibly also occurs at depth under the railway embankment South of the revised highway line. The thicknesses of this clay encountered at our borings vary from 8-1/2 to 15-1/2 feet.

The silty clay is brownish-grey in colour where it has been subjected to some desiccation or oxidation, but is grey at depth. It has a fissured, nuggety texture throughout.

The strength and consolidation characteristics of the Cornwall marine clay have been tested extensively both for the adjacent crossing line G, and at the Highway 401 - C. P. R. crossing. In addition there is some reliable published data dealing with the clay in this area. The conclusions to be reached are:

- a) The clay is very weak, with average laboratory shear strength or cohesion values of 515 p.s.f. for the grey-brown partly desiccated material, and 250 p.s.f. for the grey clay beneath.
- b) "In situ" vane shear tests (see following graph) in this material give results which are consistently 3 to 4 times higher than the laboratory shear strengths. However, on the basis of current available data, the vane shear test results are considered too high.
- c) The clay has been preconsolidated in the past, with preconsolidation stress in the order of 2600 p.s.f.
- d) In laboratory quick undrained tests, quick triaxial tests, slow triaxial tests, with pore water pressure measurements, and field vane shear tests, the clay exhibits brittle type failures at low unit strain. The inference to be drawn here is that failures in the field, should they occur, would be sudden, and with little warning.
- e) The clay is relatively sensitive (i. e. it loses most of its original strength when remoulded), with sensitivities ranging from 3 to 10.

SOIL CONDITIONS:

ii) Marine Silty Clay (Cont'd)

- i) The clay is fully saturated, with high natural moisture contents, generally close to or exceeding the liquid limit.

Between the marine silty clay and the underlying glacial till is a transitional zone some 1-1/2 to 4 feet thick of grey sandy and silty clay with many grits and rock fragments. This stratum is generally wet, and is loose to compact.

iii) Glacial Till

The major soil stratum on this site is glacial till, consisting of a grey silty fine sand with many grits and pebbles. A typical grading curve of this material is included in Appendix I.

The till is generally compact to dense, throughout the major part of the stratum, and is very dense at depth.

Natural moisture contents in the till range from 5.0% to 17.5%, and it is generally in a very moist to wet condition.

The unit weight of the material in situ is in the order of 140 p. c. f.

iv) Limestone Bedrock

Underlying the entire site is a dark grey limestone containing occasional fossils. Thin shale and sand seams are also interbedded with the limestone. It appears that in the location of borehole 1 limestone boulders or fragments occur immediately above the limestone bedrock.

There is no great variation in the top of bedrock level under the site, although it does appear to dip slightly in the Westerly and South-Westerly direction.

WATER CONDITIONS:

At the time of our investigation in April, 1958, the water table, from numerous reliable observations, was very close to the ground surface. Observations at the line "G" crossing made in January, 1958, showed exactly the same condition. However, it is likely that the ground water level will drop slightly during the summer months.

ENGINEERING CONSIDERATIONS:

Laboratory triaxial tests have been performed on samples from this site, in which measurements of the pore water pressure were made. The results of these tests permit analysis of the clay to be made in terms of effective stresses rather than total stresses. Such an analysis should give the best approximation of the behaviour of the marine clay under loadings from high embankments ("The Measurement of Soil Properties in the Triaxial Test", Bishop and Henkel. Edward Arnold Publishers Ltd, London, 1957, pp 22-25).

Since these additional test results will not affect the construction of the bridge at the revised crossing site, they are included in detail in our supplementary report number 57149A, dealing with the results of additional investigations on the marine clay at line "G".

RECOMMENDATIONS AND CONCLUSIONS:

1. It is our understanding that only the Eastbound traffic lane of divided Highway 401 is to be constructed at present. It has been considered necessary for some crossings in the Cornwall area to be constructed as a complete unit to accommodate both the Eastbound and Westbound traffic lanes. However, the soil conditions on this site do not necessitate such a form of construction.
2. The soil under the future Westbound traffic lane (North of Highway 401 centre-line) at this crossing consists mainly of dense glacial till, and therefore when this is built at some future date, no special foundation problems will be involved, and the Eastbound structure and its approaches should not be affected.
3. The weak marine clay on this site occurs only under the Eastbound lane on the West side of the railway tracks, and is only of limited thickness.
4. It is possible to found the Eastbound bridge piers and abutments on large spread footings placed directly upon the glacial till. We summarize below the safe allowable loadings, and recommended footing depths:

RECOMMENDATIONS AND CONCLUSIONS:

4. (Cont'd)

Location of Footings	Recommended Depth below present grade	Safe Allowable loading figs. 4 ft. wide or less.	Safe allowable loading figs. 16 ft. wide.
East Abutment	8 ft.	3.8 t.s.f.	3.0 t.s.f.
East Pier	8 ft.	3.8 t.s.f.	3.0 t.s.f.
West Pier	Approx. 15 ft. (Elev. 163.0)	2.4 t.s.f.	1.9 t.s.f.
West Abutment	Approx. 13 ft. (Elev. 165.0)	4.3 t.s.f.	3.7 t.s.f.

Bearing values for intermediate footing widths are approximately proportional.

5. The recommended loadings are governed by consideration of settlement rather than by the safe bearing strength of the glacial till. Accordingly, the loadings given are reduced to allow for this factor.

Thus the total settlements should not be greater than one inch, and differential settlements would be some fraction of the total settlements.

6. It is possible for plastic deformation of the clay stratum on the West side of the railway tracks under the Eastbound lane to occur as a result of the superimposed load of the embankment. This in turn would produce some lateral thrust on the West pier and West abutment. Accordingly it may be considered desirable to allow for this in the design, either by providing some form of key between the footing and the ground surface at this point, or placing the footing at a slightly greater depth in the glacial till.

7. a) All excavations on this site will probably encounter some water. The amounts of water in the excavations should not be excessive, and can probably be controlled by intermittent pumping, provision for which should be made.

RECOMMENDATIONS AND CONCLUSIONS: (Cont'd)

7. b) In view of the limited thickness of clay under the West pier and abutment of the Eastbound bridge, no undue stability problems are envisaged in the excavations for these footings, and the railway embankment will not be affected.
- c) It is considered that quicking or boiling of the glacial till at the bottom of the footing depths is unlikely to be a problem.
8. Thirty foot high approach embankments at both the East and West abutment locations will be stable, although some settlement of the West approach fill can be expected where it is located over the silty clay material.
9. In view of the relatively small quantity of marine clay in the immediate vicinity of the bridge, it may be considered expedient to remove this material completely for a distance of approximately 90 feet back from the West pier, over the core width of the embankment.

Such a course has the advantage of eliminating the possibility of plastic failure of this material around the bridge pier referred to in item 6 above.

E. M. PETO ASSOCIATES LTD.,

MM:sh







E. M. Peto, P. Eng.

BOREHOLE LOG

Job Name Highway 401 - C.N.R. Crossing Borehole No. 1
 Client Dep't. of Highways of Ontario Casing BX (2-1/2" diam.) Boring Date April 18th - 19th, 1958
 Datum Geodetic Complied By H.M.L. Checked By L.H.L.

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
Organic silty loam.	Black		0' 6" 189.6					
Silty fine sand, grits, some organic matter.	Yellowish-Brown	Loose.	5' 0"		1  S.S. 6	G	WET	W.L. = 4' 4", APR. 19, 1958 29' HOLE, 22' CASING, BAILED 19' NIGHT BEFORE W.T. = 4' 3" MAY 3, 1958
Silty fine sand with many rocks and boulders.			10' 0" 178.8		2  S.S. 43		WET	W.L. = 10' 4", APR. 21 50' HOLE, 27' CASING
Fine sand, grits and pebbles.	Mixed Brown	Dense.	15' 0"		3  S.S. 35		MOIST NAT. M.C. = 76%	
Till: Silty fine sand, grits and pebbles.	Light Grey	Dense	20' 0" 169.6		4  S.S. 60 1/2"			
As above.	Light Grey	Dense	25' 0"		5  S.S. 41		QUITE MOIST NAT. M.C. = 15.3%	
Silty fine sand, grits and pebbles.	Light		30' 0"		6  W.S. -			
Silty fine to medium sand, grits and pebbles.	Brownish-Grey		35' 0" 154.6		7  S.S. 62		MOIST	
Silty very fine sand, grits and angular rock fragments.	Dark Grey	Very Dense	40' 0"		8 BXT R.C. -		CORE RECOVERY = $\frac{8"}{60"} = 13.3\%$	
Either bedrock containing sand layers, or a series of large boulders.	Dark Grey		45' 0"		9 BXT R.C. -		RECOVERY = $\frac{22"}{60"} = 36.7\%$	
Dolomitic, some coarse grained limestone.	Dark Grey		50' 0" 139.6		10 BXT R.C. -		RECOVERY = $\frac{36"}{60"} = 60.0\%$	
Fossiliferous limestone, some shale seams.	Dark Grey							
Fossiliferous limestone, some shale seams. Some very hard limestone.	Light Grey							

HOLE TERMINATED

NOTE: POOR RECOVERY
 DUE IN PART TO GRIND
 OF THE ROCK CORE,
 THE ROCK DOES CONT
 SAND SEAMS.

BOREHOLE LOG

Checked By E.M.P.

ABBREVIATIONS

R. C. ROCK CORE

REFUSAL. PROBABLY BEDROCK

BOREHOLE LOG

Borehole No. 3
Boring Date April 7th - 8th, 1958
Checked By F.M.I.

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

[illegible]

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Highway 101 - G.N.R. Crossing Job No. 571A9B.

Borehole No. 4

Client Dept. of Highways of Ontario Casing EX. 12-1/2" diam.)





Boring Date April 15th - 16th, 1958

Datum Geodetic.

Compiled By W. J. O.

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
Organic silty and sandy loam.			0' 0" 190.8					
Silty fine sand, considerable organic matter.	Mixed Brown	Loose.			1	S.S.	5	W.L. = 2' 0", APR. 16, 1958 P.M. NO CHG. WET & W.T. = 30" MAY 3, 1958
TILL: silty fine sand, grits and pebbles.	Pale Brown	Very dense	5' 7" 184.3		2	S.S.	50/7	MOIST
Fine to medium sand, many grits and pebbles.	Pale Brown, Rust leaching	Dense			3	S.S.	36	
Fine sand, grits and pebbles.	Yellowish-brown.	Very dense	10' 0"		4	S.S.	75	MOIST NAT. M.C. = 16.4%
AS ABOVE, WITH ROCK FRAGMENTS.			14' 0" 176.8					
Silty fine sand, grits and rock fragments.	Grey	Dense			5	S.S.	45	WET NAT. M.C. = 5.0% (ROCK IN M.C. SPECIMEN)
As above.	Grey	Compact	20' 0" 169.3		6	S.S.	28	QUITE MOIST
Fine to medium sand, grits and pebbles, minor silt content.	Grey	Dense	25' 0"		7	S.S.	48	QUITE MOIST
Silty fine sand, grits and pebbles.	Grey	Dense	30' 0"		8	S.S.	32	NAT. M.C. 30' 1" = 6.6% NAT. M.C. 30' 6" = 8.4%
3" seam with minor clay content.								
Silty fine sand, grits and pebbles.	Dark Grey	Very dense	35' 0"		9	S.S.	82/6"	SLIGHTLY MOIST
As above.	Dark Grey	Very dense	40' 0"		10	S.S.	64	
			42' 6" 148.3					

REFUSAL. PROBABLY BEDROCK.





e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Highway 401 - C.C.P. Crossing Job No. 17147B Borehole No. 6
 Client Dept. of Highways of Ontario Casing 12" dia. steel Boring Date April 1966
 Datum Geodetic Compiled By M.S. Checked By M.S.

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
			0' 0"					HOLE SAVED IN ON MAY 3, 1966 BUT W.T. BELIEVED TO BE AT SURFACE.
Organic silty loam.	Black		179.7					
Medium sand.	Yellowish-brown		2' 0"					
Silty very fine sand, grits and pebbles.	Pale brown	Compact	2' 6"		1	S.S.	14	QUITE MOIST
			5' 0"					
Silty fine sand, grits and pebbles, seams of medium sand.	Pale Brown	Dense			2	S.S.	35	MOIST. NAT. M.C.=10.6%
			9' 6"					
Silty fine to medium sand, grits and pebbles.	Gray	Dense			3	S.S.	38	SAMPLE LOST, WASH SAMPLE RETAINED.
			15' 0"					
As above, numerous grits and larger angular rock fragments.	Gray	Compact			4	S.S.	28	WET.
			20' 0"					
As above.	Gray	Very dense			5	S.S.	60/8"	WET. NAT. M.C.=6.8%
			25' 0"					
Silty fine to medium sand, grits and pebbles.	Gray	Very dense			6	S.S.	70/6"	SAMPLE LOST, WASH SAMPLE RETAINED.
			30' 0"					
As above.	Gray	Very dense			7	S.S.	82/6"	WET.
			31' 6"					
			148.2					
								REFUSAL. PROBABLY BEDROCK.

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

Job Name Hwy. 401 - C.N.R. Crossing Job No. 57149D
Client Dept. of Highways of Ontario Casing BX (24" diam.)
Datum Geodetic Compiled By M. H.

Borehole No. 7
Boring Date May 1st 1958
Checked By R. H.

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
Organic silty loam.			0' 0" 177.9					W.L. = 0' 5" MAY 1, 1958 10 MIN.
Nuggetty silty clay, some sand and pebbles.	Mixed Grey-brown	Firm	1' 2" 176.7		1	S.S.	5	AFTER BAILING HOLE TO 12'
Very nuggetty silty clay, some sand content.	Greyish-brown	Firm to Stiff	5' 0"		2	S.S.	9	WETTER THAN PLASTIC LIMIT
Nuggetty silty clay.	Brownish-grey	Soft.	10' 6" 167.4		3	2" S.L.	PUSHED	NAT. M.C. = 23.8% DRIER THAN PLASTIC LIMIT. Hard in the undisturbed state.
As above, stratified with thin seams of fine sand.	Grey	Soft.	12' 6" 165.4		4	S.S.	3	CLAY WETTER THAN LIQUID LIMIT. NAT. M.C. = 30.1%
Sandy and silty clay, many grits and rock fragments.	Grey-brown	Compact.	15' 0"		5	2" S.L.	PUSHED	
Matrix of sandy and clayey silt, with grits and angular rock fragments.	Grey	Compact.	17' 0" 160.9		6	S.S.	30	COULD NOT BAIL HOLE BELOW 12' DUE TO RAPID INGRESS OF WATER.
Clayey and silty very fine sand, grits and pebbles.	Grey	Dense	20' 0"		7	S.S.	18	
			25' 0"		8	S.S.	38	WET. NAT. M.C. = 7.6%
Silty very fine sand, grits and pebbles.	Dark grey	Very dense	29' 1" 148.8		9	S.S.	60/10	QUITE MOIST

REFUSAL. PROBABLY BEDROCK.

BOREHOLE LOG

SAMPLE CONDITION

SAMPLE TYPE

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

Y. 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	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
Organic silty loam.	Black		0' 0" 180.4 179.3					HOLE BAILED TO 28' UPON COMPLETION. WT. AT SURFACE WITHIN ONE HOUR, APR. 22, 1958.
Silty fine sand, grits and rock fragments.	Brown	Compact	5' 0"		1	S.S.	22	WT=04, MAY 3, 1958. MOIST.
As above, many large angular rock fragments.	Grey	Dense			2	S.S.	45	SLIGHTLY MOIST. NAT MC=11.17
Stony till.			10' 0"					DRILLED WITH BX CASING FROM 8 TO 10 FT
Silty fine to medium sand, grits and pebbles.	Gray	Compact			3	S.S.	28	
			15' 0"					
Silty fine to medium sand, many grits and angular rock fragments.	Grey	Very dense			4	S.S.	85	
			20' 0"					
Silty very fine sand, grits and pebbles.	Dark Grey	Very dense			5	S.S.	50/6"	MOIST.
			23' 0" 157.4					
			25' 0"					
Fine to medium sand.	Light Grey	Very dense.			6	S.S.	60/3"	SATURATED.
			30' 0"					
As above.	Light Grey	Very dense.			7	S.S.	65/3"	"
			32' 0" 146.8					
								REFUSAL PROBABLY BEDROCK.

BOREHOLE LOG

Checked By E.M.P.

SOIL DESCRIPTION	COLOR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
			0' 0"					WT. AT SURFACE, APR. 26, 1958.
Organic silty loam.	Black		178.6 1'-0"					HOLE FLOWING OVER VERY SLOWLY, MAY 1, 1958.
Mucky silty clay.	Mottled Grey-Brown	Firm	5' 0"		1	S.S.	7	SLIGHTLY WETTER THAN PLASTIC LIMIT. NAT. M.C. = 50.5%
As above.	As above.	Firm			2	S.S.	6	NAT. M.C. = 52.0%
Silty clay.	Brown	Firm	9' 3" 169.3		3	S.L.		TAPPED DOWN.
Clayey and silty very fine sand, grits and pebbles.	Mixed Brown & Grey	Compact	13' 0" 165.6		4	S.S.	17	MOIST.
Fill: silty fine sand, grits and pebbles.	Light Grey	Dense	15' 0"		5	S.S.	36	MOIST.
Fill, many pebbles and some large rock fragments.	Grey	Dense			6	S.S.	51	MOIST.
			20' 0"		7	S.S.	60/6"	MOIST.
Clayey and silty fine sand, grits and rock fragments.	Grey	Very dense	25' 0"		8	S.S. W.S.	85/6"	NOTE: HOLE COULD NOT BE BAILED DUE TO RAPID INGRESS OF WATER
Silty fine to medium sand, grits and pebbles.	Brownish Grey	Very dense	30' 0"		9	S.S. W.S.	75/7"	
As above.	As above	Very dense	33' 2" 145.4					

VIRTUAL REEVAL POSSIBLY BEDROCK

BOREHOLE LOG

Borehole No. 11
Boring Date April 24th - 25th, 1958
Checked By W.M.O.F.

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

[illegible]

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





Name Highway 401 - G.N.R. Crossing Job No. 571498
ent Dept. of Highways of Ontario Casing BX (2-1/2" diam.)
turn Geodetic Compiled By M.M.

Borehole No. 11A
Boring Date April 25th - 26th, 1958
Checked By E.M.F.

SAMPLE CONDITION

SAMPLE TYPE

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

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

[illegible]

Borehole No. 12
Boring Date April 29-30, 1958.
Checked By E. S. F.

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

REFUSAL	PROBABLY	BEDROCK
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ACREHOLE LOG

Checked By E.H.P.

ABBREVIATIONS

Y.T. IN SITU VANE SHEAR TEST

Q/11 UNCONFINED COMPRESSIVE STRENGTH

W. L. WATER LEVEL IN CASING

W. T. GROUND WATER TABLE IN SOIL

WATER LEVELS, SOIL MOISTURE & REMARKS

FLOWING OVER VERY
WLY, MAY 1, 1958. W.T.=02
Y 3, 1958, BUT HOLE PARTI
CAVED IN.

of the *Staphylococcus aureus* isolates, and the *Staphylococcus aureus* isolates were found to be more sensitive to the antibiotics than the *Staphylococcus aureus* isolates.

217

20

5.5

S.S.

55

5.5

5. 5

REFUSAL PROBABLY BEDROCK

APPENDIX I

LABORATORY TEST RESULTS

95° BOREHOLE CONE PENETRATION TEST RESULTS

Job No. 17149B

REPORT NO. OF BOREHOLE PENETRATION TESTS
 100 ft. diameter dropping 10 inches.

Test No. 111 (ELEV. = 1778 APPROX.)

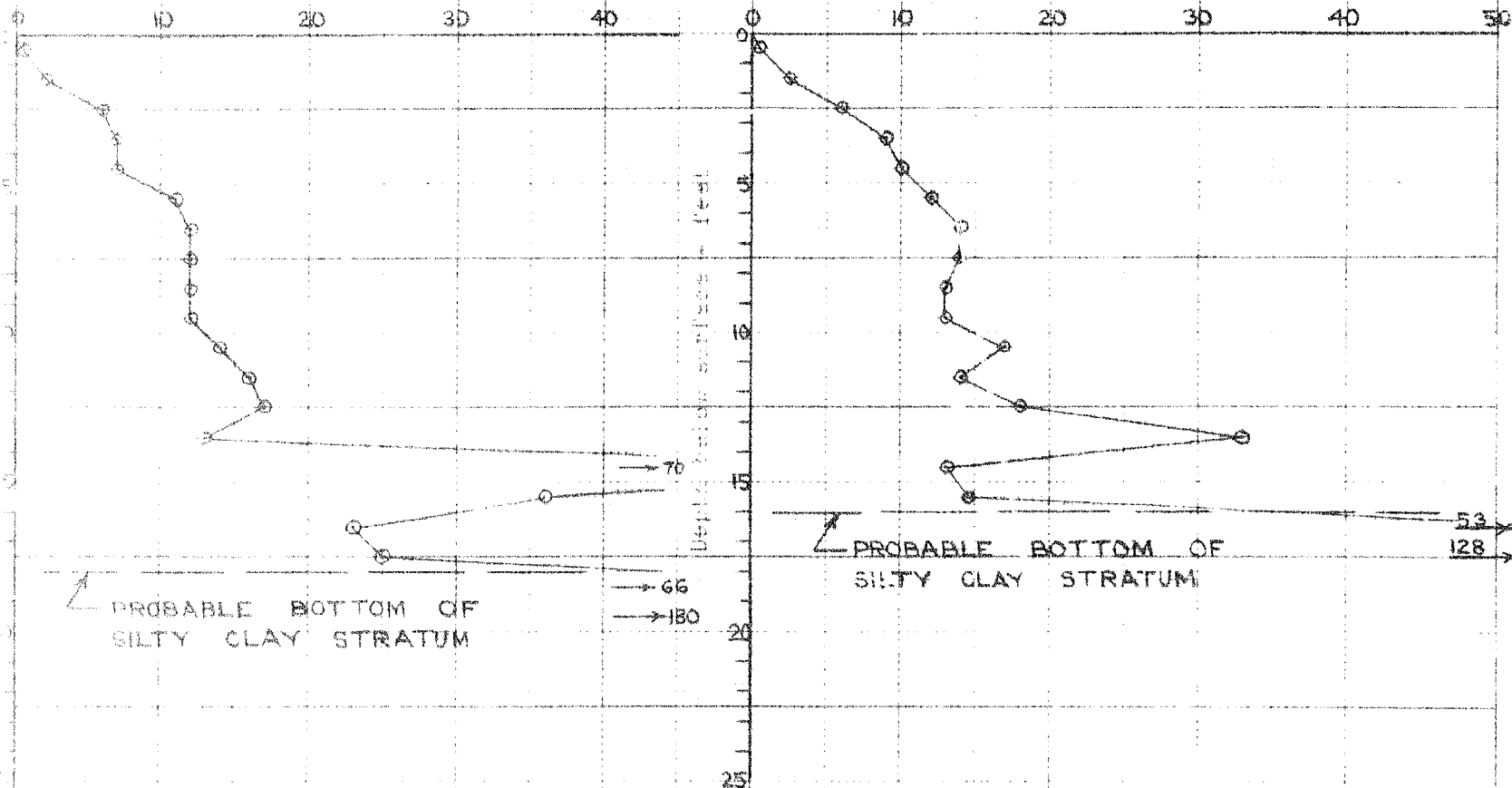
W.T. AT SURFACE, MAY 1, 1958
 W.T. = 0.3", MAY 3, 1958

Test No. 112 (ELEV. = 1779 APPROX.)

W.T. = 0.5", MAY 1, 1958
 W.T. = 0.6", MAY 3, 1958

No. of Blows

No. of Blows



e. m. peto associates ltd.
SOIL TESTING LABORATORY

LIQUID LIMIT TEST

FLOW LINE CHARTS

JOB No. 57149B PROJECT Hwy. 401 - C.N.R. Crossing

SAMPLE FROM _____

SAMPLE FROM Borehole 11, Sta. 4

DEPTH _____

DEPTH 9 1/2' - 10 1/2'

MOISTURE CONTENT %

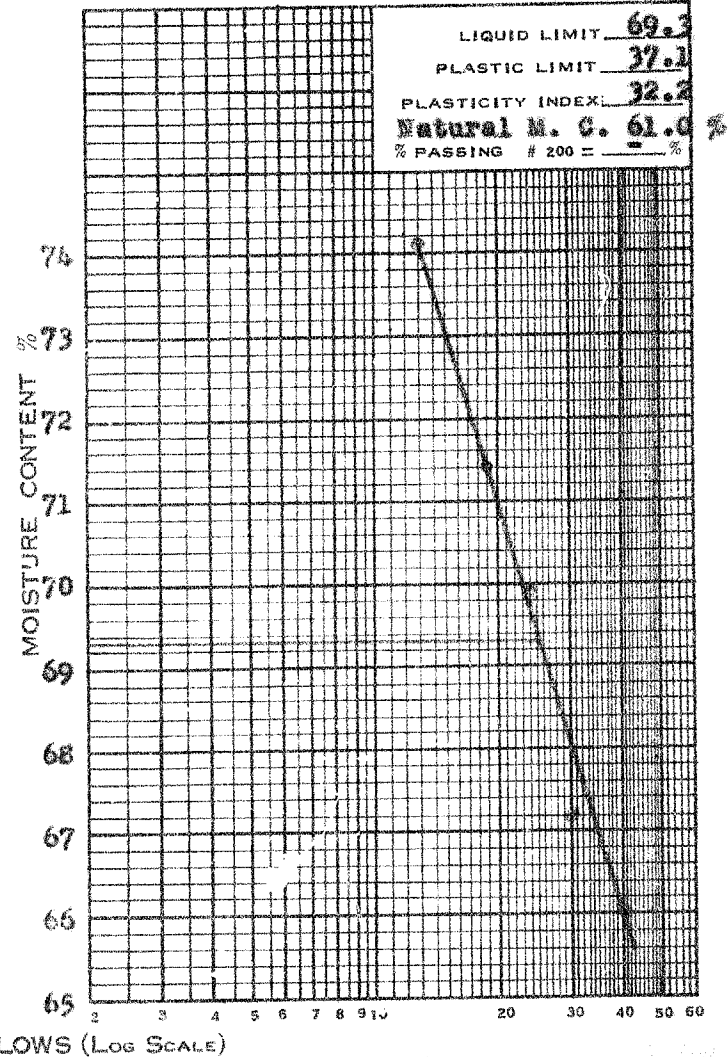
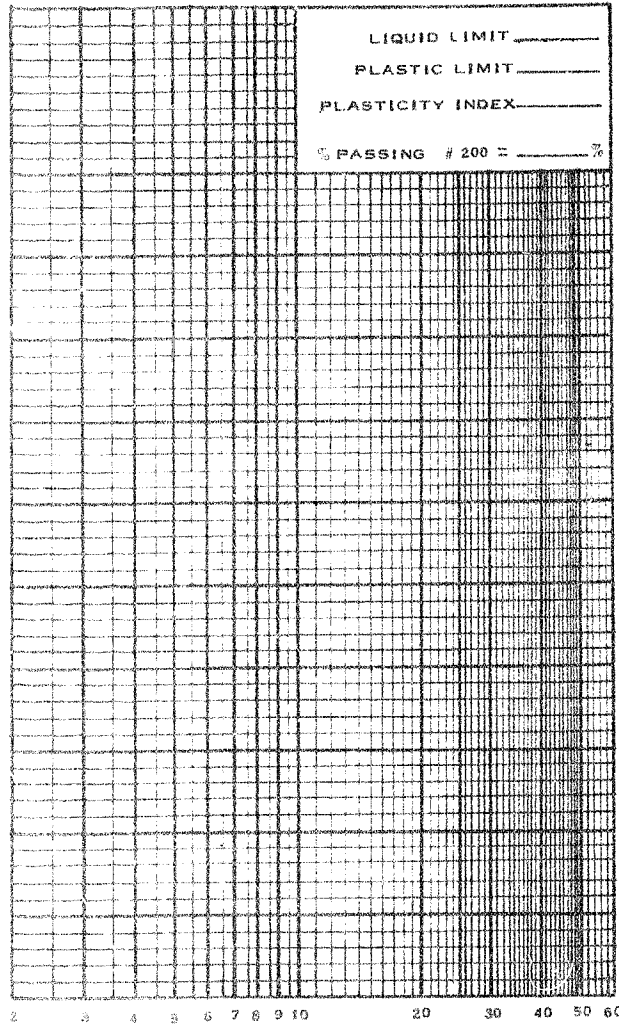
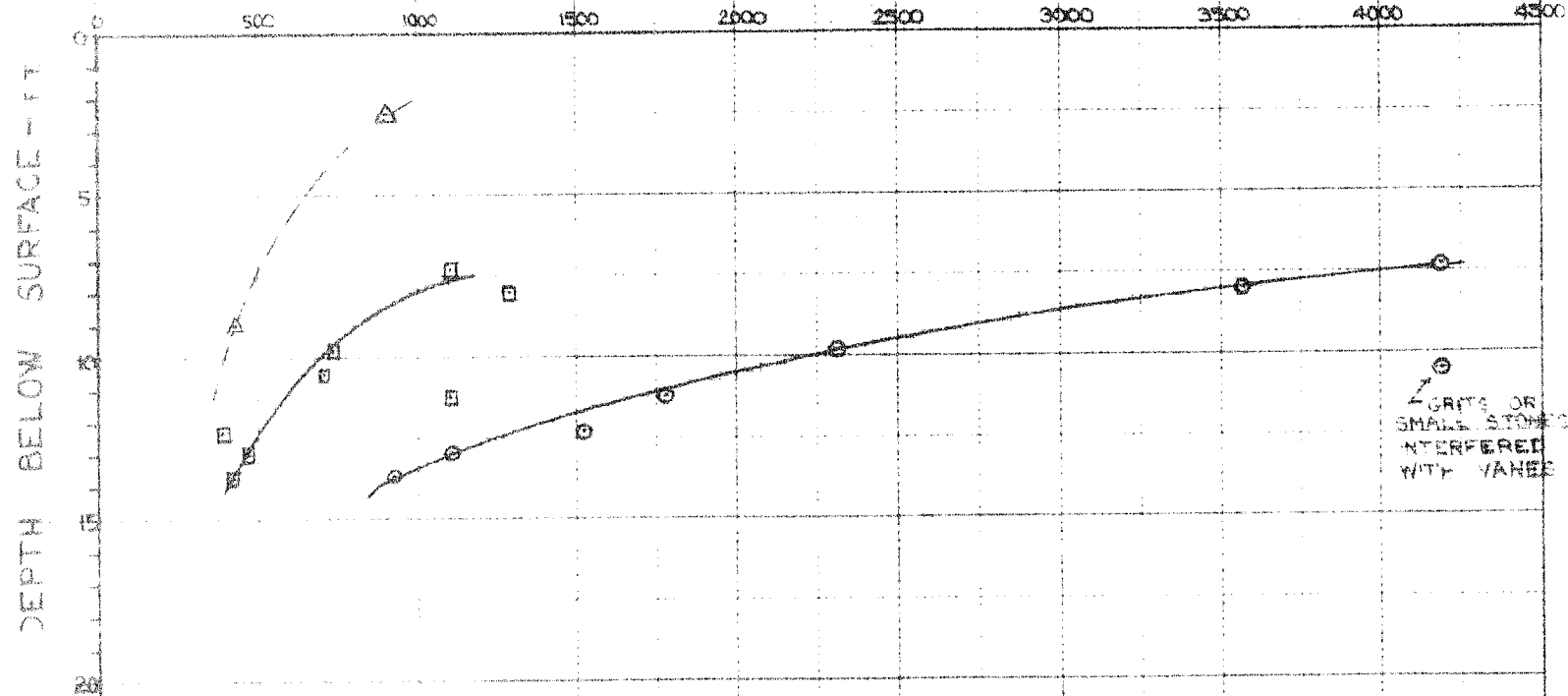


CHART FOR COMPARISON OF IN SITU VANE TESTS AND LABORATORY COMPRESSION TESTS

PLOT OF SOIL SHEAR STRENGTH VERSUS DEPTH

JOB NO 57149B
B.H. II AND B.H. IIA

SOIL SHEAR STRENGTH - P.S.F.



QUICK UNDRAINED TRIAXIAL COMPRESSION TESTS

WITH PORE WATER PRESSURE MEASUREMENTS

JOB NO 571498

BH NO 7

SAMPLE 3,

SPECIMEN NO	DEPTH	DEGREE OF SATURATION %	WET DENSITY lb/cu ft	NAT M.C %
①	9'0" - 9'3"	88.0	97.7	55.3
②	9'3" - 9'6"	88.0	97.7	55.3

σ_{TOTAL} 576 lb/sq ft

$\sigma_{EFFECTIVE}$ 432 lb/sq ft

ϕ_{TOTAL} 8° 30'

$\phi_{EFFECTIVE}$ 34° 30'

SHEAR STRESS PSI

EFFECTIVE STRESS

TOTAL STRESS

NORMAL STRESS PSI

PMA
29-5-58

QUICK UNDRAINED TRIAXIAL COMPRESSION TESTS WITH PORE WATER PRESSURE MEASUREMENTS

JOB N° 571498

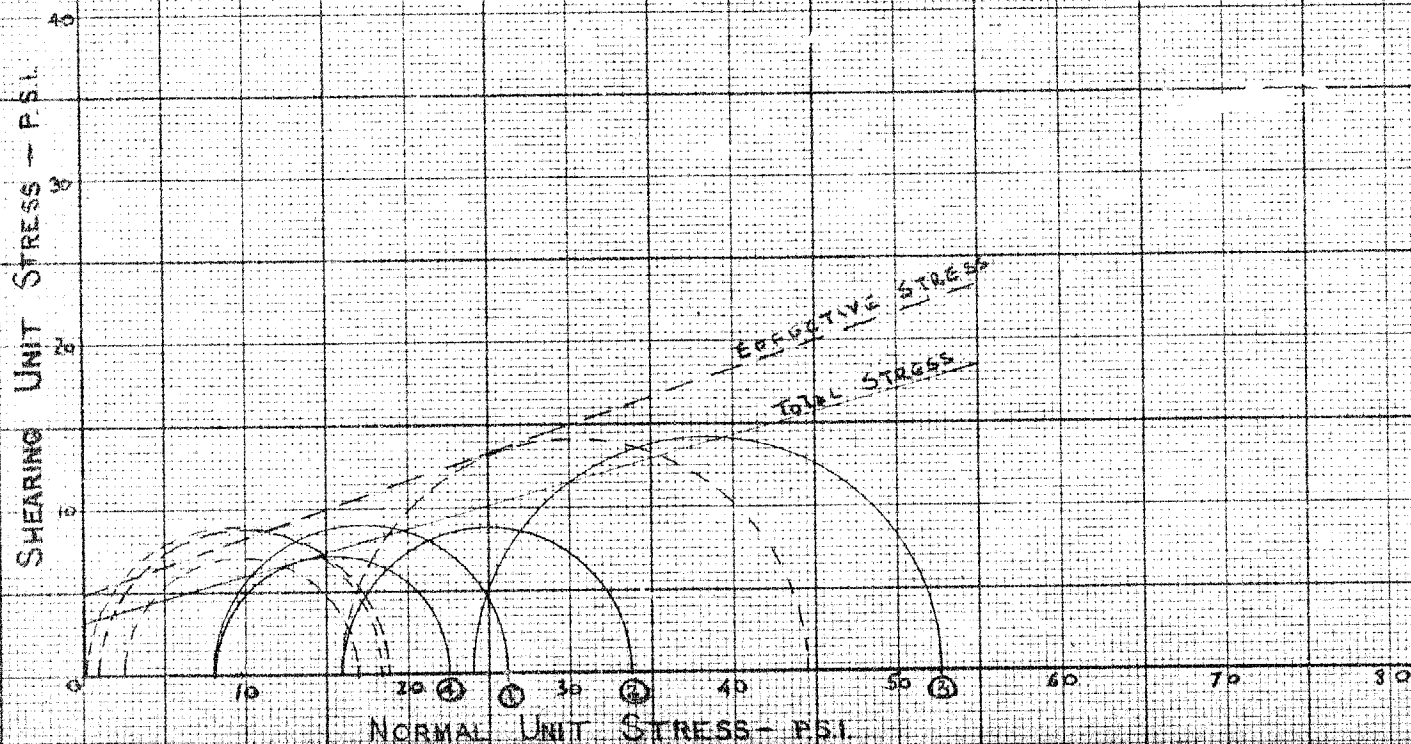
BH N° 11

SAMPLE 38, 3C

SPECIMEN N°	DEPTH	DEGREE OF SATURATION %	WET DENSITY lb/cuft.	NAT. M.C. %
①	8'6" - 8'9"	100.05	105.0	59.4
②	8'9" - 9'0"	100.05	105.0	54.6
③	9'0" - 9'3"	98.60	102.2	61.2
④	9'3" - 9'6"	98.60	102.2	60.3

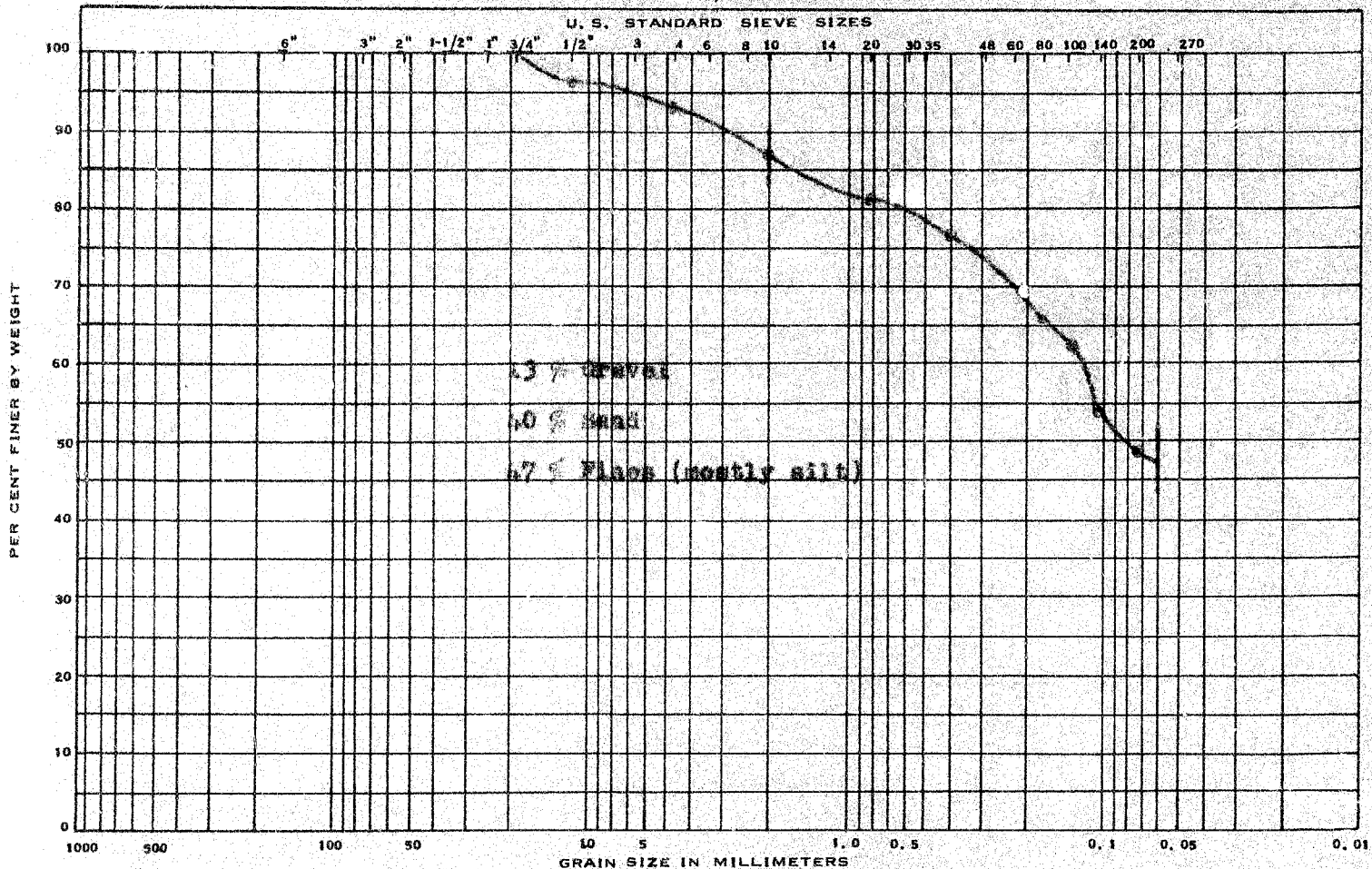
C TOTAL = 432 lb/sq ft C EFFECTIVE = 720 lb/sq ft

ϕ TOTAL = 15° 45' ϕ EFFECTIVE = 18° 40'



e. m. peto associates ltd.

TORONTO, ONTARIO



BOULDERS	STONES	GRAVEL	COARSE SAND	MED. SAND	FINE SAND	COARSE SILT	MED. SILT
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MASS. INST. OF TECH. CLASSIFICATION

JOB NAME Hwy. 401 - C.N.R. Crossing JOB NO. 57149B HOLE NO. 2 SAMPLE NO. 3 and 4
 DEPTH 7' - 12' ELEVATION 174' REMARKS Grading of typical till material at this site.

GRAIN SIZE DISTRIBUTION DIAGRAM
 COARSE MATERIALS

APPENDIX II

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" or 3" 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" 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 disturbed soil.

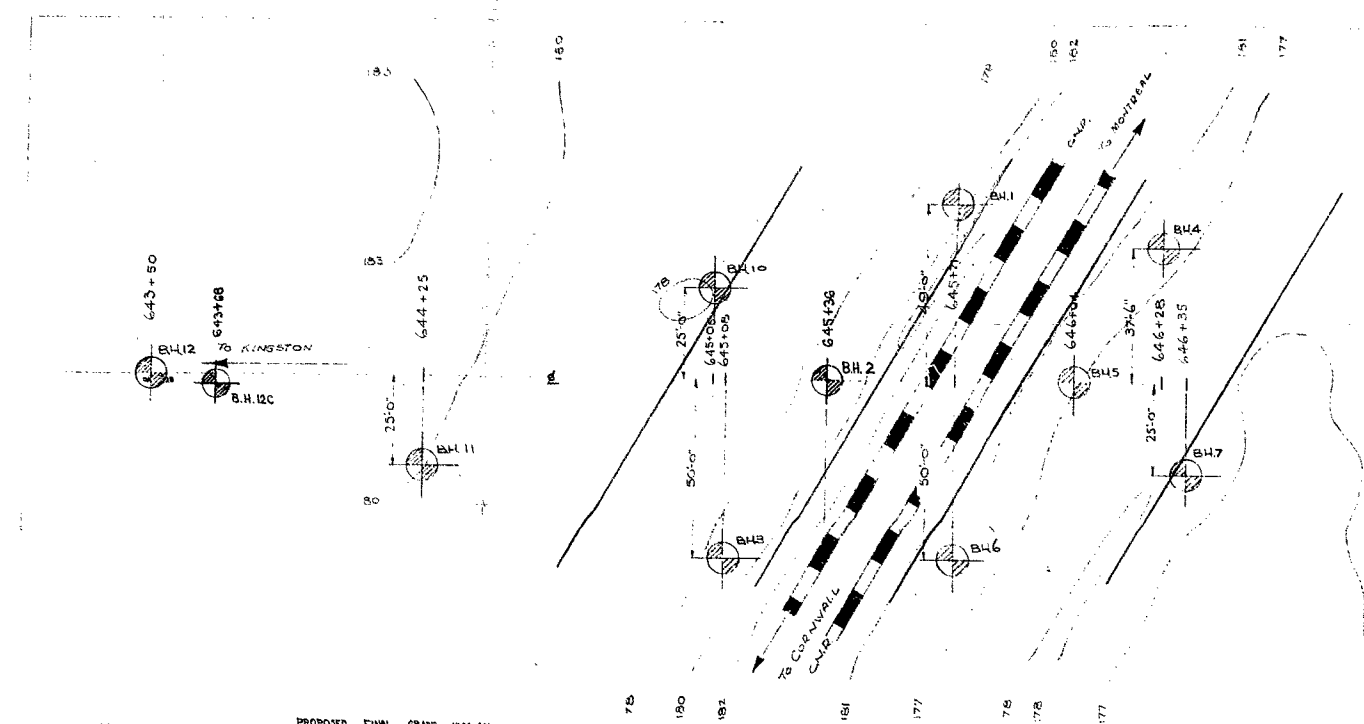
The Dutch cone probe test is made by driving the drill rods into the ground with a 2-1/4" - 30° cone tip. The number of 4200 inch pound blows per foot of penetration are recorded, as in the standard penetration test.

Where required, "in situ" shear strength tests are made ahead of the casing, using modified Aker vane test equipment.

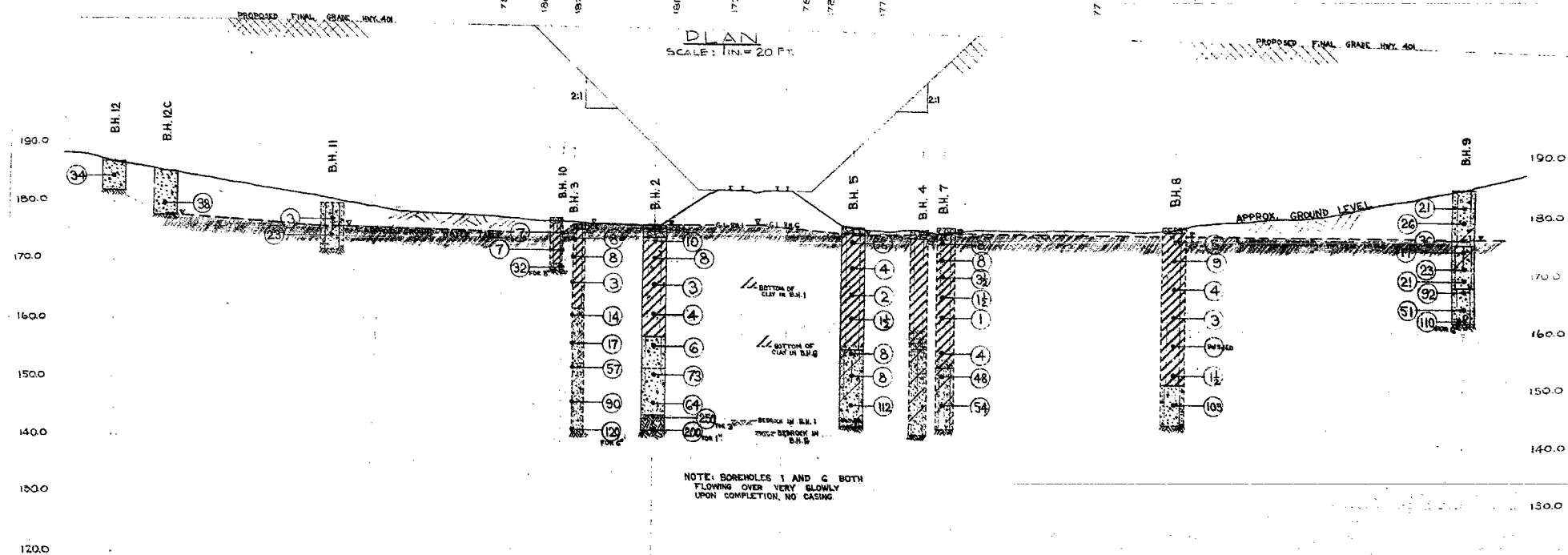
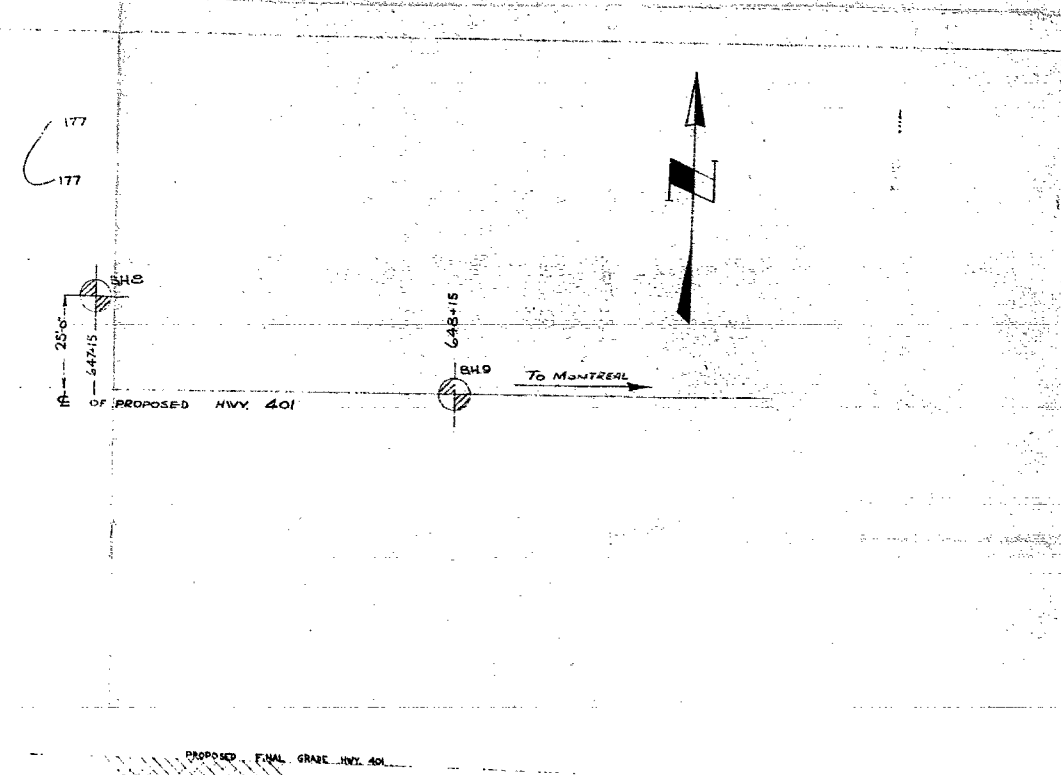
Disturbed samples are visually classified in the field, sealed in sample jars, and are re-examined, and tested as necessary, in the soils laboratory. Undisturbed samples are returned to the laboratory for later examination and testing, as required.

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.

57-F-241C
W.P. 69-57
Hwy. #401
C.N.R. CROSSING



PLAN
SCALE: 1" = 20' F.

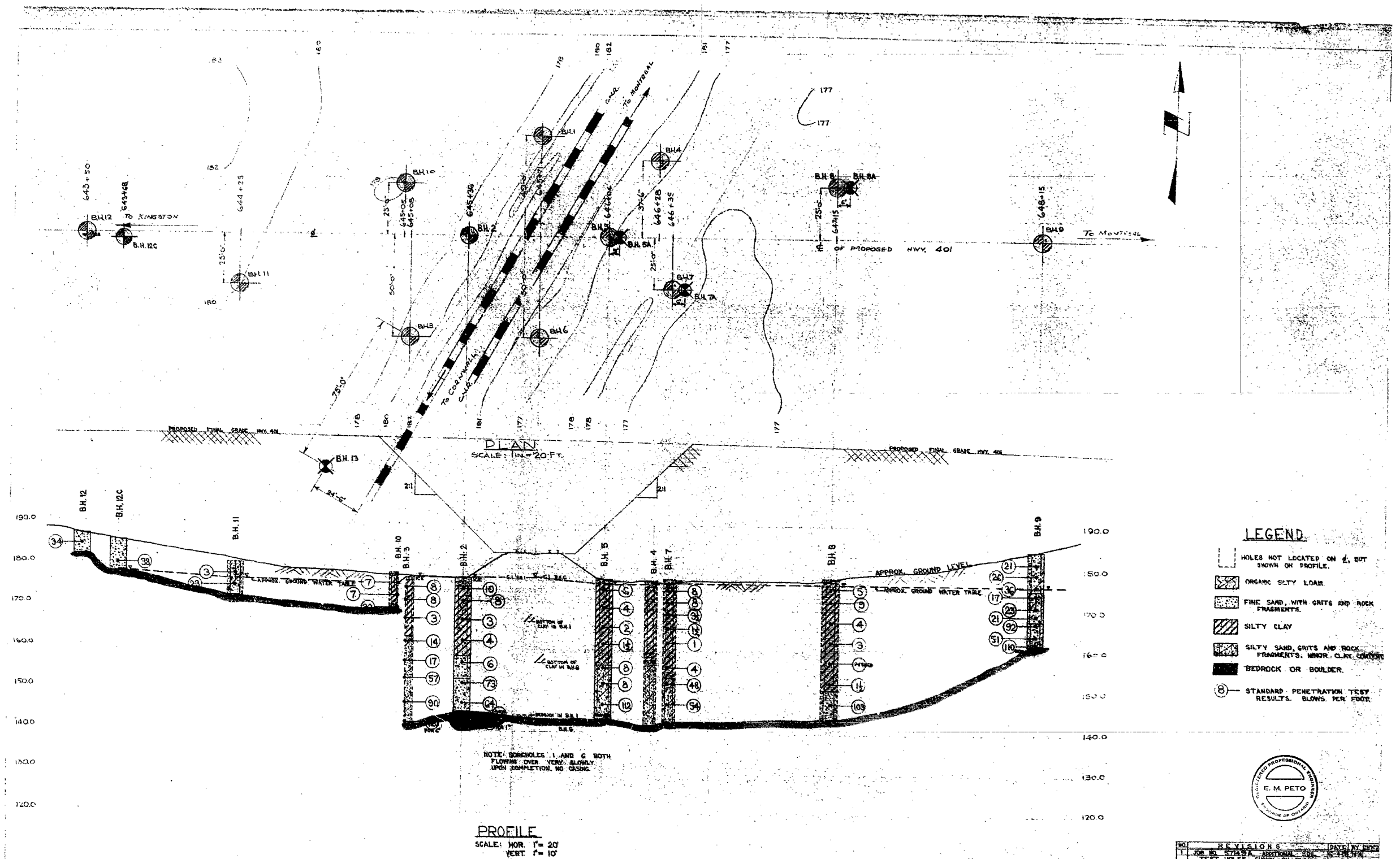


PROFILE
SCALE: HOR. 1" = 20'
VERT. 1" = 10'

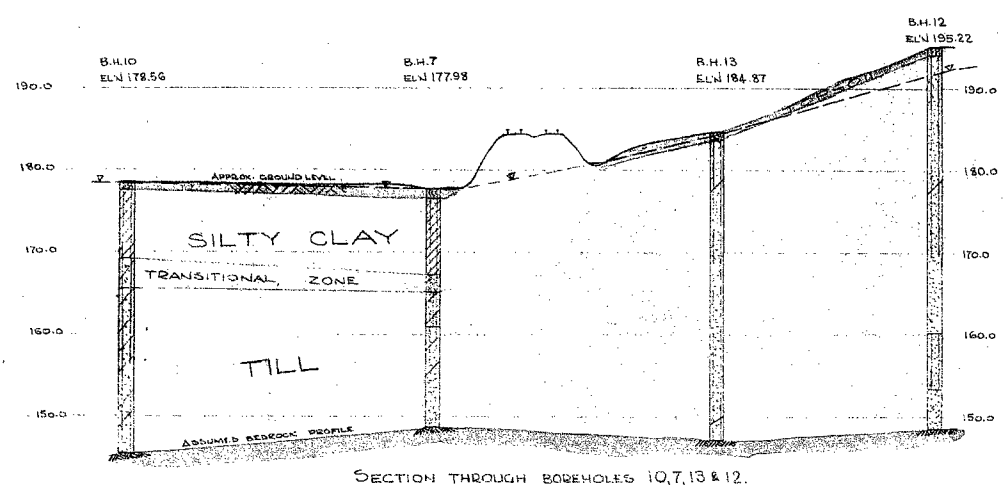
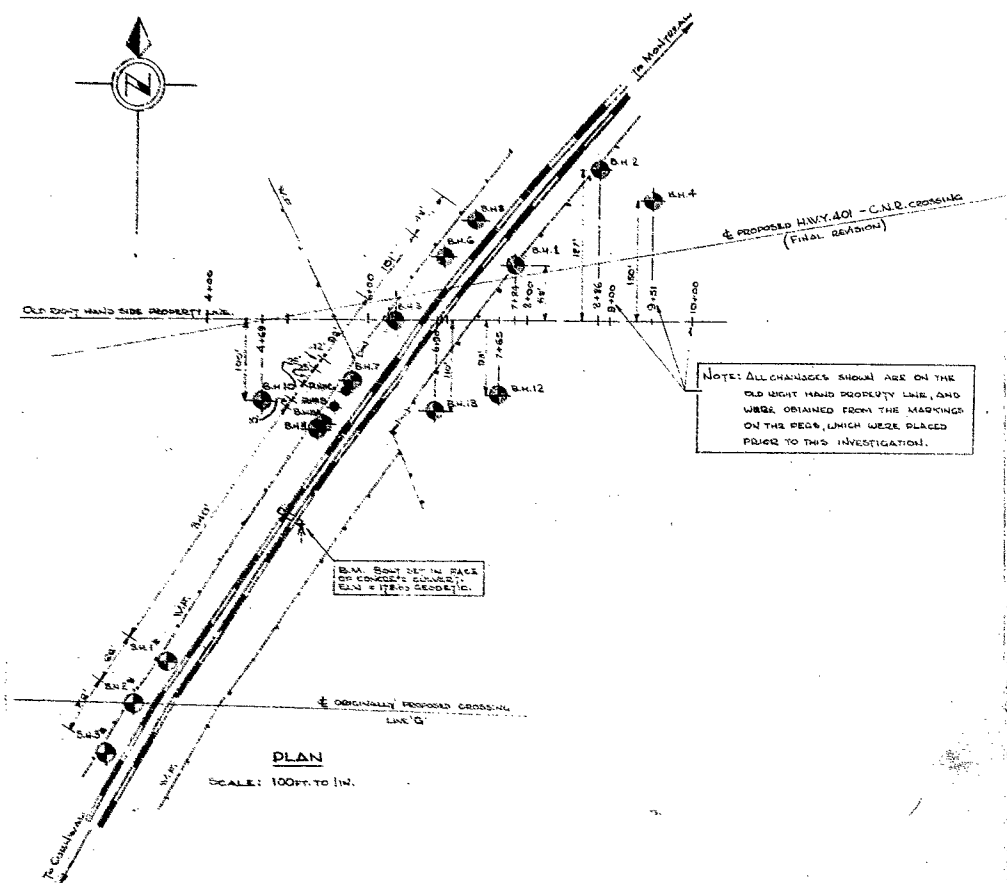
LEGEND

- HOLES NOT LOCATED ON PLAN, BUT SHOWN ON PROFILE.
- ORGANIC SILTY LOAM
- FINE SAND, WITH GRITS AND ROCK FRAGMENTS.
- SILTY CLAY
- SILTY SAND, GRITS AND ROCK FRAGMENTS, MINOR CLAY CONTENT
- BEDROCK OR BOULDER
- 8 - STANDARD PENETRATION TEST RESULTS, BLOWS PER FOOT.

e.m. peto & associates Ltd.
SOIL SITE INVESTIGATION
HWY. 401 - C.N.R. CROSSING
TOWNSHIP OF CHARLOTTENBURG
FOR
DEPT. OF HIGHWAYS OF ONTARIO.
OUR JOB No. 57-148 DATE FEB. 12, 1988
CLIENTS PLAN No. W.D. 69-578

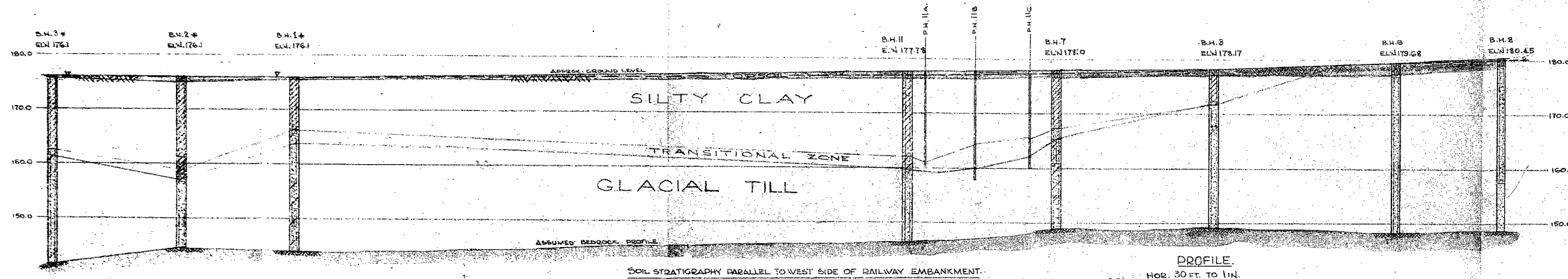


NO.	REVISIONS	DATE	BY
1	JOB NO. 57-148	1984	E.M.P.
2	TEST HOLES SHOWN ON PLAN	1984	E.M.P.
e.m. peto & associates Ltd.			
SOIL SITE INVESTIGATION			
AT			
HWY. 401 - C.N.R. CROSSING			
TOWNSHIP OF CHARLESTOWN			
FOR			
DEPT. of HIGHWAYS 5/1 ONTARIO			
OUR JOB No. 57-148			
CLIENTS PLAN No. W.P. 69-57 Bridge W.P. 69-57			



- LEGEND**
- B.H.* — HOLE DOWN IN PREVIOUS INVESTIGATION AT LINE "G".
 - PROBE HOLE WITH DUTCH CONE.
 - ORGANIC SILTY LOAM.
 - SILTY CLAY.
 - TRANSITIONAL ZONE: SANDY & SILTY CLAY WITH GRITS.
 - TILL: SILTY FINE SAND, GRITS & PEBBLES.
 - FINE TO COARSE SAND, GRITS.
 - LIMESTONE BEDROCK.
 - POSITION OF GROUND WATER TABLE ON MAY 3, 1958.

NOTE: THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. A LINEAR VARIATION IN SOIL STRATIGRAPHY HAS BEEN ASSUMED BETWEEN BOREHOLES, AND THIS MAY ACTUALLY DIFFER FROM THAT SHOWN.



e.m. peto & associates Ltd.
 SOIL SITE INVESTIGATION
 AT
HWY. 40 - C.N.R. CROSSING
 W69-57 TWP. of CHARLOTTENBURG
 FOR
 DEPARTMENT of HIGHWAYS of ONTARIO
 OUR JOB No. 57-1498 DATE MAY 15-58
 CLIENTS PLAN No. F 3165-9 (REVISED) PER G.T.