

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 40P3-46

W.P. No. _____

CONT. No. _____

W. O. No. _____

STR. SITE No. _____

HWY. No. 4

LOCATION THAMES RIVER,
N. of LONDON

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

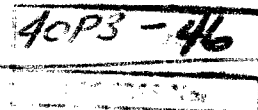
NONE

REMARKS: _____

RACEY, MacCALLUM AND ASSOCIATES LIMITED

ALSO KNOWN AS RACEY, MacCALLUM AND ASSOCIATES LTD.

Consulting Engineers
AND ASSOCIATED STAFF



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20 CARLTON STREET

Reference No. S-500/T-593

Toronto 2, Ontario,
March 21, 1957.

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

Attention: Mr. S. McCombie

RE: FOUNDATION INVESTIGATION FOR THE
PROPOSED THAMES RIVER BRIDGE ON
HIGHWAY NO. 4, NORTH OF LONDON, ONT.

Dear Sirs:

We have completed the foundation investigation at the above noted bridge site, and a summary of our conclusions and recommendations arising from this investigation is as follows:

(1) The abutments at this bridge site can be adequately supported through the use of footings bearing directly on the glacial till subsoil. A conservative estimate of the safe bearing capacity of this material has been noted as 1 ton/sq.ft. A final settlement of 1-1/4 inches has been estimated for a gross footing pressure of 2-1/2 tons/sq.ft.

(2) It is recommended that the abutment footings be founded at a minimum depth of five feet below the existing stream bed elevation. This corresponds to a footing elevation of 760.0 and insures adequate protection against river scour.

(3) Attention has been drawn to the possible spilling out of material from below the existing bridge footings during excavation for the new abutment footings. This problem can be virtually eliminated through moving the proposed bridge centre line a distance of 10 feet downstream from its present location. If this relocation is not possible, provision will have to be made to protect the existing footings during excavation through the use of sheet piling and adequate shoring.

A detailed description of the soil types encountered, as well as an outline of bearing capacity considerations, are included in the body of this report.

Reference No. S-500/T-592

- 2 -

March 21, 1957

Should any queries arise out of the reading of this report, we shall be pleased to discuss them with you at your convenience.

Yours very truly,

RACEY, MACCALLUM AND ASSOCIATES LIMITED

W. A. Trow
W. A. Trow, P. Eng.,
Divisional Soils Engineer

WAT/KA
Inclosures

Department of Highways of Ontario
Toronto, Ontario.

REPORT OF FOUNDATION INVESTIGATION
FOR THE
PROPOSED THAMES RIVER BRIDGE ON
HIGHWAY NO. 4, NORTH OF LONDON, ONTARIO.

Reference: S-507/T-593

Racey, MacCallum & Associates Limited

March 21st, 1957.

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March 21st, 1957. 5

PERMITS FOR THE CONSTRUCTION OF
THE PROPOSED BRIDGE OVER THE RIVER
ON HIGHWAY NO. 4, NORTH OF LONDON,
ONTARIO.

An investigation to assess the bearing capacity of the subsoil at the above noted river crossing, has been completed. A brief description of the work carried out as well as the recommendations pertaining to safe allowable footing loads to the subsoil are included in the body of this report.

LOCATION OF SITE

The proposed structure is parallel to and positioned immediately adjacent to the south side of an existing bridge crossing the Thames River. The river crossing is located approximately three quarters of a mile north of London, on Highway No. 4. The new abutment locations are presently occupied by the foundation members of what appears to have been the original river crossing structure. The proposed bridge centre line is approximately at right angles to the direction of flow of the river and crosses the river at a section that coincides with the exit end of a fairly broad meander loop. Sediments are presently being deposited on the east bank of the river and slight erosion is evidenced on the west bank. On the basis of the present river bottom contours, observations taken at the site of existing abutment conditions, and the very dense nature of the glacial till through which the river flows, the river is judged to be relatively inactive with respect to scour and fill characteristics.

DESCRIPTION OF FIELD WORK

The drilling and soil sampling equipment arrived on the site February 8th, 1957, and boring was started on borehole No. 7 that afternoon. The locations of the boreholes and penetration profiles were laid out under the supervision of a soils engineer. A trained soils inspector remained on the site and supervised the boring and taking of samples. The field work was completed February 20th, 1957.

The conventional method of advancing a borehole by alternately driving and washing standard diamond drill casing, could only be used to a depth of 7 1/2 feet, even with the aid of blasting with dynamite. It was necessary to resort to the use of diamond shoe bits on the casing to penetrate the dense boulder till. Standard split spoon samples were taken at five foot depth intervals, but sample recovery was very poor and the presence of boulders and gravel sizes within the till strata precludes the direct use of penetration resistance in blow per foot in any of the published empirical bearing capacity relationships.

In addition to the taking of split spoon samples, modified falling head field permeability tests were carried out during the drilling of the borehole to gain information on the location of

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March 21st, 1957. 6

DESCRIPTION OF FILL BANK (Cont'd)

relatively permeable strata. Water levels in the open boreholes were also recorded each morning and evening.

The dynamic penetration profiles were obtained by driving a 2 inch diameter cone, from ground surface to refusal depth, with a driving energy equal to that used in taking split spoon samples. Resistance has been recorded as the number of blows per foot of penetration. All samples recovered were placed in moisture proof containers and despatched to the Toronto Laboratory, for visual examination and description. The location of the boreholes and penetration profiles are shown on enclosure No. 1.

DISCUSSION OF SOIL TYPES ENCOUNTERED

The results of the two borings carried out at this site show that the subsoil is of glacial origin. The recorded variations in penetration resistances and the changes in the silt to clay percentages noted in samples recovered is typical of the heterogeneous glacial till deposits in this area. Borehole No. 2 showed that the very hard stratum of clayey till was overlain by 6 feet of relatively soft compressible material judged to be fill. The upper horizon of the till stratum has been established as elevation 769.0 at this borehole and drilling was discontinued at elevation 769.6, still within the same material. The depths at which boulders were encountered have been noted on enclosure No. 2, and core recovered from these boulders showed them to be limestone.

Borehole No. 3 on the east side of the river showed that the competent till layer was overlain by approximately 6 feet of material which has been described as a reworked till, i.e. a material different from the underlying layer only in that it exhibits an absence of the fine silt and clay sizes which have presumably been removed by water action. The upper surface of the competent till stratum was intersected at elevation 770.0 and the thickness was proven to a depth of 40.5 feet or elevation 735.5.

The penetration profiles showed refusal depths at 10 feet and 12 feet below surface at holes PC 1 and PC 2 respectively. A marked increase in resistance to penetration was noted at 8 feet below surface on both these profiles. The increase in resistance very likely coincides with the upper horizon of the dense underlying till.

SEALING CAPACITY EVALUATION

As evidenced by the very high resistance offered to the drilling and sampling of the glacial till strata, the problem of

Report: No. S-20/1-501

March 21st, 1957.

BEARING CAPACITY ESTIMATION (cont'd)

abutment support is not one of adequate bearing capacity but rather one of determining the depth of footing required to protect the abutment from river scour and subsequent foundation failure.

The subsoil profile proven by borehole No. 2 showed the till layer to have sufficient clay content to behave as a cohesive material with respect to strength and deformation properties. The consistency has been determined as very stiff to hard with a minimum shear strength of 2,000 p.s.f. This conservative strength figure gives rise to a safe allowable bearing capacity of $\frac{1}{2}$ tons per square foot incorporating a safety factor of 3 and neglecting any influence of footing shape or depth. The actual footing pressures will probably be of the order of $2\frac{1}{2}$ tons per square foot.

To arrive at an estimate of the order of settlement to be expected under a footing load of $2\frac{1}{2}$ tons per square foot, a footing width of 5 feet has been used and soil conditions as evidenced by borehole No. 2 have been assumed to a depth of 32 feet below footing elevation. A relationship between footing size, net footing loads, subsoil strength and compressibility characteristics, and resultant final settlement, has been developed by Skempton (1). Mathematically this relationship is expressed as follows:-

$$S = 5 B \frac{p_a}{p_{nf}} \times 1/K_v \times C$$

where S = settlement

B = footing width

p_a = actual net footing pressure

p_{nf} = net footing pressure to cause ultimate shear failure

K_v = reciprocal of modulus of compressibility

C = undrained shear strength

For a footing width of 5 feet and K_v assumed at 100 (a very conservative estimate) a settlement of $1\frac{1}{4}$ inches is predicted.

The east abutment footings are founded on a more granular till and values of bearing capacity and settlement are respectively greater and less than the values estimated for the west abutment.

The practice of determining the depth of footings to avoid detrimental scouring action is largely based on empirical rules tempered by the consideration of such factors as stream velocity, channel shape, rise and fall of water surface during flood periods, as well as the in-situ density, particle size and shape of river bed materials. The high degree of density of the river bed till stratum at this site plus the previously mentioned relative inactivity

REPORT No. E-500/T-193

March 21st, 1957.

FOUNDING CAPACITY EVALUATION (Cont'd)

of the river with respect to scour and fill indicate that footings could be founded at a depth five feet below lowest river bed elevation. This establishes the abutment footings at elevation 760.0.

Founding both footings at the same elevation results in more apparent scour protection at the east abutment than at the west one and it might seem justifiable to found the east abutment at a higher elevation. This is not considered advisable because future channel improvements or upstream structures could result in very different scour and fill characteristics and the present apparent additional protection at the east abutment could be readily reduced.

Reference to D.H.O. drawing number D 3060-A shows that the abutments for the new bridge are to be placed as close as practically possible to the existing bridge abutments. No information is available regarding the depth of the footings of the existing bridge abutments and should they be founded at an elevation above 760.0 consideration will have to be given to the effect of excavating to place the new footings. It would be desirable to keep the excavation sufficiently far from the existing footing to preclude any disturbance without having to resort to sheet piling and shoring operations. This limiting distance can be estimated by considering the existing footing to be founded at elevation 770.0 or below and calculating the point of emergence, at this elevation, of a Rankine passive failure plane. This plane is drawn at an angle of $(45 - \phi/2)$ from the point of intersection of a vertical line through the corner of the existing footing and a horizontal plane coincident with elevation 760.0. This gives a theoretical distance of 21 feet assuming an angle of shearing resistance of 40° for the subsoil and also assuming that the load on the existing footing equals its ultimate value. The actual footing pressure is more nearly equal to $1/3$ to $1/4$ of the ultimate capacity of the foundation material and thus the clearance distance noted is much too large. A value of 10 feet is considered adequate and consistent with the order of safety factors normally used in design.

The problem of excavation adjacent to the existing abutments is more critical for the east than the west abutment because of the more granular nature of the subsoil at the east footing location. A clearance distance of slightly less than 10 feet would be required for the more cohesive soil conditions at the west abutment. If it is not possible to alter the proposed bridge centre line to realize the clearance distance noted, it will be necessary to drive sheet piling prior to excavating and to provide adequate shoring during excavation to prevent material from spilling out from under the perimeter of the existing bridge abutment footings.

REPORT: No. 8-500/T-593

March 21st, 1957. 9

SUMMARY AND RECOMMENDATIONS

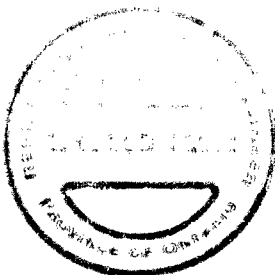
1) The results of the foundation investigation at this bridge site show that the proposed abutments can be founded on footings supported directly on the subsoil which has been classified as a glacial till. A conservative safe bearing capacity of 4 tons per square foot has been calculated for the subsoil at this site.

2) It is recommended that the new bridge abutment footings be founded at elevation 760.0. This elevation corresponds to a footing depth of five feet below the lowest stream bed elevation at this river section, and is considered adequate depth protection against scour. Both footings should be founded at this elevation.

3) Particular attention must be paid to the possibility of spilling out of material from below the existing bridge footings during excavation for the new footing, this can, of course, only occur if the existing footings are founded above elevation 760.0. Should this be the case, sheet piling and adequate shoring procedures will have to be employed. It has been calculated that this problem can be virtually eliminated through moving the proposed bridge centre line a distance 10 feet downstream from its present location.

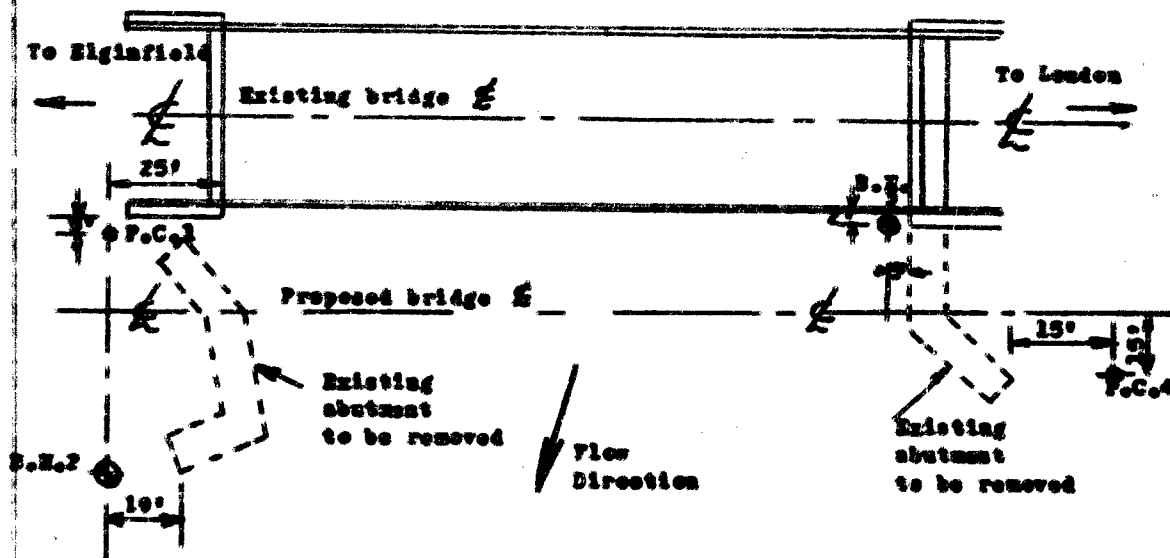
L. G. Soderman

L. G. Soderman.



Prop. 81

10



SKETCH SHOWING LOCATION OF BOREHOLE
AND PENETRATION PROFILES AT TIMES
RIVER BRIDGE, HIGHWAY NO. 4, 3/4
MILE NORTH OF LONDON, ONTARIO.

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division
Engineering Data Sheet for Boreholes 2

Project: Proposed Thames River ridge, Hwy No. 4.
Location: Approximately 3/4 mile north of London, Ont.
Hole Location: As shown on Enclosure No. 1.
Hole Elevation and Datum: 789.0 Geometric
Field Work Begun: Ended

Field Supervisor: C. White
Driller: W. Linton
Firm: L.A.S.
Checked:

URGENT

Sampling Method

2' Soil with hole

2' Shallow hole

Sampling Equipment

2' Soil with hole

2' Soil with hole

Core

Sampling

Unconsolidated compression (C_u)Time and 1' and 2' hole (C_u)

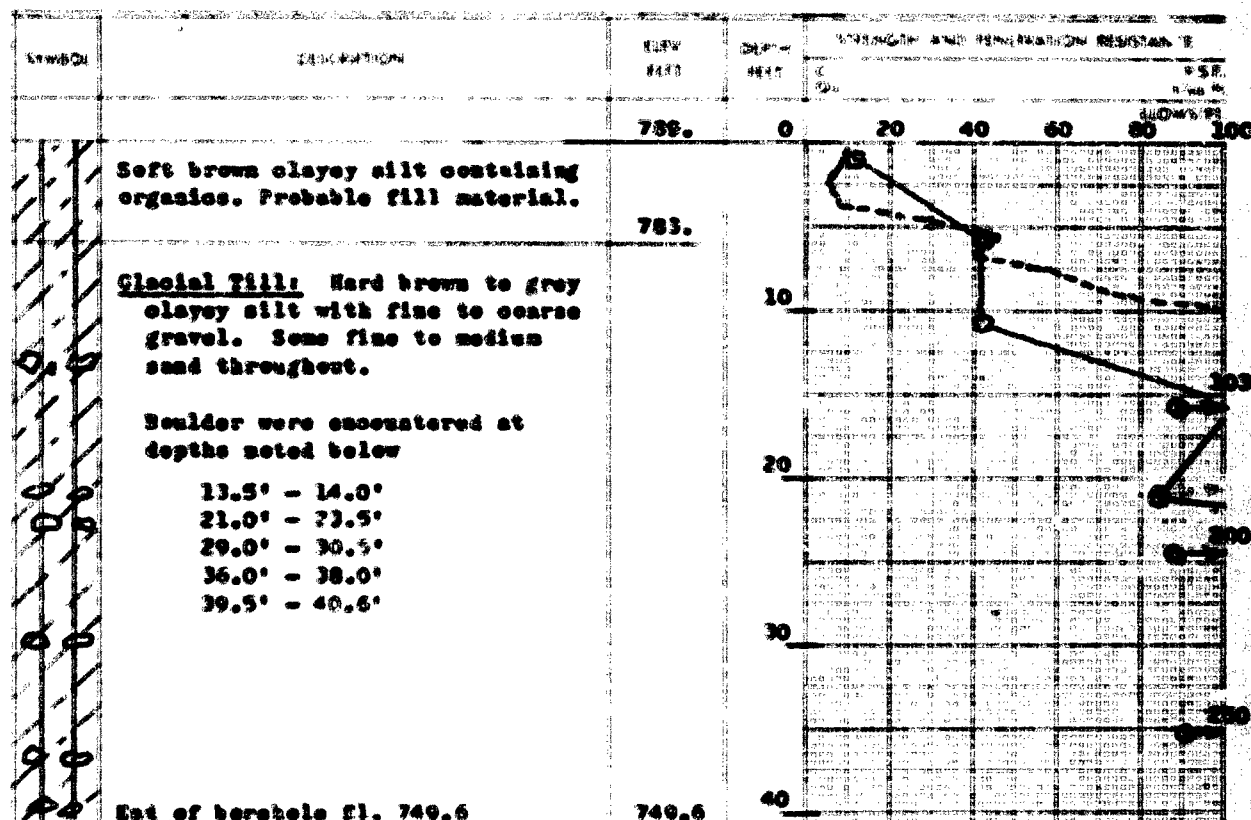
Consolidation

Natural moisture and

Liquid limit

Plastic limit

Plasticity Index

**Notes:**

- 1) 8" casing driven to elevation 779.0
- 2) 8" casing advanced by drilling from 771.0 to 765.0.
- 3) Borehole ended in boulder at elevation 749.6.

CONSTRUCTION

Engineering Data Sheet for Composite: 2

Field Supervisor: **W. White**
 Officer: **A. Linton**
 Precinct: **100 S.**
 Case Number:

Pinus sp.

CONFIDENTIAL

NAME

UNIT NO

PCN

NAME

DATE

TIME

LOCATION

REMARKS

ACTION

STATUS

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RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Estimating 3

Field Supervisor	C. White
Driver	R. Linton
Pump	L.C.S.
Checked	

and Thames Silver Bridge, Hwy No. 1.
approximately 3/4 mile north of London, Ont.
As shown on Enclosure No. 1.
and between T.T.C. Geodetic.

PLATE 1

Legend

$$\text{Superficial } \text{H}_2\text{O} \text{ and } \text{H}_2\text{O} \text{ in } \text{H}_2\text{O}$$

7. 10. 1991

2. *Senecio* *Hydrocotyle*

1997年12月 第10卷第12期

Figure 1. Schematic representation of the experimental design. The subjects were divided into two groups: the control group and the experimental group. The control group was divided into two subgroups: the control group and the control group. The experimental group was divided into two subgroups: the experimental group and the experimental group.

2. 2. 1948

^a $\chi^2 = 0.67$, $p = .81$.

Keywords: *depression, mood, mood disorder, mood disorder with anxiety, mood disorder without anxiety, mood disorder with anxiety, mood disorder without anxiety, mood disorder with anxiety, mood disorder without anxiety*

THE UNIVERSITY OF CHICAGO

Wang, Z. 2003. *China's Foreign Trade and Investment*. Beijing: China Development Press.

Summary

[illegible]

Journal of Management Education 36(7)

1992

DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE C.	PSF 1 sq ft INCHES
	776.			
base brown silty sand to medium gravel. reworked glacial till.	770.			
Fill: Very dense gray silt with fine to coarse Trace of clay through- le boulder at elevation		10		
		20		
		30		
orchole El.	736.5	40		
using advanced by drilling surface to elevation				
ole ended in probable or.				
table elevation at sion 773.0.				

CONFIDENTIAL		SAMPLE	NATURAL UNIT WT P.C.
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Hole Begun: _____ Foundation Engineering Division

Hole Ended: _____ Engineering Data Sheet for Borehole: P.C. NO. 2

Job Name: Proposed Thames River Bridge, Hwy. No. 4.

Job Located: Approximately 3/4 mile north of London, Ont.

Hole Located: As shown on Enclosure No. 1.

Hole Elevation: _____ Datum: _____

L.G.S.
Checked by

DEPTH	CL	TEMP	WATER	DESCRIPTION	TIME OF DAY	DATE	MONTH	YEAR
0								
10								
20								

Notes:

- 1) Refusal at depth of 10' below surface.
- 2) Penetration resistance recorded as blows/ft. penetration of 2" diameter cone driven with energy of 350 ft. lbs. per blow.

20 40 60 80 blows/ft.

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