

68-F-206M

BOLTON

CREEK

CROSSING

FRONTENAC

COUNTY

H. Q. GOLDER & ASSOCIATES LTD.

SOIL AND FOUNDATION ENGINEERS

HEAD OFFICE - TORONTO, ONTARIO

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**196 BRONSON AVENUE
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E. J. HEFFERNAN (OTTAWA)

REPORT

TO

J. M. TOMLINSON & ASSOCIATES LIMITED

ON

SOIL INVESTIGATION

PROPOSED BOLTON CREEK CROSSING

COUNTY OF FRONTENAC

ZEALAND

ONTARIO

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April, 1968

68760

TABLE OF CONTENTS

	<u>Page</u>
ABSTRACT	1
INTRODUCTION	2
PROCEDURE	2
SITE AND GEOLOGY	3
SUBSURFACE CONDITIONS	4
PROPOSED BRIDGE STRUCTURE	5
a) General	5
b) Foundations	6
c) Flexible Culvert	7
d) Approach Embankments	8
ABBREVIATIONS	In order
RECORD OF BOREHOLE SHEETS	Following
	Page 9.
FIGURES 1 - Boring Plan and Soil Stratigraphy	
2-3 - Grain Size Distribution Curves	

ABSTRACT

The results of an investigation to determine the subsurface conditions at the proposed Bolton Creek crossing are reported and recommendations are made for foundation design and construction of the proposed bridge or culvert structure.

The existing sand, gravel and boulder embankment fill was found to be underlain by up to 10 feet of very loose fibrous peat. The peat is underlain by deposits of firm to stiff grey clayey silt and loose to compact silty sand and silty fine sand. Bedrock was encountered at a depth of about 28 feet below roadway level at the south side of the site while on the north side of the site, dynamic cone penetration tests indicate that the bedrock surface is more than 55 below roadway level.

It is recommended that the bridge structure be founded on timber piles some 20 to 40 feet in length and designed for an allowable load of 20 tons/pile.

An alternate structure would be the use of flexible steel multi-plate culverts. All fibrous peat should be removed from below the culvert area and should be replaced with coarse granular fill to within 1 foot of invert level. The culverts should be founded on, and backfilled with, well compacted sand and gravel fill.

INTRODUCTION

H. Q. Golder & Associates Ltd, have been retained by J. M. Tomlinson and Associates Limited, Consulting Engineers, to carry out a soil investigation at the site of the proposed Bolton Creek Crossing, County of Frontenac, near Zealand Ontario. The purpose of this investigation was to determine the soil conditions at the site and, based on this information, to provide recommendations for foundation design and construction of a proposed bridge or culvert structure.

PROCEDURE

The field work for this investigation was carried out between March 18th and 22nd, 1968. Four boreholes were put down using a machine drill rig supplied and operated by the F. E. Johnston Drilling Co. Ltd, Ottawa. In addition, a dynamic penetration test was put down adjacent to borehole 1, and dynamic penetration tests were carried out from the bottom of boreholes 2 and 3. Overburden samples were taken in all boreholes and bedrock was cored in BX size in borehole 1. The field work was supervised throughout by a member of our engineering staff.

A detailed log of each boring is given on the Record of Borehole sheets 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 soil and rock core samples obtained during the investigation were brought to our laboratory for examination and testing. The results of the testing are shown on Figures 2 and 3, as well as on the Record of Borehole sheets.

The elevations given in this report are referred to the bench mark located on a nail in a cedar tree at Station 0+00, 37 feet left of the centerline of the existing road. The elevation of this bench mark was given as 108.03, as referred to a local datum.

SITE AND GEOLOGY

The site under investigation is located in the Township of Oso, County of Frontenac, about 18 miles northwest of Perth and about 1 mile north of Zealand. The topography of the Bolton Creek valley in the vicinity of the proposed crossing is relatively flat and swamp covered.

From available geological information it is known that the area is underlain by Precambrian crystalline limestone of the Grenville series. The creek valley has been filled with alluvial silts and sands. Recent deposits of peat are found in the swamp areas.

SUBSURFACE CONDITIONS

The detailed soil stratigraphy encountered in each borehole is given on the Record of Borehole sheets and is illustrated on the stratigraphic section on Figure 1. Following is a summarized account of the soil conditions at the site.

The approach embankments to the existing bridge structure consist of a compact sand, gravel, and boulder fill which at the borehole locations was up to 10 feet thick. The embankment fill is underlain by a stratum of very loose dark brown fibrous peat which ranges in thickness from 2 to 10 feet. A natural moisture content of 370 percent was obtained on a sample of this peat.

Underlying the fibrous peat in borehole 1 is a stratum of loose grey to brown sand with some gravel. The sand at this borehole location contains a 5 foot layer of firm to stiff grey clayey silt. An in situ vane test carried out in the clayey silt gave a shear strength value of 1200 lbs/sq.ft.

Underlying the peat in boreholes 2, 3, and 4 is a stratum of grey sandy silt which was proven for a depth of 10 feet in borehole 3. Evidence of clay layering was encountered in the stratum. The result of a mechanical analysis carried out on a sample from this stratum is shown on Figure 2. Standard penetration tests carried out in the sandy silt gave "N" values

ranging from 8 to 16 blows/foot indicating a relative density of loose to compact.

At a depth of about 20 feet below roadway level in boreholes 1 and 2, is a stratum of grey silty fine sand with a thickness at the borehole locations of from 6 to 10 feet. Based on "N" values ranging from 7 to 14 blows/foot, the relative density of the silty fine sand is considered to be loose to compact. A stratum of compact grey medium to coarse sand was found to underlie the silty fine sand in borehole 2. This medium to coarse sand was proven for a depth of about 7 feet.

Bedrock was encountered in borehole 1 at a depth of about 28 feet below roadway level. From the results of the boreholes and penetration tests, the bedrock is known to fall sharply towards the north. The bedrock was found to be a sound grey crystalline limestone.

A standpipe was installed in borehole 1 beneath the clayey silt. The water level in the standpipe, and in the open holes at boreholes 2, 3, and 4, on March 22, 1968, was at elevation 97.5, or about 2 feet below ground level. The water level in Bolton Creek at the time of the investigation, the spring run-off period, was also at elevation 97.5.

PROPOSED BRIDGE STRUCTURE

a) General

It is understood that it is proposed to replace the

existing narrow bridge over Bolton Creek by a longer bridge structure at the same crossing. Alternatively, the crossing would be effected by the use of 3, ten foot diameter flexible steel multi-plate culverts at the location shown on Figure 1. As presently planned, the proposed roadway grade at the crossing will be approximately the same as the existing grade.

b) Foundations

The boreholes put down at the bridge crossing indicate that the creek bed is underlain by a deposit of fibrous peat followed by loose sands and silts to a depth of about 15 feet below low water level. Spread footing foundations are therefore unsuitable at this site and it is recommended that the bridge foundation loads be transferred to the underlying sand stratum or to bedrock by the use of piles. It is considered that a timber pile would be the most economical pile type for this bridge. For a timber pile with a 12 inch butt and 8 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 20 tons per pile. Based on the low "N" values obtained in the overburden, together with the results of the dynamic penetration tests, it is considered that the piles will probably be driven to the bedrock surface or to the level at which penetration test 2 met practical refusal,

that is, some 20 to 40 feet below creek bed. Steel H piles are also a suitable pile type at this site where the depth to bedrock is variable.

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 , of 0.3, providing some minor movement of the top of the abutment walls can be tolerated.

c) Flexible Culvert

A proposed alternate to a pile supported bridge structure is the use of flexible multi-plate culverts located in the vicinity of boreholes 3 and 4. The invert level of the culverts would be at about elevation 90, and the boreholes indicate some 8 feet of loose and compressible peat below this elevation. This fibrous peat should be removed from below the culvert area and should be replaced with suitable compacted granular fill to invert level. The excavation of this peat could best be handled by a dragline operating below the water

level. Coarse granular fill similar to the embankment fill and available from a nearby pit could then be end-dumped through water to within one foot of invert grade. Well compacted sand and gravel fill conforming to D.H.O. sand cushion specifications should be used immediately below and around the culvert. Differential settlement along the length of the culverts would then be within the tolerable limits of a metal culvert.

The creek can be diverted from the construction area by a low level dyke of relatively impervious earth fill. The water level within this dyked area could be maintained at about elevation 90, during erection and backfilling of the culverts, by pumping from sumps. If construction is carried out during a period of low water in the creek, the water inflow into the dyked area will be minor.

d) Approach Embankments

It is understood that the proposed roadway grade will probably remain at about existing grade. Some consideration however, may be given to a grade increase of some 2 feet to alleviate the possibility of flooding of the roadway. It is considered that most or all of the settlement of the peat under the loading of the present embankment has taken place. This consolidation settlement under a load of 10 feet of fill would have produced a shear strength increase in the peat layer,

sufficient to overcome potential instability associated with an embankment grade raise of some 2 feet. The majority of the additional settlement associated with any grade raise will take place within a few months. The majority of the settlement resulting from filling in the existing bridge crossing, assuming culverts are used, would also take place within a few months. Final grading operations adjacent to structures, not affected by consolidation of the peat, should be delayed a few months after completion of embankment filling.

RAM/ml
68760
April, 1968.

[Signature]
R. A. Montgomery, P. Eng.

[Signature]
F. J. Heffernan, P. Eng.



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

C consolidation test
H hydrometer analysis
M sieve analysis
MH combined analysis, sieve and hydrometer¹
Q undrained triaxial²
R consolidated undrained triaxial²
S drained triaxial
U unconfined compression
V 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

e	$e = 2.7183$
$\log_e a$ 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_s	coefficient of consolidation
T_v	time factor $= c_s t / d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion intercept
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

*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 1

BORING DATE MAR 18 1968

DATUM

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER BX CASTING

SAMPLER HAMMER WEIGHT 40 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. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.		SHEAR STRENGTH C_u , LB. / SQ. FT.					WATER CONTENT, PERCENT						
							VANER + NAT. + REM. 500 1000 1500 2000					W_p W W_L						
99.5 0.0	GROUND LEVEL					100										GROUND LEVEL		
	COMPACT BROWN SAND AND GRAVEL, SOME BOULDERS (EMBANKMENT FILL)		1	BX RC														
			2	"		95												
			3	2" DO														
			4	BA														
			5	"		90												
89.3 10.2	VERY LOOSE DARK BROWN FIBROUS PEAT		6	2" DO	2													
87.3 12.0	LOOSE GREY SAND, SOME GRAVEL																	
85.5 14.0	FIRM TO STIFF GREY CLAYEY SILT		7	"	E													
81.0 18.5	LOOSE BROWN SAND, SOME FINE GRAVEL		8	"	B													
78.0 21.5	COMPACT GREY SILTY FINE SAND		9	"	14													
72.0 27.5	SOUND GREY CRYSTALLINE LIMESTONE BEDROCK		10	BX RC		70												
66.4 33.1	END OF HOLE		11	"														
						65												
						60												

15-10 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 5'

GOLDER & ASSOCIATES

DRAWN D.N.
CHECKED R.M.

RECORD OF BOREHOLE 2

LOCATION See Figure 1

BORING DATE MAR. 19, 1968

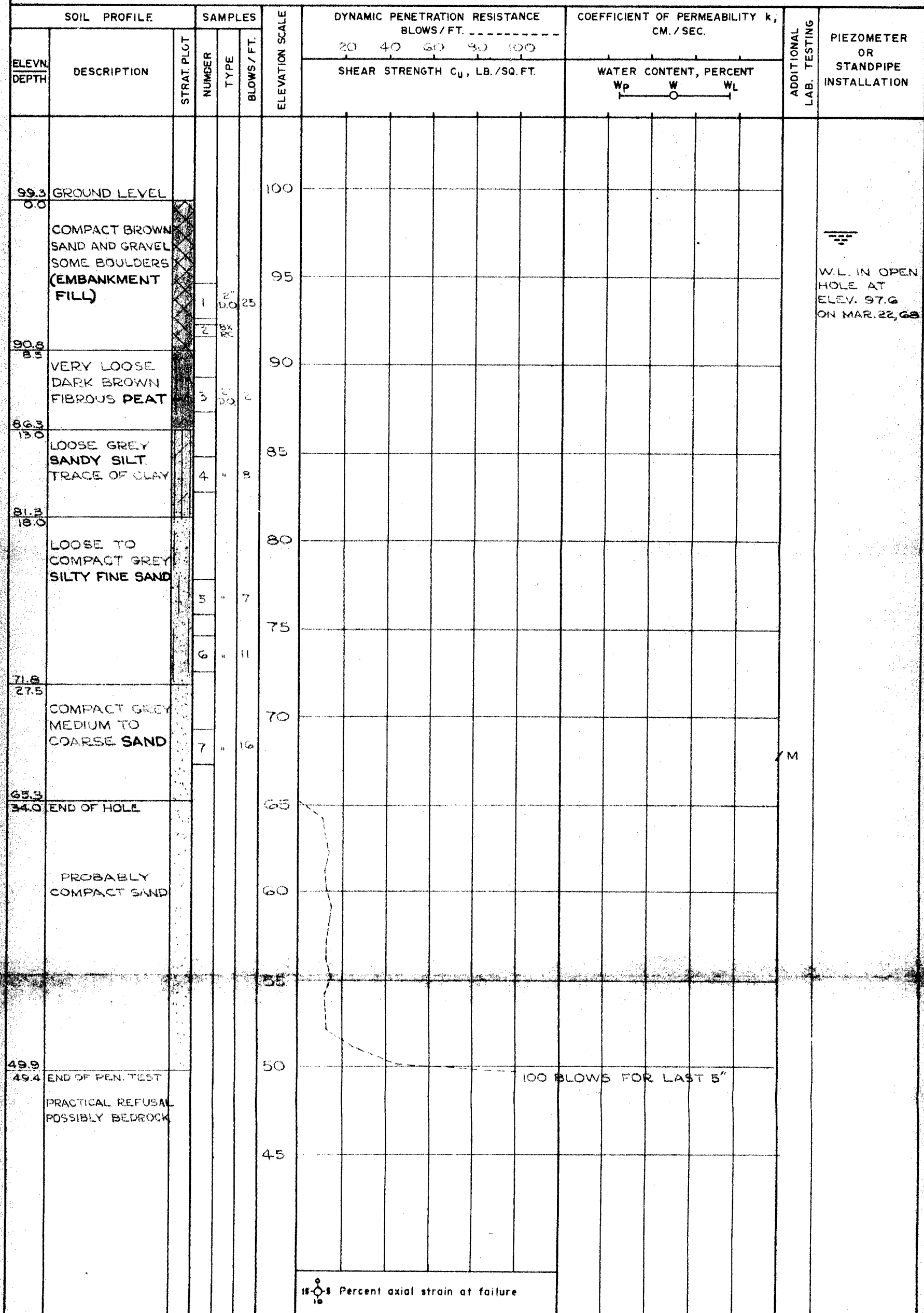
DATUM

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

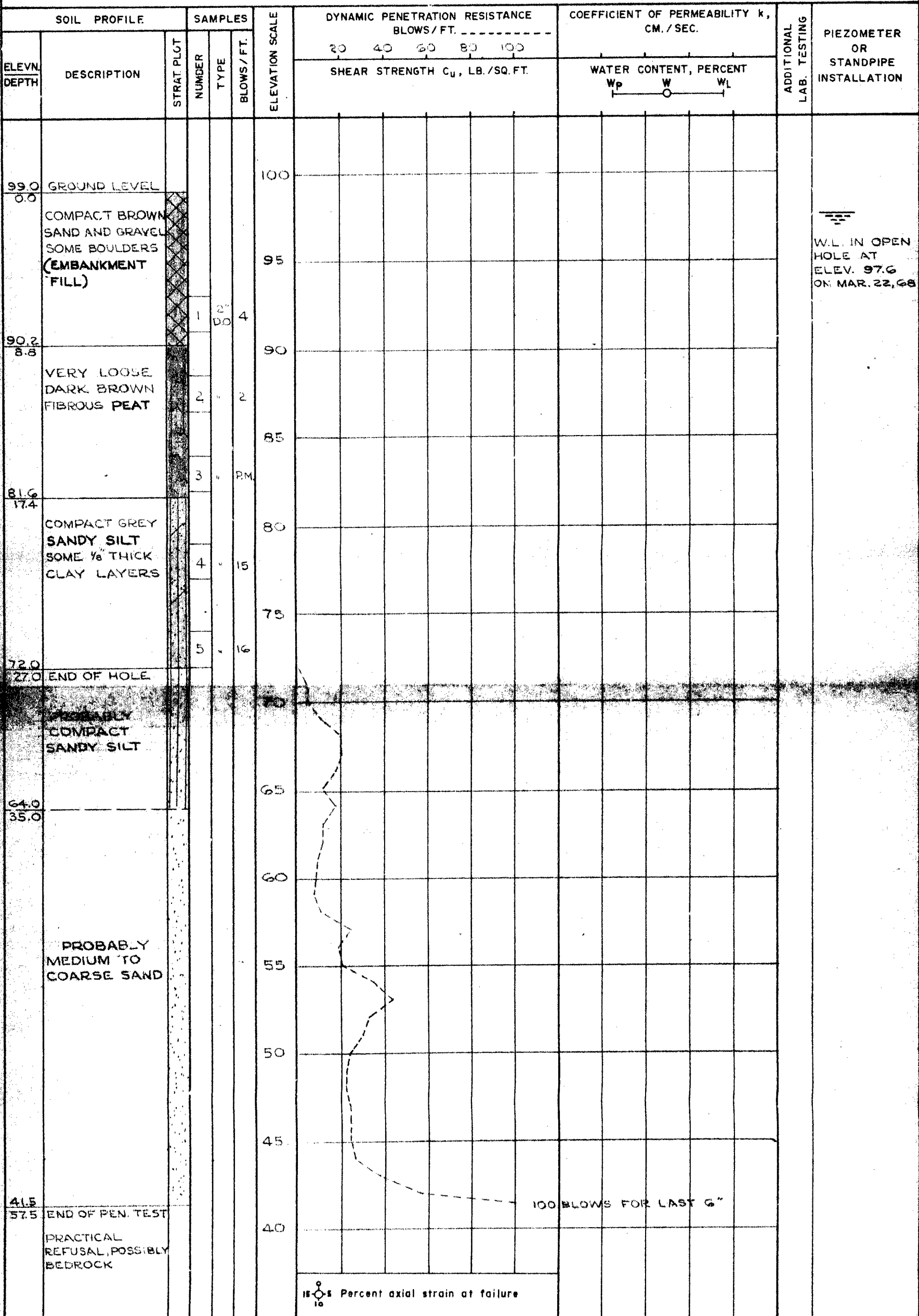
VERTICAL SCALE
1 INCH TO 5'

GOLDER & ASSOCIATES

DRAWN D.N.
CHECKED *AM*

RECORD OF BOREHOLE 3

LOCATION See Figure 1 BORING DATE MAR. 22, 1968 DATUM
BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER BX CASING
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 5'

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

GOLDER & ASSOCIATES

DRAWN R.N.

CHECKED

RECORD OF BOREHOLE 4

LOCATION

See Figure 1

BORING DATE MAR. 22, 1968

DATUM

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER BX CASING

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. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		SHEAR STRENGTH C_u , LB./SQ. FT.					WATER CONTENT, PERCENT <div style="display: flex; justify-content: space-around; width: 100%;"> W_p W W_L </div>						
99.0	GROUND LEVEL					100												
97.0	COMPACT BROWN SAND AND GRAVEL SOME BOULDERS (EMBANKMENT FILL)		1	BX RC		95												
92.1	VERY LOOSE DARK BROWN FIBROUS PEAT		2	2" SD	PM	90												
91.5			3	"	PM													
83.5							85											
81.5	LOOSE GREY SANDY SILT TRACE OF CLAY		4		9													
80.0	END OF HOLE					80												

15-10-5 Percent axial strain at failure

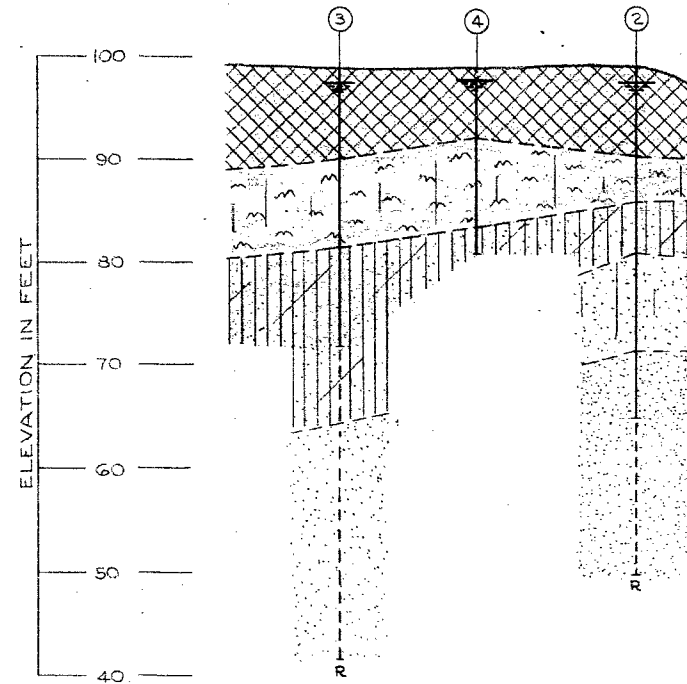
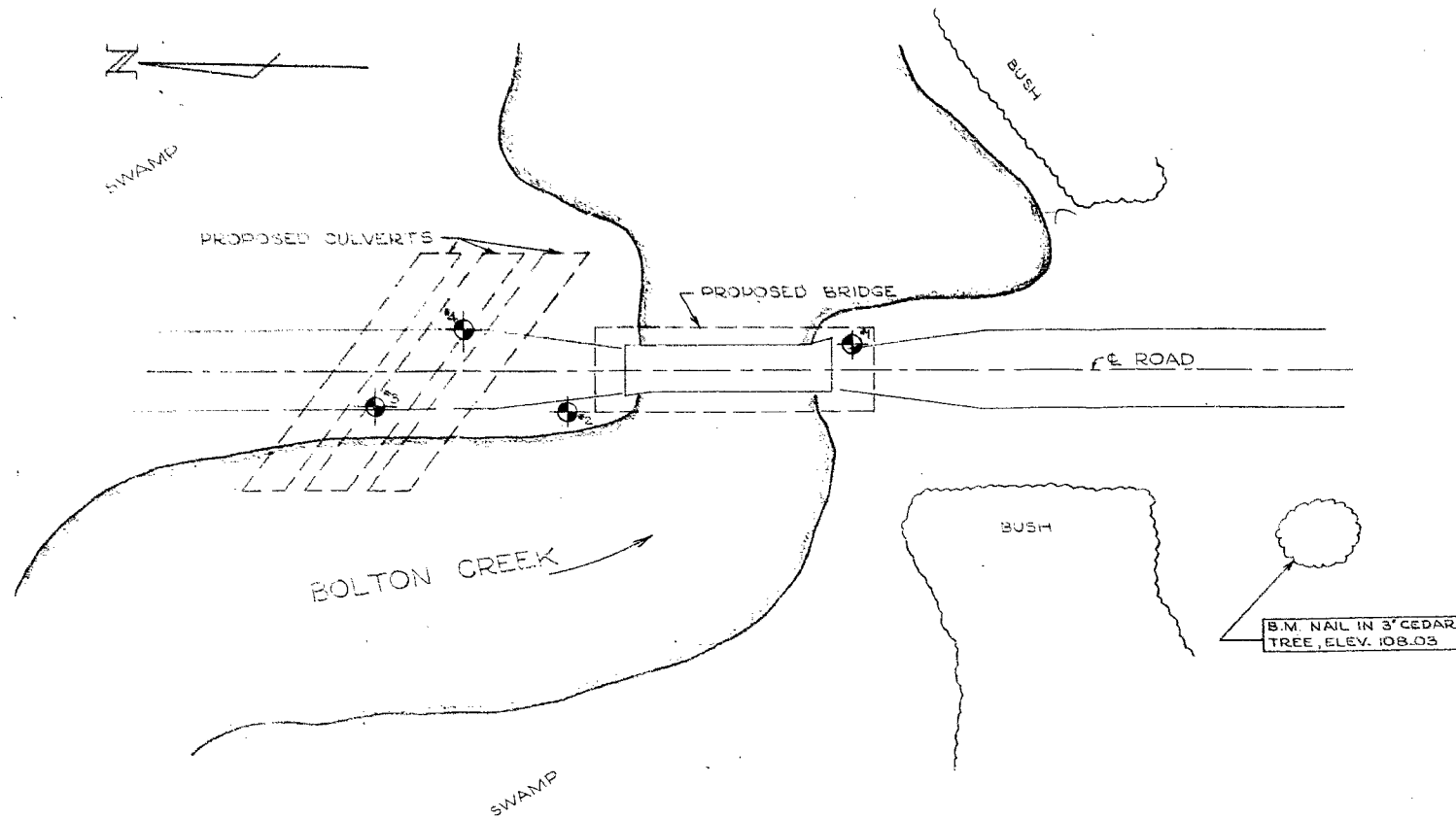
W.L. IN OPEN
HOLE AT
ELEV. 97.7
ON MAR. 22, 68

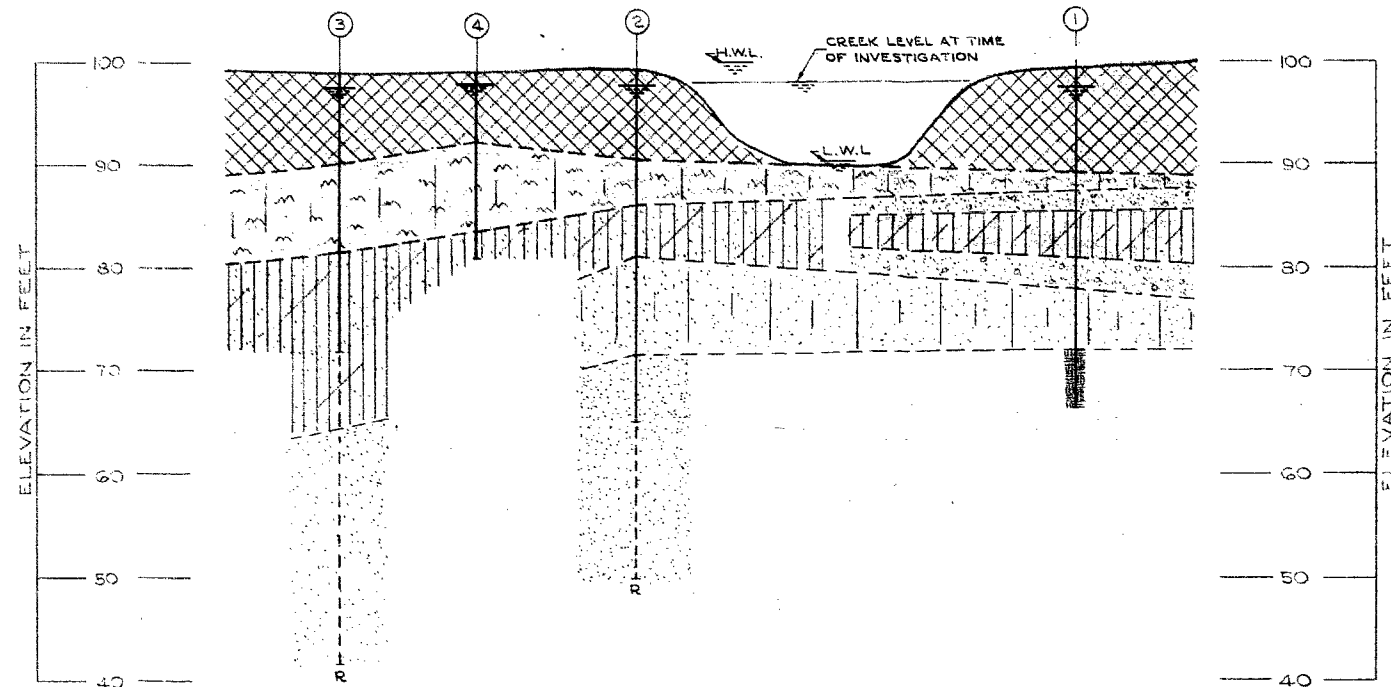
MH

 VERTICAL SCALE
1 INCH TO 5'

GOLDER & ASSOCIATES

 DRAWN D.N.
CHECKED *[Signature]*





SECTION ALONG E ROAD

SCALES: HOR. 1" TO 20'
VER. 1" TO 10'

STRATIGRAPHY

- COMPACT BROWN SAND AND GRAVEL, SOME BOULDERS (EMBANKMENT FILL)
- VERY LOOSE DARK BROWN FIBROUS PEAT
- LOOSE GREY AND BROWN SAND, SOME FINE GRAVEL
- FIRM TO STIFF GREY CLAYEY SILT
- LOOSE TO COMPACT GREY SANDY SILT, SOME CLAY LAYERS
- LOOSE TO COMPACT GREY SILTY FINE SAND
- COMPACT GREY MEDIUM TO COARSE SAND
- SOUND GREY CRYSTALLINE LIMESTONE BEDROCK

LEGEND

- BOREHOLE IN PLAN
- BOREHOLE IN ELEVATION
- PENETRATION TEST FROM END OF BOREHOLE
- WATER LEVEL IN BOREHOLE ON MAR. 22, 1968. (SPRING RUN-OFF PERIOD)
- R - PRACTICAL REFUSAL TO PENETRATION TEST

REFERENCE: DRAWING SUPPLIED BY:
J.M. TOMLINSON & ASSOCIATES LTD.,
CONSULTING ENGINEERS -
BRIDGE & CULVERT, SITE PLAN & PROFILE.

SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND DOES NOT VARY FROM THAT SHOWN.

Drawn: April 1, 1968.

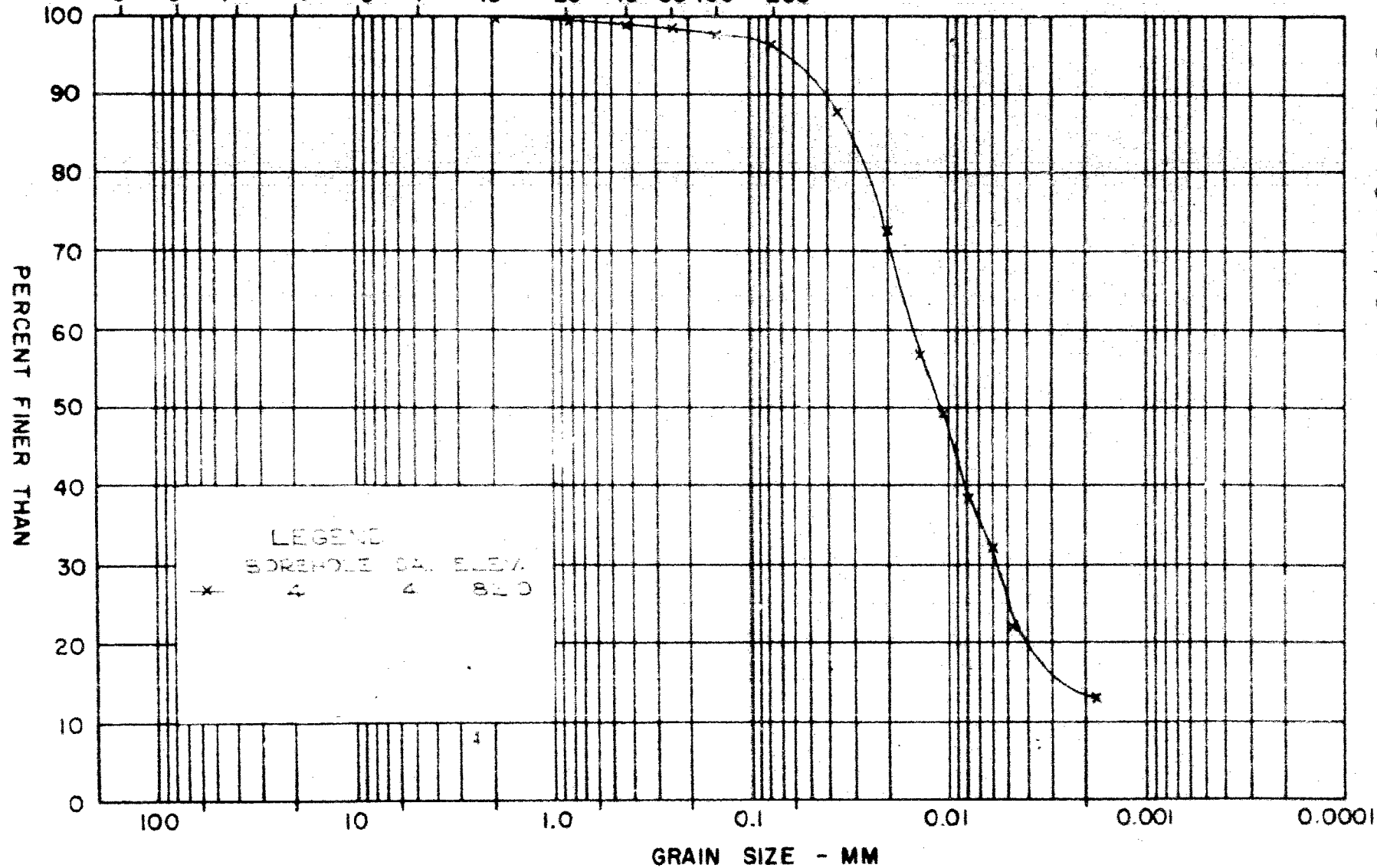
GOLDER & ASSOCIATES

Made J.N.
Child
App'd

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



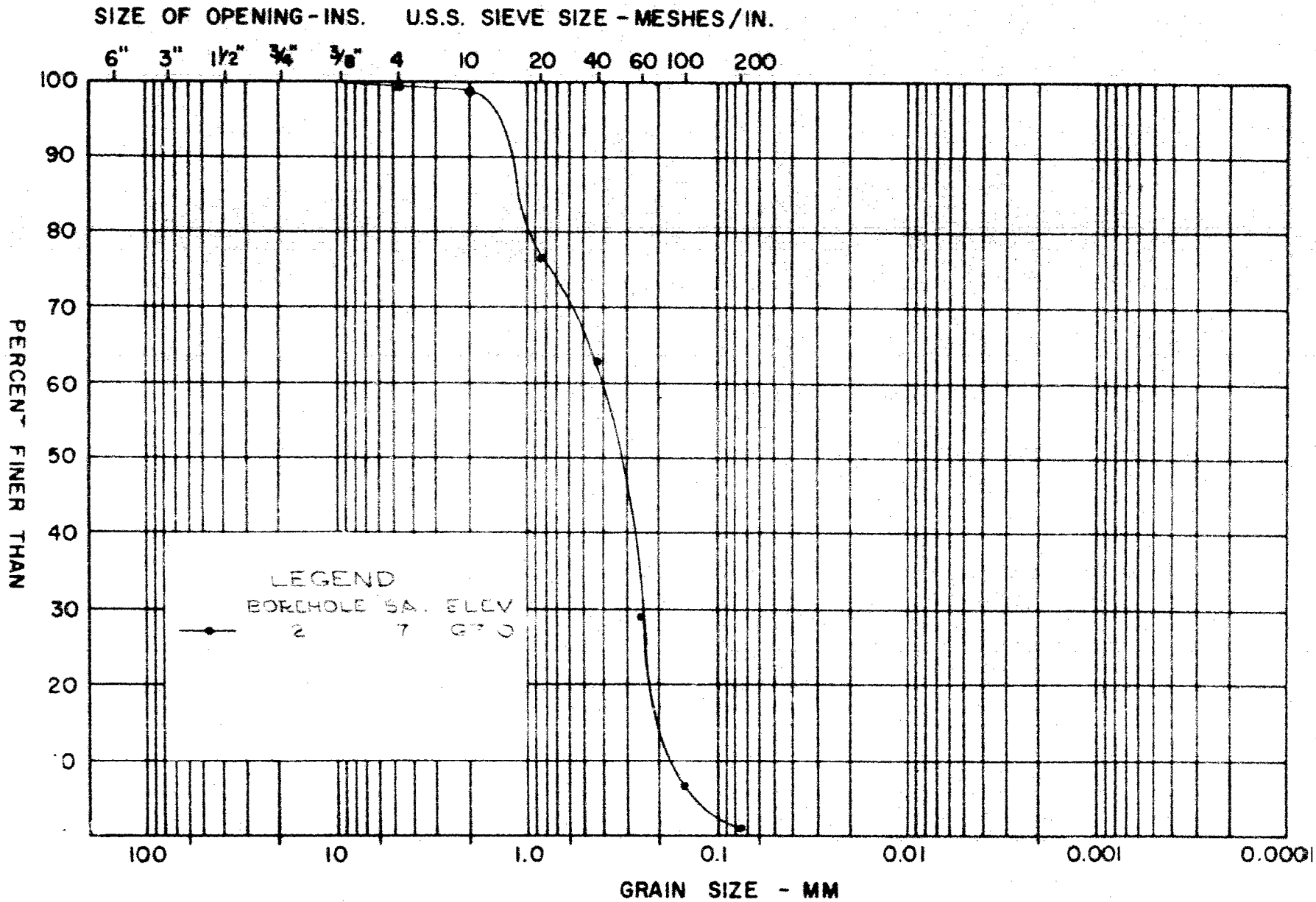
SANDY SILT, SOME CLAY LAYERS

GRAIN SIZE DISTRIBUTION

FIGURE 2

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

M.I.T. GRAIN SIZE SCALE



MEDIUM TO COARSE SAND

GRAIN SIZE DISTRIBUTION

FIGURE 3

GOLDER & ASSOCIATES

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

ayp

Mr. K. L. Kleinstreiber,
Municipal Bridge Liaison Engr.,
Bridge Division,
Admin. Bldg.

Foundation Section,
Materials & Testing Division,
Room 107, Lab. Bldg.

August 27, 1968

Bolton Creek Bridge near Zealand, Ontario
Structure Site No. 7-20
County of Frontenac - Township of Oso

BA-2860

We have reviewed the Foundation Report by H. Q. Golder and Associates, together with the Engineering Report and Preliminary Proposal by J. M. Tomlinson Associates Ltd. for the above mentioned structure. Our comments are as follows:

1) Three Multi-Plate Pipe Arch Culverts:

In our opinion, excavation of organic material (peat) should include the old creek bed and also the portion between the old creek and new structure, in order to eliminate possible differential settlements of the future road.

ii) Structure on Timber Piles:

In our experience, problems have been caused because of the presence of organic material below perched abutments, even when supported on piles. Earth movements have been known to take place in such circumstances causing damage to the structure. In view of the above mentioned facts, it is recommended that this peat be removed and replaced with suitable granular material prior to the construction of the structure.

It is believed that it would not be economical to drive timber piles to refusal in this case and, therefore, consideration should be given to the use of steel H-piles to refusal which would provide a much higher capacity.

KD/ndef

cc: Foundations Files
Gen. Files

M. Devata
M. Devata,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

P.S. -- Reports by H. Q. Golder & Assoc. Ltd.
J. M. Tomlinson & Assoc. Ltd. -
returned herewith.