

64-F-260M

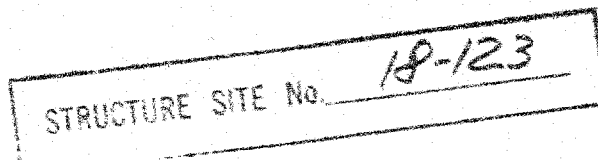
BEATY BRIDGE

CAISTOR TWO

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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



RECEIVED
BP 2 1964 1954
J. M. TOMLINSON & ASSOCIATES LTD.
2444 BLOOR STREET WEST
TORONTO 9, ONTARIO

767-9201
763-4103

October 28, 1964

64-F-260M

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

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

RE: SOIL INVESTIGATION,
PROPOSED BEATY 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. An assessment of the stability of the south river bank at this location was also to be made.

PROCEDURE

The field work was commenced on October 1, 1964 using a mobile power auger supplied and operated by the F. E. Johnston Drilling Co. Ltd. The augers met refusal at a shallow depth in borehole 2 and the boring program was completed on October 7 and 8

using a standard diamond drillrig. A total of three boreholes were put down to an average depth of about 25 feet.

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 bench mark, the elevation of which is given as 78.11 on Drawing 64.8.1 Prel. 3. The bench mark consists of a tack on the south side of a 3.5 foot diameter elm tree located about 100 feet northwest of the existing bridge.

SITE AND GEOLOGY

The site of the proposed bridge over the upper reaches of the Welland River is located on the southern extension of Lincoln County road #44 about 4 miles east of Caistorville in Caistor Township. The new bridge will be located between Lots 10 and 11 in the south part of Concession II, and about 30 feet east of the existing bridge.

The topography of the surrounding area is generally level to gently rolling. There is no pronounced valley of the Welland River at this location. There is in general, however, a

20 foot high bank on the outside curve of the river and about a 5 foot high bank followed by gently sloping land elsewhere along the river.

From available geological information it is known that the area is generally covered by glacial Lake Warren clays and silts followed by glacial drift. The underlying bedrock is dolomite of the Guelph Formation.

SOIL CONDITIONS

The river banks at the boring locations are covered by a surface stratum of clayey material of up to 11 feet in thickness. Near ground surface in borehole 1, the material is a highly plastic brown silty clay, while the remainder of the stratum is a brown silty clay of low plasticity containing a trace of sand and gravel. This latter material has the appearance of a reworked till. Standard penetration tests in the clay gave "N" values ranging from 13 to 45 blows per foot indicating a stiff to hard consistency.

The brown silty clay is underlain by a stratum of brown, becoming grey with depth, sandy silt till, which is the significant foundation stratum at the site. All the boreholes were terminated in this stratum after penetrating the till for up to 17 foot depth. The till is comprised of sand, gravel, cobbles and boulders in a matrix of silt and a trace of clay. A grain size distribution curve

obtained on a sample from this stratum is shown on Figure 2. A one foot thick layer of stiff grey silty clay was encountered within the till in borehole 2 at about elevation 65.

Based on standard penetration tests carried out in this stratum, which gave "N" values ranging between 45 and greater than 100 blows per foot, the relative density of the till is dense to very dense and generally very dense.

WATER CONDITIONS

At the time of the investigation, the water level in the Welland River at this bridge location was at about elevation 68. It is understood from local information that high water level is about 10 feet above this level.

Borehole 1, which was augered on October 2 to elevation 62, was dry on October 8. A piezometer was installed in wash boring 2 on October 8 following its completion and the water level in the piezometer was at ground level initially though insufficient time had elapsed for the water level to reach equilibrium in the relatively impervious soil. It is considered that the stabilized water level at the borehole locations is slightly above present creek level at about elevation 70 which is near the boundary between the desiccated brown and non-weathered grey sandy silt till.

DISCUSSION

General

It is understood that the existing 60 foot span Beaty bridge will be replaced by either a single or double span bridge about 90 feet in total length located some 30 feet east of the existing bridge. The alignment of the bridge will be changed to eliminate the skew angle across the river. It is also understood that the existing roadway grade will be raised several feet to decrease the grade of the south approach.

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 depth of founding will also provide some scour protection.

The "N" values for a significant depth below foundation level are in excess of 60 blows per foot. Based on these values, an allowable bearing pressure of up to 4 tons per square foot may be used in design of footings founded in the very dense sandy silt till.

With the above bearing pressure there should be no detrimental settlement of the bridge abutment and pier footings, provided precautions are taken during construction, as discussed below, 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 to the underlying till.

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, which is a limiting value, may be used in design.

It is recommended that clean 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 rigid frame abutment walls, it is recommended that an at rest earth pressure coefficient, K_0 , of 0.5 be used for the compacted granular backfill.

SLOPE STABILITY

The stability of the existing south river bank at the bridge location has been examined. The present overall slope of

this bank is about 4 horizontal to 1 vertical, though some local sections of the bank are considerably steeper and of the order of 2 horizontal to 1 vertical. Due to the presence of the competent very dense sandy silt till stratum at about ten feet below the slope surface, the overall or deep seated stability of the existing slope is adequate. As for the overlying silty clay stratum, previous triaxial testing on samples of geologically similar clay from this region gave an effective angle of shearing resistance, ϕ' , of about 25 degrees. Based on this value a 2 horizontal to 1 vertical slope in this clay, not affected by seepage forces, would be at about limiting equilibrium. As this surface clay stratum would be subjected to some seepage forces during rainy seasons of the year, it is recommended that growth of grass, shrubs and trees should be encouraged on the steeper sections of the existing bank in the proximity of the bridge to minimize surficial movement and prevent surface water erosion.

APPROACH EMBANKMENTS

The overall height of the embankment above existing ground surface will be about 12 feet at the north approach. The south approach embankment will be about 12 feet above existing ground surface at the abutment location and at about ground level some 100 feet south of the abutment. It is considered that an embankment side slope of 2 horizontal to 1 vertical would be stable,

provided well compacted suitable fill is used.

Prior to placement of the embankment fill, all topsoil and any organic pockets should be removed under the proposed embankments. Rip rap should be placed over the embankment side slopes in the area of the bridge to prevent erosion scour. The rip rap should be placed on a granular filter bed and should extend for a height of 2 feet above the maximum high water level.

CONSTRUCTION PROBLEMS

Because of the low permeability of the till and the overlying clay it is considered that foundation excavations for abutments can be carried down a few feet below the groundwater level in the sandy silt till without undue sloughing of the sides or loosening of the base due to seepage. The water inflow into the abutment excavations should be handled readily by low capacity sump pumps.

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 river bed deposits are encountered in the river they should be removed prior to construction of the dyke.

If sheeting is employed, it should extend below foundation

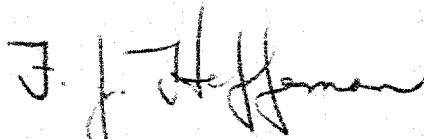
level a depth equal to the maximum water head. The driving of this sheeting could be difficult as some of the sheet piles could be hung up on boulders.

To prevent softening of the till due to entry of 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.

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.



F. J. Heffernan, P. Eng.

FJH:IMB
64122

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

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

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM 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

(h) 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)

α_v	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
C_c	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
c_s	coefficient of consolidation
T_v	time factor = c_s / 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_r	sensitivity

in terms of effective stress
 $\tau_f = c' + \sigma' \tan \phi'$

in terms of total stress
 $\tau_f = c_u + \sigma \tan \phi_u$

*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, 2 & 3

LOCATION **See Figure 1** BORING DATE **DEC 1 8 1964** DATUM **SEA LEVEL**
 BOREHOLE TYPE **PIEZOMETER WITH WATER FILLED** BOREHOLE DIAMETER **4.0 IN. (10.16 CM.)**
 SAMPLER HAMMER WEIGHT **140 LB.** DROP **30 INCHES** PEN. TEST HAMMER WEIGHT **140 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. PLOT	NUMBER	TYPE		BLOWS/FT.	SHEAR STRENGTH C_u , LB./SQ.FT.				WATER CONTENT, PERCENT W_p W W_L
86.3	GROUND LEVEL					30	1/				
80.0	VERY STIFF BROWN SILTY CLAY TRACE OF SAND AND GRAVEL WITH DEPTH		1	2'	75	80	10				
75.0			2		70						
67.4	VERY DENSE BROWN SANDY SILT, SOME GRAVEL, TRACE OF CLAY (SANDY SILT TILL)		3		65					PIEZOMETER OCT 2 1964	
61.9	VERY DENSE GREY SANDY SILT, SOME GRAVEL, COBBLES, TRACE OF CLAY (SANDY SILT TILL)		4		60						
25.0	END OF HOLE				50	NOTE: REFUSAL TO AUGERING AT ELEV. 61.9					
75.4	GROUND LEVEL					70	2/				
68.4	VERY STIFF TO HARD BROWN SILTY CLAY		1	2'	65					GROUND LEVEL SURFACE SEAL CENTRAL SEAL GRAVEL FILL PIEZOMETER	
65.2	DENSE SANDY SILT, SOME GRAVEL, TRACE OF CLAY (TILL)		2		60						
61.2	STIFF GREY SILTY CLAY		3		55						
51.8	VERY DENSE GREY SANDY SILT, SOME GRAVEL, COBBLES, BOULDER, TRACE OF CLAY (SANDY SILT TILL)		4		50						
24.6	END OF HOLE				40	NOTE: REFUSAL TO AUGERING AT ELEV. 51.8 BY TASSING ADVANCED BY DIAMOND DRILLING FROM ELEV. 52.4 TO 50.0					
76.5	GROUND LEVEL					70	3/				
67.5	STIFF BROWN SILTY CLAY TRACE OF SAND, GRAVEL AND COBBLES		1	2'	65						
55.0	VERY DENSE GREY SANDY SILT, SOME GRAVEL, COBBLES, BOULDER, TRACE OF CLAY (SANDY SILT TILL)		2		60						
21.5	END OF HOLE				50	NOTE: EX CASING ADVANCED BY DIAMOND DRILLING DOWN TO ELEV. 55.5					
						15-20 Percent axial strain at failure					

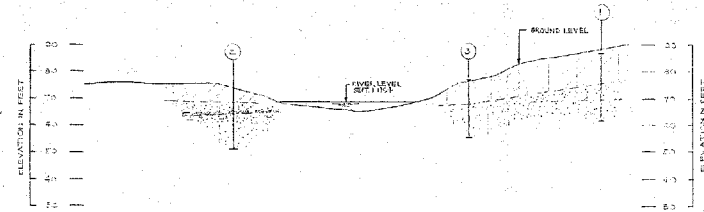
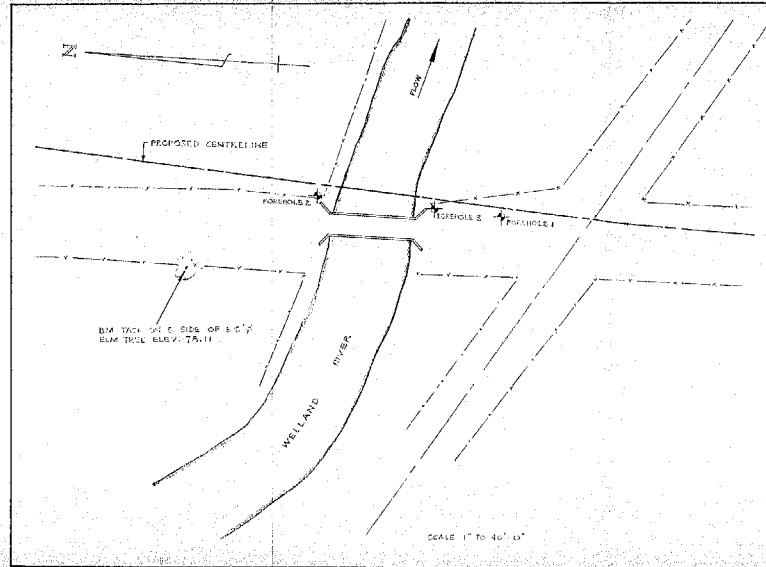
VERTICAL SCALE

1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN **WJA**CHECKED **WJA**

G4142

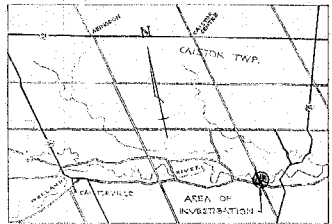


SCHEMATIC SECTION ALONG CENTERLINE PROPOSED BRIDGE
SCALE 1" TO 20'-0"

- LEGEND**
- BORING LOCATION IN PLAN
 - BORING ELEVATION

- STRATIGRAPHY**
- STIFF TO HARD PLAIN SILTY CLAY (TRACED IN SANDY SILT) AND SPECKLED
 - LIGHT TO MEDIUM SANDY SILT (SANDY SILT TILL) TRACED IN PLAIN
 - STIFF SILT SILTY CLAY

BORING PLAN & SOIL STRATIGRAPHY SECTION **FIGURE 1**



KEY PLAN

ENTER ELEVATION IN PLAN AND ELEVATION IN ELEVATION. (ELEVATION IN PLAN IS BASED ON THE SITE PLAN AND THE LOCATION OF THE BORING.)

GENERAL NOTE: NAME CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED BY VARIOUS LOCAL SOURCES. ONLY THE SOIL STRATIGRAPHY BETWEEN BORINGS HAS BEEN IMPROVED FROM GEOLOGICAL SURVEY AND SO HAS NOT BEEN FROM THAT SOURCE.

DRAWN: OCT. 7, 1964

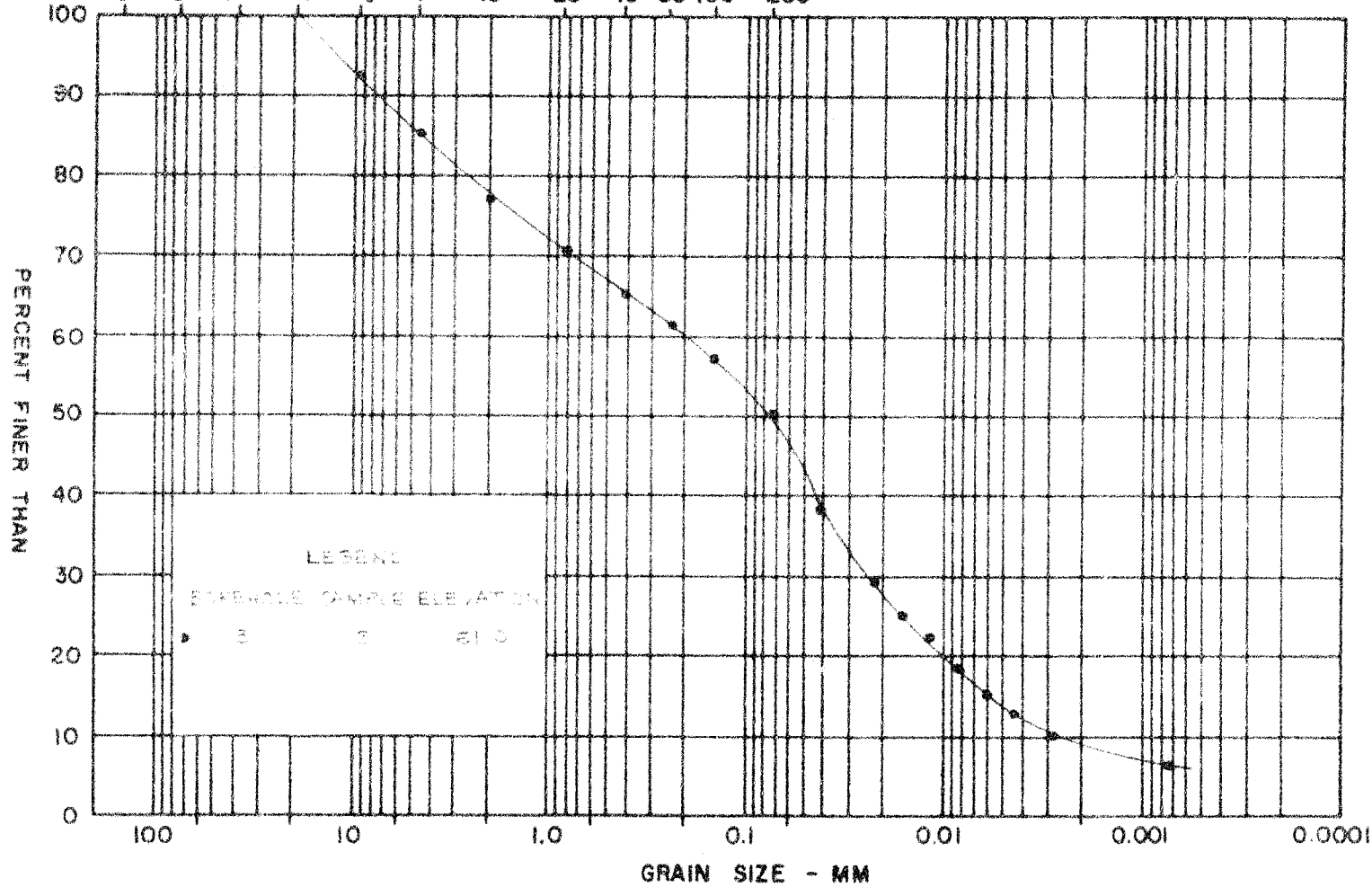
GOLDER & ASSOCIATES

Wade, J.W.
Chad, J.W.
Rood, J.W.

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.S. SIEVE SIZE - MESHES / IN.

6" 3" 1 1/2" 3/4" 3/8" 4 10 20 40 60 100 200



GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SANDY SILT

FIGURE 2