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GEOCRES No. 31G-176

W.P. No. _____

CONT. No. _____

W. O. No. _____

STR. SITE No. _____

HWY. No. _____

LOCATION Co. RD. N° 24 over
RIGAUD RIV.

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

NONE

REMARKS: _____

B.A. 1888

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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REPORT

TO

316-176

GEOCRES No.

DE LEUW, CATHER & COMPANY OF CANADA LIMITED

ON

SITE INVESTIGATION

PROPOSED DALKEITH BRIDGE

COUNTY ROAD No. 24 OVER RIGAUD RIVER

UNITED COUNTIES OF STORMONT, DUNDAS & GLENGARRY

Distribution:

- 10 copies - De Leuw Cather & Company of Canada Limited
Ottawa, Ontario.
- 2 copies - H. Q. Golder & Associates Ltd.
Toronto, Ontario.

July, 1964

64046

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ABSTRACT

The results of a site investigation for the Dalkeith bridge over the Rigaud River on County Road No. 24, north of Dalkeith, Glengarry County, Ontario are reported.

It was found that the site was covered by up to about 2 feet of topsoil or loose brown sandy silt. Beneath the topsoil is from 3 to about 9 feet of soft to stiff grey brown weathered silty clay underlain by firm to stiff grey sensitive silty clay up to 25 feet in thickness. These strata are the weathered and unweathered zones in Leda clay. Beneath the Leda clay is compact to dense silty sand and gravel till varying in thickness from about 8 to about 23 feet. Limestone bedrock is present beneath the till and was proven in two boreholes.

An artesian water condition with a head about 6 feet above the river level was noted in one borehole when the casing was at the surface of the bedrock.

Because of the limited supporting capacity and probable settlement of the Leda clay it is recommended that this bridge be founded on end bearing piles driven to refusal in the till or to bedrock.

Embankment stability is not a problem at this site but a settlement of up to 6 inches is to be expected under the weight of the embankment where it is the highest. The embankment will need some slope protection because of partial inundation at periods of high water.

INTRODUCTION

H. Q. Golder and Associates Ltd. have been retained by De Leuw, Cather and Company of Canada Limited to carry out a site investigation for the proposed Dalkeith Bridge over the Rigaud River, on County Road No. 24, north of Dalkeith, Glengarry County, Ontario. The proposed bridge is to be located approximately 600 ft. upstream of the present bridge and the project will include the realignment of approximately 1,200 feet of the road, including the approach embankments for the bridge which will have a maximum height of about 12 feet.

PROCEDURE

The field work for this investigation was carried out during the period May 8, 1964 to May 12, 1964 using diamond drilling equipment modified for soil exploratory work. Three boreholes were put down at the proposed locations of the bridge foundations, with bedrock being proven in two of these. A further borehole was put down under the proposed embankment about 150 feet south of the bridge. Dynamic penetration tests were carried out at each borehole and two additional dynamic penetration tests were carried out at other locations in the area of the proposed embankment. A piezometer was installed in the deepest borehole and

standpipes in two other boreholes to determine groundwater conditions at the site.

----- Alignment A, as shown on Figure 1, was the line proposed at the start of the site investigation; however, during the course of the investigation alignment B was decided upon. The two alignments are near enough so that the same boreholes are suitable for either case.

A detailed log for each borehole is given on the Record of Boreholes following the text of this report. The locations of the borings are shown on Figure 1 and a longitudinal cross-section giving the inferred soil stratigraphy is shown on Figure 2.

Field vane strength tests were carried out in the boreholes where applicable. Following determination of the in situ strength, the soil was remoulded by rotating the vane and the remoulded shearing strength determined to indicate the sensitivity of the soil.

The samples obtained during the investigation were brought to our laboratory for examination and testing. Quick triaxial strength tests, moisture contents, Atterberg limits

and density determinations were carried out on selected samples. The results of these tests are given on the accompanying Record of Borehole sheets. One consolidation test was carried out in order to make an estimate of the probable settlements of the embankment. The results of this test are given on Figure 3.

SITE

At the proposed bridge site the river follows a curved course. On the north side of the river, the outside of the curve, the ground surface rises, at an average slope of about two horizontal to one vertical, about 25 feet above the river bed. Marks on the present bridge and debris on the banks indicate that there had recently been a rise in the river level to about elevation 91, some 9 feet above the river level at elevation 81.7 on May 11, 1964. On the south side of the river is a relatively flat flood plain area, most of which would be covered at high water. During the high water condition noted above about 250 ft. of the approach embankment at the south end of the proposed bridge would be standing in water.

SOIL AND GROUNDWATER CONDITIONS

The borings indicate that there is up to about 2 feet

of topsoil or loose brown sandy silt, containing organic matter, in some portions of the site but that the topsoil or sandy silt cover is not continuous over the entire site.

~~Immediately below the topsoil or at ground surface~~ where the topsoil does not exist, there is a soft to stiff grey brown weathered silty clay containing organic matter and thin sand seams. This grey-brown silty clay varies in thickness from about 3 to 9 feet.

At the proposed location of the bridge the weathered silty clay is underlain by firm to stiff grey sensitive silty clay up to 25 feet in thickness. This is the Leda clay which is common in the Ottawa Valley and its tributary valleys and the grey brown silty clay above it is the weathered or oxidized surface layer of the same clay. The grey silty clay was not encountered in borehole 3, 150 feet south of the proposed bridge location.

Beneath the grey silty clay is a stratum of glacial till composed of silty sand and gravel with a trace of clay. This till is in a compact to dense state and varies in thickness from about 8 ft. at borehole 1 to 23 ft. at borehole 2.

Bedrock, which is a grey limestone, was encountered beneath the glacial till and proven in borehole 1 at a depth of 28.3 ft., elevation 56.6 and in borehole 2 at a depth of 58.0 ft., elevation 41.6.

An artesian head of about 3 feet above the ground surface, 6 feet above river level, was encountered in borehole 1 when the casing in the hole was down near the surface of the bedrock. There was a considerable flow of water from the casing when the top of the casing was near ground level, the water apparently coming from a layer of ground just above the bedrock or from the bedrock itself. A small artesian flow was also noted when the drill rods at penetration test 4 were removed.

The water levels in the borings had not become static from the boring operation before the boring supervisor left the site, so that no meaningful readings of water levels in the boreholes can be reported. The piezometer and standpipes are however available for future readings.

DISCUSSION

The brown and grey silty clay strata which underly the site of the proposed bridge are in general in a soft to firm condition. These clays have a high natural moisture content. in

most cases near or above the liquid limit, are quite sensitive and would consolidate considerably if loaded by the bridge piers and abutments. It is therefore recommended, in order to eliminate undesirable settlements, that the bridge be founded on piles driven into the underlying till or to bedrock. If the piles are driven to practical refusal in the till or to the bedrock a load of 70 tons per pile may be applied considering a 12 BP 53 lb. section.

Before placing the approach embankments all topsoil, organic material and loose flood plain deposits should be removed from under the full base width of the embankment. The strength tests carried out in the field and laboratory indicate that the underlying silty clay has an undrained shear strength sufficient to support a 12 foot high embankment with the usual two to one slopes if the embankment is constructed of well compacted granular soil. In the area immediately adjoining the south abutment where there is about 20 ft. of silty clay and the embankment is the highest, a settlement of about 6 inches is anticipated under the weight of the embankment.

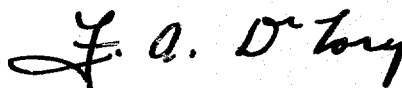
The granular fill immediately behind the abutments, for a minimum horizontal distance of 5 feet, should be free draining and non-frost susceptible to prevent the build-up of hydrostatic and ice pressures. Department of Highways granular "B" material

with less than 5 per cent passing the 200 sieve would be suitable for this purpose.

The placing of backfill behind the abutments will ~~cause a horizontal force on the abutments~~ and hence on the foundation piles. For compacted granular backfill, a coefficient of active earth pressure of 0.3 may be used to estimate the horizontal force. The earth pressure on the abutments will cause horizontal movement of the abutments and in addition there will be a tendency for the settlement of the embankment and movement necessary to mobilize the shearing strength of the underlying clay to cause horizontal movement. Therefore the bridge seats should be designed so that they can accommodate some lateral movement of the abutments without causing compression in the bridge superstructure.

It should be noted that the stability of the embankment at the bridge abutment depends on maintaining the shearing strength of the Leda clay at nearly its present value. Care should be taken to minimize remoulding of this clay as it is sensitive and the strength would be considerably lowered by remoulding. Small displacement piles, such as steel H sections should therefore be used to support the bridge structure.

Because of the probability of the embankments being eroded during periods of high water, it is recommended that the embankment side slopes, where they might be inundated, be protected by rip-rap extending 3 feet above high water level.



F. A. De Lory, P.Eng.



J. L. Seychuk, P.Eng.

FAD:HB
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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_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

(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 = $c_v t / d^2$ (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_i	sensitivity

$\left. \begin{array}{l} \text{in terms of effective stress} \\ \tau_f = c' + \sigma' \tan \phi' \end{array} \right\}$

$\left. \begin{array}{l} \text{in terms of total stress} \\ \tau_f = c_u + \sigma \tan \phi_u \end{array} \right\}$

*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 MAY 8, 1964

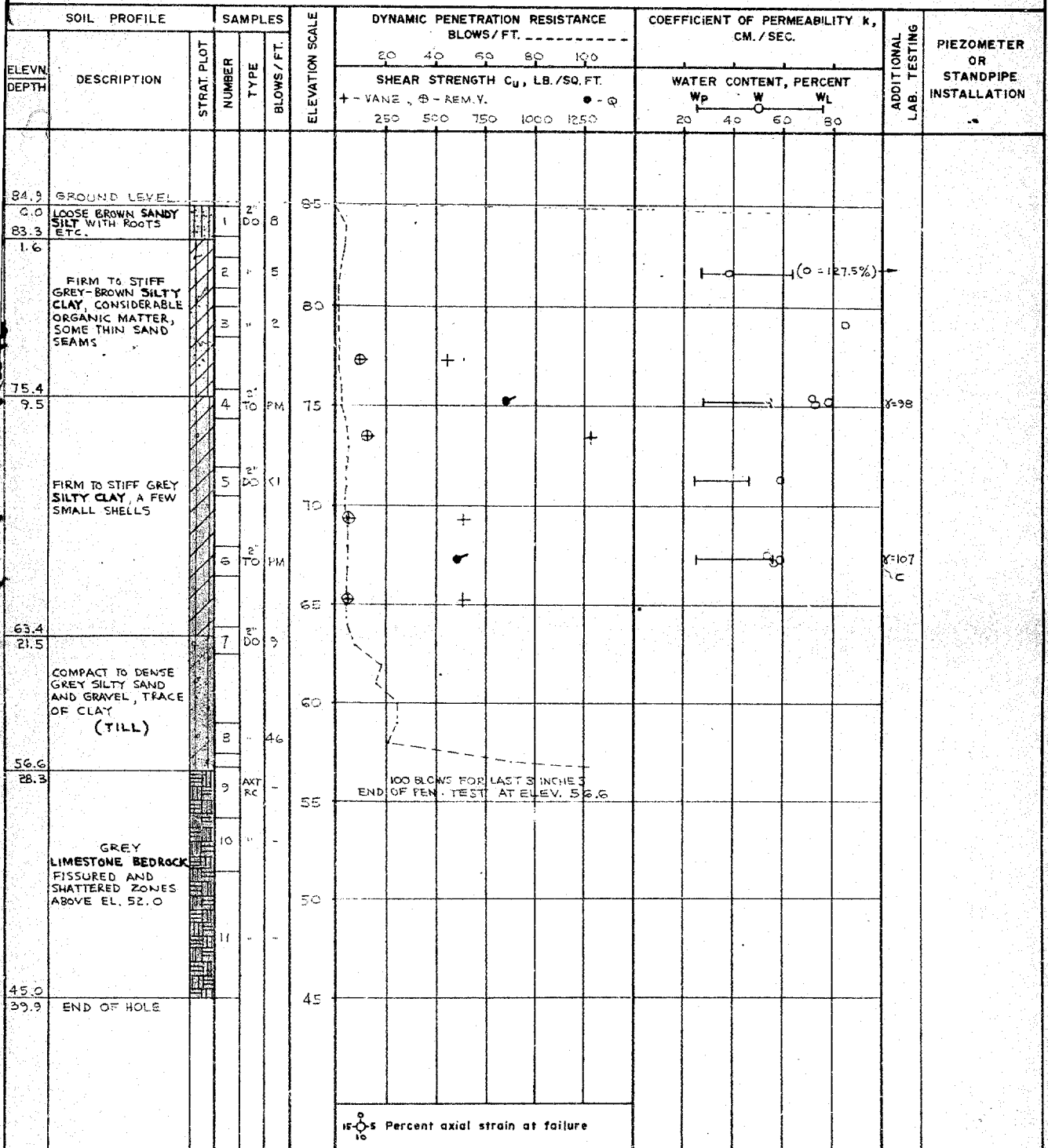
DATUM LOCAL

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX, BX & AX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN *AW*
CHECKED *F. DEL*

RECORD OF BOREHOLE 2

LOCATION See Figure 1

BORING DATE MAY 10-11, 1964

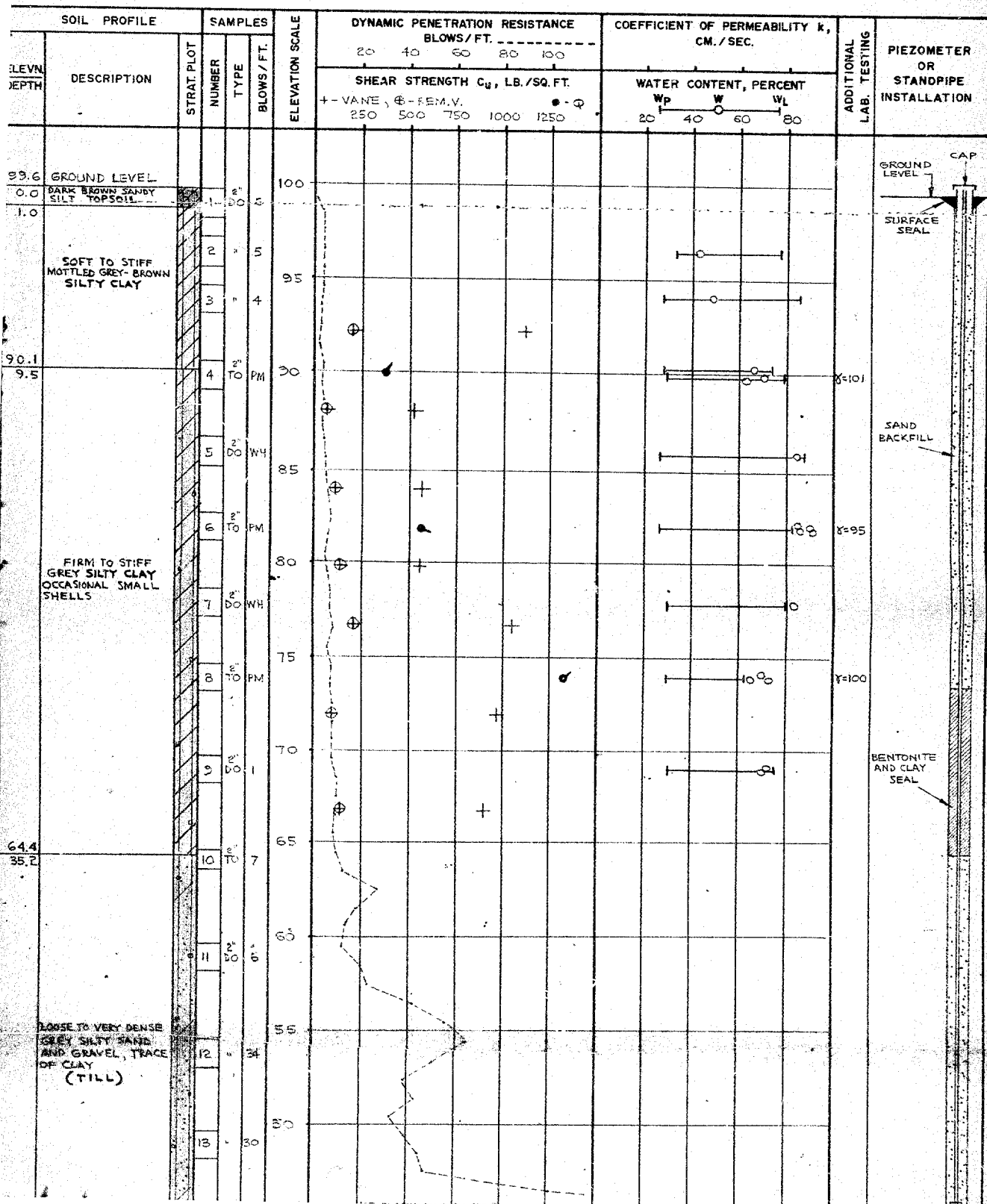
DATUM LOCAL

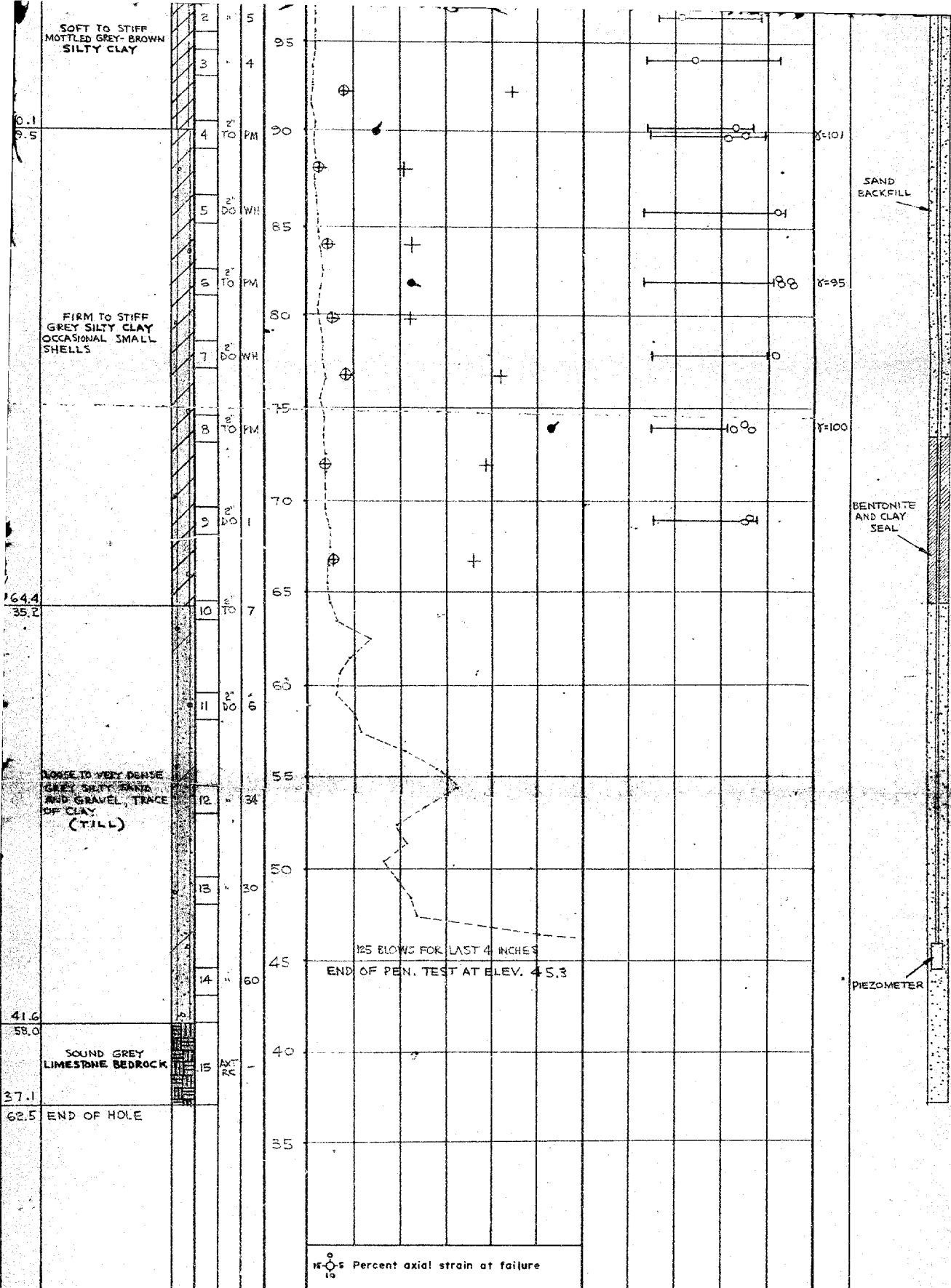
BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES





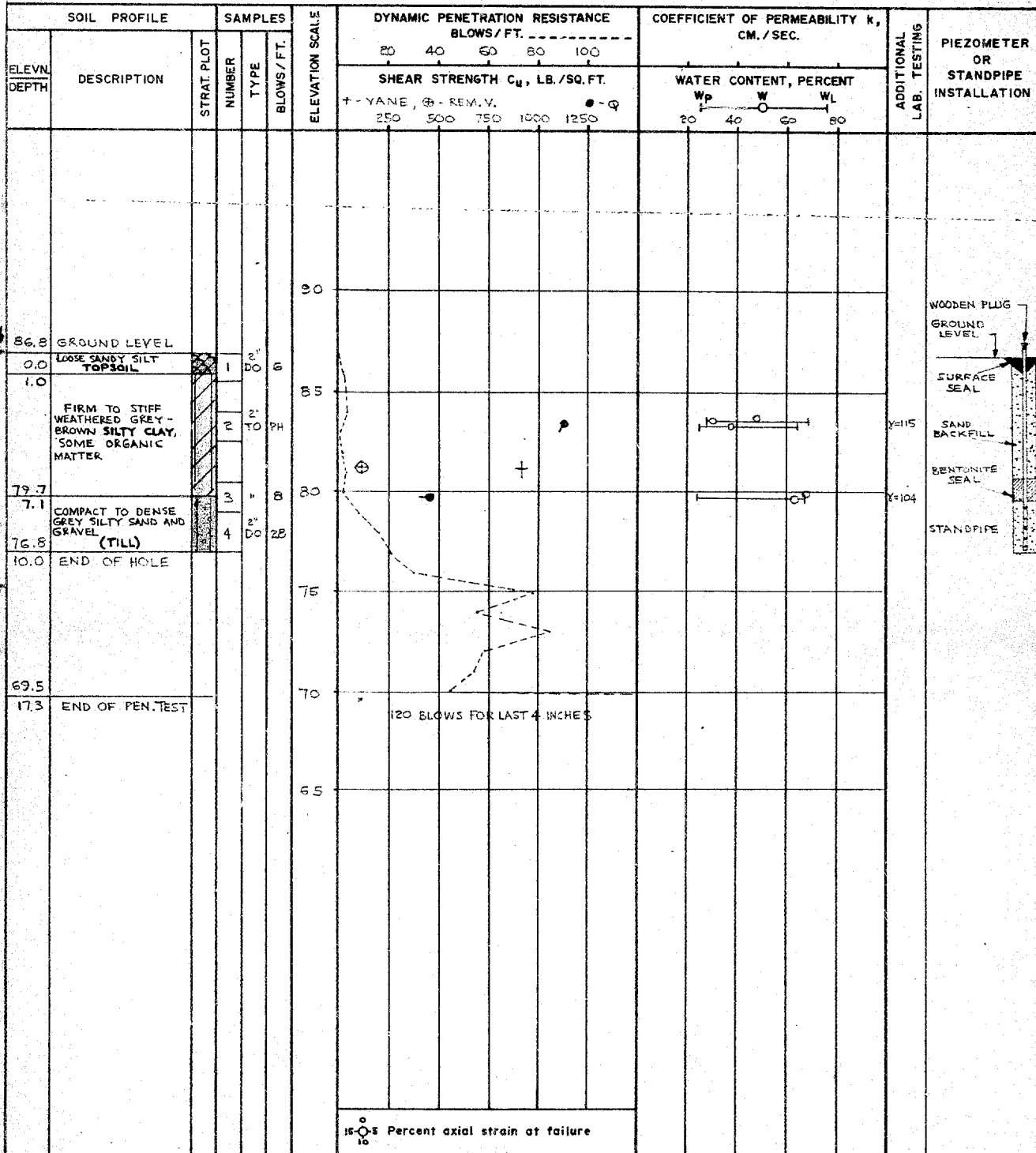
VERTICAL SCALE
1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN *A.W.*
CHECKED *F.D.L.*

RECORD OF BOREHOLE 3

LOCATION See Figure 1 BORING DATE MAY 3, 1964 DATUM LOCAL
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX CASING
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
 1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN *Mbs*
 CHECKED *F. DEL*

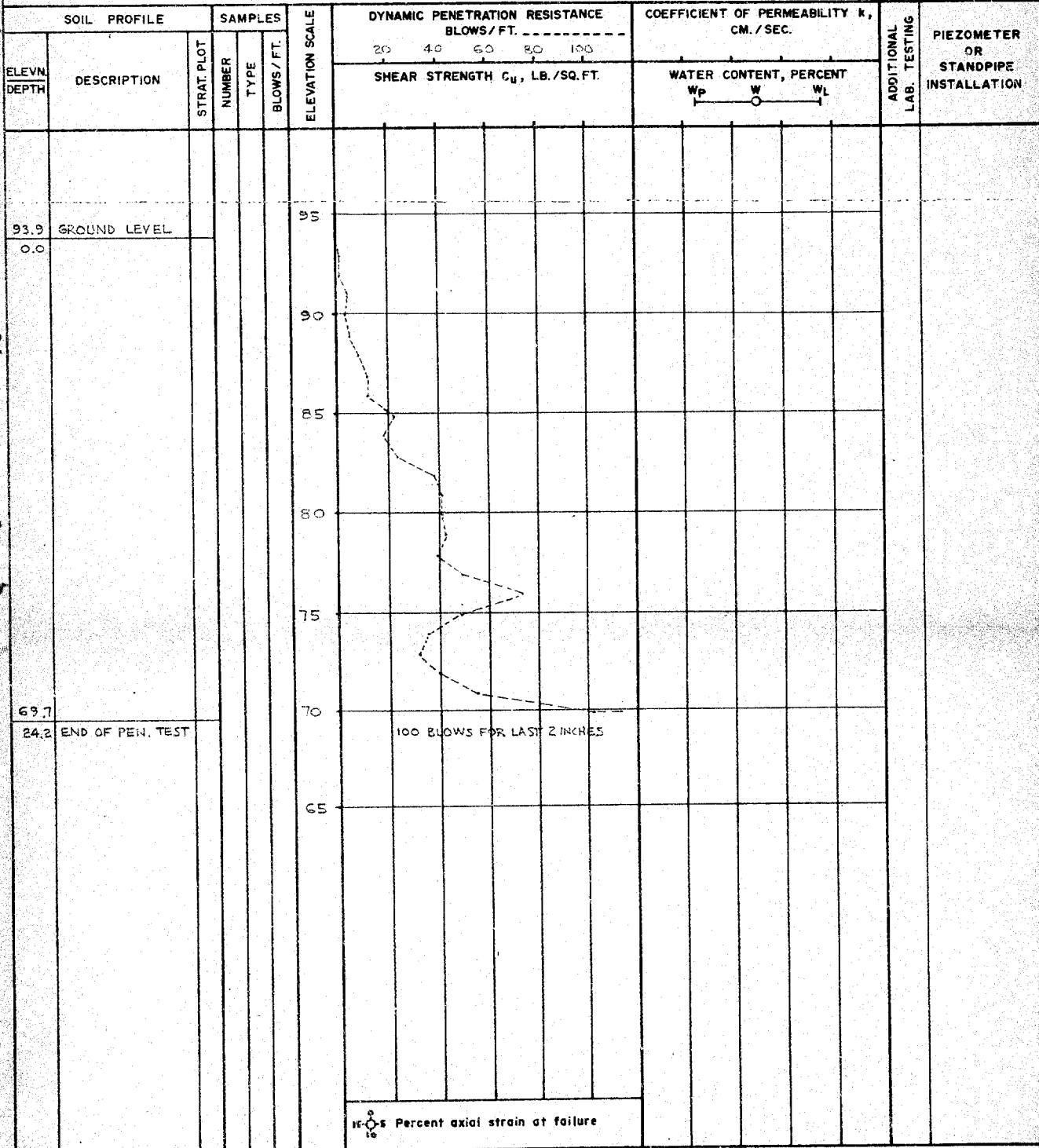
LOCATION	See Figure 4	BORING DATE	MAY 8, 1964	DATUM	LOCAL
BOREHOLE TYPE	DYNAMIC PENETRATION TEST		BOREHOLE DIAMETER	-	
SAMPLER HAMMER WEIGHT - LB.	DROP - INCHES	PEN. TEST HAMMER WEIGHT	140 LB.	DROP	30 INCHES

VERTICAL SCALE
1 INCH TO 5'-0"

DRAWN AW
CHECKED F DEL

RECORD OF BOREHOLE 5

LOCATION **See Figure 1** BORING DATE **MAY 9, 1964** DATUM **LOCAL**
 BOREHOLE TYPE **DYNAMIC PENETRATION TEST** BOREHOLE DIAMETER **-**
 SAMPLER HAMMER WEIGHT - **LB.** DROP - **INCHES** PEN. TEST HAMMER WEIGHT **140 LB.** DROP **30 INCHES**



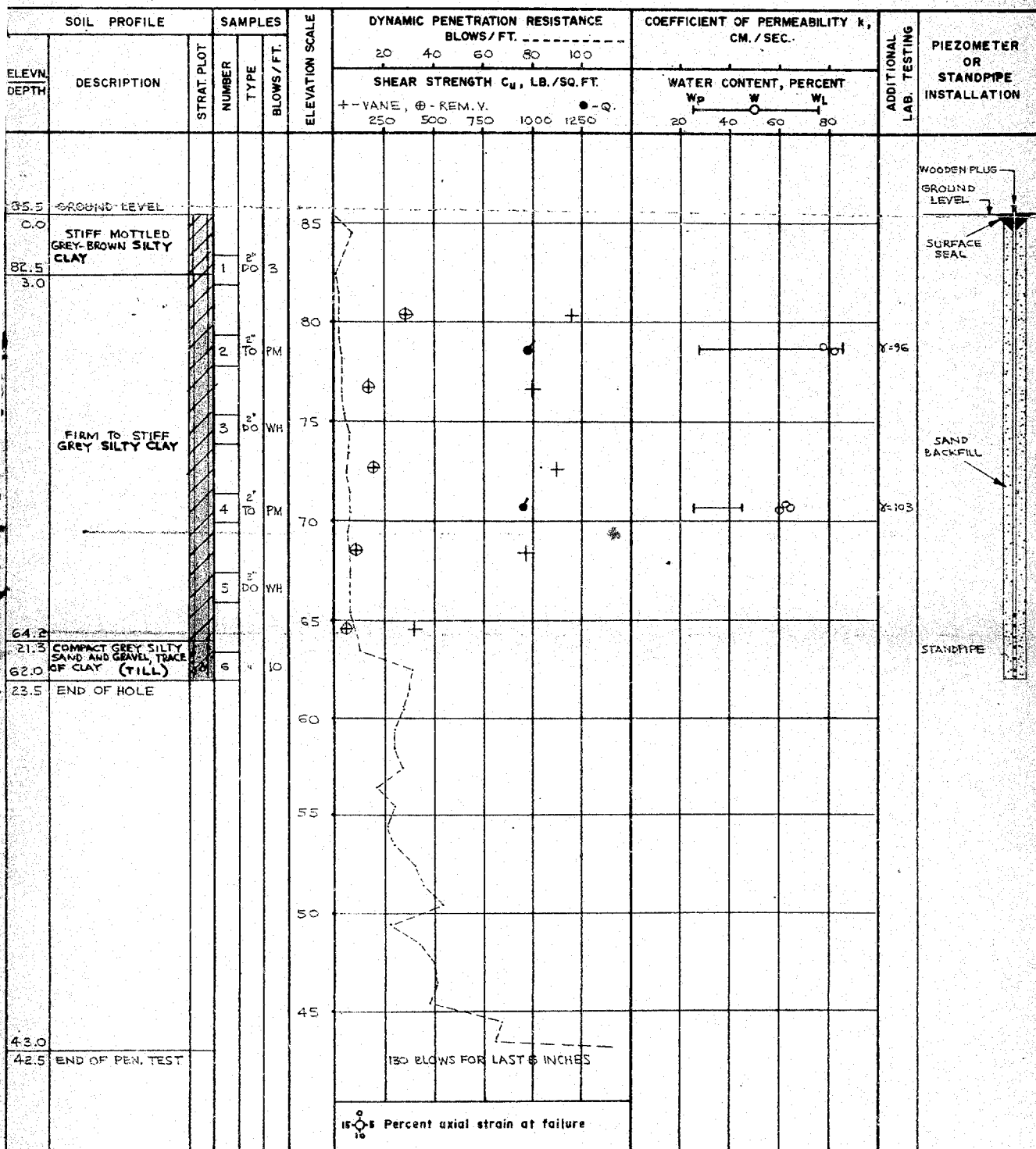
VERTICAL SCALE
 1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN *[Signature]*
 CHECKED *F. DEL*

RECORD OF BOREHOLE 6

LOCATION See Figure 1 BORING DATE MAY 11-12, 1964 DATUM LOCAL
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX CASING
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN *W.D.*
CHECKED *E.D.*

RECORD OF BOREHOLE 7

See Figure 1

BORING DATE MAY 12, 1964

DATUM LOCAL

BOREHOLE TYPE DYNAMIC PENETRATION TEST

BOREHOLE DIAMETER

SAMPLER HAMMER WEIGHT - LB. DROP - INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

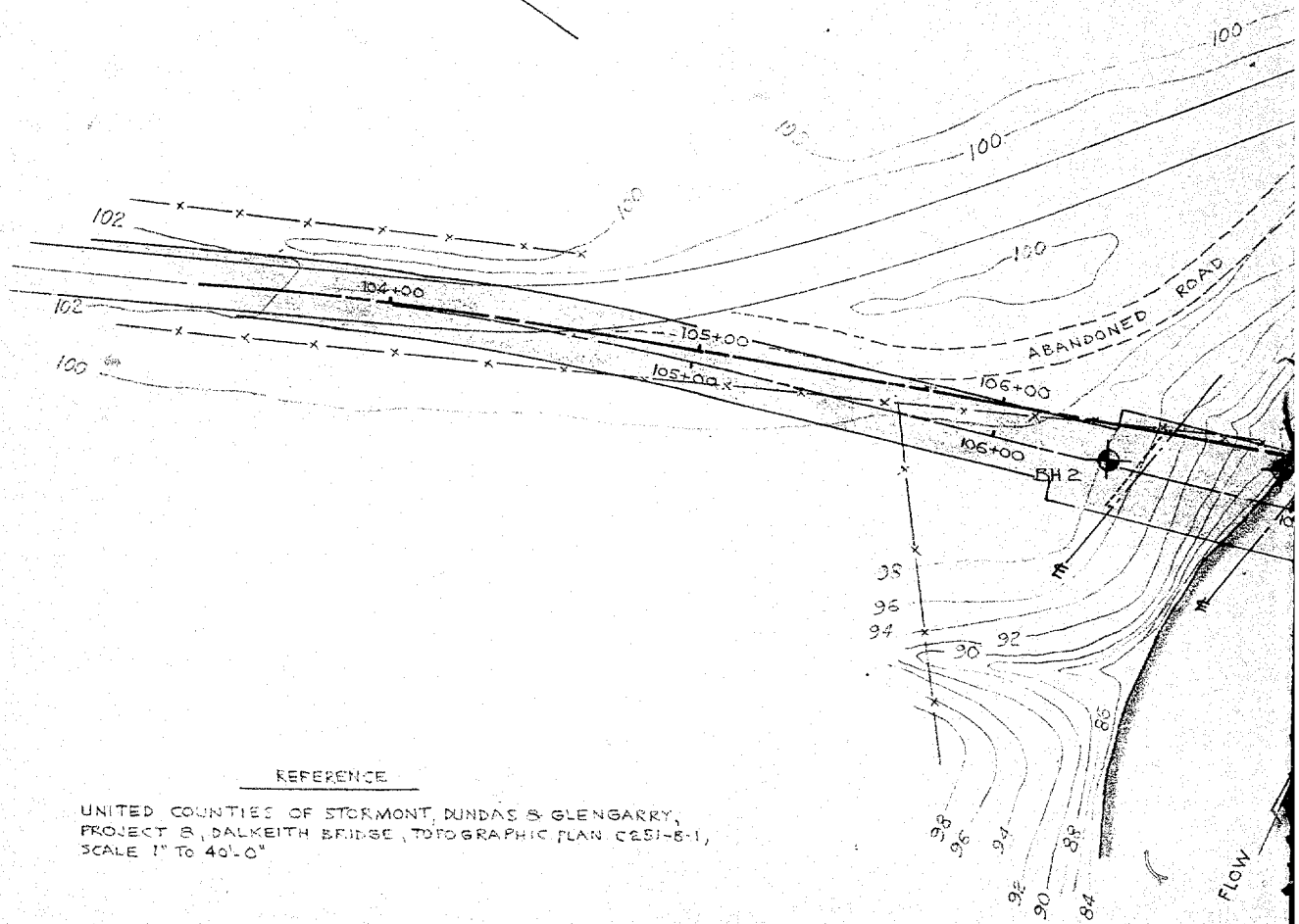
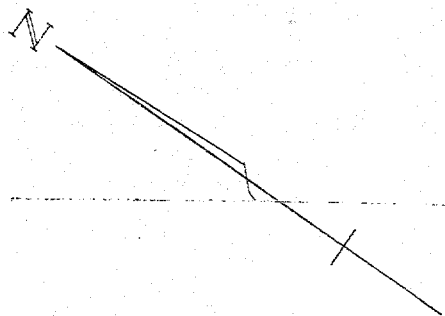
SOIL PROFILE		SAMPLES		ELEVATION SCALE		DYNAMIC PENETRATION RESISTANCE BLOWS/FT. ----- 20 40 60 80 100		COEFFICIENT OF PERMEABILITY k, CM./SEC.		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVN. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.	SHEAR STRENGTH c_u , LB./SQ. FT.	WATER CONTENT, PERCENT Wp W Wl				
64.0	GROUND LEVEL										
0.0											
53.6											
30.4	END OF PEN. TEST										

15-10 Percent axial strain at failure

VERTICAL SCALE
1 INCH TO 5'-0"

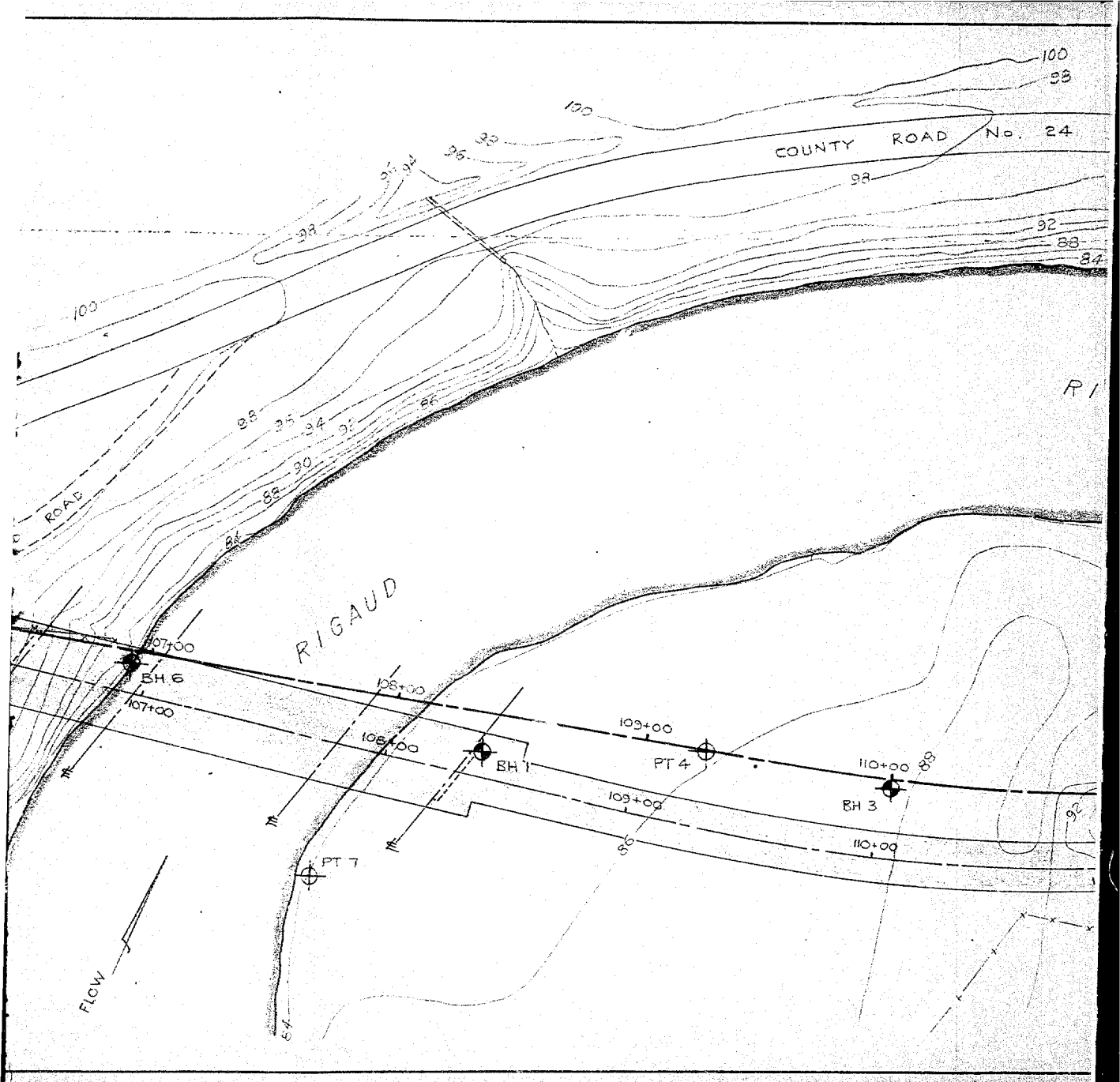
GOLDER & ASSOCIATES

DRAWN M.W.
CHECKED F. DEL.



REFERENCE

UNITED COUNTIES OF STORMONT, DUNDAS & GLENGARRY,
PROJECT 8, DALKEITH BRIDGE, TOPOGRAPHIC PLAN C251-8-1,
SCALE 1" TO 40'-0"



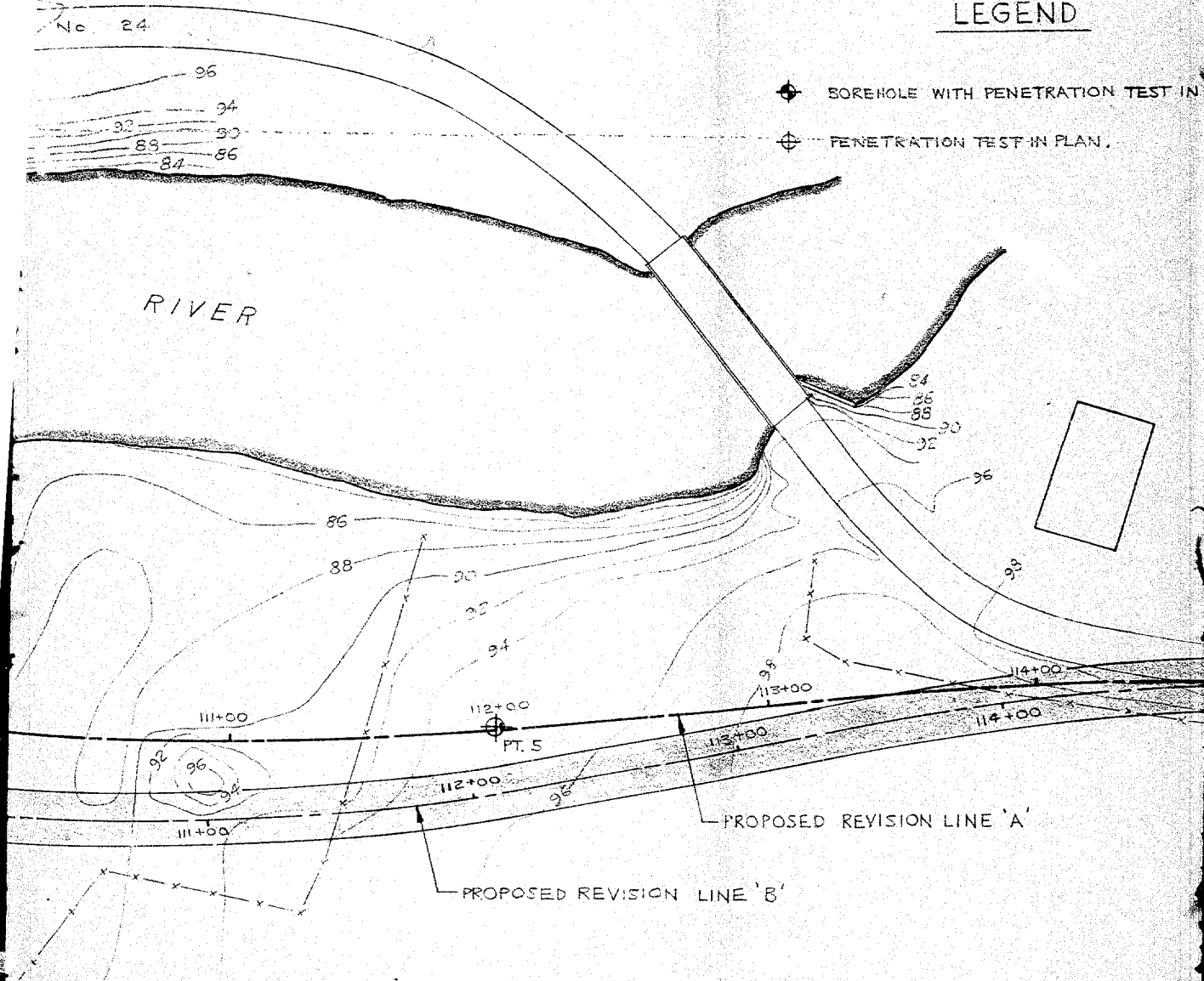
100
98
No 24

LEGEND

⊕ BOREHOLE WITH PENETRATION TEST IN

⊕ PENETRATION TEST IN PLAN.

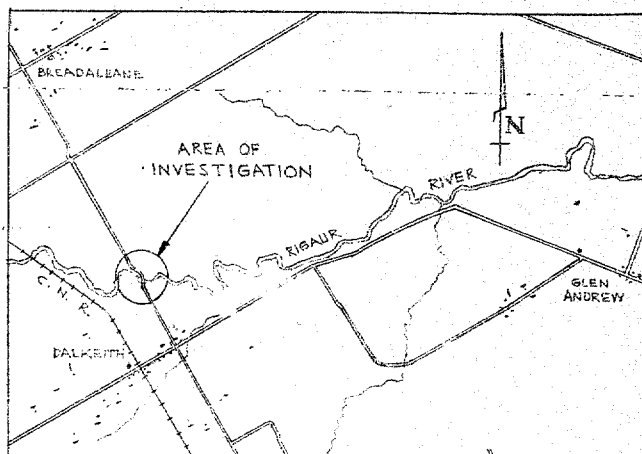
RIVER



GEND

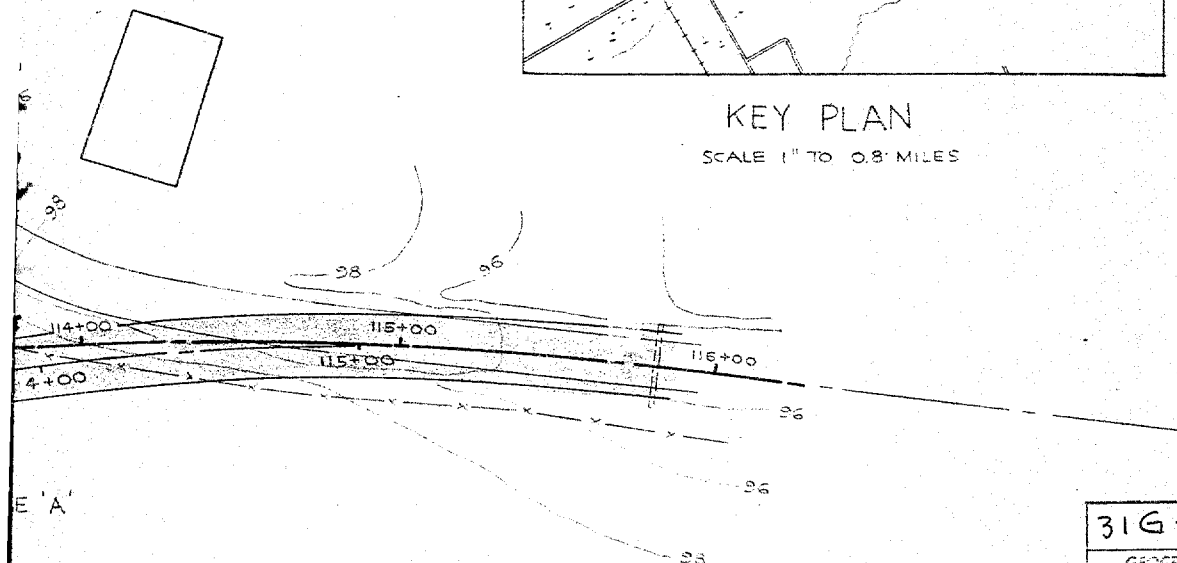
PENETRATION TEST IN PLAN

EST IN PLAN.



KEY PLAN

SCALE 1" TO 0.8 MILES



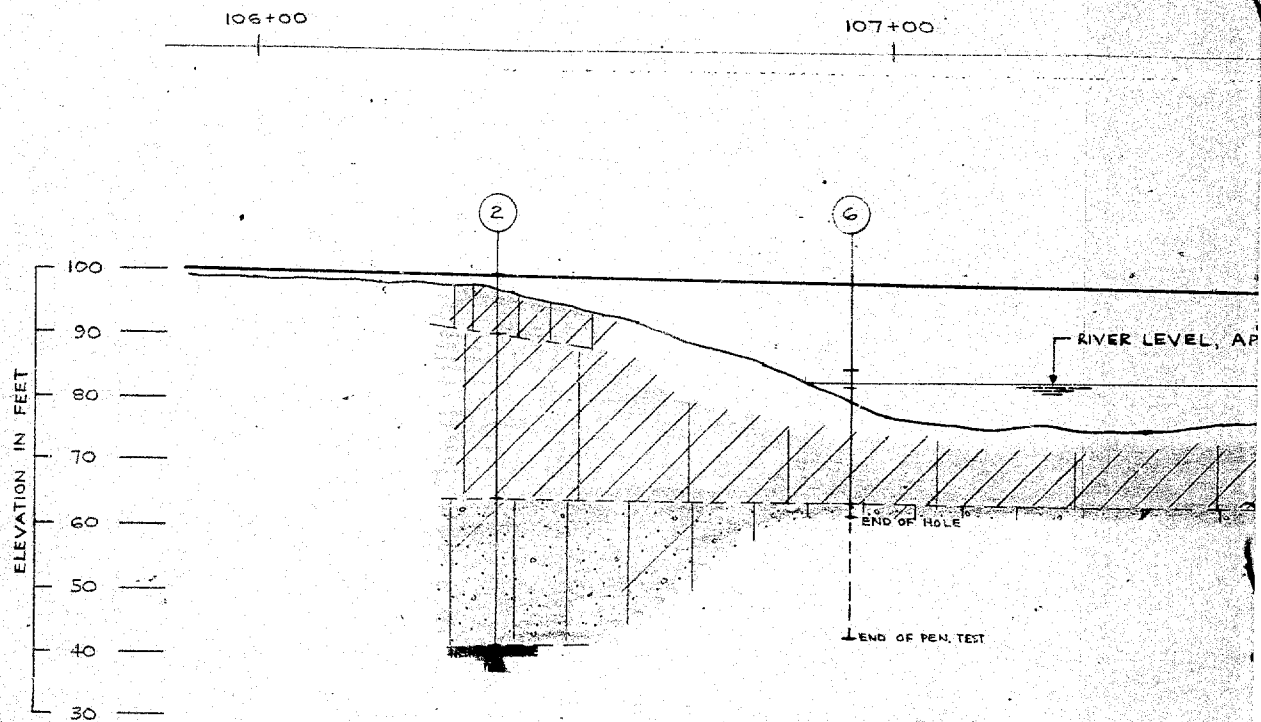
31G-176

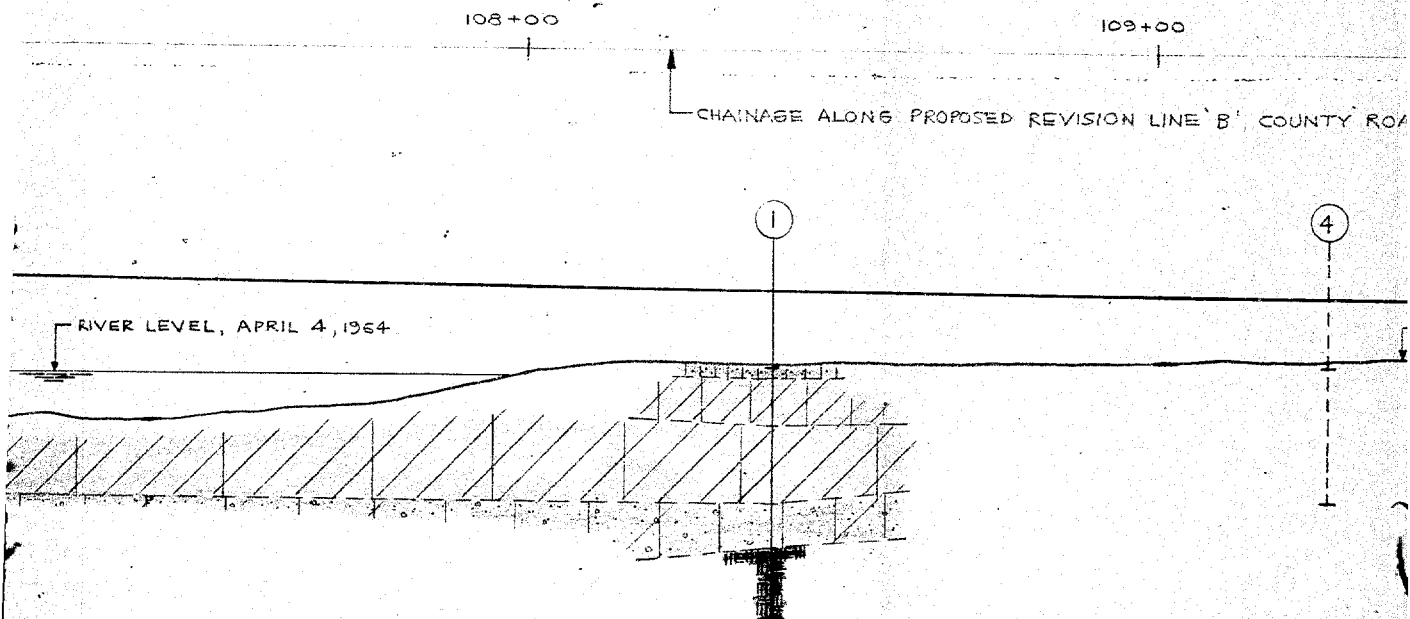
GEOGRES No.

SCALE 1" TO 40'-0"

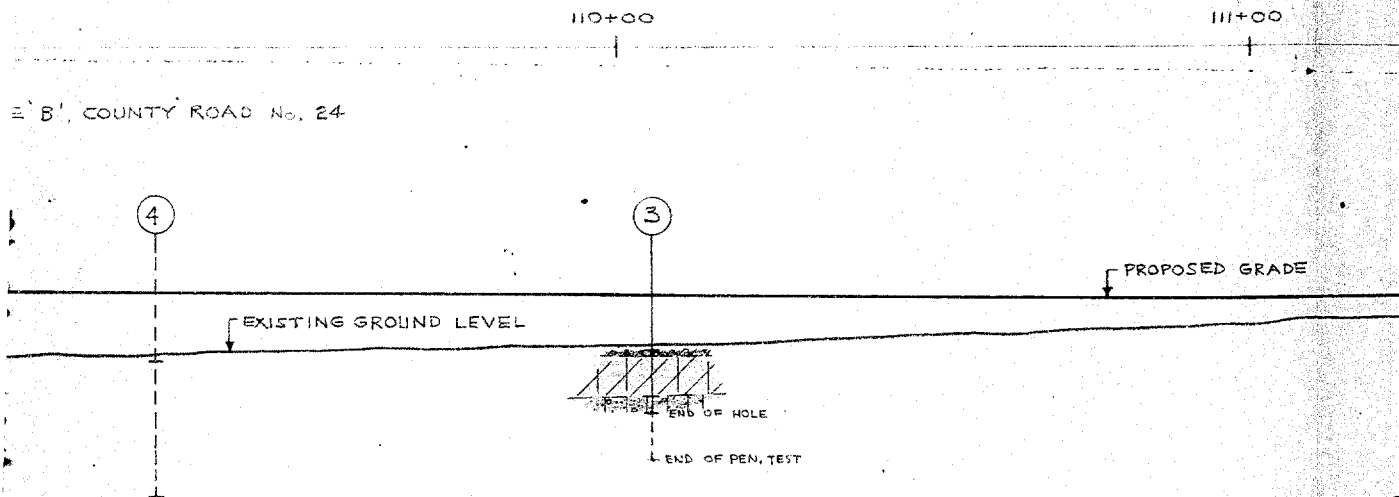
GOLDER & ASSOCIATES

Made *h.w.*
 Chkd. *E. D. G.*
 Appd. *000*





SCHEMATIC SECTION ALONG CENTRELINE OF PROPOSED REVISION LINE 'B'
SCALE: 1" TO 20'-0"



VISION LINE 'B'

SPECIAL NOTE: DATA CON-
STRATA HAVE BEEN OBTAIN-
TIONS ONLY. THE SOIL BY
BOREHOLES HAS BEEN INFER-
EVIDENCE AND SO MAY VARY

SOIL STRATIGRAPHY SECTION

STRATIGRAPHY



LOOSE SANDY SILT **TOPSOIL**



LOOSE BROWN **SANDY SILT** WITH ROOTS



SOFT TO STIFF MOTTLED GREY-BROWN
SOME ORGANIC MATTER AND THIN SAND, SEA



FIRM TO STIFF GREY **SILTY CLAY**, A FEW



LOOSE TO VERY DENSE GREY SILTY SA
GRAVEL, TRACE OF CLAY (**TILL**)



GREY LIMESTONE **BEDROCK**

LEGEND



BOREHOLE IN ELEVATION



PENETRATION TEST IN ELEVATION

111+00

112+00

5

ELEVATION IN FEET

100
90
80
70
60
50
40
30

POSED GRADE

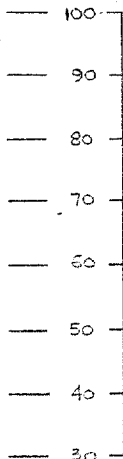
SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

STRATIGRAPHY

112+00

LOOSE SANDY SILT **TOPSOIL**LOOSE BROWN **SANDY SILT** WITH ROOTS ETC.SOFT TO STIFF MOTTLED GREY-BROWN **SILTY CLAY**
SOME ORGANIC MATTER AND THIN SAND SEAMSFIRM TO STIFF GREY **SILTY CLAY**, A FEW SMALL SHELLSLOOSE TO VERY DENSE GREY SILTY SAND AND
GRAVEL, TRACE OF CLAY (**TILL**)GREY LIMESTONE **BEDROCK**

ELEVATION IN FEET

LEGEND

1

BOREHOLE IN ELEVATION

4

PENETRATION TEST IN ELEVATION

31G-176

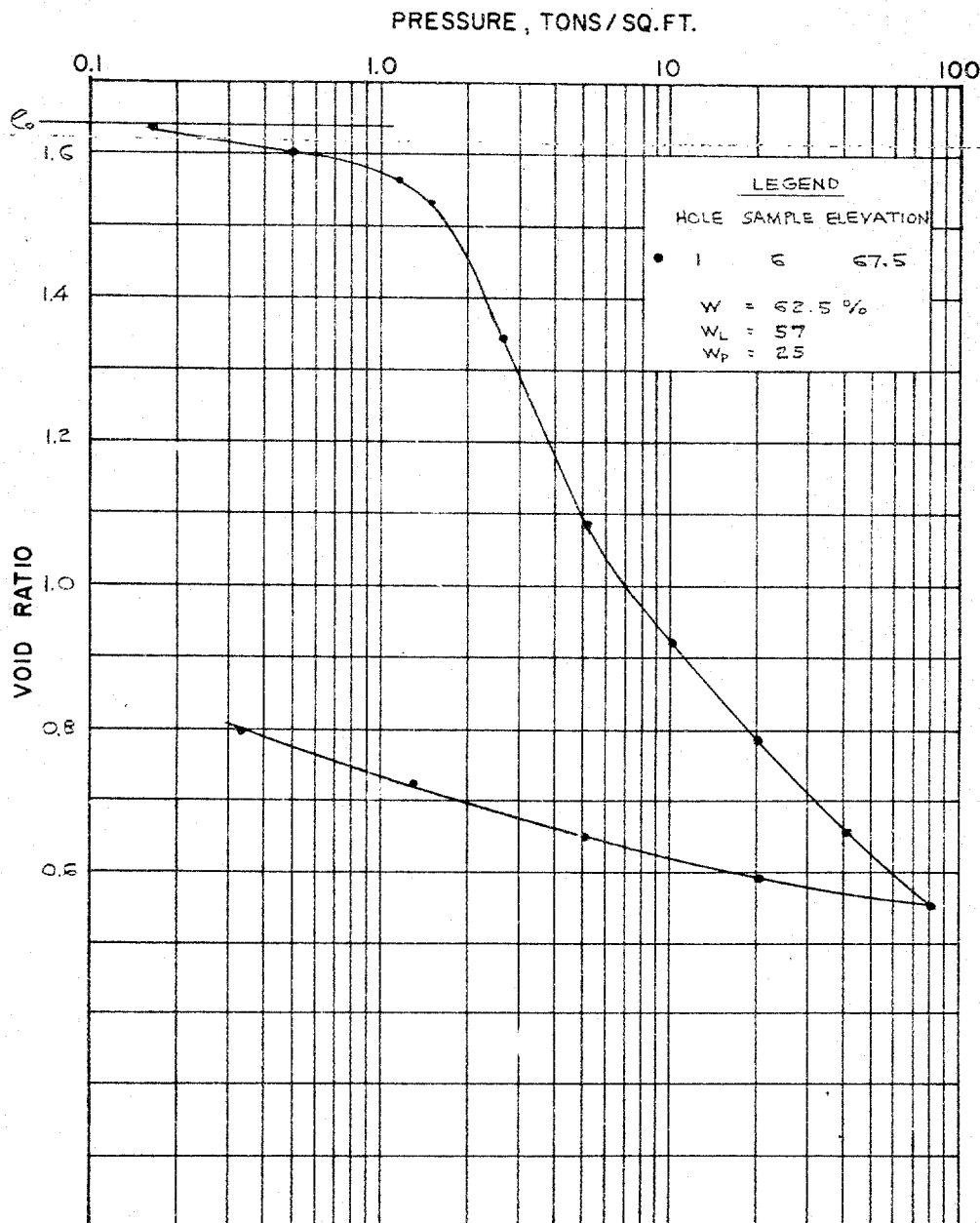
GEOCREP No.

GOLDER & ASSOCIATES

Modr *AM*
 Chkd. *EP*
 Appd. *GP*

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 3



GOLDER & ASSOCIATES