

68 - F - 222 M

GLENBROOK BRIDGE

CHARLOTTE NBURGH

BA 2825
Site 31-189

H. Q. GOLDER & ASSOCIATES LTD.

SOIL AND FOUNDATION ENGINEERS
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March 8, 1968.

68-F-222M

R. M. Kostuch Associates Ltd.,
Consulting Engineers,
238 King Street West,
P. O. Box 663,
Brockville, Ontario.

Attention: Mr. R. M. Kostuch, P. Eng.

RE: Soil Investigation,
Proposed Glenbrook Bridge,
Township of Charlottenburgh.

Dear Sirs:

This letter reports the results of a soil investigation carried out at the above bridge site. The purpose of this investigation was to determine the soil conditions at the site and based on this information to make recommendations for foundation design for the proposed bridge.

PROCEDURE

The field work for this investigation was carried out on March 1 and 2, 1968. One borehole and one dynamic penetration test was put down with a machine drill rig supplied and operated by the F. E. Johnston Drilling Co. Ltd., Ottawa. It was not possible to drive an accompanying dynamic penetration test beside the borehole due to the bouldery nature of the approach fill. After several attempts a dynamic penetration test was driven through the bouldery fill on the north side of the bridge. In situ vane testing was carried out in the borehole. The field work was supervised by a member of our engineering staff.

The location of the boring and the penetration test,

together with a stratigraphic section along the centerline of the proposed bridge, are shown on Figure 1. A detailed log of the borehole is given on the Record of Borehole sheet following the text of this report.

The soil samples were brought to our laboratory for detailed examination. The results of a laboratory test are shown on Figure 2.

The elevations given in this report are referred to the bench mark located on the railing at the southeast end of the bridge. The elevation of this bench mark was given to us as 104.98, as referred to a local datum.

SITE AND GEOLOGY

The bridge site is located some 7 miles north east of Cornwall and about 4 miles west of Williamstown. The topography of the area surrounding the South Raisin River is relatively flat.

From available geological information it is known that the area is underlain by clay deposited in the Champlain Sea (Leda clay). The clay covers glacial till, and shaly limestone of the Ottawa formation.

SUBSURFACE CONDITIONS

The detailed soil stratigraphy encountered in the borehole is given on the Record of Borehole sheet. Following is a summarized account of the soil conditions.

The approach embankments to the bridge consist of a sand, gravel and boulder fill which at the borehole location was 8.3 feet thick. The embankment fill is underlain by a stratum of firm grey silty clay some 20 feet thick. The upper 5 feet of this clay stratum has been weathered to a stiff grey brown crust. As measured by the ratio of undisturbed to remoulded vane values, the sensitivity of the clay is 10, which is typical for the sensitive Leda clay in this area. The shear strength values obtained by vane testing in the grey portion of the clay ranged from 720 to 840 lb/sq.ft. In the grey brown crust, the values of the shear strength were 960 and 1080 lb/sq.ft.

At a depth of 28 feet below grade, a stratum of dense to very dense silty sand till was encountered. The stratum was proved for a depth of 12 feet. The results of a grain size distribution test on a sample of this till is shown on Figure 2.

A piezometer was installed in the borehole in the till stratum. The water level in the piezometer on March 5, 1968, 4 days after completion of boring was at elevation 95 or about 2 feet above ice level.

PROPOSED BRIDGE STRUCTURE

a) General

It is understood that it is proposed to replace the existing narrow Glenbrook bridge over the South Raisin River by a one span simply supported bridge, 60 feet in length and two lanes in width. During reconstruction of the bridge crossing, the approach embankments will be raised by 2 to 3 feet above existing roadway level.

b) Foundations

The average shear strength of the clay stratum at and below the approximate founding elevation of 86, is about 800 lb/sq.ft. The allowable bearing value, incorporating a factor of safety of 3 against ultimate failure, is therefore of the order of 1,500 lb/sq.ft. for footings founded in the clay at this elevation. With this low allowable bearing pressure, the use of spread footings for this bridge is not considered feasible. Any significant increase in bearing pressure in excess of this amount given above would certainly exceed the preconsolidation pressure of the clay and would result in large settlement of this highly compressible deposit.

It is therefore recommended that the foundation loads be transferred to the underlying dense to very dense silty sand till by the use of piles, driven to practical refusal in the essentially granular till stratum. It is considered that a timber pile would be the most economical foundation for this bridge. For a timber pile, with a 14 inch butt and 9 inch tip diameter, driven to a final set of about 5 blows to the inch with a hammer having a rated capacity of about 12,000 ft.lb./blow, the design load may be taken as 25 tons per pile.

Closed end abutments should be backfilled for a distance of at least 5 feet horizontally with a well compacted, free draining and non-frost-susceptible granular material. Provision should be made for drainage from the backfill to prevent hydrostatic or ice pressure build up behind the walls. With full effective drainage of the backfill the abutments may be designed using a total unit weight of 135 lb/cu.ft. and an active earth pressure coefficient, K_a , = 0.3, providing some minor movement of the top of the abutment walls can be tolerated.

c) Approach Embankments


The proposed grade is to be raised some 2 to 3 feet above the existing roadway and some 10 feet above the surrounding ground surface. Based on the undrained shear strengths measured in the silty clay stratum at the site, the factor of safety against instability of the side slopes and the vertical front part of the proposed embankment at the abutment would be about 2 which is considered adequate.

From the shear strength values obtained from vane testing, it is considered that the clay has been preconsolidated by about 1 ton/sq.ft. in excess of the present overburden pressure. The addition of 2 to 3 feet to the approach embankment height would impose a loading of about 0.2 tons/sq.ft. This loading, within the preconsolidation range of the clay, would result in only a minor recompression of the clay amounting to 1 or 2 inches of settlement of the embankment surface.

We trust that this report contains sufficient information for your design purposes. If we can be of any further service to you on this project, please call us.

Yours very truly,

H. Q. GOLDER & ASSOCIATES LTD.



F. J. Heffernan, P. Eng.

FJH/ml
68755B
March, 1968.

GOLDER & ASSOCIATES



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPES

<i>AS</i>	auger sample
<i>CS</i>	chunk sample
<i>DO</i>	drive open
<i>DS</i>	Denison type sample
<i>FS</i>	foil sample
<i>RC</i>	rock core
<i>ST</i>	slotted tube
<i>TO</i>	thin-walled, open
<i>TP</i>	thin-walled, piston
<i>WS</i>	wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

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.

<i>WH</i>	sampler advanced by static weight—weight, hammer
<i>PH</i>	sampler advanced by pressure—pressure, hydraulic
<i>PM</i>	sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

<i>Relative Density</i>	<i>N, blows/ft.</i>
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) *Cohesive Soils*

<i>Consistency</i>	<i>c_u, lb./sq. ft.</i>
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer ¹
<i>Q</i>	undrained triaxial ²
<i>R</i>	consolidated undrained triaxial ²
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	field vane test

NOTES:

¹Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

²Undrained triaxial tests in which pore pressures are measured are shown as \bar{Q} or \bar{R} .

LIST OF SYMBOLS

I. GENERAL

π	= 3.1416
e	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of a
$\log_{10} a$ or $\log a$	logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress ($\bar{\sigma}$ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{xy}	shear strain
ν	Poisson's ratio (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) Unit weight

γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) Consistency

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_S	shrinkage limit
I_L	liquidity index = $(w - w_P) / I_P$
I_C	consistency index = $(w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

(c) Permeability

h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
k	coefficient of permeability
j	seepage force per unit volume

(d) Consolidation (one-dimensional)

m_v	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
C_c	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
c_c	coefficient of consolidation
T_v	time factor = c_d / d^2 (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion
ϕ'	effective angle of shearing resistance, or friction
c_u	apparent cohesion*
ϕ_u	apparent angle of shearing resistance, or friction
μ	coefficient of friction
S_i	sensitivity

$\left. \begin{array}{l} \text{in terms of effective stress} \\ \tau_f = c' + \sigma' \tan \phi' \end{array} \right\}$

$\left. \begin{array}{l} \text{in terms of total stress} \\ \tau_f = c_u + \sigma \tan \phi_u \end{array} \right\}$

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

RECORD OF BOREHOLE 1

LOCATION See Figure BORING DATE MAR 1, 1968 DATUM LOCAL
 BOREHOLE TYPE WASTE BORING BOREHOLE DIAMETER 8.5 INCHES
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT LB. DROP INCHES

SOIL PROFILE				SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FT. -----					COEFFICIENT OF PERMEABILITY K, CM. / SEC.					ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEV.N. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FT.		SHEAR STRENGTH C _u , LB. / SQ. FT. VAN DER NAGT, ⊕ DATA					WATER CONTENT, PERCENT						
							100	150	200	250	300	W _p	W	W _L				
101.0	GROUND LEVEL															GROUND LEVEL		
95.0	COMPACT BROWN SAND, GRAVEL AND BOULDERS (EMBANKMENT FILL)		1	8.5	100													
			2	"														
			3	"														
			4	"														
92.5	STIFF GREY BROWN SILTY CLAY		5	8.5	90											SAND FILL		
88.0				6	"													
83.0				7	"	80												POLY TUBING
				8	"													
	FIRM GREY SILTY CLAY SOME ORGANIC MATERIAL, TRACE OF SHELLS		9	8.5	70											PIEZOMETER		
				10	"													
				11	"	57												
72.5																		
29.2	DENSE TO VERY DENSE GREY SILTY SAND, SOME GRAVEL, TRACE OF CLAY (SILTY SAND TILL)		12	8.5												CAVED IN MATERIAL		
				13	"													
61.0	END OF HOLE															W.L. IN PIEZOMETER AT ELEV. 95.1 ON MAR. 5, 1968		
40.0																		

15--5 Percent axial strain at failure

15-0.5 Percent axial strain at failure

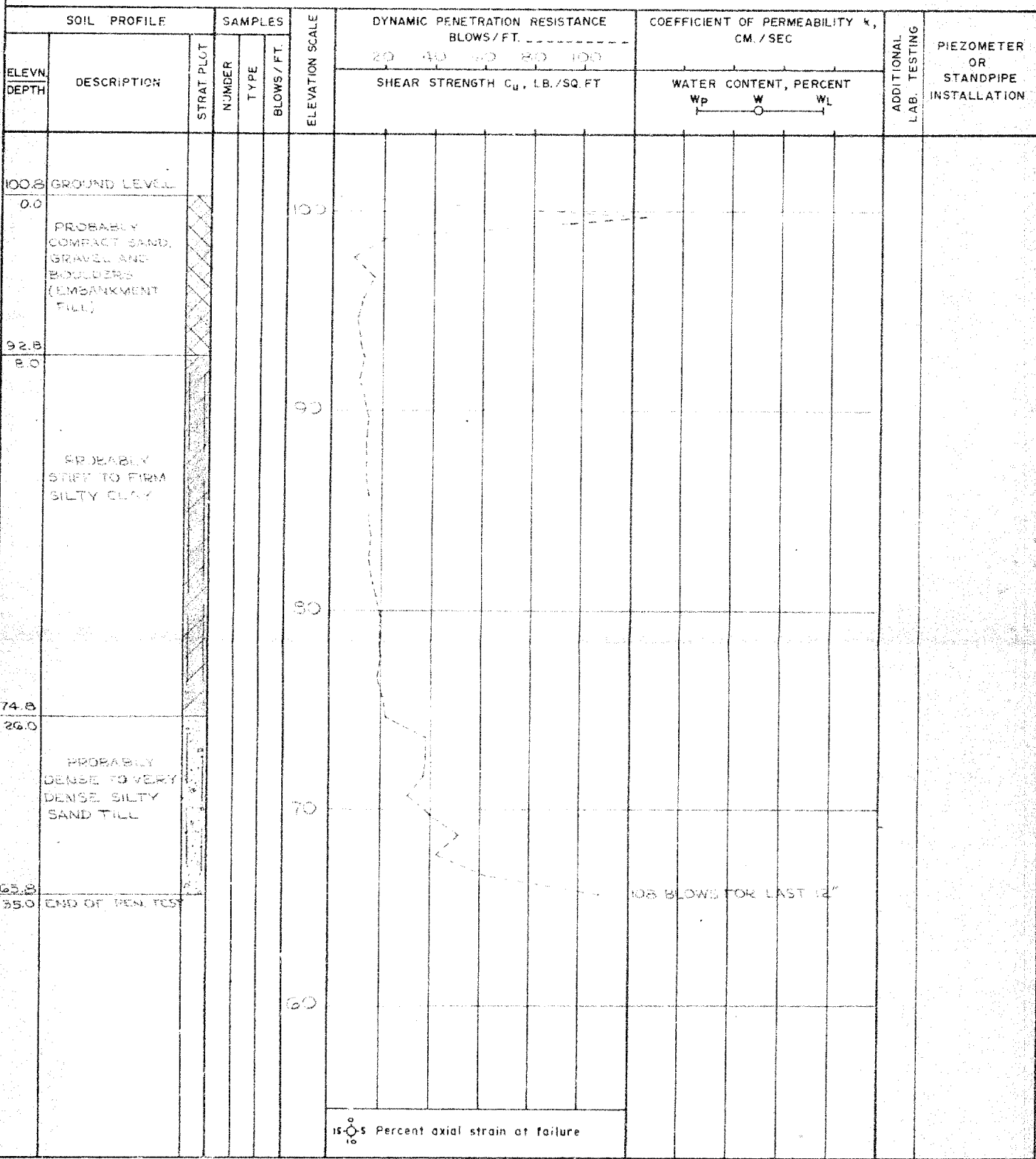
VERTICAL SCALE
1 INCH TO 5'

GOLDER & ASSOCIATES

DRAWN D.N.
CHECKED F.S.H.

PEN TEST
RECORD OF ~~BOREHOLE~~ A

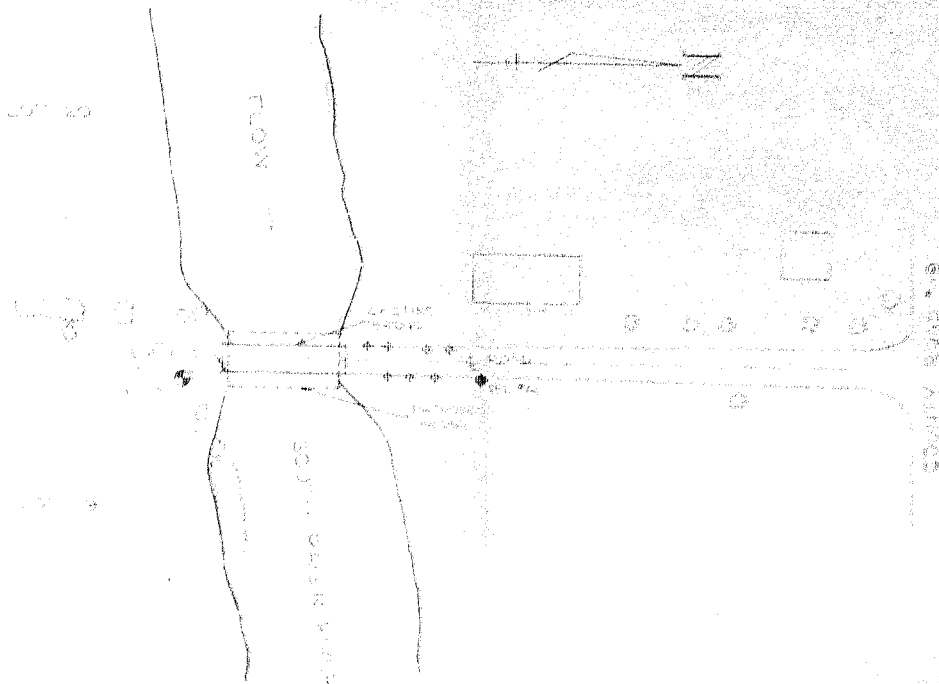
LOCATION See Figure 1 BORING DATE MAR 2, 1947 DATUM LOCAL
BOREHOLE TYPE HELL PLOT BOREHOLE DIAMETER APPROX. 2" DIA. CONE
SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
1 INCH TO 5'

GOLDER & ASSOCIATES

DRAWN D.N.
CHECKED F.H.

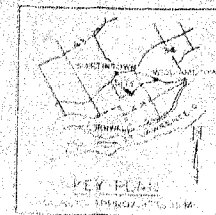


PLAN

304-17-1776-203

BORING PLAN & SOIL STRATIGRAPHY (GLENBROOK BRIDGE)

FIGURE 1



- LEGEND**
- BOREHOLE IN PLAN
 - PENETRATION TEST IN PLAN
 - PENETRATION TEST, REFUSAL IN EMBANKMENT FILL, IN PLAN
 - BOREHOLE IN ELEVATION
 - PENETRATION TEST IN ELEVATION
 - WATER LEVEL IN ELEVATION

STRATIGRAPHY

- CLAYEY BROWN SAND, GRAVEL AND BOULDERS (EMBANKMENT FILL)
- FINE GRAY SILTY CLAY SOME ORGANIC MATERIAL, TRACE OF SHELLS
- DENSE TO VERY DENSE GRAY SILTY SAND, SOME GRAVEL, TRACE OF CLAY (SILTY SAND TILL)

SECTION ALONG E OF ROAD

DATE: 1968
BY: J. B. GOLDER

SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE FULL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

DEFECTS AND NEGATIVE DUE TO
CORRECTION OF ORIGINAL DOCUMENT

Drawn, MARCH 6, 1968

GOLDER & ASSOCIATES

Made
Chkd
App'd

M.I.T. GRAIN SIZE SCALE

