

# H. Q. GOLDER & ASSOCIATES LTD.

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REPORT

TO

J. M. TOMLINSON & ASSOCIATES LTD.

ON

FOUNDATION INVESTIGATION

PROPOSED COLBORNE STREET BRIDGE REPLACEMENT  
OVER SIXTEEN MILE CREEK

OAKVILLE

ONTARIO

66-5-257M

## Distribution:

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October, 1966

66108

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ABSTRACT

This report presents the results of a subsurface investigation carried out at the site of the proposed reconstruction of the Colborne Street bridge over Sixteen Mile Creek in Oakville, Ontario. The purpose of the investigation was to determine the subsurface conditions at the site and to provide engineering recommendations for the foundation design of the proposed structure.

The site is underlain by some 10 to 40 feet of loose to dense silty sand fill followed at about elevation 240 by some 10 to 50 feet of compressible organic silt. The organic silt is generally soft in the upper portion but the consistency increases to very stiff with depth. The organic silt is generally underlain by up to 10 feet of rock fragments in a hard silty clay matrix. At the pier locations some 10 to 20 feet of either fractured limestone cobbles and boulders or badly weathered interbedded limestone and soft shale was encountered at about elevation 187. The fractured limestone or rock fragments are underlain at between elevations 170 and 225 by weathered and fractured shale bedrock.

The proposed structure may be founded on end-bearing piles driven to refusal in either the shale bedrock or limestone. For steel H-piles a design load of up to 70 tons/pile may be used. Due to shattering of the bedrock during driving all piles should be redriven to ensure that the design pile load is obtained. Settlement of the piers and abutments founded on piles driven to rock should be negligible. Careful control of pile driving should be provided to ensure no detrimental settlement of the existing bridge piers and abutments takes place.

Restrictions to the height of approach embankments are discussed.

## INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by J. M. Tomlinson & Associates Ltd. to carry out a subsurface investigation at the site of the proposed reconstruction of the Colborne Street bridge, over Sixteen Mile Creek in Oakville, Ontario. The purpose of the investigation was to determine the subsurface conditions across the site and to provide information for the design of foundations for the new structure.

## PROCEDURE

The field work for this investigation was carried out between September 6 and 27, 1966. During this period a total of 4 boreholes with adjacent dynamic penetration tests and one additional dynamic penetration test were put down to depths of between 70 and 90 feet using a skid-mounted machine drillrig supplied and operated by Canadian Longyear Limited, Toronto, Ontario. Following completion of each borehole a sealed piezometer was installed for groundwater level observation. The field work was supervised throughout by a member of our engineering staff.

The locations of the borings together with a section of the inferred subsoil conditions along the centreline of the proposed bridge are shown on Figure 1. A detailed log for each boring

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is given on the Records of Boreholes following the text of this report.

Samples obtained during the investigation were brought to our laboratory for detailed examination and testing. The results of the laboratory testing are presented on the Records of Boreholes and on Figures 2 to 7.

The borehole locations and elevations were supplied to us by J. M. Tomlinson & Associates Ltd. The elevations given in this report are referred to Geodetic datum.

#### SITE AND GEOLOGY

The site of the proposed Colborne Street (Highway 2) bridge over Sixteen Mile (Oakville) Creek and its associated flood-plain is located some 1,400 feet north of Lake Ontario in the Town of Oakville, Ontario. The existing Sixteen Mile Creek valley is about 35 feet below the surrounding ground surface and the valley crest width is some 400 feet. The valley slopes are lightly wooded. The site is presently occupied by a 5 span concrete and steel structure and associated roadway approach embankments which carries Colborne Street over Sixteen Mile Creek at a roadway grade of about elevation 276.

Oakville lies within the physiographic region known as the Lake Iroquois Plain, an area which was inundated by Lake Iroquois during the retreat of the last Pleistocene ice sheet; however at the actual bridge site Sixteen Mile Creek in conjunction with other glacial streams feeding into Lake Ontario, has eroded a deep valley into the underlying shale bedrock when the lake was at a lower level. With the rise of Lake Ontario to its present level this valley was drowned and filled by recent deposits of organic silt.

The site is underlain by Ordovician shale and limestone bedrock of the Meaford Formation. It is known to be a grey to brown fissile calcareous shale containing hard layers of impure calcareous sandstone and crystalline limestone.

#### SOIL CONDITIONS

The detailed soil stratigraphy encountered in each borehole is given on the Records of Boreholes.

The borings indicate that the site is underlain by an extensive layer of fill which varies in thickness from about 30 to 40 feet at the proposed abutment locations to about 10 feet at the pier locations. The fill consists generally of brown to grey silty sand with gravel to a trace of gravel and occasional pockets

of silty clay and organic matter. Behind the proposed east abutment location (borehole 4), the fill has an irregularly layered structure and in the lower 5 feet is composed of a stiff grey to brown silty clay with sand and a trace of gravel and wood fragments. Typical grain size distribution curves for samples of the silty sand fill are presented on Figure 2. Based on standard penetration test results which gave "N" values of between 5 and 32 blows/ft. the relative density of the fill varies from loose to dense and is generally compact.

The fill is underlain at the abutment locations, boreholes 3 and 4, by some 5 to 10 feet of grey to brown sandy silt to organic sandy silt with a trace to some clay and gravel throughout. A typical grading curve for the sandy silt is shown on Figure 3. The sandy silt is in a compact state of packing.

The sandy silt and fill deposits are underlain at about elevation 230 to 240 by an extensive stratum of organic material which varies in thickness from about 8 and 21 feet at the west and east abutments respectively, to some 50 feet at the pier locations. This organic stratum consists of dark brown to grey organic silt with a trace to some clay and sand and contains occasional shell and wood fragments. Typical grading curves for samples of the organic silt are shown on Figures 4 and 5.

Atterberg limit determinations carried out on samples of the organic silt gave liquid limits ranging from about 40 to 100 and plasticity indices of 20 to 60. The in situ water content of the organic silt varies between 30 and 60 percent giving a liquidity index of about 0.4. The organic content of the silt was found to be about 6 to 11 percent by weight which accounts for the high liquid limits and plasticity indices.

The total unit weight of the material is between 98 and 120 lb/cu.ft.

The undrained shear strength of the organic silt was determined by in situ field vane testing and by laboratory triaxial compression tests on relatively undisturbed samples. The results of this testing, which is summarized on Figure 7, indicates that the undrained shear strength of the organic silt increases from about 800 lb/sq.ft. near the top of the stratum (elevation 240) to greater than 2,000 lb/sq.ft. below about elevation 205. Based on these test results together with the standard penetration test results, summarized on Figure 7, the consistency of the organic silt increased from soft to very stiff with depth. The apparent discrepancy between the undrained shear strength results and the "N" values is probably due to the essentially granular nature of the silt.

Remoulded field vane tests gave values ranging between 300 and 500 lb/sq.ft. Based on these results the organic silt has an average sensitivity (defined as the ratio of undisturbed strength to remoulded strength) of about 2.5 to 3.

The organic silt is underlain in boreholes 1, 3 and 4 by a 2 to 8 foot thick deposit of limestone and shale fragments and gravel in a hard silty clay matrix. This stratum may be a badly re-worked upper portion of the underlying bedrock and may have formed the bed of an ancient stream or river.

#### ROCK CONDITIONS

The rock fragments and gravel or organic silt deposits are underlain at about elevation 187 in boreholes 1 and 2 put down at the east and west pier locations respectively, by a badly fractured limestone layer some 12 to 18 feet in thickness.

The recovered BXL rock core from this stratum consists of weathered and badly fractured light grey limestone apparently containing numerous fissures and occasional solution cavities. The low recovery of core (generally less than about 50 percent) and the hard nature of the recovered rock indicate that up to 50 percent of the stratum consists of material other than limestone. It is possible that the material not recovered in the core may

consist of soft badly weathered shale which was ground up during drilling operations and the layer could thus be part of the Meaford Formation; however if the interbedded material is till the layer could be a boulder pavement laid down in the bed of the ancient glacial stream which eroded the drowned valley forming the present Sixteen Mile Creek bed.

The limestone layer is underlain at about elevation 170 at the pier locations, elevation 197 behind the east abutment and about elevation 227 at the west abutment by calcareous shale bedrock. The shale bedrock, which was proved for up to about 20 feet by core drilling, is generally weathered and fractured but becomes fairly sound below about elevation 165. The shale exhibits near horizontal bedding and contains some layers up to 12 inches thick of interbedded limestone and cemented conglomerate rock.

#### GROUNDWATER CONDITIONS

Following completion of each of the boreholes a piezo-meter was installed for groundwater level observation purposes. The installation details together with the latest set of water level readings taken on October 14, 1966 are shown on the Records of Boreholes and on Figure 1.

The latest set of water level readings indicate that

the groundwater level across the site is within the fill or underlying sandy silt at about elevation 245. The water level is some 4 feet below the valley floor at the pier locations and some 28 to 30 feet below the top of the roadway approach embankments at the abutment locations.

#### PROPOSED STRUCTURES

It is understood that the existing Colborne Street bridge over Sixteen Mile Creek is to be replaced by a 64 to 68 foot wide structure located on the same centreline as the existing bridge. The replacement structure is to consist of three continuous spans, the end spans being some 100 feet long and the centre span being 140 feet long.

It is further understood that the proposed structure is to be supported by end-bearing piles driven to bedrock. To offset horizontal thrusts it is proposed to drive batter piles. The existing bridge is to be left in place for use as falsework during construction of the replacement structure.

As the Colborne Street grade is to be raised it will be necessary to increase the height of the roadway approach embankments by some 5 to 10 feet.

### Foundations

Due to the compressible and soft nature of the organic silt overburden deposit underlying the site it is unsuitable for the support of the proposed continuous structure. Therefore, the structure should be founded on piles driven to practical refusal in the weathered or fractured upper zones of the bedrock.

The driving of piles to bedrock, and particularly within several feet of the existing bridge foundations, raises a number of problems:

- i) It is difficult to define the exact depth at which piles would meet practical refusal since, from past experience at the nearby Rebecca Street bridge and elsewhere, it is known that the soft weathered shale or fractured zones of bedrock tend to shatter at the pile tip, under normal driving energy. Thus, for a single pile, a penetration of up to 10 feet into the rock could take place.
- ii) Within a pile group the effect of pile driving can cause shattering beneath adjacent previously driven piles. Thus it may be necessary to re-drive piles to ensure that they are adequately seated.

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- iii) The piled foundations of the existing bridge at the site are apparently of timber and are understood to be taken to rock. However, should these piles not be driven to rock, it is probable that they will settle as piles for the new bridge are driven. Even if these piles are on rock (and being of timber they could not have been driven to penetrate the upper shattered zone without causing brooming or failure), shattering of rock beneath the tips could take place as the new piles are driven and this could also lead to settlement.
- iv) Driving of a group of piles, particularly at the abutment locations, could cause some remoulding of the organic deposit. This effect would only be temporary since there would be regain of strength with time (say 1 to 2 months), but could reduce the stability of the existing approach embankments. In order to assess the stability of the existing embankment, an analysis has been made of the east bank failure which took place in 1922. For a factor of safety,  $F = 1.0$ , an average undrained shear strength of 800 lb/sq.ft. is computed to act on the most probable failure surface, (Fig. 8). This computed value agrees with the average of the field results for the upper part of the organic deposit.

Analysing the existing bank with this shear strength acting the computed value of  $F$  is about 1.3. Driving of piles would reduce the stability by about 10 to 15 percent,  $F$  in this case being about 1.1 to 1.2. Therefore the stability of the bank could be marginal during pile driving. It should be noted that the limits of remoulding assumed, namely 20 feet for a 10 foot wide pile group, would probably apply only to a non-displacement type pile, such as a steel H-section. Should displacement piles be used, such as pipe sections driven closed end, or pre-cast concrete piles, then the limits of remoulding could well be larger than those assumed. For this reason, apart from others, the existing approach embankments should be carefully monitored during pile driving operations and construction loads on these embankments be severely restricted.

In considering the problems discussed above, it is proposed that piled foundations be specified, meeting the following requirements:

- a) Since pile penetration will be somewhat indefinite, it will be necessary that pile lengths can be increased, where required, by splicing or extensions.

- b) Since driving stresses at the pile tip could be extremely high, either in penetrating the stratum of limestone and shale fragments in a clayey matrix, or in penetrating the upper shattered zone of the rock, attention should be paid to the design of the pile tip. It may be necessary to reinforce the end of H-sections to prevent tip over-stressing. Herkules piles are normally cast with a solid steel tip and this has proved satisfactory in the past even in penetrating cobble or boulder layers.
- c) Driving energy should be high, to ensure that the pile is adequately seated in the weathered or fractured rock. Taking for example, a 12 BP 74 lb. H-pile section, a driving energy of greater than 20,000 ft.lb/blow should be employed. For an allowable load of 60 to 70 tons the driving resistance should be at least 100 blows for the last foot of driving and the final set 20 blows to the last inch. It should further be specified that following completion of driving of a pile group that each pile in the group be re-trapped to the required final set.
- d) The piles for the proposed structure should be located so that they will not encounter any of the existing timber piles at the site. To prevent twisting or dog-legging and

the like, requirements for pile straightness in driving should be rigidly enforced.

- e) Level readings should be periodically taken on the existing piers and abutments. These readings, in conjunction with periodic careful inspection of the piers and abutments, should be used to determine if there is progressive movement which could be detrimental and which is caused by the pile driving.

For these requirements, an H-pile section would be most suitable; however, provided that there is careful control of the pile driving, the use of Herkules pre-cast concrete piles could be considered as an alternative.


#### Approach Embankments


The stability of the existing east approach embankment has been examined and the factor of safety,  $F$ , of this embankment is computed to be about 1.3, allowing for end effects. This value for  $F$  is a limiting design value and any addition of load, such as increased height of fill, could cause detrimental movement. Should it be imperative to increase the height of the approach embankment it is suggested that this should be done by replacing part of the existing fill with light weight fill and increasing the height with

light weight fill, thus ensuring that there is no overall increase in load. For example, since the unit weight of water cooled slag is about 80 lb/cu.ft., to increase the height of fill by 5 feet would mean excavation and replacement of about 8 to 9 feet of the existing fill with slag and adding a further 5 feet of slag fill. This method has several advantages, since some embankment excavation will be required for construction of a new abutment and since it will be necessary to trim the end and side slopes to 2 horizontal to 1 vertical.

To prevent surficial instability due to erosion by surface water run-off the slopes should be sodded as soon as possible following completion of construction.

Settlement of the approach embankments when constructed as outlined above should be relatively minor.

  
J. B. Davis, P.Eng.

  
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VM:JBD:hdg  
66108  
October 20, 1966.

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## 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<sub>u</sub>, 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 <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 = $-\Delta e / \Delta \log_{10} \sigma'$
$c_e$	coefficient of consolidation
$T_v$	time factor = $cd/d^2$ ( $d$ , 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
$S_t$	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 BOREHOLE 1

LOCATION See Figure 1

BORING DATE SEPT. 6, 1966

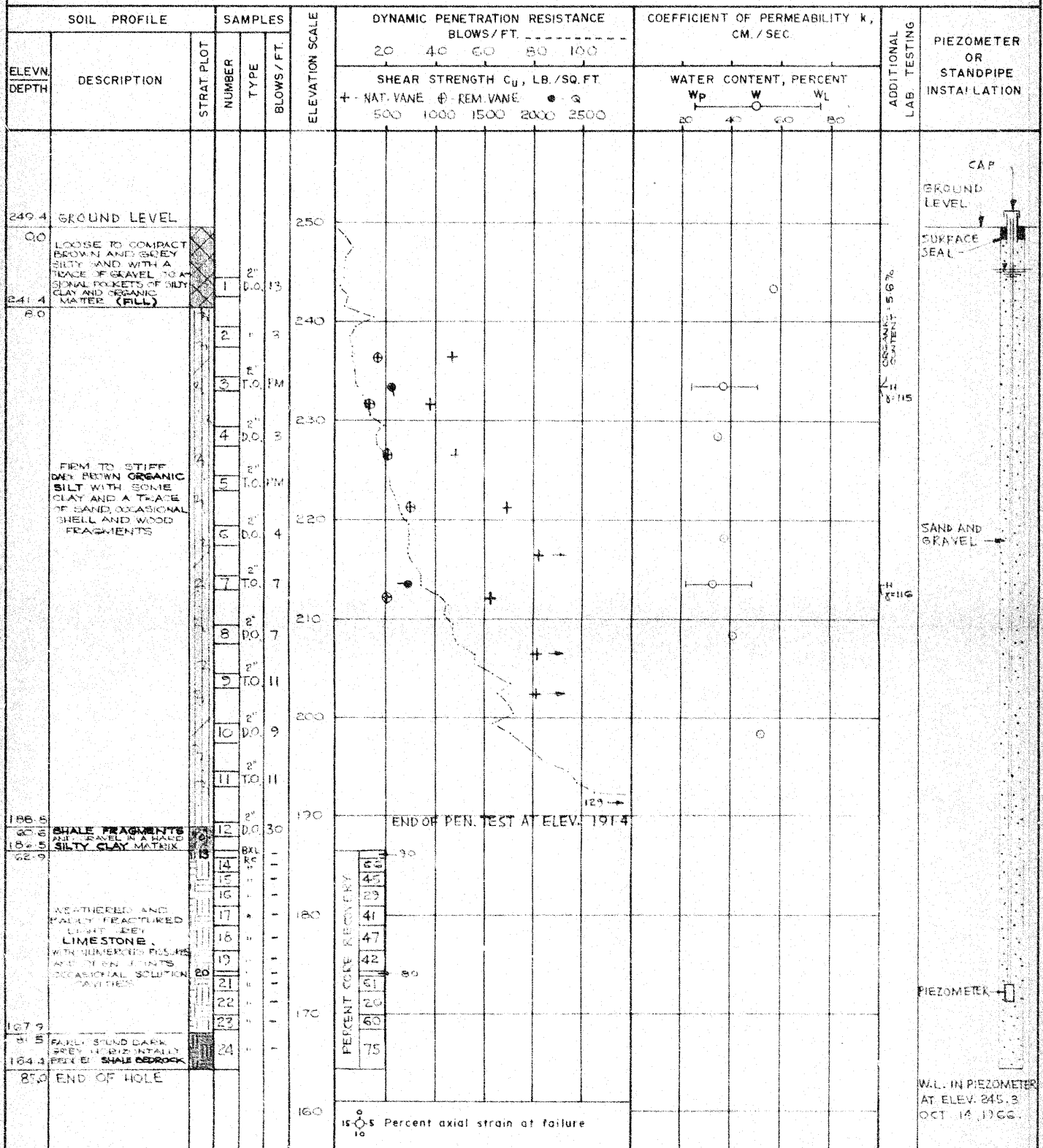
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

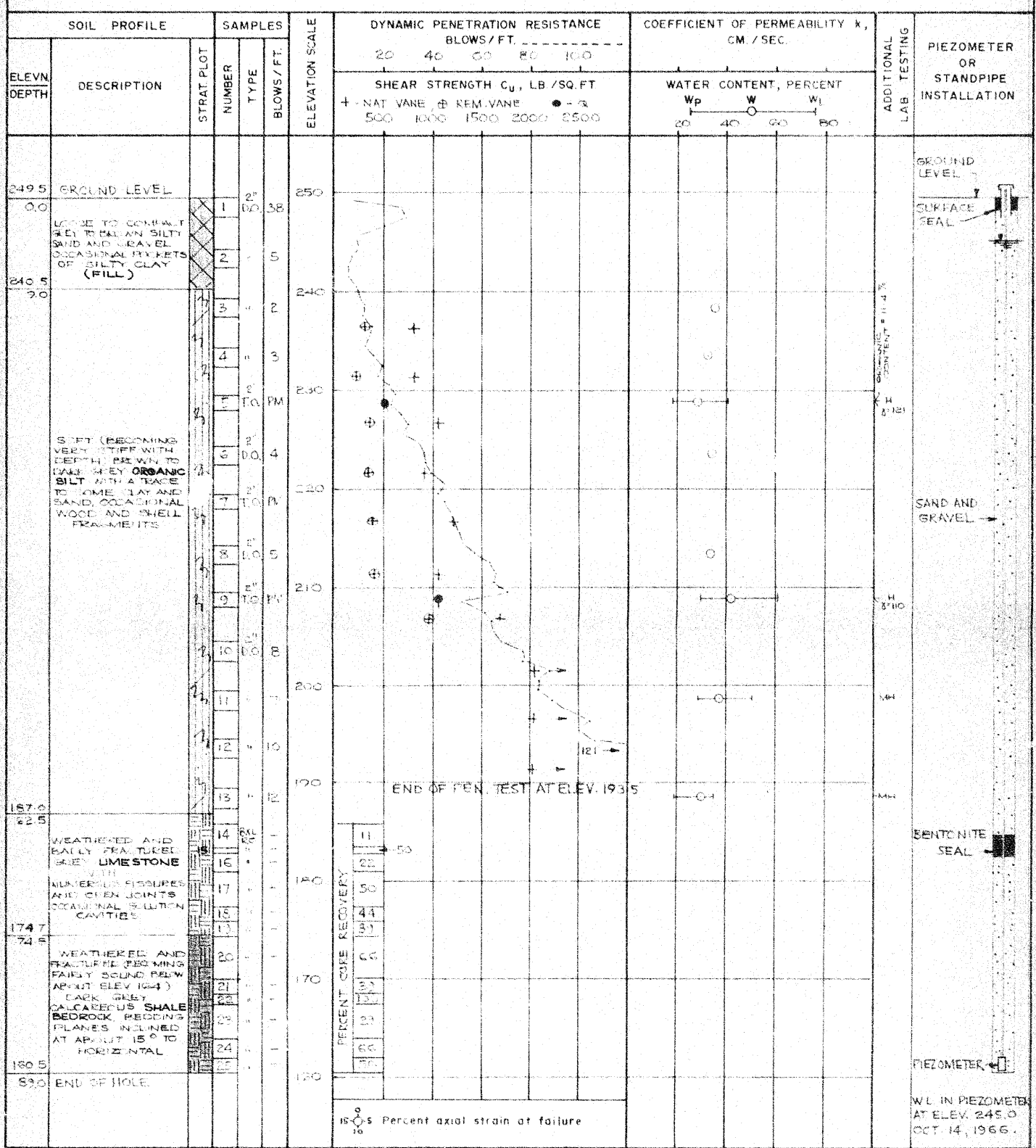

 VERTICAL SCALE  
 1 INCH TO 10' - 0"

GOLDER &amp; ASSOCIATES

 DRAWN *[Signature]*  
 CHECKED *[Signature]*

# RECORD OF BOREHOLE 2

LOCATION See Figure 1 BORING DATE SEPT. 12 - 16, 1966 DATUM GEODETTIC  
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER N X BX CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE  
 1 INCH TO 10' ±

GOLDER & ASSOCIATES

DRAWN *[Signature]*  
 CHECKED *[Signature]*

W.L. IN PIEZOMETER  
 AT ELEV. 245.0  
 OCT. 14, 1966.

# RECORD OF BOREHOLE 3

LOCATION See Figure 1

BORING DATE SEPT 16-21, 1966

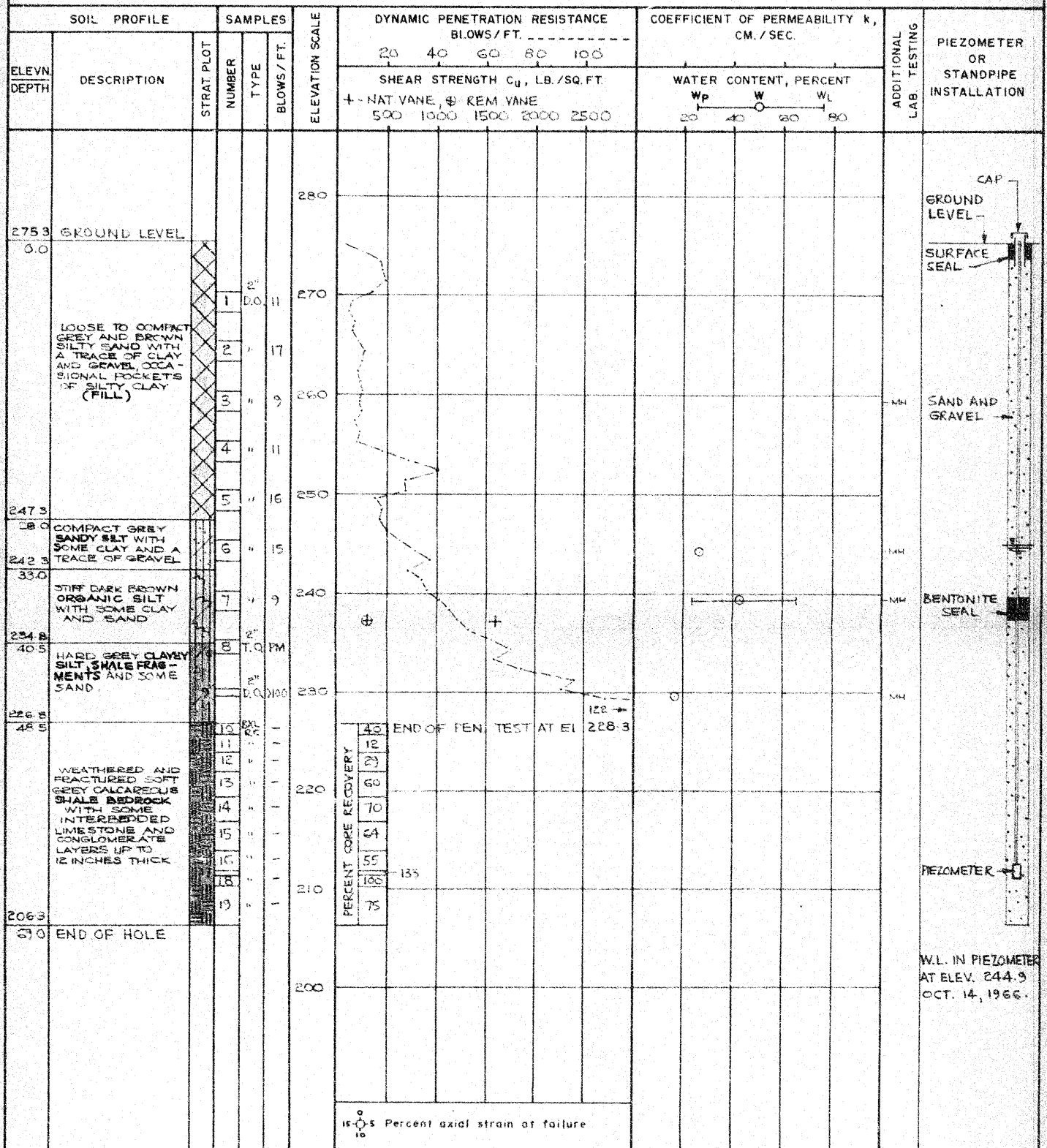
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

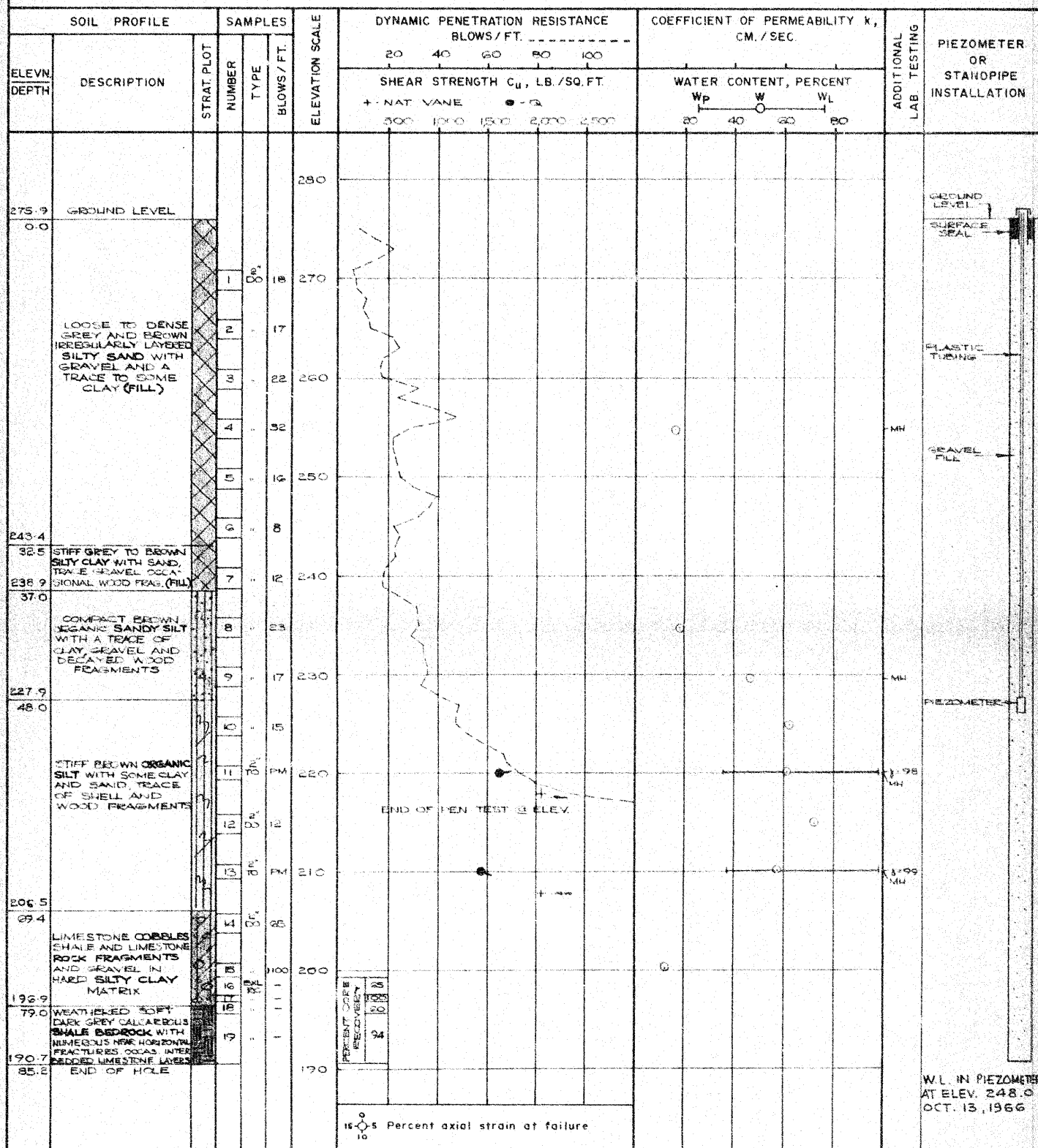

 VERTICAL SCALE  
 1 INCH TO 10' - 0"

GOLDER &amp; ASSOCIATES

 DRAWN *W.G.*  
 CHECKED *W.G.*

## RECORD OF BOREHOLE 4

LOCATION	See Figure 1	BORING DATE	SEPT. 22 - 26, 1966	DATUM	GEODETIC
BOREHOLE TYPE	WASH BORING	BOREHOLE DIAMETER	NX, PX CASING		
SAMPLER HAMMER WEIGHT 140 LB.	DROP 30 INCHES	PEN. TEST HAMMER WEIGHT 140 LB.	DROP 30 INCHES		



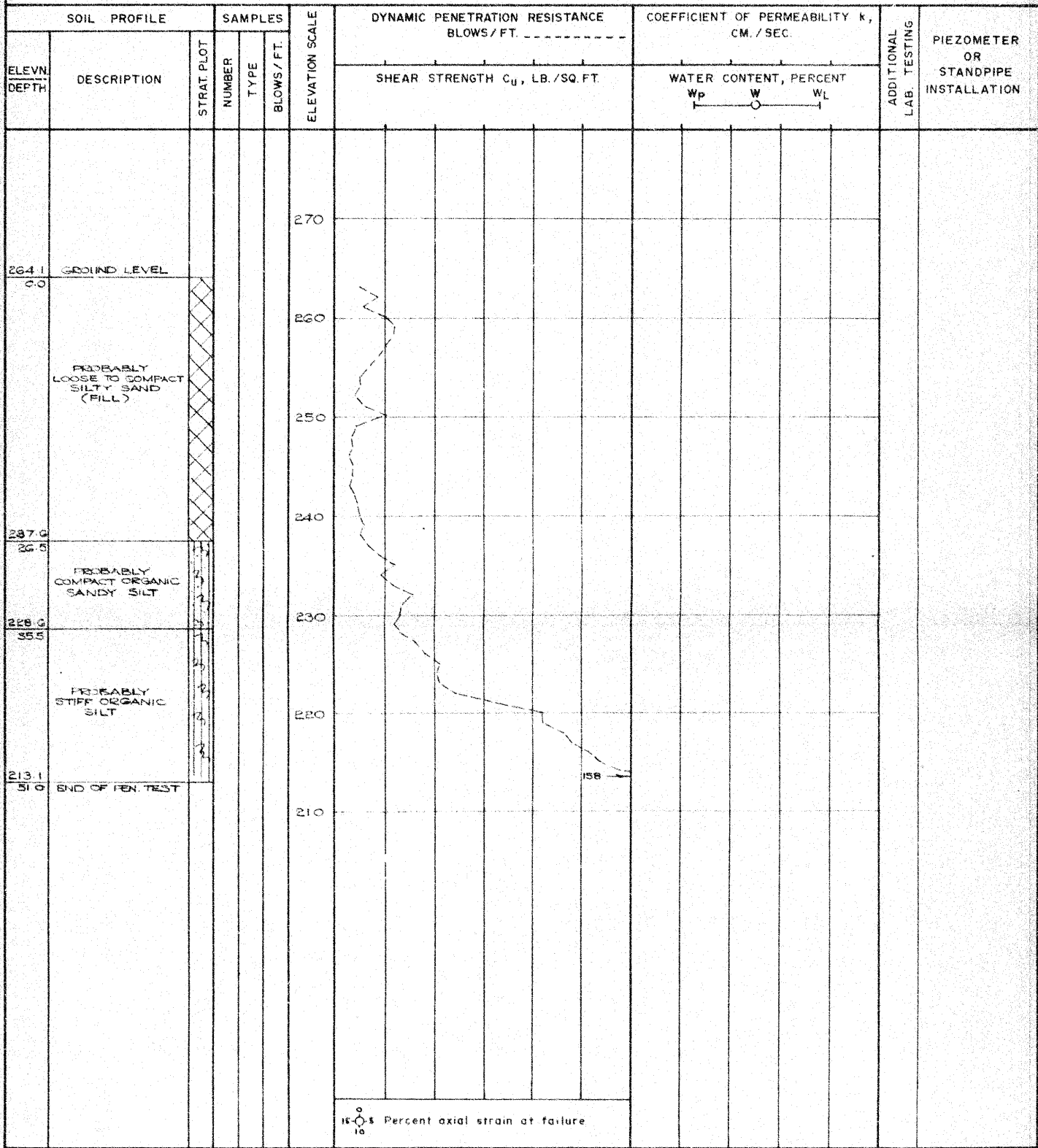
VERTICAL SCALE  
1 INCH TO 10'-0"

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DRAWN \_\_\_\_\_  
CHECKED \_\_\_\_\_

# PEN. TEST RECORD OF BOREHOLE 5

LOCATION See Figure 1 BORING DATE SEPT. 27, 1962 DATUM GEODETIC  
BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER -  
SAMPLER HAMMER WEIGHT - LB. DROP - INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

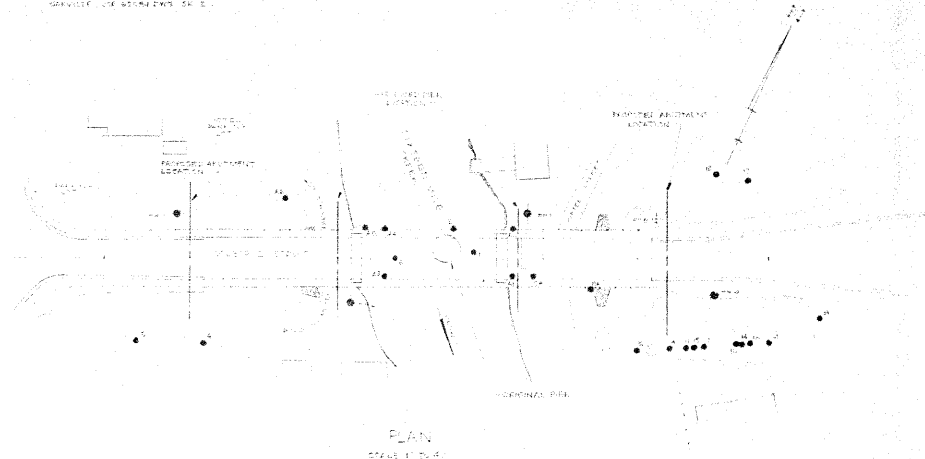


VERTICAL SCALE  
1 INCH TO 10'-0"

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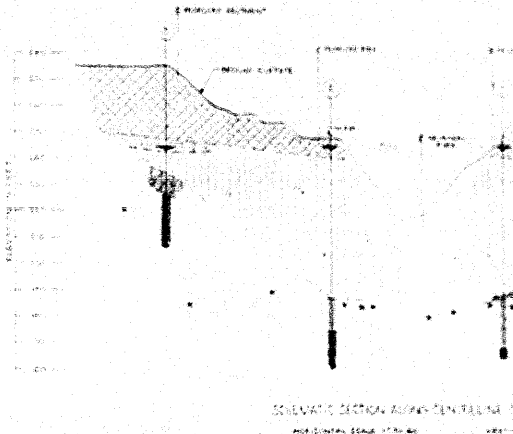
DRAWN  
CHECKED

REFERENCED LIT. DRAWING AND ASSOCIATED PRELIMINARY  
PLAN AND PROFILE, COLBORNE ST. BRIDGE,  
BRIDGEVILLE, JOE BOON PWS, SR. 2.



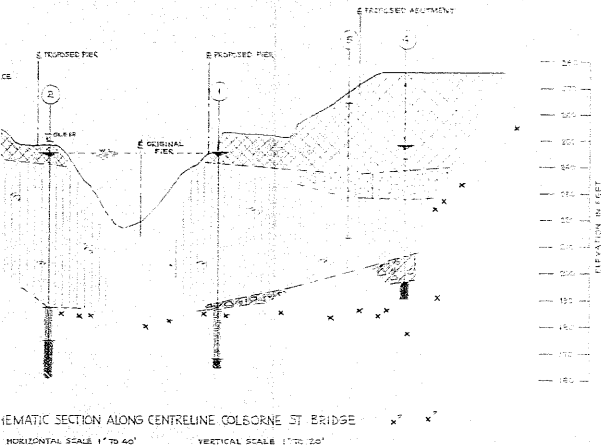
PLAN  
SCALE 1" = 40'

REFERENCE: LIT. DRAWING AND ASSOCIATED PRELIMINARY  
PLAN AND PROFILE, COLBORNE ST. BRIDGE,  
BRIDGEVILLE, JOE BOON PWS, SR. 2.



ELEVATION SECTION ALONG CENTERLINE

VERTICAL SCALE 1" = 10'



## LEGEND

- ◆ BENCHMARK AT ELEVATION 238.00
- 1 BENCHMARK AT ELEVATION 238.00
- W IN PERMETER TEST 14, 100
- PENETRATION TEST IN PLACE, DEPTH 100
- PENETRATION TEST IN PLACE, DEPTH 100
- BORING IN PLACE, DEPTH 100

## STRATIGRAPHY

- 10' TO 15' TO 20' TO 25' TO 30' TO 35' TO 40' TO 45' TO 50' TO 55' TO 60' TO 65' TO 70' TO 75' TO 80' TO 85' TO 90' TO 95' TO 100' TO 105' TO 110' TO 115' TO 120' TO 125' TO 130' TO 135' TO 140' TO 145' TO 150' TO 155' TO 160' TO 165' TO 170' TO 175' TO 180' TO 185' TO 190' TO 195' TO 200' TO 205' TO 210' TO 215' TO 220' TO 225' TO 230' TO 235' TO 240'
- 10' TO 15' TO 20' TO 25' TO 30' TO 35' TO 40' TO 45' TO 50' TO 55' TO 60' TO 65' TO 70' TO 75' TO 80' TO 85' TO 90' TO 95' TO 100' TO 105' TO 110' TO 115' TO 120' TO 125' TO 130' TO 135' TO 140' TO 145' TO 150' TO 155' TO 160' TO 165' TO 170' TO 175' TO 180' TO 185' TO 190' TO 195' TO 200' TO 205' TO 210' TO 215' TO 220' TO 225' TO 230' TO 235' TO 240'
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SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT VARIOUS LITHOLOGICAL SITES. THE SOIL STRATIGRAPHY SECTION STRATIGRAPHY HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

Drawn, OCT. 19, 1966

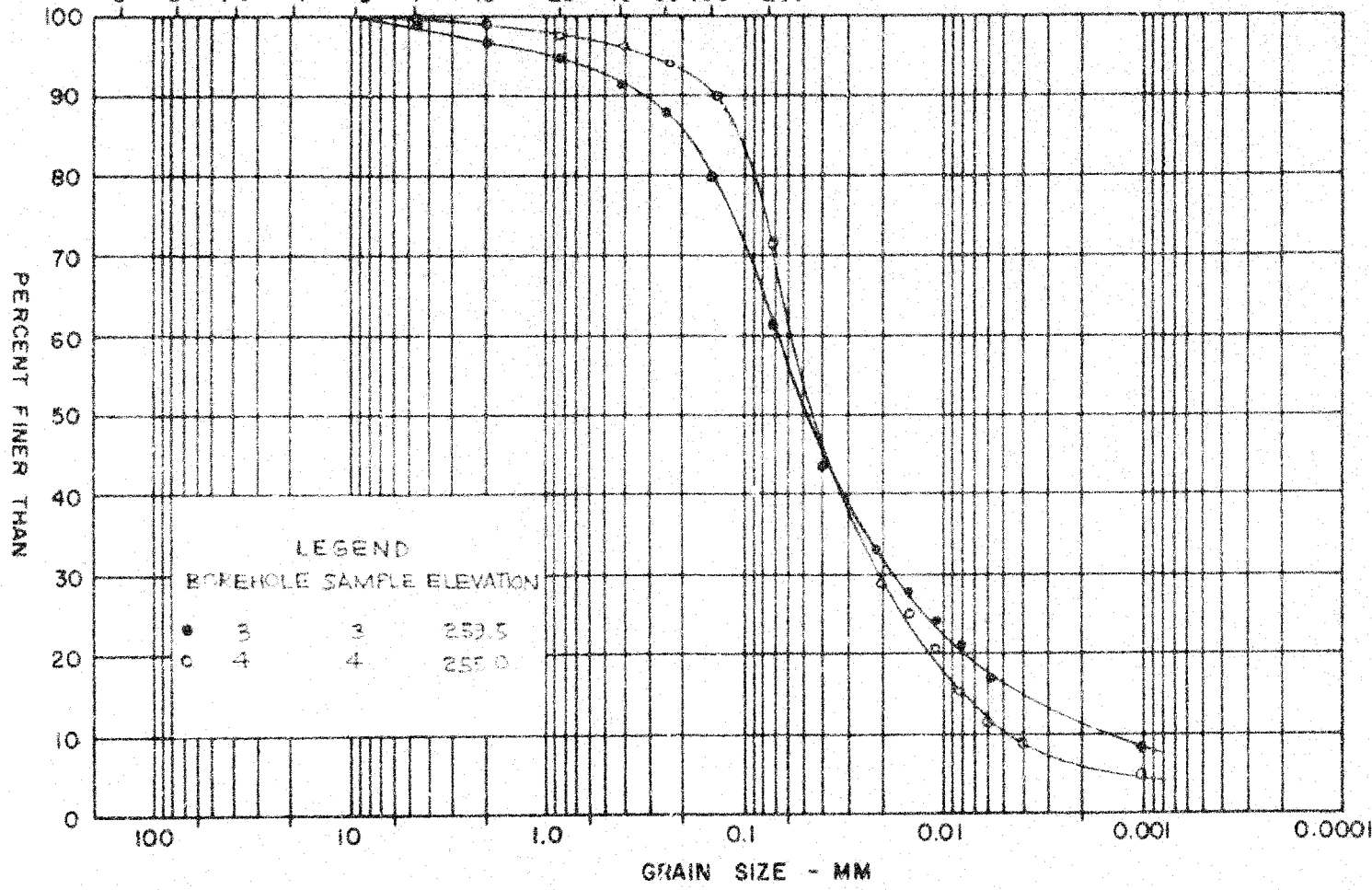
GOLDER & ASSOCIATES

MADE BY  
CHAS. GOLDER  
1966

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

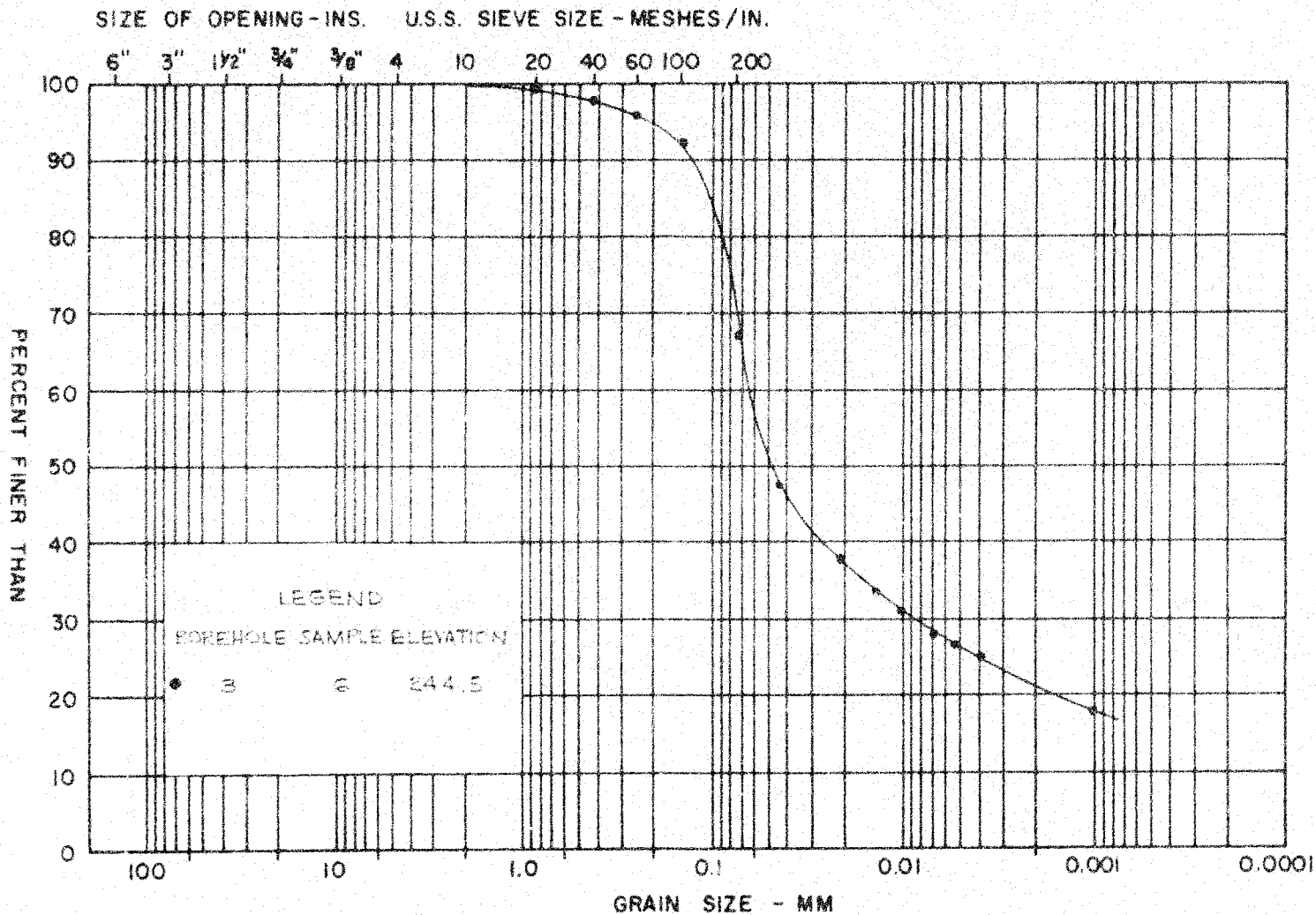


GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION  
(FILL)

FIGURE 1

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

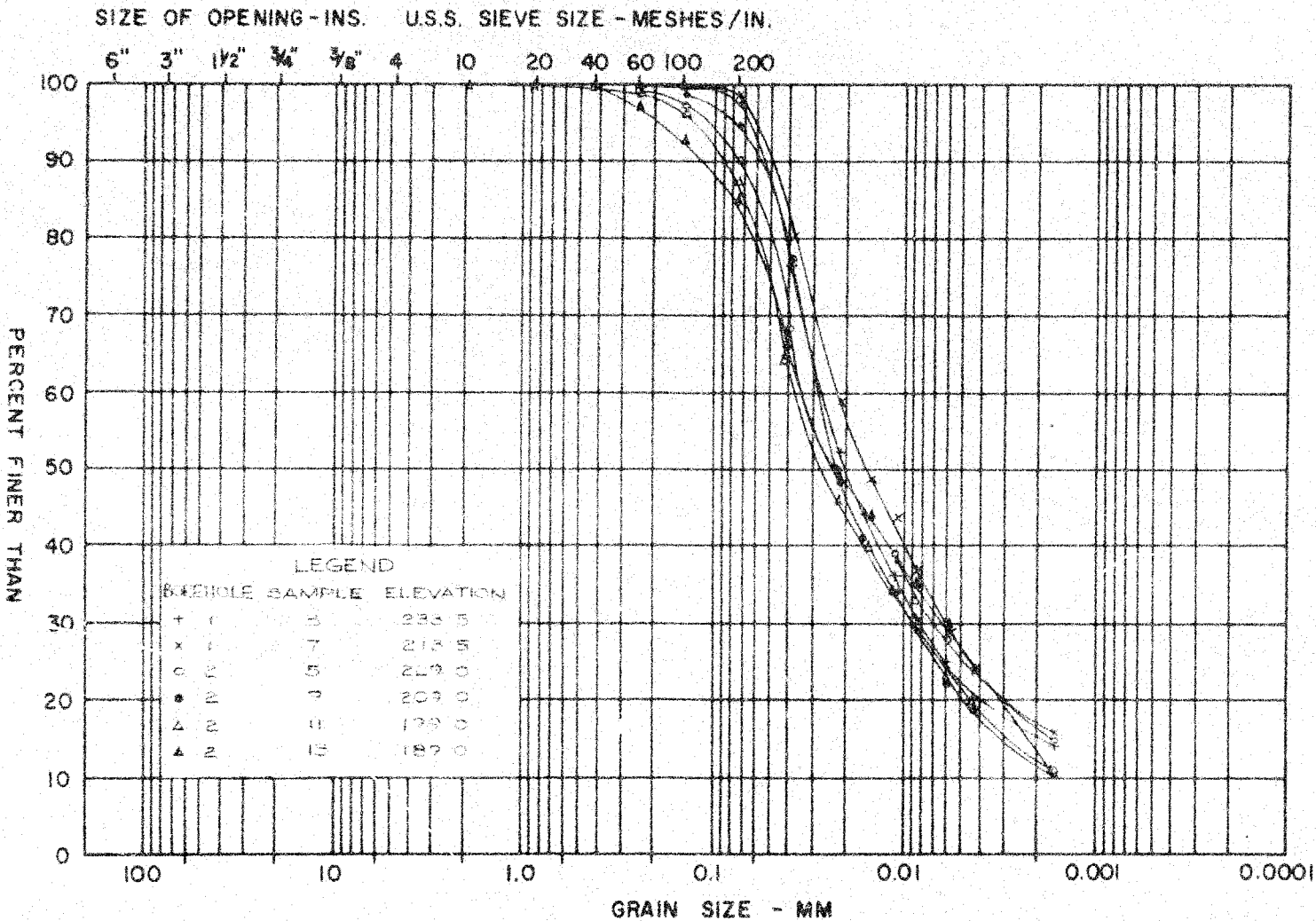
GRAIN SIZE DISTRIBUTION  
SANDY SILT

FIGURE

(1)

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

M.I.T. GRAIN SIZE SCALE



GRAIN SIZE DISTRIBUTION  
ORGANIC SILT STRATUM

FIGURE 4

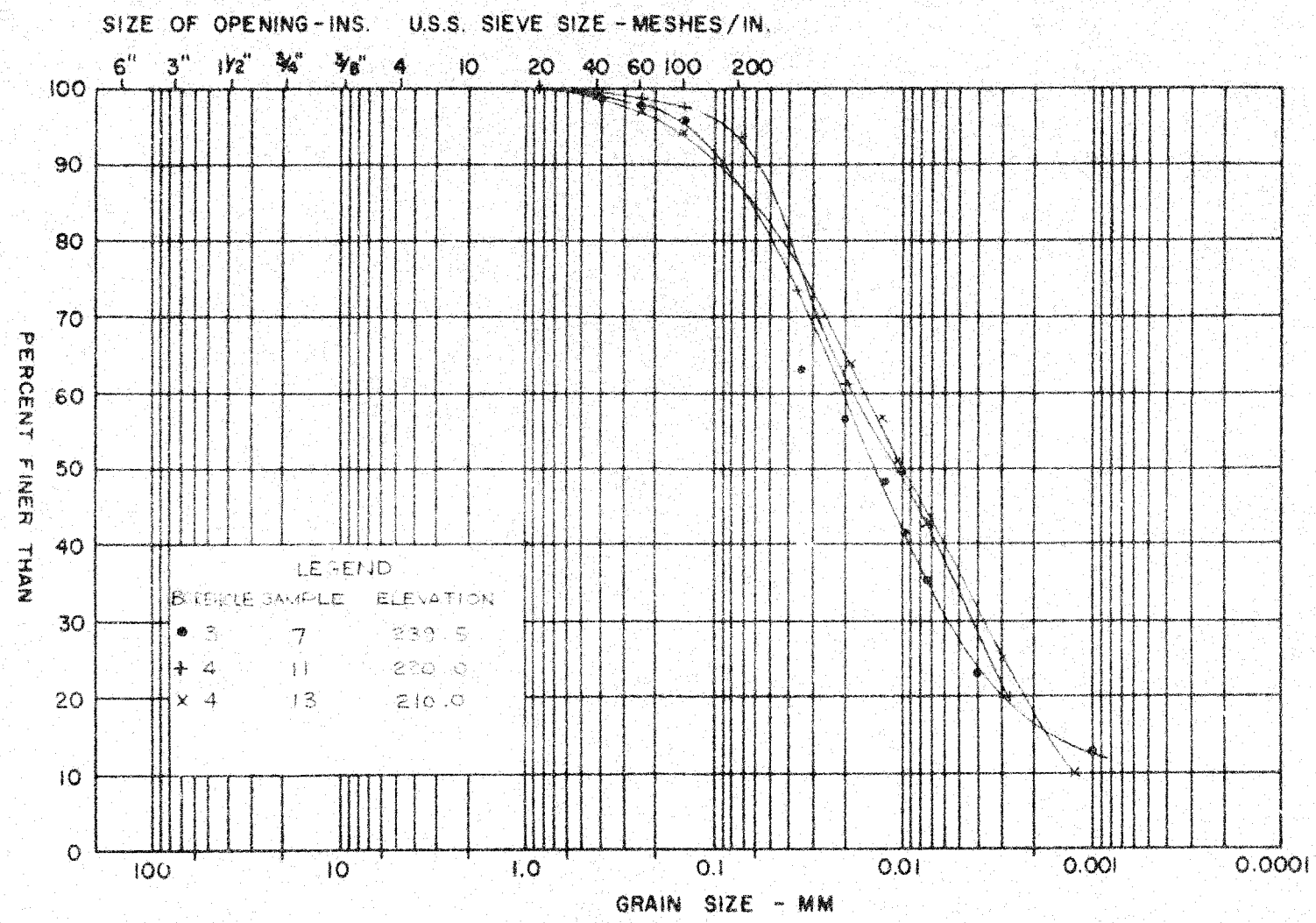
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

M.I.T. GRAIN SIZE SCALE

GRAIN SIZE DISTRIBUTION  
ORGANIC SILT STRATUM

FIGURE 2

GOLDER & ASSOCIATES

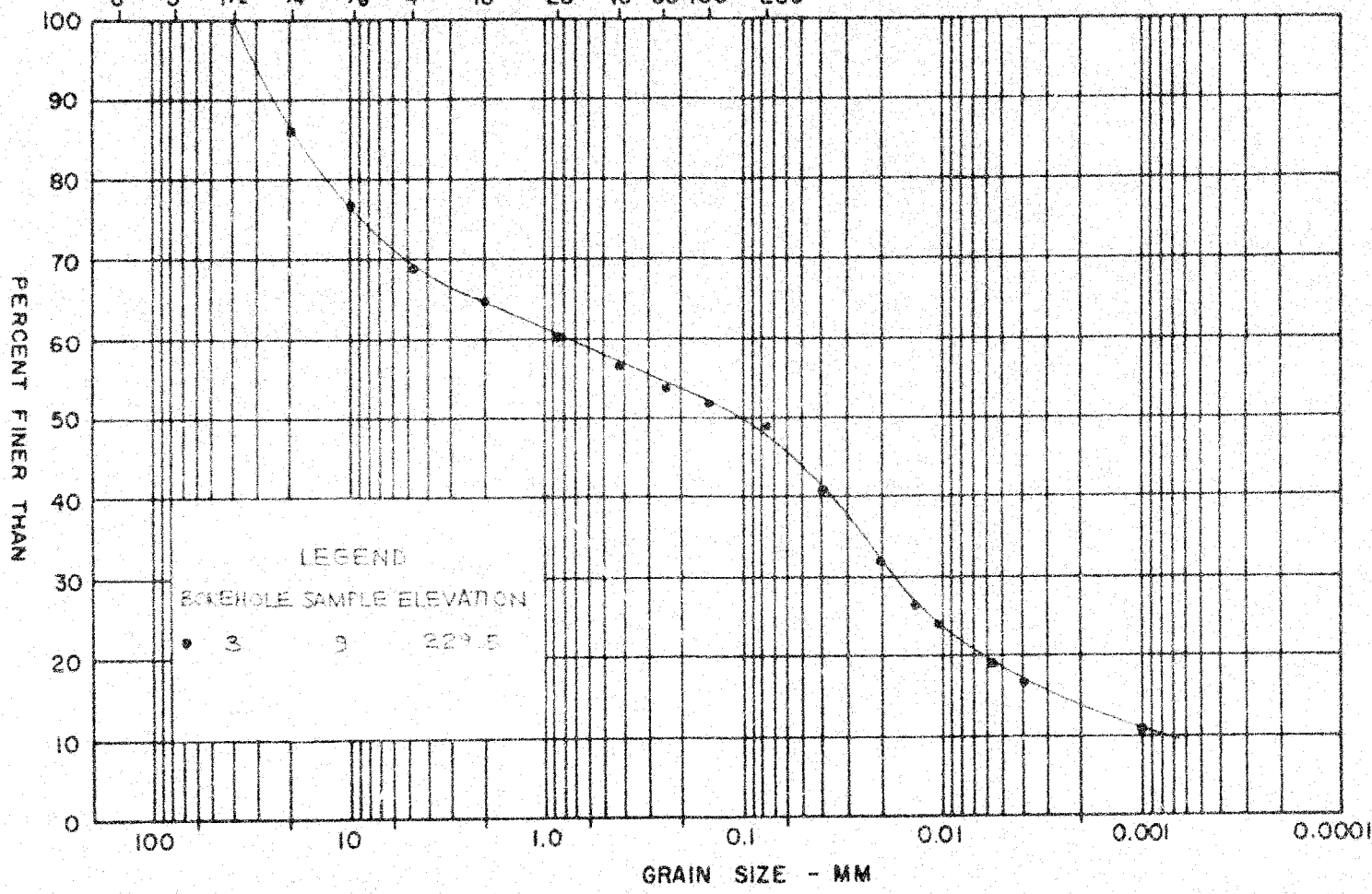


COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

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



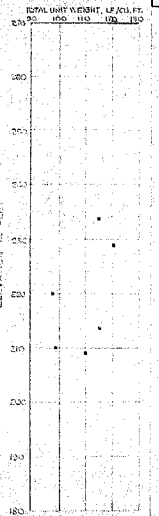
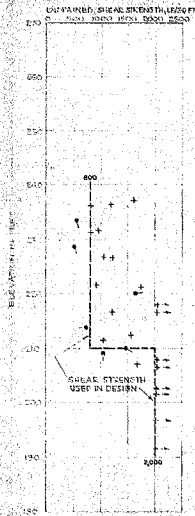
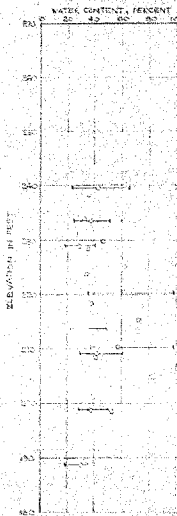
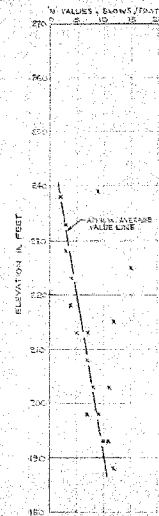
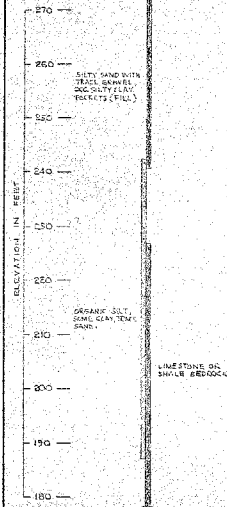
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT AND ROCK FRAGMENTS

FIGURE 10

GOLDER & ASSOCIATES

# SIMPLIFIED SOIL STRATIGRAPHY (SHOWING RANGE OF ELEVATION)



## SUMMARY OF ENGINEERING PROPERTIES ORGANIC SILT STRATUM

FIGURE 7

### LEGEND

STANDARD PENETRATION RESISTANCE PLOT:

x, N' VALUES, BLOWS/FOOT

WATER CONTENT AND ATTERBERG LIMITS PLOT:

W, WATER CONTENT, PERCENT

W<sub>p</sub>, PLASTIC LIMIT

W<sub>L</sub>, LIQUID LIMIT

W<sub>p</sub> — W — W<sub>L</sub>

UNDRAINED SHEAR STRENGTH PLOT:

+, IN SITU VANE SHEAR TEST

•, UNDRAINED TRIAXIAL COMPRESSION TEST

10 ± 5 PERCENT AXIAL STRAIN AT FAILURE

TOTAL UNIT WEIGHT PLOT:

\*, UNIT WEIGHT, LB./CU. FT.

Drawn, OCT. 19, 1986

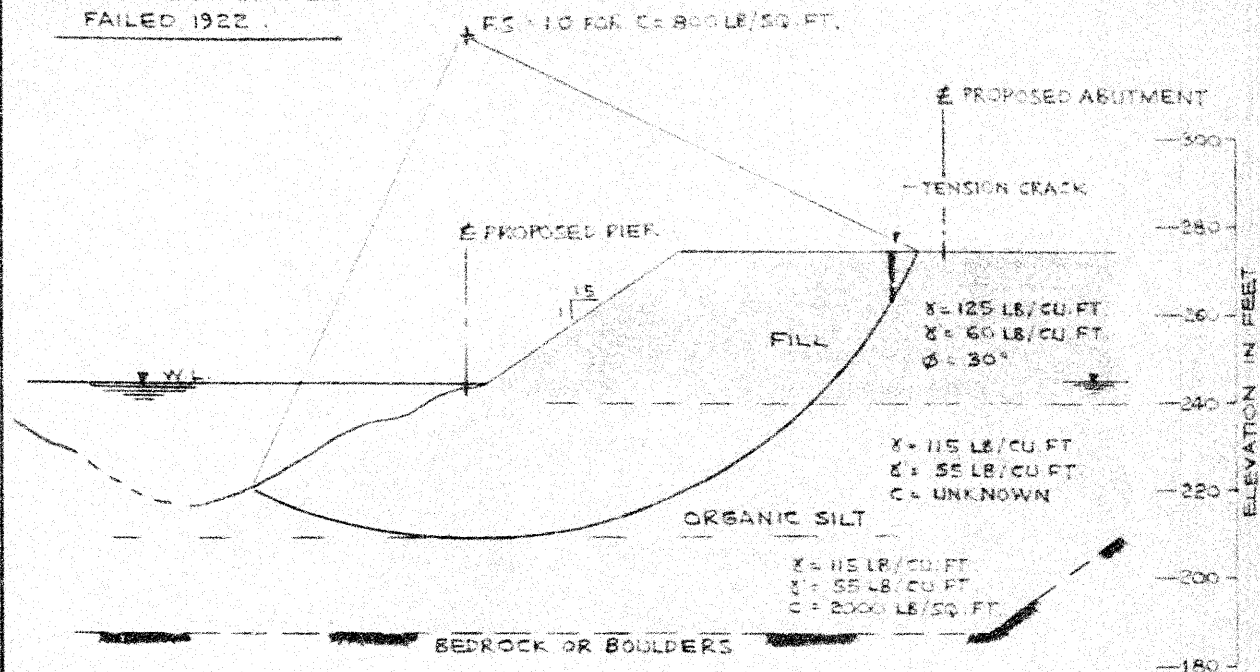
GOLDER & ASSOCIATES

MADE  
CHD  
2005

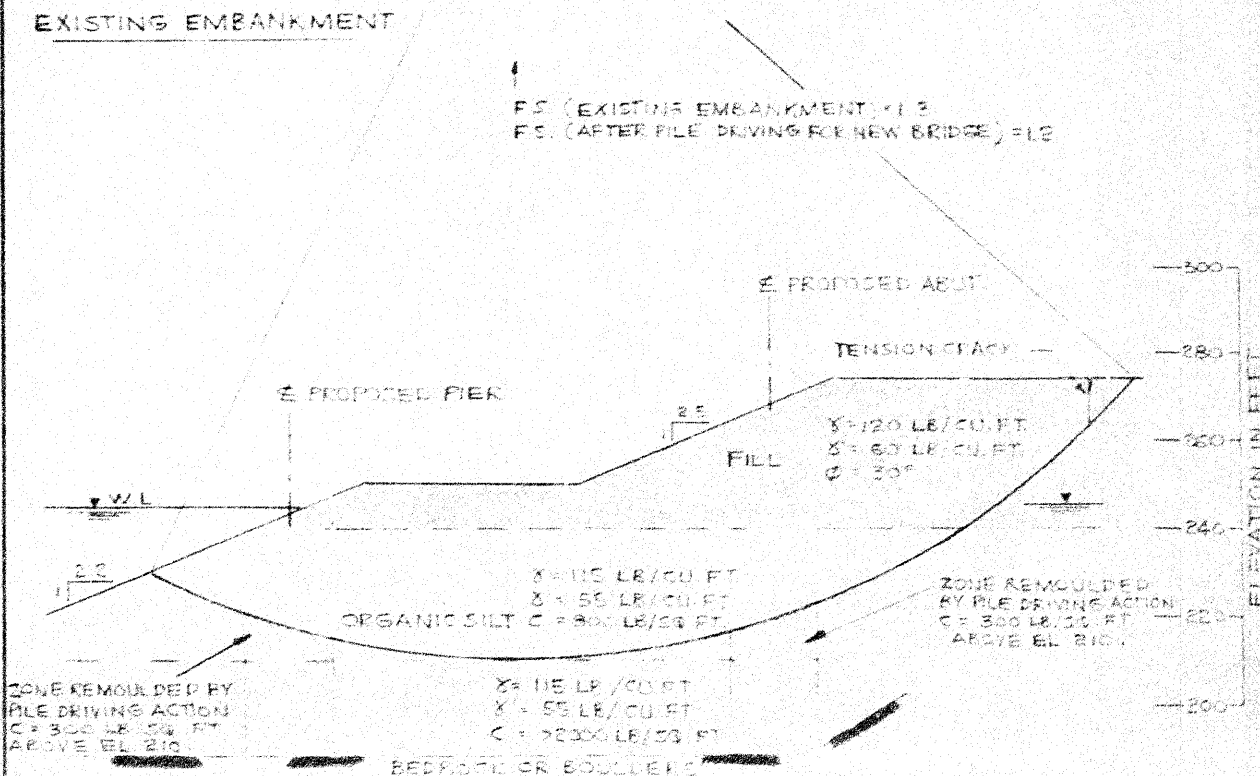
# SAMPLE RESULTS-TOTAL STRESS STABILITY ANALYSES EAST APPROACH EMBANKMENT

FIGURE 8

ORIGINAL EMBANKMENT  
FAILED 1922



EXISTING EMBANKMENT



SCALE 1" TO 40'

GOLDER & ASSOCIATES

Made *[Signature]*  
Chkd *[Signature]*  
Appd *[Signature]*