

58-F-304C

W. P. 78-58

Hwy. #65

MOOSE CREEK

BA 786

RACEY, MacCALLUM AND ASSOCIATES
LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

Consulting Engineers
AND ASSOCIATED STAFF

MONTREAL  VANCOUVER

TORONTO

DONALD C. MACCALLUM, B.ENG., M.E.I.C., P.ENG.

H. JOHN RACEY, B.S.C., M.E.I.C., P.ENG.

A. ERIC RANKINE, B.S.C., M.E.I.C., A.M.I.ELEC.E., P.ENG.

TORONTO DIVISION
27 CARLTON STREET
Toronto 2.

Reference: S-500/T-1280

5 September, 1958

58-F-304C

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

Attention: Mr. J. McAllister.

RE: FOUNDATION INVESTIGATION FOR A BRIDGE
ACROSS THE MOOSE CREEK AT HIGHWAY #65, NEAR
THE VILLAGE OF JUDGE, ONTARIO.

WP 78-58

Dear Sir,

The enclosed report presents the results of our soil investigation at the above location.

We hope the report is satisfactory to you; if you have any questions about it do not hesitate to get in touch with us.

Thank you for this opportunity of being of service to you.

Yours sincerely,
RACEY, MacCALLUM AND ASSOCIATES LIMITED,

Ronald F. Scott.

Ronald F. Scott, P.Eng.,
Divisional Soil Engineer.

RFS:YDP

RETURN TO D.H.O.
BRIDGE MAINTENANCE
SECTION

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

FOUNDATION INVESTIGATION FOR A BRIDGE
ACROSS THE MOOSE CREEK AT HIGHWAY #65,
NEAR THE VILLAGE OF JUDGE - ONTARIO.

Reference: S-500/T-1280

Racey, MacCallum and Associates
Limited.

5 September, 1958.

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TORONTO DIVISION
27 CARLTON STREET

Toronto 2.

Reference: S-500/T-1280

5 September, 1958.

FOUNDATION INVESTIGATION FOR A BRIDGE ACROSS THE MOOSE CREEK AT HIGHWAY #65, NEAR THE VILLAGE OF JUDGE - ONTARIO.

This investigation was carried out in order to determine the subsoil conditions at the site of a proposed new bridge, located near the existing structure. It consisted of four borings to a maximum depth of 100 feet including field vane shear strength testing and two cone penetration tests. A series of laboratory tests was carried out on undisturbed samples. This report presents a summary of the field and laboratory results, and recommendations regarding the engineering properties of the subsoil.

FIELD WORK :

The site is located near the North boundary of Lake Timiskaming in what is known as the Timiskaming Clay Plain or "Little Clay Belt". Typical features of this area are interspersed outcrops of pre-Cambrian rock between low-lying clay plains. The latter consist of usually thick layers of varved clay, which is thought to originate from lacustrine glacial deposits. The site is located on one of these clay plains where the ground level is not more than one or two feet above the average level of Lake Timiskaming. The surrounding land is all cultivated and is traversed by several rivers and creeks, one of which is Moose Creek.

The difference in lowest and highest water level of the Creek is about 7 feet. In the summer and autumn the Creek level at the location of the site is near Elevation 586 feet, or 2 feet above Lake Timiskaming level. In the spring it rises in level and floods some of the low-lying surrounding farmland.

Drilling was carried out using a standard type of diamond drill, equipped with 3 $\frac{1}{2}$ -inch heavy duty casing and set up on the existing embankment, to avoid laborious moving up and down of the rig.

Undisturbed samples were taken at regular intervals, using 2-inch inside diameter Shelby tubes, while field vane tests were

- 2 -

Reference: S-500/T-1260

5 September, 1958.

- Continued -

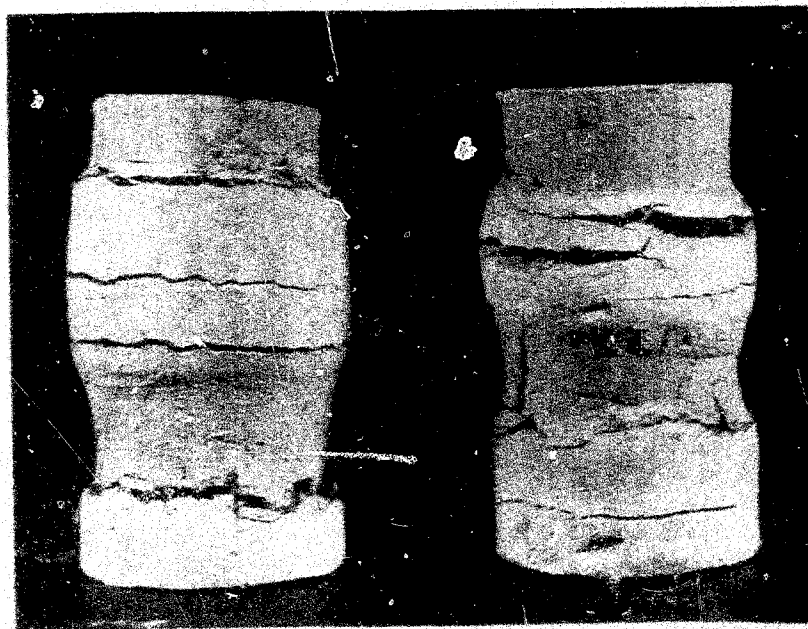
carried out in order to obtain a continuous picture of the shear properties of the soil. Two cone penetration tests were taken down to 100 feet below the surface, for the purpose of indicating a possible refusal level. However, when a local farmer mentioned that his well on a nearby farm was taken down 240 feet through the clay into a gravel bed, attempts to reach firmer soil at a depth greater than 100 feet were abandoned.

Notwithstanding the relatively great depth of the borings no serious difficulties were encountered and the field work was completed in one week, from 14 to 21 July, 1958.

LABORATORY WORK :

The undisturbed samples of the varved clay were subjected to several laboratory tests, including unconfined compression and triaxial tests to give information on the shear strength. Atterberg limit determinations were carried out on many samples, and consolidation tests were performed in order to establish the settlement properties.

The main problem of determining the engineering properties of the varved clay lies in its heterogeneous character, which is well illustrated in the accompanying photograph.



Reference: S-500/T-1280.

5 September, 1958.

- Continued -

The difference in shrinkage amounts distinguishes between the light "summer" varves and the dark "winter" varves. These differences are also evident in the plasticity, compressibility and shear strength properties of the material. In view of this characteristic, it is felt that the shear strength of the soil in the mass can be determined with an accuracy which increases with the size of the sample tested. Consequently, the field vane test results on a soil cylinder approximately 6 inches high will yield more dependable values than the laboratory test results on 4 and 3 inch high samples. The height of one light and one dark varve together is consistently about $1\frac{1}{2}$ inches at all depths from which samples were obtained.

Two consolidation tests were carried out, on samples No 4 and No 5 of Borehole No 3. The former sample was a dark grey or winter seam, the latter, a light grey seam, with some of the dark soil at the top and the bottom of the sample. The results are indicated on Enclosures No's 6 and 7.

From one sample, No 16 of Borehole No 3, all light and dark varves were separated, and the moisture content and Atterberg limits of each type were determined. The results are indicated on the Engineering Data Sheets of Borehole No 3. The soil of the light varves could be classified, in accordance with the Casagrande chart, as a silt of low compressibility, the dark soil is an inorganic clay of high plasticity. Some of the light soil tested from other samples is more clayey and of higher plasticity, however.

DISCUSSION OF RESULTS :

The results of the borings and associated laboratory tests are presented on the Engineering Data Sheets, Enclosures No 2 to 5. Considering the uniform conditions encountered, it was felt that one deep boring with undisturbed samples at 5 feet intervals, and three borings with samples down to the probable depth of the necessary excavation and a series of field vane tests to greater depth, would supply sufficient information. The cone penetration tests taken down to 100 feet at two of the borings were only carried out to ensure that no sudden density changes in the soil would be overlooked.

The top 5 - 10 feet of soil appears to be fill material, and at two of the borings traces of the original topsoil were found at about 7 foot depth. At boring No 2, however, sand and gravel were found to a depth of 17 feet below the top of the 5 to 7 foot high embankment. This may have been deposited by the creek during a period of high currents after previous erosion.

Below the mainly granular overburden a thin crust of somewhat oxidised and desiccated varved clay overlies the grey varved clay

Reference: S-500/T-1280

5 September, 1958

- Continued -

deposit which, as mentioned earlier, may be over 200 feet thick. From the field vane results it may be considered soft to medium stiff. The sensitivity values indicated on the Engineering Data Sheets cannot be considered of much importance, since it is evident that there is a great difference between the sensitivity of the clay in the dark varves and that of the silty clay or clayey silt in the light varves, a conclusion which is reached through the difference in Atterberg limits and natural moisture contents. Both the dark and the light varves have natural moisture contents very close to the liquid limit. The light silty soil would lose less of its strength, however, after remoulding than the dark clay.

The results of the consolidation tests on the two samples tested appear to indicate a rather high compressibility. It is noteworthy that the coefficients of compressibility from these two tests are considerably higher than those determined from the liquid limit - coefficient of compressibility relationship :

$$C_c = .009 \times (L_w - 10\%)$$

established by Skempton and others.

Apparently the properties of individual varves vary more than could be expected from visual inspection. Therefore, an average minimum and maximum C_c value will have to be determined for practical purposes. The time rate of settlements is even more difficult to determine, as the silt seams are certain to give rise to a considerable amount of horizontal drainage which cannot be imitated in the laboratory test. It may only be assumed that consolidation will take place more rapidly than can be determined from the laboratory results. Uniform settlements in excess of normally tolerated maxima and taking place over a long period of time must be anticipated.

RECOMMENDATIONS :

The above results are sufficient evidence that construction of a higher embankment and a new and larger bridge at this site will meet with a number of difficulties. The varved clay extends to too great a depth to consider end bearing piles. The bearing capacity of friction piles will be virtually impossible to determine without trial loadings. Consequently, the only alternative is to design a light structure of wide footings and to determine the settlements accordingly.

The proposed new height of the embankment is only about 3 feet above the present height. This should not cause any appreciable settlements of the clay, which has been preconsolidated under approximately 10 feet of fill and granular soil.

The bridge abutments will have to be founded at some depth

Reference: S-500/T-1280

5 September, 1958.

- Continued -

below the creek depth, in order to prevent scour damage. This is very important in this case, because the varved clay would offer little resistance against fast running water. The existing bridge does not appear to have suffered badly, but no information could be obtained about the depth of its foundations. It would seem reasonable to assume a foundation depth of 26 feet below proposed grade, or Elevation 574 feet. This is 5 feet below the deepest point of the creek bed, as indicated on the Department of Highways of Ontario profile. The average shear strength at that depth is of the order of 600 psf, and based on this value the ultimate bearing capacity of a wide footing at that depth would be of the order of 4200 psf plus the weight of the overburden.* The first part is generally divided by a factor of safety of three to obtain the safe bearing capacity. The weight of the overburden in the case under consideration is 1800 - 2000 psf on the land side of the abutments. If toe protection in the form of sheet piling were applied to prevent a shear failure towards the creek, the full overburden weight on the land side may be added to the safe bearing capacity. This would give a final value of 3200 psf.

It is considered that the settlements under such a load can best be determined using an average value of the compression indices obtained from the consolidation test results and those based on the liquid limits. The maximum possible average for dark and light varves together is felt to be of the order of 0.9, and the minimum possible value around 0.4. Based on these figures the maximum and minimum settlements under a 10 foot wide footing will be of the order of 10 inches maximum and 5 inches minimum respectively, under a 3200 psf load. Consequently, it seems that the footing loads can only be taken slightly higher than the present overburden weight of about 2000 psf at that depth, and a light-weight construction will be essential for the new bridge.

The problem will be complicated by the lateral thrust of approximately 26 feet of backfill on the bridge abutments. This will cause high toe pressures on the abutment footings, and after consolidation has taken place the abutments may be found to lean forward.

On Enclosure No 8 a light reinforced concrete abutment is sketched, on a wide footing with a large toe. The footing loads can be reduced considerably by using a light-weight backfill above the heel of the abutment. The forces acting on the abutment are determined on the basis of a lowest water table at Elevation 584 feet. The forces indicated on the sketch include dead weight, fill weight and active and passive thrusts, but not the weight of the bridge span. The maximum pressure at the toe of the footing in the demonstrated case would still be of the order of 3200 psf, while the average foundation pressure is not more than 1600 psf. This does not take the passive thrust P_3

* G. G. Meyerhof : "The Bearing Capacity of Foundations", Geotechnique Vol. II.

Reference: S-500/T-1260

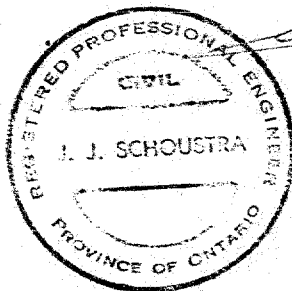
5 September, 1958.

- Continued -

into account, because it would only act when movement of the abutment takes place. It would be necessary, therefore, to obtain more lateral support, either by anchoring the abutment wall or by shoring it against the other abutment by means of a rigid construction.

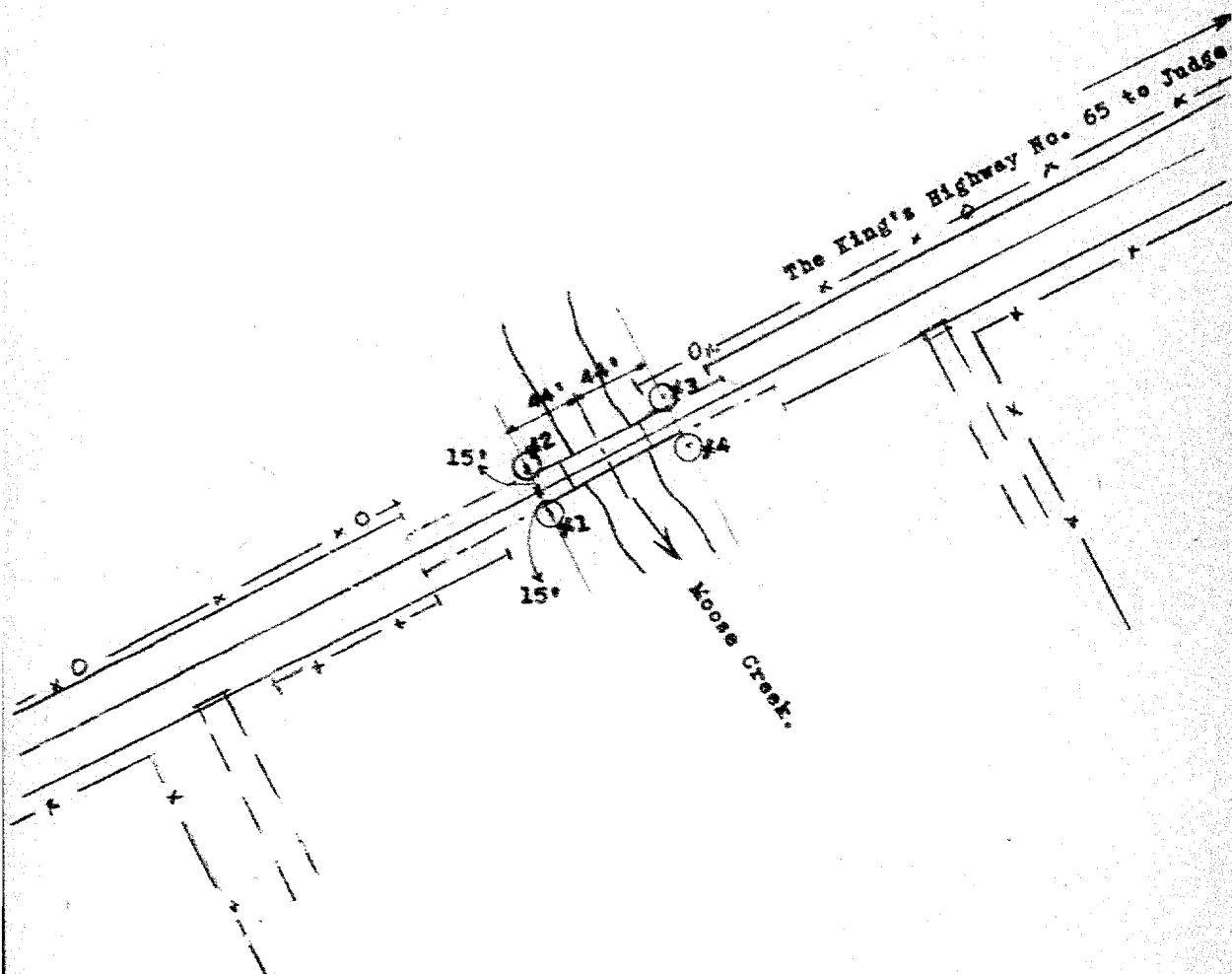
Summarising the results of this investigation, the following conclusions may be drawn :

1. The soil at the site consists, apart from a fill layer, of a very thick layer of varved clay.
2. Although the individual properties of the different varves vary considerably, the clay as a whole can be considered highly compressible and of medium stiffness.
3. The permissible bearing capacity for a footing foundation is limited by the extent of consolidation settlement to a value which is little more than the present weight of overburden.
4. A light-weight construction with a very wide footing is recommended; extra lateral support for the abutments must be found in the form of anchorage or frame-work
5. Because of the possibility of scour damage, the footing depth should be at least 5 feet below the present depth of the creek, or Elevation 574 feet.

J. J. Schoustra, P.Eng.,

JJS:YDP.

Prep. By J.J.S.

HIGHWAY NO. 65 AND MOOSE CREEK.

Location of borings.

From D.H.O. sketch plan.

Scale : 1 inch = 100 feet.

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: **1**Project: **HIGHWAY NO. 65 AND MOOSE CREEK**Location: **NEAR JUDGE, ONTARIO.**Hole Location: **See Enclosure No. 1.**Hole Elevation and Datum: **596.3 Ft. M.S.L.**Field Supervisor: **H.G. Prep.: J.S.**Driller: **O.R. Checked: J.S.**Date: **19. 8. 1958****LEGEND**

Shear Strength (C)

Unconfined compression

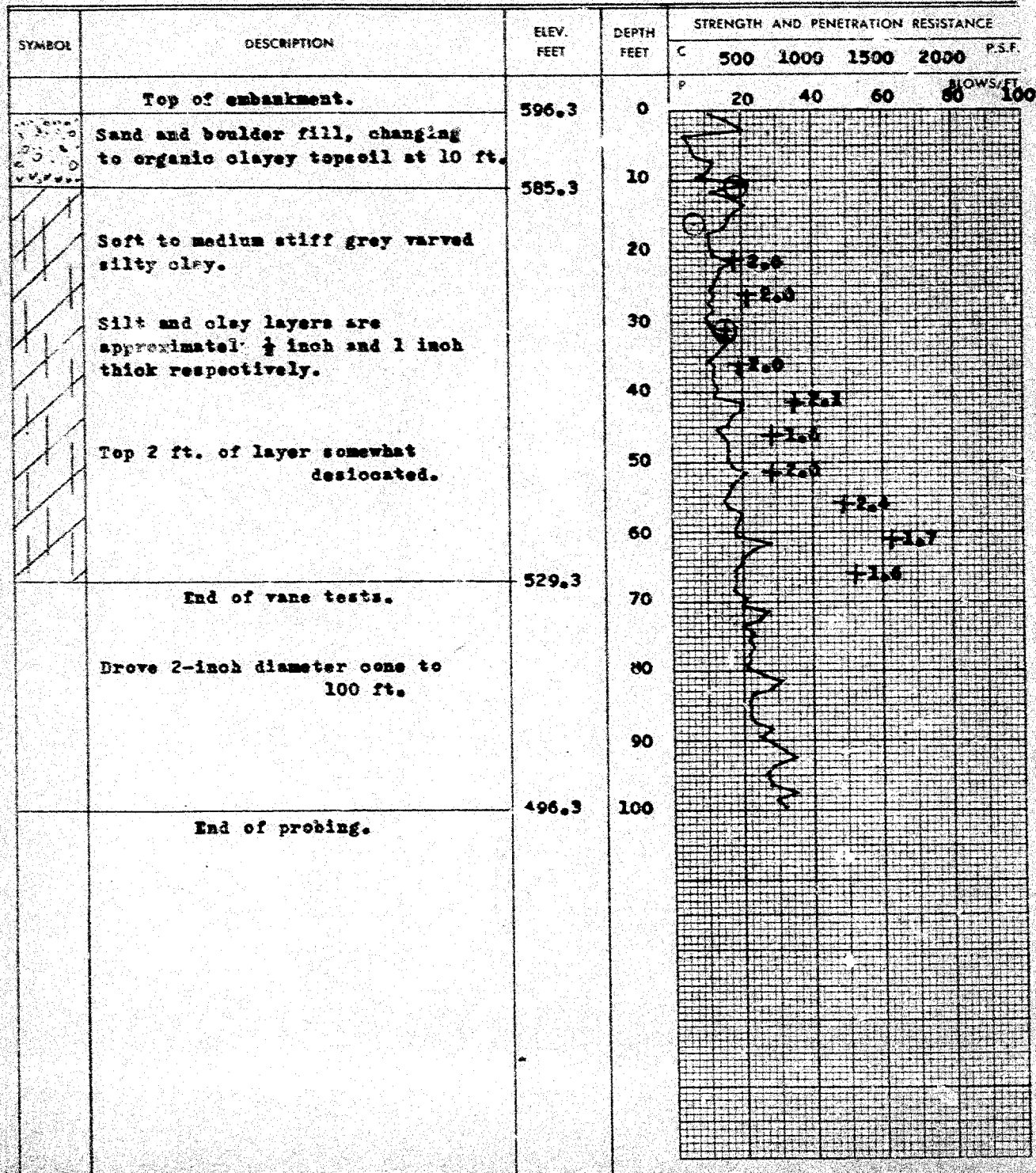
Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing



RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1

LEGEND**Consistency**

Natural moisture and
Liquidity Index (LI)
Liquid limit
Plastic limit



X LI

-O

-P

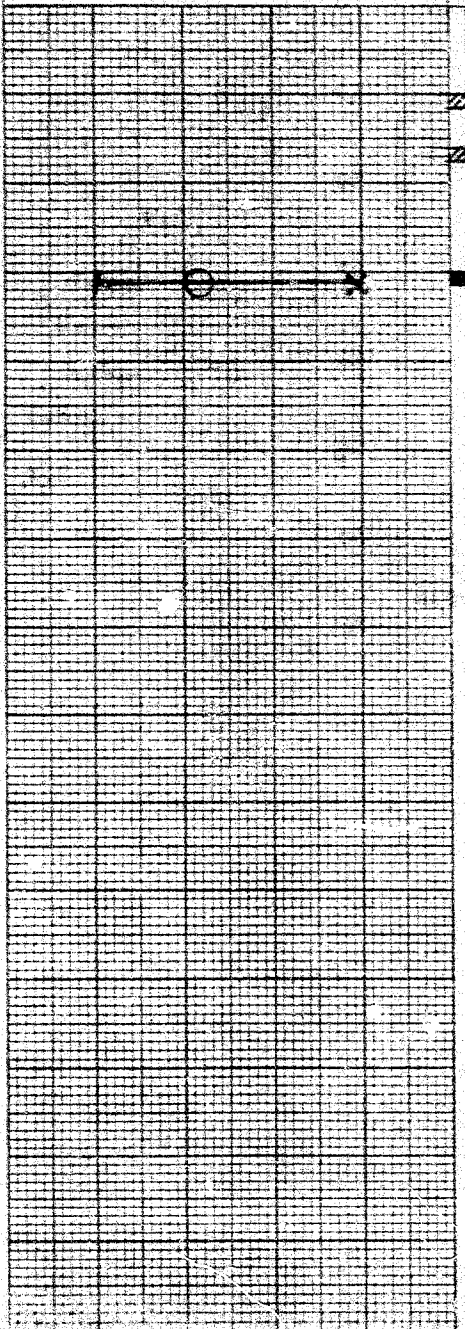
Sampling Method

2" Dia. split tube



2" Shelby tube



DEPTH FEET	CONSISTENCY					SAMPLE	NATURAL UNIT WT. P.C.F.	REMARKS
	MOISTURE CONTENT, - DRY WEIGHT							
	20	40	60	80	100			
0								
10						1		
20						2		
30						3	106	Quick - undrained triaxial test : C = 650 psf.
40								
50								
60								
70								

OVER

OVER

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1

Project: HIGHWAY NO. 65 AND MOOSE CREEK

Location: NEAR JUDGE, ONTARIO.

Hole Location: See Enclosure No. 1.

Hole Elevation and Datum: 595.6 Ft. M.S.L.

Field Supervisor: H.G. Prep.: J.S.

Driller: O.R. Checked: J.S. Date: 19. 8. 1958

LEGEND

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube








2" Dia. Cone

Casing

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4⁹

⊕ ⊕

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE					
				C	500	1000	1500	2000	P.S.F.
	Top of embankment.	595.6	0	P	20	40	60	80	100
	Medium dense, medium to coarse sand with gravel and boulders. Probably fill.	578.6	10						
			20						
			30						
			40						
			50						
			60						
			70						
	End of boring.	528.6							

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 2

LEGEND**Consistency**

Natural moisture and

Liquidity Index (LI)

Liquid limit

Plastic limit

— * —

x LI

— O —

—

Sampling Method

2" Dia. split tube

2" Shelby tube



DEPTH FEET	CONSISTENCY					SAMPLE	NATURAL UNIT WT. P.C.F.	REMARKS
	MOISTURE CONTENT, % DRY WEIGHT							
0	20	40	60	80	100			
10						1		Only disturbed samples were obtained from the value.
20						2		
30								
40								
50								
60								
70								

OVER

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 3

Project: **HIGHWAY NO. 65 AND MOOSE CREEK**Location: **NEAR JUDGE, ONTARIO.**Hole Location: **See Enclosure No. 1.**Hole Elevation and Datum: **595.0 Ft. M.S.L.**Field Supervisor: **H.G. Prep.: J.S.**Driller: **O.R. Checked: J.S. Date: 19. 8. '58****LEGEND**

Shear Strength (C)

Unconfined compression

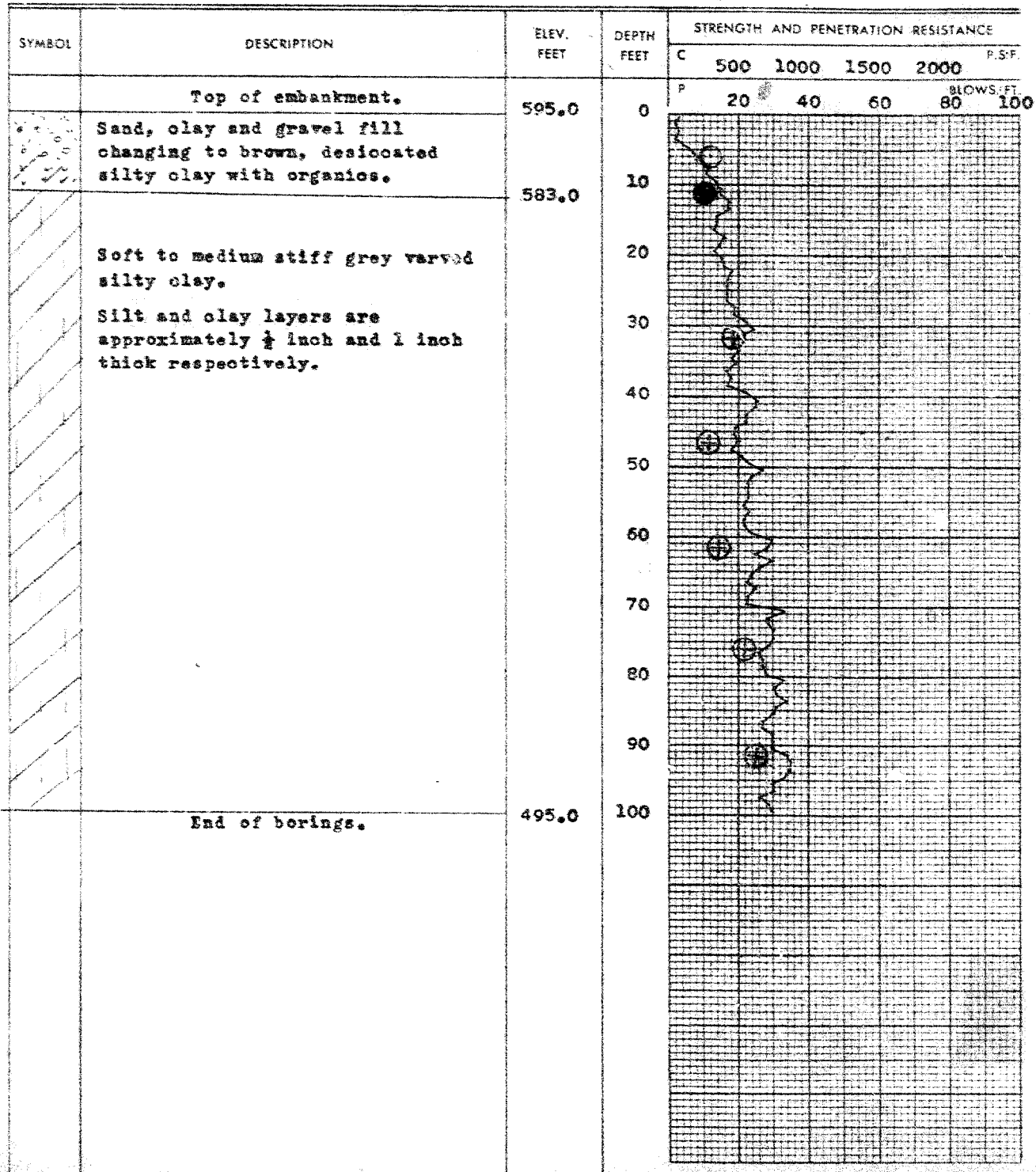
Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

2" Dia. Cone

Casing



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Foundation Engineering Division

Engineering Data Sheet for Borehole: 3

LEGEND**Consistency**

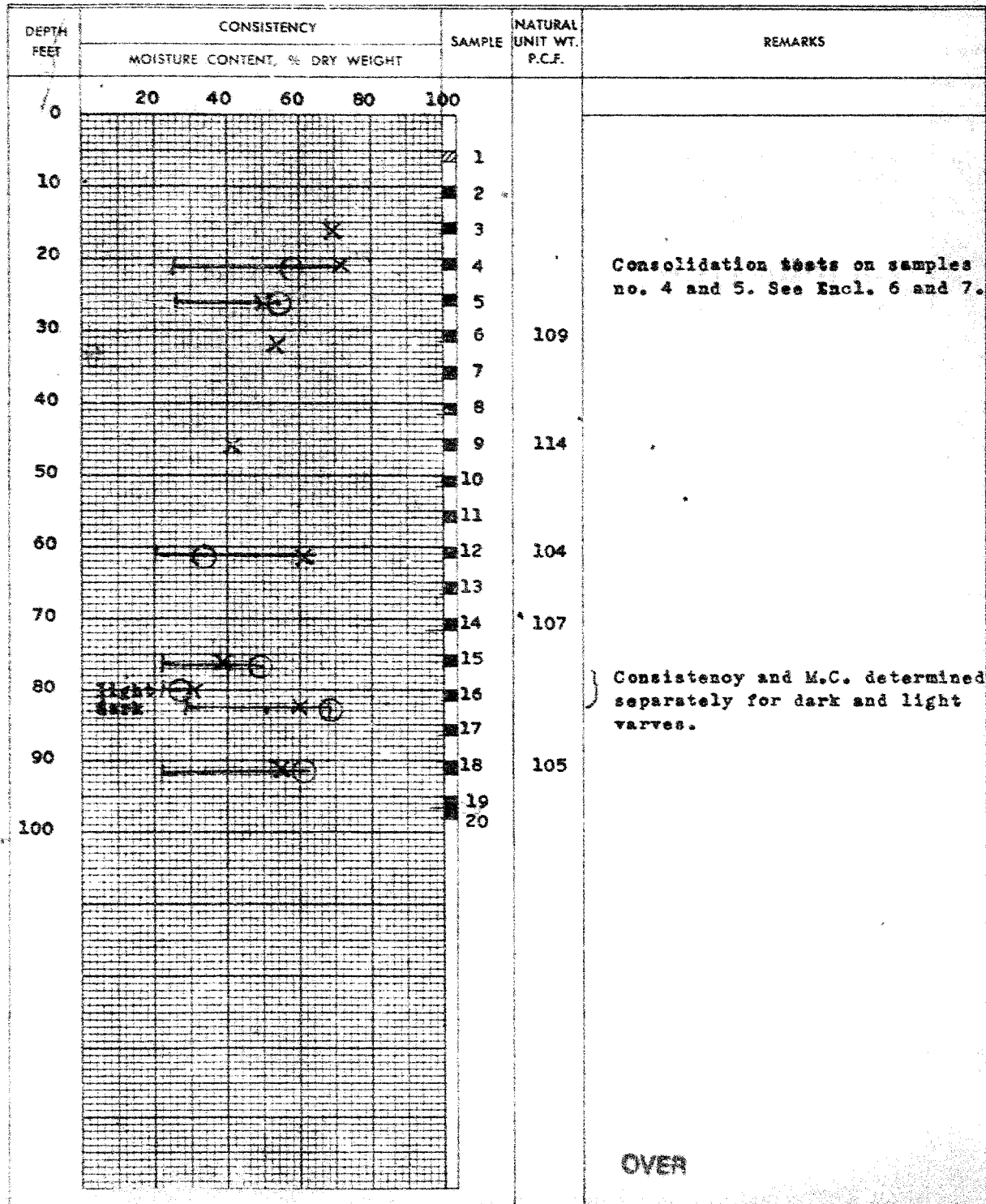
Natural moisture and
Liquidity Index (LI)
Liquid limit
Plastic limit

**Sampling Method**

2" Dia. split tube



2" Shelby tube



OVER

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: L₁Project: HIGHWAY NO. 65 AND MOOSE CREEKLocation: NEAR JUDGE, ONTARIO.Hole Location: See Enclosure No. 1.Hole Elevation and Datum: 595.3 Ft. M.S.L.Field Supervisor: H.G. Prep.: J.S.Driller: O.R. Checked: J.S.Date: 19. 8. 1958**LEGEND**

Shear Strength (C)

Unconfined compression

Vane test and sensitivity (S)

Penetration Resistance (P)

2" Split tube

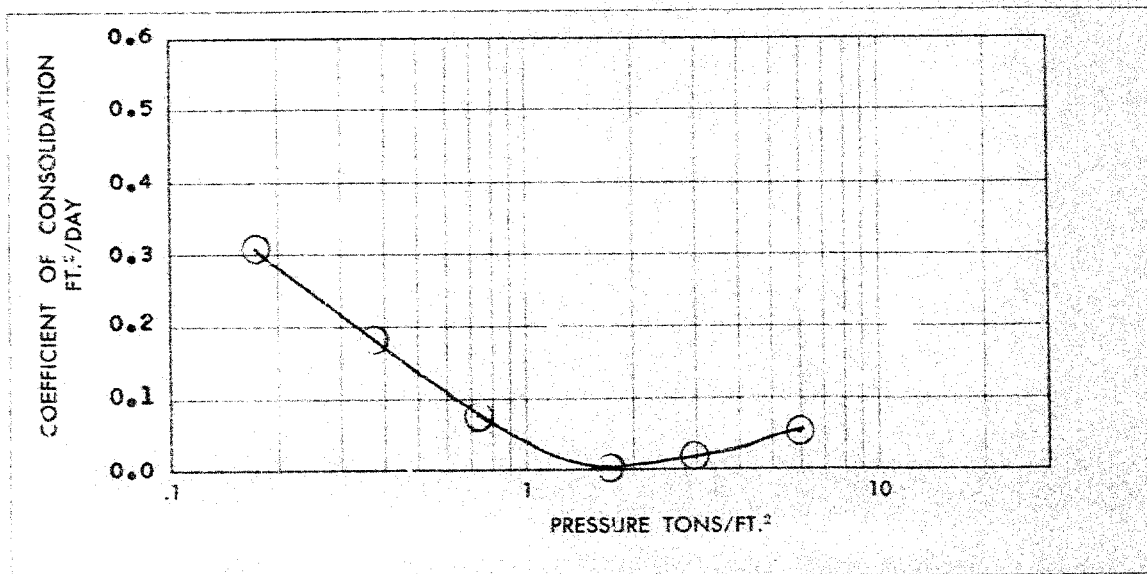
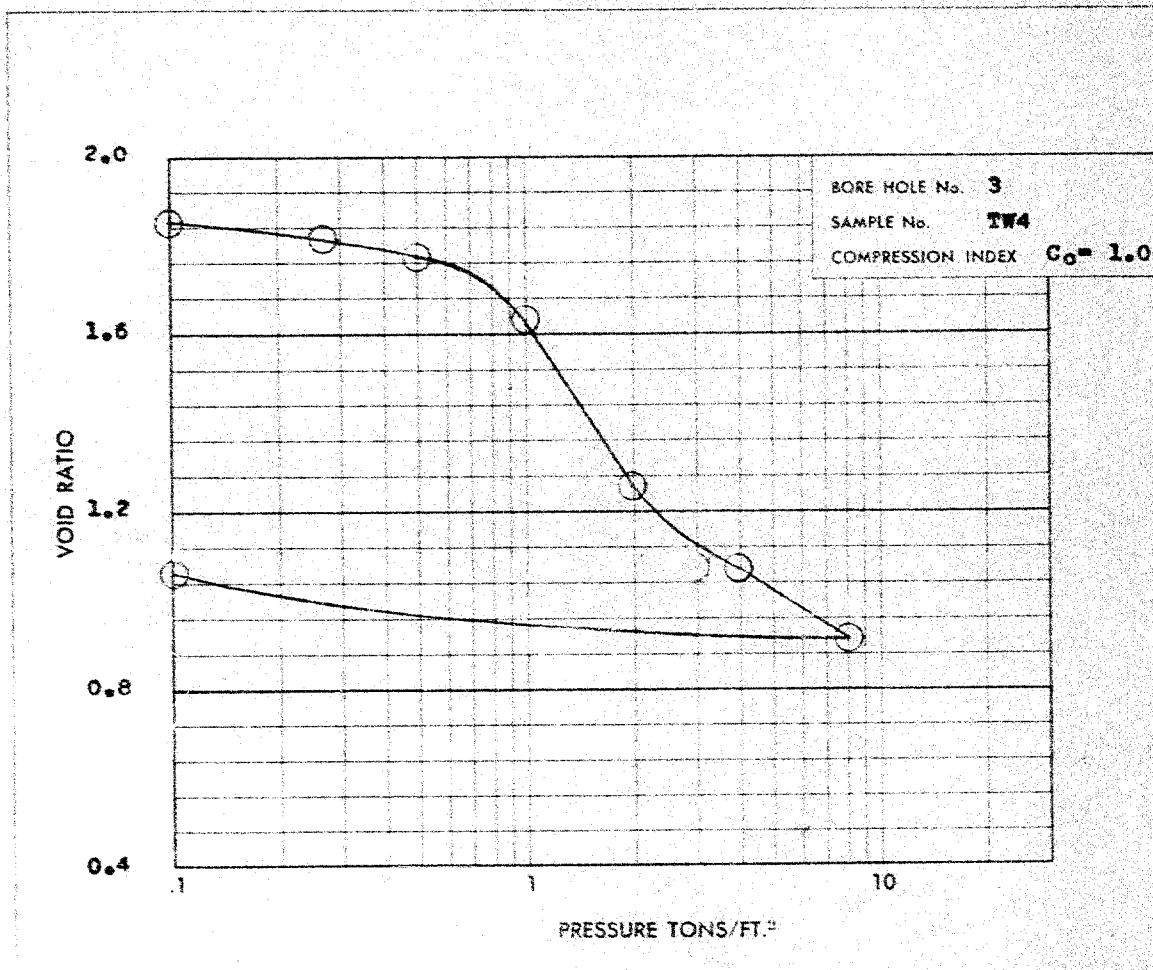
2" Dia. Cone

Casing

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE					
				C	500	1000	1500	2000	P.S.F.
	Top of embankment.	595.3	0	P	20	40	60	80	100
	Sand, clay and gravel fill changing to brown organic clayey topsoil at 10 ft.	582.3	10						
	Soft to medium stiff grey varved silty clay.	530.3	20		+ 2.3				
					+ 2.3				
			30		+ 2.1				
					+ 2.5				
			40		+ 2.4				
					+ 3.7				
	End of boring.	530.3	50		+ 2.1				
					+ 3.1				
			60		+ 2.0				
					+ 1.9				
			70						
			80						
			90						

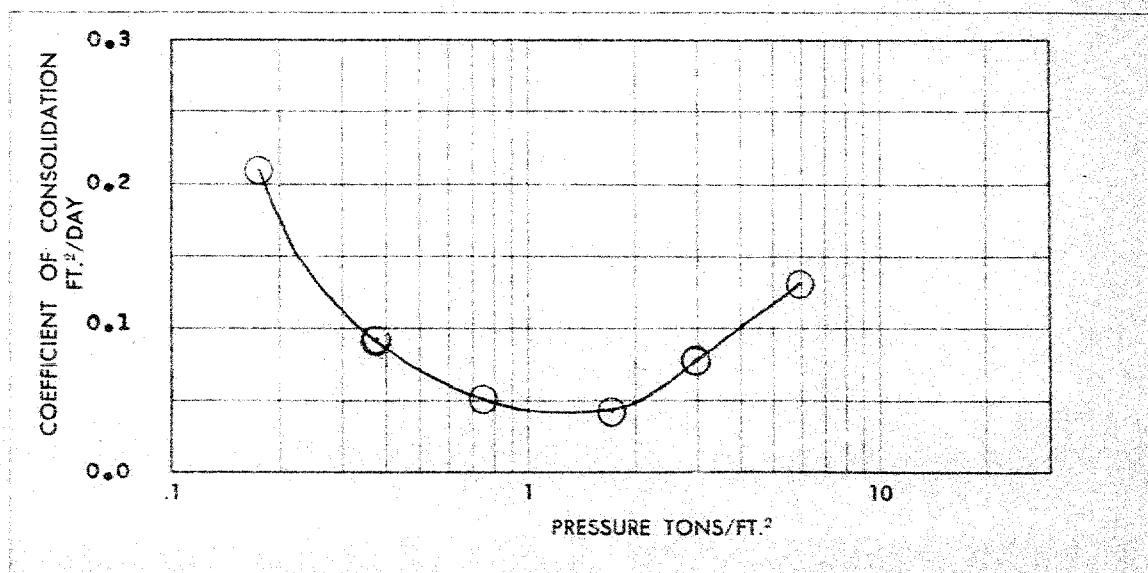
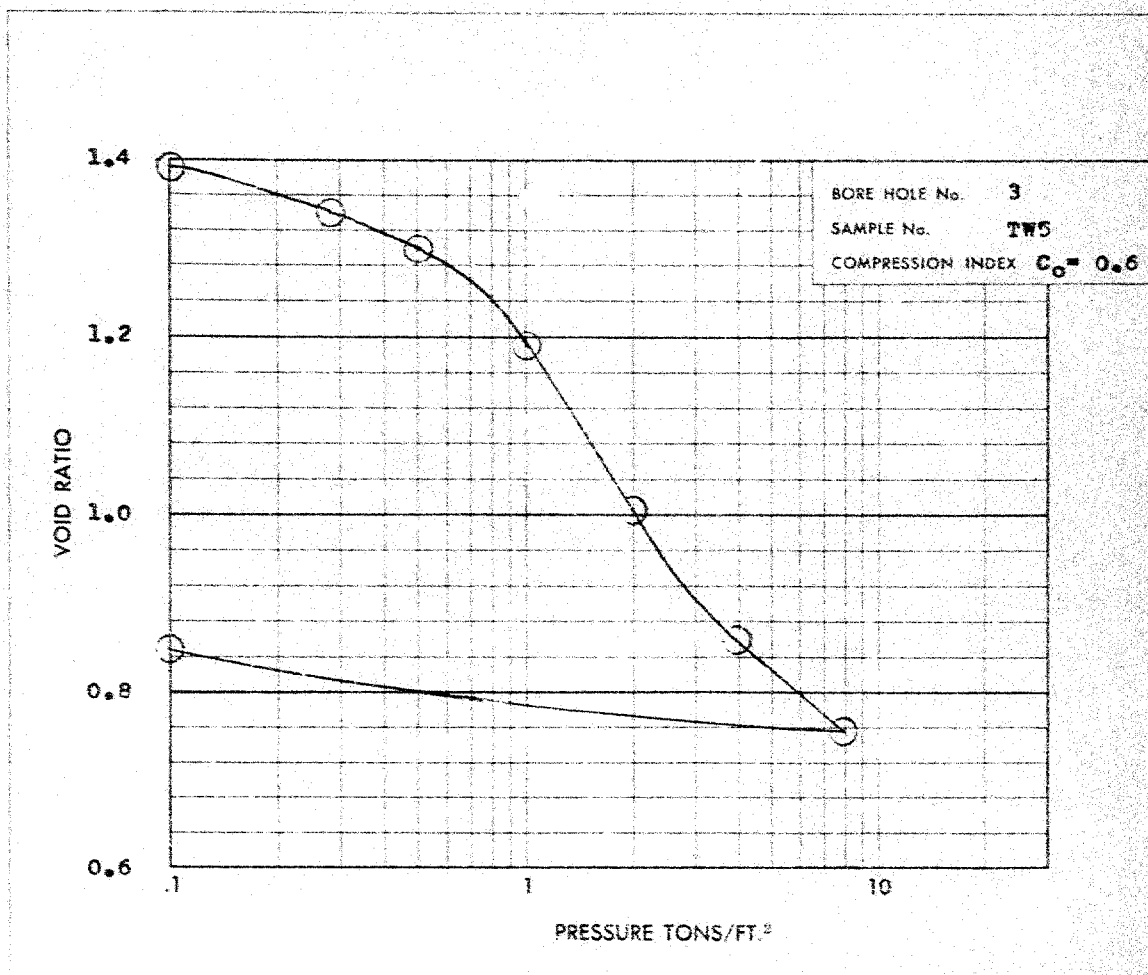
RACEY McCALLUM AND ASSOCIATES LTD.

CONSOLIDATION TEST

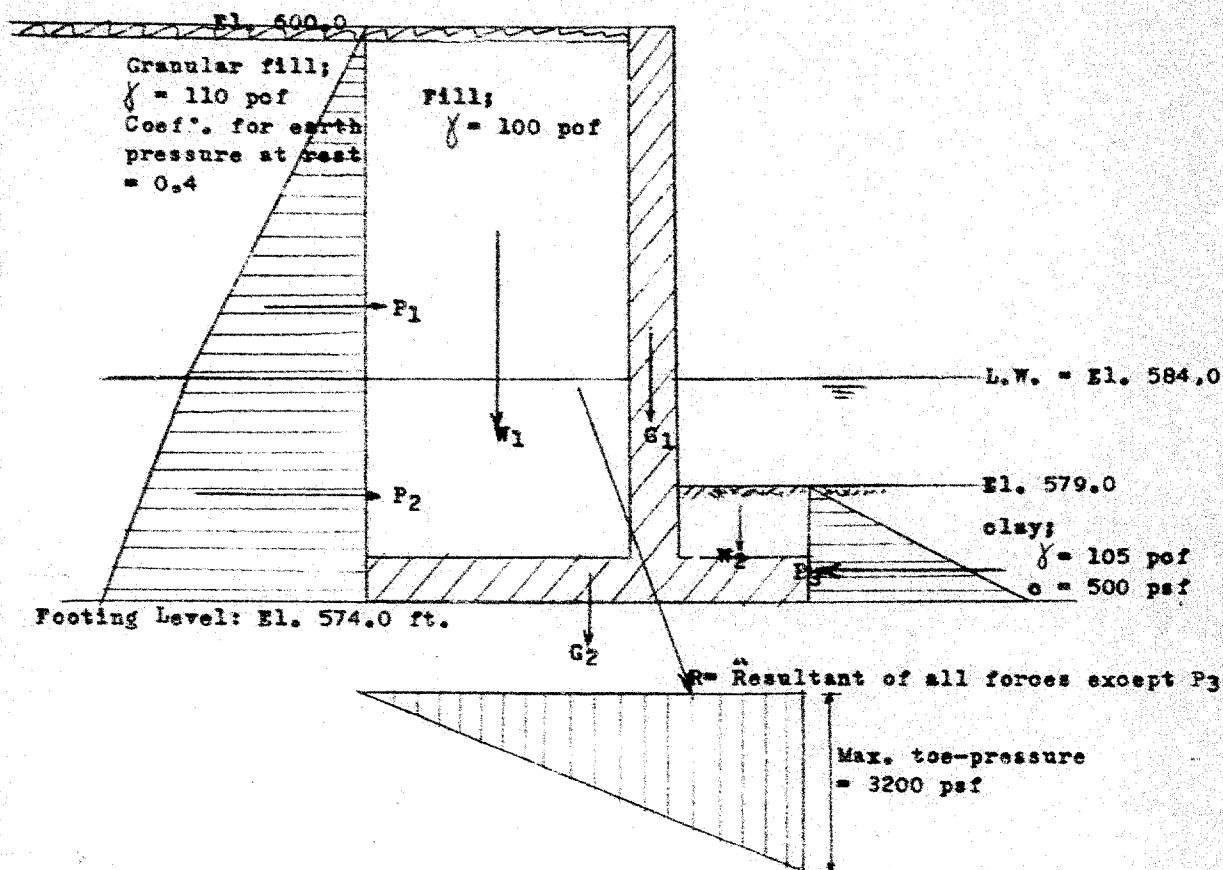


RACEY MacCALLUM AND ASSOCIATES LTD.

CONSOLIDATION TEST



Prep. By J.S.



FORCES ACTING ON BRIDGE ABUTMENT.

Vertical:

W_1 11.4 tons
 W_2 0.4 "
 G_1 2.8 "
 G_2 1.8 "
 Bridge span . unknown.

Horizontal:

P_1	2.9 tons	} These forces will be reduced by about $\frac{1}{4}$ when lateral movement occurs.
P_2	4.3 "	
P_3	3.5 tons	(This force will not reach its full value until movement begins)

5A786 A

W. P. 78-58, DISTRICT 14
DEPARTMENT OF HIGHWAYS, ONTARIO

58-F-304 C

BORINGS
THROUGH APPROACH FILL
MOOSE CREEK BRIDGE
HIGHWAY 65
NEAR JUDGE, ONTARIO

Distribution:

18 Copies - Department of Highways, Ontario,
Toronto, Ontario.

2 Copies - H. Q. Golder & Associates Ltd.
Toronto, Ontario.

June, 1961

6124

GOLDER & ASSOCIATES

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section,
(Foundations Office).

June 30, 1961.


FOUNDATION INVESTIGATION REPORT
by: H.Q. Golder & Assoc., Ltd.
for: D.E.O.

Re: Borings Through Approach Fill,
Moose Creek Bridge, Highway 65,
Near Judge, Ont., District 14.
W.P. 78-58.

This memo accompanies the factual report prepared
by the Consultant, H. Q. Golder & Associates, Ltd.

Borings for the above noted site were requested
by you in order to define the size and number of boulders
present in the area of the approach fills to the structure.

LGS/MdeF
Attach.


L. G. Soderman,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
G. K. Hunter
R. S. Chapman
E. R. Saint
J. Roy
T. J. Kovich
J. E. Gruspier
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Foundations Office
Gen. Files

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
V. MILLIGAN

2446A BLOOR ST. W.
TORONTO 9
RO. 7-9201

June 19, 1961

Department of Highways, Ontario,
Materials and Research Section,
Downsview, Ontario.

Attention: Mr. A. G. Stermac, P. Eng.,
Supervising Foundation Engineer

RE: BORINGS,
THROUGH APPROACH FILL,
MOOSE CREEK BRIDGE,
HIGHWAY 65,
NEAR JUDGE, ONTARIO

Dear Sirs:

This letter reports the factual results of the above work carried out at the site of the existing Moose Creek Bridge on Highway 65, between lots 7 and 8 at Concession II & III in the Township of Casey, County of Timiskaming, a few miles west of the Ontario-Quebec boundary. The purpose of this investigation was to determine, at the locations marked by you on Fenco Drawing No. D4753-1, the size and number of boulders present within the roadway approach fills to the existing bridge in order to assess the difficulties which may be encountered in driving closed steel sheeting for the new bridge.

The field work was carried out between May 30th, 1961 and June 6th, 1961 using a standard skid-mounted machine drillrig. A total of 4 boreholes was put down to the depths specified by

you. The borings were drilled through the roadway fill in BX casing size using a EX casing shoe and jetted down in the underlying varved silty clay using A rods. At three of the boring locations a hole was dug by hand to a depth of about 2 to 3 feet to remove the large size rip-rap cover prior to commencement of drilling.

The locations of the borings are shown on Figure 1. Detailed logs of each borehole showing the depths at which boulders or granular and clayey fill were encountered are given on the Records of Boreholes.

The core samples obtained during the investigation will be stored in our laboratory until January 1st, 1962 at which time you will be notified regarding their disposal.

All elevations in the report are referred to Geodetic datum and were obtained by reference to the existing bridge structure for which elevations are shown on Fenco Drawing No. D4753-1.

In general the roadway approaches to the existing bridge are comprised of granular to clayey fill consisting of gravel and sand with a trace of clay together with cobbles and boulders. The cobbles and boulders placed towards the sides of the approach embankments and as a rip-rap cover for the toe

of the fill are rock sizes generally obtained from mining operations in the area. The maximum boulder size observed at the toe of the fill was about 5 feet with an average boulder size of about 2 to 3 feet.

We believe that this letter contains the factual information necessary for you to finalize your foundation design. If we can be of further service to you, please call us.

JLS/jb
6124



J. L. Seychuk, P. Eng.

LIST OF STANDARD ABBREVIATIONS

The standard abbreviations commonly employed on each "Record of Borehole", on the figures, and in the text of the report are as follows:

SAMPLE TYPES

A.S. - Auger Sample	R.C. - Rock Core
C.S. - Chunk Sample	S.T. - Slotted Tube
D.O. - Drive Open	T.O. - Thin-walled, Open
D.S. - Denison Type Sample	T.P. - Thin-walled, Piston
F.S. - Foil Sample	W.S. - Wash Sample

PENETRATION RESISTANCES

Dynamic Penetration Resistance - The energy required to drive a 2 inch diameter, 60 degree cone attached to the end of the drilling rods into the ground: expressed in blows per foot, where each blow represents 4,200 inch-pounds of energy.

Standard Penetration Resistance, N - The number of blows by a 140 pound hammer dropped 30 inches required to drive a 2 inch drive open sampler one foot into the ground.

Sampler advanced by static weight	- weight, hammer	- Wh
Sampler advanced by pressure	- pressure, hydraulic	- Ph
Sampler advanced by pressure	- pressure, manual	- Pm

SOIL DESCRIPTION

The standard terminology for the descriptions of the relative density of cohesionless soils and the consistency of cohesive soils is as follows:

<u>Relative Density</u>	<u>N, Blows/ft.</u>	<u>Consistency</u>	<u>c, lb/sq. ft.</u>
Very Loose	0 to 4	Very Soft	Less than 250
Loose	4 to 10	Soft	250 to 500
Compact	10 to 30	Firm	500 to 1,000
Dense	30 to 50	Stiff	1,000 to 2,000
Very Dense	over 50	Very Stiff	2,000 to 4,000
		Hard	over 4,000

SOIL TESTS

C - Consolidation Test	Q - Undrained Triaxial
H - Hydrometer Analysis	Qc - Consolidated Undrained Triaxial
M - Sieve Analysis	S - Drained Triaxial
MH - Combined Analysis, Sieve and Hydrometer	U - Unconfined Compression
	V - Field Vane Test

Note: Undrained triaxial tests in which pore pressures are measured are shown as Q' or Q'c.

SOIL PROPERTIES

γ - Total Unit Weight	K - Coefficient of Permeability
γ_d - Dry Unit Weight	c - Undrained Shear Strength ($\frac{1}{2}$ Compressive Strength)
γ_b - Submerged Unit Weight	St - Sensitivity
L _L - Liquid Limit	ϕ' - Effective Angle of Shearing Resistance
P _L - Plastic Limit	c' - Effective Cohesion Intercept
W - Natural Water Content	Cc - Compression Index
G - Specific Gravity	Cv - Coefficient of Consolidation
e - Void Ratio	

RECORD OF BOREHOLE 1

LOCATION SEE FIG. 1

BORING DATE

MAY 31, 1961

DATUM

GEODETIC

BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER

BX CASING

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES

SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT	LIQUID LIMIT L_L PLASTIC LIMIT P_L WATER CONTENT W		WATER CONTENT, PER CENT
ELEVATION DEPTH	DESCRIPTION	START PLAT	NUMBER TYPE	BLOWS / FT.		SHEAR STRENGTH C , LB / SQ FT.			
557.2	GROUND LEVEL								
556.8									
555.2	F								
554.6	B								
553.6	B								
552.2	VARVED								
551.6	SILTY CLAY								
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RECORD OF BOREHOLE 2

LOCATION SEE FIG. 1 BORING DATE JUNE 6, 1961 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER BX CASING
 SAMPLER HAMMER WEIGHT — LB DROP — INCHES PEN TEST HAMMER WEIGHT LB DROP INCHES

SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT L_L PLASTIC LIMIT P_L P_L W L_L WATER CONTENT W			
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE		SHEAR STRENGTH S , LB / SQ. FT.				WATER CONTENT, PER CENT			
591.1	GROUND LEVEL				592								
590.6	EXCAVATED BY HAND - 4 BOULDERS 0.5' 1' 0.5' AND 0.5' IN SIZE RESPECTIVELY IN A MATRIX OF CLAYEY TO SANDY FILL				591								
588.4					590								
588.3	B				589								
586.9	F				588								
586.0					587								
585.8	B				586								
585.6	F				585								
585.3	B				584								
584.9	F				583								
584.2					582								
	VARVED				581								
					580								
	SILTY CLAY				579								
578.4					578								
577.5	END OF HOLE				577								

LEGEND

F FILL
 B BOULDER

VERTICAL SCALE
 1 INCH TO 2'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
 CHECKED *dy*

RECORD OF BOREHOLE 3

LOCATION SEE FIG. 1

BORING DATE JUNE 1-5, 1961

DATUM

GEODETIC

BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER

8x CASING

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

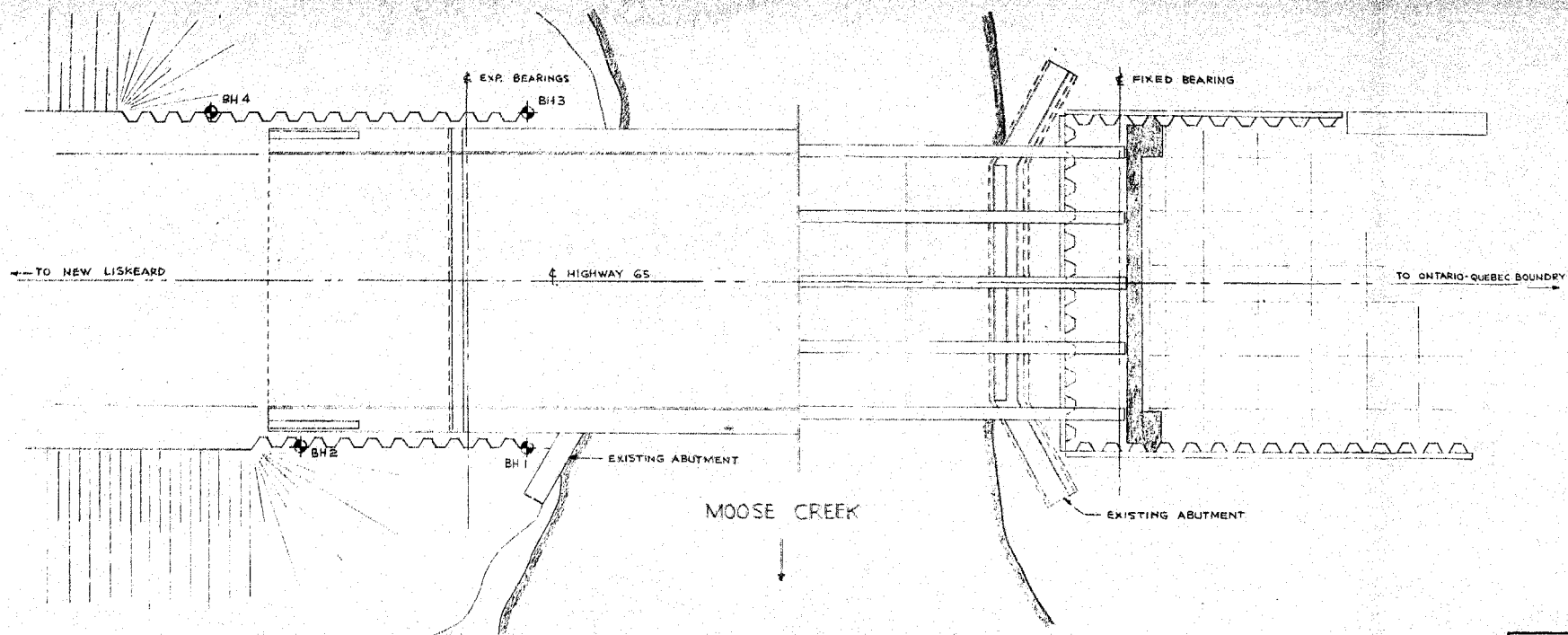
PEN TEST HAMMER WEIGHT — LB. DROP — INCHES

SOIL PROFILE			SAMPLES		DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT L_L PLASTIC LIMIT P_L WATER CONTENT W			
ELEVATION DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.	ELEVATION SCALE	SHEAR STRENGTH C , LB / SQ FT.		WATER CONTENT, PER CENT	
587.8	GROUND LEVEL					588.0				
586.8	EXCAVATED BY HAND - COBBLES AND BOULDERS UP TO 1.0' LIE IN A MATRIX OF CLAYEY TO SANDY FILL					587.4				
585.8						587.0				
584.8						586.6				
584.2						586.2				
583.7						585.8				
583.2						585.4				
582.7						585.0				
582.2						584.6				
581.7						584.2				
581.2						583.8				
580.7						583.4				
580.2						583.0				
579.7						582.6				
579.2						582.2				
578.7						581.8				
578.2						581.4				
577.7						581.0				
577.2						580.6				
576.7						580.2				
576.2						579.8				
575.7						579.4				
575.2						579.0				
574.7						578.6				
574.2						578.2				
573.7						577.8				
573.2						577.4				
572.7						577.0				
572.2						576.6				
571.7						576.2				
571.2						575.8				
570.7						575.4				
570.2						575.0				
569.7						574.6				
569.2						574.2				
568.7						573.8				
568.2						573.4				
567.7						573.0				
567.2						572.6				
566.7						572.2				
566.2						571.8				
565.7						571.4				
565.2						571.0				
564.7						570.6				
564.2						570.2				
563.7						569.8				
563.2						569.4				
562.7						569.0				
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561.2						567.8				
560.7						567.4				
560.2						567.0				
559.7						566.6				
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558.7						565.8				
558.2						565.4				
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550.2						559.0				
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547.7						557.0				
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542.7						553.0				
542.2						552.6				
541.7						552.2				
541.2						551.8				
540.7						551.4				
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533.7						545.8				
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499.7						518.6				
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498.7						517.8				
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497.7						517.0				
497.2										

RECORD OF BOREHOLE 4

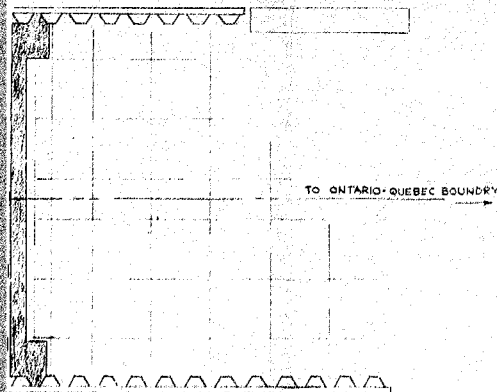
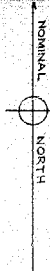
LOCATION SEE FIG. 1 BORING DATE JUNE 5-6, 1961 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER 5X CASING
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES

SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT		LIQUID LIMIT L_L PLASTIC LIMIT P_L P_L W L_L WATER CONTENT W	
ELEVATION DEPTH	DESCRIPTION	START. PLOT	NUMBER	TYPE		SHEAR STRENGTH C , LB./SQ.FT.		WATER CONTENT, PER CENT	
581.4	GROUND LEVEL				581.4				
580.8	EXCAVATED BY HAND - 4 BOULDERS 3.7' IS. LG AND 6.1' IN SIZE RESPECTIVELY IN A MATRIX OF CLAYEY FILL				580.8				
580.4					580.4				
580.0					580.0				
579.6					579.6				
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510.8					510.8				
510.4					510.4				
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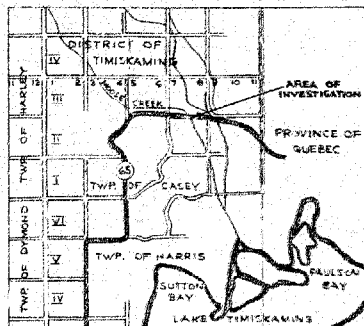


DRWG. No.	
D 4753-1	FOUNDATI CORPORATI OF HIGHWA BRIDGE, 2

FIXED BEARING



EXISTING ABUTMENT



KEY PLAN
SCALE: 1" TO 2 MILES

LEGEND

◆ BOREHOLE IN PLAN

REFERENCE	
DRWG. No.	DESCRIPTION
D 4753-1	FOUNDATION OF CANADA ENGINEERING CORPORATION LIMITED FOR DEPARTMENT OF HIGHWAYS ONTARIO - MOOSE CREEK BRIDGE, GENERAL ARRANGEMENT.

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO
BORINGS THROUGH APPROACH FILL
MOOSE CREEK BRIDGE
HIGHWAY 65
NEAR JUDGE
BORING PLAN
ONTARIO

GOLDER & ASSOCIATES
CONSULTING CIVIL ENGINEERS

DATE: JUNE 26, 1961 SCALE: 1" TO 8'-0"

MADE
J.A.

CHKD.
J.S.

APPD.
K7

FIGURE 1