

#69-F-226 M

SITE 6-254

HURON CHURCH

LINE BRIDGE

H. Q. GOLDER & ASSOCIATES LTD.

SOIL AND FOUNDATION ENGINEERS

HEAD OFFICE - TORONTO, ONTARIO

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

747 HYDE PARK ROAD  
LONDON, ONTARIO  
471-9600

REPORT

TO

M. M. DILLON LIMITED

ON

SUBSURFACE INVESTIGATION

FOR

PROPOSED HURON CHURCH LINE BRIDGE

WINDSOR

69-F-2264 ONTARIO

Distribution:

15 copies - M. M. Dillon Limited,  
Windsor, Ontario.

2 copies - H. Q. Golder & Associates Ltd.,  
London, Ontario.

February, 1969

69305

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ABSTRACT

The results of a subsurface investigation at the site of the proposed bridge crossing of the Grand Marais Drain by Highway 3 in the City of Windsor, Ontario, are reported. The new structure will replace an existing two lane bridge.

At this site, a major stratum of very stiff to stiff grey silty clay approximately 100 feet thick overlies a thin basal dense sand and till layer. Probable bedrock is approximately 110 feet below existing ground surface.

The bridge abutments may be supported on spread footings founded on the very stiff grey silty clay at elevation 583.5 as proposed by M. M. Dillon Limited, Consulting Engineers responsible for the design of the structure. Settlement of each abutment has been calculated to be less than 1 inch using the average net foundation pressure of 0.9 tons per square foot proposed. Stability analyses carried out on the slopes at the bridge location show that both the end of construction and long term stability conditions are adequate. Protection of the slopes at the abutment locations with a concrete slab surface as proposed is mandatory.

## INTRODUCTION

H. Q. Golder & Associates Ltd. has been retained by M. M. Dillon Limited, Windsor, to carry out a subsurface investigation at the site of the proposed new bridge crossing of the Grand Marais Drain by Highway 3 in the City of Windsor, Ontario.

The purpose of the investigation was to define the soil and ground-water conditions at the site and to make recommendations for the design of abutment foundations. This involved a study of the bearing capacity and settlement characteristics of the various soil strata and an analysis of the effect of the additional surcharge due to abutment foundations on the slopes of the proposed widened and realigned Grand Marais Drain.

## PROCEDURE

Two boreholes were put down to assumed bedrock at depths of 110 and 111 feet during the period of January 23 to 31, 1969, using a trailer mounted power auger supplied and operated by P. V. K. and Sons. The borehole locations are shown in the sketch plan in Figure 1.

The boreholes were sampled at 2.5 foot intervals to about 40 feet below existing ground level and at 5.0 to 10.0 foot intervals thereafter. Standard penetration tests taken to determine "N" values of the various strata were supplemented with 2 7/8 inch diameter thin walled Shelby tube samples taken to obtain relatively undisturbed samples for strength and compressibility testing. Field vane tests were carried out between the above noted samples to determine the undrained shear strength of the stiff silty clay.

All samples were brought to our London laboratory for detailed examination and representative testing. Detailed water content determinations with accompanying Atterberg limits and grain size distribution tests were carried out to assist in soil classification of the various strata. Two consolidation tests were performed on relatively undisturbed samples of the silty clay to obtain reliable consolidation data for settlement analysis.

Details of all field and laboratory testing are shown in the Record of Boreholes sheets and in Figures 2 to 5.

Ground-water conditions in the boreholes were observed during the field work which was supervised throughout by a soils engineer from our staff. Existing ground level elevations at the borehole locations were supplied by M. M. Dillon Limited, Windsor, and are referred to geodetic datum.

A study of the slope stability of the realigned and widened Grand Marais Drain is presently being carried out by H. Q. Golder & Associates Ltd. (report No. 68722 to be published) for M. M. Dillon Limited, Windsor. Effective stress shear strength parameters determined for the silty clays have been used to analyse the stability of the slopes at the abutment positions for this structure.

#### SITE AND GEOLOGY

The proposed new overpass of the Grand Marais Drain by Highway 3 is situated in the west end of the City of Windsor, Ontario on the main route to the Ambassador Bridge. The topography is very flat being typical of that found in the Windsor area.

The majority of the present overburden at the site is part of the extensive clay deposit known as the St. Clair Clay Plain which varies from 90 to 140 feet in thickness throughout the Windsor area. This clay deposit is believed to have been laid down during the later stages of the Wisconsin glaciation. The upper 20 feet of the stratum has been heavily overconsolidated due to desiccation with the remainder of the deposit being only slightly overconsolidated above existing overburden pressures.

#### SOIL CONDITIONS

Underlying a thin surficial layer of fill and loose sand, there exists a major stratum of grey silty clay about 95 feet thick. Beneath the clay stratum there is a thin layer of dense sand and basal till which overlies the probable bedrock formation.

Underlying a 1 foot layer of topsoil, there is a 3 to 3.5 foot layer of compact grey to black sandy silt fill over a thin layer of loose medium to coarse sand which terminates about elevation 589.



Beneath these surficial deposits, there is a major stratum of grey silty clay which is very stiff to about elevation 570. Below this, the clay becomes stiff exhibiting an average undrained shear strength of 1250 pounds per square foot. The silty clay gave "N" values as measured in the standard penetration tests in the range of 9 to 25 blows per foot with the higher values being obtained in the upper 20 feet of the stratum. Values of undrained shear strength versus depth for the stiff clay are plotted on the Record of Borehole sheets.

The water content of the clay stratum above elevation 547 lies between 16 and 27 per cent with an average value being 20 per cent. This deposit of low to medium plasticity has liquid and plastic limits of 32 and 17 respectively. Typical grain size distribution curves for this silty clay are shown in Figure 5.

Two consolidations tests were carried out on trimmed specimens from the tube samples recovered from 20 and 40 foot depths in borehole 1. The results which gave a rebound compression index,  $C_r$ , of 0.03 and a virgin compression index of 0.2 are plotted on

Figures 2 and 3. The apparent preconsolidation pressures at depths of 20 and 40 feet as determined from the consolidation tests are 4.0 and 2.5 tons per square foot respectively as shown on Figure 4.

Below elevation 547, there is a 10 foot layer of stiff stratified grey silty clay. The average water content of this layer is in the range of 21 to 35 per cent with 28 per cent being a representative value. This stratified layer has liquid and plastic limits of 36 and 17 respectively.

The portion of the silty clay stratum which lies beneath the above described deposit is in a very stiff condition and has index properties similar to the clay found above elevation 547. This lower till-like clay gave "N" values of 16 to 25 blows per foot and has an average water content of 20 per cent increasing to 30 per cent at the bottom of the stratum where the material becomes more plastic in nature.

Directly underlying the clay stratum, a 6 to 10 foot stratum of dense medium to coarse sand was encountered

overlying the dense to very dense basal till. This sandy till is approximately 4 feet thick and overlies the probable bedrock which offered complete refusal to both augering and the standard penetration tests.

Ground-water conditions were observed in the boreholes during drilling and water levels stabilized at about elevation 592. The boreholes were dry during augering but filled to about elevation 570 when the lower sand deposit was intercepted.

#### DISCUSSION

Our comments and recommendations pertaining to the influence of soil conditions at the site on the proposed design are submitted for your consideration under the following headings:

1. Allowable Bearing Capacity

The shear strength of the silty clay stratum immediately below the proposed founding elevation of 583.5 for abutment footings is adequate to safely support the average design bearing pressure

of 0.9 tons per square foot. The calculated safe allowable net bearing pressure (factor of safety of 3.0 applied to the average undrained shear strength of 2000 pounds per square foot) making allowance for the position of the footing with respect to the slope of the storm drain channel was found to be 1.5 tons per square foot. The maximum footing edge pressure should not exceed 2.0 tons per square foot (factor of safety of 2.0 on ultimate bearing capacity).

## 2. Settlements

In order to estimate the long term settlements of the abutments positioned and founded at the proposed elevation of 583.5, two consolidation tests were carried out on representative undisturbed samples taken from the strata within the zone of influence of the loaded footing. The pressure-void ratio curves obtained from the tests show the material to be overconsolidated. The vertical stress distribution with depth relative to existing effective vertical overburden stress as well as

the probable preconsolidation pressure profile used in settlement calculation are shown on Figure 4. The maximum total settlement due to consolidation resulting from the applied footing pressure of 0.9 tons per square foot was calculated to be about 1 inch. This value is well within the allowable settlement for single span structures of this type.

### 3. Slope Stability

The proposed structure spans the Grand Marais storm drain channel which is cut through the stiff desiccated crust of the silty clay deposit in this area of Windsor. An extensive study of the stability of the channel slopes for this drain has been made by H. Q. Golder & Associates Ltd. for M. M. Dillon Limited and the complete results will be presented in our report no. 68722 presently being prepared.

Stability analyses have been carried out using both the total stress and effective stress method

of analysis for slopes varying from 1 horizontal to 1 vertical to 2 horizontal to 1 vertical. For the 1 1/2 horizontal to 1 vertical slope proposed at the bridge site location, the end of construction case factor of safety is 4.0. The long term stability was found to govern the slope angle and for the 1 1/2 horizontal to 1 vertical slope proposed, the factor of safety was found to be 1.27. This value was determined using effective shear strength parameters of  $c' = 100$  pounds per square foot and  $\phi' = 28$  degrees. It has been assumed that the open channel slopes will be lined with concrete slabs and that a granular bedding will be provided beneath the concrete slabs to provide drainage. Details of slope treatment for various channel depths will be provided in our report on the channel slopes previously referred to, numbered 68722.

#### 4. General

Excavation for the abutment footings will be in the shallow surface stratum of sand to sandy silt

between elevation 597 and 589. Side slopes of at least 1 horizontal to 1 vertical will be required through this water bearing sand layer. Excavation in the stiff clay below elevation 589 may be carried out with vertical side walls. Seepage into the excavation will be minor and easily handled by low capacity sump pumps.

*V. R. Eakin.*

V. R. Eakin, P. Eng.



*L. G. Soderman*

L. G. Soderman, P.Eng.

VRE/LGS:cmm  
69305  
February, 1969

## 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 logarithm 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 intercept
$\phi'$	effective angle of shearing resistance, or friction
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\mu$	coefficient of friction
$S_r$	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 BOREHOLE 1

**LOCATION**

See Figure 1

**BORING DATE** JAN 23-28, 1969

DATUM GEODETIC

BOREHOLE TYPE POWER AUGER BORING

**BOREHOLE DIAMETER** 4.5'

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST	HAMMER WEIGHT	LB.	DROP	INCHES
1	10	10	10	10
2	10	10	10	10
3	10	10	10	10
4	10	10	10	10
5	10	10	10	10
6	10	10	10	10
7	10	10	10	10
8	10	10	10	10
9	10	10	10	10
10	10	10	10	10
11	10	10	10	10
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13	10	10	10	10
14	10	10	10	10
15	10	10	10	10
16	10	10	10	10
17	10	10	10	10
18	10	10	10	10
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80	10	10	10	10
81	10	10	10	10
82	10	10	10	10
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85	10	10	10	10
86	10	10	10	10

[illegible]

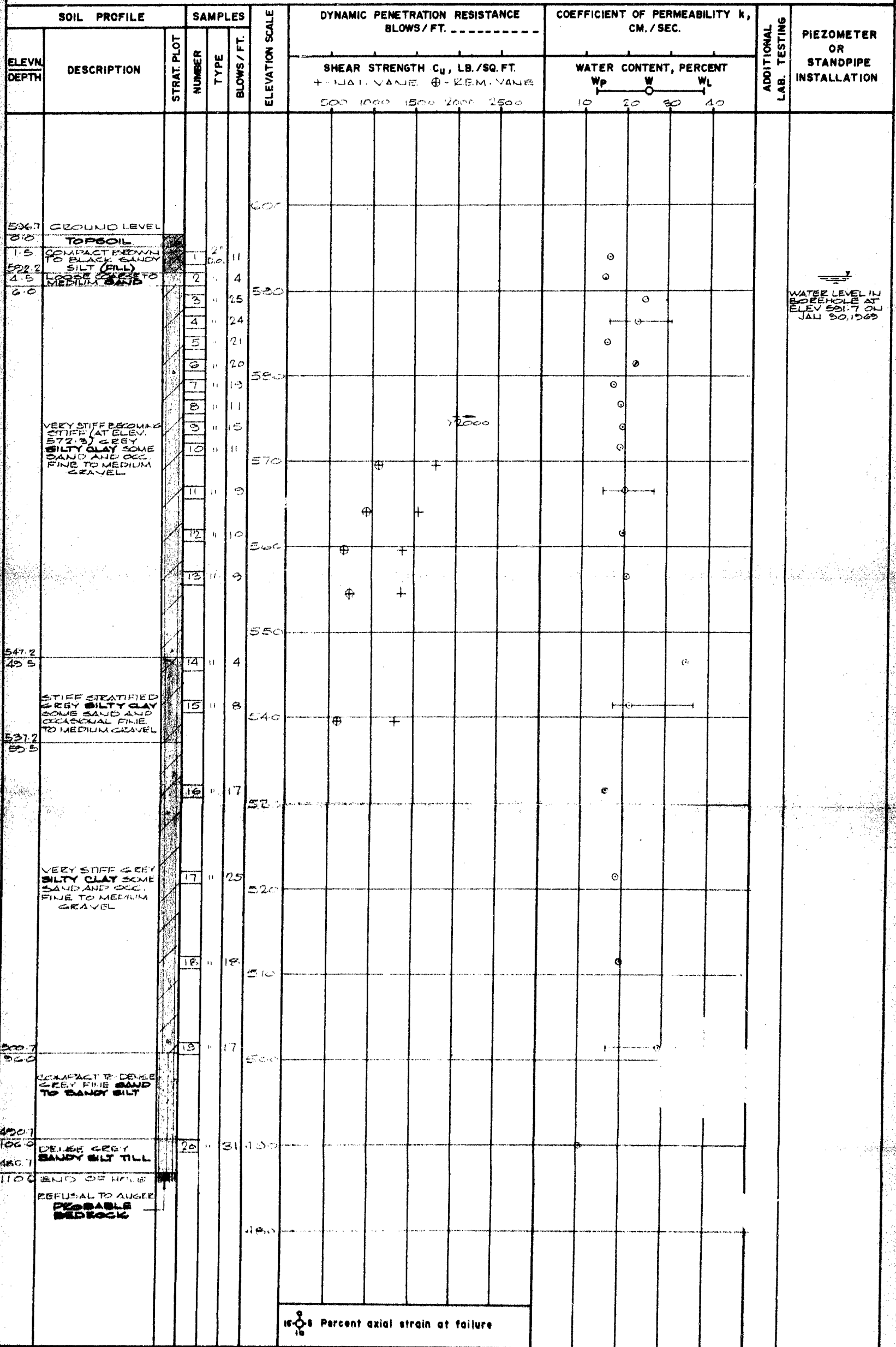
VERTICAL SCALE  
1 INCH TO 10'-0"

**GOLDER & ASSOCIATES**

DRAWN   
CHECKED 

RECORD OF BOREHOLE 2

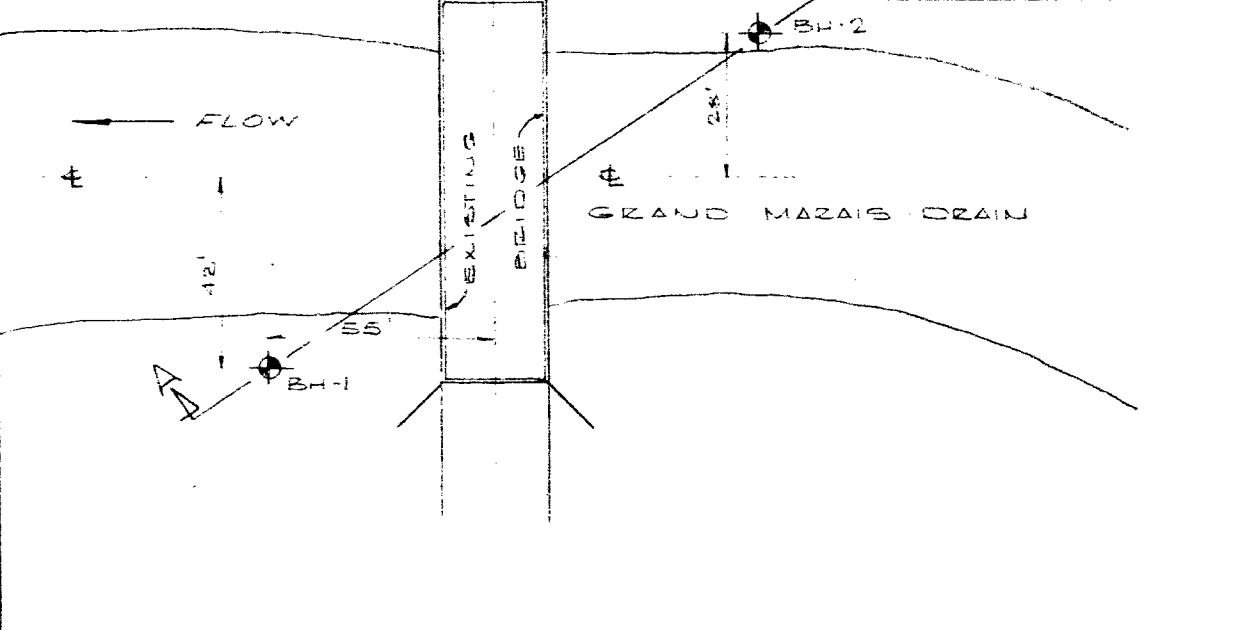
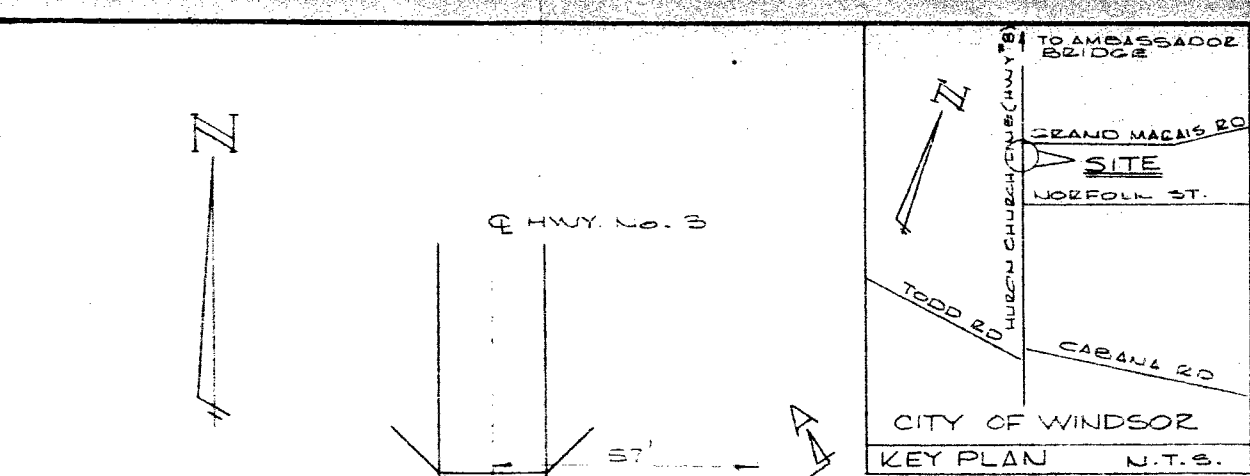
LOCATION See Figure 1 BORING DATE JAN. 30 & 31, 1969 DATUM GEODETIC  
BOREHOLE TYPE POWER AUGER BORING BOREHOLE DIAMETER 4.5"  
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT - LB. DROP - INCHES



VERTICAL SCALE  
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *JK*  
CHECKED *ED*

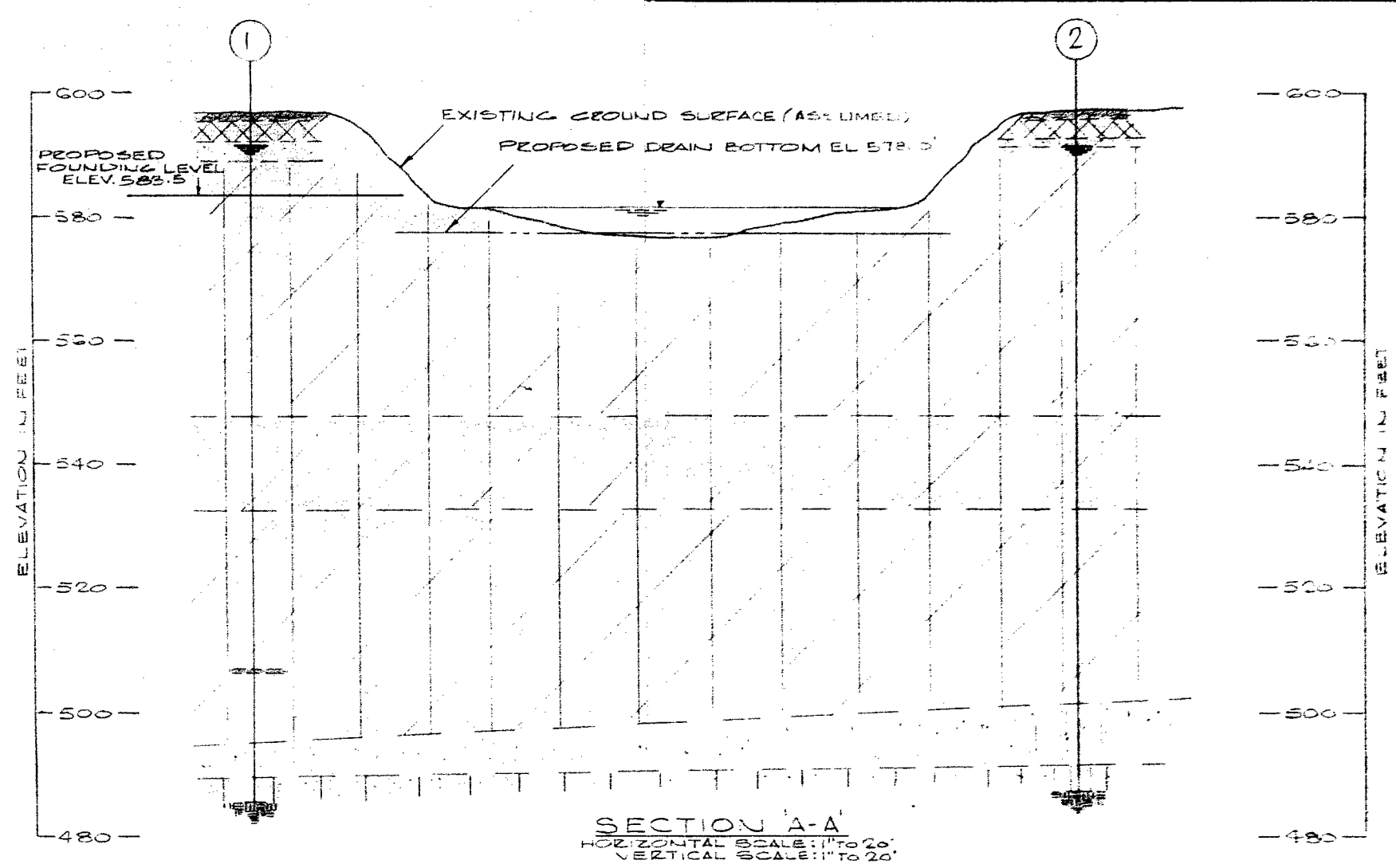


PLAN  
NOT TO SCALE

- LEGEND**
- - BOREHOLE IN PLAN
  - ② - BOREHOLE IN ELEVATION
  - - WATER LEVEL IN BOREHOLE  
JAN 30 & 31, 1969

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 SO MAY VARY FROM THAT SHOWN.

PLAN AND SOIL STRATIGRAPHY **FIGURE 1**



**STRATIGRAPHY**

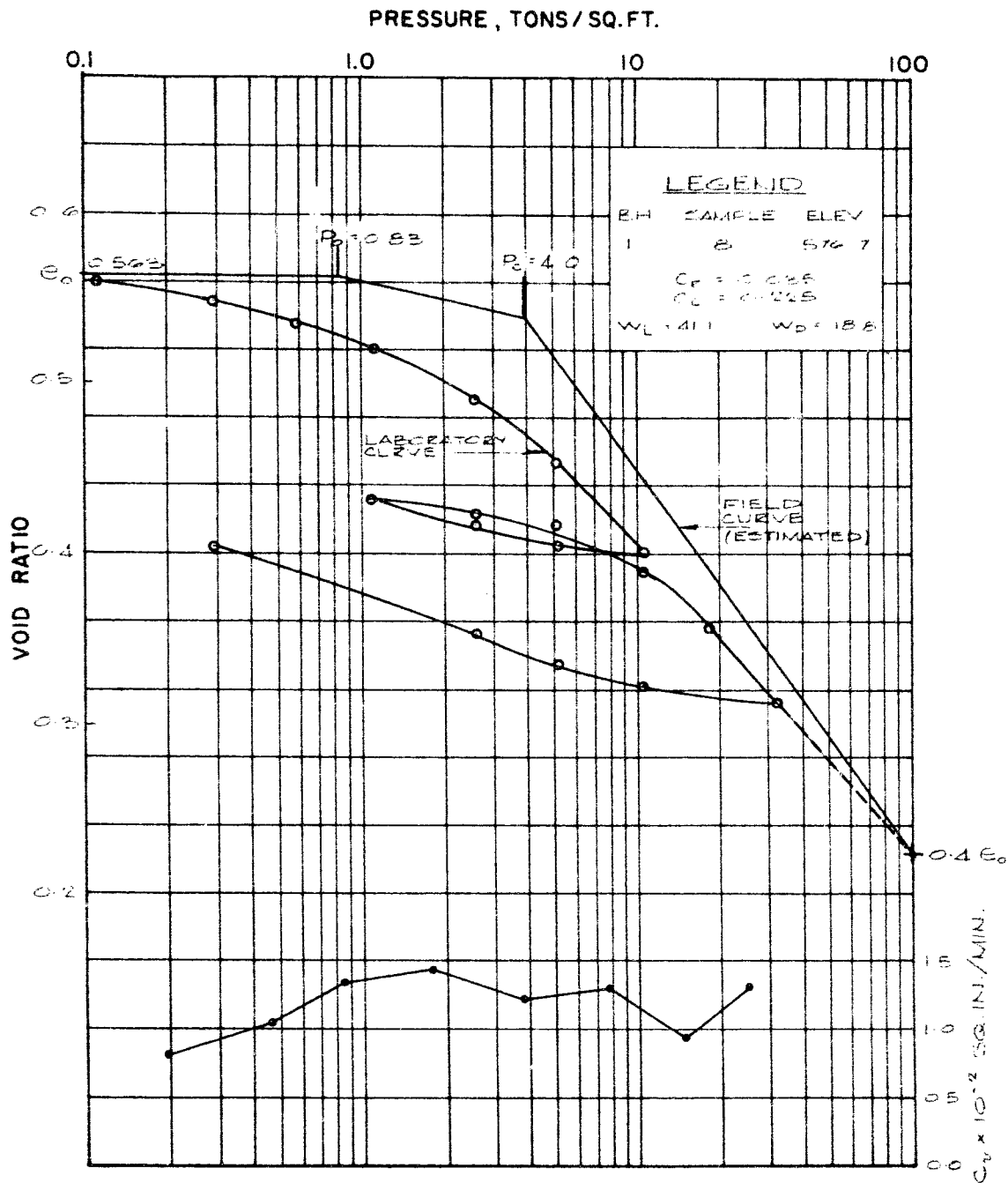
- TOP SOIL
- COMPACT TO LOOSE SANDY SILT (FILL)
- LOOSE TO DENSE BROWN TO GREY SAND TO SANDY SILT
- STIFF TO VERY STIFF GREY SILTY CLAY
- STIFF STRATIFIED GREY SILTY CLAY
- VERY DENSE GREY SANDY SILT TILL
- PROBABLE BEDROCK

**Golder Associates**  
D. J. J. FEB. 12, 1969

Made by *VZ*  
Chkd. by *VZ*  
App'd. by *LEB*

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

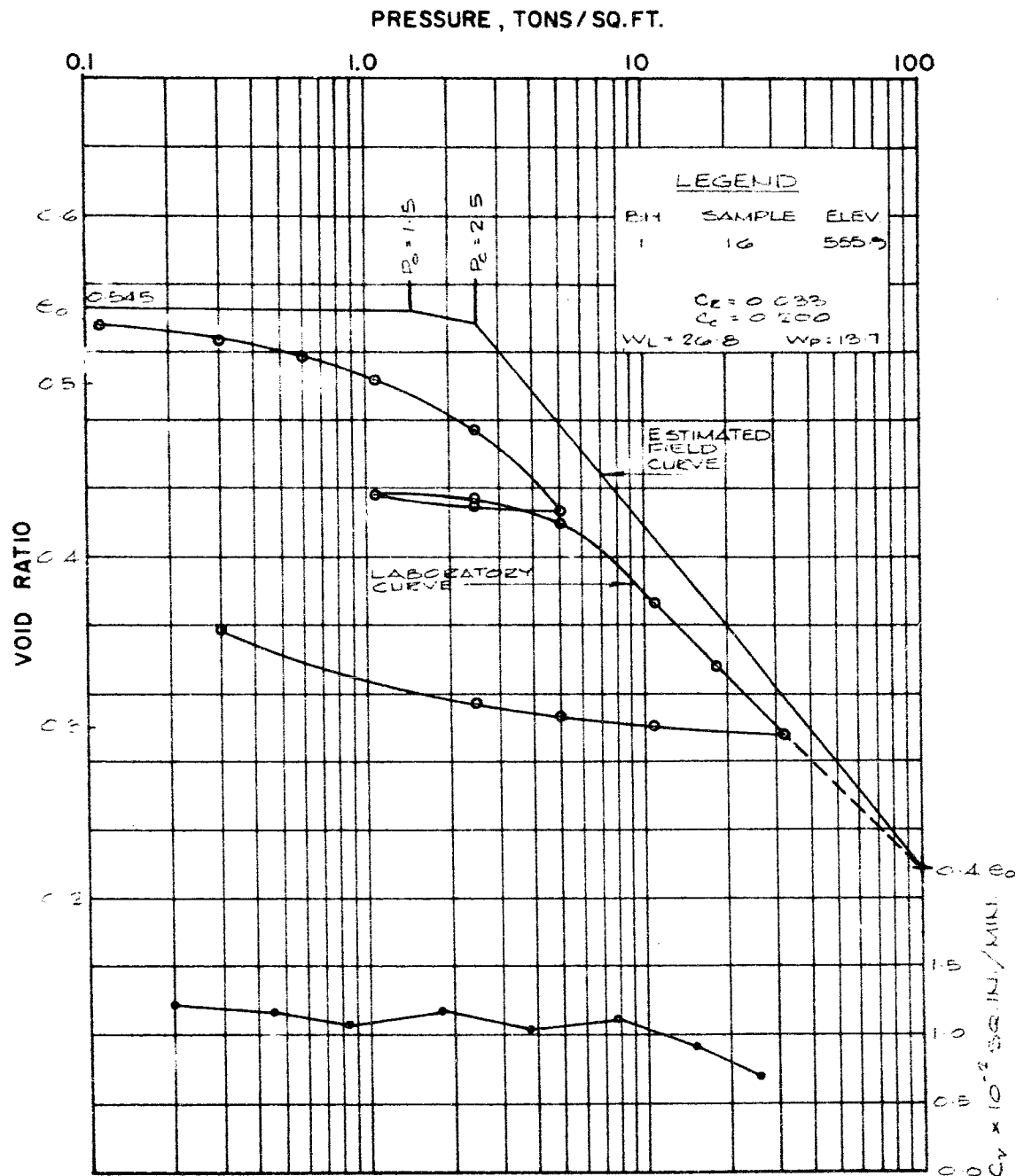
FIGURE 2



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# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

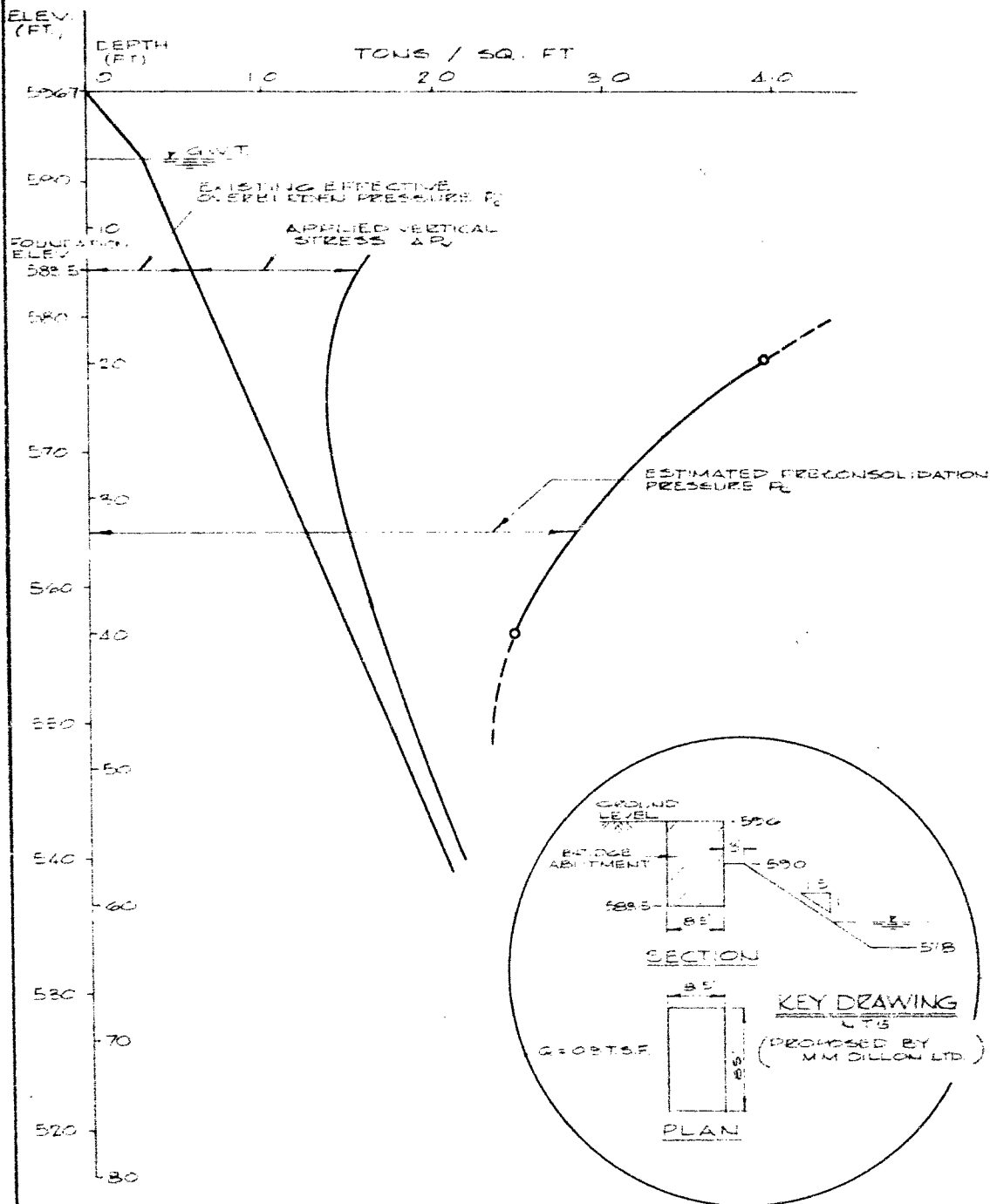
FIGURE 3.



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# EFFECTIVE DEPTH PROFILE

FIGURE 4



GOLDER & ASSOCIATES

Made  
Chkd.  
Appd.

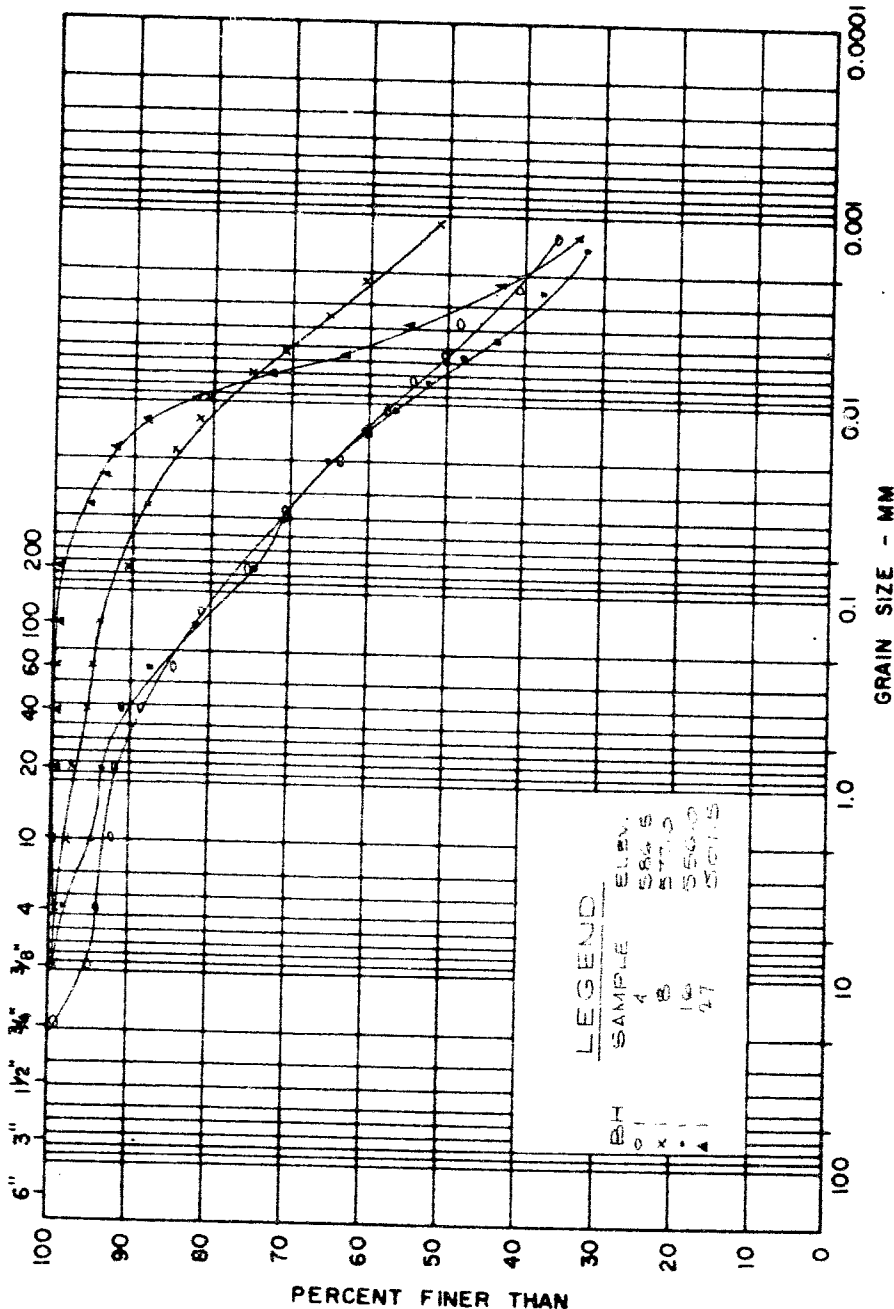
# GRAIN SIZE DISTRIBUTION

GREY SILTY CLAY

FIGURE 17

M.I.T. GRAIN SIZE SCALE

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



GOLDER & ASSOCIATES

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