

63-F-271 M

COUNTY ROAD #6

MCADAM BRIDGE

LOTS 10/11. CONIX.

WEST WILLIAMS

TWP.

Plans returned to Bridge Office on request

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

TO: Mr. A. Stermac
Principal Foundations Eng.,
Materials & Research Section,

FROM: G.C.E. Burkhardt

DATE: Bridge Division,
March 15, 1963.

OUR FILE REF. BA 1606

IN REPLY TO

SUBJECT: County of Middlesex,
McAdam Bridge,
County Road #6,
Township of West Williams,
Lots 10/11, Con IX,
Structure Site #20-89,

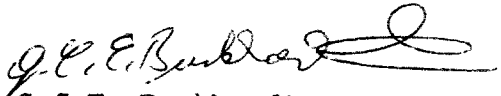
Attached please find one copy of the Foundation Report, by Golder and Associates, and one copy of the Preliminary Plans for above mentioned structure.

We would appreciate it very much if we could have your comments especially in regard to the type of footings proposed including construction problems.

We hope to approve of the plans at the beginning of April 1963 and would appreciate it very much, if we could have your comments within the next two weeks.

Since we don't have enough copies of the plans, we would like to have the plans back, which we are forwarding to you to-day.

GCEB/dm


G.C.E. Burkhardt,
for K.L. Kleinstelber,
Municipal Bridge Liaison Engineer.

*No concrete- bored caissons are feasible and
contractor properly equipped for such job should
have no difficulty.*

*For phone to G.C.E.B.
March 19, 1963.*

A. J. Thompson,

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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

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

REPORT

TO

R. C. DUNN & ASSOCIATES LIMITED

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED COUNTY BRIDGE NO. 30
(McAdam Bridge)

COUNTY OF MIDDLESEX, ONTARIO

Distribution:

10 copies - R. C. Dunn & Associates Limited,
London, Ontario.

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

February, 1963

6270

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ABSTRACT

The results of an investigation carried out at the site of the proposed Middlesex County Bridge No. 30, McAdam Bridge, over the Ausable River in West Williams Township, Ontario, are reported.

It was found that the floodplain on the south side of the river is covered by an alluvial deposit of very loose to loose sandy to clayey silt between about 7 to 17 feet thick. The north bank of the river, which is some 40 feet higher than the south bank, is covered by about 10 feet of loose to compact silt of interglacial origin followed by a stratum of stiff lacustrine layered clayey silt and silty clay about 36 feet thick. The lacustrine deposit on the north bank and the alluvial floodplain deposit on the south side are underlain by a generally very stiff stratum of silty clay till which is some 20 feet thick at the river channel increasing to between 40 and 50 feet in thickness at the river banks. The clay till below about elevation 105 is underlain by a 30 foot thick lacustrine deposit of stiff to very stiff layered silty clay with silt and sand seams up to about 6 inches thick. This lacustrine clay is in turn underlain by about 17 feet of hard clayey till and 20 feet of very stiff lacustrine clay followed by hard clayey till.

Readings taken in piezometers installed in the clay strata showed that the piezometric water level in the subsoil was some 20 feet above river level during the investigation and within about 5 feet of the reported river high water level. A perched water level was observed in the floodplain deposit overlying the upper clay till.

Recommendations are made to support the piers and abutments of the proposed 3 span continuous bridge on bored cast in place concrete caissons and bearing in the very stiff upper clay till between elevations 115 and 120, using an allowable bearing pressure of 5 tons per square foot.

To provide long term stability and minimize horizontal creep movements, the north bank of the river should be trimmed to a slope of 3 horizontal to 1 vertical and protected by a granular blanket as discussed in the report.

In order to ensure the stability of the approach embankment on the south side of the river, it is recommended that the floodplain material overlying the clayey till be removed prior to placement of the fill in the abutment area.

INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by R.C. Dunn & Associates Limited, Consulting Engineers to the County of Middlesex, to carry out an investigation at the site of proposed County Bridge No. 30 (McAdam Bridge) on County Road No. 6 between Lots 10 and 11, Concession IX in West Williams Township. The purpose of this investigation was to determine the subsoil conditions at the site and to interpret these conditions in relation to foundation design of the proposed structure and the stability of approaches to the bridge on the banks of the Ausable River.

PROCEDURE

The field work for this investigation was carried out during the period December 5, 1962 to January 12, 1963. A total of 6 boreholes and 3 dynamic penetration tests were put down using a skid-mounted machine drillrig supplied and operated by F. E. Johnston Drilling Company Ltd. The borings were advanced in NX and BX casing size to an average depth of about 70 feet. Samples of the subsoil were taken at intervals of depth not exceeding 5 feet. After completion of each deep borehole a piezometer was installed to determine the piezometric groundwater level.

The locations of the borings put down in this investigation are shown on the site plan in Figure 1. A section of the inferred soil stratigraphy across the site is given on Figure 2. Detailed logs for each borehole put down are given on the Records of Boreholes at the end of the report.

The samples obtained during the investigation were brought to our laboratory for detailed examination and testing. The results of the laboratory testing are shown on the Records of Boreholes and on Figures 3 to 14, inclusive.

The elevations in this report are referred to local datum and were determined by reference to a bench mark located on a tree about 350 feet south of the river at the crossing and 30 feet west of the road allowance centreline. The elevation of this bench mark was set at 166.72 by the County of Middlesex. All the survey work connected with this investigation was carried out by R.C. Dunn & Associates Ltd.

SITE TOPOGRAPHY AND GEOLOGY

The site is located on County Road No. 6 where it crosses the Ausable River in the Township of West Williams some 10 miles south of Parkhill, Ontario. The general area is characterized by gently rolling topography. The river at this location flows in a channel about 90 feet wide with a floodplain about 150 feet wide on the south side. The floodplain rises to a height of about 35 feet above creek bed level to form the south bank. The north bank of the river which rises some 75 feet above the creek bed is steep.

From available information and previous work in this locality, it is known that the general stratigraphy in this area consists of a complex succession of glacial clay tills and interglacial or lacustrine deposits of clays, silts and sands of the

Pleistocene period extending to bedrock. The bedrock, which is a Norfolk sedimentary formation of the Devonian Period, is more than 100 feet below general ground surface at the site.

SOIL CONDITIONS

The sequence of soil strata encountered by the borings put down at the site conforms to the general geological pattern discussed above. The detailed stratigraphy encountered in each borehole is given on the Records of Boreholes at the rear of this report. A section of the stratigraphy across the site inferred from this data is presented on Figure 2. Following is an account of the soil conditions at the site.

A geologically recent alluvial deposit forms the floodplain on the south side of the river. The deposit generally increases in thickness from about 7 feet near the top of the bank to about 17 feet near the river. It is comprised essentially of sandy silt with a trace of clay and organic matter but varies from a clayey silt to silty medium sand. The organic content occurs mainly in the form of small decomposed woody chunks. The upper portion of the north bank of the river is covered by about 10 feet of material similar in composition to the floodplain deposit but of different geological origin, probably lacustrine. The lower portion of the recent deposit on the south bank of the river at borehole 2 is similarly probably of lacustrine or lake deposit origin and not floodplain material throughout.

Grain size distribution curves obtained on samples from the recent alluvium are given on Figure 3. Organic content determinations on three samples from borehole 3 gave values between 7 and 10 percent. Based on the results of standard penetration tests which gave "N" values ranging between 2 and about 10 blows per foot, together with the results of the dynamic penetration tests given on the Records of Boreholes, the alluvial deposit is generally very loose to loose.

The silt forming the upper deposit on the north bank of the river is underlain by a stratum of layered clayey silt and silt which grades down into a layered silty clay and clayey silt with depth. This stratum is considered to be of interglacial origin and is referred to as upper lacustrine clay in this report. The layered silt and clay which occurs only at this location on the site is about 36 feet thick and extends down to about elevation 152. The layering in the stratum is generally indistinct to visual examination and where it is evident the individual clayey silt and silty clay layers are up to several inches in thickness and form no regular pattern. The stratum contains occasional fine gravel sizes of sedimentary rock origin throughout and thin fine sand seams which generally increase to several inches in thickness with depth.

The upper lacustrine clay on the north bank of the river and the alluvial floodplain deposit on the south side are underlain by a stratum of dark grey silty clay, a glacial till deposit. This stratum which underlies the whole site under

consideration extends down to about elevation 105 and is about 20 feet thick at the river channel increasing to between 40 and 50 feet at the banks. The stratum is in general homogeneous in structure and is comprised of silty clay with a trace of sand and fine gravel dispersed throughout, as shown by the grading curves on Figure 4. The silty clay, referred to as upper clay till in this report, contains occasional small pockets and thin layers of fine sand and clayey silt throughout. The silt and sand layers generally predominate below about elevation 115.

A stratum of lacustrine clay about 30 feet thick underlies the upper clay till across the site. The stratum is comprised of brownish grey and dark grey silty clay layers of varying thickness, which represent a minor variance in grain size, and contains small pockets of grey silt, thin clayey silt and fine sand seams throughout. A grading curve obtained on a sample from the stratum is given on Figure 5. The stratum, below about elevation 90, grades into essentially a clayey silt with silty clay seams and fine sand seams about 6 inches thick.

The lacustrine clay is in turn underlain by a stratum of dark grey silty clay till, referred to as the middle till in this report. The till stratum in borehole 4, where it was completely penetrated, is 17 feet thick and extends down to about elevation 60. The middle clay till is similar in composition to the upper till but is more homogeneous in structure and does not contain any visible seams or lenses of sand or silt.

A lower lacustrine clay deposit, about 19 feet thick and extending down to about elevation 40, underlies the middle till stratum in borehole 4. The lacustrine deposit is comprised of layers of brownish grey and dark grey silty clay. The individual layers, which are variable in thickness, represent mainly a colour change and a minor variance in gradation. Small pockets and thin seams of grey silt together with occasional fine sand seams and fine gravel sizes are dispersed throughout the stratum.

The lower lacustrine clay, as the upper and middle lacustrine clay deposits, is underlain by glacial till. The till was penetrated for a depth of about 12 feet in borehole 4 where refusal to wash boring on a large boulder or bedrock was encountered at about elevation 29. The basal or lower till is comprised of homogeneous silty clay to clayey silt, with a trace of sand and occasional fine gravel in the upper portion of the stratum. The sand and gravel content increases with depth to form about 50 per cent of the stratum near the base.

Atterberg limit tests were carried out on samples of the clay strata discussed above and the results for individual boreholes are presented on the Records of Boreholes. A plot of all the Atterberg limit and natural water content test results versus elevation is given on Figure 6. Reference to this summary chart shows that, with the exception of the upper lacustrine and lower till deposits which contain variable proportions of sand and silt, the plasticity of the lacustrine and till groups is relatively uniform.

The liquid limit for the upper and middle tills is about 30 and the plasticity index about 15. The liquidity index, which is the ratio of natural water content minus the plastic limit to plasticity index, is less than 0.5 and generally about 0.2 for the upper till and near zero for the middle till. For the middle and lower lacustrine clay deposits a liquid limit between about 35 and 45 and a plasticity index of 15 to 20 were obtained. In the lower clayey silt portion of the middle lacustrine stratum the liquid limit is as low as about 25. The average liquidity index is about 0.3 for the middle and about 0.2 for the lower lacustrine deposits.

A number of undrained triaxial compression tests were carried out on samples of the clay strata. The results of these tests are presented on the borehole logs and on Figure 7 which gives a plot of undrained shear strength versus elevation. Typical stress-strain curves from these tests are given on Figure 8 to 12, inclusive.

The undrained shear strength measured for the upper lacustrine clay ranges between about 1,000 and 2,000 pounds per square foot. The range for the middle lacustrine deposit is between about 1,000 and 3,000 pounds per square foot with an average value of about 2,000 pounds per square foot. The shear strength as measured for the lower lacustrine clay is generally in excess of 2,000 pounds per square foot.

The range in shear strength for the upper clay till, which is the significant stratum for foundation design at the site, is generally between about 2,000 and 4,000 pounds per square foot

with an average value of about 3,000 pounds per square foot. Several values as low as 1,500 pounds per square foot were obtained in this stratum.

Based on the results of the strength tests summarized on Figure 7, together with the standard penetration resistance values given on the Records of Boreholes, the upper lacustrine clay deposit is generally stiff, the upper till and middle lacustrine deposits range between stiff and very stiff, the lower lacustrine clay is very stiff and the middle and lower tills are hard.

A plot of unit weight values obtained on samples of the clay strata is given on Figure 13. The results show that the total unit weight for the till deposits is between about 135 and 140 pounds per cubic foot and between about 125 and 130 pounds per cubic foot for the lacustrine deposits.

The results of two consolidated drained direct shear tests carried out on samples of the upper lacustrine clay are plotted on Figure 14. The failure envelope gives a drained angle of shearing resistance, ϕ_d , of about 25 degrees with a negligible cohesion intercept, c_d , on the envelope.

WATER CONDITIONS

A porous pot piezometer was installed in each of the 4 boreholes which penetrated into or through the middle lacustrine clay to determine the piezometric groundwater level within the clay strata underlying the site. The piezometer installation details and the stabilized reading obtained in the piezometers at the con-

clusion of the field work are given on the Records of Boreholes.

The readings taken show that the piezometric water level was some 8 feet above ground surface and at about elevation 152 in the boreholes near the river. The level in the boreholes on top of the river banks was at about elevation 156 to 158. This is some 20 feet above the river level during the period of the investigation and within about 5 feet of the reported river high water level at elevation 160. This apparent artesian effect is probably produced by the presence of permeable sand seams within the lacustrine clay strata.

A perched water level was observed in the surface alluvial deposit overlying the relatively impervious clay till on the south side of the river. The perched level was found to slope down to river level from about elevation 158 on top of the bank at borehole 2. As the relatively permeable recent alluvium is in direct communication with the river water, the perched water level may be expected to fluctuate with the river level.

DISCUSSION

General

It is understood that a new bridge on the County Road No. 6 crossing over the Ausable river is to be constructed at the location shown on Figure 1. The proposed bridge, to be known as the McAdam Bridge, is to be a three span continuous reinforced concrete structure. The centre span is to be about 150 feet in length with the end spans each about 110 feet long. The approximate locations of the piers and abutments are shown in section on

Figure 2. The bridge is to be about 45 feet wide and the loading of each pier including dead and live loads is to be approximately 1,200 tons. The total abutment loading will be about 500 tons.

The roadway grade at the north abutment of the bridge will be at about elevation 185 and at about elevation 180 at the south abutment. To accommodate the approach grade a cut of about 15 feet will be required on the north bank of the river and about 20 feet of fill will have to be placed on the south bank.

Bridge Foundations

The significant stratum at the site for the support of the bridge piers and abutments is the upper clay till immediately underlying the floodplain deposit on the south side of the river and the stiff upper lacustrine clay on the north bank. The upper till is generally very stiff and extends down to a depth of about 20 feet below river bed level. Reference to the undrained shear strength profile on Figure 7 shows that the lacustrine clay underlying the upper till is softer than the till. Further, the middle lacustrine clay contains sand seams under artesian pressure, as discussed under a previous section of this report. Based on the above considerations it is concluded that for economical design, the foundations of the bridge structure should be placed within the upper till, as no advantage would be gained by penetrating into the underlying middle lacustrine deposit.

Consideration has been given to the use of spread footings for the support of the bridge piers and abutments. Spread

footings would have to be placed at about elevation 140 and 150 at the south and north abutment locations, respectively. For river scour protection the footings at the pier locations should be taken down to about elevation 120. Taking a shear strength value of 2,000 pounds per square foot for the upper till above elevation 120, a net allowable bearing value of 2 tons per square foot could be used for design of the spread footings.

The use of spread footings at the site would necessitate some 20 to 30 feet of excavation below existing ground surface at the pier and abutment locations. The excavations would have to be sheeted and strutted, particularly on the south side of the river to retain the relatively pervious floodplain material and prevent entry of river water into the excavation. Because of the deep excavation required during the construction of spread footings, we consider that economically piles or caissons would give a more suitable foundation.

To avoid possible difficulties due to artesian water in sand seams and lenses which predominate within the middle lacustrine clay, piles used to support the bridge piers and abutments should not penetrate into this deposit. Further, to minimize the effects of consolidation settlement in the softer lacustrine clay underlying the till, the base of the piles or caissons should be placed in the till stratum as high as possible above the lacustrine clay.

Various pile types could be used. Taking for example, 12 inch diameter steel pipe piles or H piles of equivalent size

driven to elevation 110 at the south pier location, the allowable load per pile is computed to be about 25 tons. This allowable load is based on an ultimate effective shaft adhesion of 1,500 pounds per square foot for the upper till and taking a factor of safety, $F=3$ on the computed ultimate load. For such piles the end bearing effect is small. Consequently, to take advantage of the relatively high strength of the upper till deposit, we recommend that large diameter bored cast in place concrete caissons be used to support the bridge piers and abutments.

The caissons should be taken to about elevation 115 at the pier locations and to elevation 120 at the abutment locations. Taking an average shear strength of 3,000 pounds per square foot for the till at the above elevations, an allowable end bearing capacity of 5 tons per square foot is computed for design. Considering 10 caisson units at a pier location, a base diameter of about 5.5 feet is required for each unit. If a 30 inch diameter caisson shaft is used, the required base diameter could be provided by bellling out the lower portion of each caisson. No allowance of caisson shaft adhesion or friction should be made in the design of end bearing caissons.

It is recommended that the base of each bored caisson be examined to ensure that no slough material or disturbed till is present prior to placement of the concrete. To prevent sloughing of particularly the deposits overlying the till, casing of the bored hole will generally be required to the top of each belled out base.

The settlement of the bridge piers founded on end bearing caissons as discussed above, due to consolidation in the underlying clay strata, is estimated to be about 2 inches. This estimate is based on experience with overconsolidated clays of similar plasticity, liquidity and strength. A compression index, C_r , of 0.05 was assumed for the clays at the site and the minimum allowable spacing between adjacent edges of the belled out portion of the caissons was taken as 2 feet. The possible differential settlement between a pier and abutment is similarly estimated to be about 1 inch. This differential movement is most likely to take place between the south pier and abutment due to some additional settlement caused by the approach fill to the south abutment.

With provision of spread footings at a higher elevation to support the bridge piers and abutments as discussed previously, the total and differential settlements would probably be of the order of $1\frac{1}{2}$ to 2 times greater.

Approaches

The existing north bank of the river is about 70 feet high and slopes down at about 2 horizontal to 1 vertical. To accommodate the approach grade a cutting of about 15 feet will be made on top of the bank. The existing slope is not surficially stable as evidenced by zones of water seepage and sloughing on the face.

To provide surficial slope stability, and to minimize horizontal creep effects on the abutment in the lacustrine clay,

the existing bank should be trimmed. Based on the results of drained strength tests on samples of the lacustrine clay given on Figure 14, we recommend that the north bank be trimmed to a slope of 3 horizontal to 1 vertical, as shown on Figure 2.

The trimmed slope should be covered by a blanket of free draining granular material at least 2 feet in thickness to prevent surface water erosion, minimize softening of the clay due to frost action and to control seepage of water from the face of the bank. Growth such as grass and shrubs should be encouraged on the granular blanket. The portion of the blanketed slope below river high water level should further be protected against scour by rip rap. Provided that the north abutment foundations are carried into the glacial till underlying the lacustrine deposit, the long term overall stability of the trimmed slope is adequate.

The side slopes of the roadway cut in the upper portion of the north bank should be made no steeper than 2 horizontal to 1 vertical. The side slopes should similarly be covered by a granular blanket. In order to prevent erosion of the trimmed river bank, the surface water runoff from the roadway cut should be channelled into drains and not allowed to spill over the bank.

The approach fill to the south abutment of the bridge will be about 20 feet in height above existing ground surface. This portion of the site is underlain by very loose to loose sandy to clayey silt with organic matter, a floodplain deposit. To ensure the stability of the approach embankment it is recommended that the floodplain material be removed prior to placement of the fill. The

floodplain material should be removed beneath the complete end slope of the embankment and for a minimum distance of 5 feet back of the approach slab to the abutment. As it is understood that the roadway is not to be paved for several years following construction of the embankment, it is not essential to excavate back of the above approach slab limit, provided some minor local instability and settlement of the fill during construction is acceptable.

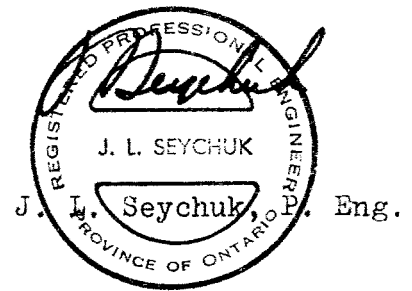
It is recommended that the end and side slopes of the approach embankment be made 2 horizontal to 1 vertical. The fill should be well compacted in place in lifts generally not exceeding 12 inches in thickness. A protective rip rap cover or the like should be provided on the end and side slopes of the approach fill below river high water level to guard against scour. Growth on the side slopes elsewhere should be encouraged to prevent surface runoff gullying.

With removal of the recent alluvium to the glacial till and the construction of the fill as discussed above, the overall stability of the approach fill and abutment founded in the till is adequate.

It is recommended that free draining granular backfill, compacted in 9 inch lifts, be placed behind both the south and north abutment of the bridge. The granular backfill should extend horizontally from the back face of the abutment walls for a minimum distance of 5 feet.

We consider that no economic advantage would be gained by moving the south abutment location much closer to the river, as the thickness of floodplain material increases in this direction and more excavation and approach fill would be required. Furthermore, more problems due to seepage of river water into excavations for removal of the floodplain material would be expected closer to the river.

JLS/jb
6270



February, 1963

for  L.G. Soderman, P. Eng.

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. - Fail 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

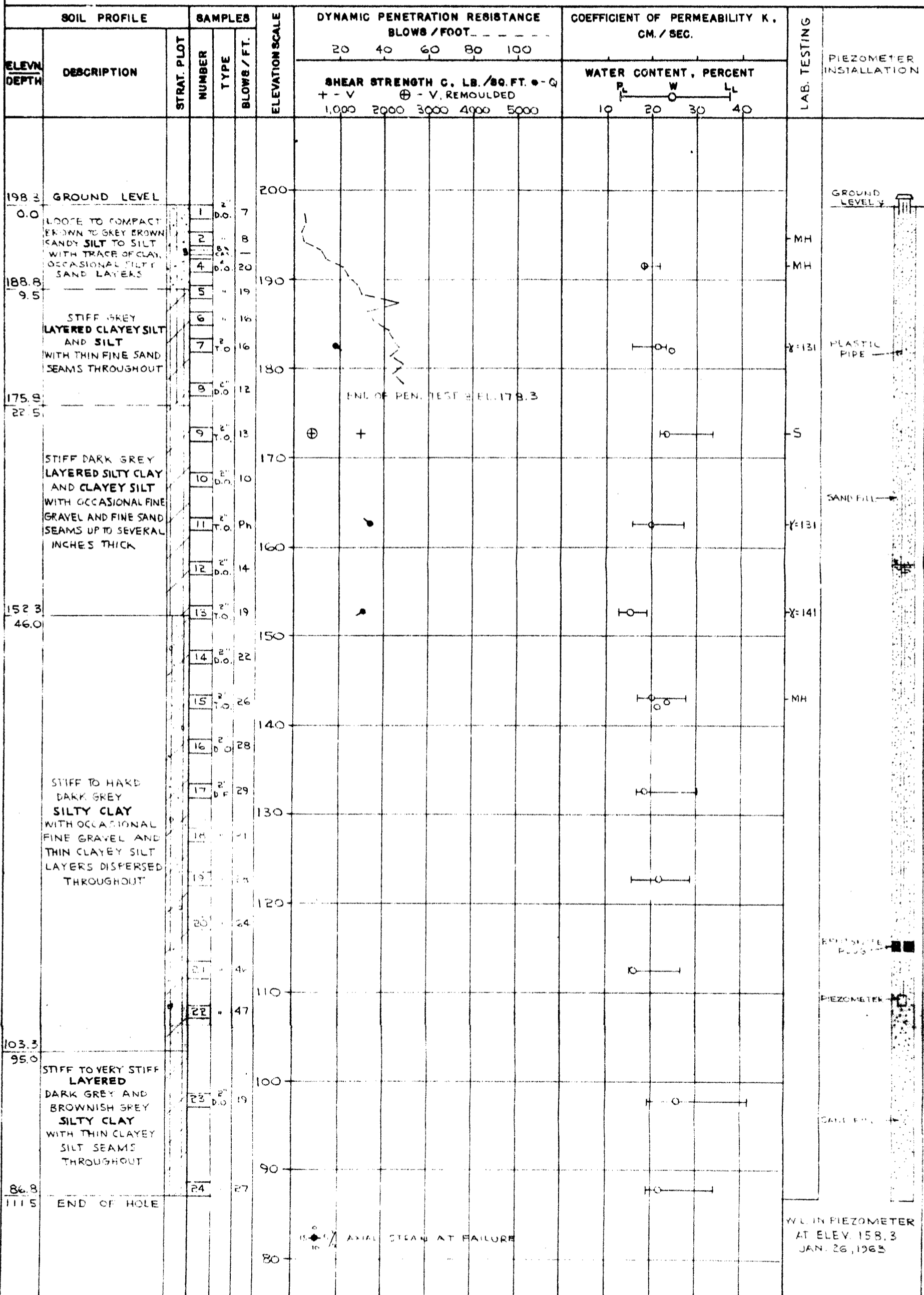
Note: Undrained triaxial tests in which pore pressures are measured are shown as Q' or Q'c.

SOIL PROPERTIES

γ - Total Unit Weight	K - Coefficient of Permeability
γ_d - Dry Unit Weight	c - Undrained Shear Strength ($\frac{1}{2}$ Compressive Strength)
γ_b - Submerged Unit Weight	St - Sensitivity
L _L - Liquid Limit	ϕ' - Effective Angle of Shearing Resistance
P _L - 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 1

LOCATION SEE FIGURE 1 BORING DATE DEC 5 - 17, 1962 DATUM LOCAL
 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



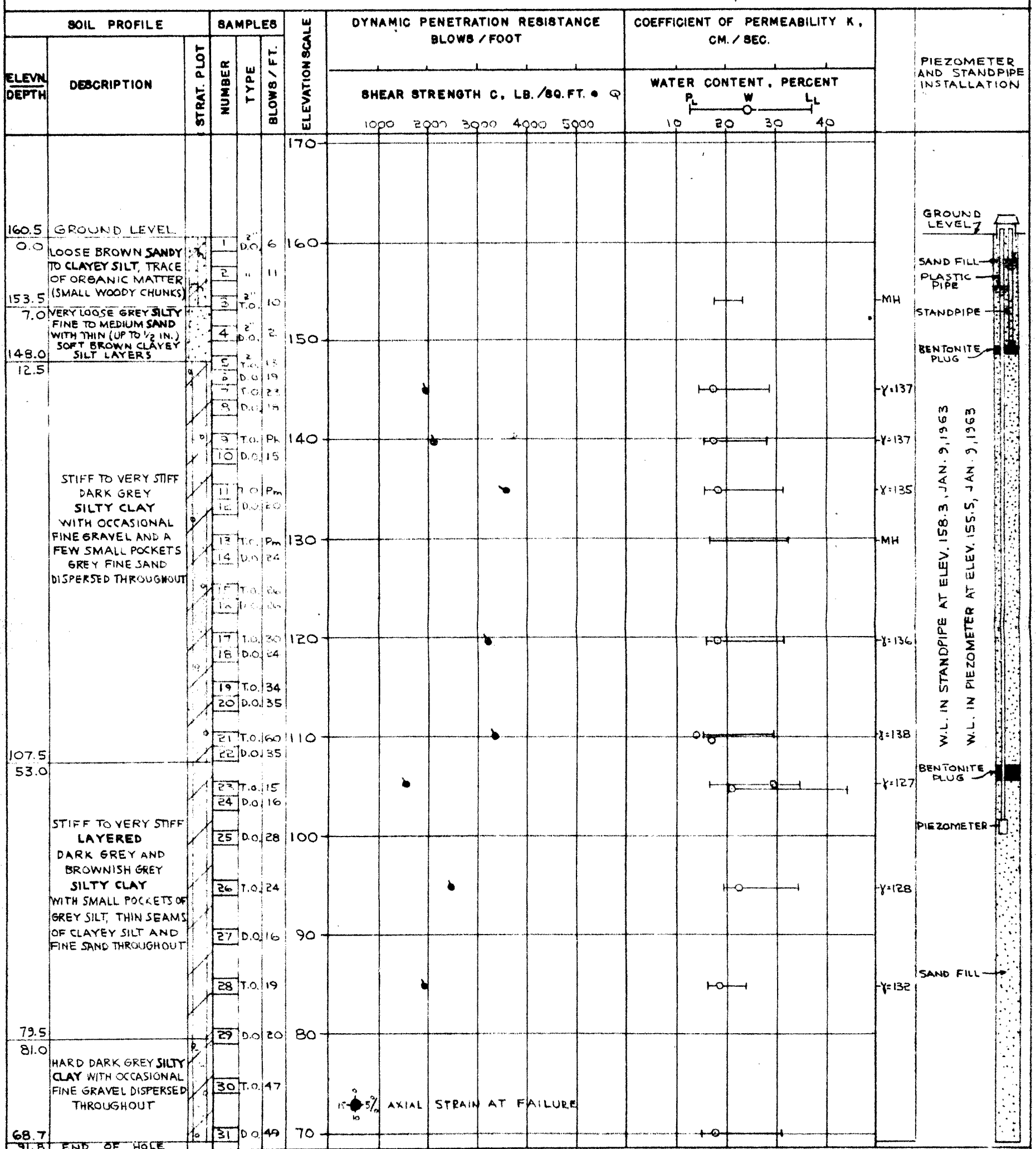
VERTICAL SCALE
 1 INCH TO 10'-0"

COLDER & ASSOCIATES

DRAWN J.A.
 CHECKED

RECORD OF BOREHOLE 2

LOCATION SEE FIGURE 1 BORING DATE DEC 21, 1962 - JAN 4, 1963 DATUM LOCAL
BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER EXCAVATING
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT LB. DROP INCHES



RECORD OF BOREHOLE 3

LOCATION SEE FIGURE 1

BORING DATE DEC. 17 - 20, 1962

DATUM LOCAL

BOREHOLE TYPE

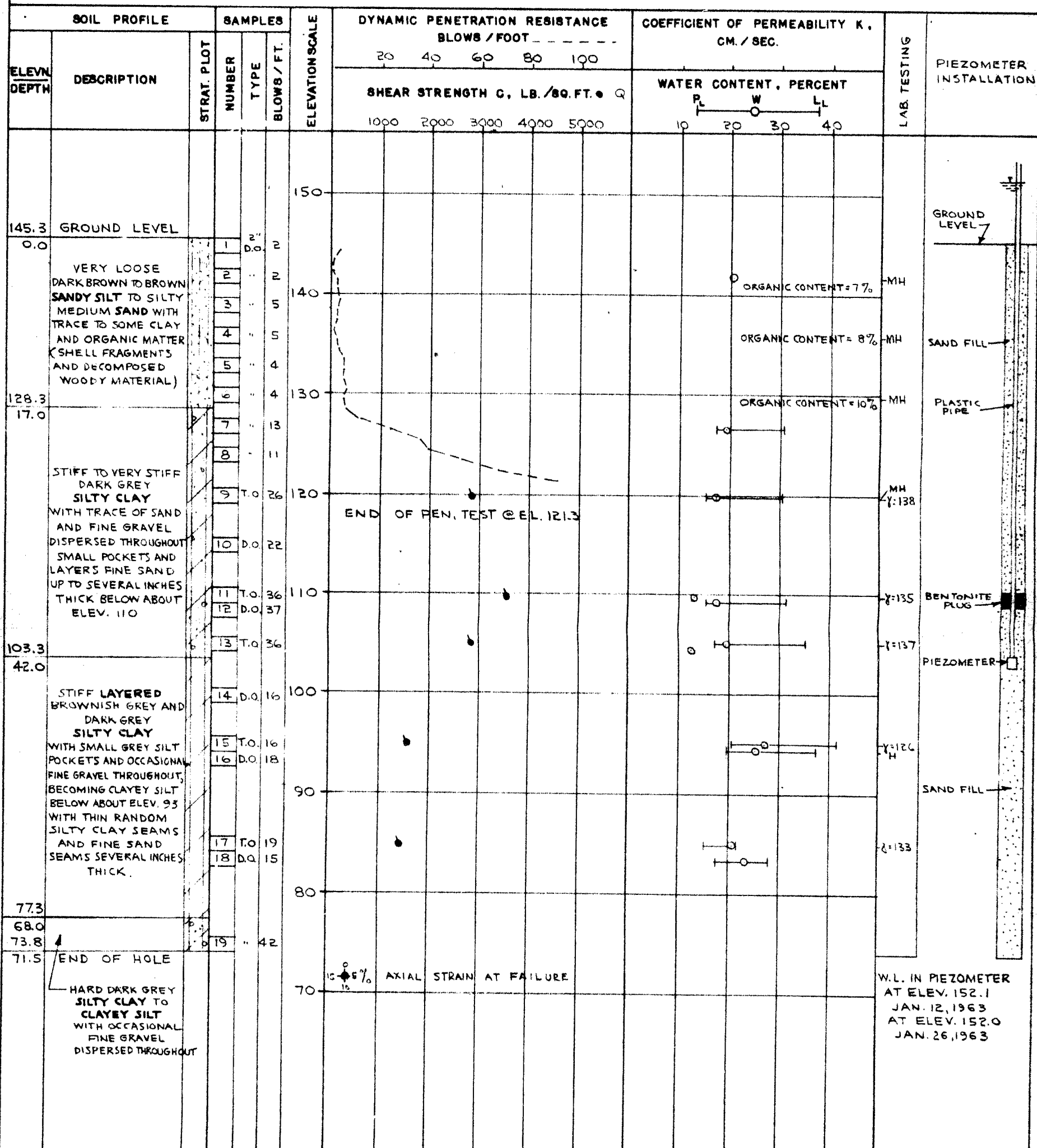
WASH BORING

BOREHOLE DIAMETER

BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

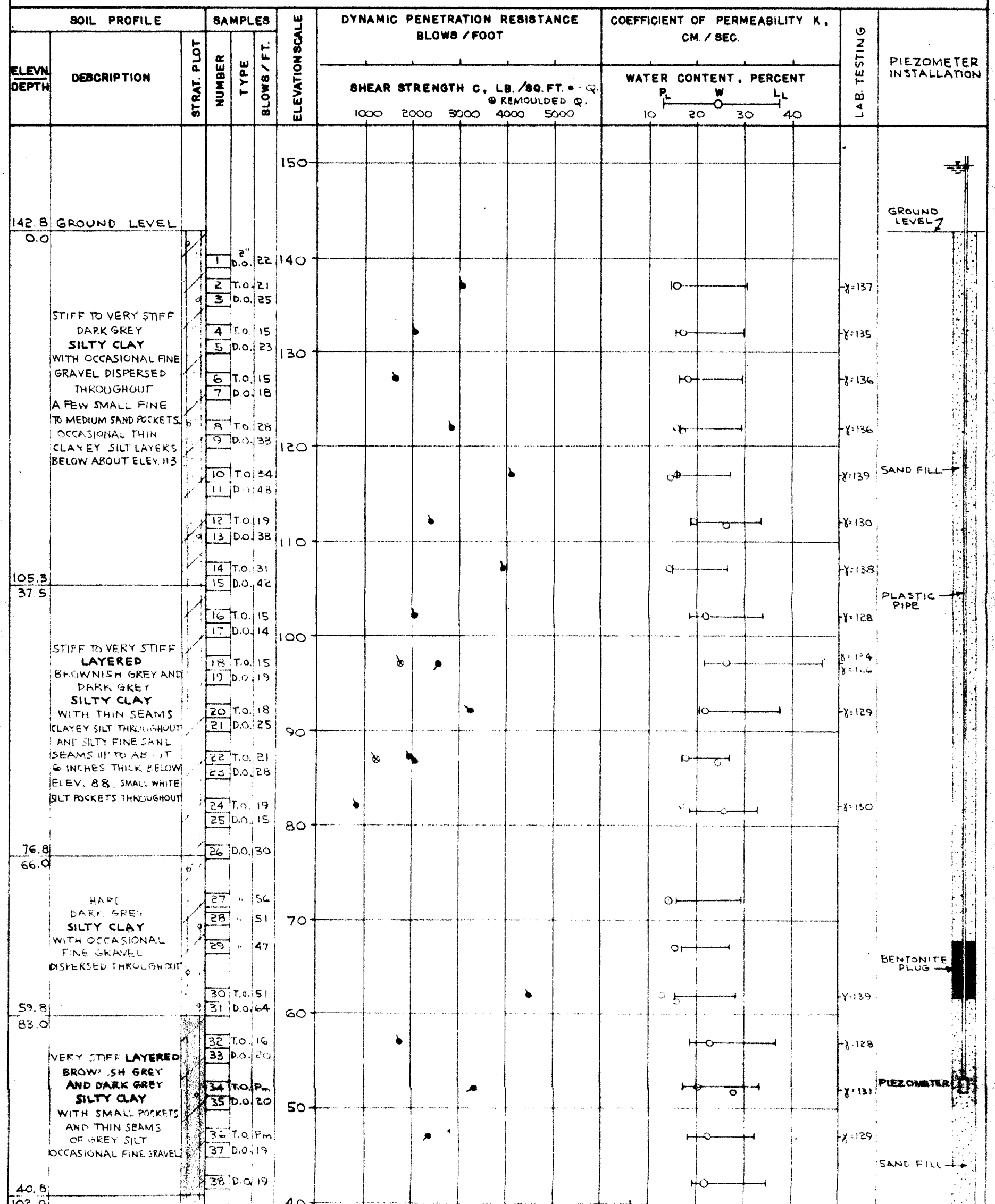
VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED J.S.

RECORD OF BOREHOLE 4

LOCATION SEE FIGURE 1 BORING DATE JAN. 5 - 9, 1963 DATUM LOCAL
BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NY CASING
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES



RECORD OF BOREHOLES 7, 8, 9

LOCATION SEE FIGURE 1

BORING DATE JAN. 12, 1963

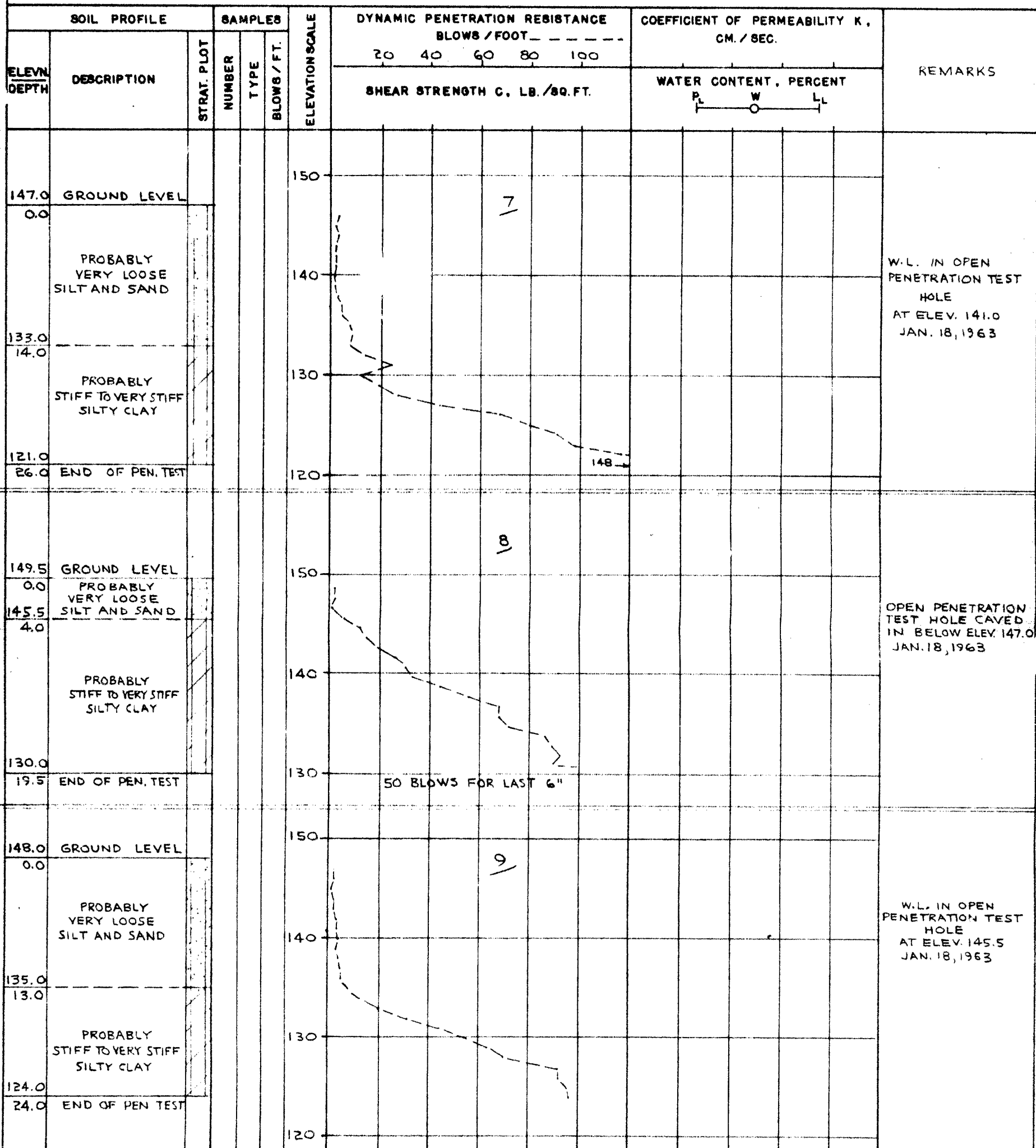
DATUM LOCAL

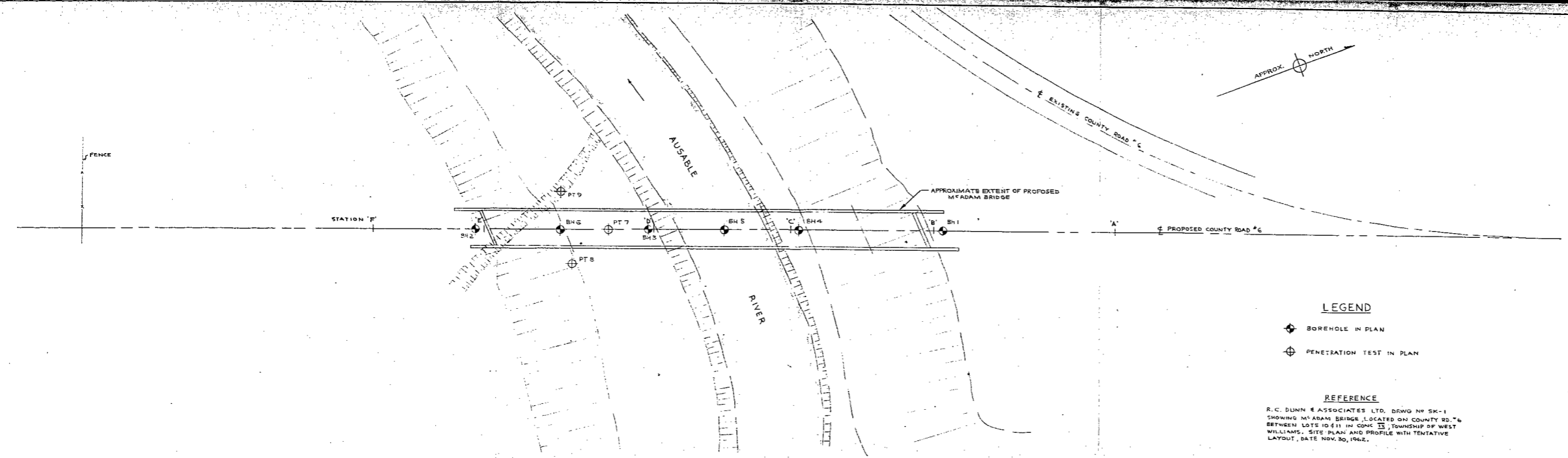
BOREHOLE TYPE PENETRATION TESTS

BOREHOLE DIAMETER

SAMPLER HAMMER WEIGHT — LB. DROP — INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES





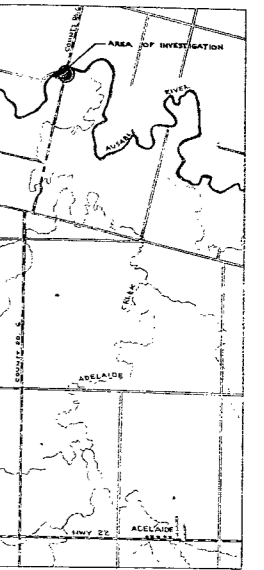
LEGEND

- ◆ BOREHOLE IN PLAN
- ⊕ PENETRATION TEST IN PLAN

REFERENCE

R.C. DUNN & ASSOCIATES LTD. DRWG NO SK-1
 SHOWING MEADAM BRIDGE, LOCATED ON COUNTY RD. #6
 BETWEEN LOTS 10 & 11 IN CONC. 13, TOWNSHIP OF WEST
 WILLIAMS, SITE PLAN AND PROFILE WITH TENTATIVE
 LAYOUT, DATE NOV. 20, 1962.

SCALE: 1" TO 40'-0"



KEY PLAN
 SCALE: 1" TO 0.8 MILES

GOLDER & ASSOCIATES

Made: J.A.
 Chkd: J.S.
 Appd: J.S.

STRATIGRAPHY

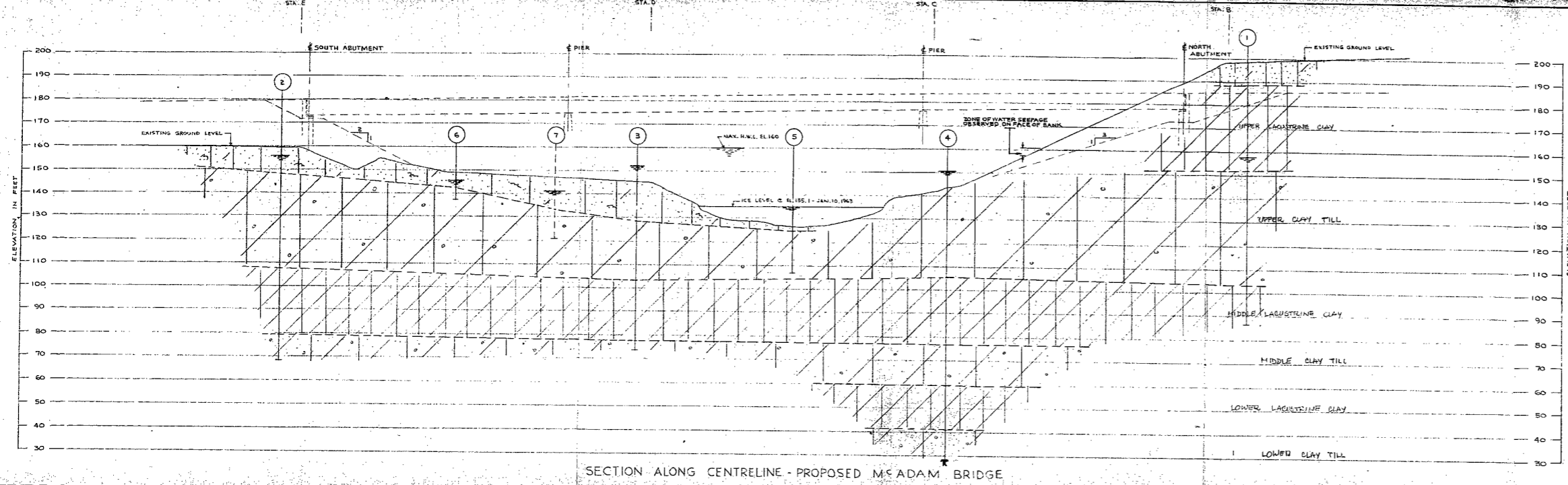
- VERY LOOSE TO LOOSE DARK BROWN TO BROWN CLAYEY SILT TO SILTY SAND WITH TRACE OF ORGANIC MATTER (RECENT ALLUVIUM)
- STIFF GREY LAYERED CLAYEY SILT TO SILTY CLAY WITH THIN FINE SAND SEAMS, OCCASIONAL GRAVEL (UPPER LACUSTRINE CLAY)
- STIFF TO VERY STIFF DARK GREY SILTY CLAY WITH OCCASIONAL FINE GRAVEL THROUGHOUT, A FEW SMALL POCKETS OF LAYERS FINE SAND (UPPER CLAY TILL)
- STIFF TO VERY STIFF LAYERED BROWNISH GREY AND DARK GREY SILTY CLAY WITH THIN CLAYEY SILT AND FINE SAND SEAMS THROUGHOUT BECOMING CLAYEY SILT BELOW ABOUT ELEV. 90 WITH SILTY CLAY SEAMS & FINE SAND LENSES UP TO 2" THICK (MIDDLE LACUSTRINE CLAY)
- HARD DARK GREY SILTY CLAY WITH OCCASIONAL FINE GRAVEL THROUGHOUT (MIDDLE CLAY TILL)
- VERY STIFF LAYERED BROWNISH GREY AND DARK GREY SILTY CLAY (LOWER LACUSTRINE CLAY)
- HARD DARK GREY SILTY CLAY WITH TRACE TO SOME SAND AND GRAVEL (LOWER CLAY TILL)
- REFUSAL - BOULDER OR BEDROCK

LEGEND

- 1 BOREHOLE IN ELEVATION
- 7 PENETRATION TEST IN ELEVATION
- W.L. IN BOREHOLE, JAN. 1963

SPECIAL NOTE: DATA CONCERNING THE DESIGN OF THE BRIDGE ARE BASED ON THE STRATIGRAPHY SHOWN ON THIS SECTION. THE STRATIGRAPHY IS BASED ON THE DATA OBTAINED FROM THE BORING LOGS AND PENETRATION TESTS. THE STRATIGRAPHY IS NOT A GUARANTEE OF THE ACTUAL SOIL CONDITIONS. THE STRATIGRAPHY IS FOR INFORMATION ONLY.

SCALE: 1" TO 20'
DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT



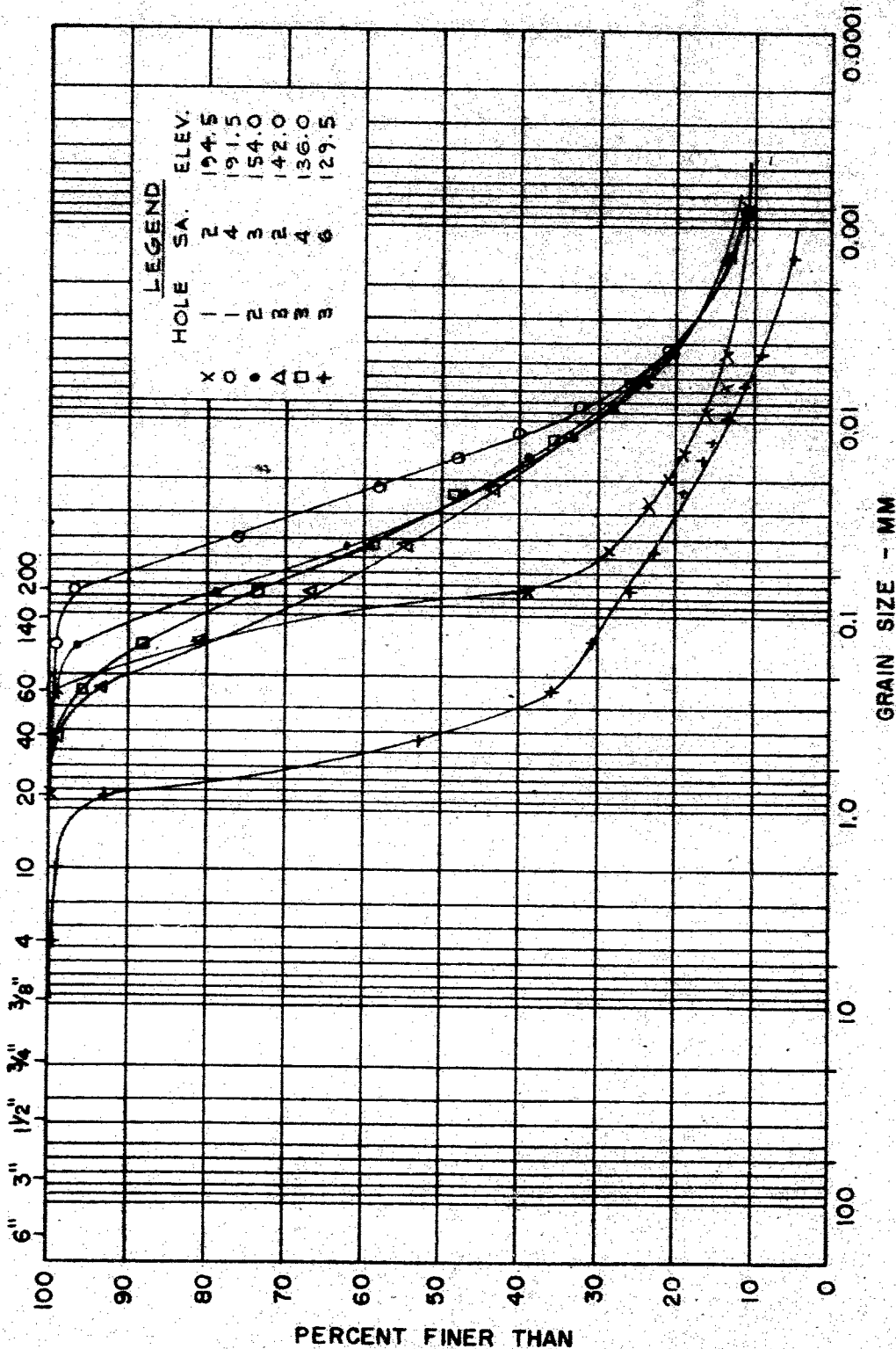
SECTION ALONG CENTRELINE - PROPOSED McADAM BRIDGE

GRAIN SIZE DISTRIBUTION RECENT ALLUVIUM

FIGURE 3

M.I.T. GRAIN SIZE SCALE

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



PERCENT FINER THAN

GRAIN SIZE - MM

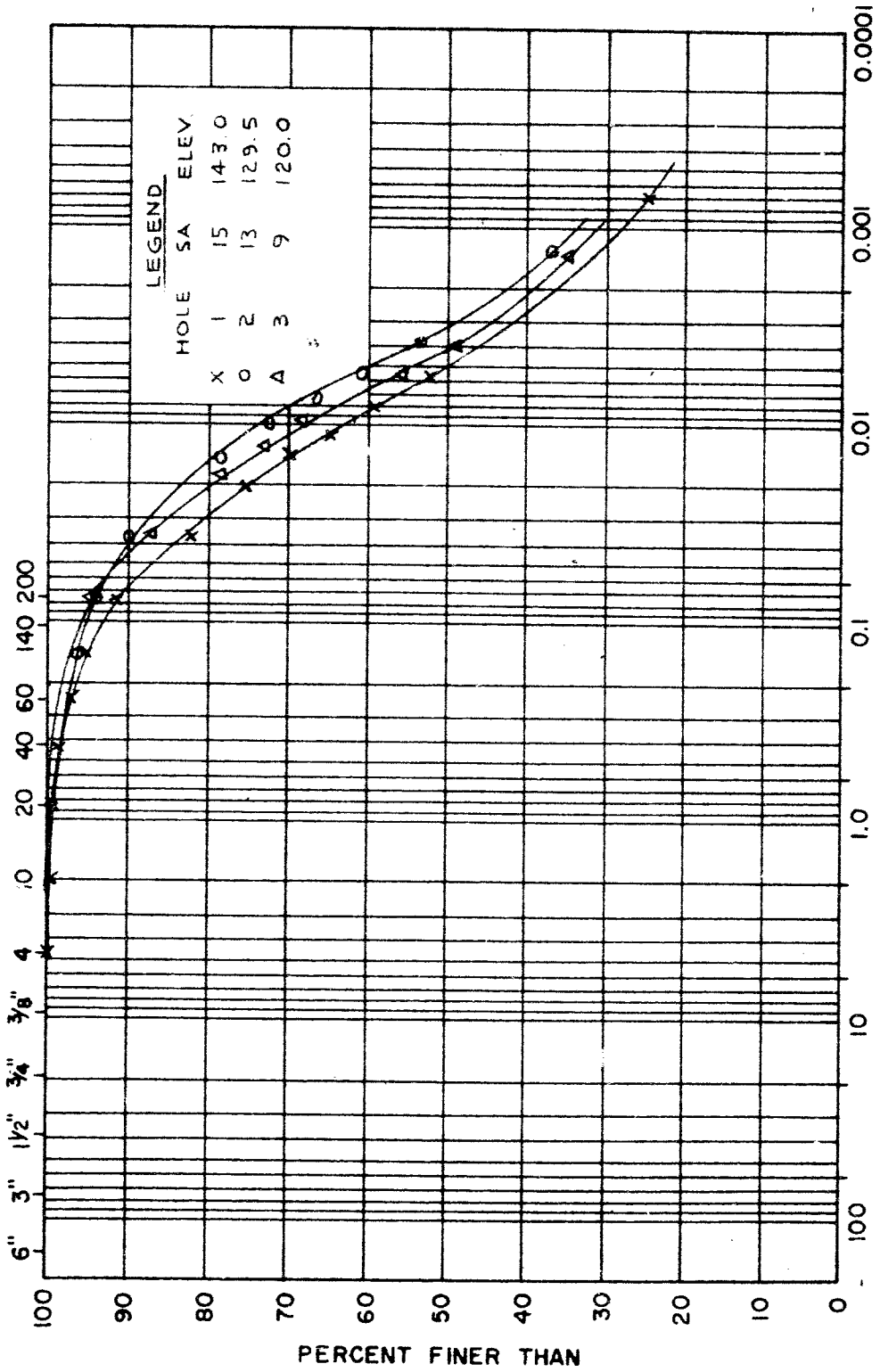
COBBLE SIZE	COARSE	MEDIUM	FINE	SAND SIZE			SILT SIZE		CLAY SIZE	
	GRAVEL SIZE			COARSE	MEDIUM	FINE	FINE GRAINED			

GRAIN SIZE DISTRIBUTION UPPER CLAY TILL

FIGURE 4

M.I.T. GRAIN SIZE SCALE

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

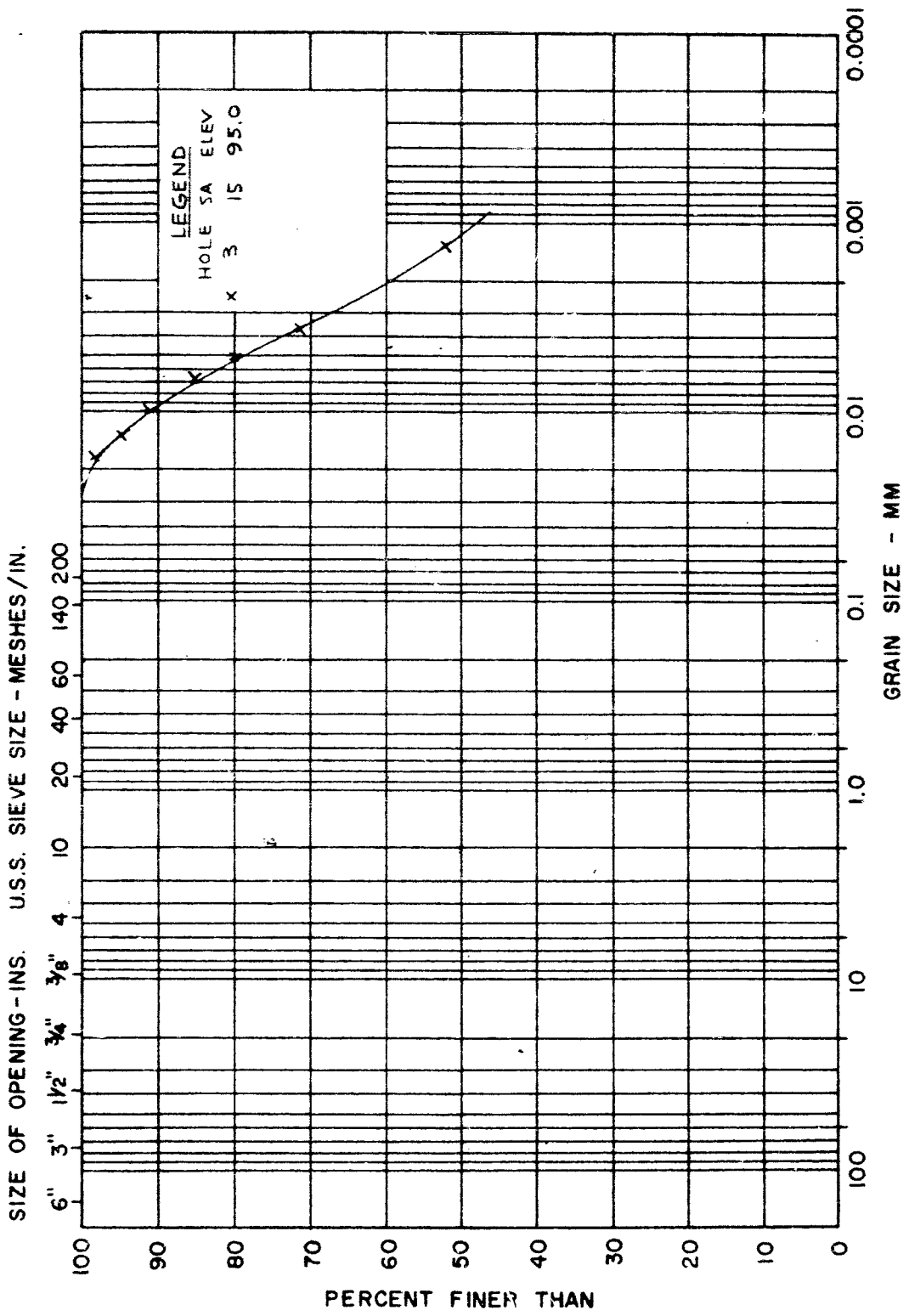


COBBLE SIZE	COARSE	MEDIUM	FINE	SAND SIZE			SILT SIZE	CLAY SIZE	
	GRAVEL SIZE			COARSE	MEDIUM	FINE		FINE GRAINED	

GRAIN SIZE DISTRIBUTION MIDDLE LACUSTRINE CLAY

FIGURE 5

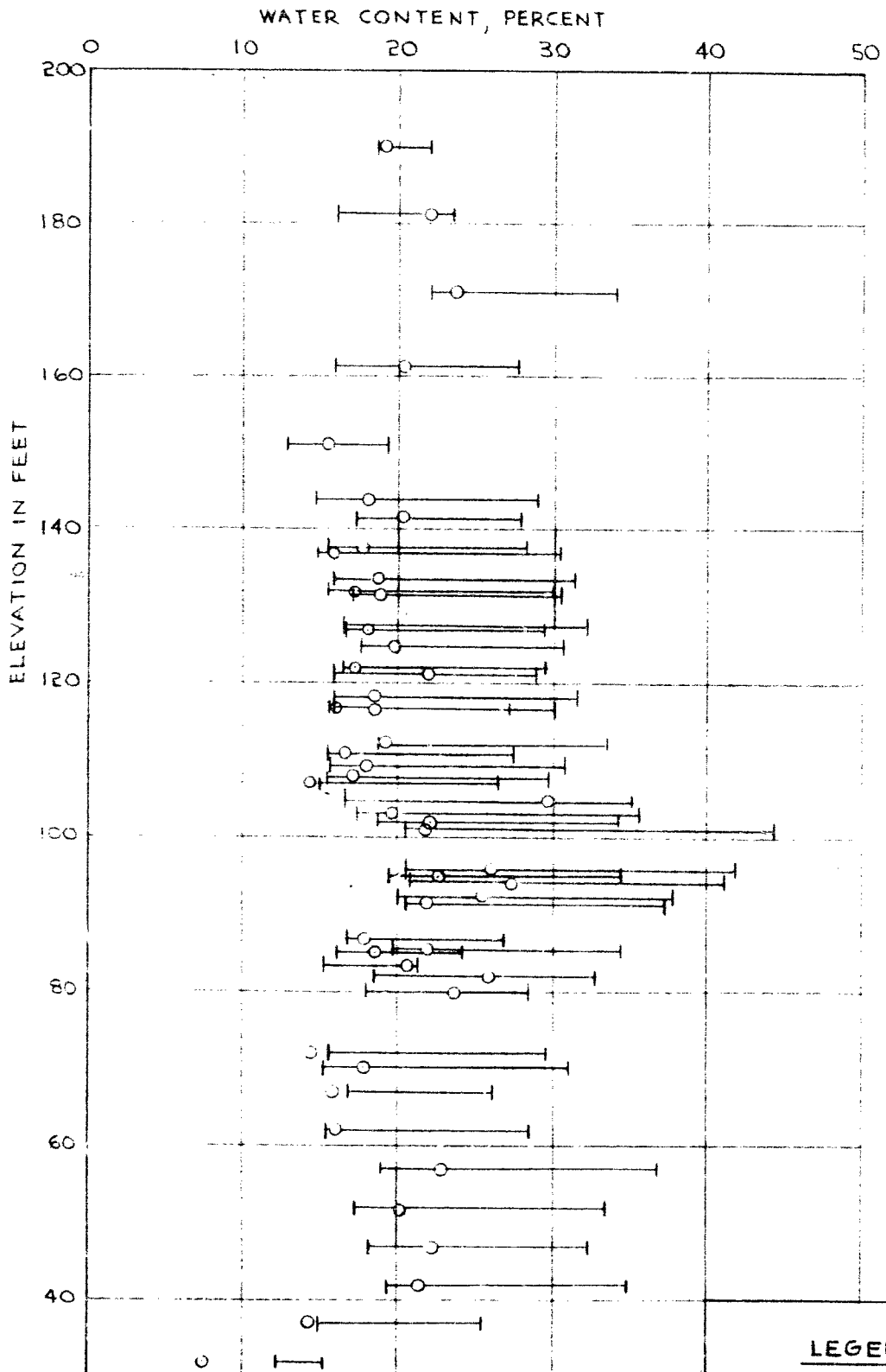
M.I.T. GRAIN SIZE SCALE



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

ATTERBERG LIMITS & NATURAL WATER CONTENTS VS ELEVATION

FIGURE 6



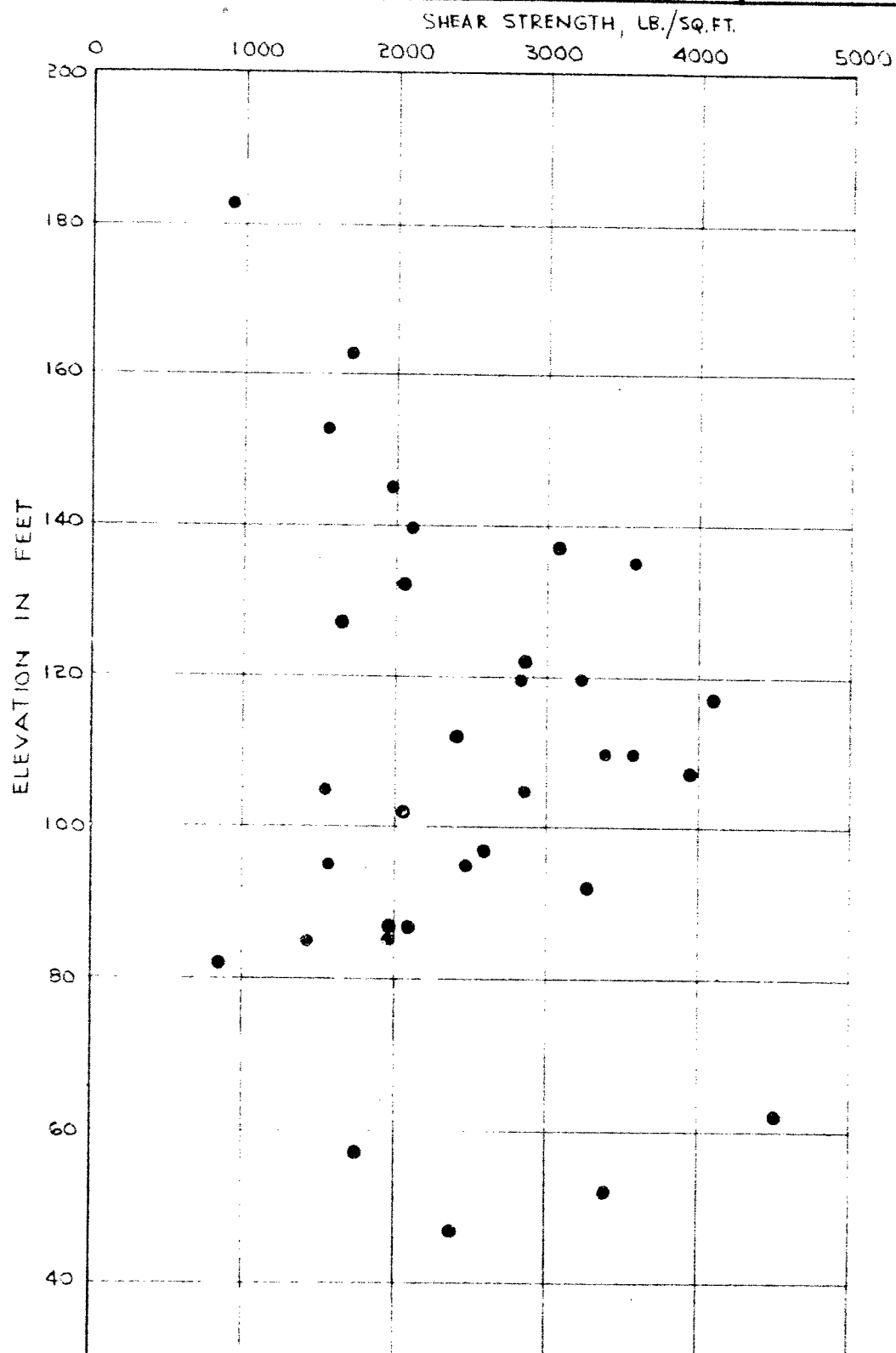
GOLDER & ASSOCIATES

LEGEND

WATER CONTENT
PLASTIC LIMIT ———— ○ ———— LIQUID LIMIT

UNDRAINED SHEAR STRENGTH VS ELEVATION

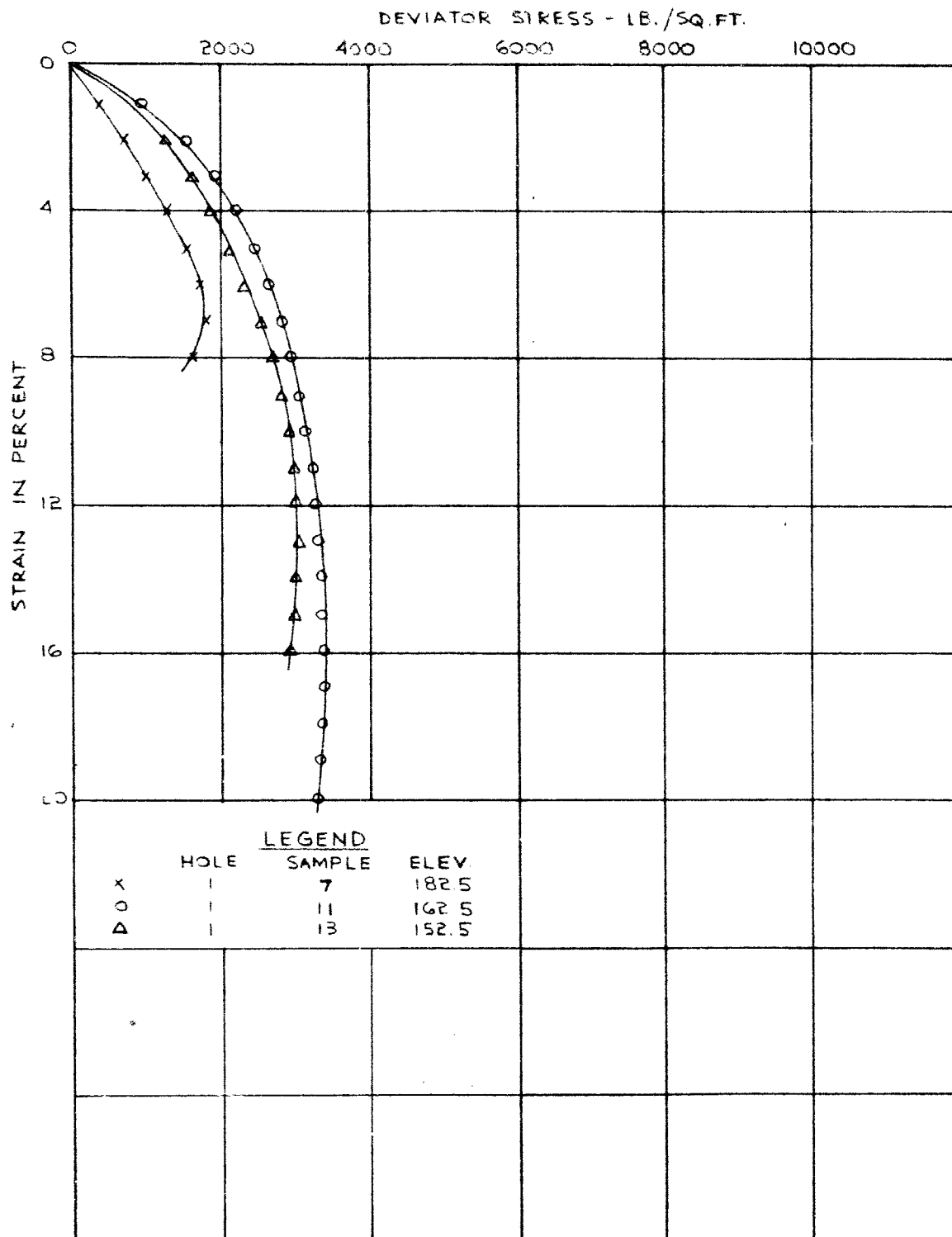
FIGURE 7



GOLDER & ASSOCIATES

UNDRAINED TRIAXIAL COMPRESSION TESTS STRESS - STRAIN CURVES UPPER LACUSTRINE CLAY

FIGURE 8

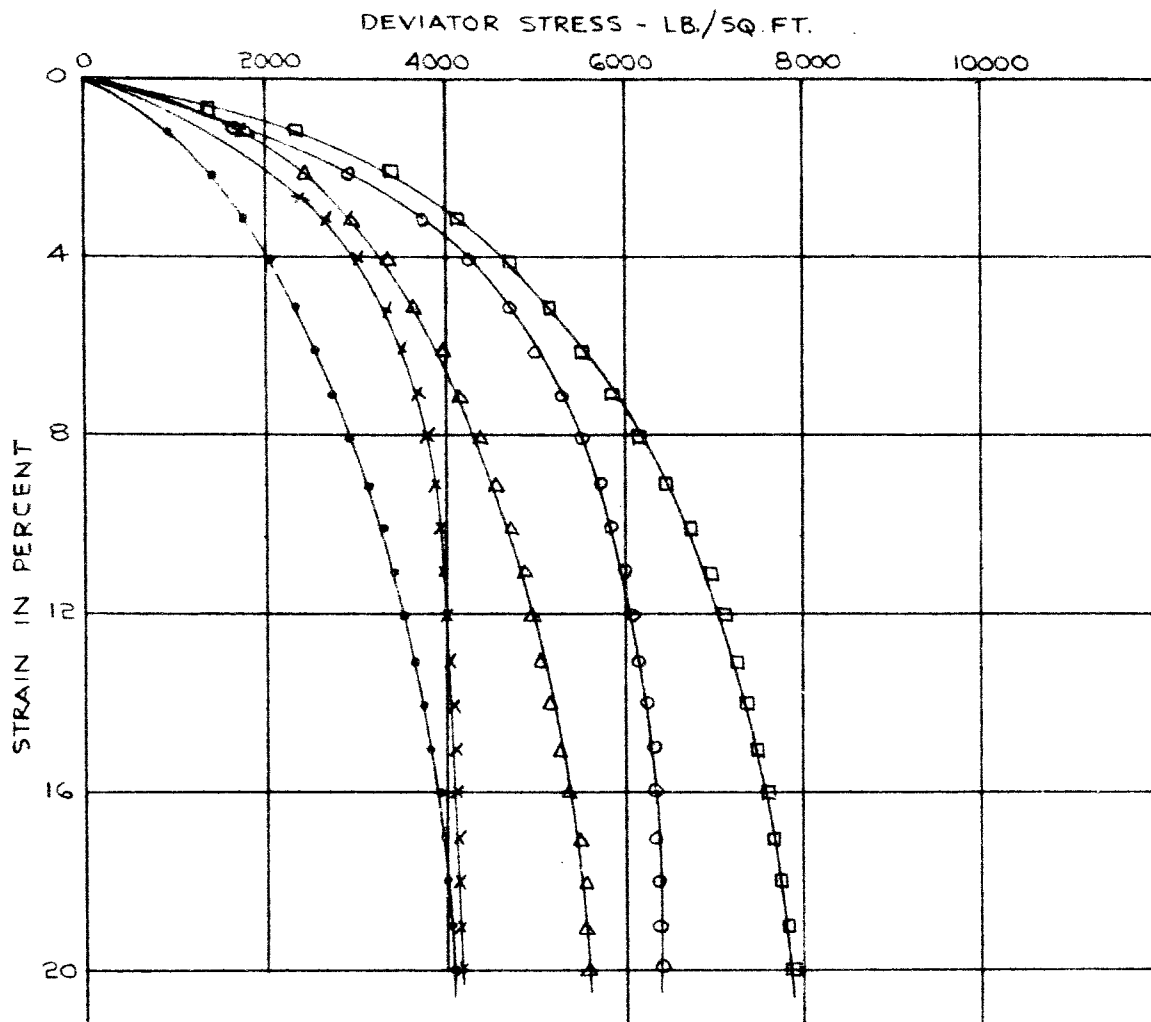


UNDRAINED TRIAXIAL COMPRESSION TESTS

TYPICAL STRESS-STRAIN CURVES

UPPER CLAY TILL

FIGURE 9

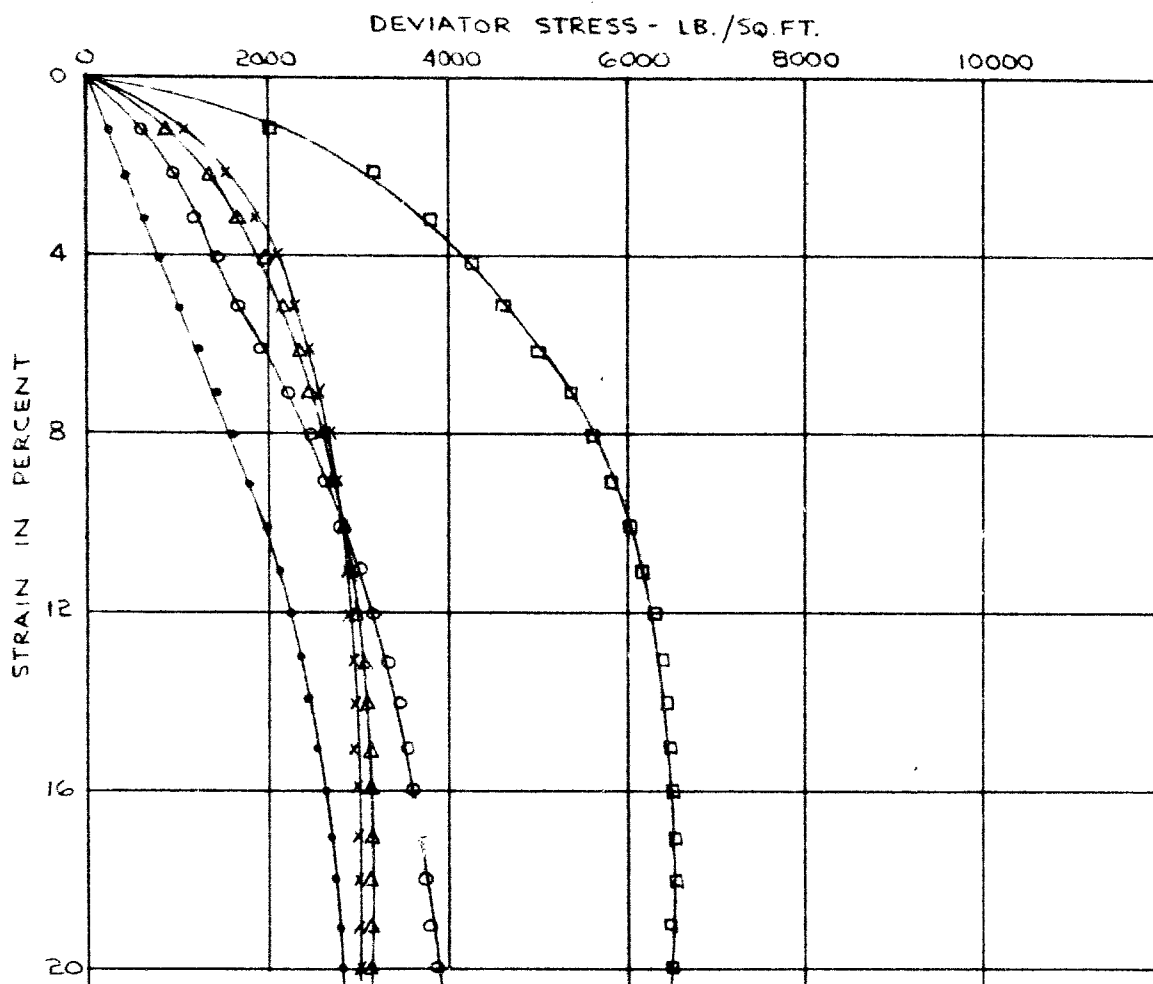


UNDRAINED TRIAXIAL COMPRESSION TESTS

TYPICAL STRESS-STRAIN CURVES

MIDDLE LACUSTRINE CLAY

FIGURE 10



LEGEND

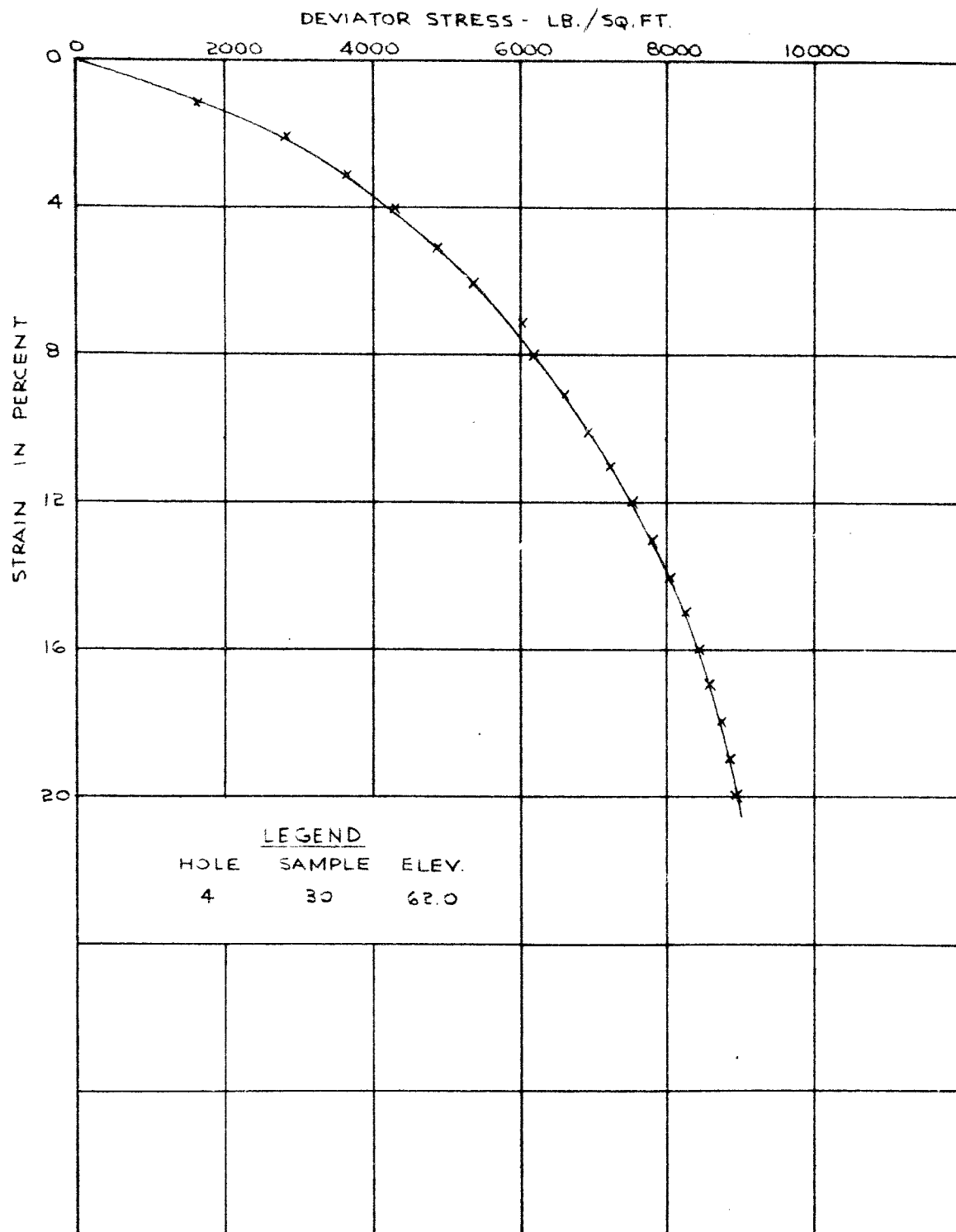
	HOLE	SAMPLE	ELEV.
X	2	23	105.0
O	2	28	85.0
Δ	3	15	95.0
•	3	17	85.0
□	4	20	92.0

UNDRAINED TRIAXIAL COMPRESSION TESTS

STRESS-STRAIN CURVE

MIDDLE CLAY TILL

FIGURE 11



GOLDER & ASSOCIATES

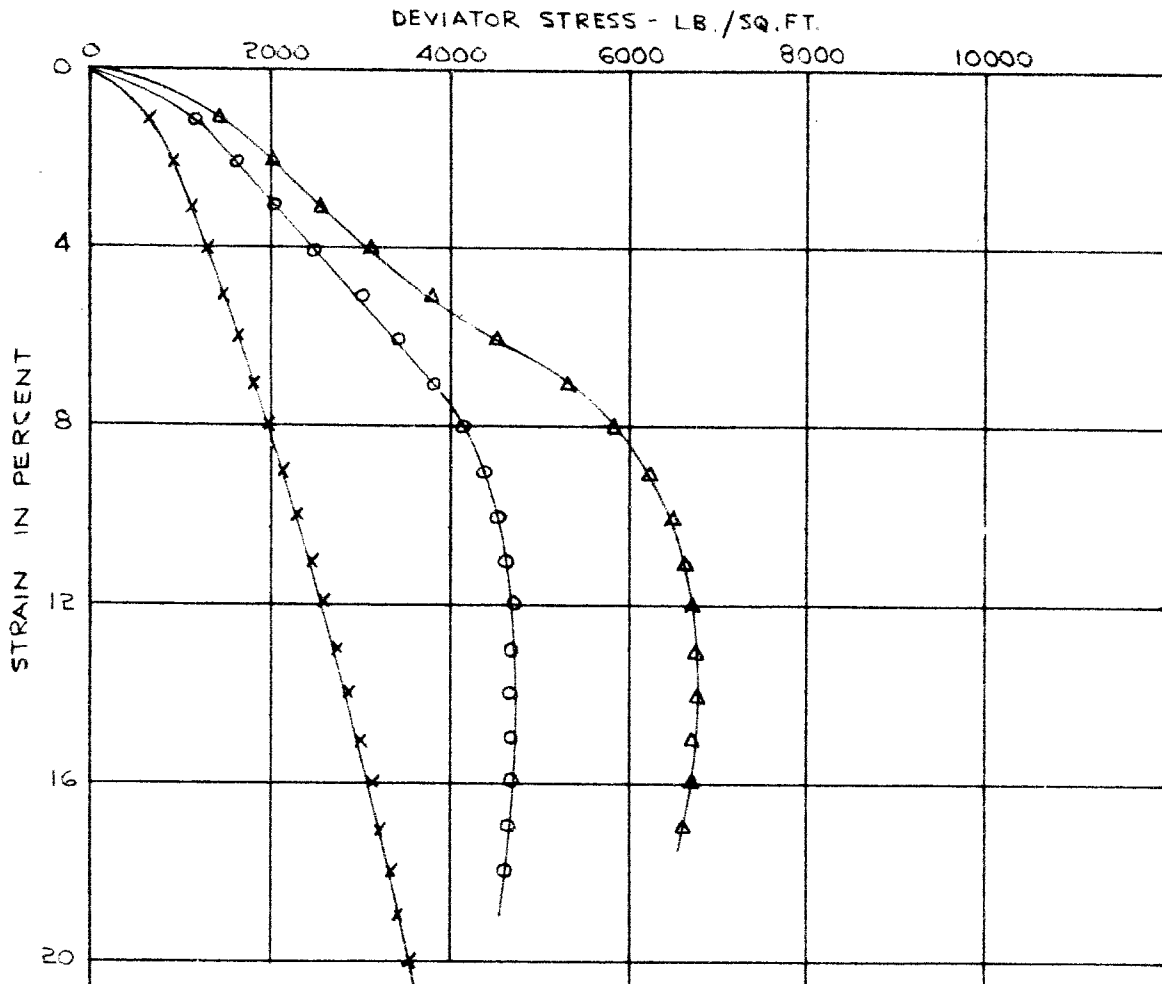
PROJECT No. 6279

UNDRAINED TRIAXIAL COMPRESSION TESTS

STRESS - STRAIN CURVES

LOWER LACUSTRINE CLAY

FIGURE 12



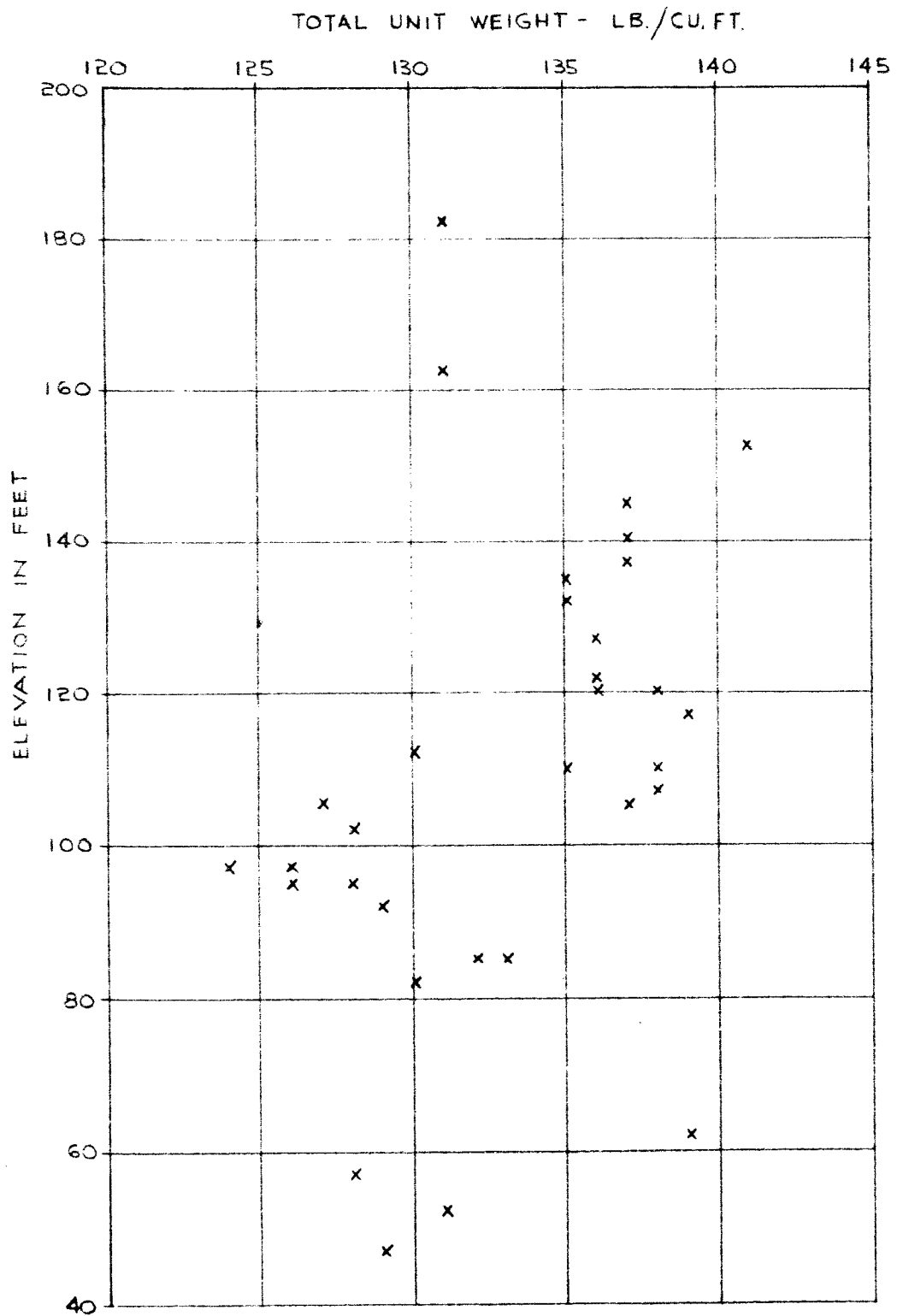
LEGEND

	HOLE	SAMPLE	ELEV.
x	4	32	57.0
Δ	4	34	52.0
O	4	36	47.0

PROJECT No. 6279

UNIT WEIGHT VS. ELEVATION

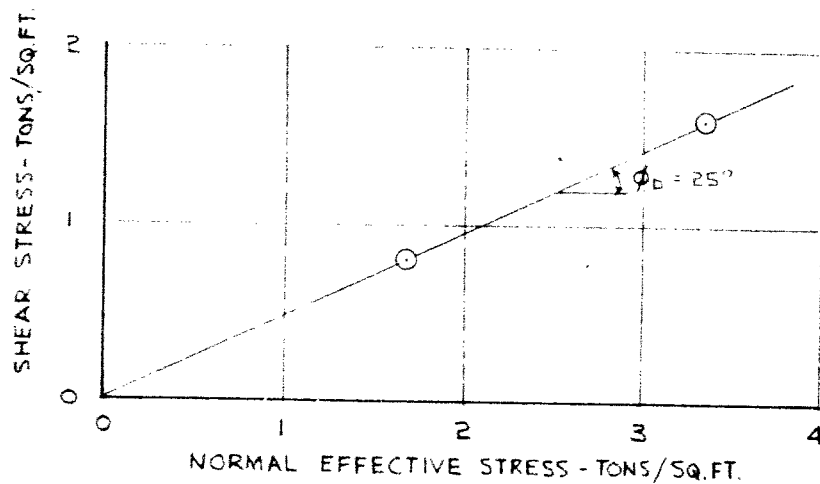
FIGURE 13



GOLDER & ASSOCIATES

CONSOLIDATED DRAINED DIRECT SHEAR TESTS

FIGURE 14



SPECIMEN	NORMAL EFFECTIVE STRESS TONS/SQ. FT.	SHEAR STRESS TONS/SQ. FT.	WATER CONTENT ON FAILURE PLANE AT END OF TEST PERCENT
1	1.68	0.82	21.6
2	3.34	1.61	20.8

RATE OF STRAIN = 0.0007 IN./MIN.
TIME TO FAILURE - SPECIMEN 1 - 7 HRS. 34 MIN.
SPECIMEN 2 - 6 HRS. 40 MIN.

LEGEND

BOREHOLE	SAMPLE	ELEV.	LL	PL	W
1	9	172.0	34	22	23.5