

#62-F-228C

WP # 279-62

Hwy # 401 &

CREEK CROSSING

NEAR LA RUE

MILLS

Mr. A. M. Toye,  
Bridge Engineer,  
Bridge Division.

Attention: Mr. S. McCombie.

Mr. A. G. Stermac,  
Principal Foundation Engr.,  
Foundation Section,  
Materials & Research Division.

January 2, 1963.

FOUNDATION INVESTIGATION REPORT BY -  
H. Q. Golder & Associates, Limited,  
Proposed Creek Crossing near La Rue Mills,  
Hwy. 401 - Line G, Cananoque, Ontario.  
W.P. 279-62, Dist. 8.

Attached, we are sending you the above-mentioned report submitted by the consultant, H. Q. Golder and Associates.

We have reviewed the report and find the factual information and data well presented and the discussion and recommendations conclusive. Concerning the approach fill recommendation (page 9 of report), we would like to add that apart from the backfill, the approach embankment can be constructed of any acceptable material, following the normal compaction requirements.

It is believed that the information contained in the report will be adequate for your further work. However, should additional data be required, please feel free to call on our Office.

AGS/MeeF  
Attach.

cc: Messrs. A. M. Toye (2) ✓

H. A. Tregaskes  
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CONSULTING CIVIL ENGINEERS

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REPORT  
TO  
DEPARTMENT OF HIGHWAYS, ONTARIO  
ON  
SOIL CONDITIONS AND FOUNDATIONS  
PROPOSED CREEK CROSSING NEAR LA RUE MILLS  
HIGHWAY 401 - LINE G  
WP 279-62  
GANANOQUE ONTARIO

Distribution:

15 copies - Department of Highways, Ontario,  
Toronto, Ontario.

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

December, 1962

6266

## ABSTRACT

The results of an investigation to determine the soil conditions at the site of a proposed creek crossing on the proposed revision of Highway 401, Line G, near Gananoque, Ontario are reported and recommendations are made for the foundation design of the proposed structures and approach embankments.

The site is underlain by a stratum of generally stiff layered clay and silt which rests on bedrock at depths ranging from zero to 40 feet. A thin deposit of silty sand was encountered immediately above bedrock surface in a few borings.

The piers and abutments for the proposed structures, with the exception of the north abutment on the western structure, may be founded on bedrock. The north abutment for the proposed western structure may either be founded on a spread footing in the layered silt and clay or carried to bedrock, as discussed in the report.

Settlement of the structures and approach embankments, if founded as recommended, should be very minor.

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## INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by the Department of Highways, Ontario by letter dated November 15th, 1962 to carry out a soil investigation for a proposed creek crossing on proposed Highway 401, Line G, near Gananoque, Ontario.

The purpose of the investigation was to determine the soil conditions at the site and to make recommendations concerning the foundation design of the proposed structure and its approach embankments.

## PROCEDURE

The field work was carried out between November 29th, and December 11th, 1962. During this period 7 borings, 6 with accompanying dynamic penetration tests and 6 additional dynamic penetration tests were put down by a machine drillrig.

The location of all borings and dynamic penetration tests put down during the investigation are shown on Figure 1 and sections of the inferred soil stratigraphy are given on Figure 2. A detailed log of each boring and dynamic penetration test is given on the Records of Boreholes.

The soil samples obtained during the investigation were returned to our laboratory for examination and testing. The results of the laboratory testing are plotted on the Records of Boreholes and on the figures.

The borings were located with respect to the centre-line of proposed Highway 401, Line G, as staked by others in the field. The borehole elevations were referred to a bench mark located in the north root of a 2.5 feet oak tree 221 feet right of station 280+49 on proposed Highway 401, Line G. The elevation of this bench mark is given as 279.87, Geodetic, on Department of Highways, Ontario Plan E-4136-1, dated August, 1962.

#### SITE TOPOGRAPHY AND GEOLOGY

The proposed creek crossing is situated approximately 0.4 miles west of La Rue Mills, Ontario. La Rue Mills is about 16 miles northeast of Gananoque, Ontario. This area lies within the physiographic region known as the "Leeds Knobs and Flats" (Chapman and Putnam, 1951) and consists primarily of scattered knobs of rock between which lie clay deposits laid down by the Champlain sea. The clay plains are typically gently undulating farmed land. The creek to be crossed meanders through one of these plains.

Bedrock in this area consists of various types of altered sedimentary rocks, crystalline limestones and dolomites, gneisses and quartzites of the Grenville series of Precambrian Age, which are intruded, metamorphosed and deformed by bodies of granite, syenite and other igneous rocks (Wilson, 1946). The surface elevation of bedrock can vary appreciably within small areas.

## SOIL CONDITIONS

Shallow surficial deposits, up to 4 feet in thickness, of sand and boulders or layered silt and organic matter were encountered in boreholes 5 and 8.

The principal stratum encountered by the borings put down at the site was a layered silty clay and clayey silt. This material was found to underlie the topsoil or surface organic matter in boreholes 1 to 4, 6 and 8 and reached a maximum thickness of about 41 feet in borehole 1. The upper 10 feet approximately of this stratum was generally a mottled grey and brown silty clay with a chunky or fissured structure. Below this upper portion, 'banding' of the samples was generally well defined and consisted of alternate layers of a fissured silty clay and faintly stratified clayey silt or silt. The silty clay layers were typically  $1/4$  to  $3/8$  inches in thickness but occasionally were up to 1 inch in thickness. The fissures in this material, which is sensitive to remoulding between the fingers, (sensitivity about 5 to 10) formed "blocks" of clay typically  $1/8$  to  $1/2$  inches in size. The clayey silt layers generally were 1 to 2 inches in thickness but occasionally up to 3 inches. Typical grain size distribution curves for the separate layers are shown on Figure 3 while the Atterberg limits for these materials are summarized on the plasticity chart on Figure 5. A few  $1/4$  inch to  $3/4$  inch thick silty sand or sand layers were noted in the lower portions of the deposit.

Undrained triaxial compression tests were carried out



on samples of the stratum and the results are summarized on Figure 7 in a plot of undrained shear strength versus elevation, the shear strength being assumed to be half the compressive strength. Stress-strain curves for four of the triaxial compression tests are shown on Figure 6.

It may be noted that the undrained shear strength of heavily fissured samples of the deposit was low compared to apparently non-fissured samples. The minimum shear strength of heavily fissured samples was about 500 pounds per square foot.

A thin deposit of compact to dense sandy silt with some clay was encountered below the layered silt and clay in boreholes 1 and 6. Two grain size distribution curves for this material are shown on Figure 4.

Bedrock was encountered below the layered silty clay and clayey silt or the sandy silt and was cored in AXT size in boreholes 2, 3, 4 and 5. The rock is a hard grey or pink crystalline rock. In boreholes 3 and 4 the rock had a structure similar to granite. Some fissures were noted in the upper 3 to 4 feet of the rock core recovered during the investigation.

#### WATER CONDITIONS

Water level observation pipes were installed in 5 of the borings at the site. Details of these installations are given on the Records of Boreholes. An artesian head of water up to 2 feet above present ground level was noted in boreholes 2, 3, 4 and 6.

The latest available water levels in the observation pipes are given on the Records of Boreholes and on Figure 2.

## DISCUSSION

### General

It is proposed to span a small creek on the proposed revision of Highway 401, Line G, near La Rue Mills, Ontario by two bridge structures. Each bridge will have 3 spans of 25, 34 and 25 feet and will be at skew angles of about 60 and 68 degrees to the centreline of proposed revision of Highway 401, Line G. The proposed grade profile will necessitate approach embankments to the bridge structures up to about 20 feet in height above the existing ground level.

### Foundation Design

In view of the relatively shallow depths to bedrock at the proposed locations of the south abutments and more <sup>northerly</sup> southerly of the two piers of each of the structures, these abutments and piers could be founded on spread footings resting on bedrock. For footings founded below any upper weathered or fissured zone in the rock an allowable bearing pressure up to 40 tons per square foot may be used for design. Based on an examination of the rock core recovered during the investigation, the weathered zone of bedrock, if any exists, is only a few inches in thickness; however, some fissures were noted in the rock cored to a depth of about 4 feet below rock surface.

Consider now what type of foundations would be suitable for the ~~north~~ abutments and ~~northerly~~ piers of the proposed structures. Spread footings founded in the layered silt and clay stratum is a possibility. Such footings would have to be founded at least 5 feet below creek bottom to avoid the possibility of scour around the footings. This would be about elevation 253. Based on the plot of the undrained shear strength of the layered silt and clay versus elevation, as shown on Figure 7, a net bearing pressure of 3,000 pounds per square foot may be used for design of such spread footings. To construct the footings would require a sheeted excavation down to elevation 253. In view of the possibility that the artesian water pressure noted in boreholes 2, 3, 4 and 6 close to bedrock surface could cause bottom heave in such excavations, computations were carried out to determine the probable factor of safety against heave for these excavations. Figure 9 shows a sketch of the situation for an assumed width of footing of 12 feet. Consider the vertical equilibrium of the block of soil ABCD in this figure. The artesian head of water acts in an upward direction on DC. The weight of the block of soil plus the shearing resistance along AD and BC act downwards. If the upward force exceeds the downward force the excavation will fail.

A typical stability computation is given on Figure 9 and the results of all computations are summarized on this figure as a plot of factor of safety against bottom heave versus the thickness of the clay below elevation 253. In the calculations the

shearing resistance along AD and BC of the block of soil has been assumed to be the remoulded value of the clay, estimated at about 200 pounds per square foot. From Figure 9, it may be seen that for clay thicknesses less than about 12 feet the factor of safety against bottom heave at the time when excavation is at elevation 253 is less than unity.

Figure 2 shows that the thickness of clay below footings at elevation 253 at the more northerly pier of the proposed western bridge structure and both the north pier and north abutment of the proposed eastern bridge structure is less than 12 feet. To increase the factor of safety we could found the footings at higher elevations in order to increase the thickness of the clay below the footings, reduce the excess water pressure at the rock surface, or carry the foundation down to rock. The former solution is undesirable as there would be a danger of river scour around the footing. The second solution would probably be ineffective as reducing the water pressure to normal hydrostatic water pressure will still give a low factor of safety against heave in the excavation. We therefore recommend that the north pier and north abutment of the eastern bridge structure and the north pier of the western structure be carried to bedrock. Water entering excavations should be controlled by pumping.

The remaining north abutment of the western structure may either be founded on a spread footing at elevation 253 in the layered silty clay and clayey silt or carried down to rock.

The depth of clay below this footing ranges from about 18 feet at the west end of the proposed footing to about 22 feet at the east end. The factor of safety against bottom heave of the excavation should then be at least 1.4 to 1.5 (Figure 9). The excavation for a spread footing should be carried out carefully due to the sensitivity to remoulding of the silty clay. A thin layer of lean concrete should be laid down immediately the excavation is down to foundation elevation.

In summary therefore, the piers and abutments for the proposed structures with the exception of the north abutment on the western structure may be founded on spread footings or caissons resting on bedrock. The north abutment for the proposed western structure may either be founded on a spread footing in the layered silt and clay at elevation 253 or carried to bedrock in a manner similar to the other piers and abutments.

Settlement of footings founded on bedrock will be negligible. Settlement of the north abutment of the western structure should be of the same order as the settlement of the approach embankment as discussed later in this report.

Where the rock surface slopes, or where river scour could be a danger to footings, they will have to be stepped and dowelled into the rock to prevent possible shear at the footing-rock interface.


Free draining granular backfill should be placed behind the proposed abutments. This backfill should extend at least 4 feet horizontally away from the abutment wall and have provision for drainage to ensure that no excess hydrostatic or ice pressures build up behind the walls. In the design of the abutments it is recommended that an earth pressure coefficient,  $K$ , of 0.3 be used, provided that some minor movement of the top of the abutment can be accommodated. Rip rap should be placed in front of the proposed abutments as a protection against river scour.

Concerning the stability of approach fills, which are not greater than 20 feet in height and that only for a limited distance, the average shearing strength of the clay deposit is certainly adequate being of the order of 1,000 to 2,000 pounds per square foot. While the shear strength of certain fissured samples is very low we can assume that, under the loading conditions imposed by the embankments, there will be little tendency for the fissures to open (as they would in sampling and extruding) and consequently the stress behaviour of the fissured material would not control the stability of the embankments.

The approach earthfill embankment may be constructed of well compacted granular borrow or rockfill. The side slopes of granular borrow should not exceed 2 horizontal to 1 vertical while the side slopes of rockfill should not exceed  $1\frac{1}{2}$  horizontal to 1 vertical.

Based on the results of a laboratory consolidation test on a typical layer of fissured silty clay (Figure 8) and assuming that the silty clay comprises approximately 50 percent of the total thickness of the strata, we estimate that the maximum settlement of the embankments due to consolidation of the clayey subsoil should not exceed 1 to 2 inches.

McC/jb  
6266



*N. R. McCammon*  
N. R. McCammon, P. Eng.

December, 1962

*V. P. Milligan*  
V. Milligan, P. Eng.

REFERENCES

CHAPMAN, L.J., AND PUTNAM, D.F., "The Physiography of Southern Ontario", University of Toronto Press, 1951.

WILSON, A.E., "Geology of the Ottawa - St. Lawrence Lowland, Ontario and Quebec", Geological Survey Memoir No.241, Canada Department of Mines and Resources, Ottawa, 1946.



## LIST OF STANDARD ABBREVIATIONS

The standard abbreviations commonly employed on each "Record of Borehole", on the figures, and in the text of the report are as follows:

### SAMPLE TYPES

A.S. - Auger Sample	R.C. - Rock Core
C.S. - Chunk Sample	S.T. - Slotted Tube
D.O. - Drive Open	T.O. - Thin-walled, Open
D.S. - Denison Type Sample	T.P. - Thin-walled, Piston
F.S. - Foil Sample	W.S. - Wash Sample

### PENETRATION RESISTANCES

Dynamic Penetration Resistance - The energy required to drive a 2 inch diameter, 60 degree cone attached to the end of the drilling rods into the ground: expressed in blows per foot, where each blow represents 4,200 inch-pounds of energy.

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 into the ground.

Sampler advanced by static weight - weight, hammer - Wh
Sampler advanced by pressure - pressure, hydraulic - Ph
Sampler advanced by pressure - pressure, manual - Pm

### SOIL DESCRIPTION

The standard terminology for the descriptions of the relative density of cohesionless soils and the consistency of cohesive soils is as follows:

<u>Relative Density</u>	<u>N, Blows/ft.</u>	<u>Consistency</u>	<u>c, lb/sq. ft.</u>
Very Loose	0 to 4	Very Soft	Less than 250
Loose	4 to 10	Soft	250 to 500
Compact	10 to 30	Firm	500 to 1,000
Dense	30 to 50	Stiff	1,000 to 2,000
Very Dense	over 50	Very Stiff	2,000 to 4,000
		Hard	over 4,000

### SOIL TESTS

C - Consolidation Test	Q - Undrained Triaxial
H - Hydrometer Analysis	Qc - Consolidated Undrained Triaxial
M - Sieve Analysis	S - Drained Triaxial
MH - Combined Analysis, Sieve and Hydrometer	U - Unconfined Compression
	V - Field Vane Test

Note: Undrained triaxial tests in which pore pressures are measured are shown as Q' or Q'c.

### SOIL PROPERTIES

$\gamma$ - Total Unit Weight	K - Coefficient of Permeability
$\gamma_d$ - Dry Unit Weight	c - Undrained Shear Strength (1/2 Compressive Strength)
$\gamma_b$ - Submerged Unit Weight	St - Sensitivity
L <sub>L</sub> - Liquid Limit	$\phi'$ - Effective Angle of Shearing Resistance
P <sub>L</sub> - Plastic Limit	c' - Effective Cohesion Intercept
W - Natural Water Content	Cc - Compression Index
G - Specific Gravity	Cv - Coefficient of Consolidation
e - Void Ratio	



## RECORD OF BOREHOLE 2

LOCATION SEE FIGURE 1

BORING DATE

NOV. 30 - DEC. 3, 1962

DATUM

GEODETIC

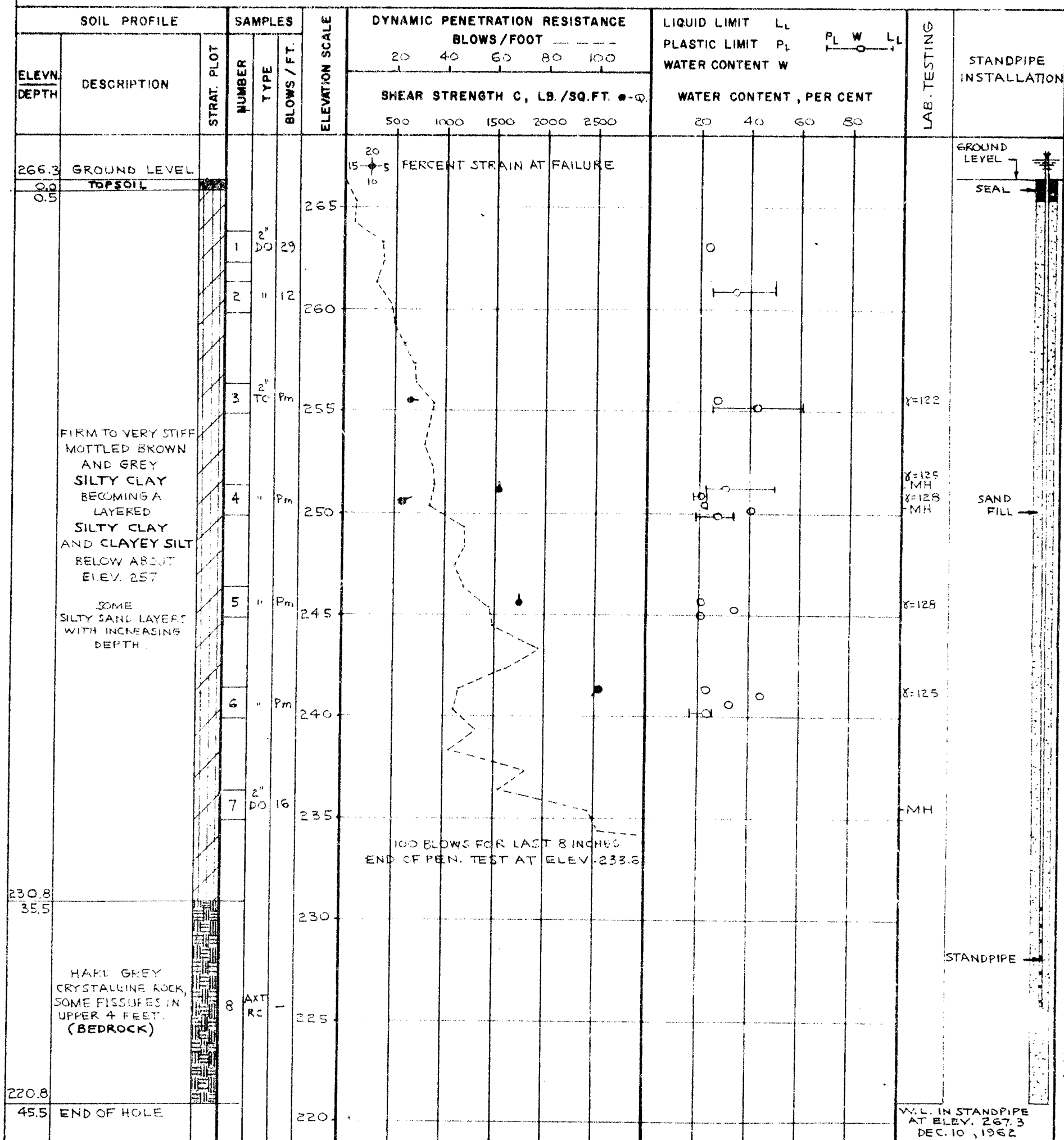
BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER

BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

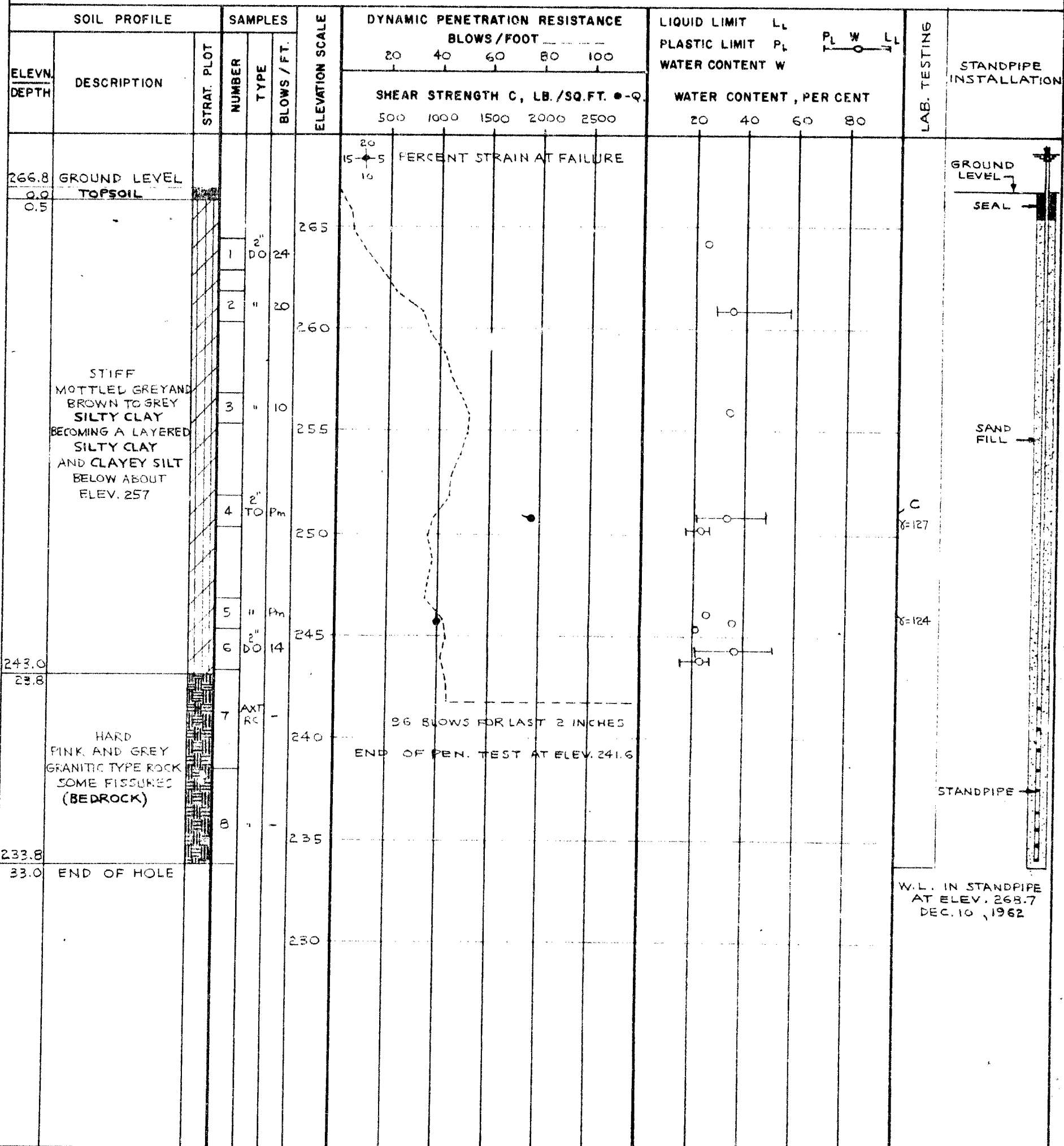
VERTICAL SCALE  
1 INCH TO 5'-0"

GOLDER &amp; ASSOCIATES

DRAWN M.W.  
CHECKED M.W.

RECORD OF BOREHOLE 3

LOCATION SEE FIGURE 1 BORING DATE DEC. 4, 1962 DATUM GEODETIC  
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



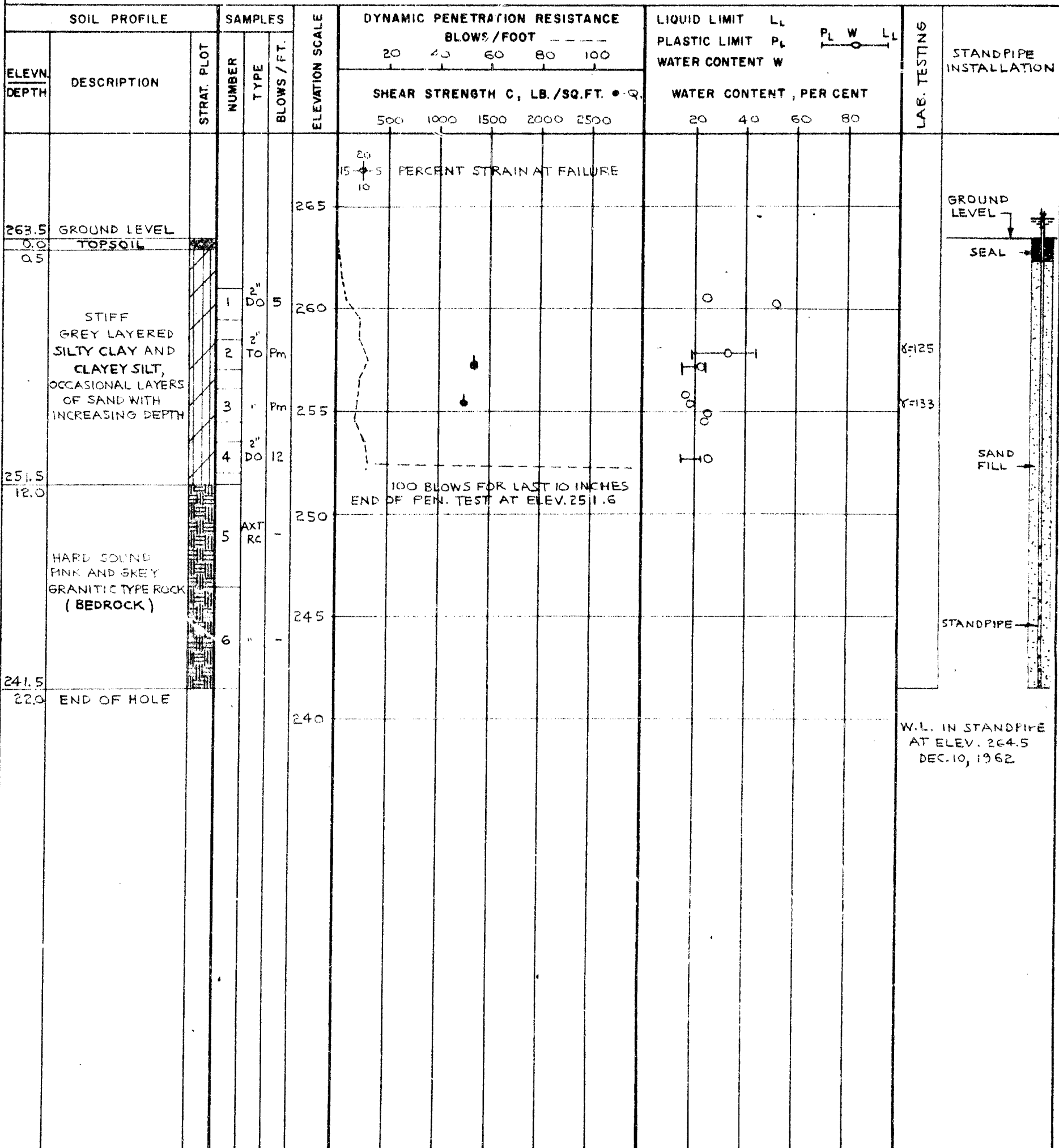
VERTICAL SCALE  
1 INCH TO 5'-0"

GOLDER & ASSOCIATES

DRAWN M.W.  
CHECKED M.W.

# RECORD OF BOREHOLE 4

LOCATION SEE FIGURE 1 BORING DATE DEC. 7, 1962 DATUM GEODETIC  
 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'-0"

GOLDER & ASSOCIATES

DRAWN M.W.  
CHECKED H. n/c

## RECORD OF BOREHOLE 5

LOCATION SEE FIGURE 1

BORING DATE DEC. 10, 1962

DATUM GEODETIC




BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER 8X CASING

SAMPLER HAMMER WEIGHT - LB. DROP - INCHES

PEN. TEST HAMMER WEIGHT - LB. DROP - INCHES

SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT					LIQUID LIMIT $L_L$ PLASTIC LIMIT $P_L$ $\frac{P_L}{L_L}$ WATER CONTENT $W$					REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		SHEAR STRENGTH $C$ , LB./SQ.FT.					WATER CONTENT, PER CENT					
266.4	GROUND LEVEL				270										W.L. IN BOREHOLE AT ELEV. 265.9 DEC. 10, 1962 	
0.0 264.5 1.9	SAND AND BOULDERS				265											
	HARD GREY CRYSTALLINE ROCK, FEW FISSURES MAINLY IN UPPER 2.5 FEET. (BEDROCK)			AXT RC	260											
253.9 12.5	END OF HOLE				255											
					250											

VERTICAL SCALE

1 INCH TO 5'-0"

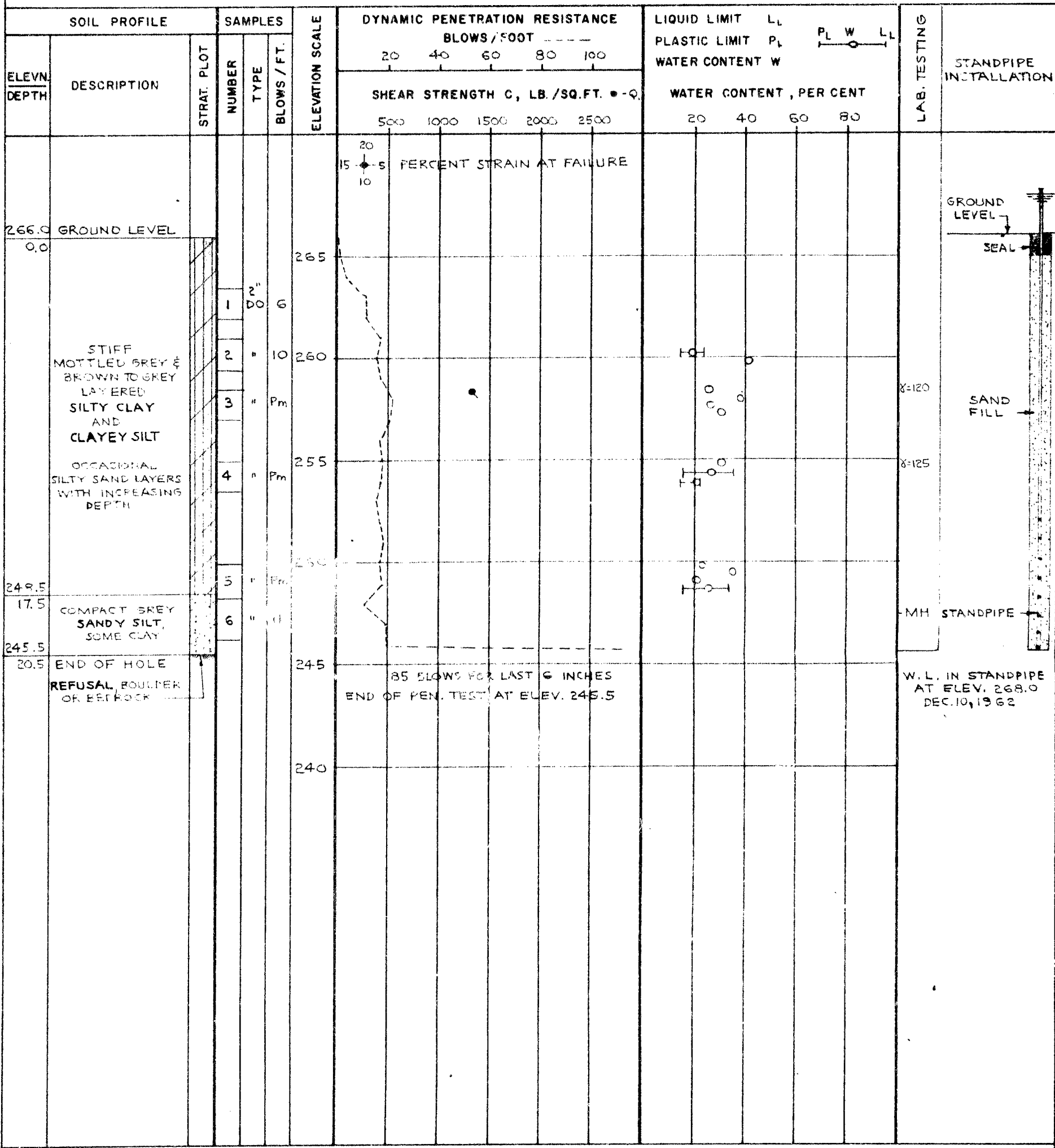
GOLDER &amp; ASSOCIATES

DRAWN M.W.

CHECKED *W.M.G.*

RECORD OF BOREHOLE G

LOCATION SEE FIGURE 1 BORING DATE DEC. 8, 1962 DATUM GEODETIC  
BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER 8X 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 M.W.  
CHECKED M.W.





## RECORD OF BOREHOLE 8

LOCATION SEE FIGURE 1

BORING DATE DEC. 7, 1962

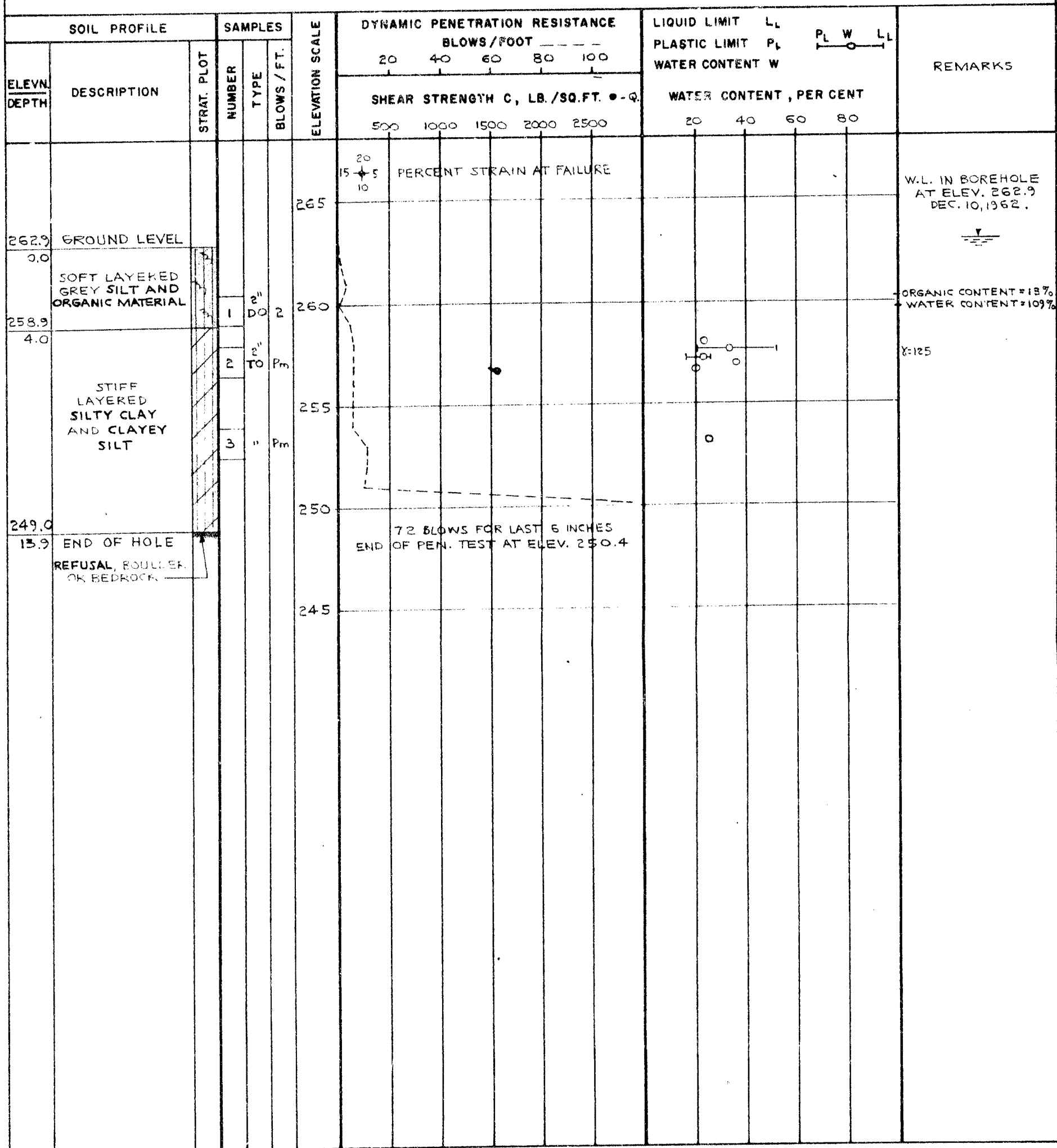
DATUM GEODETIC

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'-0"

GOLDER &amp; ASSOCIATES

DRAWN M.W.  
CHECKED M.W.





## RECORD OF BOREHOLE 12

LOCATION SEE FIGURE 1

BORING DATE DEC. 6, 1962

DATUM GEODETIC

BOREHOLE TYPE

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					LIQUID LIMIT $L_L$ PLASTIC LIMIT $P_L$ $P_L$ $W$ $L_L$ WATER CONTENT $W$						
ELEVN DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FT.		BLOWS / FOOT ----- 20    40    60    80    100					SHEAR STRENGTH C, LB./SQ.FT.						WATER CONTENT, PER CENT
262.6	CREEK LEVEL					265												
0.0						260												
257.6	CREEK BOTTOM					255												
5.0 ±						250												
245.7						245												
16.9	END OF PEN. TEST REFUSAL, PROBABLY BOULDER OR BEDROCK.					240												

VERTICAL SCALE

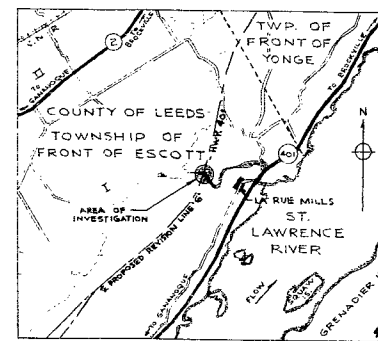
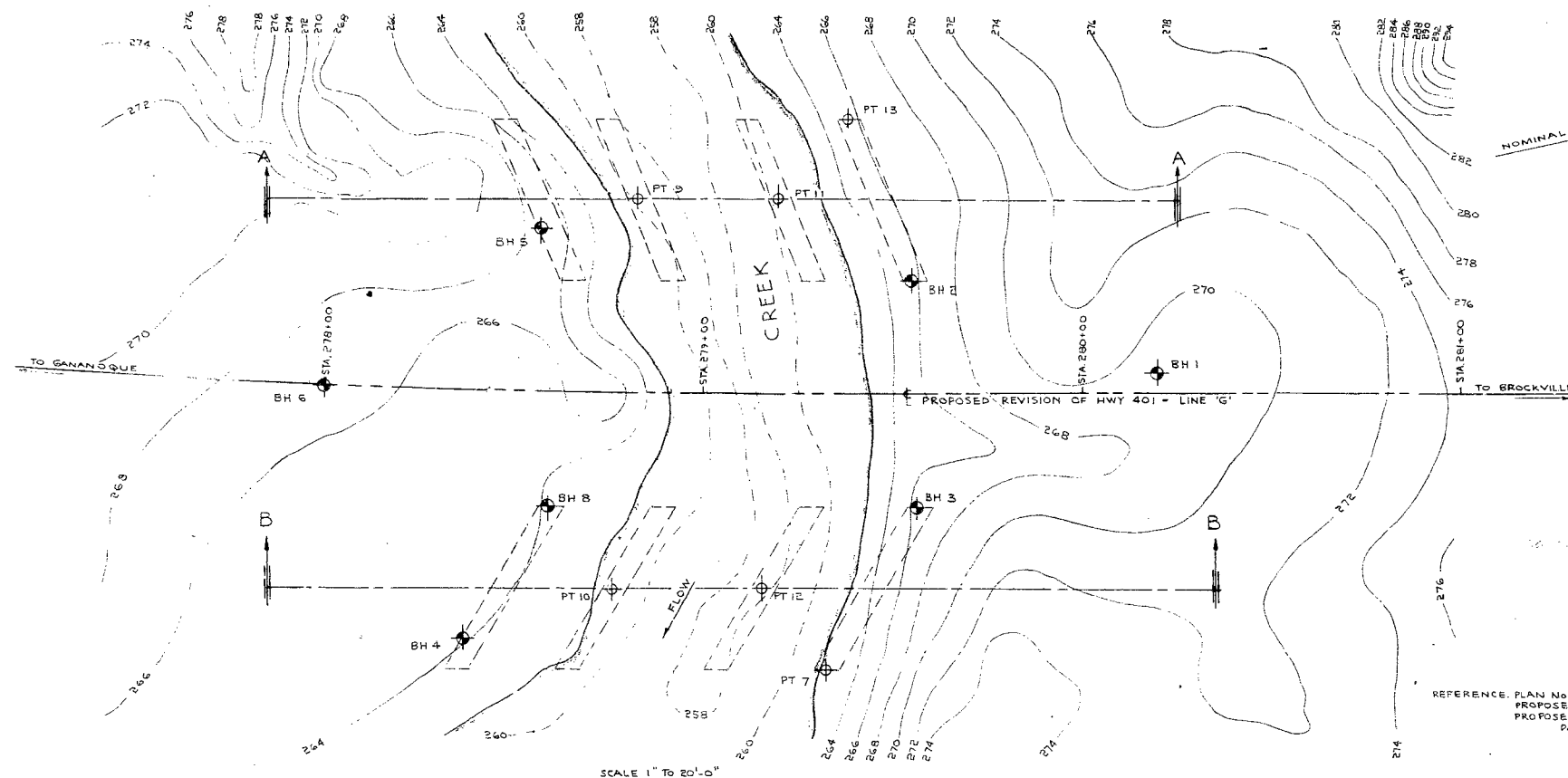
1 INCH TO 5'-0"

GOLDER &amp; ASSOCIATES

DRAWN M.W.

CHECKED *H. M. G.*

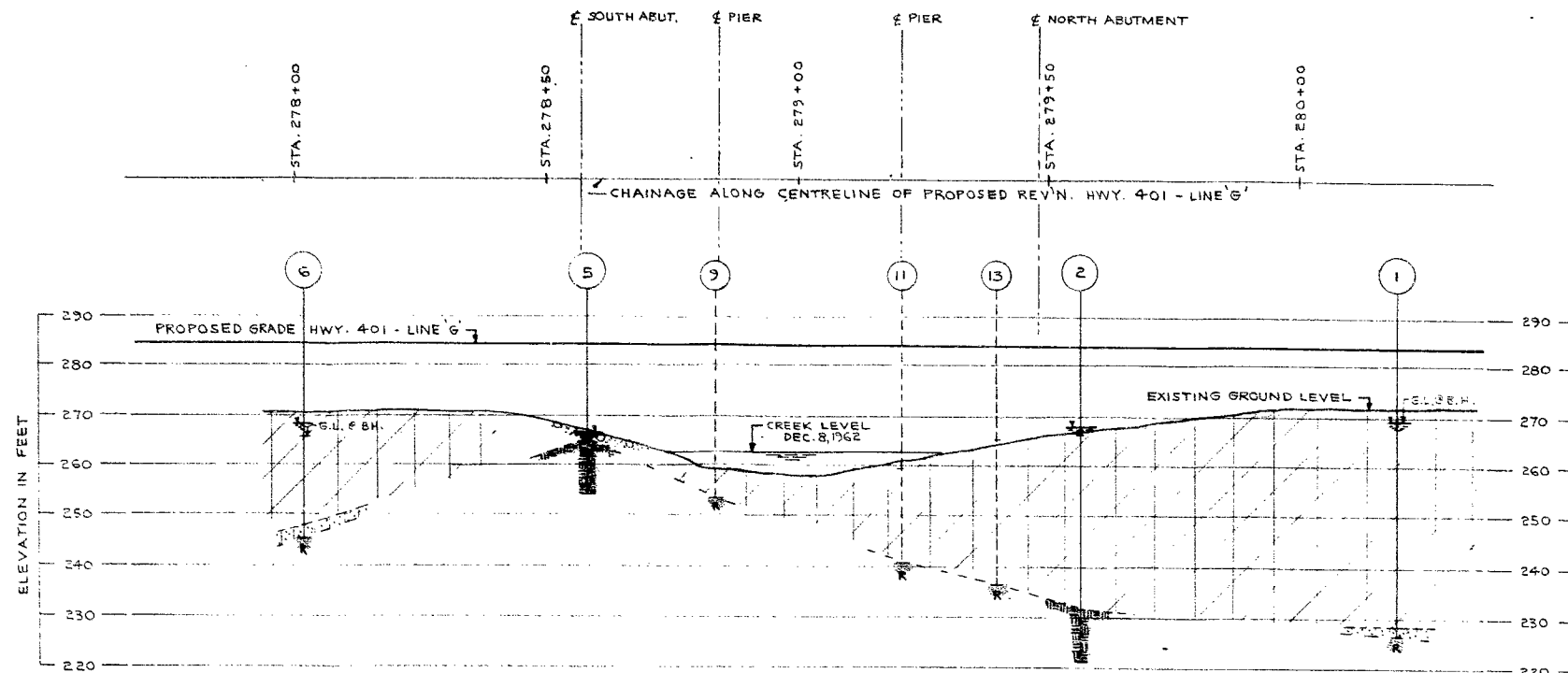




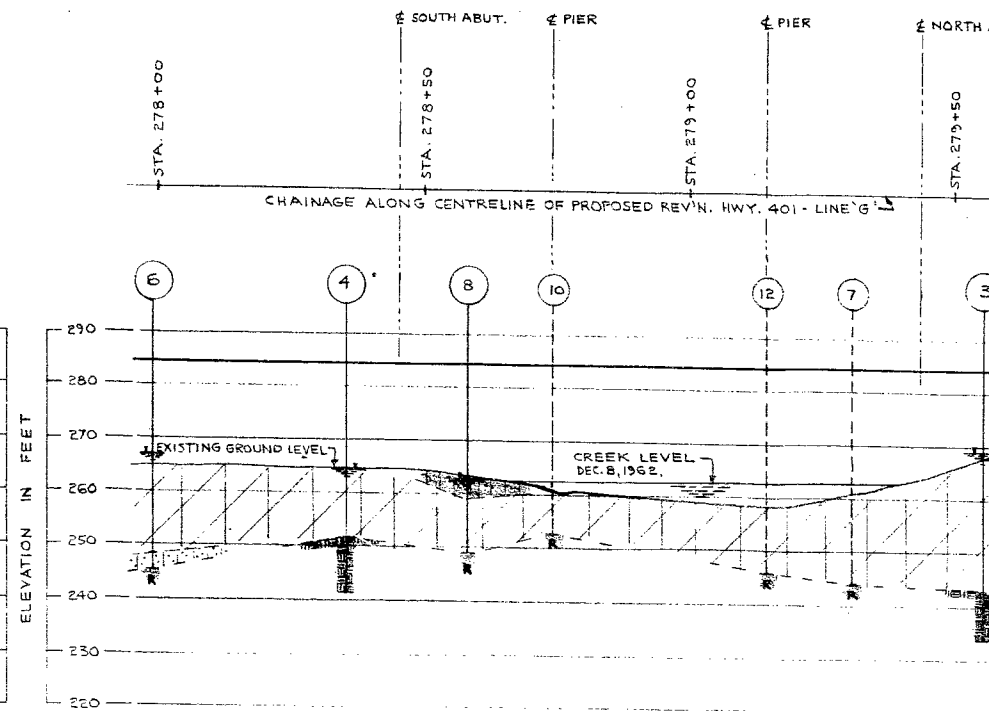
REFERENCE: PLAN No. E4136-1 D.H.O. DISTRICT No. 8  
 PROPOSED CROSSING AT CREEK AND  
 PROPOSED REV. OF HIGHWAY 401 - LINE 'G'.  
 DATED: AUG. 1962

GOLDER &amp; ASSOCIATES

Made by  
 Chkd. by  
 App'd. by






SECTION A-A









SECTION B-B

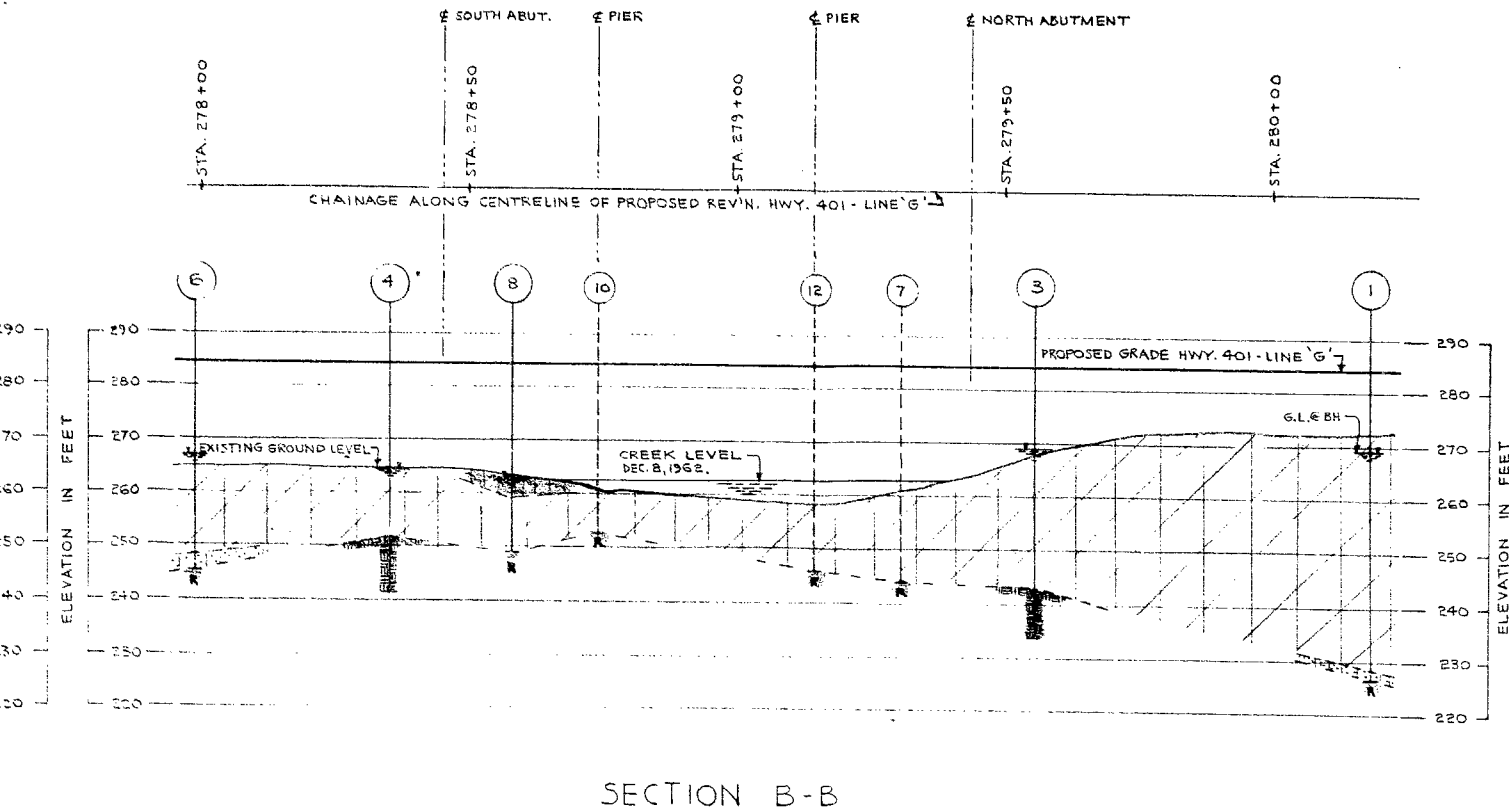
SCALE 1" TO 20' - 0"

## LEGEND

-  BOREHOLE IN ELEVATION  
 PENETRATION TEST IN ELEVATION  
 WATER LEVEL IN BOREHOLES, DEC. 10, 1962

## STRATIGRAPHY

-  SOFT LAYERED GREY SILT AND ORGANIC MATERIAL.  
 SAND AND BOULDERS  
 SOFT TO VERY STIFF GREY AND BROWN TO GREY SILTY CLAY BECOMING A LAYERED SILTY CLAY AND CLAYEY SILT, FEW SILTY SAND LAYERS.  
 COMPACT TO DENSE SANDY SILT, SOME CLAY.  
 BEDROCK  
 REFUSAL, BOULDER OR BEDROCK



SCALE 1" TO 20'-0"

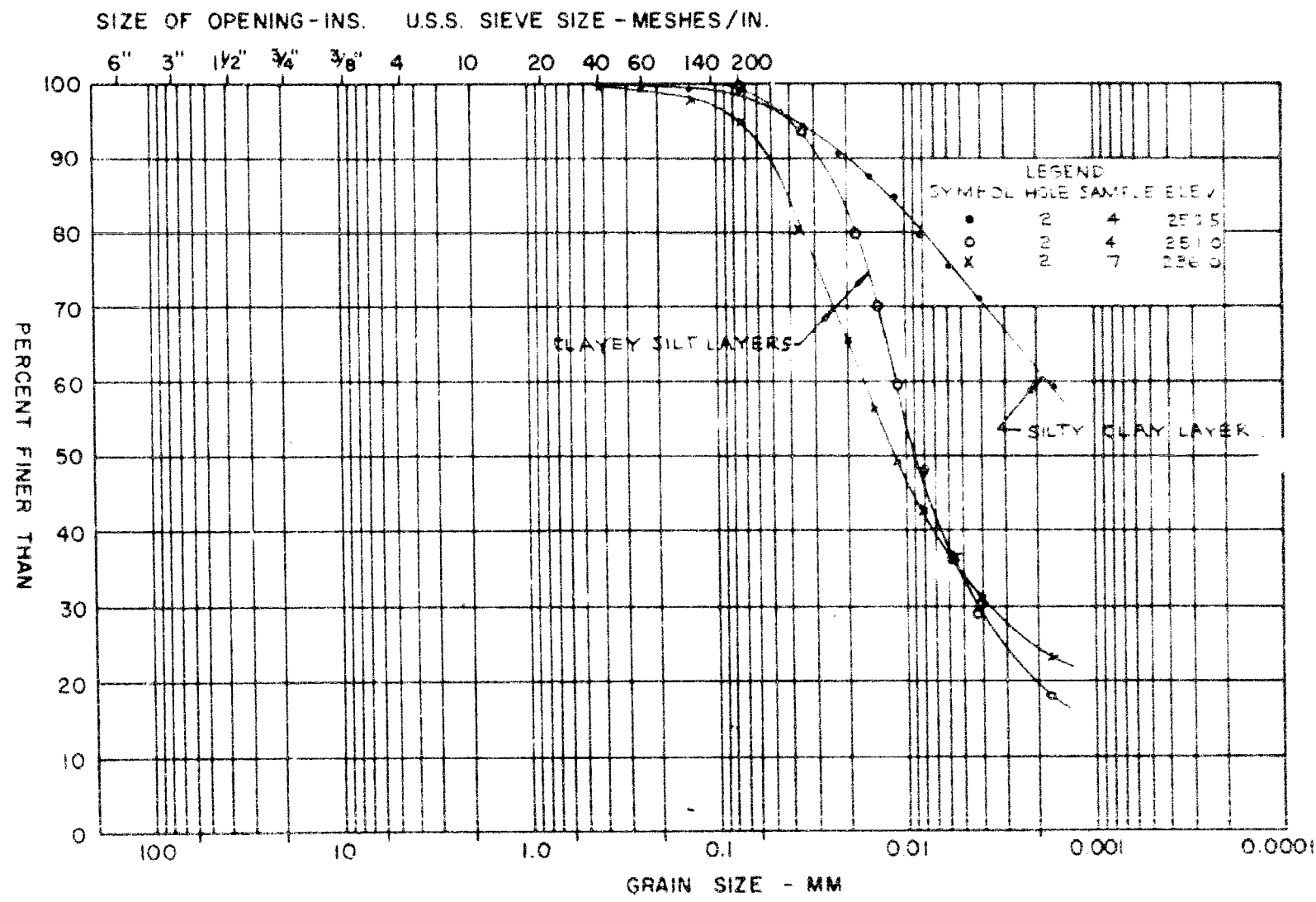
SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT VARIOUS LOCAL POINTS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

GOLDER &amp; ASSOCIATES

Made *M. W.*  
 Chkd. *T. J. G.*  
 Appd. *h. G.*



M.I.T. GRAIN SIZE SCALE



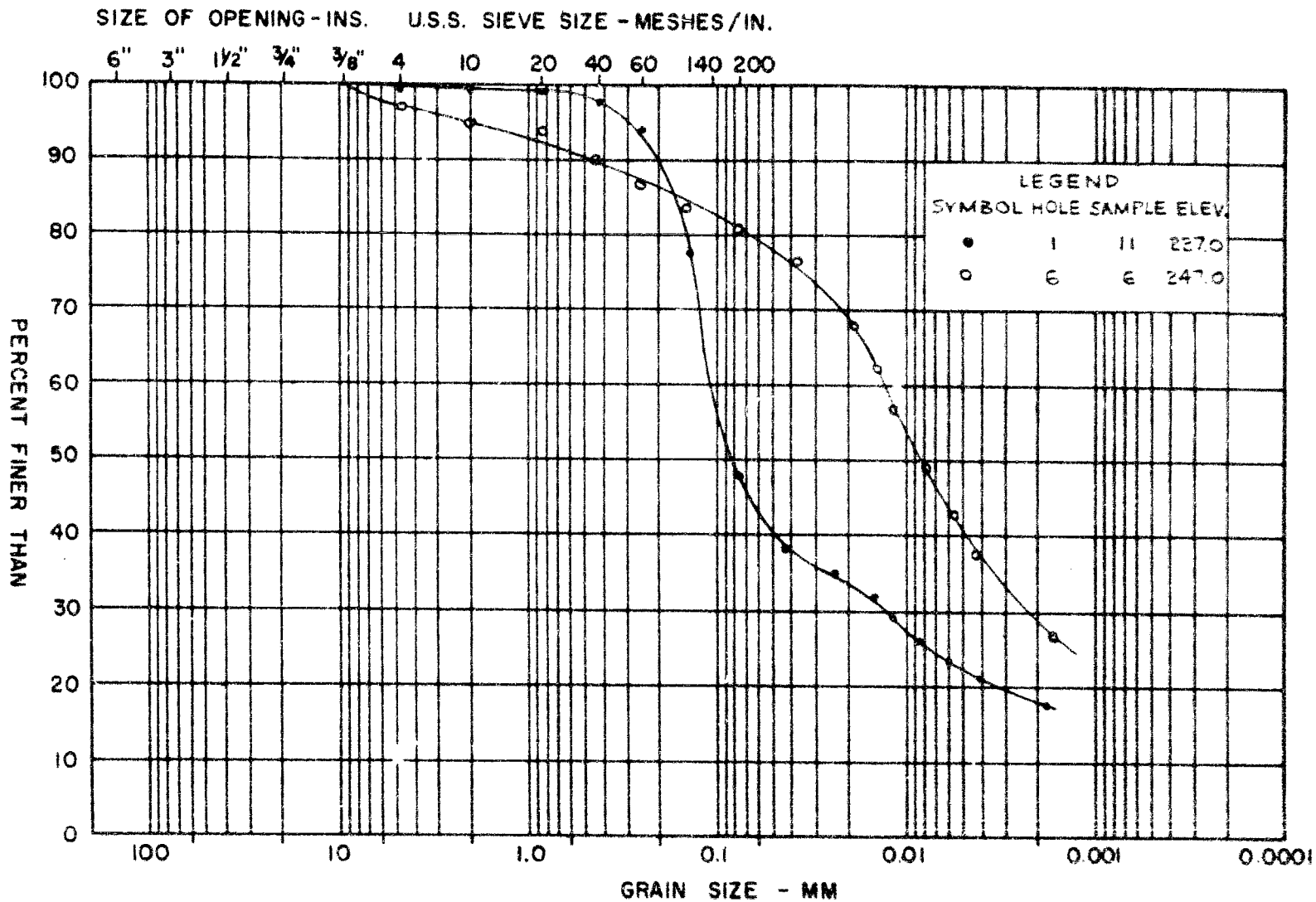
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
	GRAVEL SIZE			SAND SIZE			FINE GRAINED			

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GRAIN SIZE DISTRIBUTION  
LAYERED SILTY SAND AND CLAYEY SILT

FIGURE 1

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION  
SANDY SILT, SOME CLAY

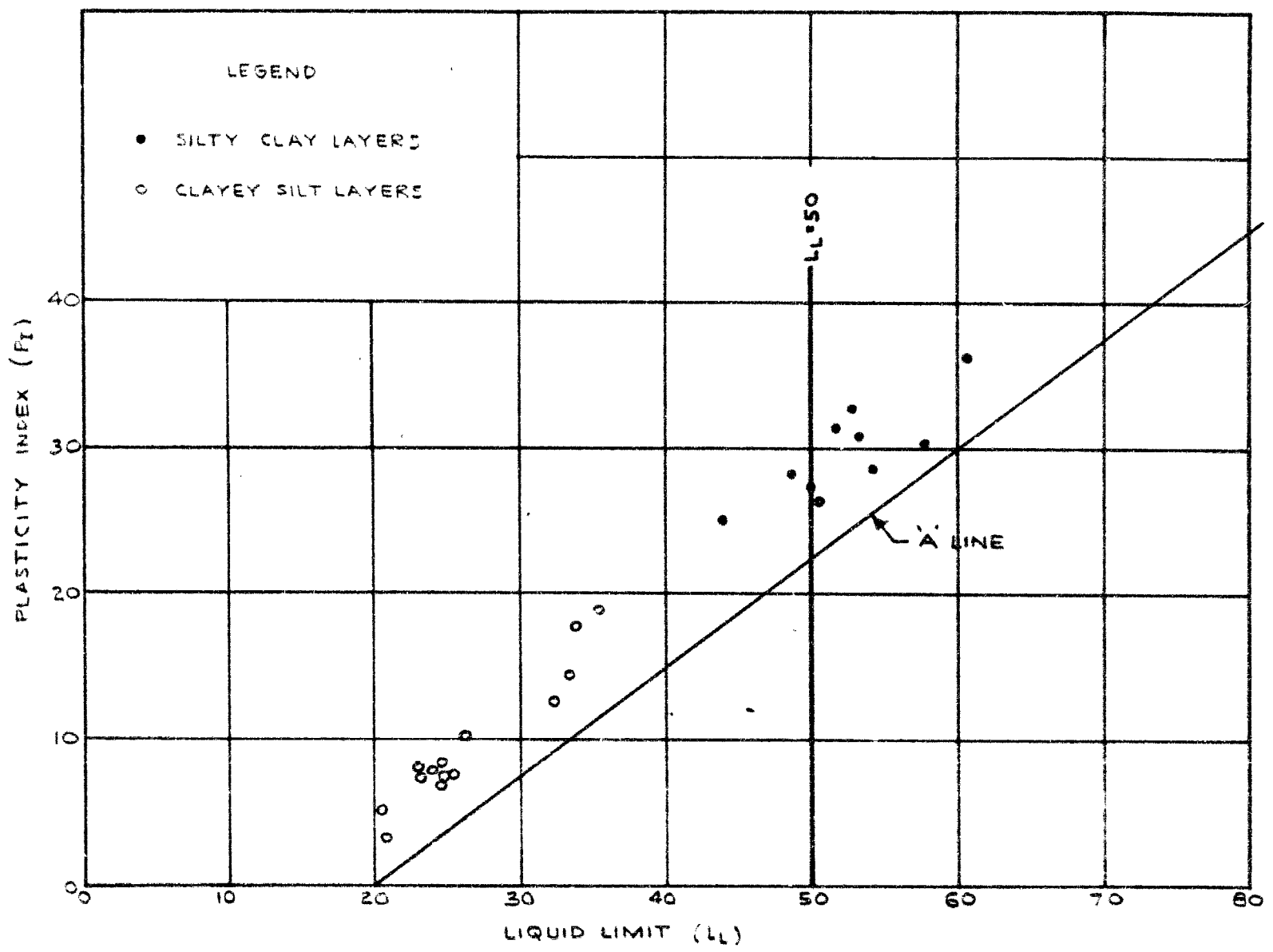
FIGURE 4

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

PLASTICITY CHART  
LAYERED SILTY CLAY AND CLAYEY SILT

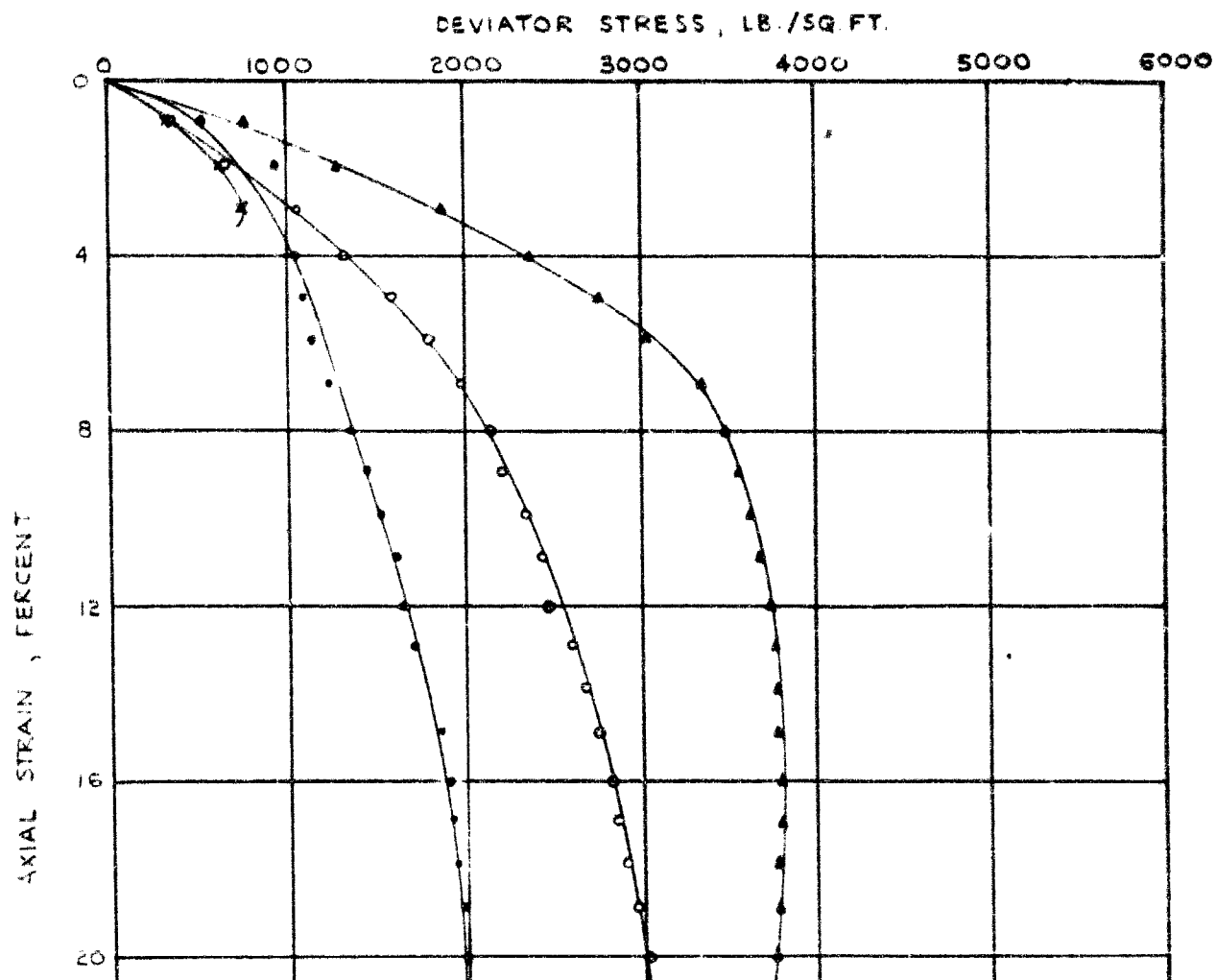
FIGURE 5

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# UNDRAINED TRIAXIAL COMPRESSION TESTS TYPICAL STRESS - STRAIN CURVES CLAYEY STRATA

FIGURE 6



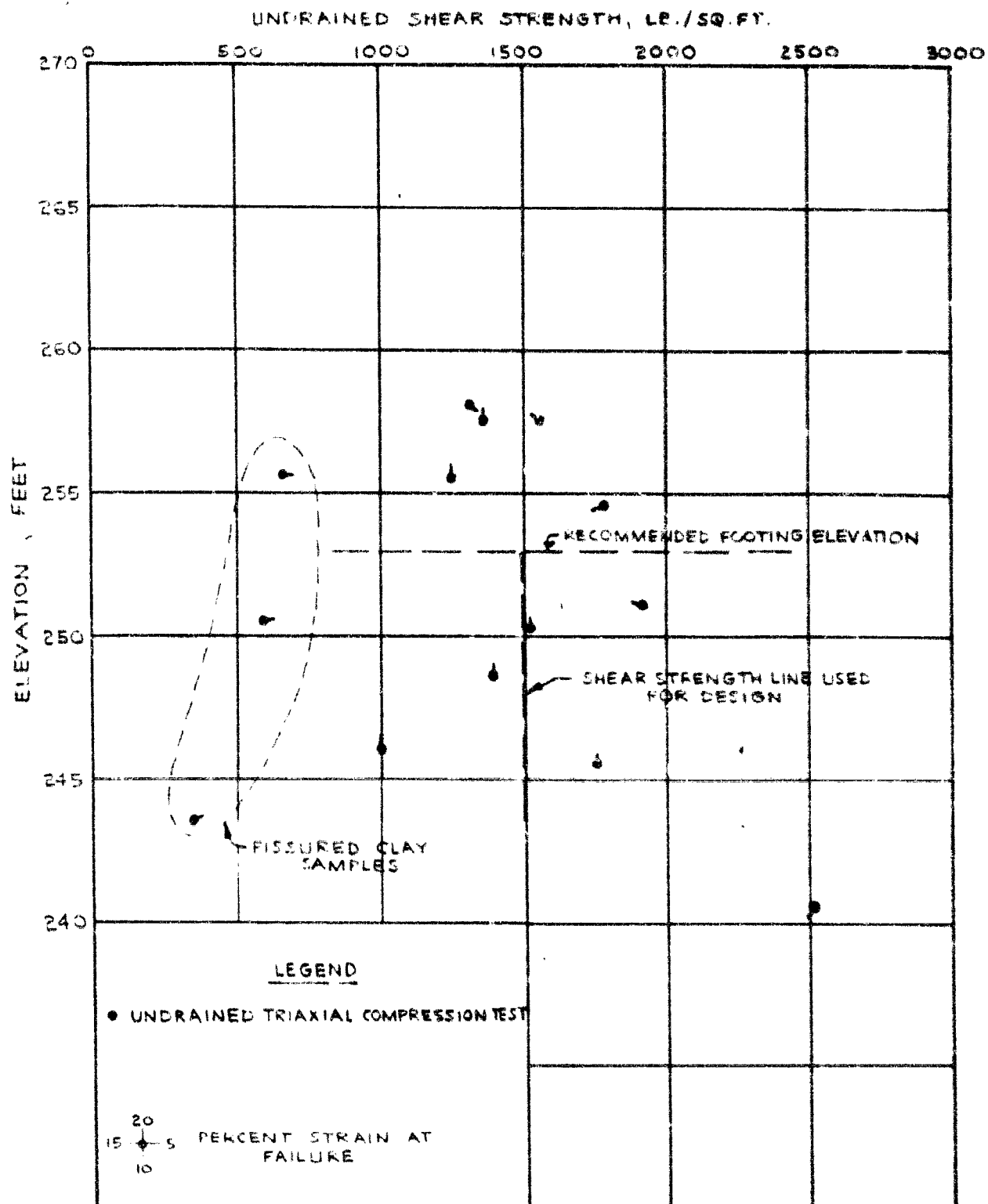
## LEGEND

SYMBOL HOLE SAMPLE ELEVATION

x	1	7	243.5
o	2	4	250.5
▲	3	4	251.0
●	3	5	246.0

# UNDRAINED SHEAR STRENGTH VS ELEVATION CLAYEY STRATA

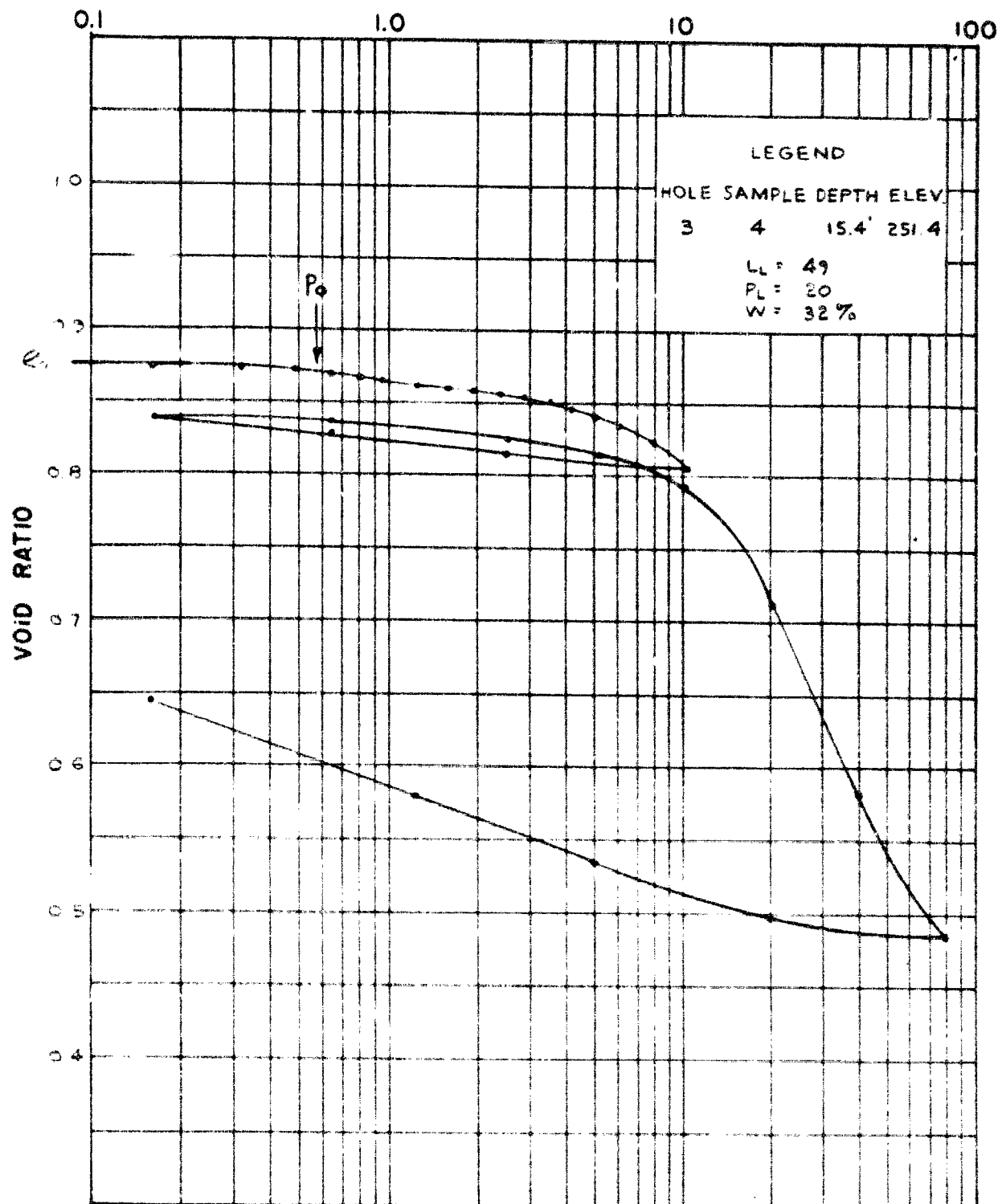
FIGURE 7



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 8

CLAY LAYER IN LAYERED SILT AND CLAY  
PRESSURE, TONS/SQ.FT.



GOLDER & ASSOCIATES

PROJECT No. 6266

CONSIDER THE VERTICAL EQUILIBRIUM OF THE BLOCK OF SOIL ABCD. BEDROCK AT ELEV. 243

UPWARD FORCE DUE TO ARTESIAN PRESSURE =  $26 \times 62.4 \times 12 = 19,500$  LB./FT. RUN

DOWNWARD RESISTING FORCE =  $(10 \times 2 \times 200) + (10 \times 120 \times 12)$

$= 4000 + 14,400 = 18,400$  LB./FT. RUN

$$FS_{HEAVE} = \frac{\text{DOWNWARD FORCE}}{\text{UPWARD FORCE}} = \frac{18,400}{19,500} = 0.94$$

