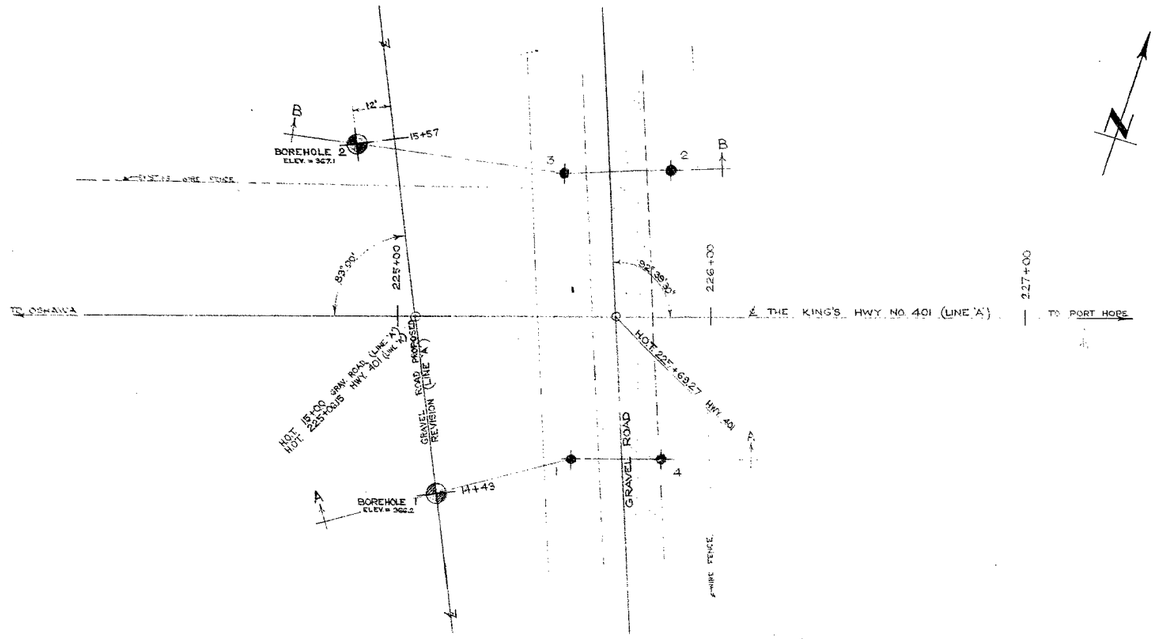
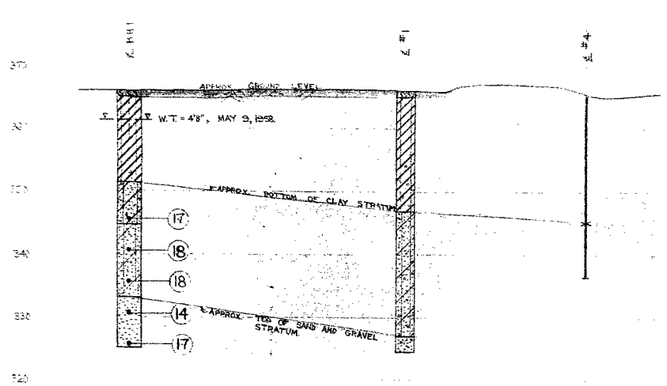
#58-F-232-C
W.P. 86-57
HWY. #401
HOPE TWP. #8

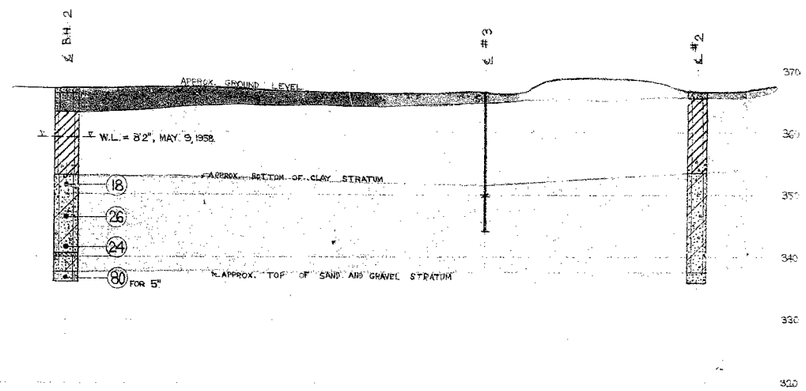


PLAN

SCALE: 1" = 20'



SECTION A-A



SECTION B-B

PROFILES

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

LEGEND

- HOLES DRIVEN MAY, 1957 BY E.M.P. SOILS CREW
- ORGANIC SILTY AND SANDY LOAM TOPSOIL
- VERY PALE BROWN SILTY CLAY
- CLAYEY AND SILTY FINE SAND, MANY GRITS AND PEBBLES.
- FINE TO COARSE SAND, SOME GRAVEL
- (18) STANDARD PENETRATION TEST RESISTANCE, BLOWS PER FOOT.

REGISTERED PROFESSIONAL ENGINEER
E. M. PETO
PROVINCE OF ONTARIO

e.m. peto & associates ltd.

SOIL SITE INVESTIGATION
AT
HOPE TOWNSHIP BRIDGE #8
HWY. 401 - GRAVEL ROAD UNDERPASS
FOR
DEPT OF HIGHWAYS OF ONTARIO

OUR JOB No.- 5850 DATE: MAY 16, 1958.
CLIENTS PLAN No.: F-57-15A PER: [illegible]

DEPARTMENT OF HIGHWAYS OF ONTARIO

HIGHWAY 401 - GRAVEL ROAD UNDERPASS
W. P. 86 - 57 HOPE TWP. BRIDGE NO. 8

SOILS REPORT

by

E. M. PETO ASSOCIATES LTD.

TORONTO, ONTARIO

May, 1958

AN INDEPENDENT ORGANIZATION PROVIDING A COMPLETE SOIL ENGINEERING & INVESTIGATION SERVICE

e. m. peto associates ltd.

YOUR REFERENCE:-

OUR REFERENCE:- 5850

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

May 22/1958.

Mr. A.M. Toye,
Chief Bridge Engineer,
Dept. of Highways of Ontario,
280 Davenport Rd.,
Toronto, Ont.

Attention: Mr. J. C. McAllister

Re: Supplementary Site Investigation
Highway 401 - Gravel Road Underpass
W.P. 80-57 Hope Twp. Bridge No. 8

Dear Sirs,

We are enclosing herewith three (3) copies of our report on this site for your attention. In accordance with Mr. McAllister's verbal request, one copy of this report has been forwarded directly to the Consulting Engineer, P. A. Benn & Associates, in order that they might expedite their design.

We were verbally instructed to carry out this investigation to confirm the continuity of soil stratification, which had been previously determined by the Dept. of Highways field personnel at the existing gravel road location some 70 feet to the East of the present site. No special testing was envisaged and speed was of the essence in completing the investigation and report.

We have considered the site conditions in detail in the soils report attached hereto together with supporting appendices. Here for your convenience is a summary of our findings and recommendations:

1. The soil conditions on this site consist of:

   a) A surface layer of approximately one foot of top soil followed by a three foot thick layer of loose fine sand at test hole 2 only.

   b) A stratum of pale brown, very silty clay between 10 and 14 ft. thick, with an average cohesion or shear strength value of the order of 1500 p.s.f.

c) A stratum some 15 1/2 to 20 feet thick of unsorted glacial material classified as a sandy loam with stones. This stratum is saturated and is only compact, with standard penetration test results averaging only 19 blows.

d) A heterogeneous sand and gravel bed below the sandy loam, of variable densities, very dense at borehole 2, but only compact at hole 1.

2. The water table on completion of the field work in May appeared to be approximately 5 feet below ground surface. No seepage water was noted in the test holes until the very silty clay stratum was completely penetrated, whereupon an appreciable ingress of water was noted, at approximately the 15 foot depth at hole 1.

3. Review of the soil conditions at this site leads us to suggest the following types of foundations for your consideration:

a) Spread footings placed at approximately the 3 foot depth, at elevation 358.0, with an allowable bearing value of 2500 p.s.f. Some settlement of the foundation may be anticipated with footings placed at this depth.

b) Pile foundations;

i) Franki compressed concrete piles based approximately at elevation 350.0, providing some very minor settlements can be tolerated. Pile settlements may be negligible if low pile load values are used.

ii) Monotube piles driven into the sandy loam stratum. There will be considerable variation in the length of these piles. Some of the piles will be virtually end-bearing if driven to the gravel bed, and some may be principally friction piles.

iii) Steel H - piles driven to virtual refusal. Some of these piles might terminate near the 30 foot depth, while at some other locations, as at test hole 1, steel H - piles may penetrate below the 42 foot depth.

c) Spread footings founded on the upper surface of the sandy loam stratum, at about elevation 351.0. The safe allowable bearing capacity for a settlement not greater than 1 inch, for a footing 7 1/2 feet wide, is 1900 p.s.f. The safe allowable bearing value, with a safety factor of 3 against shear failure, is 3000 p.s.f.; the maximum total settlement would then become as much as 1.6 inches. The differential settlement is, of course, some fraction of the

theoretical total. Ingress of water into the excavation can be expected at this depth; sufficient head of water existed at the test hole locations to suggest that quicking might occur in the bottom of the excavation at elevation 351.0

Should you require any additional information or advice with regard to this report, we shall be pleased to be of further service.

Yours very truly,

E. M. PETO ASSOCIATES LTD.

E. M. Peto, P. Eng.

EMP/

e. m. peto associates ltd., 850 roselawn avenue, Toronto 10, Ontario

Job No. 5850 Client's Ref. No. Date May 21st, 1958.

Report on

SOIL SITE INVESTIGATION

at

HIGHWAY 401 - GRAVEL ROAD UNDERPASS

W. P. 86-57 HOPE TOWNSHIP BRIDGE NO. 8

for

DEPARTMENT OF HIGHWAYS OF ONTARIO

INTRODUCTION:

We were instructed to carry out a brief investigation of the above site by Mr. J. C. McAllister, acting on behalf of Mr. A. M. Toye, Chief Bridge Engineer, under cover of his letter dated May 5th, 1958. Our investigation was to confirm and amplify the findings of a D. H. O. report at the existing gravel road some 70 feet to the East.

PROGRAMME OF WORK:

May 6th, 1958: Equipment trucked from Belleville to site.

May 7th, 1958: Test holes staked out, field elevations taken, field work commenced.

May 8th, 1958: Finished driving the second test hole.

May 9th, 1958: Final check of ground water levels, equipment loaded and returned to Toronto.

GENERAL INFORMATION:

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

2. The two soil test holes were not driven to bedrock, but were terminated in the dense sand and gravel stratum, as per verbal instructions from the Client.

GENERAL INFORMATION: (Cont'd)

3. The ground water data at test hole 1 is reliable; the measurement at test hole 2 was made shortly after the casing was removed from the ground. We believe that the true ground water level at test hole 2 is higher than that indicated on our borehole log and site plan.

4. Detailed individual borehole logs and an appendix of laboratory test results are included, together with a site plan and two profiles. The positions of our two test holes, the two D. H. O. test holes, and the two D. H. O. dutch cone probe holes are shown on the site plan.

5. All elevations in this report are to Geodetic datum, and are referred to a D. H. O. bench mark in the form of a nail and washer in the North root of an 18" elm 282. 0' right of station 225 + 97.

6. The soil samples obtained during the course of this investigation will be retained for a period of at least 30 days from issuance of this report, and will thereafter be discarded unless we are otherwise notified.

SITE AND GEOLOGY:

The topography at the site is gently undulating, and the surface drainage is fair.

The soils occupying this site are of glacial origin, and are calcareous in nature. The glacial drift is relatively deep over the limestone bedrock.

SOIL CONDITIONS:

Soil conditions on this site are uniform, although the soil strata tend to dip gently to the East.

Sandy and Silty Loam

The site is overlain by approximately one foot of topsoil, an organic sandy and silty loam.

Beneath the topsoil at borehole 2 only, a three foot thick stratum of loose fine sand was encountered.

SOIL CONDITIONS:

Very Silty Clay

Beneath the surface deposits is a 10 ft. to 14 ft. thick stratum of pale brown, very silty clay. This material is firm to stiff in consistency. The shear strengths as calculated from the unconfined compressive strength tests range from 500 p.s.f. to 2,725 p.s.f.; corresponding shear strengths from quick triaxial tests range from 526 p.s.f. to 2,448 p.s.f. A conservative average shear strength value for this silty clay is 1500 p.s.f. The internal friction is negligible.

Five Atterberg Limit tests gave values of 22.6 - 36.9, 13.9 - 20.5, and 8.7 - 20.1, for the liquid limit, plastic limit and plasticity index respectively. The natural moisture contents of the silty clay range from 12.1% to 30.3%, with 23.3% being the average value. This is in excess of the plastic limit. The silty clay is virtually saturated, with degrees of saturation ranging from 89.5% to 97.0% to 107.5%.

Although no consolidation tests were performed on the silty clay, we have calculated the approximate compressive index on the basis of the Atterberg Limit tests, and found this clay to be of low to medium compressibility. The average compressive index is in the order of 0.191.

The wet densities of the silty clay stratum range from 121.6 p.c.f. to 130.3 p.c.f., with 125 p.c.f. being a good average value.

Sandy Loam with Stones

Below the silty clay stratum is a layer, some 15-1/2 to 20 ft. thick, of unsorted glacial material, which could be termed a sandy loam. In actuality, this stratum consists of a pale brown to pale grey-brown, clayey and silty fine sand with many grits and pebbles. Immediately beneath the clay at borehole 1, the sand, grits and pebbles are in a matrix of silty clay.

The sandy loam stratum is generally only compact in density, and is wet to saturated. There are occasional thin horizontal seams of medium sand.

The wet density of the sandy loam in its present saturated state is 143.4 p.c.f. However, since this material is below the water table, only the submerged unit weight of 81.0 p.c.f. is effective and should be used in any design calculations.

SOIL CONDITIONS: (Cont'd)

Sand and Gravel

Our test test holes, and the two D. H. O. soil test holes were all terminated in a heterogeneous, light brown to grey, sand and gravel stratum. This stratum was found to be dense at three of the test holes, but at borehole 1 only a compact fine to medium sand with no stones was encountered.

There is a possibility that the sand and gravel stratum is only of limited thickness, and that the soil reverts to a somewhat denser sandy till at depth.

Apart from the anomaly at borehole 1, any piles driven on this site would certainly encounter refusal in the sand and gravel stratum if not before.

WATER CONDITIONS:

Despite the lack of long term ground water observations, we believe that the ground water table on this site at the time of our investigation was at elevation 361, approximately 5 feet below existing grade.

No free water was noted in the test holes until the silty clay stratum had been completely penetrated. The bottom of the silty clay is roughly between elevations 350 to 353, some 8 to 11 feet below the apparent free ground water table.

There is every indication that any excavation through the silty clay stratum will encounter a flow of water from the sandy loam below. The results at borehole 1, where an appreciable ingress of water into the casing was noted from the 15 foot depth, and the water level in the hole rose 6 feet above the water-bearing stratum in a few minutes, strongly suggest that any excavation in the sand stratum in this area would become quick. This potential condition is also noted at D. H. O. boring number 1 at the 35 foot depth, where the D. H. O. field crew reported "sand backed up the casing".

WATER CONDITIONS: (Cont'd)

With borehole 2 driven to the 31' depth, and cased to 30', our field crew was unable to bail lower than the 17 foot depth due to ingress of water from the bottom of the casing. Within 10 minutes of bailing, the water level at hole 2 had risen to 8 feet below surface.

ENGINEERING CONSIDERATIONS:

1. For a structure which could tolerate some settlement, it should be possible to use large spread footings founded in the silty clay stratum at elevation 358.0 roughly 9 feet below present grade.

   Using the previously selected average cohesion value for the silty clay of 1500 p.s.f., the ultimate bearing capacity for relatively shallow footings having a long rectangular shape is 5.70 x C = 5.70 x 1500 = 8550 p.s.f.

   The maximum shear stress which would be induced in the subsoil at the face of the abutment and at a depth of 9 feet due to the 22 foot high gravel road fill, has been calculated to be 665 p.s.f. This is in accordance with presently accepted soil theory ("Theoretical Problems in Soil Mechanics" - Reynolds and Protopapadakis; Crosby Lockwood and Son, 1948).

   The soil shear strengthavailable to carry the bridge load is then 8550 - 665 = 7885 p.s.f.

   Therefore the safe allowable bearing capacity for the bridge footings (S.F. = 3) is $\frac{7885}{3}$ = 2628 p.s.f.

   If we further take into account possible seasonal fluctuations in water table which may tend to increase the effective soil stresses, then the safe allowable bearing capacity used should be 2500 p.s.f.

2. The approximate compressive index of the silty clay material based on the Atterberg limit determinations, is 0.19, indicating a soil of low to medium compressibility. The silty clay stratum below elevation 358.0 varies in thickness from 5 to 7-1/2 feet. Based only on the approximate compressive index, the maximum theoretical differential settlement is of the order of 2-1/4".

ENGINEERING CONSIDERATIONS: (cont'd)

3. Consideration may be given to placing the bridge footings below the silty clay stratum at approximately elevation 351.0. The 15 to 20 foot thick layer of saturated sandy loam below elevation 351.00 gave average standard penetration test results of 19 blows per foot. Based on the water conditions encountered at our two test holes, this sandy loam is considered to be completely beneath the water table.

   The safe allowable bearing capacity at which the safety factor against breaking into the ground, or shear failure, is 3, is 3000 p.s.f.

   The safe allowable bearing capacity for a settlement not greater than 1 inch (bearing in mind the water table), for a footing 7-1/2 ft. wide is 1900 p.s.f. If a bearing value of 3000 p.s.f. is used at elevation 351.0, the maximum total settlement would be of the order of 1.0 inches; the differential settlement would be some fraction of this theoretical total settlement.

4. The placement of large footings at elevation 351.0 will involve some construction difficulties with water and potential quicking of the bottom of the excavation. There is no indication that water would be a problem in shallow excavations to elevation 358.0.

5. If a rigid frame structure is contemplated, or if settlement cannot be tolerated, pile foundations must be considered for this site. The four test holes and two Dutch cone penetration tests carried out on the two adjacent sites provide clear evidence of a considerable variation in the density of the till at depth.

   a) Franki Compressed concrete piles might be driven to shallow depths and based on the sandy loam stratum. However, if the normal Franki pile load values are used, some minor settlement might be anticipated in view of the relatively low field penetration test results obtained in this stratum at the new bridge location.

   b) Monotube or displacement piles will tend to refuse at variable depths in the sandy loam stratum, in some cases within a few feet of penetration of the top surface of this stratum. Virtually complete refusal would be encountered at the 30 ft. depth in the area of borehole 2, but no such dense stratum was recorded at borehole 1 to termination of the hole. Monotube piles would tend to obtain their bearing values largely from friction at some locations on this site.

ENGINEERING CONSIDERATIONS: (Cont'd)

5.

c) Steel H-piles could be driven most easily on this site, but it is impossible from the limited information available, to determine the ultimate refusal depth.

E. M. PETO ASSOCIATES LTD.,

E. M. Peto, P. Eng.

MM:sb

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name: Port Hope Twp. Bridge No. 8 | Job No.: 5850 | Borehole No.: 1

Client: Dept. of Highways of Ontario | Casing: BX (2-1/2" diam.) | Boring Date: May 7th - 8th, 1958.

Datum: Geodetic. | Compiled By: M.M. | 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
			0' 0"					
Organic sandy loam.			366.2					
Clayey silt.	Lt. Brown	Firm to Stiff.			1	S.S.	10	DRIER THAN PLASTIC LIMIT
			5' 0"					▽ W.T. = 4'8". MAY 9, 1958.
Clayey silt.	Very Pale Brown	Stiff			2	S.S.	19	NAT. M.C. = 22.1%
Very clayey silt.	As above.	Stiff.			3	S.S.	17	NAT. M.C. = 24.2%. WETTER THAN PLASTIC LIMIT.
			10' 0"					
Very silty clay, some 1/4" layers of clayey silt.	Very Pale Brown.	Firm.			4	S.S.	10	NAT. M.C. = 30.3% MUCH WETTER THAN PLASTIC LIMIT
As above, silt seams at 1" intervals.	Very Pale Brown	Firm.			5 A, B, C	S.L. 2"	PUSHED	NAT. M.C. = 25.3%
As above, some stones.			14' 9" 351.4		6	S.S.	10	
Matrix of Silty clay, with sand, many grits and pebbles.	Pale Brown							
			20' 0"					
Sandy and silty clay, many grits and pebbles.	Pale Brown	Compact	21' 6" 344.7		7	S.S.	17	NAT. M.C. = 14.0% SATURATED
Silty fine sand.	Light Brown				8	W.S.	—	
			25' 0"					
Clayey and Silty Fine Sand, many grits and pebbles.	Pale Brown	Compact			9	S.S.	18	SATURATED.
			30' 0"					
As above.	Pale Brown	Compact			10	S.S.	18	NAT. M.C. = 11.0% SATURATED.
			33' 0" 333.2					
			35' 0"					
Fine to medium sand.	Light Brown	Compact			11	S.S.	14	SATURATED.
As above.	Light Brown	Compact	41' 0" 325.2		12	S.S.	17	

HOLE TERMINATED

FOR TRIAXIAL TEST RESULTS SEE ATTACHED GRAPHS.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Port Hope Twp. Bridge No. 8 Job No. 5856 Borehole No. 2

Client Dept. of Highways of Ontario Casing BX (2-1/2" diam.) Boring Date May 8th - 9th, 1958.

Datum Geodetic. Compiled By [illegible] Checked By [illegible]

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" 367.1					
Organic sandy loam.								
Fine sand, minor silt content, some organic matter.	Brown	Loose.	4' 0" 363.1		1	S.S.	9	MOIST.
Very silty clay.	Pale Brown	Stiff			2	S.S.	14	NAT. M.C. = 22.4%. SLIGHTLY WETTER THAN PLASTIC LIMIT. Q_u = 2025 P.S.F.
Very silty clay.	Pale Brown	Stiff			3	S.S.	18	W.T. = 8'2" MAY 9, 1958. AFTER PULLING CASING.
			10' 0"					
As above	Pale Brown	Stiff			4a	2" S.L.	PUSHED	
As above, some layers of clayey silt.	as above	Firm			5	S.S.	7	NAT. M.C. = 26.5 %. MUCH WETTER THAN PLASTIC LIMIT.
As above, scattered grits.	Pale Brown	Firm	14' 0" 353.1		6	S.S.	8	NAT. M.C. = 23.6%. Q_u = APPROX. 1000 P.S.F.
Clayey and silty fine sand, grits and rock fragments.	Pale Grey-Brown.	Compact			7	S.S.	18	WET.
			20' 0"					
As above, layer of medium sand.	As above	Compact			8	S.S.	26	WET.
			25' 0"					
Clayey and silty fine to medium sand, odd grits.	Pale Grey-Brown.	Compact	26' 6"		9	S.S.	24	SATURATED.
			29' 6" 337.6					
Silty fine to coarse sand, grits & pebbles.	Grey	Very dense	31' 0" 336.1		10	S.S.	80/5"	SAMPLE LOST, WASH SAMPLE RETAINED.
			HOLE TERMINATED.					FOR TRIAXIAL TEST RESULTS SEE ACCOMPANYING GRAPHS.

APPENDIX 1

LABORATORY TEST RESULTS

e. m. peto associates ltd.

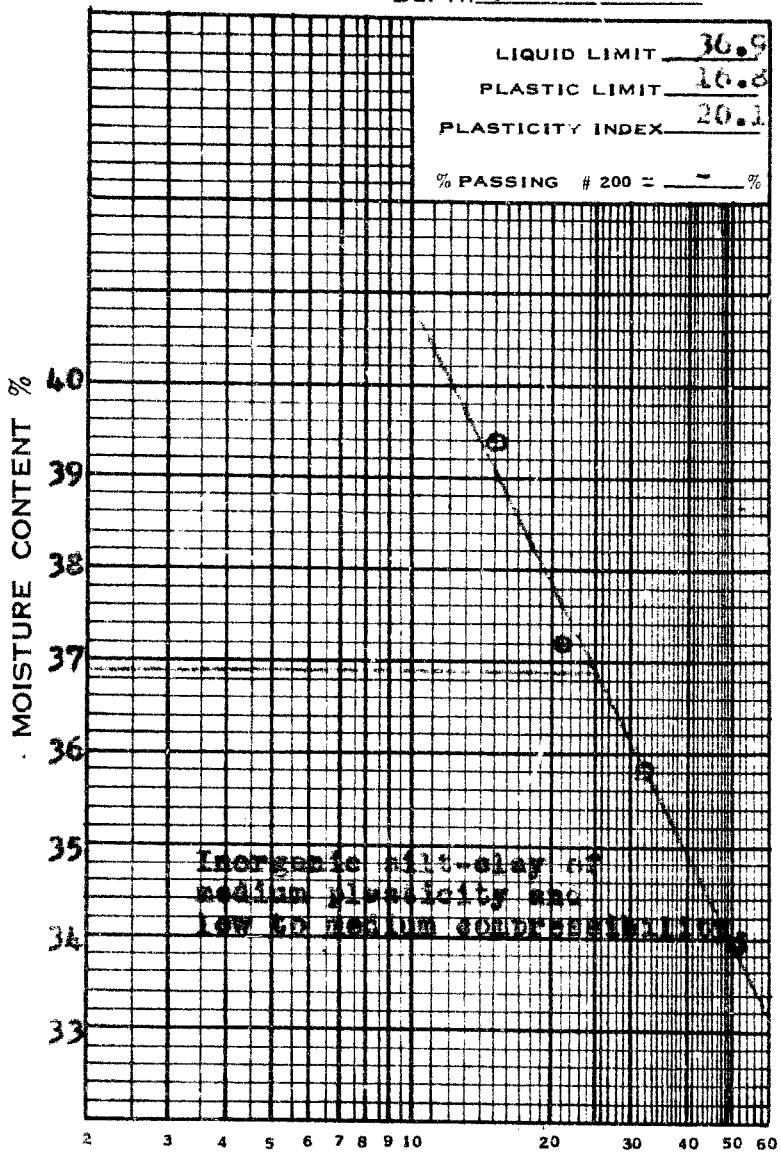
SOIL TESTING LABORATORY

LIQUID LIMIT TEST — FLOW LINE CHARTS

Job No. 5450 Project Port Hope Township Bridge #3

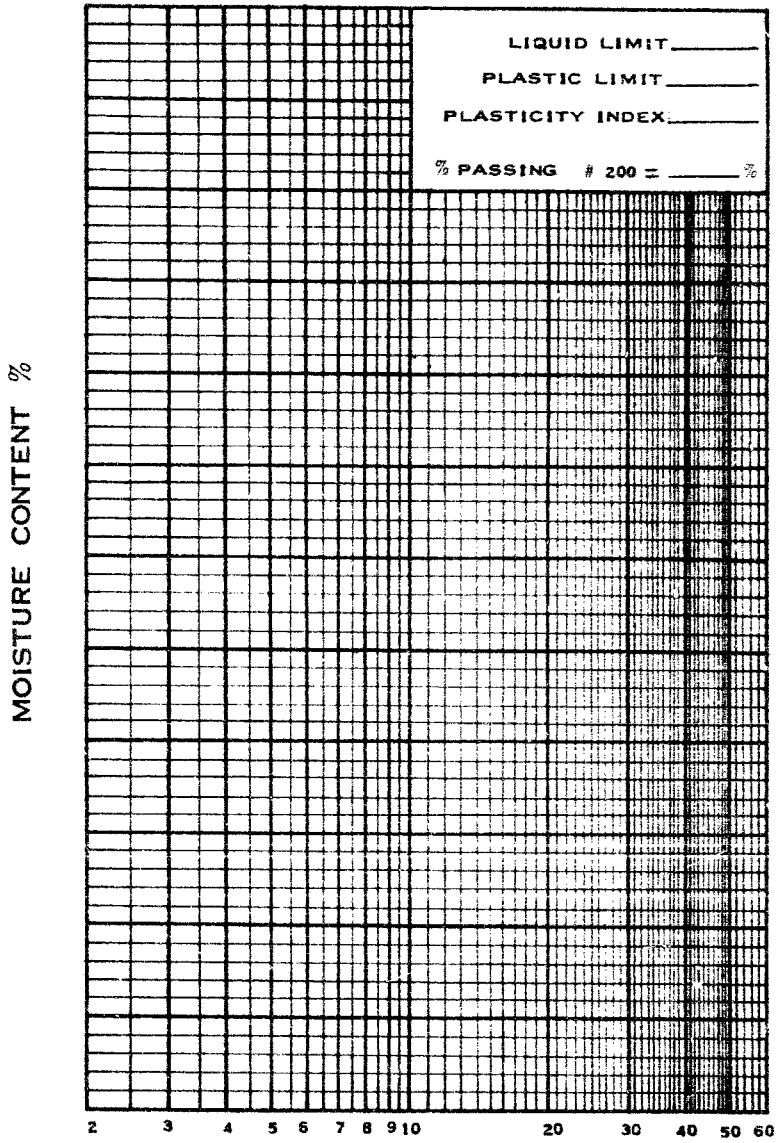
Sample From Borehole 2, St. 2 and 3

Depth 5' - 8'



Sample From

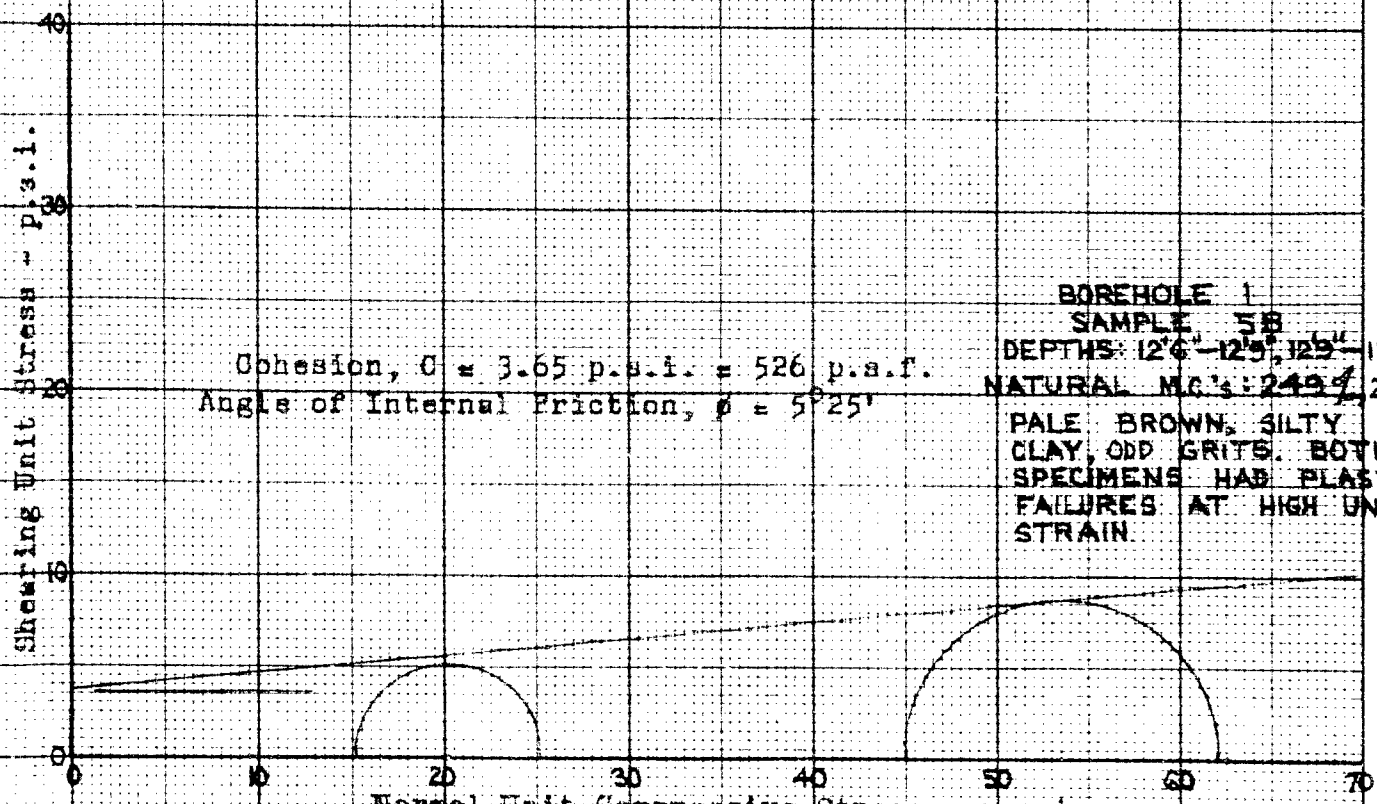
Depth



NO. OF BLOWS (Log Scale)

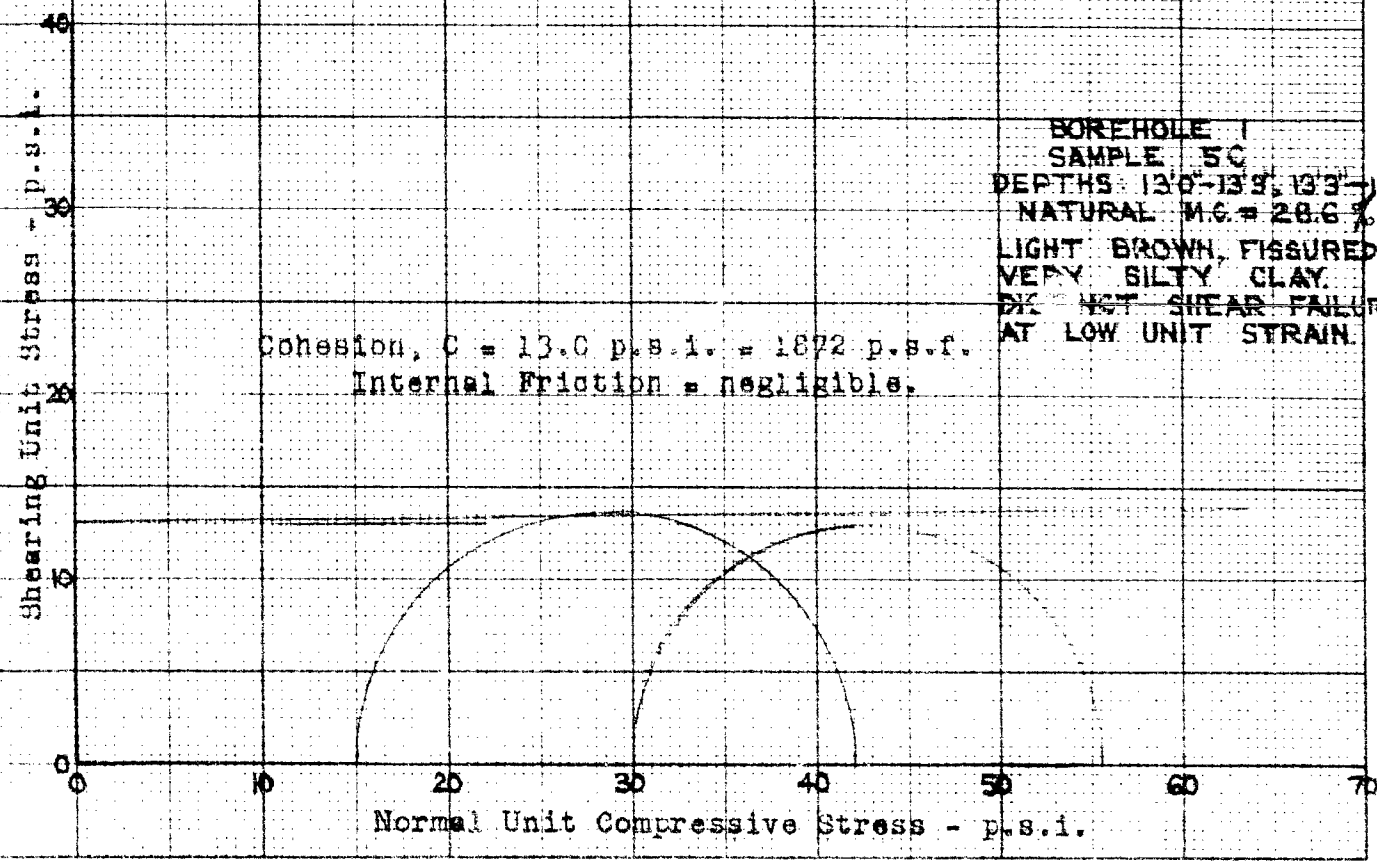
Job No. 5850

QUICK, UNDRAINED TRIAXIAL COMPRESSION TESTS



Cohesion, C = 3.65 p.s.i. = 526 p.s.f.
Angle of Internal Friction, ϕ = 5°25'

BOREHOLE 1
SAMPLE 5B
DEPTHS 12'6"-12'9", 12'9"-13'0"
NATURAL M.C.'s 24%, 27%
PALE BROWN, SILTY CLAY, ODD GRITS. BOTH SPECIMENS HAD PLASTIC FAILURES AT HIGH UNIT STRAIN.



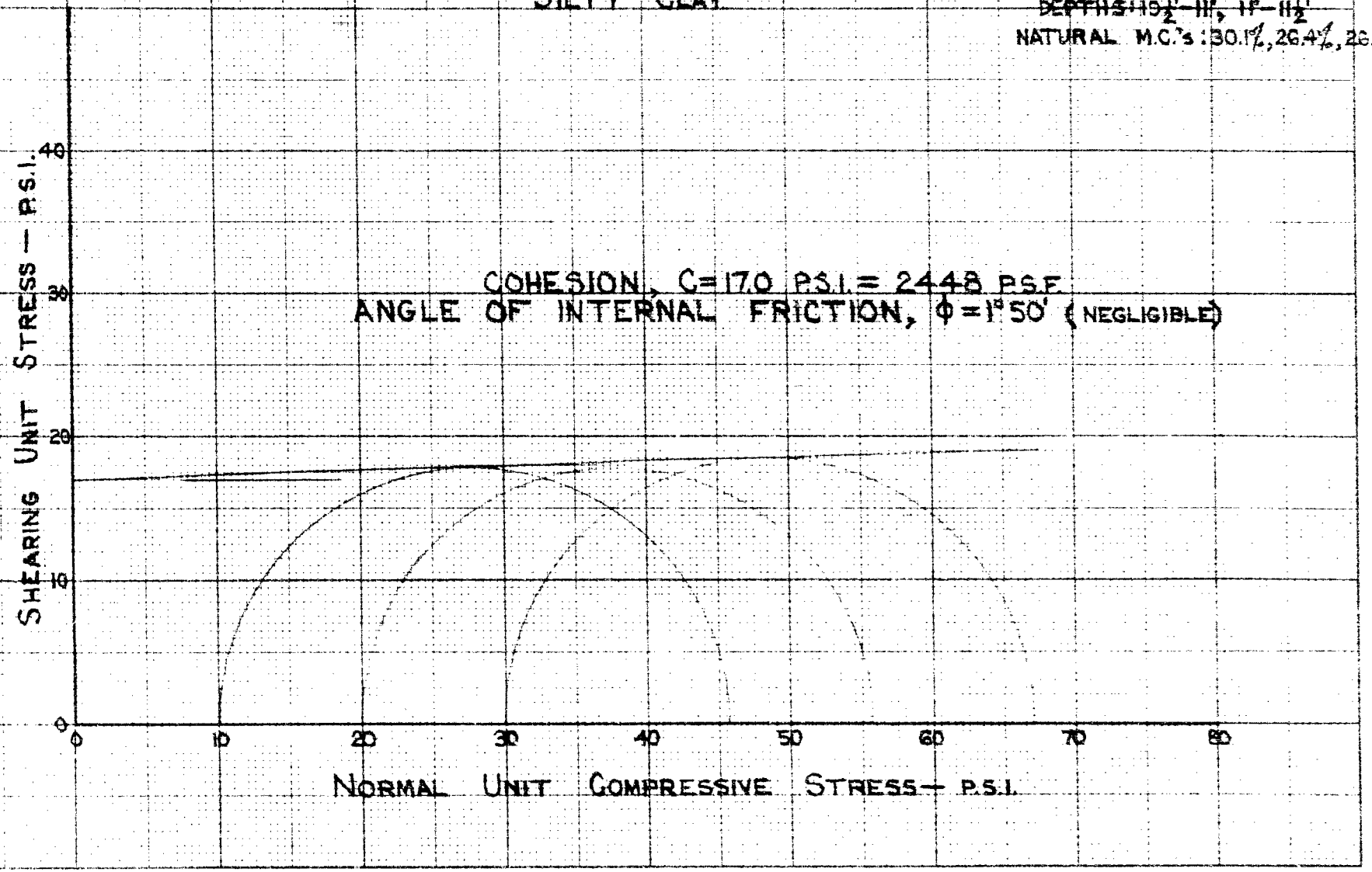
Cohesion, C = 13.0 p.s.i. = 1872 p.s.f.
Internal Friction = negligible.

BOREHOLE 1
SAMPLE 5C
DEPTHS 13'0"-13'3", 13'3"-13'6"
NATURAL M.C. = 28.6 %
LIGHT BROWN, FISSURED VERY SILTY CLAY. DID NOT SHEAR FAILURES AT LOW UNIT STRAIN.

QUICK, UNDRAINED TRIAXIAL COMPRESSION TEST

ON SAMPLES OF PALE BROWN VERY SILTY CLAY

JOB NO. 5850
BOREHOLE 2
SAMPLES 4B, 4C
DEPTHS: 10½'–11', 11'–11½'
NATURAL M.C.'s: 30.1%, 26.4%, 28.1%



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, 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 undisturbed soil.

The Dutch cone probe test is made by driving the drill rods into the ground with a 2-1/4" - 90° cone tip. The number of 4200 inch pound blows per foot of penetration are recorded, as in the standard penetration test.

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

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

The test holes are bailed at the end of the day and on completion. Subsequent water level readings are taken for the duration of the field work. Water pressure readings are recorded when artesian water conditions are encountered. Moisture content samples are recovered at frequent intervals to assist in the soil classification and the interpretation of water table results.