

MCCORMICK, RANKIN & ASSOCIATES
LIMITED

CONSULTING ENGINEERS

PORT CREDIT

OTTAWA

8 STAVEBANK ROAD
PORT CREDIT, ONTARIO
TELEPHONE 274-3477

66F223C

November 17th, 1967.

Mr. C. S. Grebski, P. Eng.,
Bridge Design Engineer,
DEPARTMENT OF HIGHWAYS,
Administration Building,
DOWNSVIEW, Ontario.

Attention: Mr. B. S. Richardson, P. Eng.

RE: BEECH RIVER BRIDGE
4.7 Miles North of Highway 530
at Carnarvon.
W. P. 178-64 - Site No. 40-9,
Highway 35 - District 11.
Our File W. O. 359-67

Dear Sir:

We have reviewed the results of the pile load tests carried out by the Foundation Section for the above-noted project, and would report as follows:

In discussion with Mr. K. Selby of the Foundation Section, we were informed that considerable difficulty was encountered in driving the 12. 3/4" o.d. steel tube piles for the load test and, therefore, the use of 16" o.d. steel tube piles for the pier bents is not considered advisable.

The possibility of using Franki piles as an alternative to steel "H" piles was considered. Mr. A. Prior of Franki, Canada, Ltd. estimates the bearing strata for 70 ton (design load) Franki piles to be approximately EL. 980. The estimated cost per foot of Franki pile is \$14.00 plus \$5,000.00 for the supply of pile driving equipment.

A comparative cost estimate for the use of steel 'H' piles and Franki piles is enclosed, herewith.

Mr. C. S. Grebski, P. Eng.

Due to the uncertain nature of the subsoil, it is recommended that the original design, using steel tube piles as columns, be amended. Two new schemes have been investigated.

In Scheme 1, five 24 inch diameter concrete columns would be used to support the pier cap. The pier footing would be supported by ten 60 tons steel 'H' piles, 70 feet in length. At the abutments an equal number of steel 'H' piles would replace the steel tube piles as shown. D. 6026-3.

In Scheme 2, the pier footing would be supported by five Franki piles placed at the same location as the 24 inch diameter concrete columns. At the abutments an equal number of Franki piles would replace the steel tube piles.

One other possible scheme might be the use of steel 'H' piles extended to the pier cap and encased in concrete. However, due to the uncertain nature of the subsoil, this scheme has not been given further consideration due to the more unequal load distribution from the deck to the foundation and the limited lateral support provided by the subsoil for vertical piles.

Considering the comparative cost estimate and the fact that the steel 'H' pile foundation would provide additional load capacity and stability, we recommend that the pier bents be redesigned using steel 'H' piles supporting concrete columns.

If this recommendation meets with your approval, we would be pleased to make the necessary changes to the contract drawings.

Yours very truly,
MCCORMICK, RANKIN & ASSOCIATES LIMITED

R. D. Nairn, P. Eng.

RDN/MA

c. Mr. K. Selby, P. Eng. ✓

COMPARATIVE COST ESTIMATE FOR FOUNDATIONS

BEECH RIVER BRIDGE

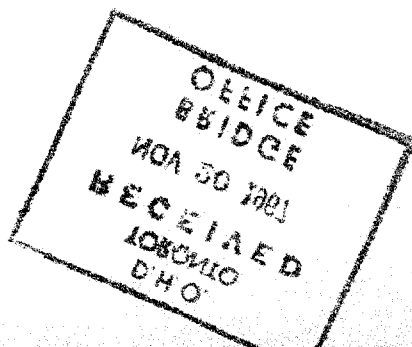
W.P. 178-64 - SITE No. 40-9

SCHEME 1 - STEEL 'H' PILES

Supply Equipment for Driving Piles	\$ 3,000.00
Supply & Drive Steel 'H' Piles - 2,750 L. F. @ \$5.00	<u>13,750.00</u>
	\$ <u>16,750.00</u>

SCHEME 2 - FRANKI PILES

Supply Equipment for Driving Piles	\$ 5,000.00
Supply & Drive Franki Piles - 1,000 L. F. @ \$14.00	<u>14,000.00</u>
	\$ <u>19,000.00</u>



Mr. A. E. McKim,
Bridge Contract Engineer,
Bridge Division,
Admin. Bldg.

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

Attention: Mr. K. Howe

July 7, 1967

Piles for Proposed Pile Load and Extraction Tests
At Beach River Crossing Hwy. 35 - W.P. 178-64
-- District #11 (Huntsville) --

As discussed with you by phone today, we are requesting you to requisition the following materials for use in the above project:

Steel Tube Piles:

12 $\frac{1}{2}$ " x 0.203" wall - 9 pieces at 45'
- 3 pieces at 40'

Steel H-Piles:

12 BP at 53 - 3 pieces at 45'

Timber Pile:

No. 14 (Creosoted) - 1 piece at 50'

It is intended to start the work on July 24, 1967. The Contractor was requested that the materials be made available to him a week in advance of this date.

As yet, no Work Order has been issued, but this should be expedited shortly.

The materials should be made available to the Contractor, Franki Canada Ltd., F.O.B., D.H.O. Yard, Downsview.

KGS/EdF

K. G. Selby
K. G. Selby,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

cc: Foundations Files ✓
Gen. Files

alp

Mr. C. S. Grebski,
Bridge Design Engineer,
Bridge Division,
Admin. Bldg.

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

March 1, 1967

Beech River Bridge (Gull River) -
Preliminary Plan #D-6026-P1,
W.P. 178-64 -- Site #40-9
Hwy. #35 - District #11 (Huntsville).

We have reviewed the preliminary plan for the above mentioned proposed structure, and submit the following comments:

(1) Although not mentioned in the Consultant's Foundation Report (H. Q. Golder & Associates), we recommend that the organic deposits within the limits of the approach fills and up to a point about 10 feet beyond the pier locations, be excavated and replaced with suitable fill.

(2) We recommend that consideration be given to the use of concrete displacement caissons as an alternative to the steel tube piles shown on the plan. Subsoil conditions appear to be best suited to the expanded base type of pile.

(3) If steel tube piles are definitely decided upon, it is recommended that a pile load test be carried out in the field to ascertain the appropriate working load and to obtain valuable data for use at other sites with similar soil conditions.

4-1-67 Selby

KGS/W1eF

cc: Messrs. S. McCombie
J. B. Curtis

K. G. Selby,
SUPERVISING FOUNDATION ENGINEER
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

Foundations Files.
Gen. Files.

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac, Principal Foundation Engineer,
Room 107, Lab. Building

Mr. J.B. Curtis,
Regional Bridge Location Engineer,
North Bay Regional Office

Bridge Division,
Downsview, Ontario

February 13, 1967

Beech River Bridge (Gull River)
4.7 Mi. North of Sec. Hwy. 530 at Carnarvon
N.P. 178-64, Site No. 40-9
Highway 35, District No. 11

Attached herewith are prints of the Preliminary Bridge
Plan Drawing D-6026-P1 for the above-mentioned structure.

The estimated cost of the proposed structure is \$92,300.
This cost includes tender, materials, engineering and sundry
construction.

Any comments or revisions you may have should be submitted
within three weeks.

CSG:rd

C.S. Grebaki,
Bridge Design Engineer

Attach.

c.c. E. Forrest
E. Cross
A. Stermac
S. McCombie

May. 401 & Leslie St
Toronto, Ontario

May 30, 1966

Materials and Testing Division

H. J. Galter and Associates Ltd.,
1444 Bloor Street West,
Toronto, Ontario.

Attention: Mr. J. L. Saychuk

- Re: Foundation Investigations - Letter of Authority -
- (1) S.F. 191-64 - The Batterville Falls Bridge,
May. No. 33, Proposed Revision, Line 'C', Dist. #11.
 - (2) S.F. 178-64 - The Beech River Bridge,
May. No. 33, Dist. #11.
 - (3) S.F. 66-64 - Site #11-3,
Rapineau Green, May. No. 177, Dist. #10.

Dear Sir:

This is to authorize you to carry out the foundation investigations at the above mentioned sites. The plans and all the necessary information pertaining to the jobs were given to your Mr. J. L. Saychuk on May 27, 1966. The names and telephone numbers of personnel to be contacted in connection with survey information and/or assistance, were also given to Mr. Saychuk.

The urgency of these investigations was discussed, and it was arranged that two of the investigations will be started on Monday, May 30, 1966, and the third one, immediately upon completion of the investigation closest to it.

You are requested to contact our office as soon as enough information becomes available and a meeting can be held with the designer. The final reports (10 copies of each project) will follow at a later stage; however, every effort should be made to have them delivered to our office as soon as possible.

May 30, 1966

Since the drawings accompanying the foundation reports, showing the location of borings, the inferred subsoil conditions, etc., are to become contract drawings, you are requested to prepare them in accordance with the B.M.C. standards. To enable you to do this, we are supplying you with a sample drawing with all the necessary explanations, together with linen sheets for your drawings. You are also requested to provide us with Greenflex copies of the drawings.

Charges for the work performed will be in accordance with your Schedule of Rates, dated October 1, 1965, and invoices to be addressed to the attention of the undersigned.

We are attaching the following Purchase Orders:

J 34810 - M.P. 191-64 (Batterville Falls Bridge).

J 34811 - M.P. 178-64 (Beach River Bridge).

J 34812 - M.P. 66-64 (Rapineau Creek Site #11-5).

covering the purchase of any new material required for this work, in order that you may use these as a basis for exemption from the Federal Tax for such purchases. The Exemption Certificate is printed thereon.

Yours very truly,

A. Sachs,

MATERIALS & TESTING ENGINEER

432/4127

Attach.

cc: Yours.

E. J. Solder
J. L. Seychuk
J. E. Miller
J. E. Jones
J. E. Callaghan
J. J. Devien
J. E. Crispier

cc: 1. Steinberg

J. E. Solder
J. E. Solder
J. E. Solder (2)
Foundations Office
Gen. Files (2)

DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

Mr. B. A. Davis
Bridge Engineer,
Bridge Division.

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

July 22, 1966

JUL 22 1966

Attention: Mr. B. A. Davis

FOUNDATION INVESTIGATION REPORT BY:
H. G. Collier and Associates, Limited -
Proposed Beach River Crossing, Hwy. 35,
Boshkung Lake, Ont. - District 11 (Muntville)
W.P. 178-64

Attached, please find the above mentioned report prepared and submitted by the consultant, H. G. Collier and Associates Ltd.

We have reviewed the report and are of the opinion that it contains all the information needed for your future design work.

Should you have any questions in connection with the above project that you would like to discuss, please feel free to contact our Office.

AGG/EdieP
Attach.

cc: Messrs. B. A. Davis (2)
H. A. Tregaskes
D. K. Parren
E. McArthur
A. E. Jones
J. Curtis
T. J. Kovich
A. Watt

Foundations Office
Gen. Files

H. G. Collier
for A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

66 F 223 C 23-70229
H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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

2444 BLOOR STREET WEST
TORONTO 9, ONTARIO
763-4103
767-9201

W.P. 178-64

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED BEECH RIVER CROSSING

HIGHWAY NO. 35

BOSHKUNG LAKE

ONTARIO

Distribution:

- 10 copies - Department of Highways, Ontario,
Toronto, Ontario.
- 2 copies - H. Q. Golder & Associates Ltd.,
Toronto, Ontario.

July, 1966

66079

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ABSTRACT

The results of an investigation to determine the sub-surface conditions at the site of a re-aligned crossing of Highway 35 over the Beech River, some $1\frac{1}{2}$ miles north of Carnarvon, Ontario are reported. Two possible re-alignments were included in the investigation. Recommendations are made for the foundation design of the proposed structure and approach embankments.

It was found that the site is covered by a thin topsoil mantle supporting tree growth. Directly underlying the surficial topsoil mantle is a stratum of loose to compact fine to medium sand some 50 to 90 feet thick. Included within the sand stratum, on the river banks, is a zone of compact layered silt and fine sand generally about 7 to 15 feet thick. Underlying the sand stratum, is a further deposit of compact layered silt and fine sand. The river level at the time of the investigation was at about elevation 1010 with the maximum depth of the river in the vicinity of the site about 9 feet. The measured piezometric groundwater level was generally at or about 1 foot above river level.

It is recommended that the piers and abutments of the proposed structure be founded on piles. It is considered that the most suitable pile type would be either a driven closed-end pipe pile or pre-cast concrete pile some 80 feet long or alternatively a concrete expanded base pile some 15 to 30 feet long. The piles should be installed as discussed in the report and a load test carried out on a representative pile to determine the final design load.

There should be no overall stability problem with approach embankments having 2 horizontal to 1 vertical side slopes provided that the surficial organic topsoil mantle is removed down to the surface of the sand deposit.

INTRODUCTION

H. Q. Golder and Associates Ltd. have been retained by the Department of Highways, Ontario, to carry out a soil investigation for the proposed bridge replacement over the Beech River on Highway 35 some 1½ miles north of Carnarvon, Ontario. The purpose of the borings put down during this investigation was to determine the subsoil and groundwater conditions at the proposed pier and abutment locations and to make recommendations for foundation design and construction of approach embankments. Two possible re-alignments were investigated in this report.

PROCEDURE

The field work for this investigation was carried out between May 31 and June 22, 1966. A total of 5 boreholes, each accompanied by a dynamic cone penetration test, as well as 8 additional dynamic cone penetration tests were put down using a skid-mounted diamond drillrig supplied and operated by the F.E. Johnston Drilling Company Ltd. For overwater boreholes the drillrig was mounted on a drum raft. Boreholes 1, 3 and 8 and penetration tests 2, 4, 5, 6, 7 and 9 were put down along or near proposed Revision Line "G", while boreholes 11 and 12 and penetration tests 10 and 13 were put down along proposed Revision Line "D". The borings, which were started in NX casing size and were completed in BX size, were

put down to depths ranging from 70 feet to 110 feet; practical refusal to casing driving was encountered in the deeper boreholes. The piezometric groundwater level was observed during the investigation by readings taken in sealed piezometers installed in all the borings, except borehole 11. The field work was supervised throughout by a member of our engineering staff.

A detailed log for each of the borings and dynamic cone penetration tests is given on the Record of Borehole and Record of Penetration Test sheets following the text of this report. The location of the borings put down in this investigation together with a section of the inferred soil stratigraphy across (a) proposed Revision Line "G" and (b) proposed Revision Line "D", is shown on Figures 1 and 2, respectively.

Samples obtained during the investigation were brought to our laboratory for detailed examination and testing. The results of the laboratory testing are shown on Figures 3 to 8, inclusive.

The elevations given in this report are referred to a benchmark consisting of a nail in the south root of a 1 foot diameter poplar tree located about 78 feet left of centre-line chainage 96+12 (Revision Line "C'"). The elevation of this benchmark is 1024.23 as referred to Geodetic datum.

SITE AND GEOLOGY

An existing one span bridge presently provides access across the Beech River; this bridge is located between the two proposed re-alignments. The Beech River in the vicinity of the site is about 125 feet wide with a maximum depth of about 9 feet; at the time of the investigation the river level was at about elevation 1010. The west river bank rises steeply (side slope $1\frac{1}{2}$ horizontal to 1 vertical) while on the east side the ground level rises gradually; the crest elevation of both banks is about elevation 1030. The terrain surrounding the river is quite heavily treed; numerous bedrock outcrops are visible to the north and east, about $\frac{1}{4}$ mile from the bridge site.

Based on available geological information it is considered that the site consists of deep fluvial deposits of interbedded sands and silts probably of post glacial origin. The bedrock in the area is metamorphic in origin, comprised principally of granite gneiss of Pre-Cambrian Age.

SUBSOIL CONDITIONS

The detailed stratigraphy encountered in each of the five boreholes and inferred at the penetration test location is given on the Record of Borehole and Record of Penetration Test sheets. The

stratigraphy interpolated from this data is presented on Figure 1 (Revision Line "G") and on Figure 2 (Revision Line "D"). Following is a summarized account of the inferred soil conditions at the site.

A thin mantle of sandy topsoil, about 3 to 6 inches thick occurs over most of the site. The river bed along Revision Line "D" is covered by about 4 feet of very loose organic sandy silt.

Medium to Fine Sand Stratum

Directly underlying the surficial deposits is the pre-dominant overburden stratum at the site, a fine to medium sand. The overall thickness of this stratum varies from about 50 feet to at least the full depth of penetration at borehole 1 (some 94 feet). The sand stratum is brown in colour in the upper portion changing to grey with depth. The sand stratum has a trace to some fine gravel sizes scattered throughout with the gravel content generally tending to increase with depth. Typical grading analyses carried out on representative samples of the upper portion of the sand are shown on Figures 3 and 4, while typical grading curves of the sand at depth are shown on Figures 6 and 7. A comparison of these grading results indicates the sand is relatively uniform throughout its depth and that the sand contains only a trace of silt sizes.

Standard penetration tests were carried out in the sand

stratum. Reference to the logs indicate that the "N" values in the upper portion of the sand (above elevation 980) range from about 2 blows/ft. to 17 blows/ft., being typically about 6 blows/ft. in the vicinity of Revision Line "G" and about 12 blows/ft. in the vicinity of Revision Line "D". From these results the relative density of the upper portion is considered to be loose along Revision Line "G" and loose to compact along Revision Line "D". In the lower portion of the stratum the "N" values range from about 5 blows/ft. to 30 blows/ft. being typically of the order of 15 blows/ft. It is inferred that most of the "N" values recorded that are less than 10 blows/ft. are probably indicative of some unavoidable loosening of the sand below the casing. Based on the above the relative density of the lower portion is considered to be compact.

Layered Silt and Fine Sand

Sandwiched between the upper and lower portion of the sand stratum, within and about 20 to 30 feet below the river banks, is a deposit of compact grey layered silt and fine sand. With depth the material changes to a layered fine sand with thin silt lenses. This deposit was not encountered beneath the river bed. The thickness of the layered material is typically about 7 feet in the vicinity of Revision Line "D" and 15 feet in the vicinity of Revision Line "G". Typical grading analyses carried out on representative samples of the layered silt and fine sand are shown on Figure 5.

Lower Deposits

Underlying the sand stratum across the site, at a depth of about 60 feet below river level, is a deposit of compact grey layered silt and fine sand. This deposit is 18 feet thick at borehole 3, the only borehole where this layered deposit was penetrated. Four grading analyses carried out on representative samples of the deposit are shown on Figure 8.

Directly underlying the layered silt and fine sand deposit at borehole 3 is a deposit of generally compact grey fine to medium sand. The boring was terminated within this deposit.

GROUNDWATER CONDITIONS

Piezometric groundwater level readings were taken during the period of the investigation in sealed piezometers installed in all the borings except borehole 11. Details of these installations are given on the Records of Boreholes. The final set of readings taken on June 22, 1966 indicate that the piezometric groundwater level across the site varies from about river level to about 1 foot above river level. At the time of the investigation the river water level was at elevation 1009.7. No artesian pressure was encountered within the overburden at the site.

DISCUSSION

General

It is understood that a bridge crossing of the Beech River is planned either along (a) Revision Line "G" some 130 feet north of an existing bridge or (b) Revision Line "D" some 40 feet south of the existing bridge. The proposed bridge will form part of Highway 35 between Dorset and Minden, Ontario. The location of the two re-alignments are shown on Figures 1 and 2.

It is understood that the proposed bridge is to be a three span structure, with the end spans being about 60 feet long and the central span being about 80 feet in length. It is not known at this time whether the abutments, which will be located slightly back from the creek edge, will be spill-through or of the retaining type. If Revision Line "G" is adopted the proposed highway grade is to be at about elevation 1038, that is some 28 feet above river level, necessitating a roadway approach embankment about 20 feet high on the east bank.

Foundations

Due to the variable and generally loose density of the upper zone of the sand stratum at the site, which extends from river bed to as much as 25 foot depth, the sand is not considered a suitable

bearing stratum for the support of the bridge abutments and piers on spread footings. For this reason, and because of the susceptibility of the sand to scour, it is recommended that a piled foundation be employed at this site due to the more favourable subsurface conditions at depth. Suitable pile types are discussed below.

(i) Driven Displacement Piles

It is considered that a driven closed-end pipe pile filled with concrete is a pile type that would be suitable at this site. From preliminary computations carried out for a pipe pile, 12 inches in diameter and 80 feet long, or driven to a final set of 20 blows/inch with a hammer developing 20,000 ft.lb. of energy/blow, it is considered that an allowable working load of about 70 tons/pile would be realized. This figure may be taken for preliminary design but it is recommended that at least one pile loading test be carried out on a representative pipe pile to determine the allowable working load prior to final design.

Another possibility would be a driven pre-cast concrete pile supplied with an interlocking splice. The allowable working load for use in preliminary design for such a pile about 80 feet long with a 12 inch section should be taken as 70 tons/pile. As discussed previously a full scale pile loading test should be carried out prior to final design.

(ii) Expanded Base Piles

An alternative to a driven displacement type pile is a Franki type pile. The feasibility of this type of pile as the possible foundation solution for the proposed structure was discussed with Mr. W. E. Lardner, P.Eng. of Franki of Canada Limited on July 6, 1966.

At the pier locations, it is considered that at a depth greater than 15 feet, the expelling of a large base by a high number of blows with a drop hammer will increase the density of the sand underlying the base sufficiently to allow a design working load of 150 tons/pile. In this case, the actual working load chosen below this maximum capacity would depend on structural considerations such as pile shaft diameter and spacing of bridge girders.

At the western abutment location a base expanded some 15 to 20 feet below existing ground surface would be located some 10 feet above the layered silt and fine sand stratum. It is recommended that the design load be limited to 100 tons/pile in this case.

At the eastern abutment location, the layered silt and fine sand stratum is about 20 feet below ground surface. It may be

difficult to expand a proper base above this fine granular material. It is therefore recommended that basing operations be carried out in the compact fine to medium sand and fine sand stratum at or below about elevation 980. The maximum allowable working load for design purposes may then be taken as 150 tons/pile. As before it is recommended that a pile load test be carried out to check the pile loads used in design.

As relatively dense sand exists from proposed pile tip depth to the maximum depth of exploration in the boreholes (110 feet below ground surface) the settlement of a single test pile should be less than $\frac{1}{4}$ inch when supporting the design load. The settlement of the pile group will depend on a number of factors including (a) the dimensions of the pile group, (b) pile spacing and (c) pile diameter etc. It is considered however, that the settlement of the pier pile groups should not exceed 1 inch. The actual expected settlement of the pile groups may be computed from the proposed test pile results, once the piling layout at the pier and abutment locations has been finalized.

For frost protection purposes foundations for the abutments should be protected by at least 5 feet of earth cover.

If retaining type abutments are used, it is recommended

that the backfill behind the abutments should be comprised of free-draining, non-frost-susceptible granular material compacted in horizontal lifts of about 9 inches. This backfill should extend horizontally from the back face of the abutment walls a minimum distance of 5 feet. Provision for drainage from this material should be made. With full effective drainage behind the walls it is recommended that a coefficient of earth pressure at rest, $K_o = 0.4$, and a total unit weight, γ , of 135 lb/cu.ft. be used for the compacted granular backfill in design of the walls. If some movement of the top of the abutment "retaining" walls can be tolerated an active earth pressure coefficient, $K_a = 0.3$ may be used. The abutments should be designed for a factor of safety of at least 1.5 against sliding based on a coefficient of friction of concrete to the natural sand of 0.45.

Approach Embankments

The maximum height of earth embankment fill will be placed immediately behind the abutment located on the east bank of the river; this fill will be of the order of 25 feet high. Prior to placement of any fill it is recommended that the surficial topsoil cover be stripped. The approach embankments directly behind the abutments should then be constructed on the sand stratum underlying the topsoil using 2 horizontal to 1 vertical side slopes. In this regard it is considered that the locally available natural fine to

medium sand would be suitable for use as the required embankment fill. For an embankment of properly compacted granular fill there should be no overall stability problem at this site during or following construction.

The earth fill embankment will experience some minor settlement due to compression of the loose granular subsoil. For an embankment section 25 feet high this settlement could be of the order of 2 to 3 inches. Most of this settlement will take place during the construction period.

Rip-rap should be placed over the side of the approach embankments to at least 3 feet above the high water level in order to prevent erosion scour and undermining of the embankments. Above the rip-rap, the embankment slopes should be sodded or seeded and mulched to minimize surface water erosion and gullying.

B. T. Darch,

B. T. Darch, P.Eng.



Hugh Q. Golder

H. Q. Golder, P.Eng.

BTD:hdg
66079
July 18, 1966.

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
DS	Denison type sample
FS	foil sample
RC	rock core
ST	slotted tube
TO	thin-walled, open
TP	thin-walled, piston
WS	wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH	sampler advanced by static weight—weight, hammer
PH	sampler advanced by pressure—pressure, hydraulic
PM	sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Relative Density	<i>N</i> , blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	<i>c_u</i> , lb./sq. ft.
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer ¹
Q	undrained triaxial ²
R	consolidated undrained triaxial ²
S	drained triaxial
U	unconfined compression
V	field vane test

NOTES:

¹Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

²Undrained triaxial tests in which pore pressures are measured are shown as \bar{Q} or \bar{R} .

LIST OF SYMBOLS

I. GENERAL

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 = $c_s t / d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion
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

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 MAY 31 - JUNE 2, 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

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FT.	COEFFICIENT OF PERMEABILITY K, CM. / SEC.	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE		BLOWS / FT.	SHEAR STRENGTH C _u , LB./SQ. FT.		
	TOPSOIL							
1022.0	GROUND LEVEL							
0.3			1 2" D.O.	4				
	LOOSE BROWN TO GREY BROWN FINE TO MED. SAND		2 "	5				
			3 "	8				
			4 "	7				
1005.0			5 "	7				
27.0	COMPACT GREY LAYERED SILT		6 "	17				
999.0			7 "	10				
33.0	LOOSE GREY BROWN FINE TO MEDIUM SAND		8 "	9				
990.5			9 "	5				
41.5	LOOSE GREY LAYERED FINE SAND AND SILT		10 "	8				
984.0			11 "	16				
48.0			12A WS	-				
			12 D.O.	19				
	LOOSE TO COMPACT GREY FINE TO MEDIUM SAND		13 "	7				
			14A WS	-				
			14 D.O.	18				
			15A WS	-				
			15 D.O.	16				
			16A WS	-				
			16 D.O.	19				
949.0			17A WS	-				
2.0	COMPACT GREY FINE SAND		17 D.O.	19				
			18A WS	-				
			18 D.O.	17				
938.0			19 WS	-				
94.0	END OF HOLE							

1040

1030

1020

1010

1000

990

980

970

960

950

940

930

157

END OF PEN. TEST AT EL. 958.0

0

5

10

Percent axial strain at failure

CAP

GROUND LEVEL

SURFACE SEAL

M

MH

MH

MH

MH

MH

MH

MH

PIEZOMETER

W.L. IN PIEZOMETER AT ELEV. 1010.3 JUNE 22, 1966

VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

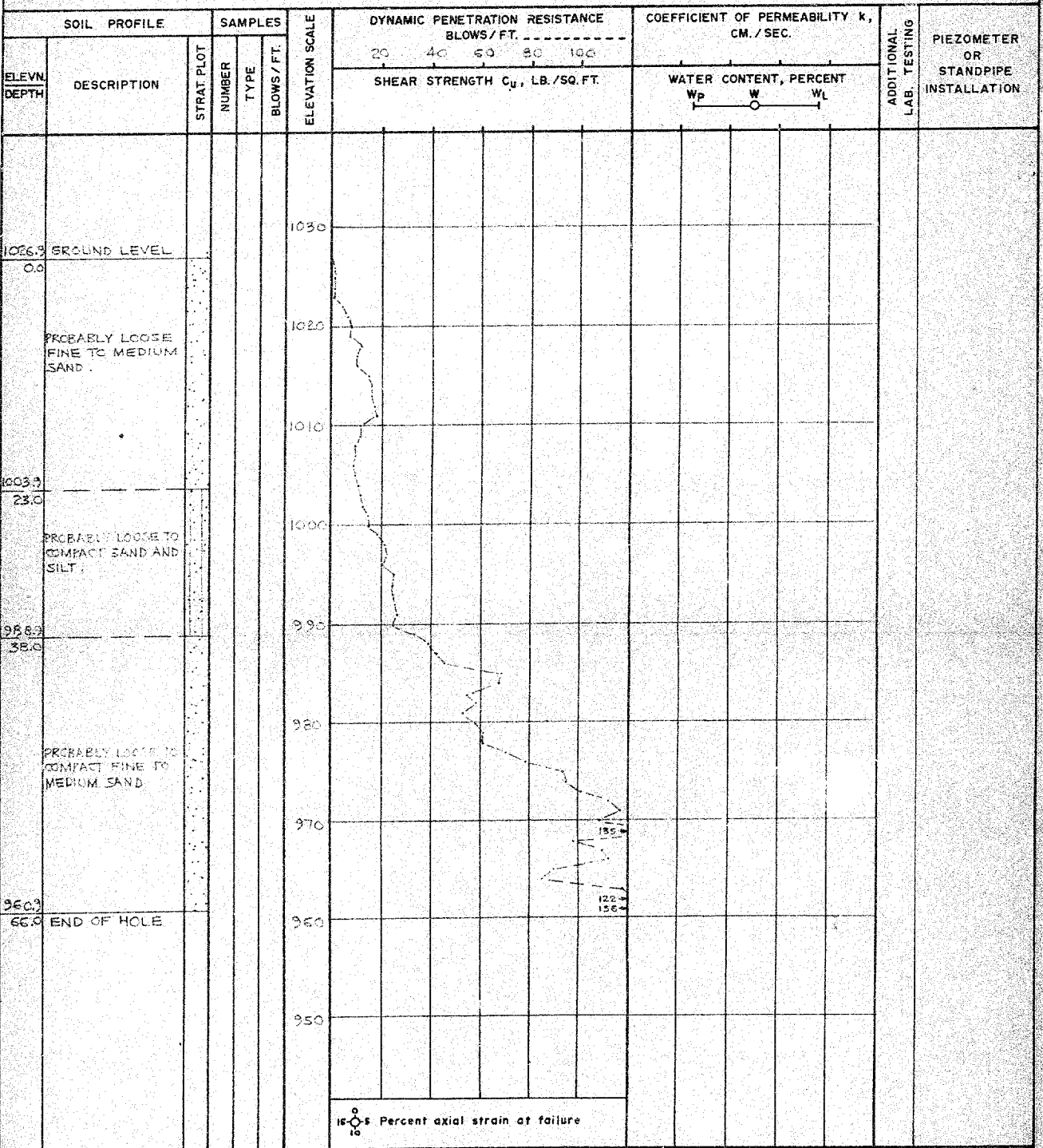
DRAWN W. A. D.
CHECKED _____

RECORD OF PENETRATION TEST 2

LOCATION See Figure 1 BORING DATE JUNE 3, 1963 DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



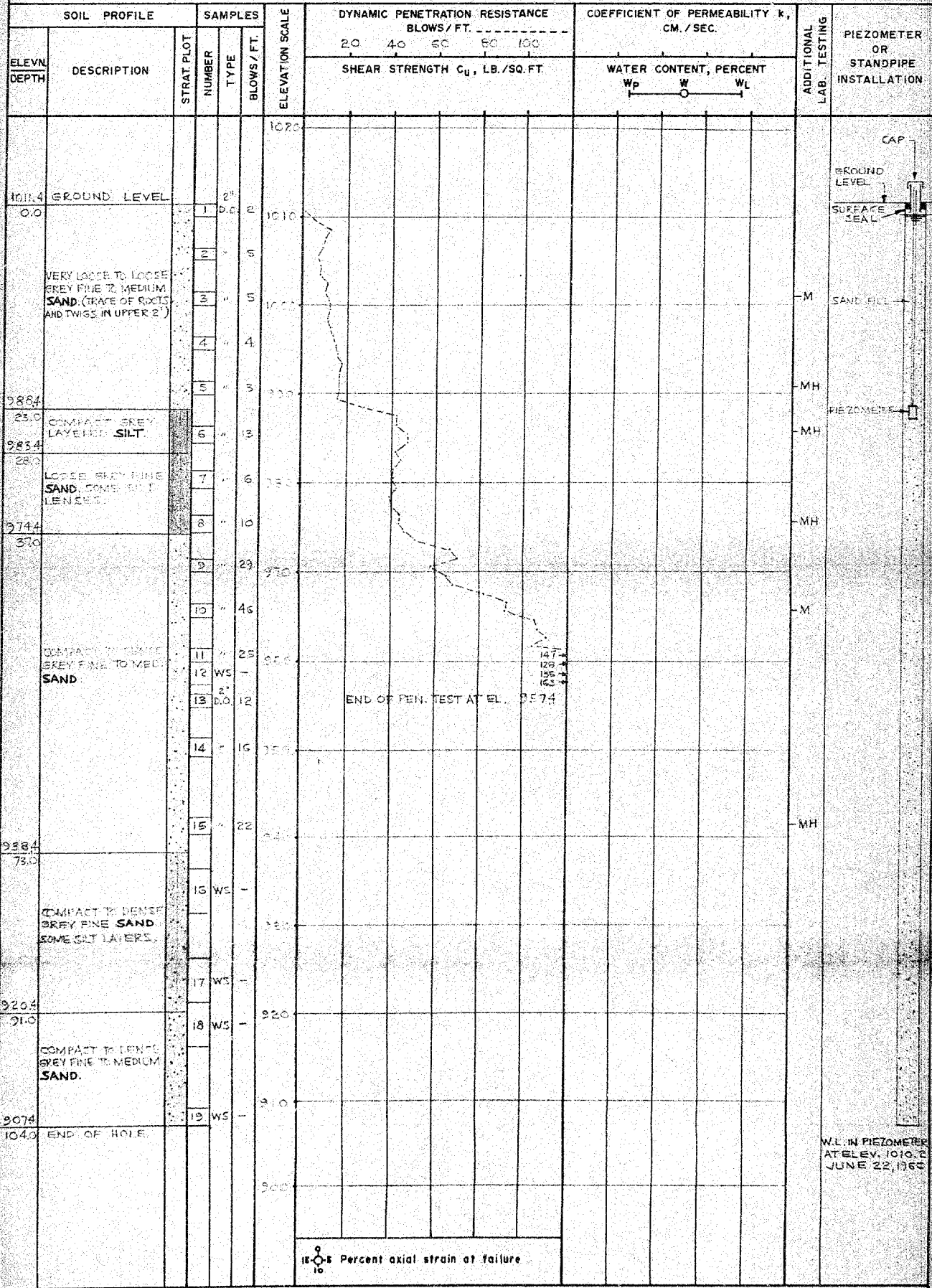
VERTICAL SCALE
1 INCH TO 10' 0"

GOLDER & ASSOCIATES

DRAWN *[Signature]*
CHECKED

RECORD OF BOREHOLE 3

LOCATION See Figure 1 BORING DATE JUNE 6-8, 1965 DATUM GEODETIC
BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER AX, BX, NX CASING
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



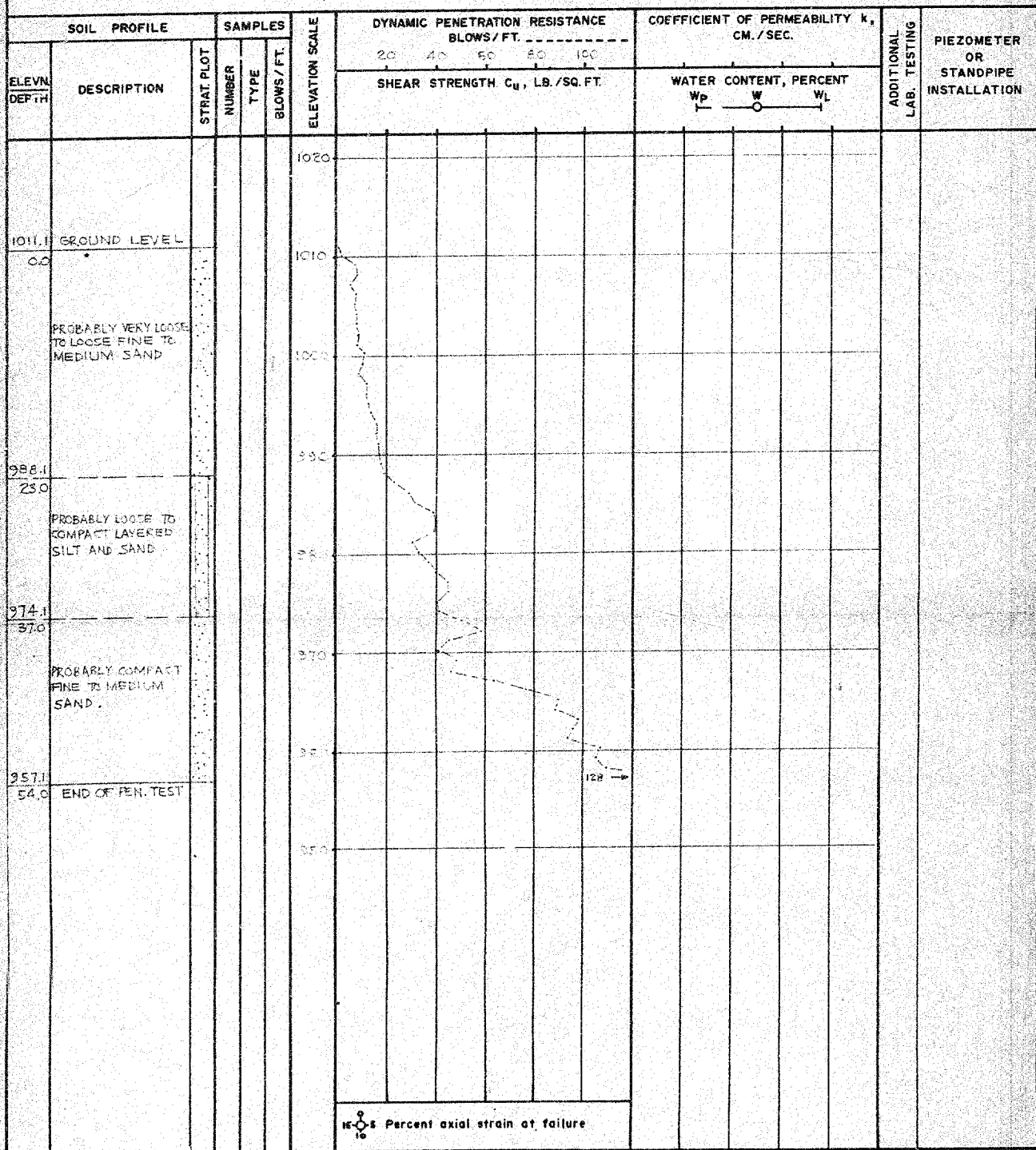
VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *mlw*
CHECKED

RECORD OF PENETRATION TEST 4

LOCATION See Figure 1 BORING DATE JUNE 9, 1966 DATUM GEODETIC
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER -
 SAMPLER HAMMER WEIGHT - LB. DROP - INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



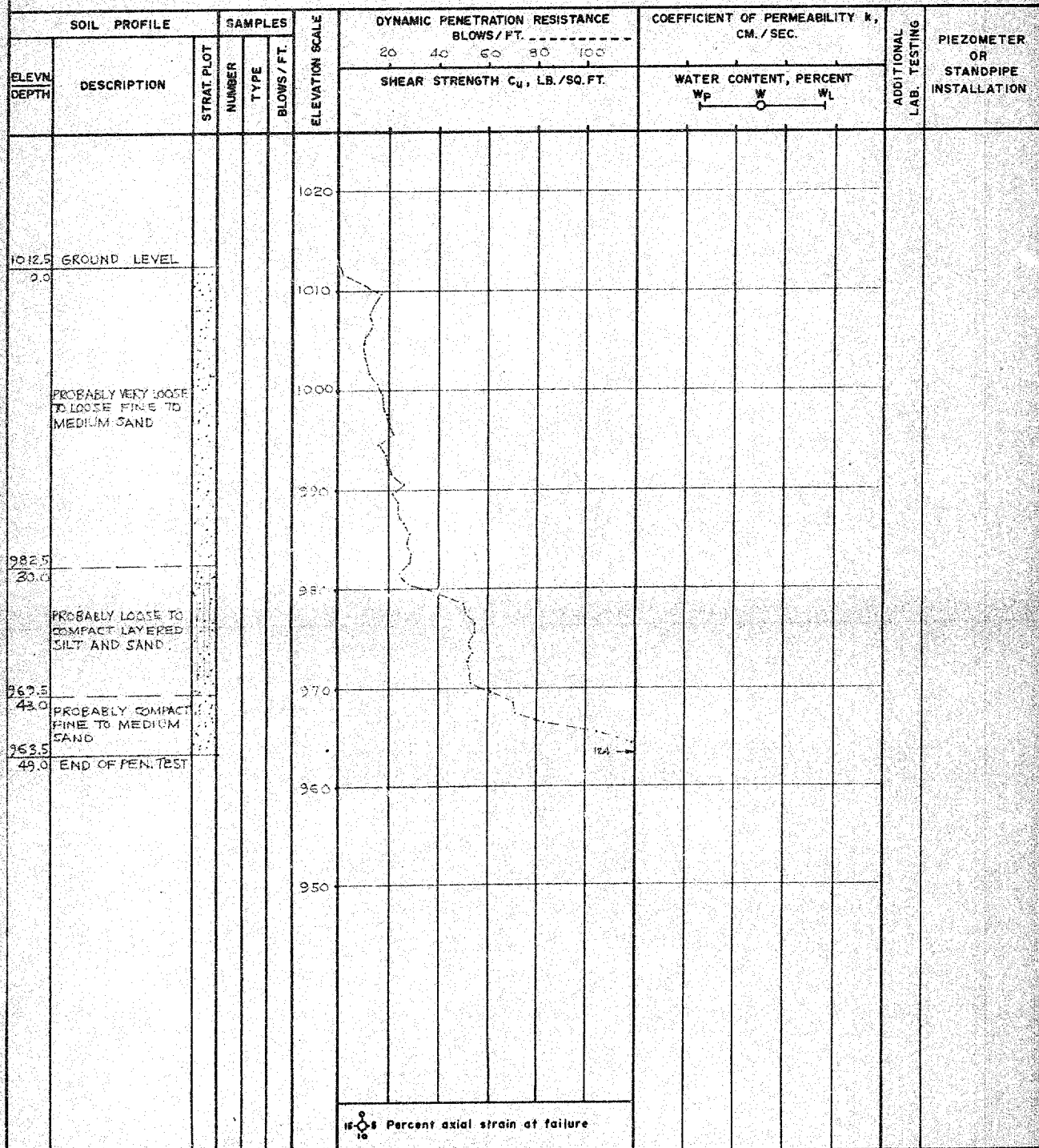
VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *[Signature]*
CHECKED

RECORD OF PENETRATION TEST 5

LOCATION See Figure 1 BORING DATE JUNE 9, 1966 DATUM GEODETIC
 BOREHOLE TYPE PENETRATION BOREHOLE DIAMETER —
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT LB. DROP INCHES



VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *[Signature]*
CHECKED

RECORD OF PENETRATION TEST 6

LOCATION See Figure 1

BORING DATE JUNE 9, 1966

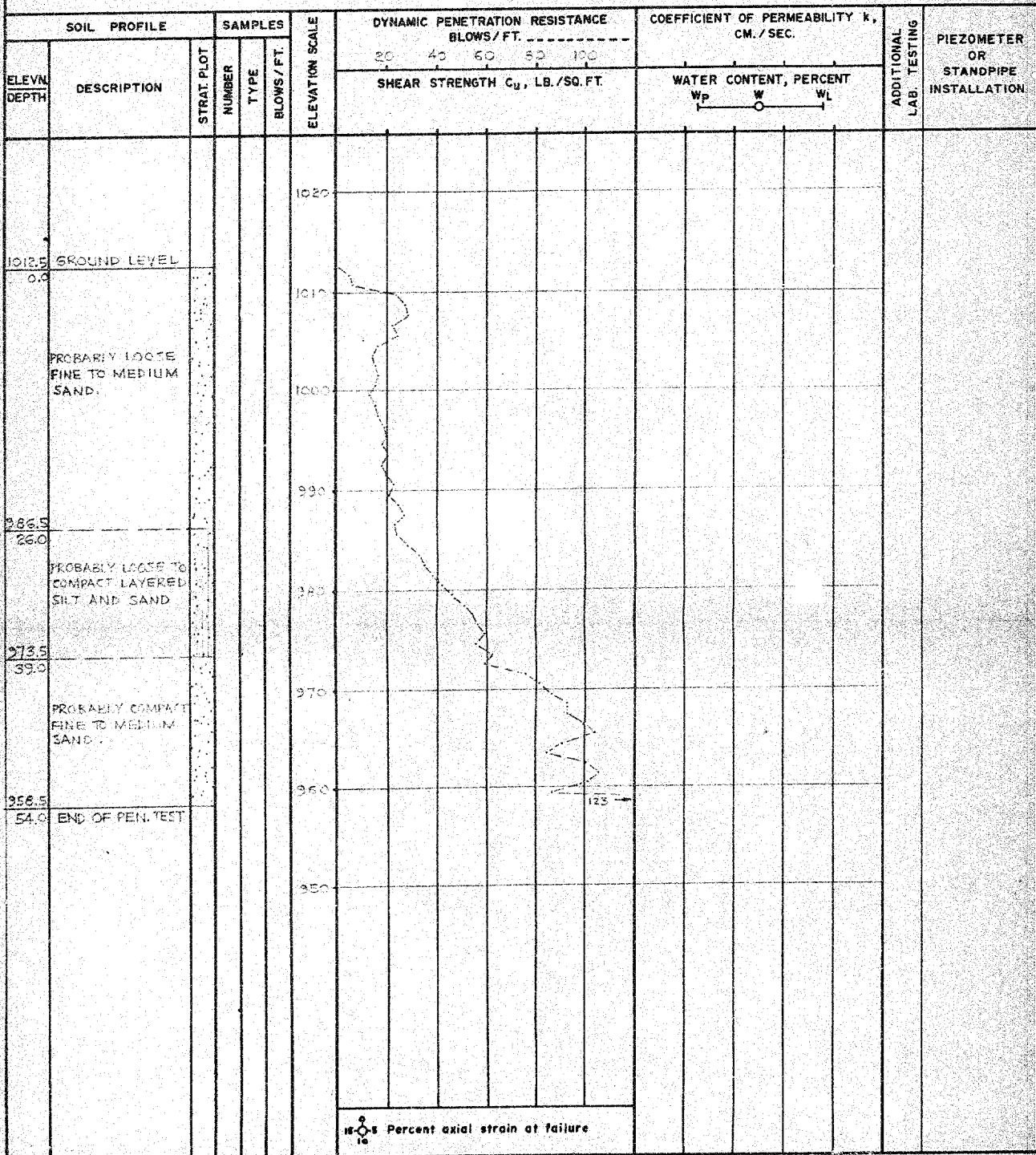
DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER —

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



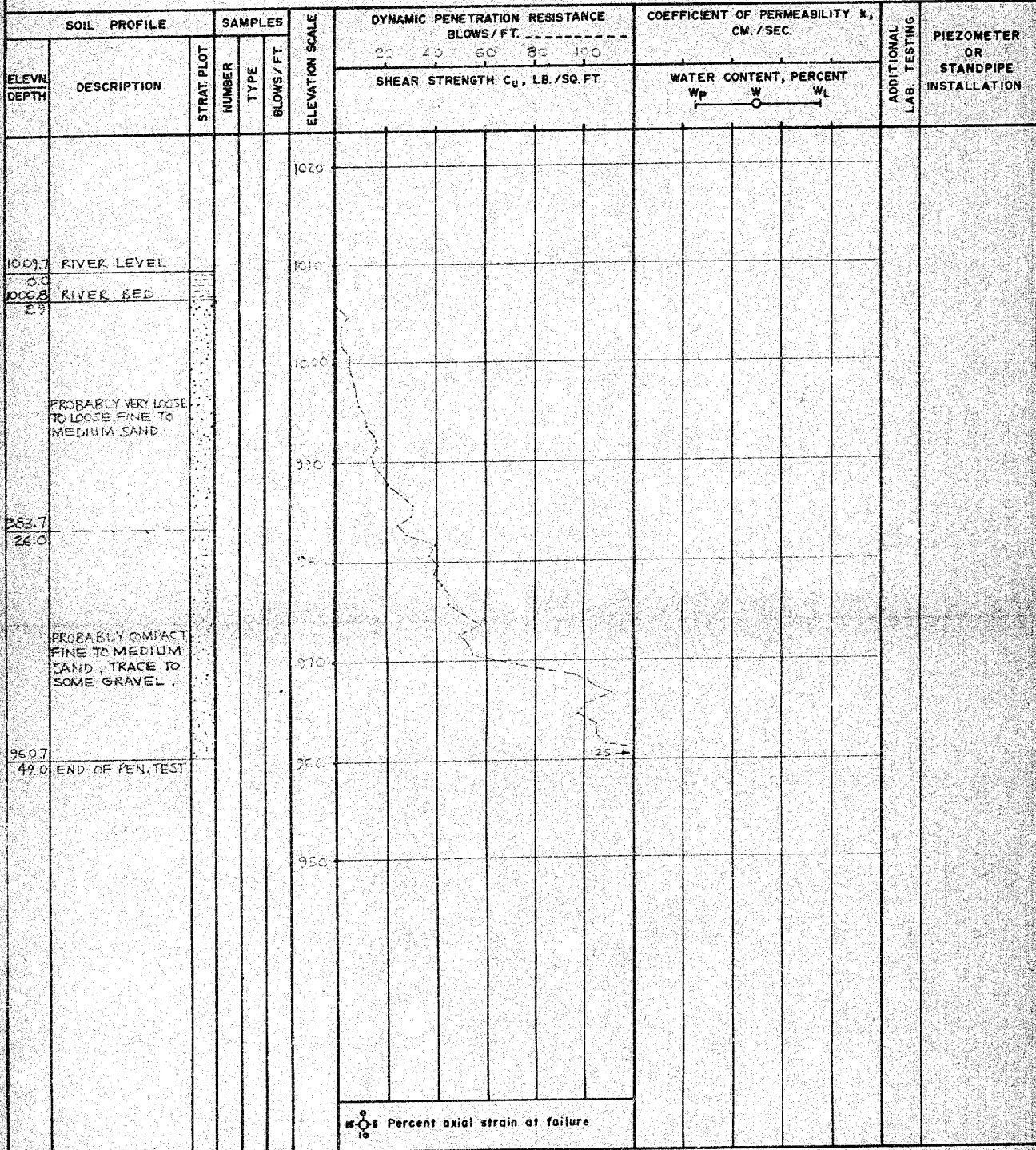
VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *mw*
CHECKED

RECORD OF PENETRATION TEST 7

LOCATION See Figure 1 BORING DATE JUNE 10, 1966 DATUM GEODETIC
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



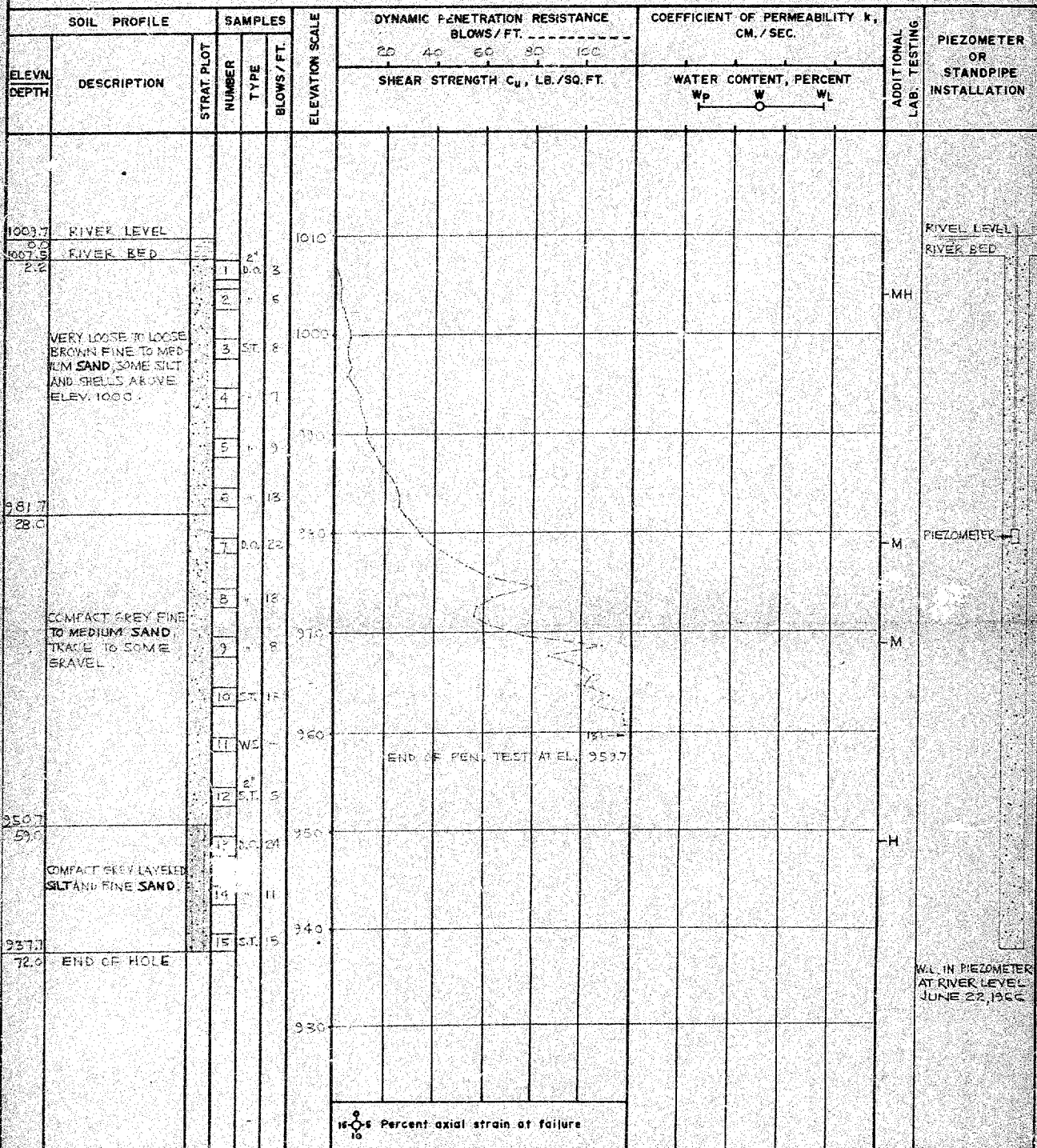
VERTICAL SCALE
 1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN MW
 CHECKED

RECORD OF BOREHOLE 8

LOCATION See Figure 1 BORING DATE JUNE 10-15, 1966 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, SX CASING
 SAMPLE HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



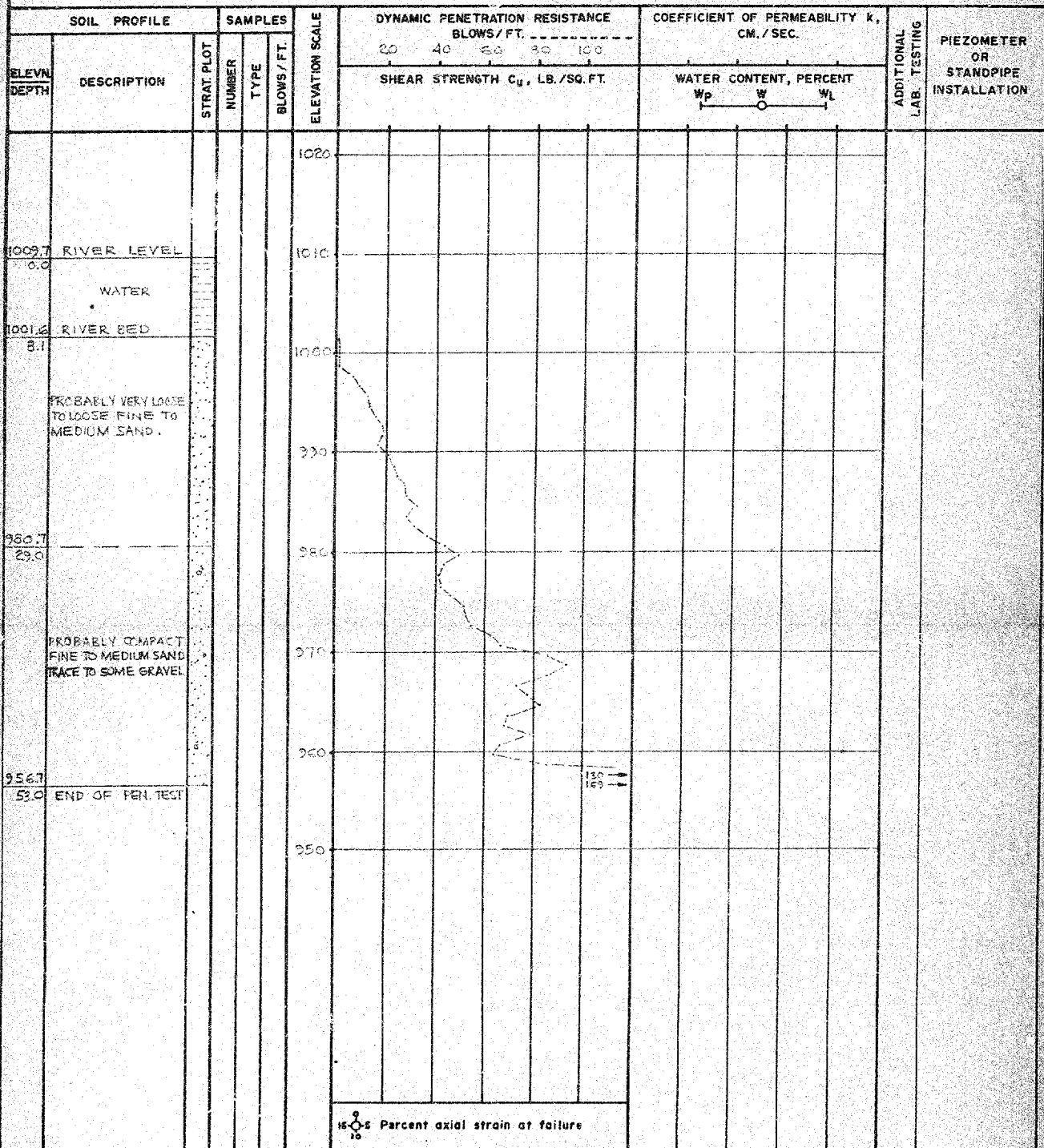
VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *[Signature]*
CHECKED

RECORD OF PENETRATION TEST 3

LOCATION See Figure 1 BORING DATE JUNE 16, 1966 DATUM GEODETIC
BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —
SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



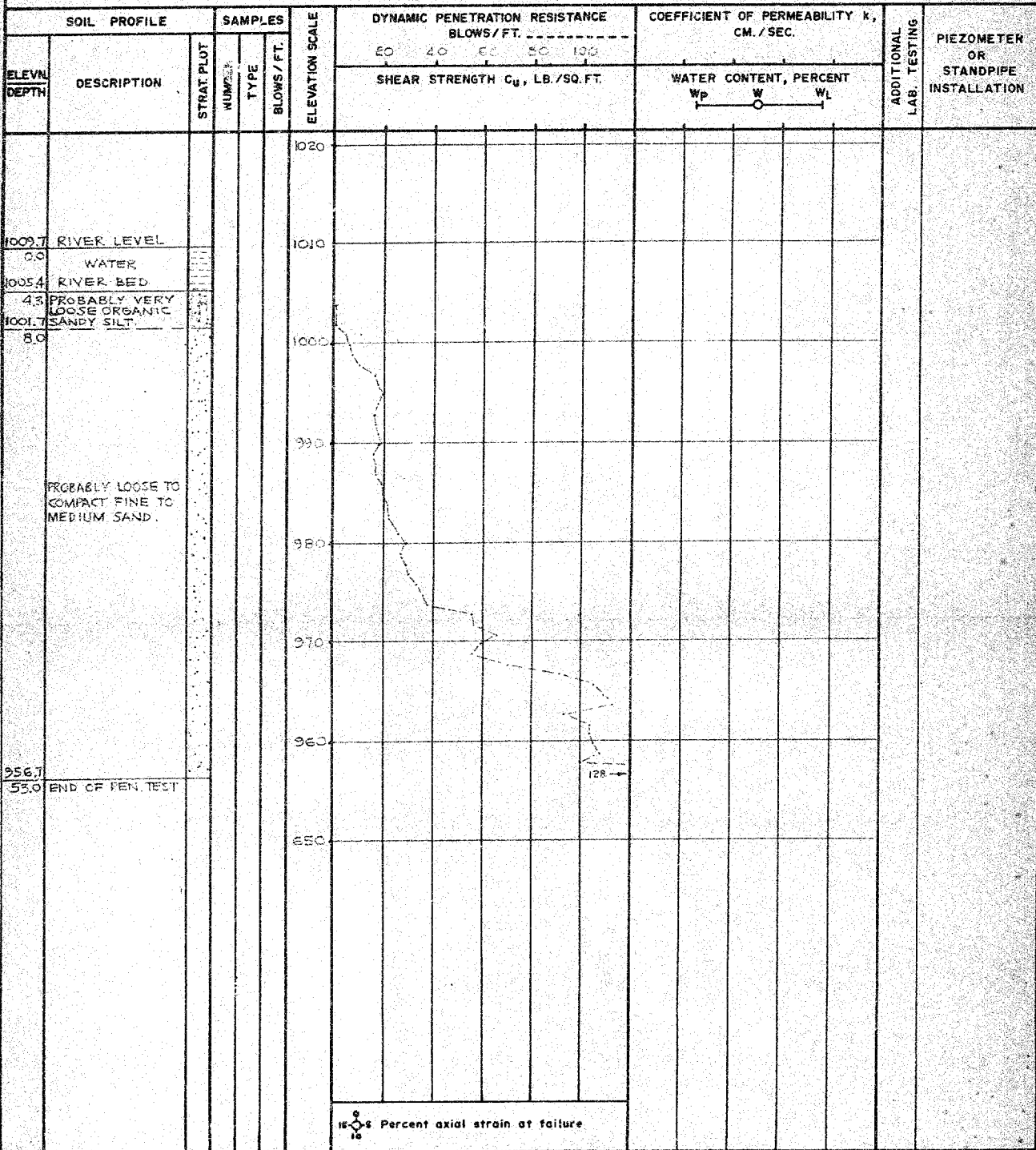
VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *[Signature]*
CHECKED

RECORD OF PENETRATION TEST 10

LOCATION See Figure 1 BORING DATE JUNE 16, 1966 DATUM GEODETTIC
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



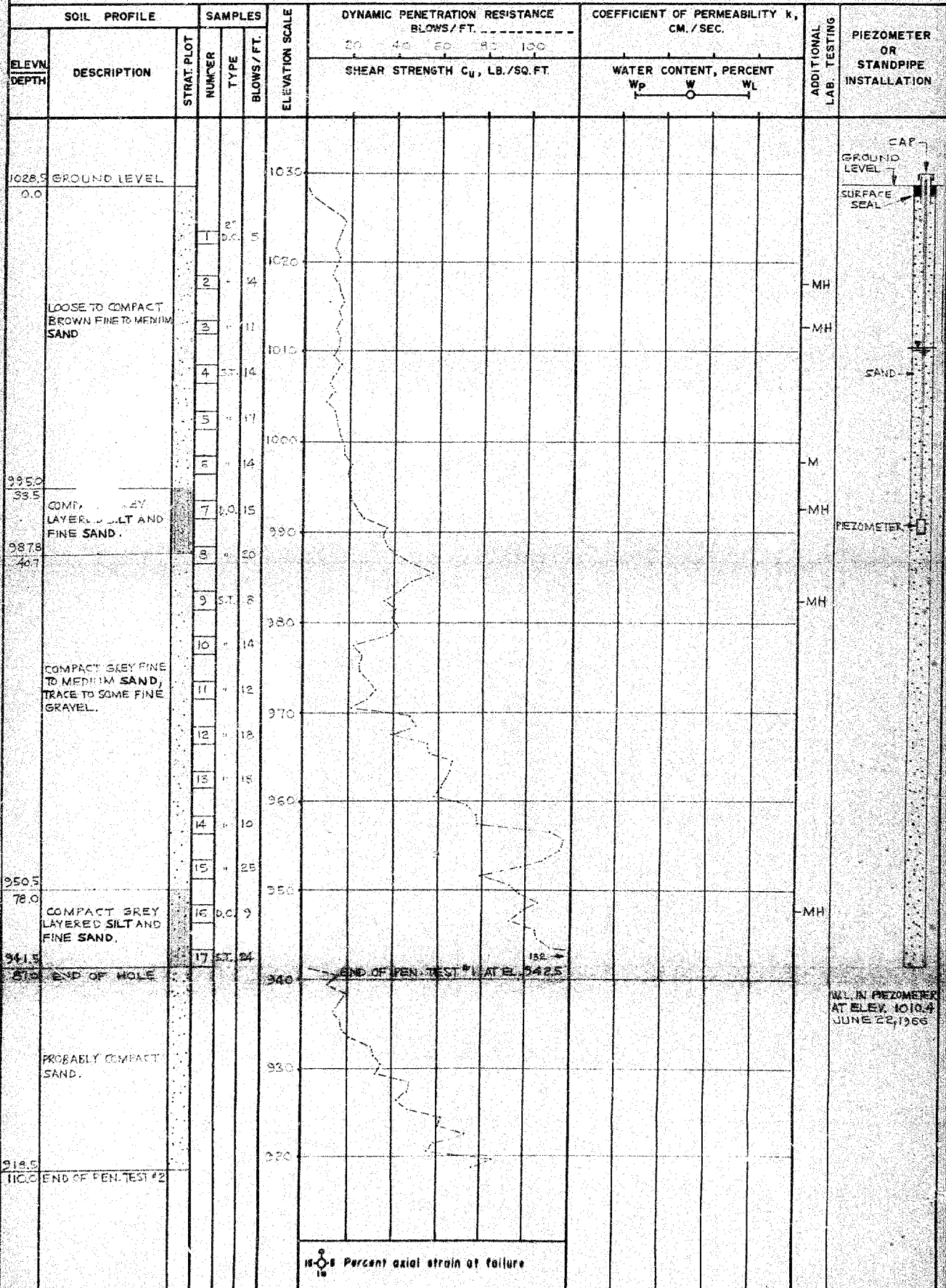
VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *[Signature]*
CHECKED

RECORD OF BOREHOLE 12

LOCATION See Figure 1 BORING DATE JUNE 18-21, 1966 DATUM GEODETIC
BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER EX. NX 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 & ASSOCIATES

DRAWN *mb*
CHECKED

RECORD OF PENETRATION TEST 13

LOCATION See Figure 1

BORING DATE JUNE 22, 1965

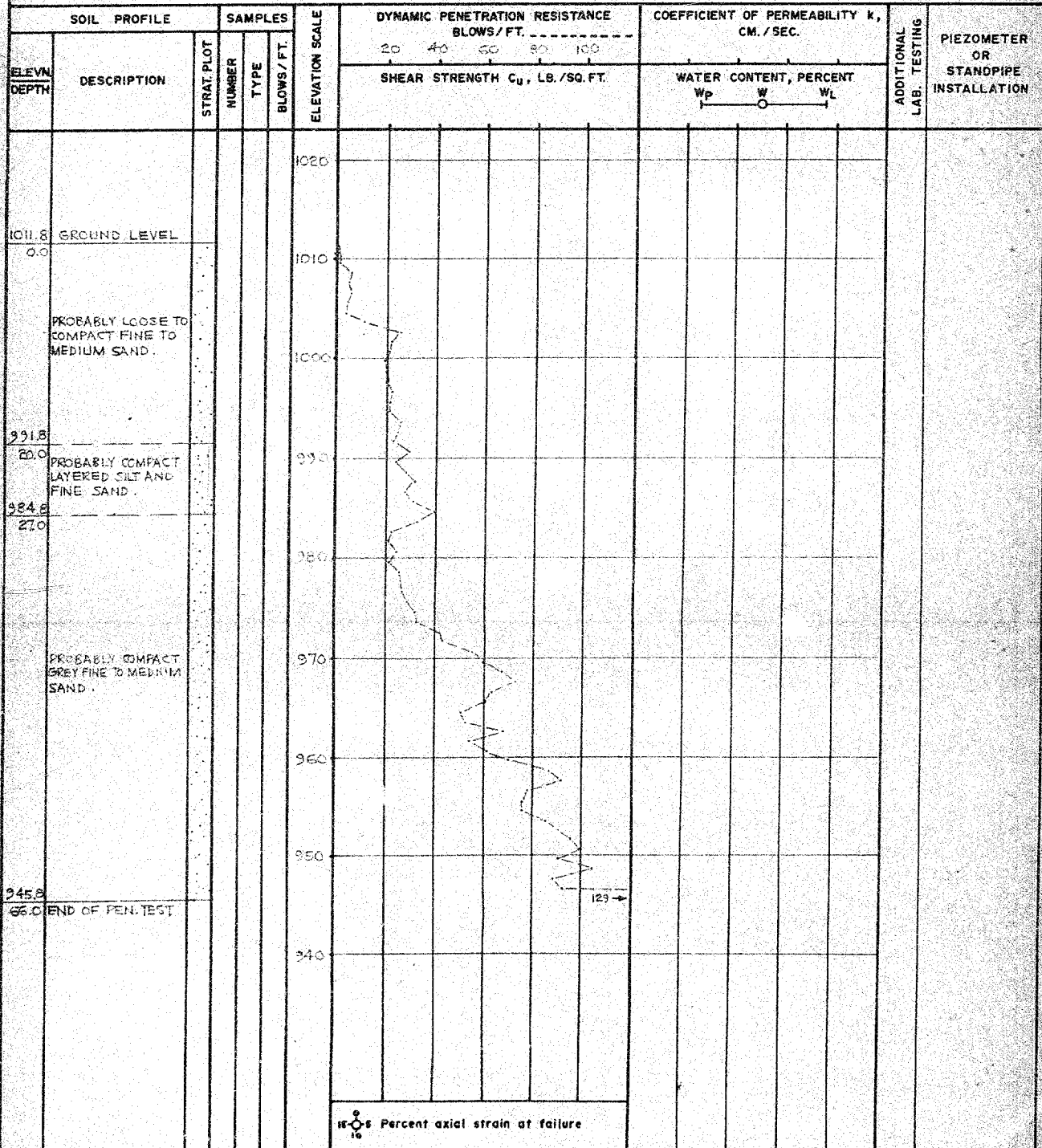
DATUM GEODETTIC

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER —

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

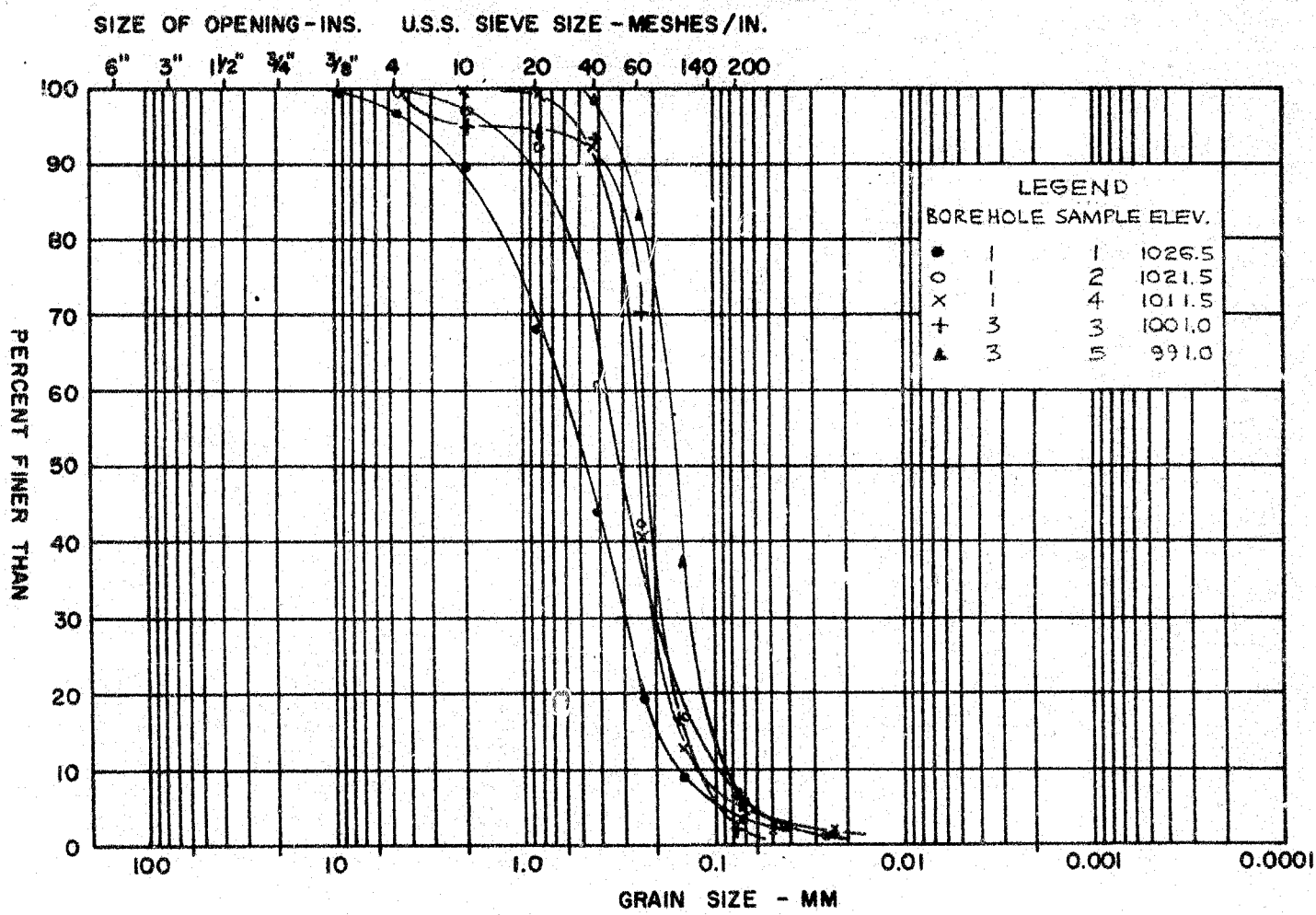
PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *asa*
CHECKED

M.I.T. GRAIN SIZE SCALE



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

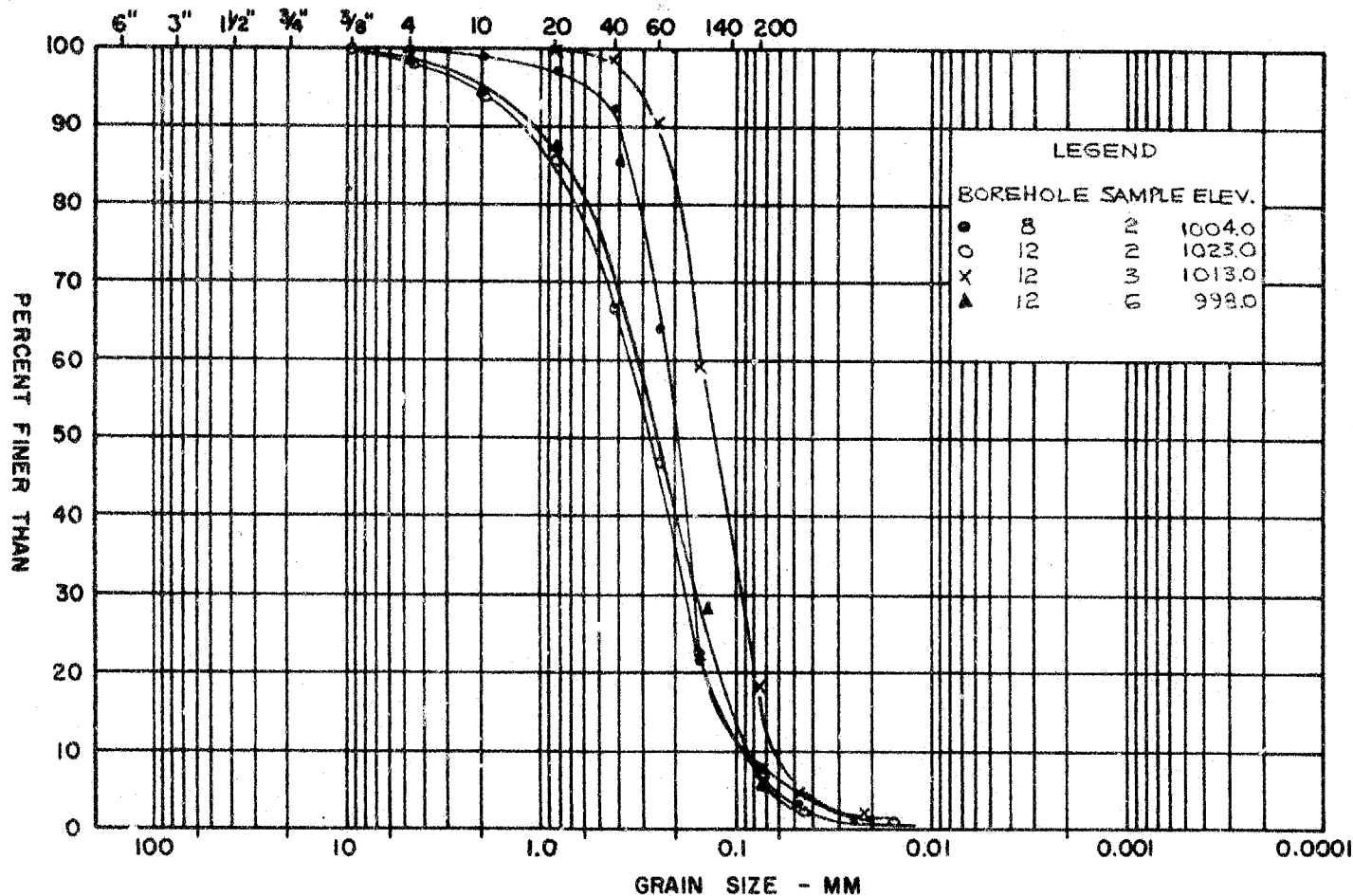
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
FINE TO MEDIUM SAND

FIGURE 3

M.I.T. GRAIN SIZE SCALE

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



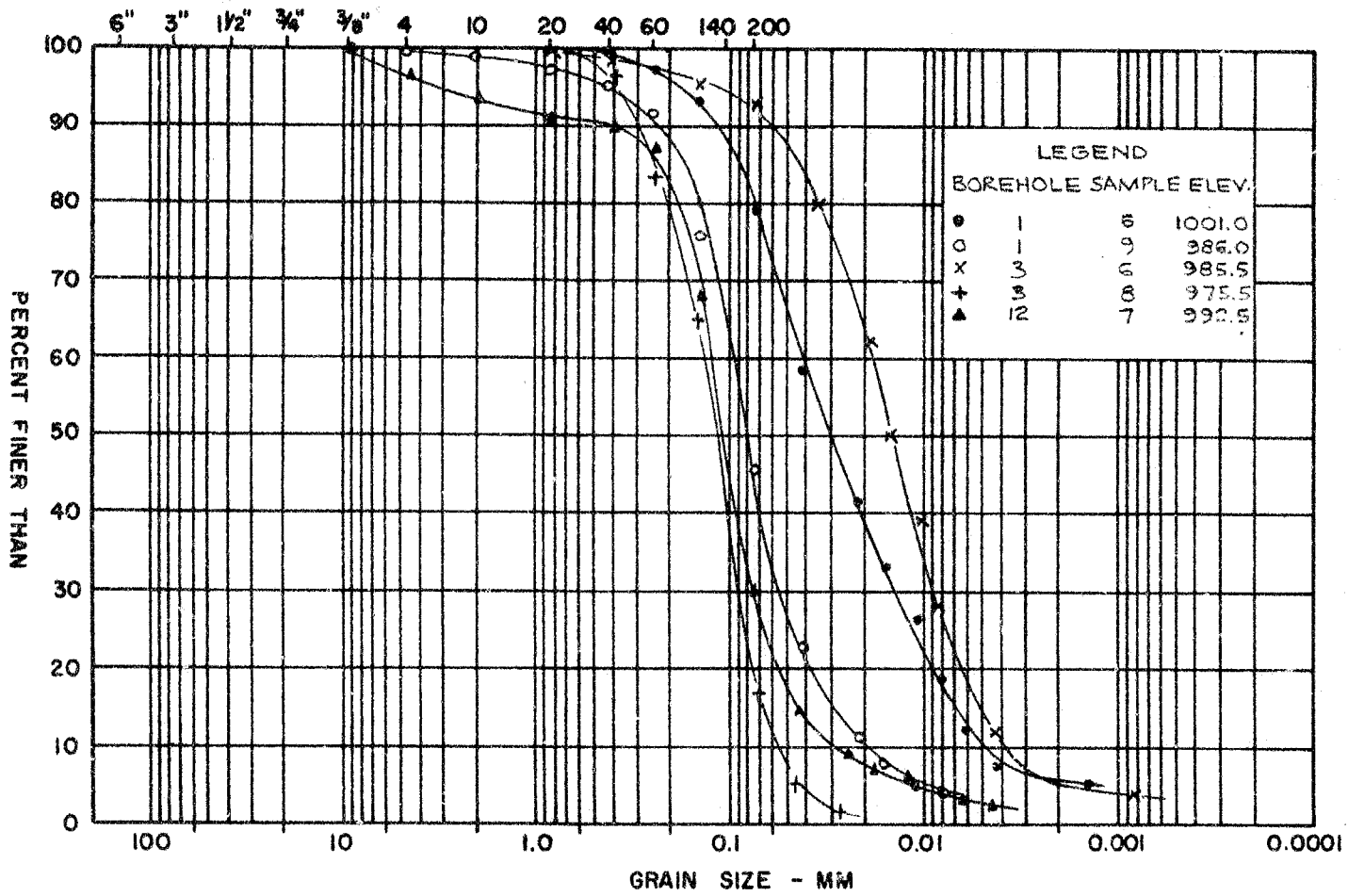
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
FINE TO MEDIUM SAND

FIGURE 4

M.I.T. GRAIN SIZE SCALE

SIZE OF OPENING - INS. U.S.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			

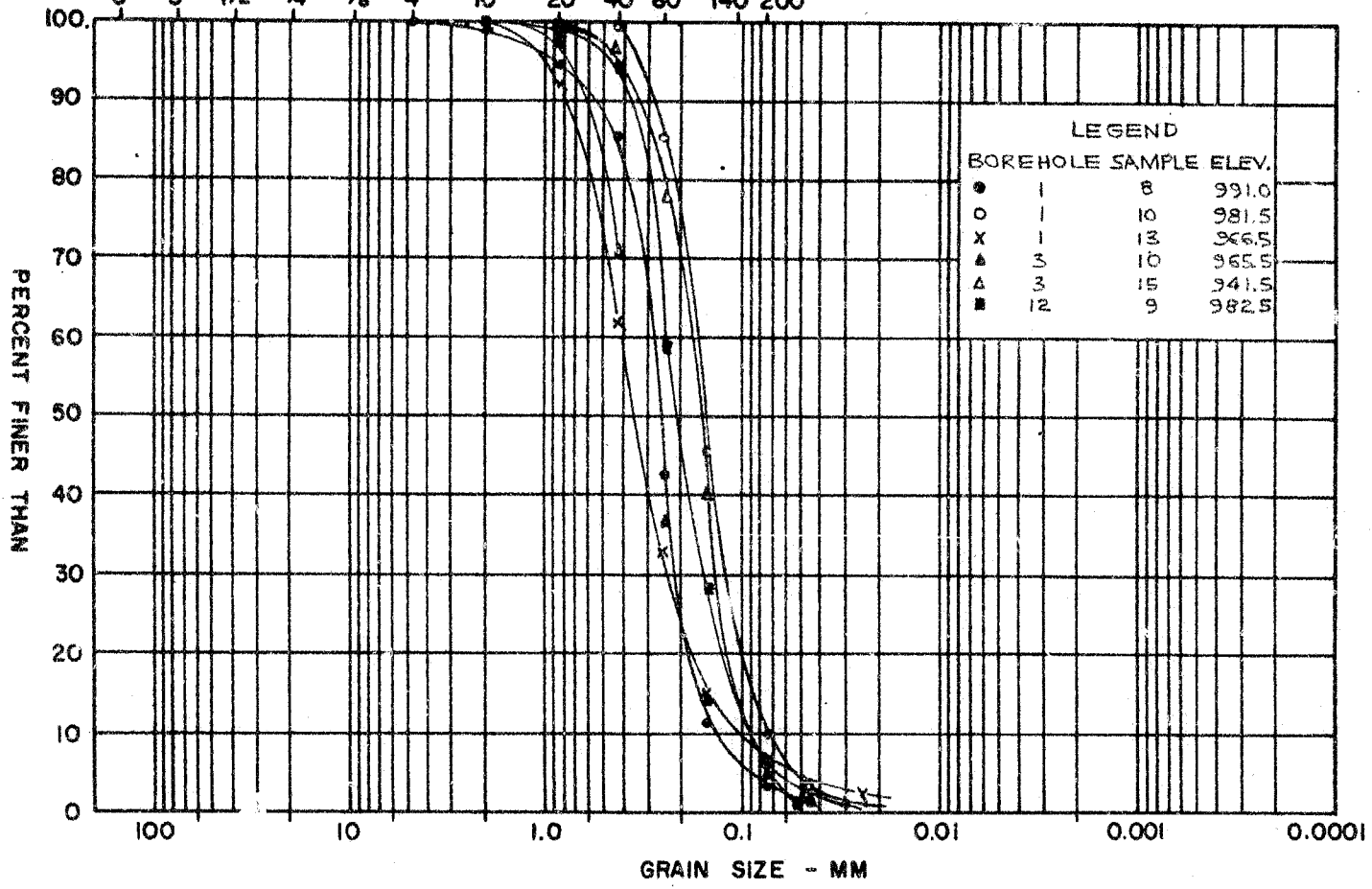
GRAIN SIZE DISTRIBUTION
LAYERED SILT AND FINE SAND

FIGURE 5

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 60 140 200



GOLDER & ASSOCIATES

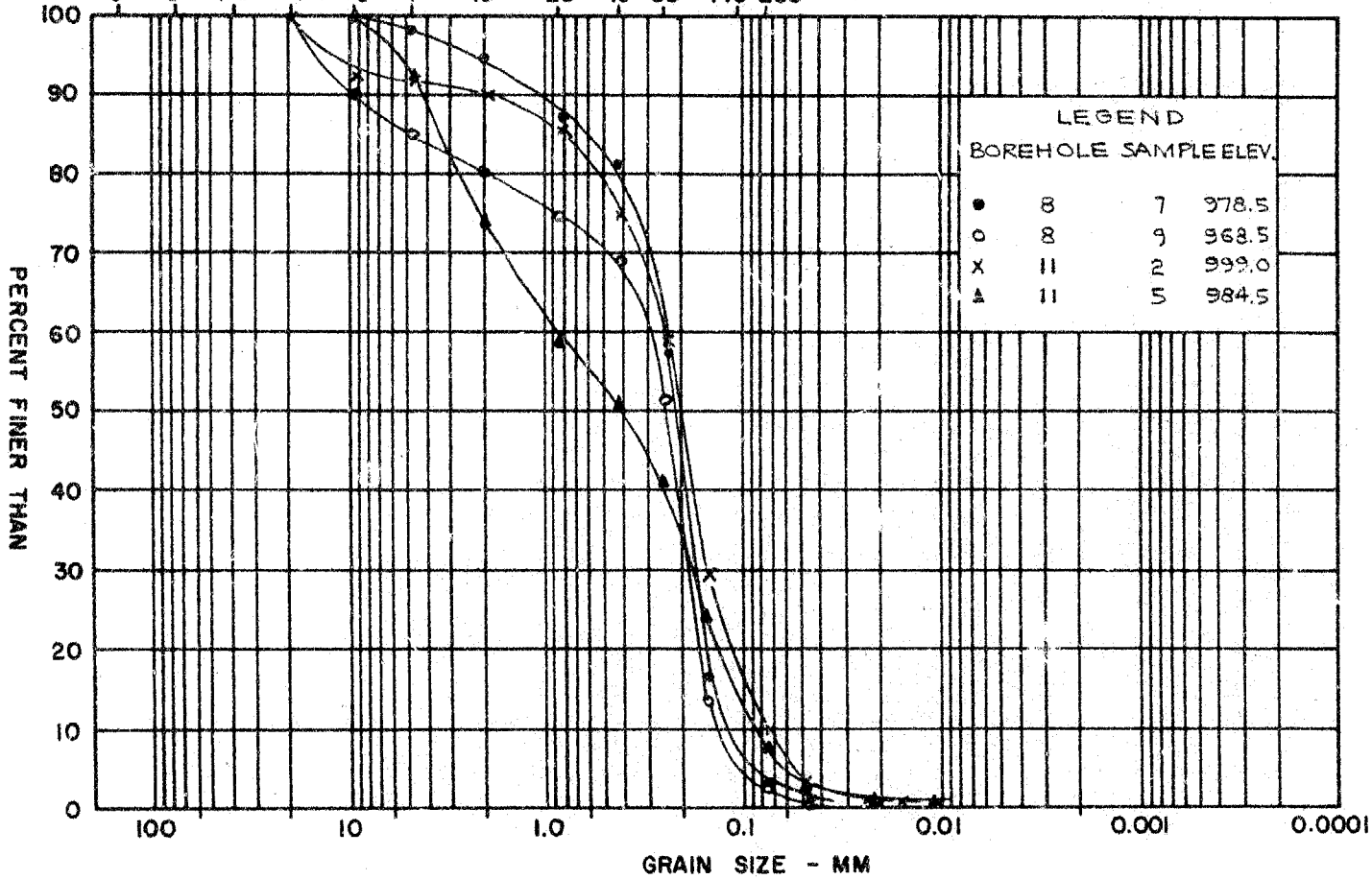
GRAIN SIZE DISTRIBUTION
FINE TO MEDIUM SAND

FIGURE 6

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 60 140 200



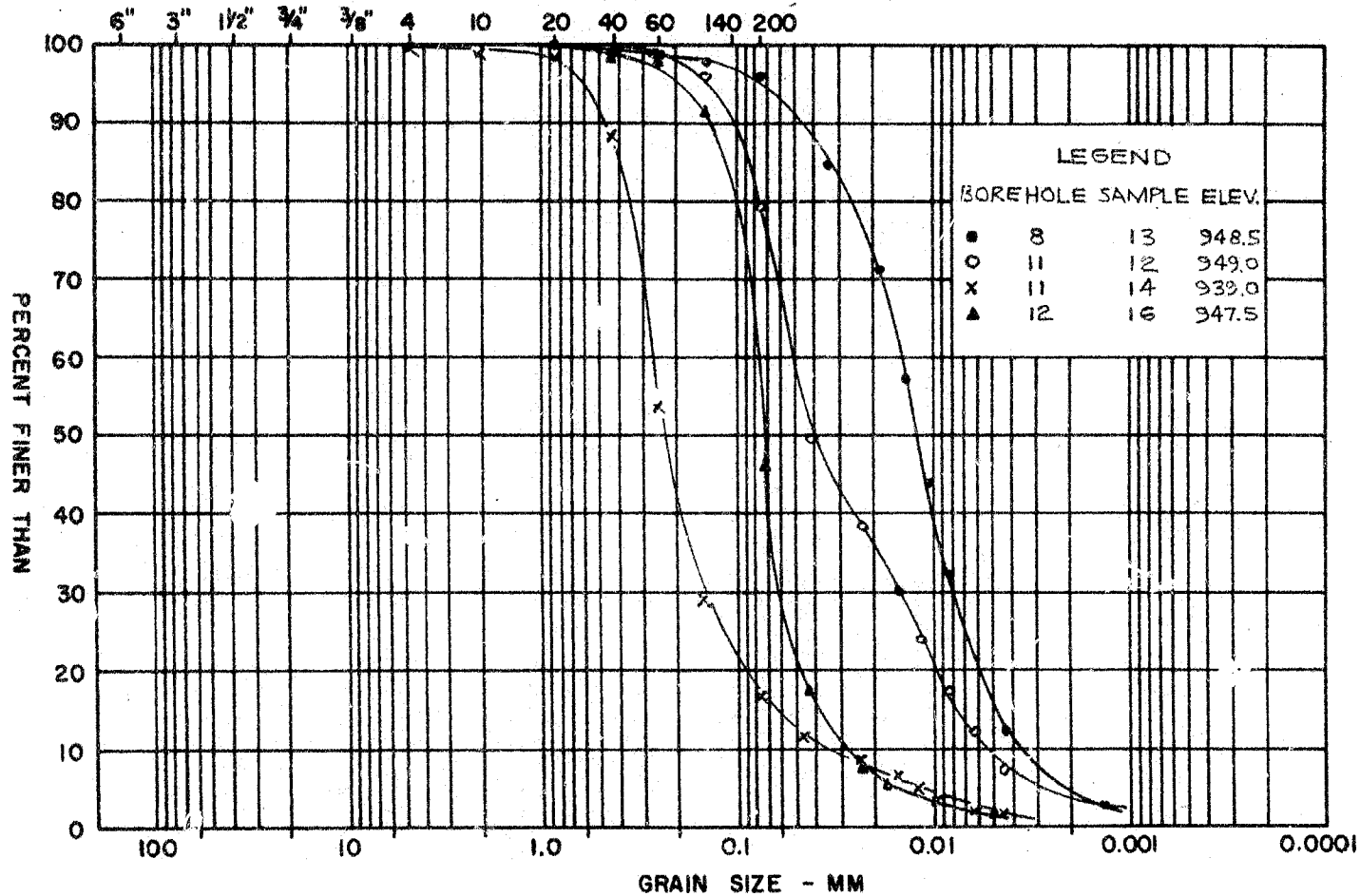
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
FINE TO COARSE SAND

FIGURE 7

M.I.T. GRAIN SIZE SCALE

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



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

GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
LAYERED SILT AND FINE SAND (LOWER DEPOSITS)

FIGURE

B

66-F-223-C

W.P. # 178-64

HWY. # 35 &

BEECH RIVER

BRIDGE

