

64-F-262M

SISLER BRIDGE

CAISTOR TWP

B.A. 1955

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER  
V. MILLIGAN  
L. G. SODERMAN

2444 BLOOR STREET WEST  
TORONTO 9, ONTARIO  
767-9201  
763-4103

October 22, 1964.

J. M. Tomlinson & Associates Ltd.,  
593 Brant Street,  
Burlington, Ontario.

18-730

Attention: Mr. D. J. S. Tefft, P.Eng.

64-F-262 M

RE: SOIL INVESTIGATION,  
PROPOSED SISLER BRIDGE,  
CAISTOR TOWNSHIP,  
LINCOLN COUNTY, ONTARIO.

Dear Sirs:

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

PROCEDURE

The field work was begun on September 30, 1964, using a mobile power auger supplied and operated by the F.E. Johnston Drilling Co. Ltd. The augers met refusal at a depth of 5 to 7 feet on boulders in several attempts. The boreholes were completed on October 5 and 6 using a standard drillrig. Two boreholes

were put down to a depth of about 25 feet and one borehole penetrated the grey dolomite rock for 3 feet by diamond drilling methods.

A detailed log for each boring is given on the Records of Boreholes following the text of this report. The locations of the borings together with a section of the inferred soil stratigraphy across the site are shown on Figure 1.

The elevations given in this report are referred to a local bench mark, the elevation of which is given as 84.88 on drawing 64.8.1 Prel. 2. The bench mark consists of a tack in the southwest wing of the existing bridge abutment.

#### SITE AND GEOLOGY

The site of the proposed bridge over the upper reaches of the Welland River is located at the first river crossing west of Lincoln County Road #44 about  $3\frac{1}{2}$  miles east of Caistorville in Caistor Township. The new bridge will be located between Lots 11 and 12 in the south part of Concession 2 and about 40 to 50 feet east of the present bridge.

The topography of the surrounding area is generally level to gently rolling. There is no distinct valley of the Welland River at this location. There is in general however a 15 to 20 foot bank on the outside curves of the river and about

a 5 foot bank followed by gently sloping land elsewhere along the river.

From available geological information it is known that the area is drift covered with a complex intermixture of tills and glacial Lake Warren clays and silts. The underlying bedrock is dolomite of the Guelph Formation.

#### SOIL CONDITIONS

A layer of dark grey organic sandy silt of 3 foot thickness was encountered at ground surface in borehole 2. This layer is believed to be a flood plain deposit of the river.

Underlying this sandy silt layer in borehole 2 and ground surface at borehole 1 is a stratum of brown to grey sandy silt till. This is the significant foundation stratum at the site. The sandy silt till extends to a depth of about 25 feet below ground level, that is, to about elevation 50. The till is comprised of sand, gravel, cobbles and boulders with a binder of silt and a trace of clay. A grain size distribution curve obtained on a sample from this stratum is shown on Figure 2.

Based on standard penetration tests carried out in the stratum, which gave "N" values generally in excess of 100 blows per foot, the relative density of the till is very dense.

The till is underlain by dark grey brown finely crystalline dolomite rock. The rock was cored in AX casing and AXT size for 3 feet. Full core recovery was obtained and the rock appears to be sound.

#### WATER CONDITIONS

At the time of the investigation, the water level in the Welland River at this bridge location was at about elevation 69. It is understood from local information that normal highwater level is about 10 feet above this level at about elevation 80.

Piezometers were installed in the boreholes after completion of the field work. The water level in the piezometers on October 8, 1964, was 3 to 4 feet below ground level. Insufficient time had elapsed, however, to allow the piezometers to come to equilibrium in the relatively impervious soil. It is considered that the stabilized water level at the borehole locations is slightly above the creek level at about elevation 70, near the boundary between the brown and grey sandy silt till.

#### DISCUSSION

##### General

It is understood that the existing 60 foot single span Sisler bridge will be replaced by either a single or double span

bridge some 80 to 90 feet in total length and located about 40 to 50 feet east or downstream from the existing crossing. It is also understood that the existing roadway grade and the bridge deck level will be raised by several feet.

#### Foundation Design

It is recommended that the abutments and the centre pier of the proposed bridge be founded on spread footings within the very dense sandy silt till. To provide adequate frost protection, the footings should be taken down at least 4 feet below the creek bed. This would provide some scour protection.

The "N" values for a significant depth below foundation level are generally in excess of 100 blows per foot. Based on these values, an allowable bearing pressure of 4 tons per square foot may be used in design of footings founded in the sandy silt till. With this bearing pressure there would be no detrimental settlement of the bridge abutment or pier footings, provided precautions are taken during construction to prevent loosening and softening of the sandy silt till at foundation grade. If a central pier is provided for the bridge, the footing should be taken down below any recent river deposits covering river bottom.

In the computation of sliding resistance between a rough concrete footing base and the undisturbed sandy silt till subsoil, a coefficient of friction of 0.4 may be used.

It is recommended that free draining granular backfill compacted in horizontal lifts of about 9 inches be placed behind the bridge abutments. The granular backfill should be non-frost susceptible and should extend horizontally from the back face of the abutment walls a minimum distance of 4 feet. Provision for drainage from this material should be made.

In the design of the abutment walls, it is recommended that an earth pressure coefficient,  $K_0$ , equal to 0.5 be used for the compacted granular backfill.

No steep slopes exist at the site at this time. There should be no stability problem with the approach embankments, which are to be raised several feet above the present level, provided they are constructed of properly compacted suitable material using side slopes of 2 horizontal to 1 vertical. Prior to placement of embankment fill, all topsoil and any organic pockets overlying the glacial till should be removed under the proposed embankments. Rip rap should be placed on a clean granular fill bed about 2 feet in thickness and the rip rap should extend for a height of 2 feet above the maximum high water level.

Construction Problems

Because of the low permeability of the till (about  $10^{-5}$  to  $10^{-6}$  cm/sec), it is considered that foundation excavations for abutments can be carried down a few feet below the water table in the sandy silt till without undue sloughing of the sides or loosening of the material due to seepage. The water inflow into the abutment excavation could be handled by pumping from sumps.

The foundation excavation for the central pier could be carried out within a perimeter clay dyke or braced closed sheeting. The dyke or sheeting should be constructed to a sufficient height to prevent flooding of the excavation during a flash run off period. If shallow pervious deposits are encountered covering the river bed either these should be removed prior to construction of the dyke, or the river bed blanketed outside the dyke for about 5 ft. to 10 ft. from the outer toe with at least 2 feet of dumped till to reduce seepage.

If sheeting is employed, it should extend below foundation level a depth equivalent to the maximum water head. The driving of this sheeting could be difficult as some of the sheet piles would be hung up on boulders. In this case, such boulder obstructions should be removed during excavation and the sheeting then driven to the required elevation.

To prevent softening of the till due to surface water or construction operations, it is recommended that the base of footing excavations, once foundation grade is reached, be inspected and immediately covered by a mud mat of lean concrete. If any soft spots are encountered at foundation grade they should be removed and replaced by lean concrete or highly compacted granular fill.

We trust that the above information is sufficient to enable you to proceed with the design of the proposed bridge structure. Should you require any additional information or if we can be of any further service to you, please call us.

Yours faithfully,

H. Q. GOLDER & ASSOCIATES LTD.

A handwritten signature in dark ink, appearing to read "F. J. Heffernan". The signature is fluid and cursive, with the first letters of the first and last names being capitalized and prominent.

F. J. Heffernan, P.Eng.

FJH/hb  
64121

**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</i> , blows/ft.
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<sub>u</sub></i> , lb./sq. ft.
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 <sup>1</sup>
<i>Q</i>	undrained triaxial <sup>2</sup>
<i>R</i>	consolidated undrained triaxial <sup>2</sup>
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	field vane test

### NOTES:

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

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

## LIST OF SYMBOLS

### I. GENERAL

$\pi$	= 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
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\bar{\sigma}$ is also used)
$\tau$	shear stress
$\epsilon$	linear strain
$\epsilon_{xy}$	shear strain
$\nu$	Poisson's ratio ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

#### (a) Unit weight

$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	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 = $c_v t / d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

#### (e) Shear strength

$\tau_f$	shear strength
$c'$	effective cohesion
$\phi'$	effective angle of shearing resistance, or friction
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\mu$	coefficient of friction
$S_i$	sensitivity

\*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 BOREHOLES 1 &amp; 2

LOCATION

See Figure 1

BORING DATE

DATUM


BOREHOLE TYPE POWER AUGER W/ SAMPLER

BOREHOLE DIAMETER 4 INCHES

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 14 LB. DROP 30 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. DEPTH	DESCRIPTION	STRAT PLAT	NUMBER	TYPE		SHEAR STRENGTH C <sub>u</sub> , LB./SQ.FT.					WATER CONTENT, PERCENT W <sub>p</sub> W      W <sub>L</sub>						
80.0	GROUND LEVEL								1/								
72.1	VERY DENSE BROWN TANN SILT, SOME GRAVEL, SCATTERED ROOTS, BUT NO TRACE OF CLAY (SANDY SILT TILL)		1	100													
6.8			2	100													
			3	100													
			4	100													
			5	100	70												
	VERY DENSE GREY SANDY SILT, SOME GRAVEL, COARSE BOULDERS, TRACE OF CLAY (SANDY SILT TILL)		6	100													
			7	100													
			8	100													
26.0	END OF HOLE																

 Percent axial strain at failure

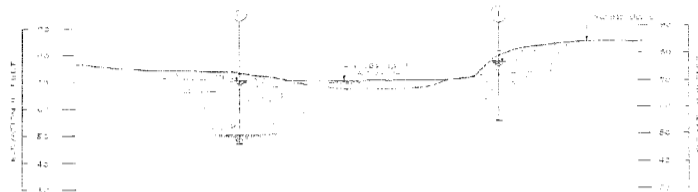
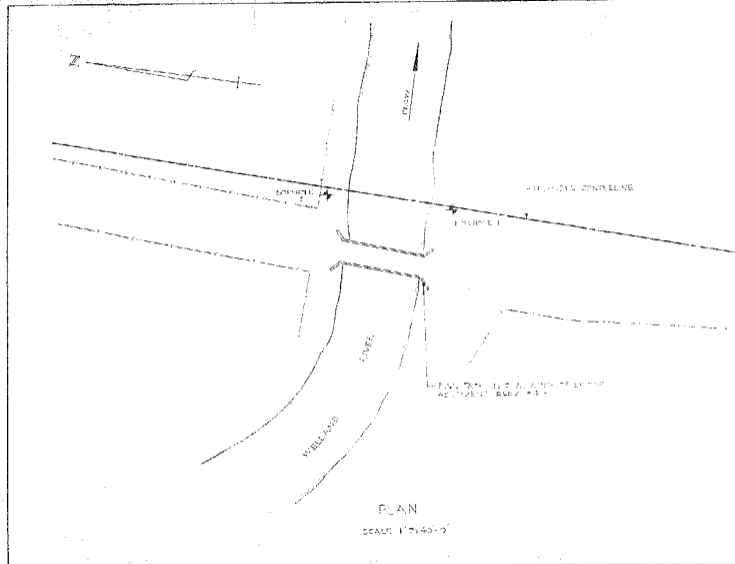
VERTICAL SCALE

1 INCH TO 10' = 1"

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DRAWN

CHECKED F.H.

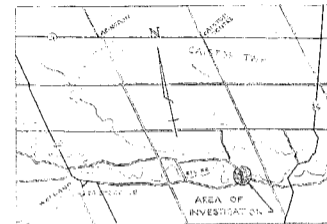


LEGEND

- BOREHOLE IN PLAN
- BOREHOLE IN ELEVATION
- WATER LEVEL IN BOREHOLE

STRATIGRAPHY

- SAND AND GRAVEL
- SANDY SILT TILL
- SILTY SAND TILL
- SILTY CLAY TILL
- SAND AND GRAVEL WITH DRIFTED BOULDERS
- SAND AND GRAVEL WITH DRIFTED BOULDERS



REFERENCE: U.S. GEOLOGICAL SURVEY, BULLETIN 1064, 1934, P. 1064.

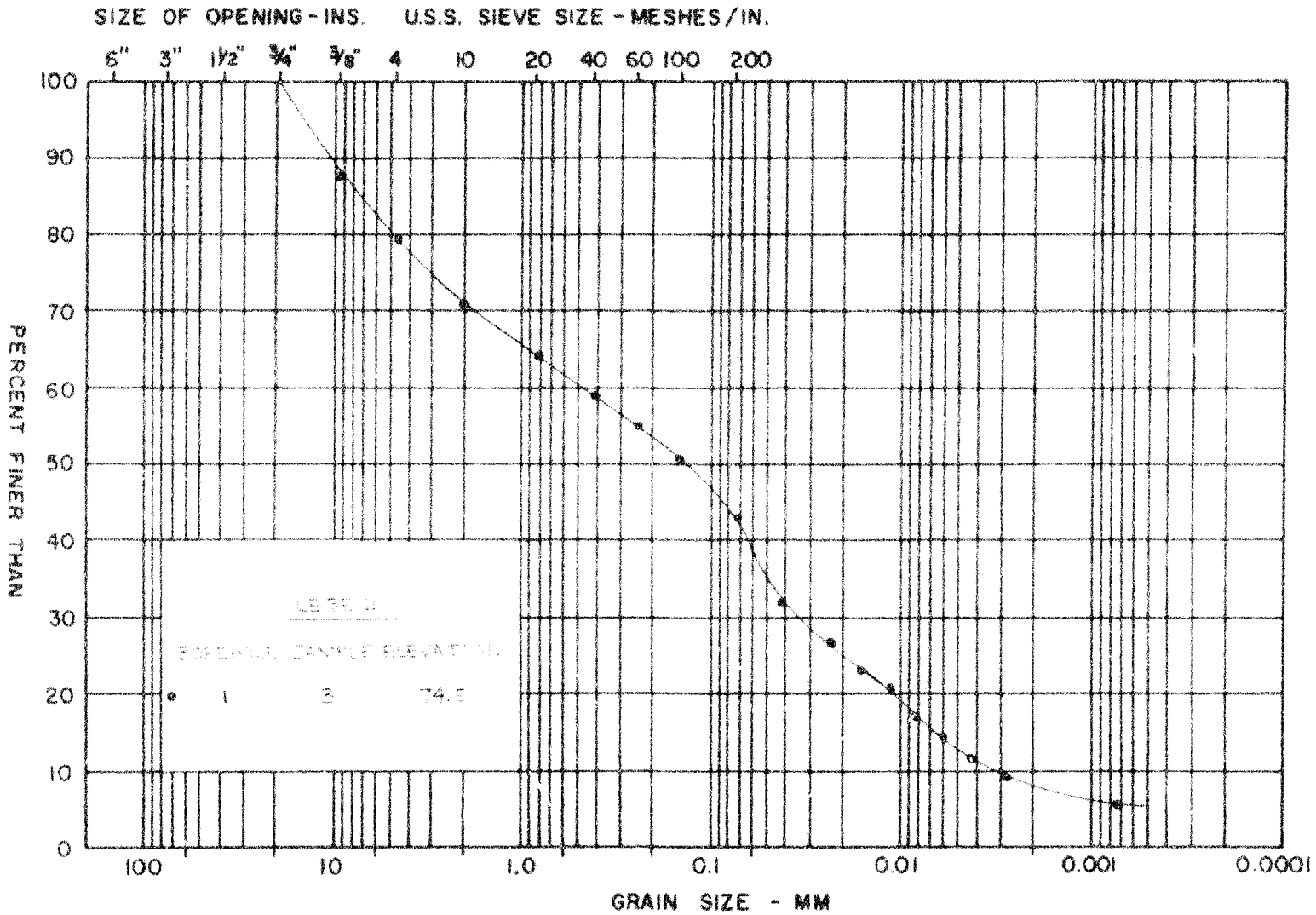
REMARKS: DATA CONCERNING THE TYPICAL STRATA HAVE BEEN OBTAINED AT SEVERAL LOCALITIES ONLY. THE SOIL STRATIGRAPHY BETWEEN THESE LOCALITIES HAS BEEN INTERPOLATED FROM EXISTING EVIDENCE AND MAY VARY FROM THAT SHOWN.

DRAWN: OCT. 5, 1954.

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Made by  
ONE  
APP.

M.I.T. GRAIN SIZE SCALE



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GRAIN SIZE DISTRIBUTION

FIGURE

COBBLE SIZE	COARSE GRAVEL SIZE	MEDIUM GRAVEL SIZE	FINE GRAVEL SIZE	COARSE SAND SIZE	MEDIUM SAND SIZE	FINE SAND SIZE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	