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LEMIEX

BRIDGE

PRESOTT
&

RUSSELL

COUNTIES

B.A. 1802

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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REPORT

TO

C. C. PARKER & ASSOCIATES LIMITED

ON

SITE INVESTIGATION

PROPOSED LEMIEUX BRIDGE OR CULVERT

COUNTIES OF PRESCOTT AND RUSSELL

LEMIEUX

ONTARIO

64-1-2901

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April, 1964

64007

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ABSTRACT

The results of the site investigation for the proposed replacement of a bridge crossing Moose Creek at Lemieux, Ontario are reported. The creek runs in a steep, narrow valley and it was found that the valley slopes are formed of 50 to 60 feet of firm to stiff silty clay topped by some 10 to 20 feet of sand. The clay is underlain by limestone bedrock. The creek has eroded to bedrock in the valley and subsequently redeposited up to 16 feet of alluvium composed predominantly of loose silts and soft organic clay upstream from the existing bridge and sand and gravel downstream from the bridge.

It was found that the overall stability of the natural slopes at the site is adequate although there is considerable surficial instability. It is recommended that, to avoid decreasing the stability of the existing slopes, a culvert be used for the proposed crossing, to be founded on the bedrock beneath the floor of the valley. Design criteria are given for the culvert and embankment, and construction procedures and treatment of existing or proposed slopes and creek banks are discussed.

INTRODUCTION

H. Q. Golder & Associates Ltd. has been retained by C. C. Parker and Associates Limited on behalf of the United Counties of Prescott and Russell to carry out a site investigation for the proposed replacement of a bridge crossing Moose Creek at Lemieux, Ontario. The purpose of the investigation was to determine soil and groundwater conditions at the site and to provide criteria for the design and construction of foundations and approaches for a bridge or culvert.

PROCEDURE

The field work for the investigation was carried out in the period January 23 to 29, 1964, using two machine drillrigs. A total of six boreholes were put down at the locations shown on Figure 1. The holes were put down in NX and BX size by wash boring methods and samples, both disturbed and undisturbed, were taken of the soils encountered. In situ vane shear tests were also carried out in the holes where appropriate, and bedrock was cored in AX and BX size. Detailed logs of each hole are given on the Records of Boreholes immediately following the text of the report.

A program of laboratory testing was carried out on the soil samples obtained during the field work and the results are plotted on the Records of Boreholes and on the figures following.

Locations and elevations of boreholes together with a stadia contour plan of the site were provided by the County Engineer, Mr. A. J. Lynch, of the Roads Department of the United Counties of Prescott and Russell. It is understood that all elevations are referred to Geodetic Datum.

SITE AND GEOLOGY

The site is located on Moose Creek at Lemieux immediately upstream from its confluence with the South Nation River. Moose Creek occupies a narrow meandering valley which has been cut deeply into the Prescott and Russell sand plains through which it runs (1951, Chapman and Putnam). The general floor of the valley is at about Elevation 160, the creek bed is at about Elevation 150 and the surrounding plain is at about Elevation 220.

The former single span bridge and approaches at Lemieux suffered a washout in the fall of 1963 and the bridge has been replaced temporarily by a Bailey Bridge placed across the old abutments, which, it is believed, were founded on bedrock.

The field work at Lemieux revealed that the valley slopes are composed mainly of a firm to stiff, but very sensitive, silty clay. Further, it was observed that, although the abutments and approaches of the former bridge appear stable, there is evidence that many slopes of the valley at the site are surficially unstable

and in the general area there is considerable evidence of former slope failures. It is therefore pertinent to consider the geological origin of the clay and its general regional behaviour in order to provide a reasonable basis for dealing with the practical design problems at the site.

The clay was deposited in the upper reaches of an inland sea called the Champlain Sea which occupied the St. Lawrence and Ottawa lowlands in the period during the latter stages of the Wisconsin and the final retreat of the last great ice sheet. The extensive clay deposits associated with this sea from Pembroke and Kingston to the ocean are generally classified as "Leda" clay. In spite of the large area occupied by deposits of this clay, there are important similarities in the characteristics and engineering behaviour, wherever it is encountered.

One of the most important characteristics of Leda clay is that it is highly sensitive to disturbance and in extreme cases it becomes a viscous liquid when remolded, even though it may be in a stiff condition in its natural, undisturbed state. This characteristic forms the basis for one of its important aspects of engineering behaviour, namely earth flows or flow slides. Earth flows are usually initiated by a relatively small bank failure which then retrogresses in a series of slides in which

part of the soil is completely remolded and flows away carrying relatively undisturbed clay with it. Commonly a pear-shaped crater results, of which the small portion marks the point of the initial bank failure. The size of these craters is usually large, ranging from a few acres to thousands of acres in area and involving millions of cubic yards of earth. (For a few examples of slides in the Ottawa-St. Lawrence lowlands see Crawford, 1961). A recent example of a moderately large flow slide is in Hawkesbury, Ontario, some 30 miles from Lemieux, where a slide involving an area of 12 acres and 500,000 cubic yards of material occurred on December 7, 1955 (Eder, 1956). The above does not mean to imply that slides which occur in Leda clay are always of the flow type but it does emphasize a need for caution in the design and construction of projects where it is present.

The Leda clay in this area is generally overlain by some 10 to 20 feet of alluvial sands and silts which is the basis for the area's physiographic classification as a sand plain. Underlying the clay is glacial drift and Ordovician limestones of the Eastview formation (Wilson, 1946).

SOIL CONDITIONS

The inferred soil stratigraphy across the site is shown on Figure 2. It was found that the slopes of the valley

of Moose Creek are composed of stiff to firm silty clay to a depth of 45 feet from Elevation 190. The head of the valley walls are at about Elevation 220 and from observations at the site it appears that the upper 20 feet or so of the sides of the valleys are made up of silty fine sand. It is inferred therefore that the general thickness of the clay is approximately 50 to 60 feet.

The clay is underlain by a black limestone bedrock, the upper few feet of which is often weathered and broken. The elevation of the sound rock ranges between about Elevation 140 and 150 across the site.

Moose Creek has apparently cut down through the clay to bedrock and has subsequently redeposited an alluvium composed predominantly of loose organic sandy silts and soft organic silty clays up to 16 feet in thickness in the area upstream from the former bridge. (See Records of Boreholes 1, 2, 3). There are some indications that the constriction caused by the construction of the existing bridge may have influenced the pattern of deposition, because in Borehole 4 located downstream of the bridge no silts or clays were encountered. At this location there was 7 feet of silty sand and gravel with boulders overlying bedrock. This of course does not preclude the fact that pockets of loose or soft sediments may be found immediately downstream of the former bridge.

Grading curves for the silts in Boreholes 1, 2 and 3 and the sand and gravel in Borehole 4 are plotted on Figure 3.

The results of laboratory tests carried out on samples of the clay are plotted on the Records of Boreholes and on Figures 4 to 6 inclusive. Figure 4 is a summary of undrained shear strength plotted against elevations and includes the results of vane tests carried out in the boreholes. This summary indicates that the average undrained shear strength of the clay is approximately 1,000 pounds per square foot with a minimum of about 800 pounds per square foot between Elevation 165 and 175. The vane tests indicate strengths appreciably lower than the undrained triaxial tests, which is not normally the case. These low vane results have been ignored, because adjacent samples have indicated higher shear strengths in the laboratory. Because of the high sensitivity of 10 or more indicated by the vane tests, it is considered that the soil in which the low vane tests were taken had been disturbed by the washing process during the advance of the boreholes.

The results of two consolidation tests indicate that the clay is overconsolidated by approximately 1 ton per square foot in excess of the present overburden pressure (see Figures 5 and 6). These results agree with many results published for the Ottawa valley area (Crawford, 1961).

GROUNDWATER CONDITIONS

Piezometers installed in the boreholes indicated that groundwater levels in the bedrock control those in the clay, and that both are controlled by creek level. All readings in piezometers and boreholes were approximately that of creek level, which during the investigation was at about Elevation 155.

STABILITY OF EXISTING SLOPES

Because of evidence of instability in some of the existing slopes, the stability of three slopes at the site has been analyzed and the results are summarized on Figure 7. An undrained form of analysis has been employed using an average undrained shear strength of 1,000 pounds per square foot. The factors of safety computed for slopes of about 2 to 1 or steeper ranged from 1.3 to 1.2. The corresponding factors of safety using an average shear strength of 800 pounds per square foot would be 1.1 to 1.0. Field observations indicated that these slopes were at least surficially unstable. (See Figure 7, Sections AA, BB).

The slope which appeared most stable was on the north side of the existing road, and west of Moose Creek (See Figure 7, Section CC). This slope is slightly steeper than 4 to 1 and has a computed factor of safety of 1.8 for a shear strength of 1,000 pounds per square foot.

SELECTION OF TYPE OF STRUCTURE

Two types of structures have been considered for the site, namely a bridge of 3 simply supported spans approximately 70 feet long and a reinforced concrete culvert with a span of about 40 feet and a rise of about 20 feet. The proposed alignment and grade of the road across the creek is shown on Figures 1 and 2.

The selection of the type of structure to be used at the site should be influenced by the stability of the existing slopes. From the discussion above it is known that the factor of safety of the existing slopes is about 1.2 to 1.3 based on an un-drained analysis and an average shear strength of 1,000 pounds per square foot. It is considered that a factor of safety of 1.3 is satisfactory, provided that some surficial instability can be tolerated.

The choice of a bridge employing the proposed spans and grade would require the addition of an approach fill of the order of 10 feet in height to the west bank which would reduce the stability of the slope. This is undesirable and in order to avoid the necessity of an approach fill it would be necessary to lower the grade and/or increase the length of spans. Even then the abutment of the bridge, which would have to be founded on piles to rock, would be subject to some movement due to the surficial instability of the existing banks.

Therefore, it is recommended that for approximately equal cost, which it is understood is the case, the more desirable structure for the site would be the culvert. The design and construction of the culvert and its associated features are discussed below.

DESIGN CRITERIA FOR CULVERT

The proposed location of the culvert is shown on Figure 8. The culvert should be founded on the sound limestone bedrock using a unit loading of up to 25 tons per square foot on the footings. The footings should be doweled into the bedrock, if necessary, to provide adequate resistance to the lateral pressures exerted on it by the overlying embankment.

Lateral pressures on the culvert should be computed using a coefficient of earth pressure at rest, K_0 , of 0.5, provided that a free draining granular fill is used for the embankment and as backfill for the culvert.

DESIGN CRITERIA FOR EMBANKMENT

The road embankment will be some 35 to 40 feet above the creek bed. It should be constructed of well-compacted free draining granular material meeting DHO specifications for Granular "B". Use of this material will permit side slopes of 2 to 1.

The loose silt and soft organic silty clay encountered in Boreholes 2 and 3 upstream of the existing bridge are unsuitable foundation material for the embankment and should be removed and replaced with granular fill. The excavation for the culvert will remove much of this material as a matter of course and will furnish an indication of its further extent. From the results of the boreholes it is expected that the silt and organic clay will be confined largely to the west bank of the creek, upstream from the existing bridge.

The removal of the silt and clay should be under the direction of the supervising engineer and should be carried out with care in order to avoid general oversteepening of the existing slopes as discussed below under Construction Procedures.

TREATMENT OF EXISTING SLOPES

It is recommended that the existing slope on the west bank of the creek upstream of the proposed culvert be trimmed to a slope of $2\frac{1}{2}$ to 1 as shown on Figure 8. This will increase the factor of safety of this area to about 1.5 ($c_u = 1,000$ pounds per square foot). Vegetation, including trees, should be planted on the slope to reduce surficial instability.

It would be impractical to carry out extensive treatment of the remaining slopes, which are generally of the order of

2 to 1 or flatter. However, the planting of trees or shrubs on these slopes will help to reduce the observed surficial instability.

RELOCATION OF CREEK AND BANK PROTECTION

Some relocation of the creek will be required and the proposed extent of this is shown on Figure 8. Also shown on Figure 8 is the recommended limit of relocation of the creek towards the east bank on the upstream side of the culvert to avoid oversteepening the slopes.

It will also be important to protect the banks of the new creek channel to prevent erosion, particularly upstream of the culvert where local oversteepening of banks could lead to slope failures which may block the culvert. Therefore, it is recommended that the areas shown on Figure 8 be protected by riprap or similar measures.

CONSTRUCTION PROCEDURE

It is understood that the contractor for this project would be required to construct the culvert in two stages. In the first stage, the portion upstream or south of the existing bridge and embankment would be constructed while keeping the existing road open to traffic. In the second stage a temporary detour would be built over the completed portion of the culvert, the existing bridge and abutments would be removed, and the downstream

portion of the culvert would be constructed. It is anticipated that construction would be carried out in the summer when the flow of Moose Creek is low and would be easily diverted by means of a shallow ditch or temporary culvert.

As noted previously, the excavation for the foundations of the culvert and the removal of the material unsuitable as a foundation for the culvert fill will require care and supervision. The most troublesome area will probably be on the west bank upstream of the existing bridge where there is up to 15 feet or so of material to be removed.

It is recommended that excavations near the toe of the existing slopes in this area be cut at an angle no steeper than $2\frac{1}{2}$ to 1 to avoid oversteepening the slopes. This procedure is illustrated in Figure 9 which also indicates that cut slopes of this order may be required in any case to remove the unsuitable material.

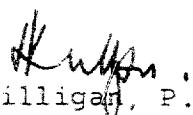
It is inferred that, because of the height and apparent stability of the existing approach fill on the west bank, it is founded on competent material such as that encountered in Borehole 4. Part of the fill will have to be removed for the construction of the proposed culvert, but the remainder may be incorporated in the culvert embankment. Because the height of

the embankment is some 35 feet or so above the base of the excavation for the proposed culvert, it is recommended that cuts in the fill be limited to 2 to 1 or flatter.

All of the existing west abutment and at least some of the east abutment will have to be removed for construction of the culvert and embankment. Because of the very high sensitivity of the clay in the valley slopes and consequently its extreme susceptibility to shock, it is recommended that blasting not be allowed in the removal of the abutments.



A. A. Gass, P. Eng.



V. Milligan, P. Eng.

AAG:IMB
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GOLDER & ASSOCIATES

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- CHAPMAN, L.J. and PUTNAM, D.F. (1951) "The Physiography of Southern Ontario" University of Toronto Press.
- CRAWFORD, C.B. (1961) "Engineering Studies of Leda Clay"; Soils in Canada; The Royal Society of Canada, Special Publication No. 3; University of Toronto Press.
- EDEN, W.J. (1956) "The Hawkesbury Landslide"; Proceedings of the Tenth Canadian Soil Mechanics Conference, December 17 and 18, 1956; National Research Council Technical Memorandum No. 46; June, 1957
- WILSON, A.E. (1946) "Geology of the Ottawa-St. Lawrence Lowland, Ontario and Quebec"; Geological Survey Memoir 241; Canada Department of Mines and Resources, Queen's Printer, Ottawa.

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPES

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

II. PENETRATION RESISTANCES

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

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

<i>WH</i>	sampler advanced by static weight—weight, hammer
<i>PH</i>	sampler advanced by pressure—pressure, hydraulic
<i>PM</i>	sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) *Cohesionless Soils*

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

(b) *Cohesive Soils*

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

IV. SOIL TESTS

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

NOTES:

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

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

LIST OF SYMBOLS

I. GENERAL

π	= 3.1416
e	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of a
$\log_{10} a$ or $\log a$	logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress ($\bar{\sigma}$ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{xy}	shear strain
ν	Poisson's ratio (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) Unit weight

γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) Consistency

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_S	shrinkage limit
I_L	liquidity index = $(w - w_P) / I_P$
I_c	consistency index = $(w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

(c) Permeability

h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
k	coefficient of permeability
j	seepage force per unit volume

(d) Consolidation (one-dimensional)

m_v	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
C_c	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
c_r	coefficient of consolidation
T_v	time factor = $c_r t / d^2$ (d , drainage path)
U	degree of consolidation

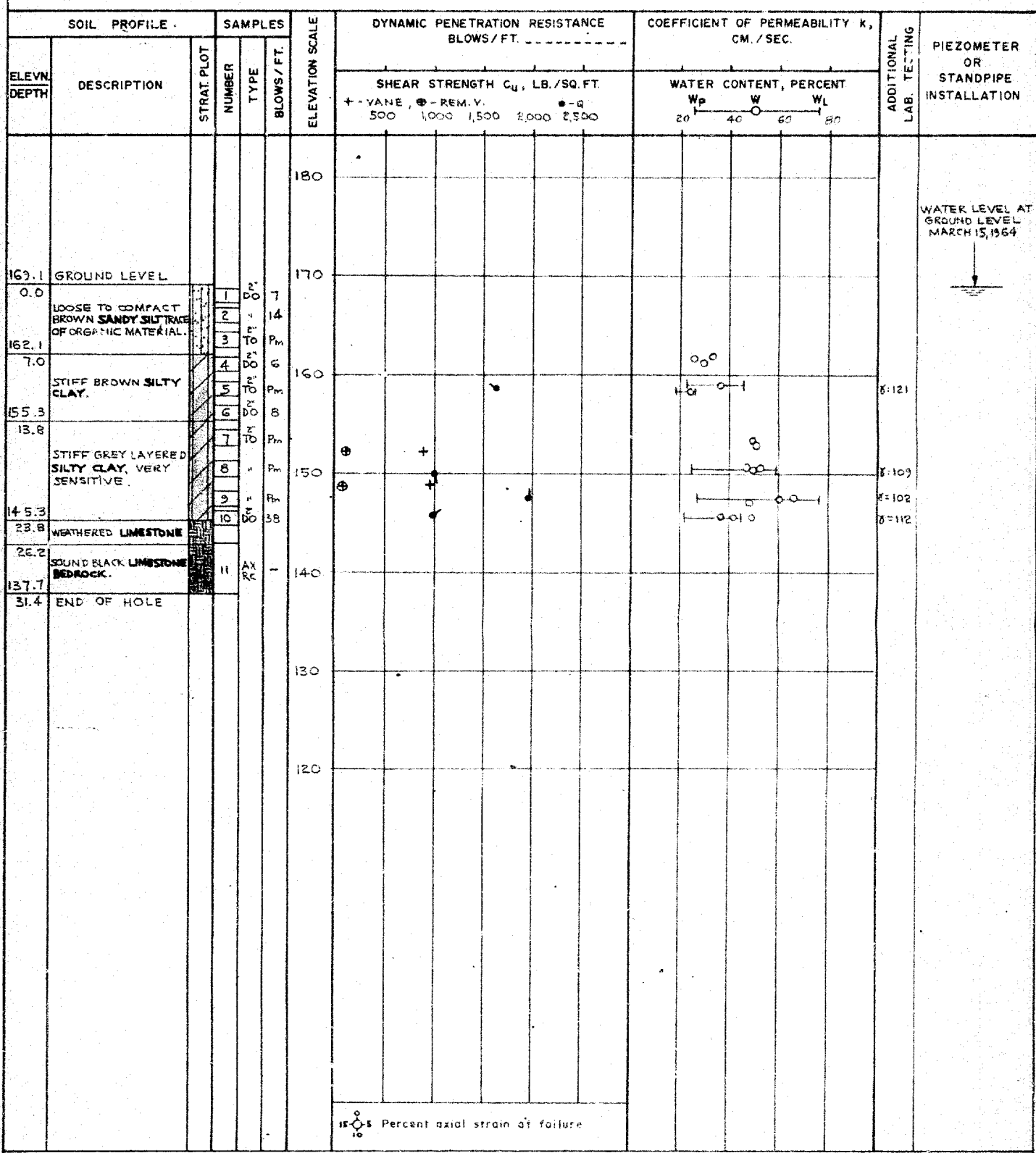
(e) Shear strength

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_r	sensitivity

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

RECORD OF BOREHOLE 1

LOCATION See Figure 1 BORING DATE JAN 24-27, 1964 DATUM GEODETTIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX & BX CASING
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT - LB. DROP - INCHES



VERTICAL SCALE
 1 INCH TO 10' - 0"

COLDER & ASSOCIATES

DRAWN *[Signature]*
 CHECKED *[Signature]*

RECORD OF BOREHOLES 2 & 3

LOCATION See Figure 1

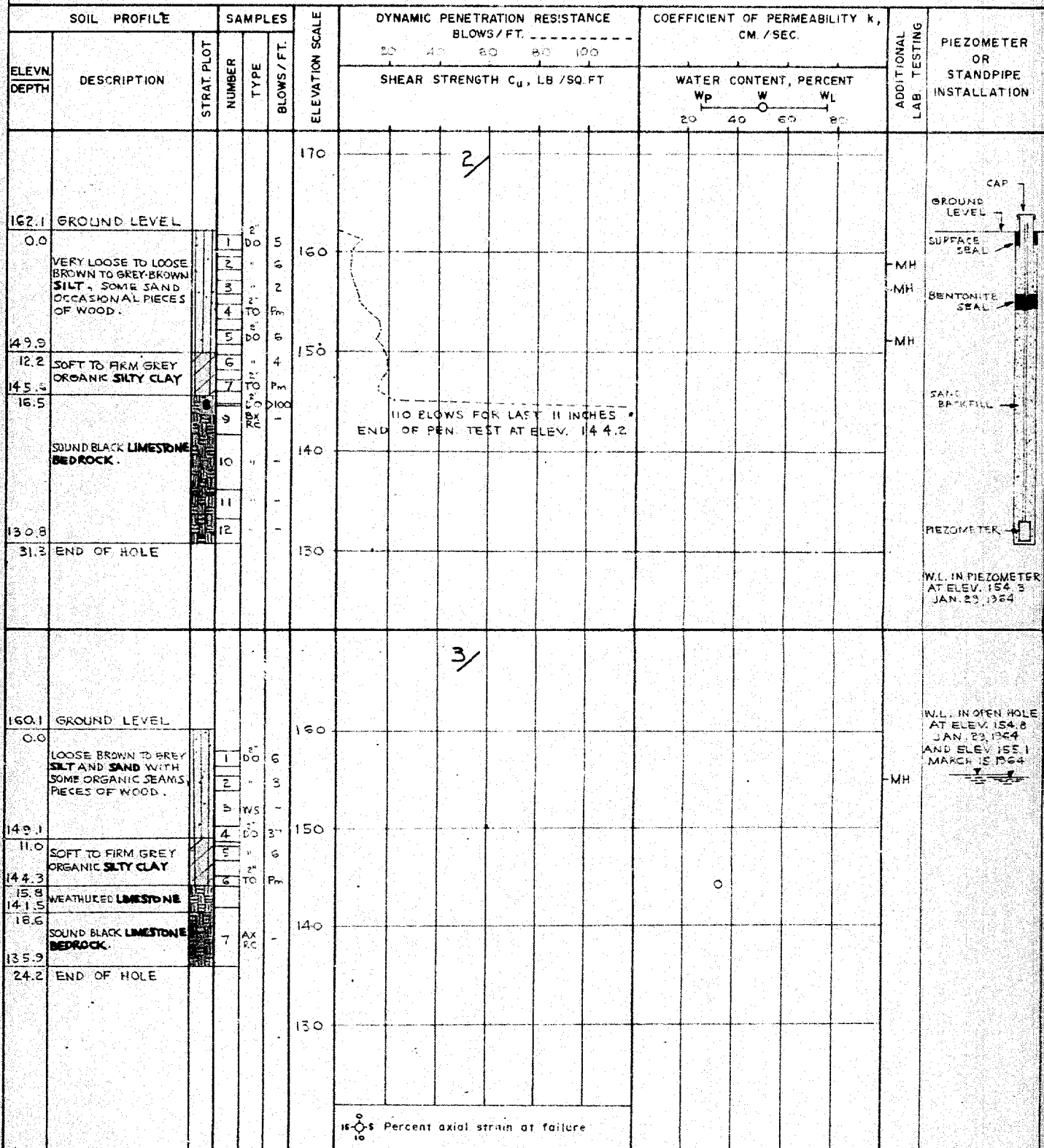
BORING DATE JAN. 23-24 & 28, 1964 DATUM GEODETTIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN M. J.
CHECKED AB

RECORD OF BOREHOLE 4

LOCATION See Figure 1

BORING DATE JAN. 23, 1964

DATUM GEODETTIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER 8X CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT - LB. DROP - INCHES

SOIL PROFILE		SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT.		COEFFICIENT OF PERMEABILITY K, CM / SEC		ADDITIONAL LAB TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEVATION DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		BLOWS/FT.	SHEAR STRENGTH C_u , LB./SQ.FT.	WATER CONTENT, PERCENT W_p W W_L			
155.3	GROUND LEVEL										
0.0	COMPACT BROWN SILTY SAND AND GRAVEL, SOME CLAY, BOULDERS TO 12" SIZE.		1	28							
14.8.2			2	150							
7.1			3								
132.8	SOUND BLACK LIMESTONE BEDROCK.		4	1							
12.5	END OF HOLE										

W.L. IN OPEN HOLE
AT ELEV. 154.1
JAN. 23, 1964

Percent axial strain at failure

 VERTICAL SCALE
 1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

 DRAWN *M. J.*
 CHECKED *D.*

RECORD OF BOREHOLE 5

LOCATION - See Figure 1

BORING DATE JAN 27-28, 1964

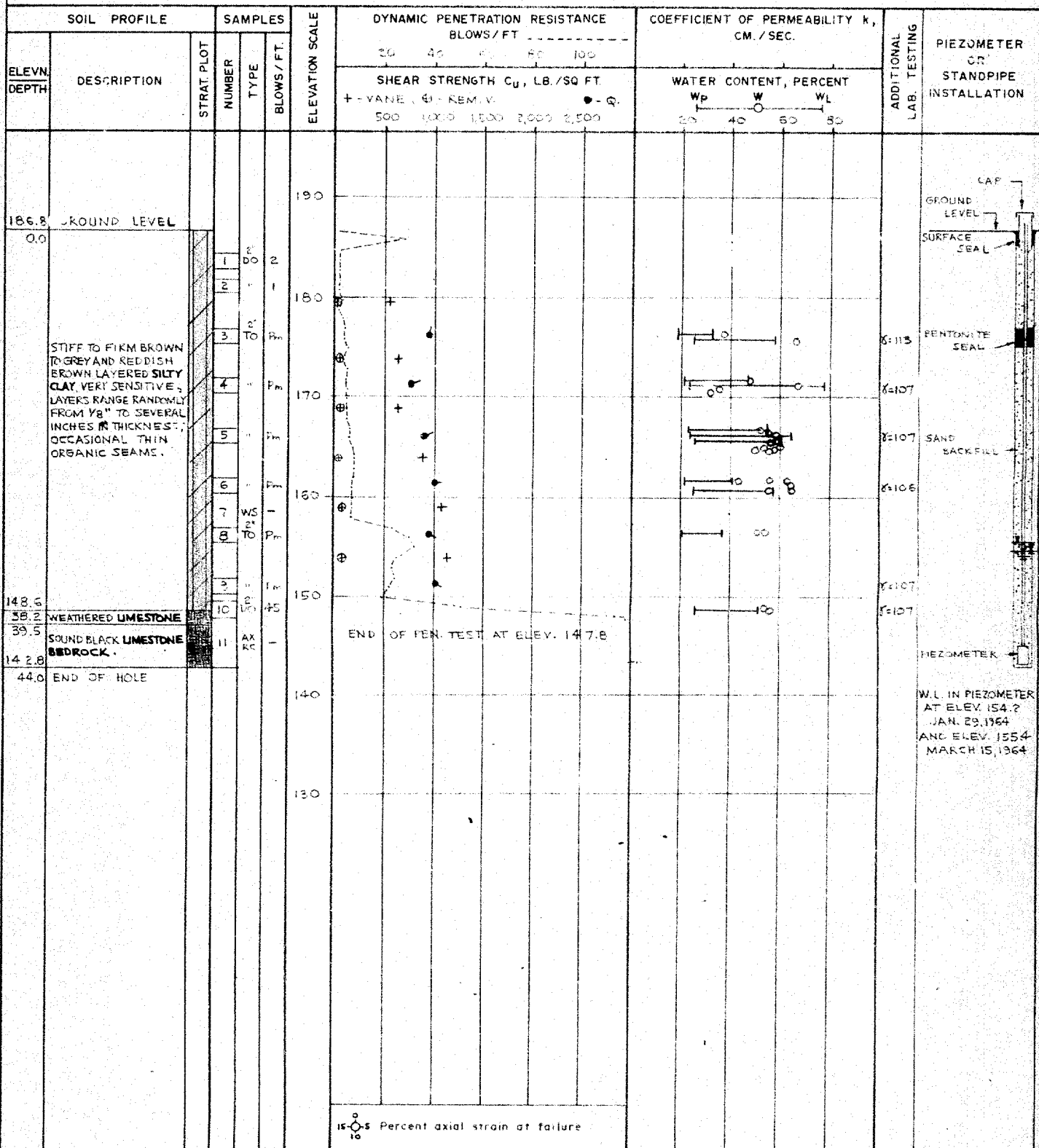
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10' - 0"

COLDER & ASSOCIATES

DRAWN *MLW*CHECKED *th*

RECORD OF BOREHOLE G

LOCATION

See Figure 1

BORING DATE JAN. 23-27, 1964

DATUM GEODETIC

BOREHOLE TYPE

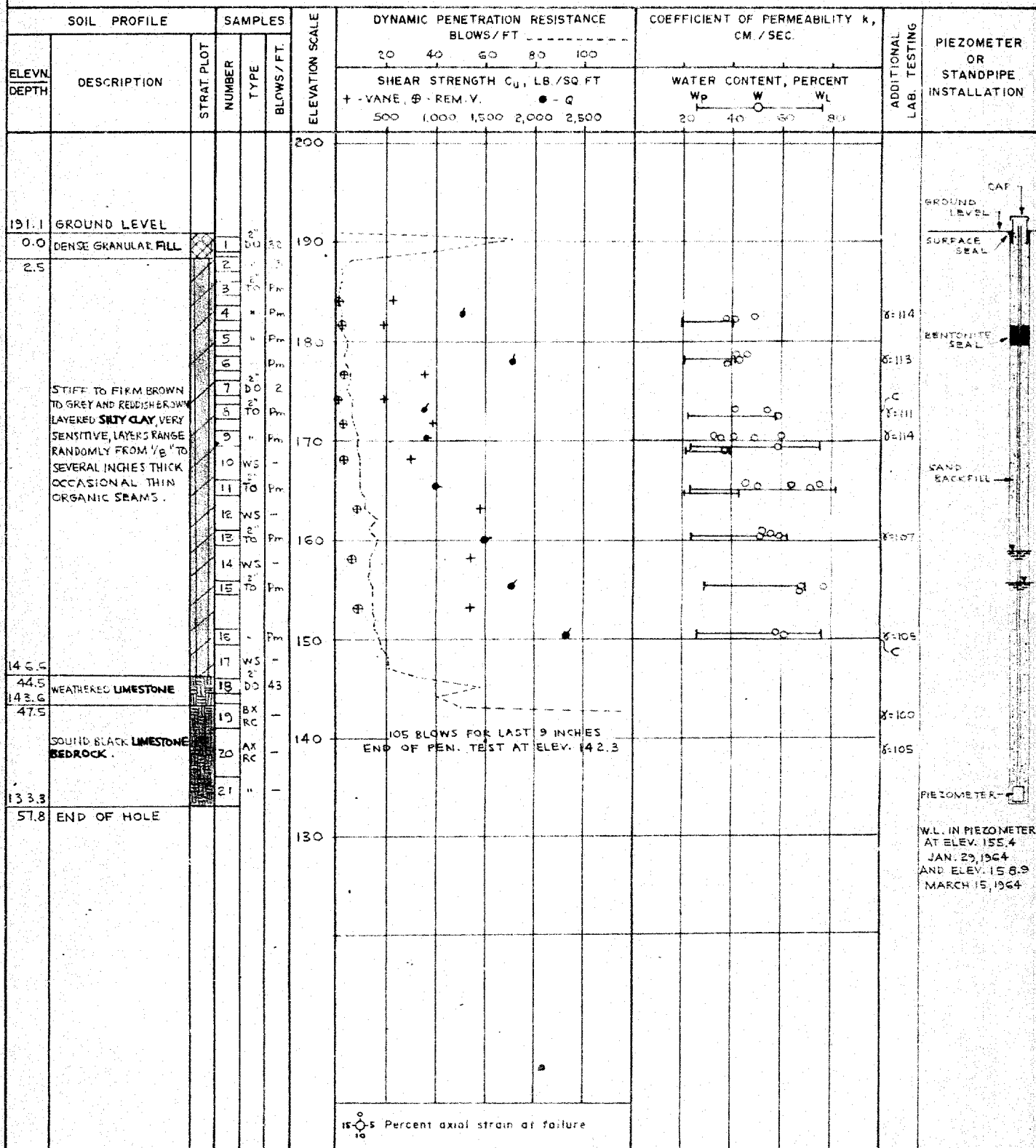
WASH BORING

BOREHOLE DIAMETER NX, BX & AX CASING

SAMPLER HAMMER WEIGHT 140 LB.

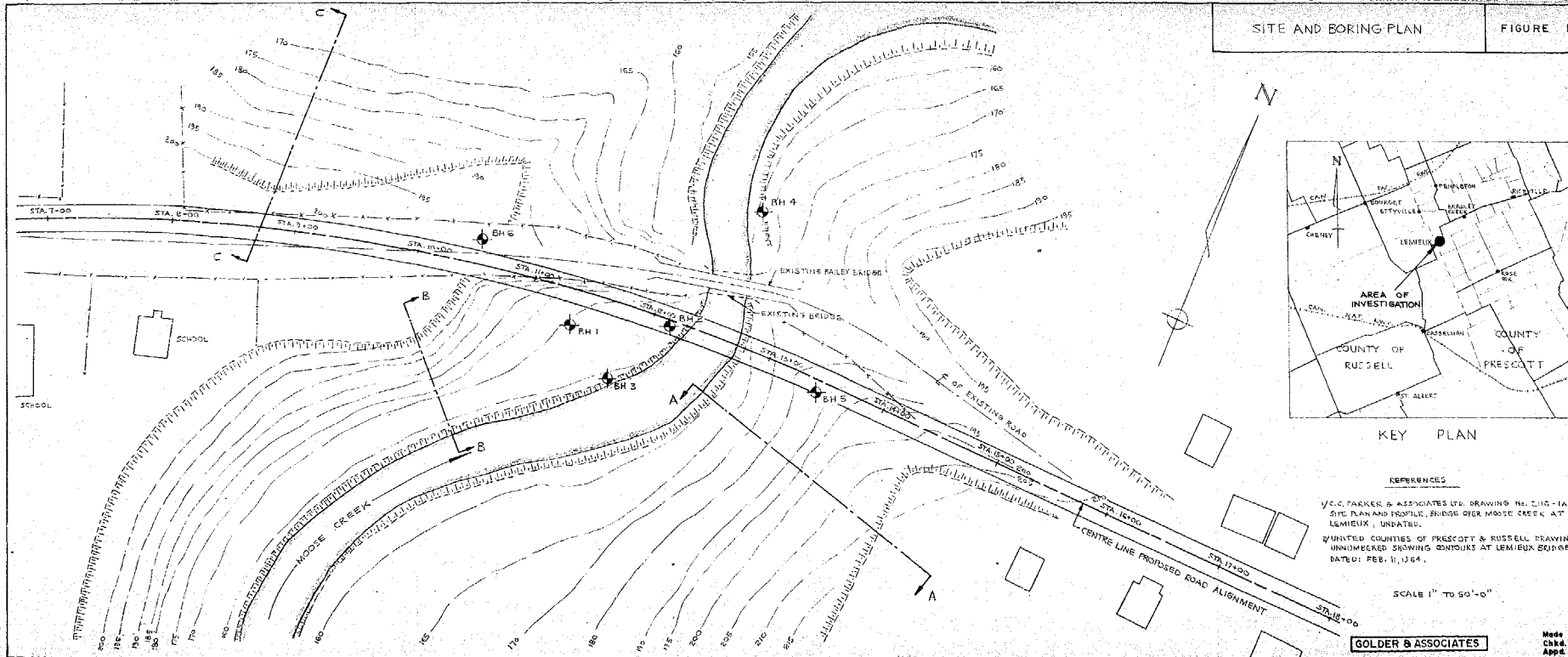
DROP 30 INCHES

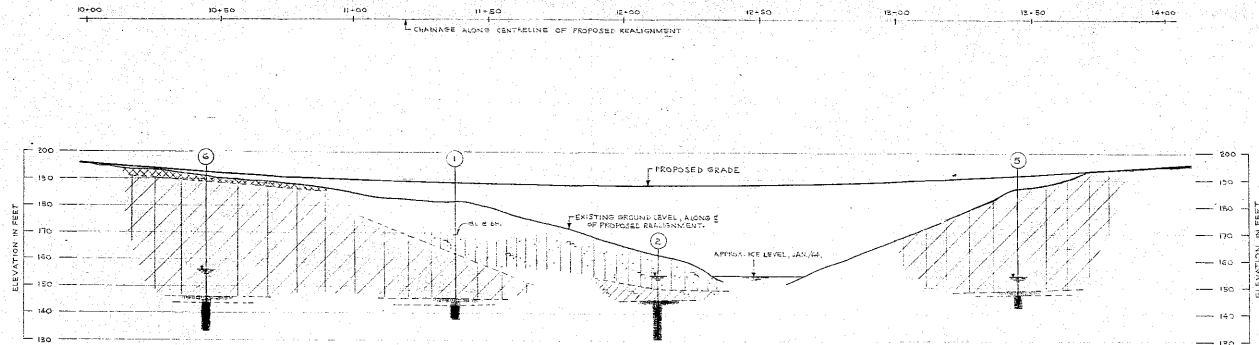
PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *M.W.*
CHECKED *M.W.*





SCHEMATIC SECTION ALONG CENTRELINE OF PROPOSED ROAD REALIGNMENT

SCALE 1" TO 20'-0"

LEGEND

- BOREHOLE - IN ELEVATION
 WATER LEVEL IN BOREHOLE, JAN. 29, 1944.

STRATIGRAPHY

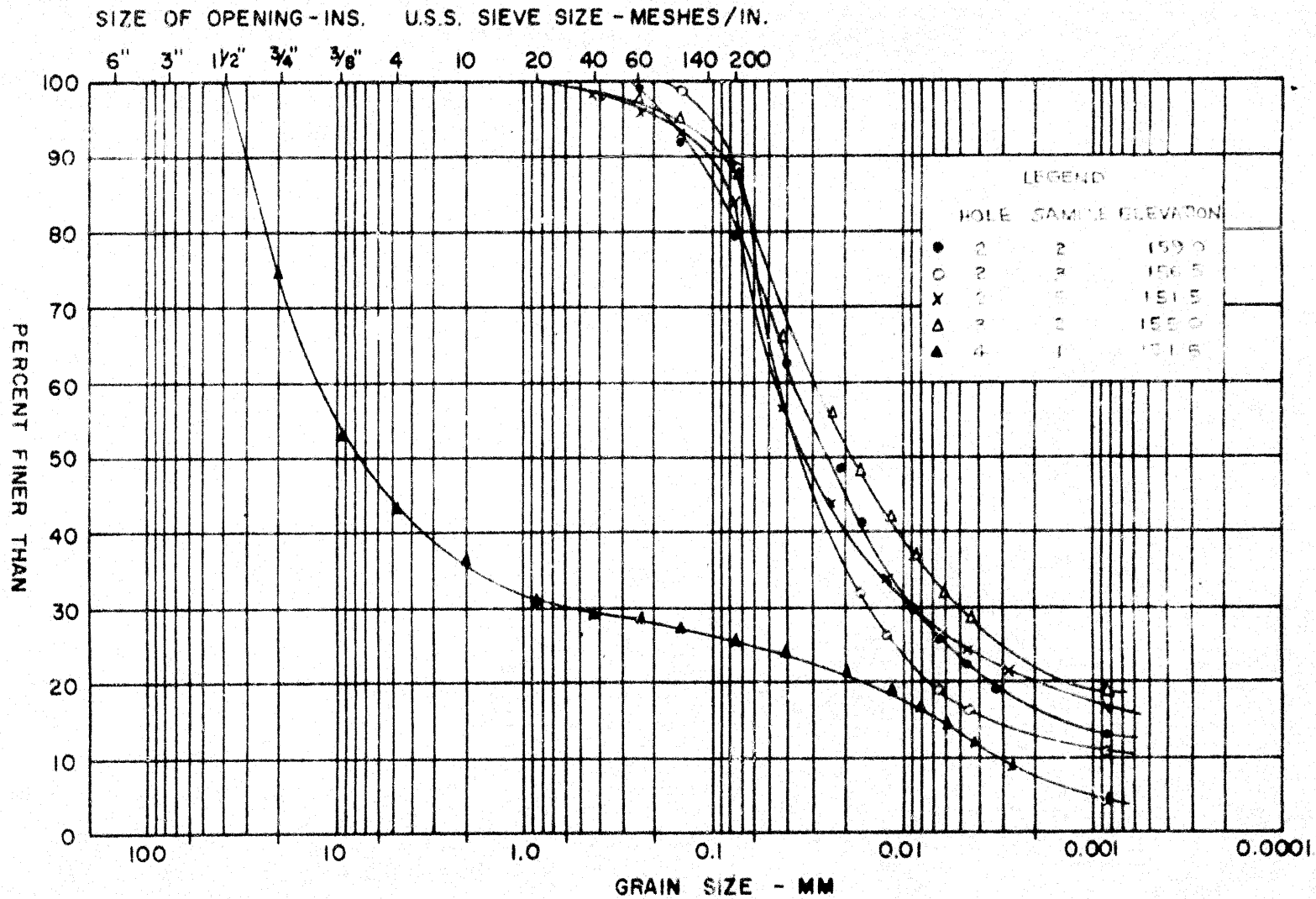
- DENSE GRANULAR FILL
 VERY LOOSE TO COMPACT BROWN SANDY SILT, TRACES OF ORGANIC MATERIAL.
 SOFT TO FIRM GREY ORGANIC SILTY CLAY.
 GENERALLY FIRM TO STIFF BROWN TO GREY AND REDDISH BROWN SILTY CLAY, VERY SENSITIVE, SOME ORGANIC ZEOLITE.
 WEATHERED LIMESTONE.
 SOLID BLACK LIMESTONE BEDROCK.

REFERENCE

C. C. PARKER AND ASSOCIATES LTD. DRAWING No. 215-1, SITE PLAN AND PROFILE, BRIDGE OVER MOOSE CREEK AT LENEXA, DNE DATED: MARCH 20, 1944.

SPECIAL NOTE: DATA OBTAINING THE VARIOUS STRATIGRAPHIC UNITS OBTAINED AT BOREHOLE LOCAL TOWN CLAY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLE HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

M.I.T. GRAIN SIZE SCALE



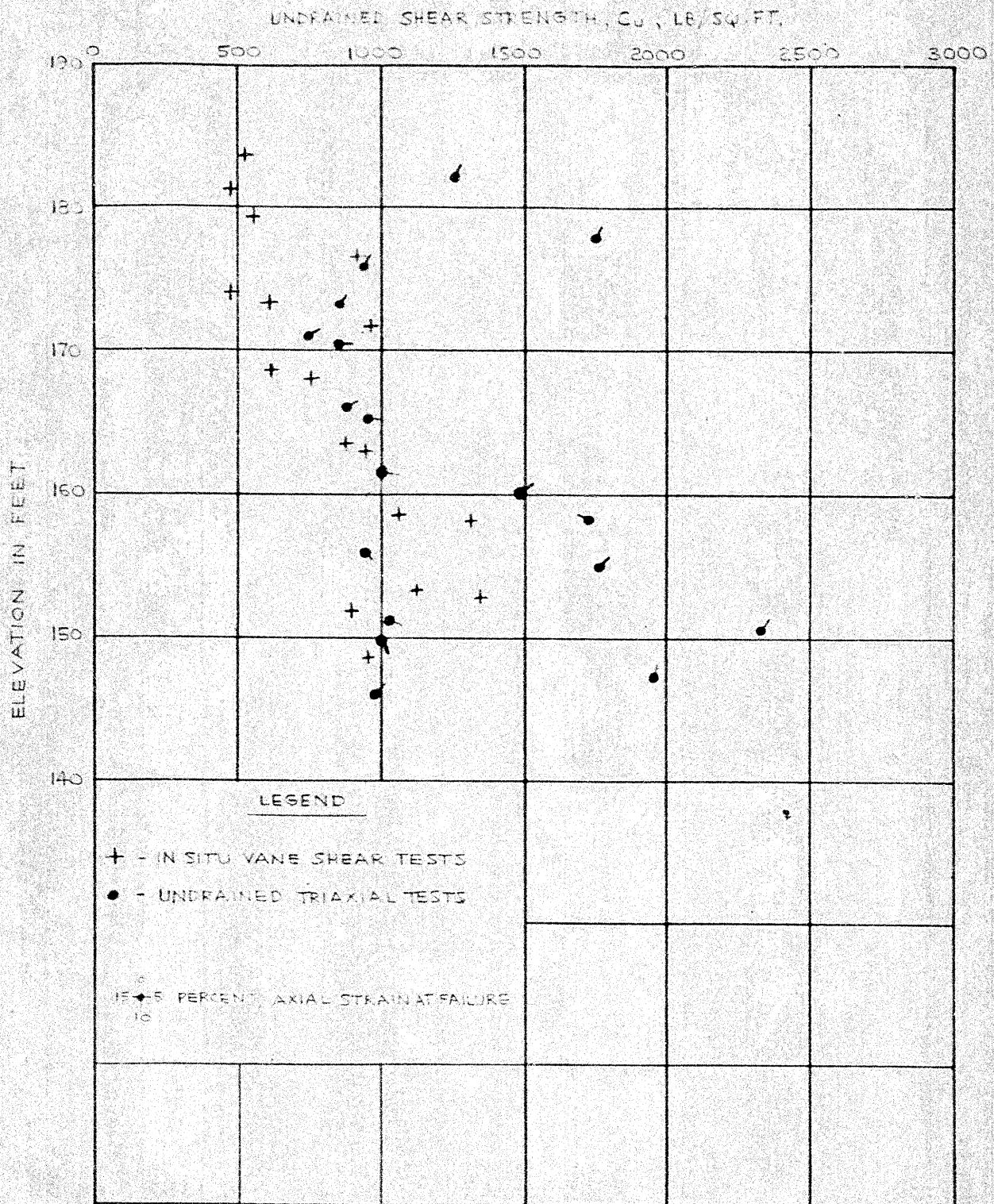
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

FIGURE

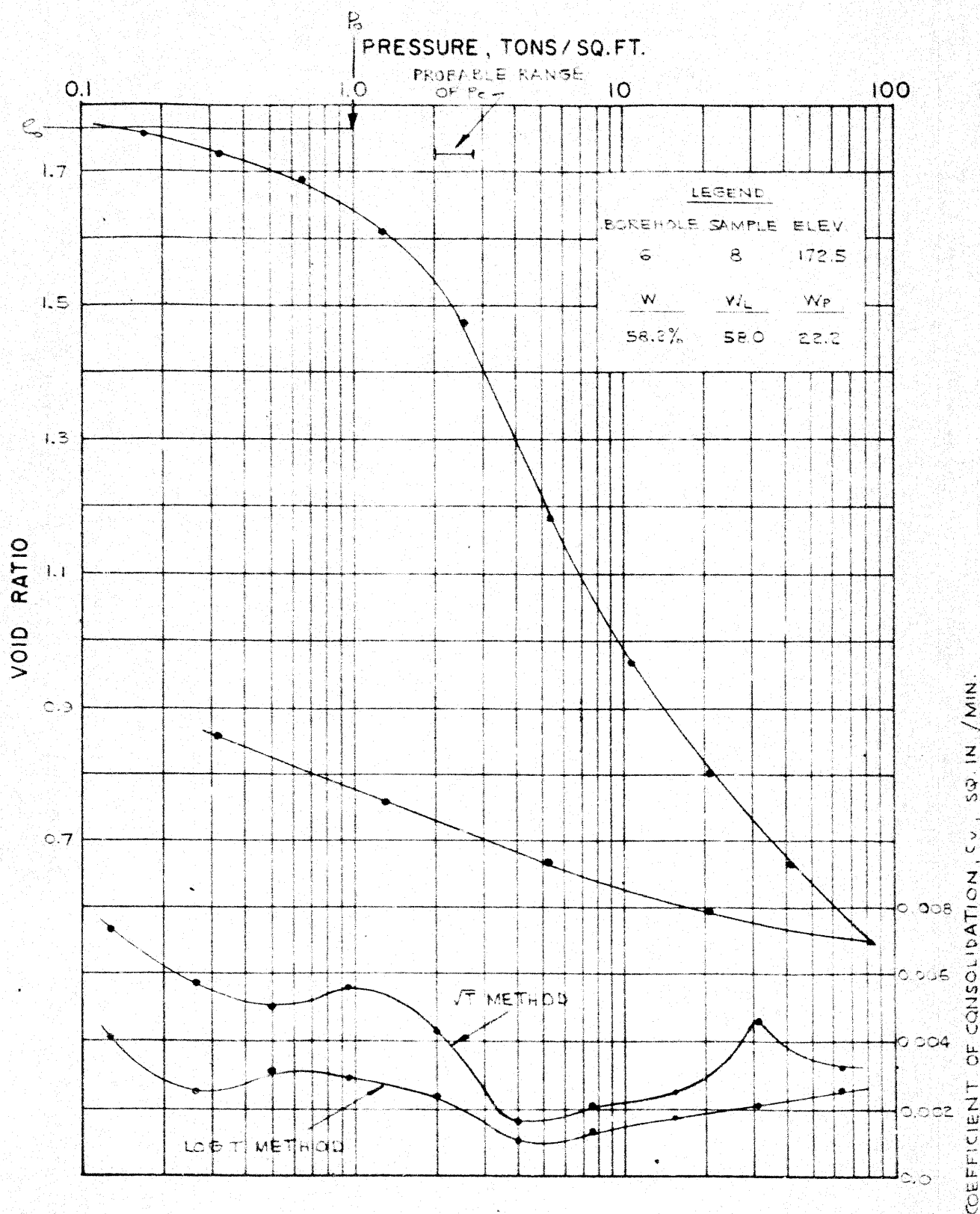
3

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



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

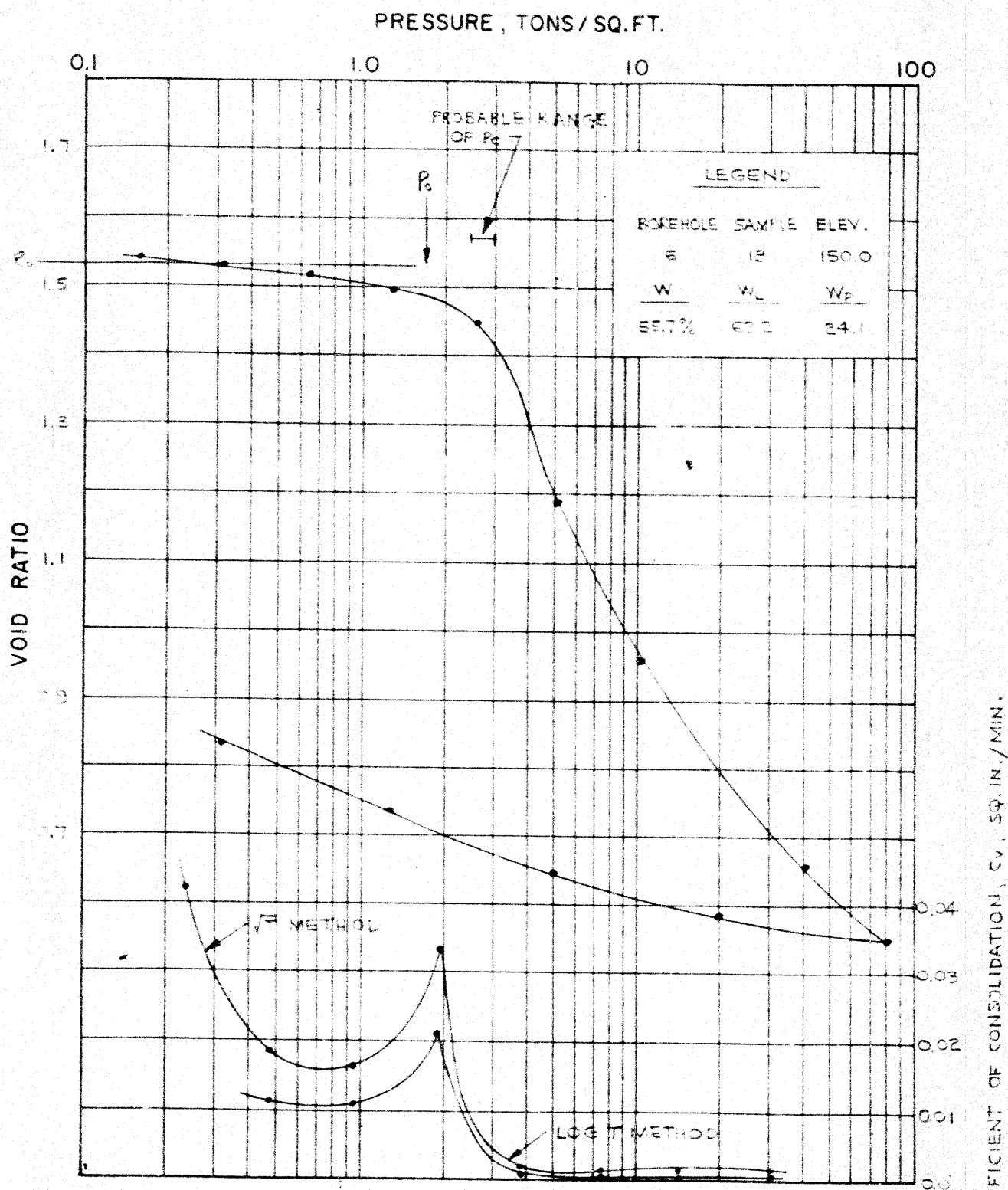
FIGURE 5



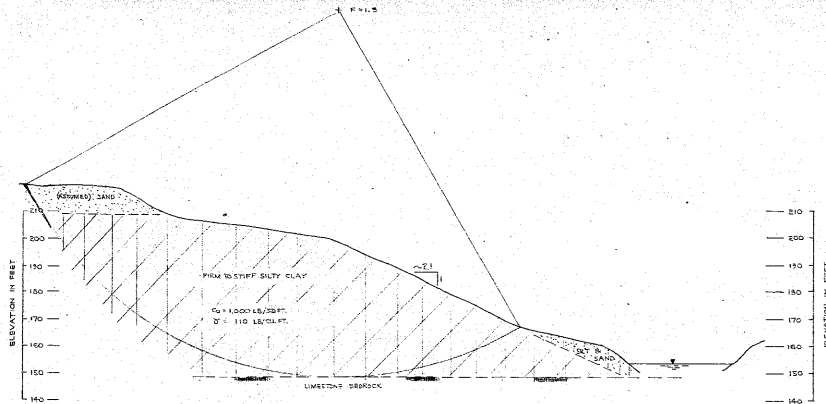
GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

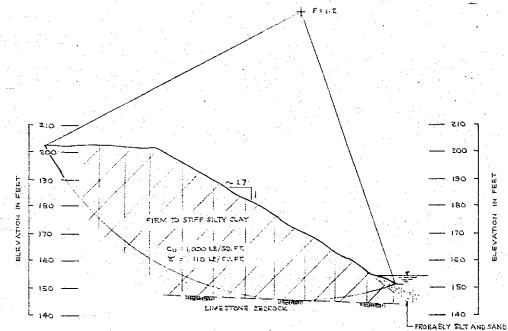
FIGURE 6



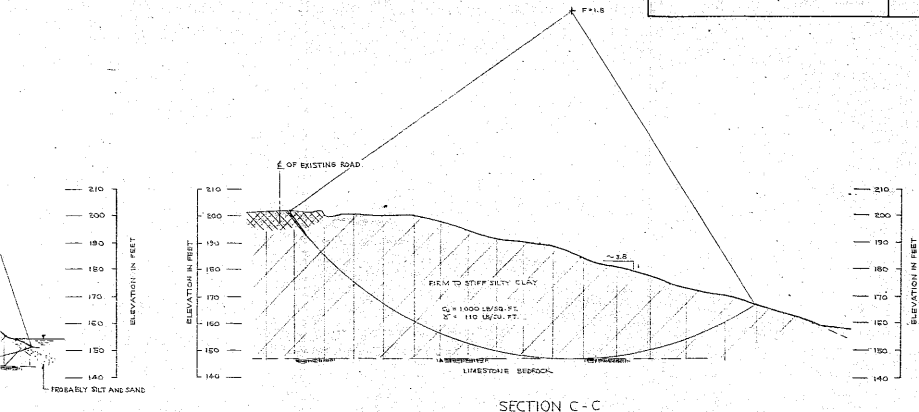
GOLDER & ASSOCIATES



SECTION A-A



SECTION B-B

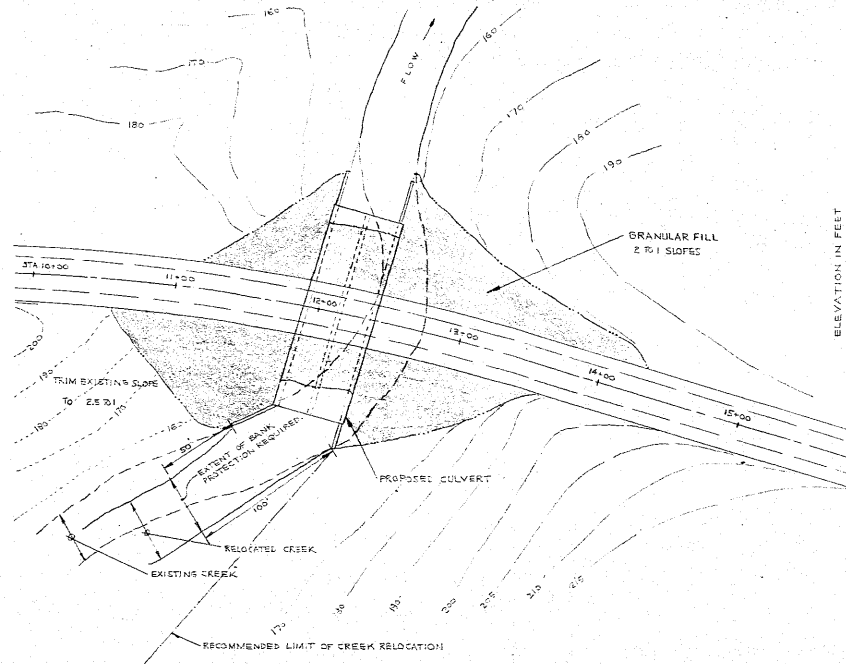


NOTES

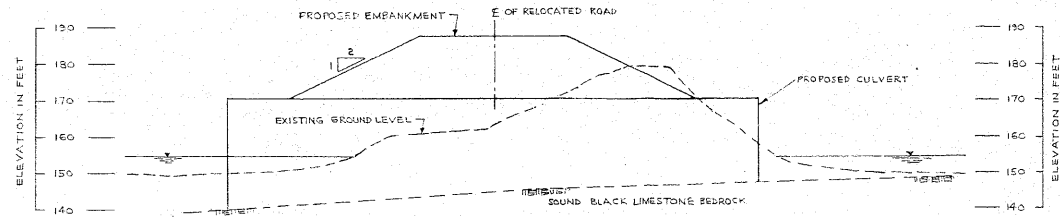
- 1/ SEE FIGURE 1 FOR LOCATIONS OF SECTIONS.
- 2/ SURFICIAL INSTABILITY OBSERVED AT SECTION A-A.
- 3/ DEFINITE INDICATIONS OF INSTABILITY OBSERVED AT SECTION E-E.
- 4/ SECTION C-C APPEARED TO BE STABLE IN ALL RESPECTS.

SCALE 1" TO 20'-0"

GOLDER & ASSOCIATES



SCALE 1" TO 50' - 0"

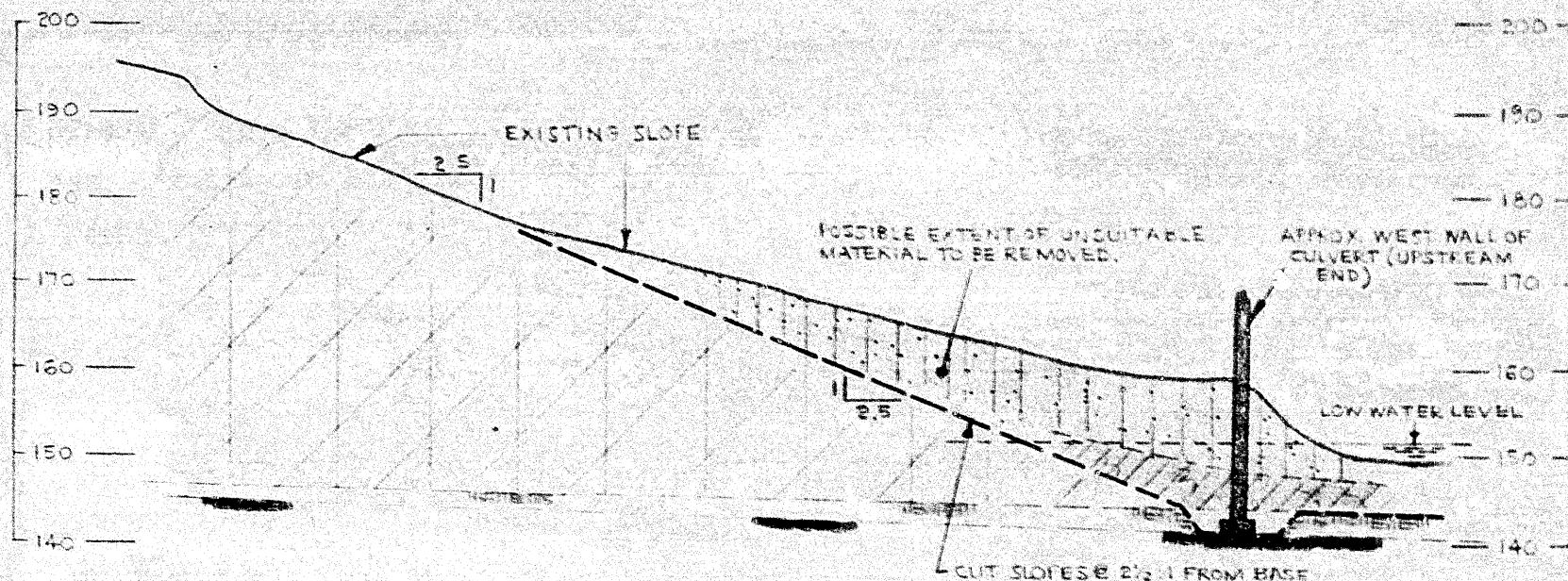


SCHEMATIC SECTION ALONG CENTRELINE OF CULVERT

SCALE 1" TO 20' - 0"

REFERENCES






1. C.C. PARKER AND ASSOCIATES LTD. DRAWING No. 2116-1, SITE PLAN AND PROFILE, BRIDGE OVER MOOSE CREEK AT LEMIEUX, DATED: MARCH 10, 1964.
2. UNITED COUNTIES OF PRESCOTT AND KUSSEL UNNUMBERED DRAWING, LEMIEUX BRIDGE, CONTOUR PLAN, DATED: FEB. 11, 1964.



SCHEMATIC SECTION THROUGH WEST BANK AT UPSTREAM
END OF CULVERT

SCALE 1" TO 20'-0"

LEGEND

- | | |
|--|--|
|  STIFF TO FIRM BROWN TO GREY SILTY CLAY |  WEATHERED LIMESTONE |
|  LOOSE TO COMPACT SANDY SILT |  LIMESTONE BEDROCK |
|  SOFT ORGANIC CLAY | |