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H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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REPORT

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TO

DE LEUW, CATHER & COMPANY OF CANADA LTD. "ED

ON

SITE INVESTIGATION

PROPOSED CANAL ROAD BRIDGE NO. 38

STAGE IV INTERCHANGE

OTTAWA QUEENSWAY, W.P. 954-59

OTTAWA

ONTARIO

Distribution:

**10 copies - De Leuw, Cather & Company of Canada Limited,
Ottawa, Ontario.**

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Toronto, Ontario.**

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64006

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ABSTRACT

The results of a site investigation carried out for the proposed Canal Road Bridge No. 38 and associated retaining wall to the south on the Queensway in Ottawa, Ontario, are reported.

It was found that the site is covered by some 12 to 19 feet of heterogeneous fill. The fill over most of the site is essentially granular in composition and very loose to loose, but varies at a few borehole locations to a soft to firm silty clay with depth. The fill is underlain by a thin layer of geologically recent loose to compact sand or directly by a stratum of firm to stiff silty clay to clayey silt between about 5 and 10 feet thick. Loose to compact silt, some 6 to 11 feet thick, underlies the silty clay followed by a stratum of sandy silt glacial till, 10 to 20 feet in thickness. The upper portion of the till is in a loose to compact state of packing and is a softened and reworked material. The till in the lower few feet is unmodified and dense to very dense. Shale bedrock underlies the till some 40 to 50 feet below ground surface.

The groundwater level across the site during February, 1964, was found to be within the fill and some 7 to 14 feet below ground surface.

It is recommended that the bridge structure and retaining wall be founded on steel 'H' piles driven to practical refusal in the bedrock underlying the site.

The significant deposit at the site which will control the overall stability of the approach embankments, bridge abutments and retaining wall is the very loose to loose heterogeneous fill. To ensure stability, prevent lateral movements and minimize settlement of approach embankments, it is recommended that the existing fill be removed in the bridge abutment area and along the retaining wall and replaced with compacted granular fill.

In areas where the existing fill will not be removed, it is recommended that the approach embankments be constructed in advance of bridge construction and surcharged where possible.

INTRODUCTION

H. Q. Golder & Associates Ltd. has been retained by De Leuw, Cather & Company of Canada Limited, Consulting Engineers, to carry out a final soil investigation at the site of the proposed Canal Road Bridge No. 38 on the Ottawa Queensway in Ottawa, Ontario. The purpose of this investigation was to determine the subsoil conditions at the site and to provide information for the foundation design of the proposed bridge structure, retaining wall and approach embankments.

The results of a preliminary site investigation carried out previously to determine the general soil conditions for the proposed Queensway-Nicholas Street Interchange in this area are presented in our report 6339, dated December, 1963.

PROCEDURE

The field work for this investigation was carried out during the period January 23, 1964, to February 4, 1964, using mobile power auger and diamond drilling equipment. A total of 7 boreholes (numbered 102 to 108, inclusive) was put down across the site to depths ranging between 22 and 64 feet. Four of the borings were put down at the proposed bridge pier and abutment locations and the remaining 3 holes along the line of the proposed retaining wall. A dynamic penetration test was carried out adjacent

to 6 of the boreholes and a piezometer or standpipe installed in 5 of the boreholes following their completion to determine the groundwater level.

A detailed log for each borehole is given on the Records of Boreholes at the end of this report. The locations of all the borings put down in this investigation together with the location of a pilot borehole put down at this bridge site during the previous investigation are shown on Figure 1. A section of the inferred soil stratigraphy across the site is also given on Figure 1.

The samples obtained during this investigation were brought to our laboratory for examination and testing. The results of the laboratory testing are shown on the Records of Boreholes and on the figures.

All elevations in this report were supplied by De Leuw, Cather & Company of Canada Limited and are referred to Geodetic datum.

SITE TOPOGRAPHY AND GEOLOGY

The bridge site is located at about chainage 449+00 on the proposed Queensway between the Rideau Canal and the Rideau River in Ottawa, Ontario. The Queensway in this locality follows

an abandoned railway right of way on an embankment some 5 feet high. The general ground surface across the site slopes down in a southeasterly direction from about elevation 205 on the north side of the railway right of way to about elevation 195 on the south side. To the north of the Queensway centreline the site is covered by numerous dumped piles of debris and rubbish.

Geological information indicates that bedrock at the site is an Ordovician shale of the Billings formation. The bedrock in this area is covered by glacial deposits laid down during the Wisconsin stage of glaciation. The glacial deposits consist chiefly of sandy glacial till. The till includes much reworked material that grades downward into unmodified till.

Following retreat of the ice sheet, the area was invaded through the Ottawa River and Tributary Valleys by an arm of the ocean known as the Champlain Sea. During this period of submergence, silts and clays were laid down over the glacial till. These marine deposits were then exposed by subsequent uplift which occurred after retreat of the glaciers. In certain areas clay of non-marine origin covers the marine clay. This younger clay probably was derived in great part from the marine clay by wave and current action during the last stages of the Champlain Sea and during subsequent estuarine and fluvial action. Geologically

recent deposits of sand, silt and peat form the surface cover in localized areas.

SOIL CONDITIONS

The borings show that the site is covered by fill ranging between about 12 and 19 feet in thickness. The elevation of the base of the fill is about 183 to 184, except in borehole 107 in the southern portion of the site where it drops off to about 175. The fill is variable in composition ranging from essentially a granular to a clayey material. The granular portion of the fill, which covers the major portion of the site, consists mainly of silty sand but varies to an organic sandy silt and includes chunks of clay, cinders, gravel, ashes, pieces of wood and rubbish. In boreholes 103, 104 and 107 the lower 5 to 8 feet of the fill is clayey and is comprised of chunky sensitive silty clay with pockets of sand and silt and occasional organic matter. Grading curves for several samples of fill are shown on Figure 2.

The heterogeneous granular fill, based on the standard penetration resistance values given on the Records of Boreholes, ranges between very loose and compact and is generally in a loose state of packing. Three triaxial compression tests were carried out on samples of the clay fill. The results of these

tests, which gave a range in the undrained shear strength value from about 400 to 1,300 lb/sq.ft., are given on the Records of Boreholes. The strength tests, together with one field vane test result of 1,400 lb/sq.ft., indicate that the clay fill varies in consistency between soft to stiff. However, the higher values obtained may represent the strength of relatively undisturbed chunks or blocks of clay and the general overall consistency of the clay fill is probably in the soft to firm range.

The fill in borehole 105 and in borehole 4 put down during the previous investigation is underlain by a geologically recent deposit of loose to compact grey silty sand, some 1 to 3 feet in thickness.

Beneath the fill deposit covering the site or the thin layer of sand, a stratum of silty clay to clayey silt was encountered in all the borings. This stratum is between about 5 and 10 feet in thickness and extends down to about elevation 177 at the bridge location dropping off to about elevation 171 at borehole 107 to the south.

The clay stratum has a variable silt content and ranges in composition between a silty clay and clayey silt. A grading curve for one sample of the clay is shown on Figure 3. The

stratum contains thin layers of silt and sand and on occasion exhibits a layered structure consisting of alternate layers of silty clay and clayey silt to silt, each about 1/4 to 1 inch in thickness.

Atterberg limits carried out on samples of the clay gave a range in the liquid limit from about 23 to 57 with the corresponding range in the plasticity index between about 5 and 35. The liquidity index of the clay ranges between about 0.2 to 2.6 and is commonly of the order of 1.0 to 1.5.

The strength of the clay was measured by in situ vane testing in the field and by laboratory triaxial compression tests on relatively undisturbed samples. The shear strength results obtained, which range from 600 to about 2,000 lb/sq.ft., are given on the Records of Boreholes. In general, however, the range of shear strength is of the order of 800 to 1,500 lb/sq.ft., which is in the firm to stiff range of consistency. The sensitivity of the clay, as determined by the field vane measurements, is between about 4 and 10.

The clay across the site is underlain by a stratum of grey silt some 6 to 11 feet thick. Several grading curves for samples of the silt are given on Figure 4. These show that the stratum is comprised essentially of silt with a trace to some clay

and fine sand. Based on the standard penetration test results the silt is loose to compact and generally compact.

Underlying the silt and extending to bedrock at about elevation 147 to 154 is a stratum of dark grey glacial till about 10 to 20 feet in thickness. The glacial till is comprised of a well graded composite of silt, sand and gravel with a trace to some clay and contains occasional cobbles. Grading curves for a number of samples of the till, obtained using 1½ inch diameter sampling equipment, are shown on Figures 5 and 6.

The results of standard and dynamic penetration tests carried out in the till, with recorded 'N' values as low as 2 blows per foot, show that the upper portion of the stratum down to about elevation 155 is loose to compact and generally compact. This indicates that the till in the upper portion is a softened and reworked material. In the lower few feet of the stratum, the till is unmodified and in a dense to very dense state of packing with 'N' values generally in excess of 50 blows per foot.

The bedrock underlying the till is a dark grey shale. The shale is generally sound, except in boreholes 105 and 107 where it was found to be weathered and partially fractured in the upper 6 to 9 feet.

GROUNDWATER CONDITIONS

Piezometers or water level observation pipes were installed in 5 of the boreholes put down in this investigation. The details of the installation are given on the Records of Boreholes. Periodic readings were taken in these installations during February, 1964, and a set of results are recorded on the Records of Boreholes and on the stratigraphy section on Figure 1.

The readings showed that the groundwater level during February, 1964, was at about elevation 188 at the bridge structure location dropping off to about elevation 185 at the southern end of the proposed retaining wall. This is some 7 to 14 feet below ground surface and generally within the lower portion of the fill deposit covering the site. Because of the relatively impervious clay stratum underlying the fill, it can be expected that the groundwater level will rise in the spring or during periods of heavy precipitation.

DISCUSSION

General

It is understood that the proposed bridge is to be a 3 span simply supported steel or precast reinforced concrete structure with spill through abutments. The approach embankments to the bridge will be some 20 to 25 feet in height and will be

constructed with side and end slopes no steeper than 2 horizontal to 1 vertical. To accommodate a low level ramp along the east side of the south approach embankment, a retaining wall which will extend south about 350 feet from the southeast wing wall of the bridge is to be provided. The height of the retaining wall above existing ground surface is to be about 20 feet at the bridge abutment decreasing to about 10 feet at its southern end. The approximate extent of the retaining wall is shown on Figure 1.

Foundations

The heterogeneous fill covering the site is not a competent foundation stratum for the support of the bridge and retaining wall structures. The structures could be founded on spread or strip footings placed in the silty clay to clayey silt stratum underlying the fill using a maximum net allowable bearing pressure of 1 ton/sq.ft.; however, the use of a spread or strip footing foundation will necessitate excavation through the fill and below groundwater level. This would be quite difficult. Further, the settlement of the footings would be generally greater than about 2 inches, the differential settlement between an abutment and pier being of the same order. Consequently, it is considered that end bearing piles would give a more suitable and economical foundation for the bridge and retaining wall structures.

A piled foundation would avoid deep sheeted excavation during construction and prevent consolidation settlement of the structures. Steel 'H' piles are recommended in order to minimize disturbance of the sensitive silty clay during driving. Assuming a 12 inch steel 'H' section driven to practical refusal in the shale bedrock underlying the site, an allowable load of about 70 tons per pile may be used for design. It is suggested that the design pile load be confirmed by a load test carried out prior to construction.

For adequate frost protection at least 6 feet of earth cover should be provided above the underside of the concrete pile cap footings.

Approach Embankments

The significant deposit at the site which will control the stability of approach embankments together with the overall stability of the bridge abutments and the retaining wall is the very loose to loose heterogeneous fill. The fill, although variable in composition, is mainly granular in nature, except for the lower portion in several boreholes where it is clayey. There should, in general, be no major problem with the overall stability of approach embankments about 20 feet above existing ground surface at the bridge abutments and behind the retaining wall; however, there could be some settlement and lateral movement of the roadway embank-

ments because of the variable character and very loose to loose density of the fill covering the site.

It would be desirable to eliminate settlement and possible lateral movement immediately adjacent to the bridge abutments and retaining wall structures. To accomplish this, the existing fill should be removed in the bridge abutment area and along the retaining wall and the excavation backfilled with granular fill well compacted in place. The extent of such excavation and replacement is illustrated by the sketch on Figure 7.


In the areas where the existing fill will not be removed, the settlement of embankments could be minimized by constructing them as far in advance of the bridge and retaining wall as possible. This would permit a substantial portion of the movement to take place if the embankments are left in place for about a year before construction of the structures and paving operations. If time of this order is not available, the embankments could be constructed to 150 percent of the final design height to provide a surcharge. It is estimated that the surcharge fill would have to be in place for a period of about 6 months prior to bridge and retaining wall construction. The magnitude and time rate of the movements should be carefully monitored in order to indicate when the major portion of movement has taken place.

Retaining Wall

It is recommended that at least 6 feet, in horizontal extent, of non-frost susceptible and free draining granular material be placed behind the retaining wall which should be founded on piles driven to practical refusal in the bedrock underlying the site. An active earth pressure coefficient, $K_a = 0.3$ can be used for design assuming compacted granular backfill. With full effective drainage provided by the free draining granular material behind the retaining wall, the unit weight of the compacted fill for computing earth pressures on the wall may be taken as 130 lb/cu.ft.



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LIST OF ABBREVIATIONS

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

I. SAMPLE TYPES

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

II. PENETRATION RESISTANCES

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

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

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*

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

(b) *Cohesive Soils*

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

IV. SOIL TESTS

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

NOTES:

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

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

LIST OF SYMBOLS

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Unit weight

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

(b) Consistency

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

(c) Permeability

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

(d) Consolidation (one-dimensional)

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

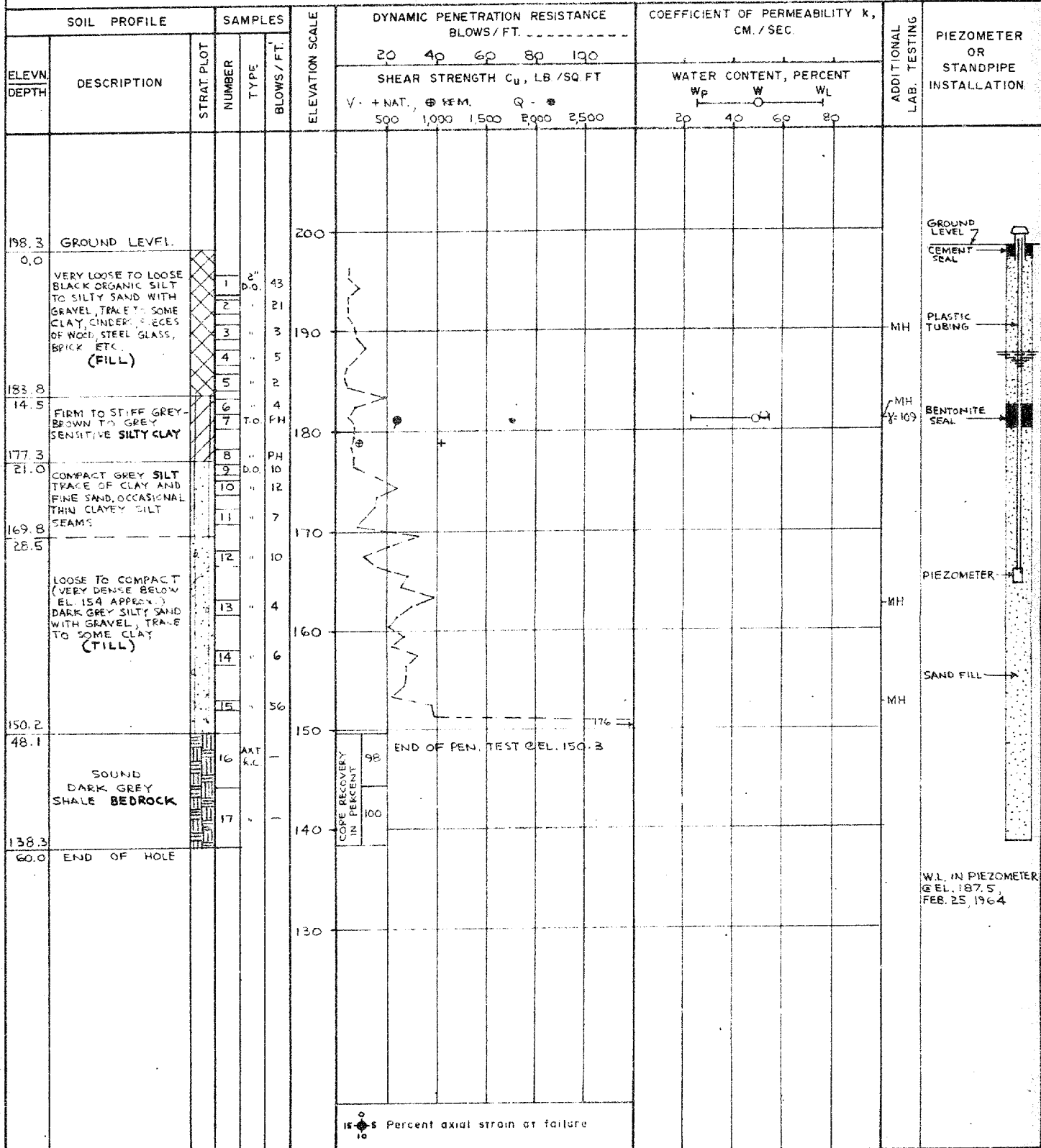
(e) Shear strength

τ_f	shear strength
c'	effective cohesion
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

*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 102

LOCATION See Figure 1 BORING DATE JAN. 27-30, 1964 DATUM GEODETIC
 BOREHOLE TYPE POWER AUGER & WASH BORING BOREHOLE DIAMETER 4.5" & BX CASING
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



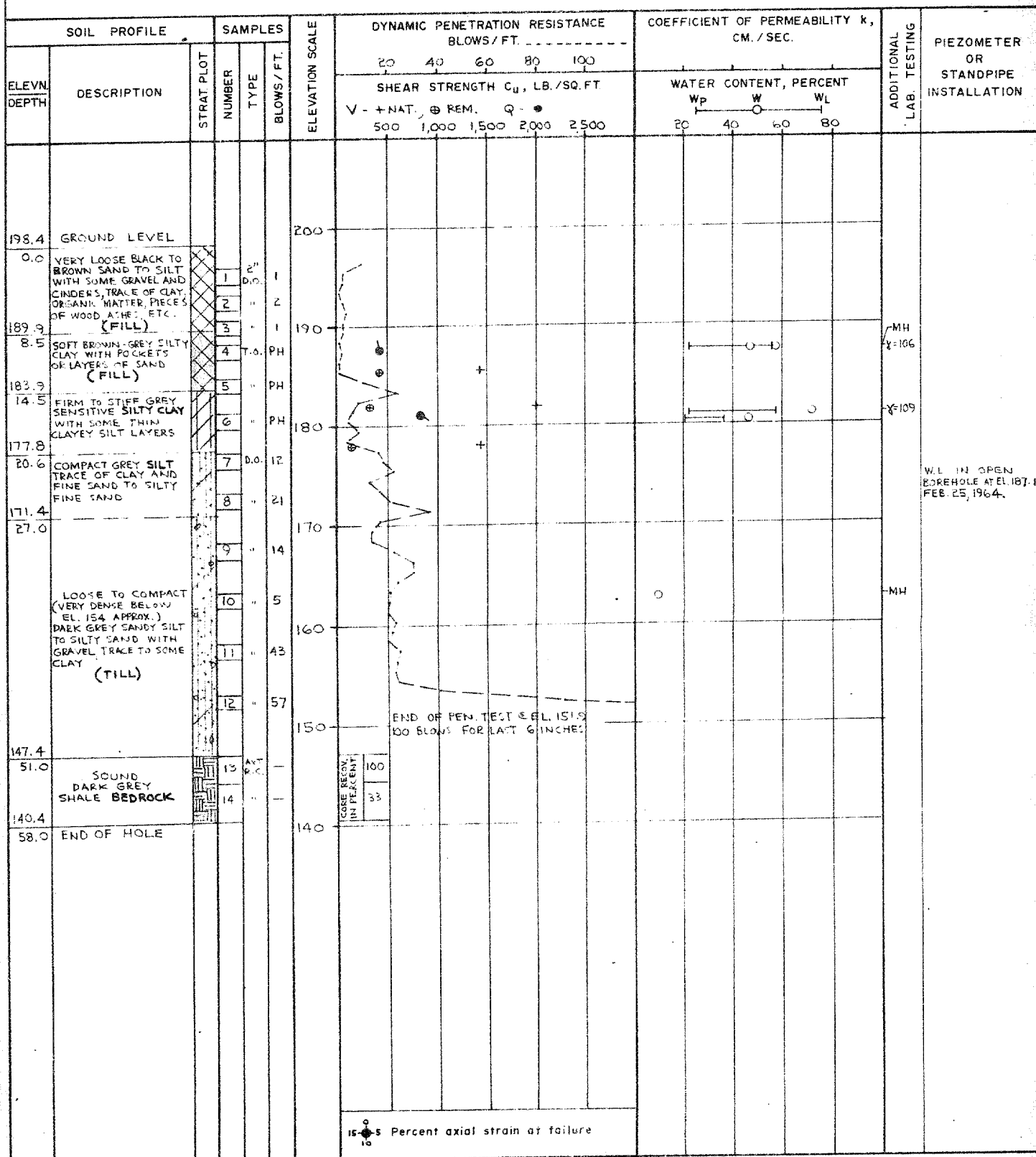
VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED J.A.

RECORD OF BOREHOLE 103

LOCATION See Figure 1 BORING DATE JAN. 28 - FEB. 4, 1964 DATUM GEODETIC
 BOREHOLE TYPE POWER AUGER & WASH BORING BOREHOLE DIAMETER 4.5" & 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.T.

RECORD OF BOREHOLE 104

LOCATION

See Figure 1

BORING DATE JAN. 29-31, 1964

DATUM

GEODETIC

BOREHOLE TYPE

POWER AUGER & WASH BORING

BOREHOLE DIAMETER

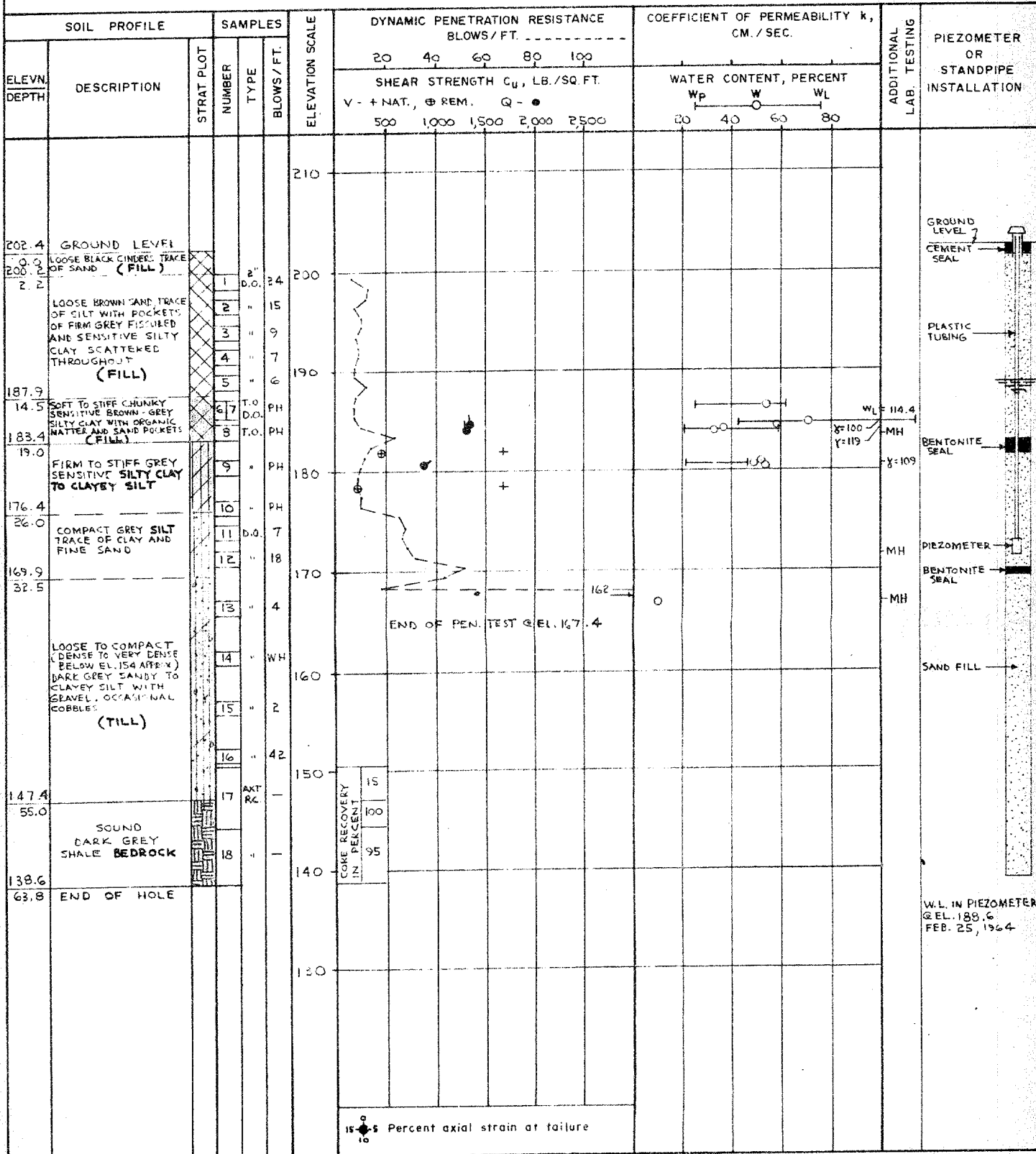
4.5" ϕ BX CASING

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.M.

RECORD OF BOREHOLE 105

LOCATION See Figure 1

BORING DATE JAN. 23-25, 1964

DATUM

GEODETIC

BOREHOLE TYPE

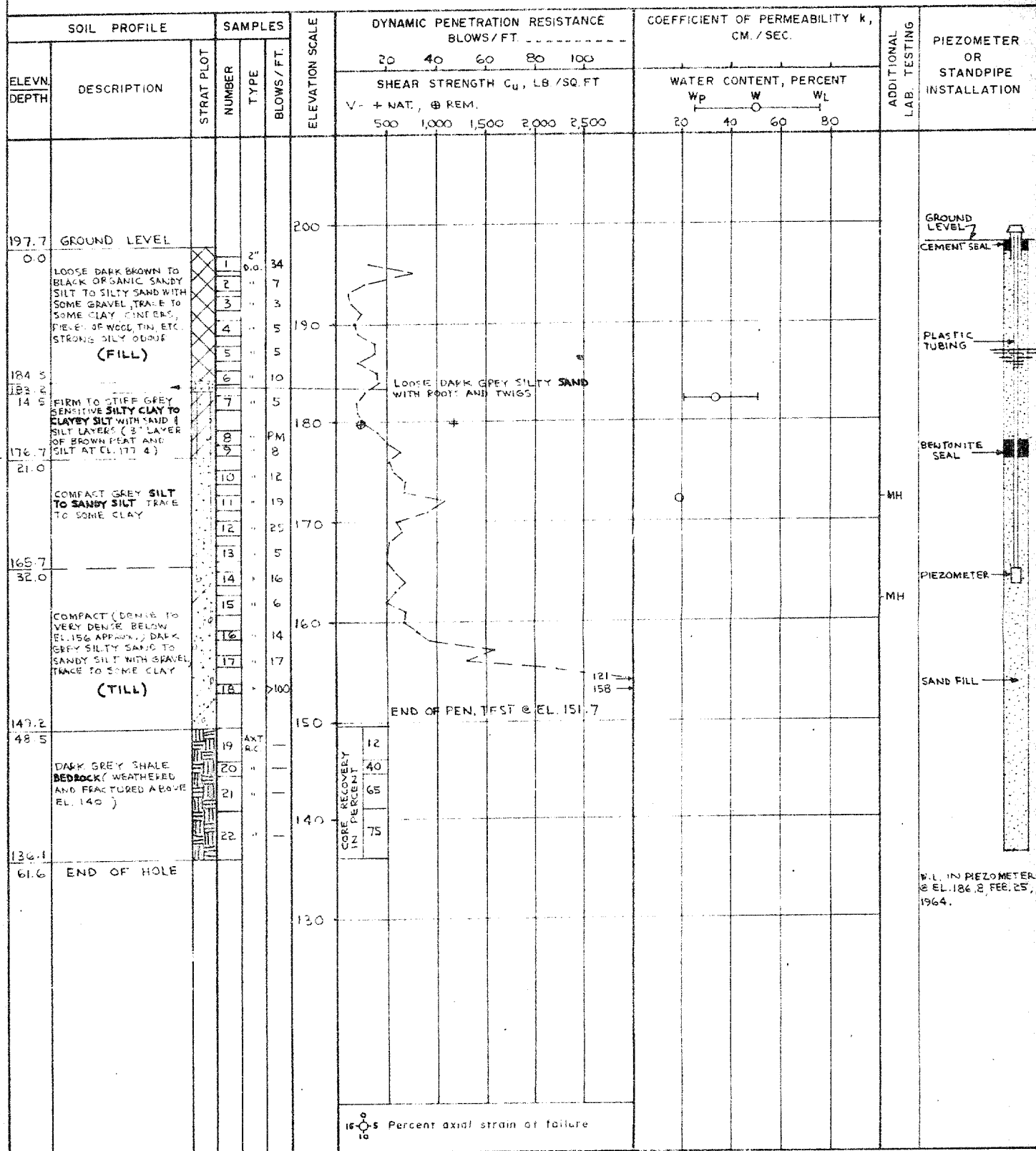
POWER AUGER & WASH BORING

BOREHOLE DIAMETER

4.5" & 8X CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



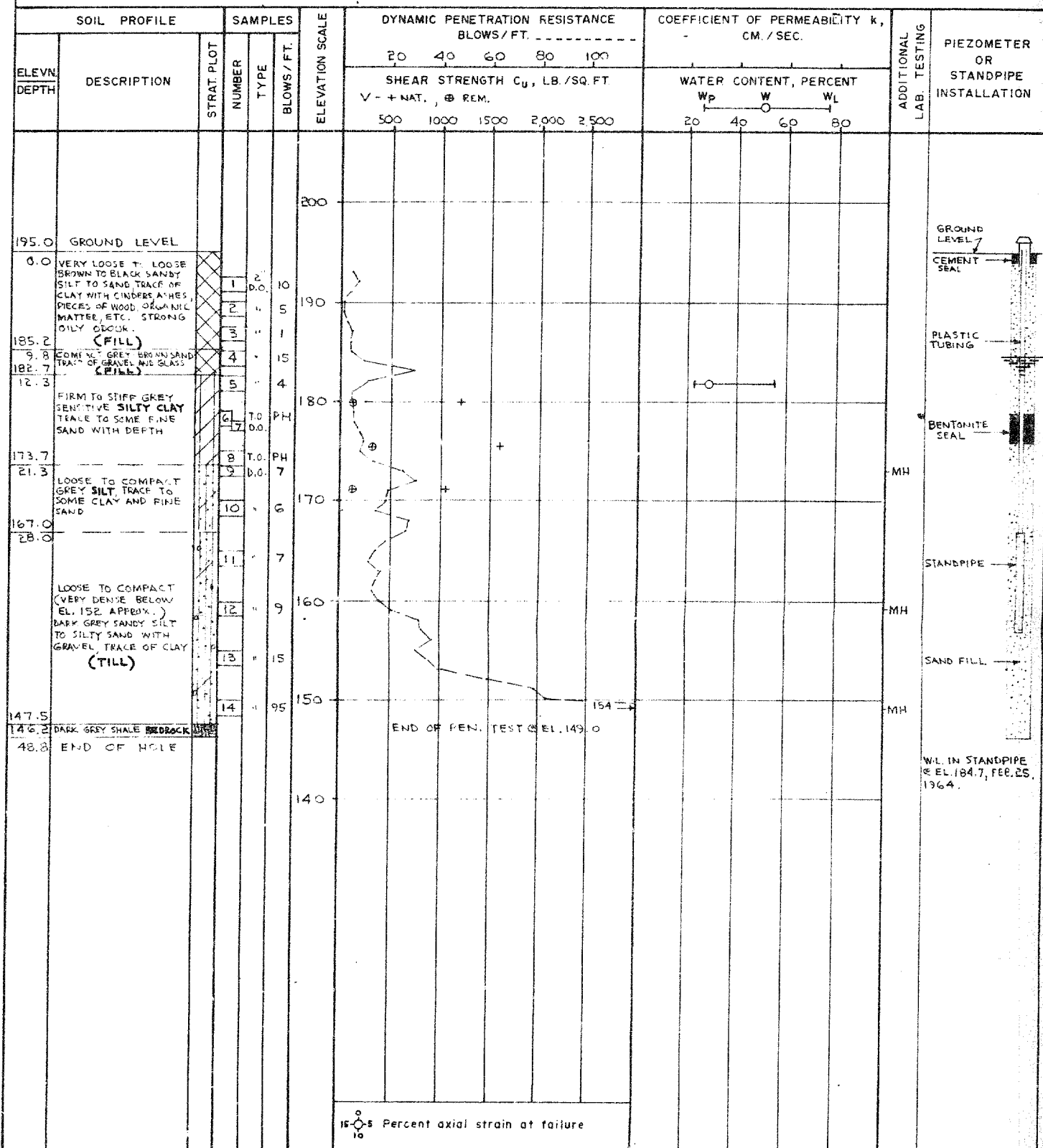
VERTICAL SCALE
1 INCH TO 10 "

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED DTC

RECORD OF BOREHOLE 106

LOCATION See Figure 1 BORING DATE JAN. 30-31, 1964 DATUM GEODETIC
 BOREHOLE TYPE POWER AUGER BORING BOREHOLE DIAMETER 4.5"
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



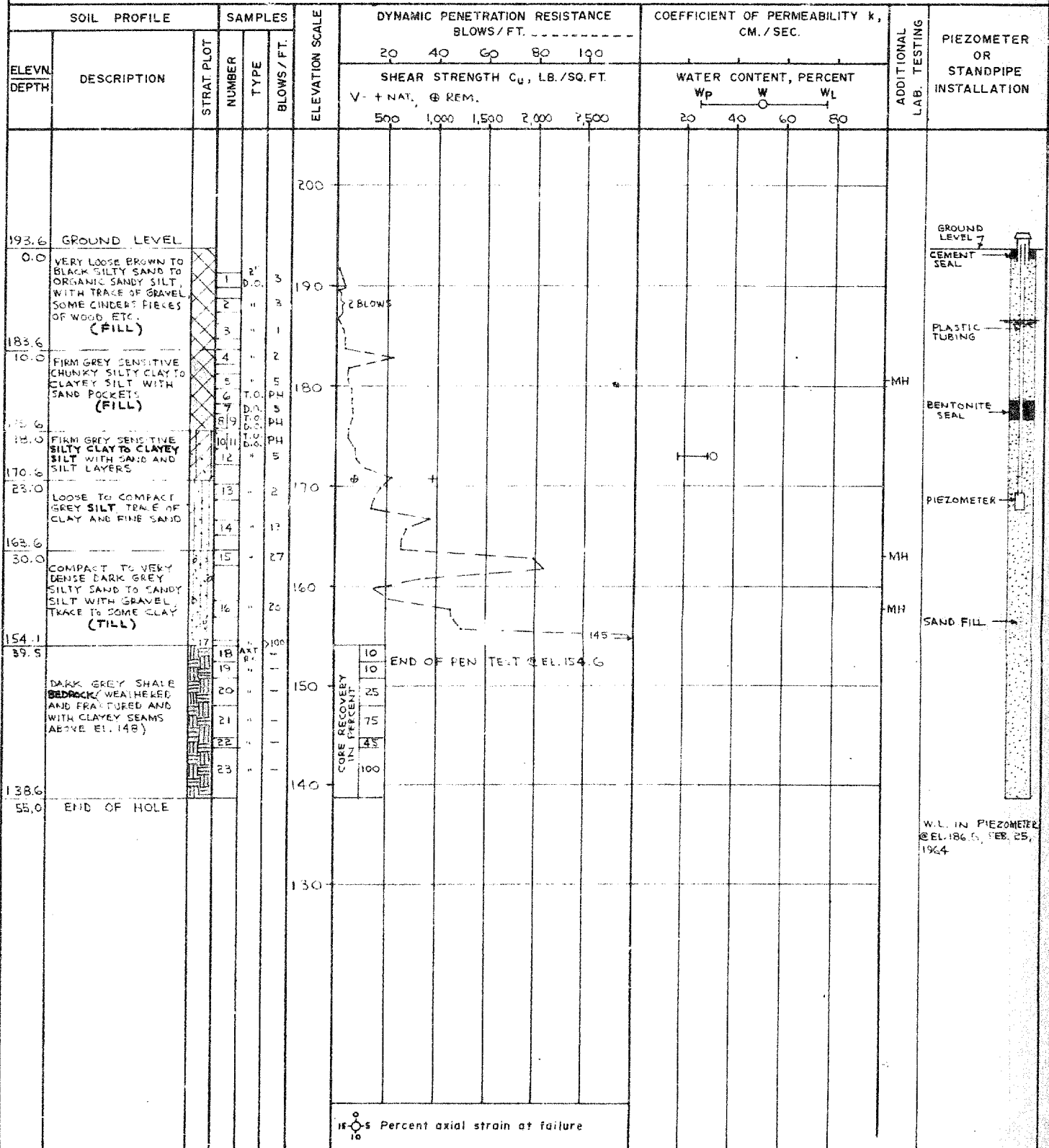
VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
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RECORD OF BOREHOLE 107

LOCATION See Figure 1 BORING DATE JAN 30 - FEB. 3, 1964 DATUM GEODETIC
 BOREHOLE TYPE POWER AUGER & WASH BORING BOREHOLE DIAMETER 4.5" 4 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.A.

RECORD OF BOREHOLE 108

LOCATION See Figure 1

BORING DATE FEB. 3, 1964

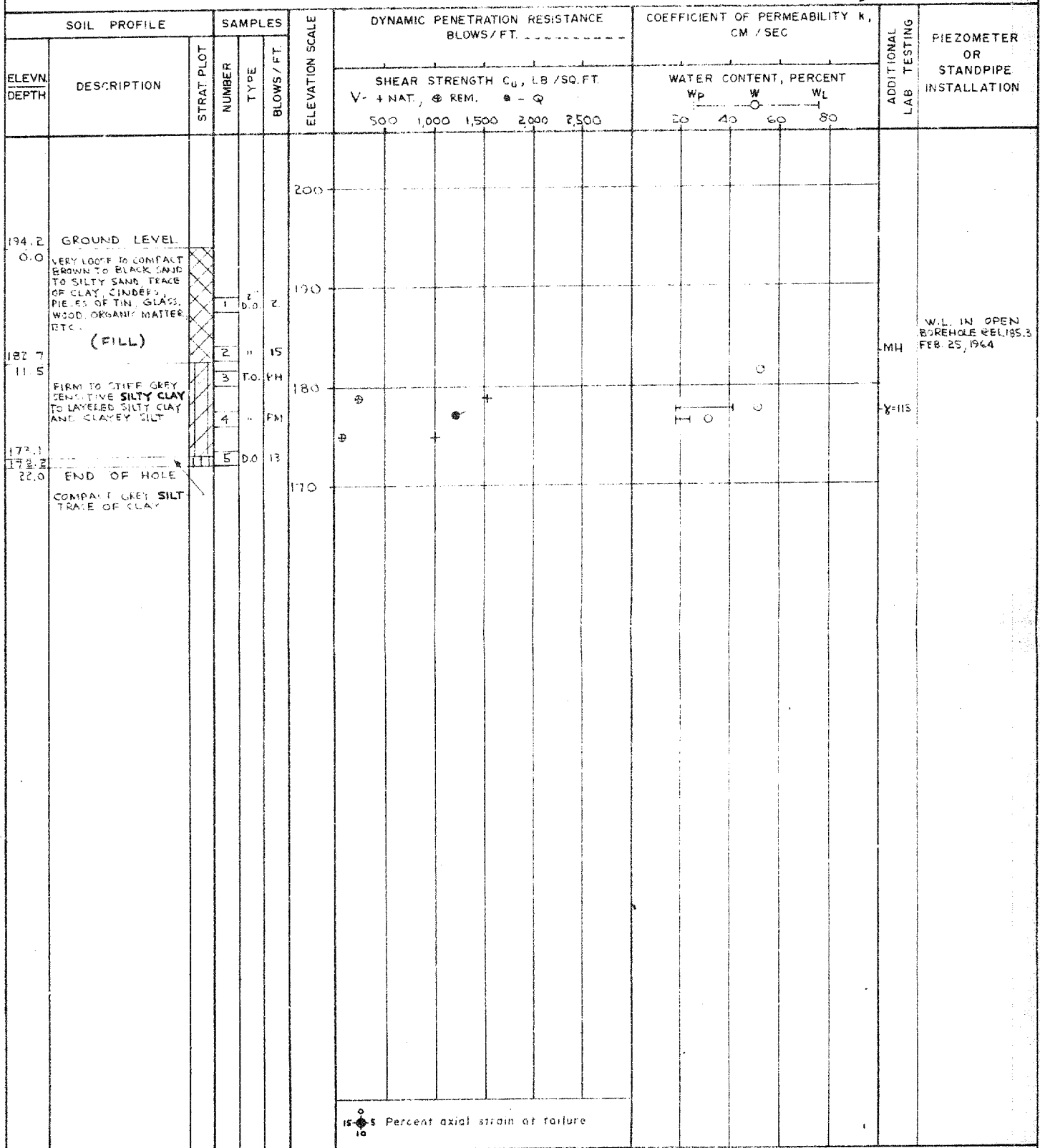
DATUM GEODETIC

BOREHOLE TYPE POWER AUGER BORING

BOREHOLE DIAMETER 4.5"

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES



W.L. IN OPEN BOREHOLE REL. 185.3 FEB. 25, 1964

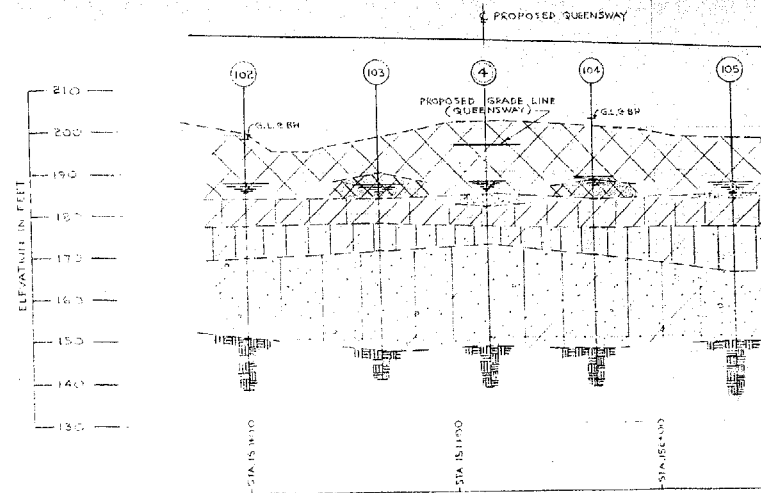
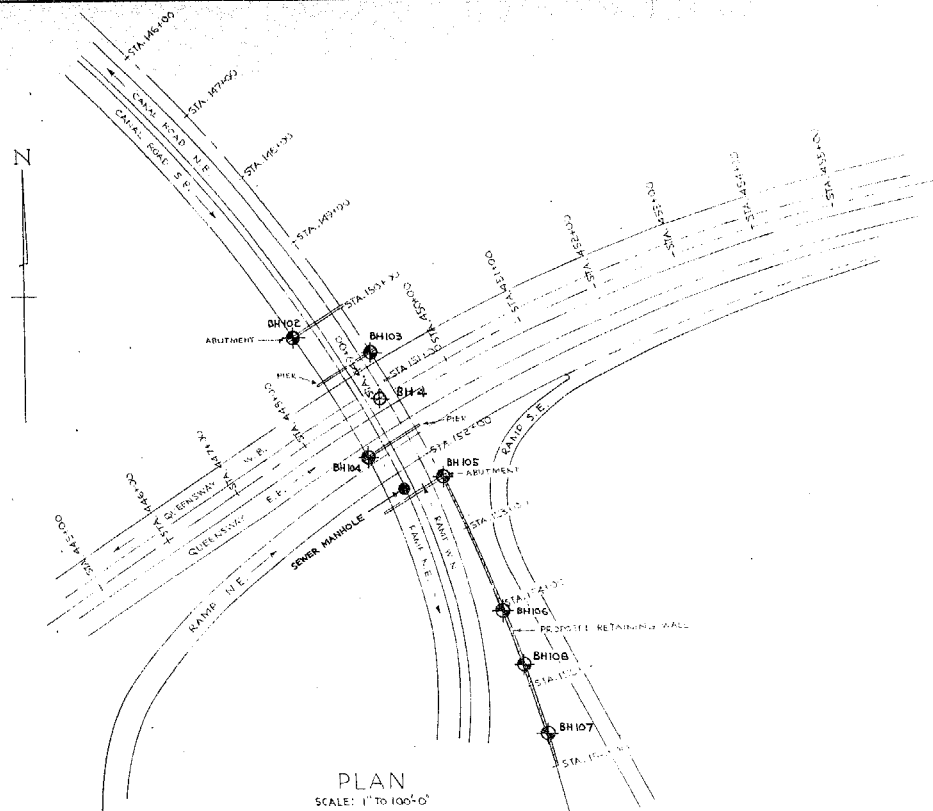
MH

Y=113

VERTICAL SCALE
1 INCH TO 10'-0"






GOLDER & ASSOCIATES

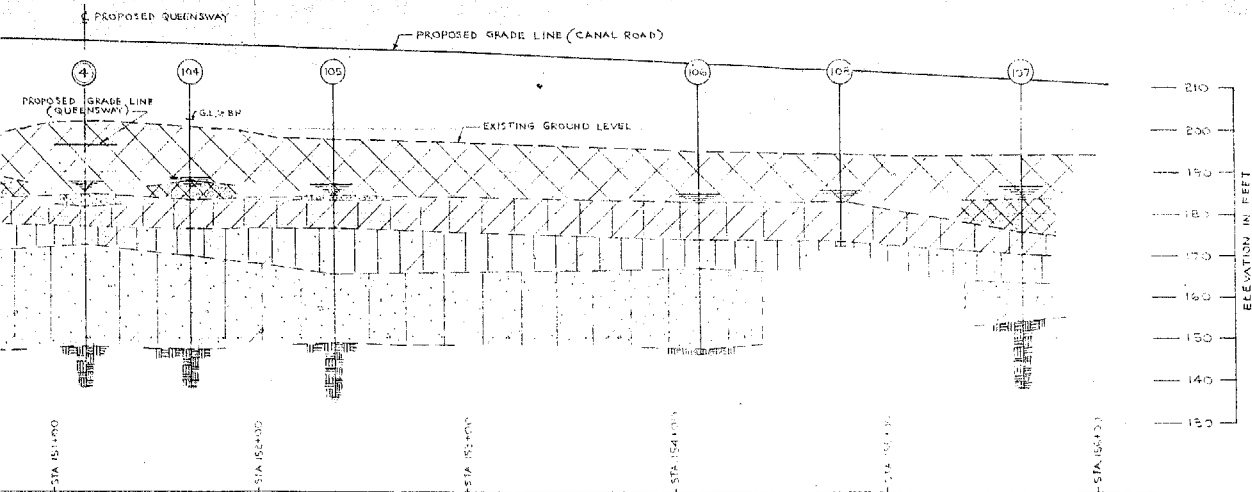
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SCHEMATIC SECTION ALONG CENTRE
SCALE: 1" TO 100'-0"

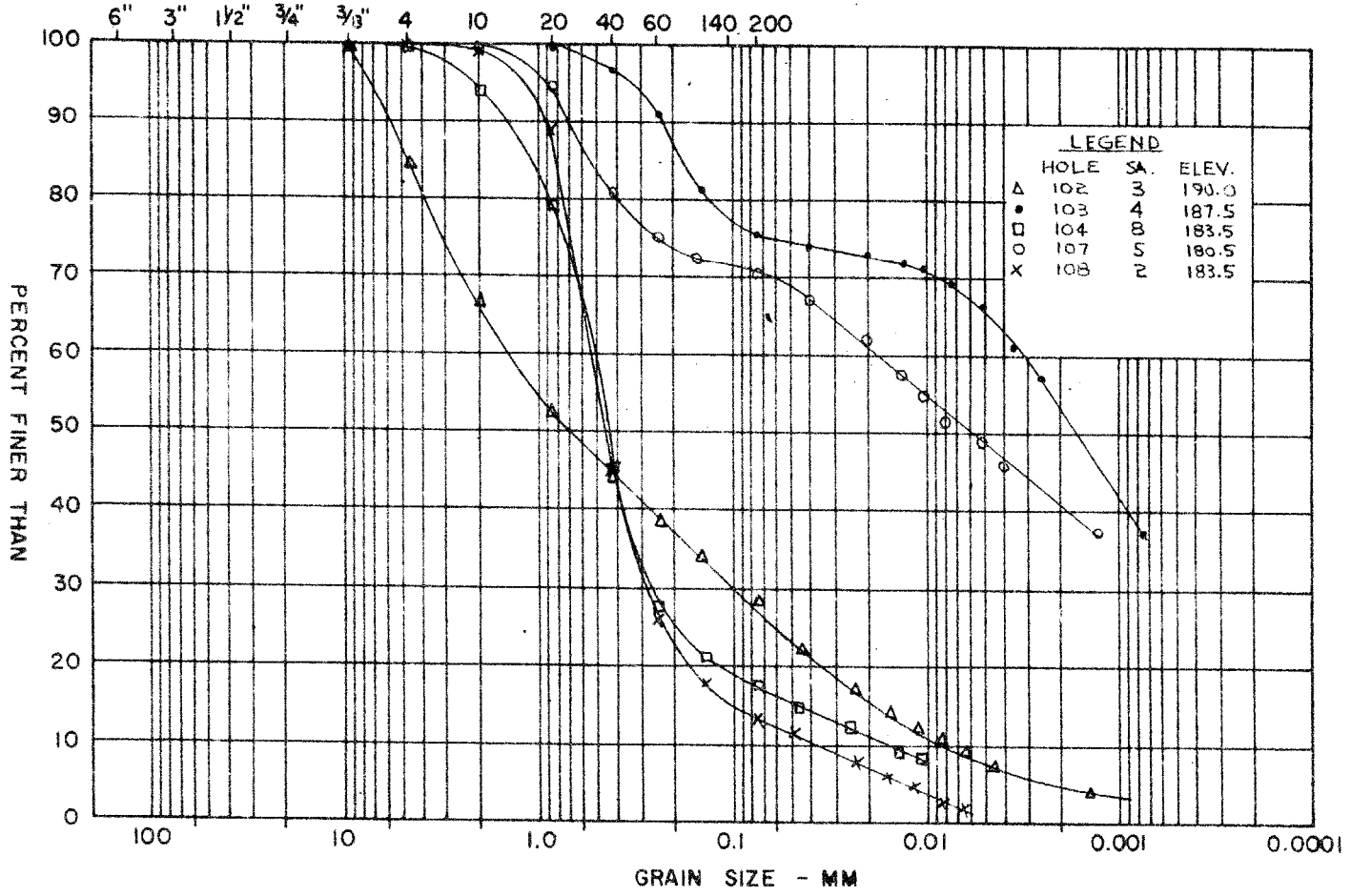
LEGEND

-  BOREHOLE IN PLAN (PRESENT INVESTIGATION),  (PREVIOUS INVESTIGATION, OUR REPORT DATED DEC., 1962)
-  BOREHOLE IN ELEVATION (PRESENT INVESTIGATION),  (PREVIOUS INVESTIGATION)
-  W.L. IN BOREHOLE, FEB. 25, 1964



M.I.T. GRAIN SIZE SCALE

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



GOLDER & ASSOCIATES

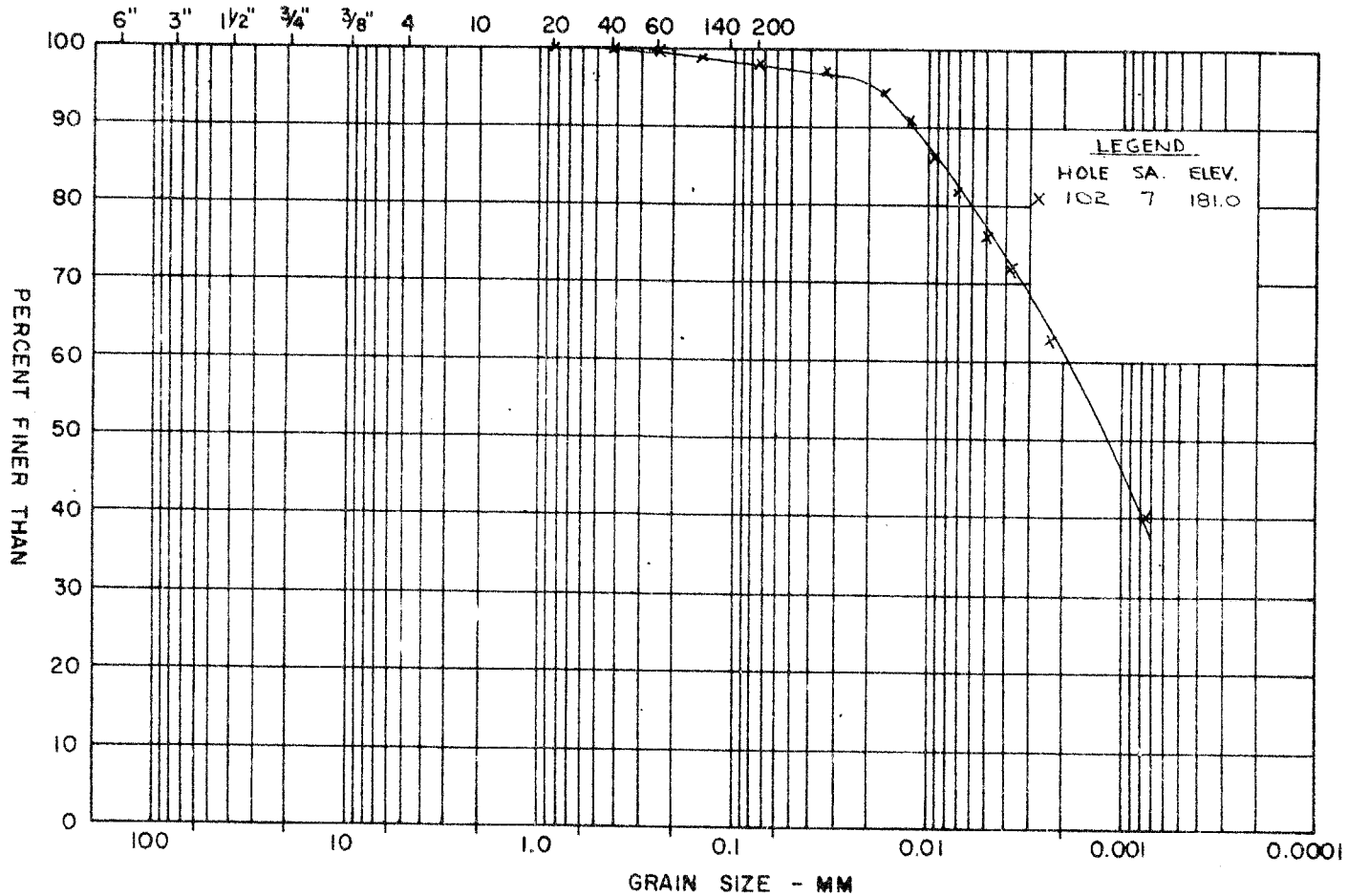
GRAIN SIZE DISTRIBUTION
FILL

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

FIGURE 2

M.I.T. GRAIN SIZE SCALE

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

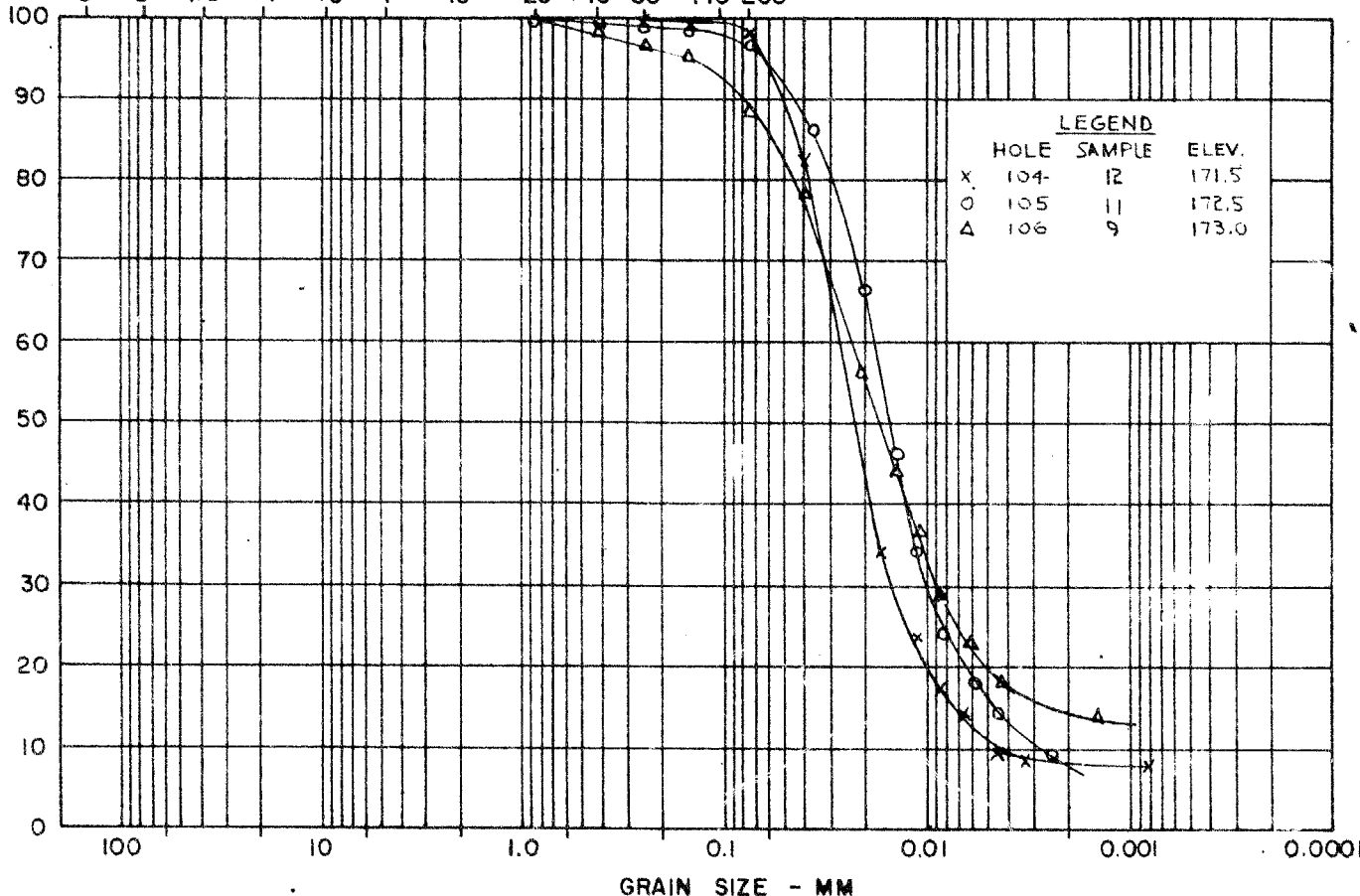


M.I.T. GRAIN SIZE SCALE

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

PERCENT FINER THAN



GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SILT

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

FIGURE 4



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

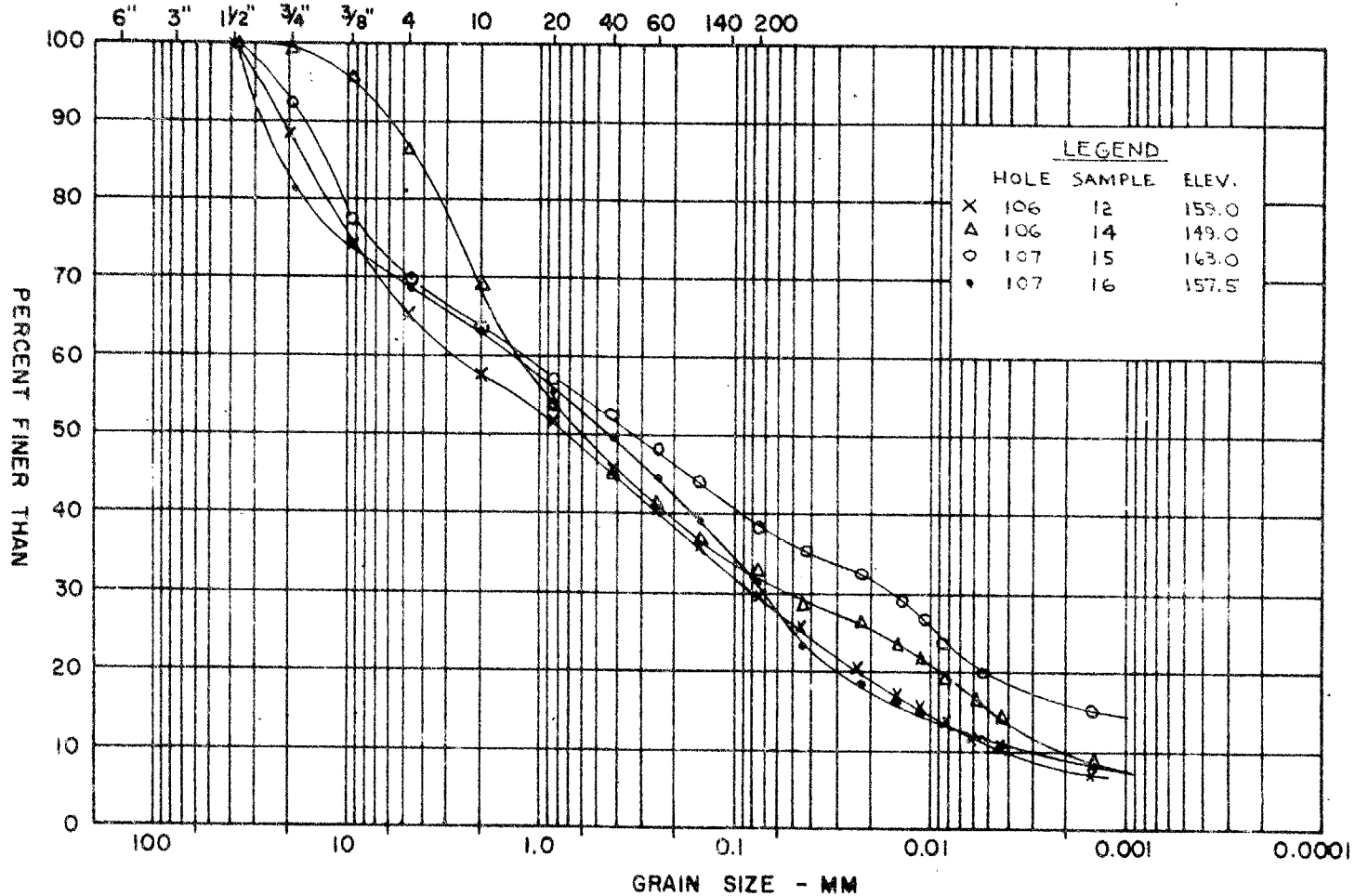
GRAIN SIZE DISTRIBUTION

TILL

FIGURE 5

M.I.T. GRAIN SIZE SCALE

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

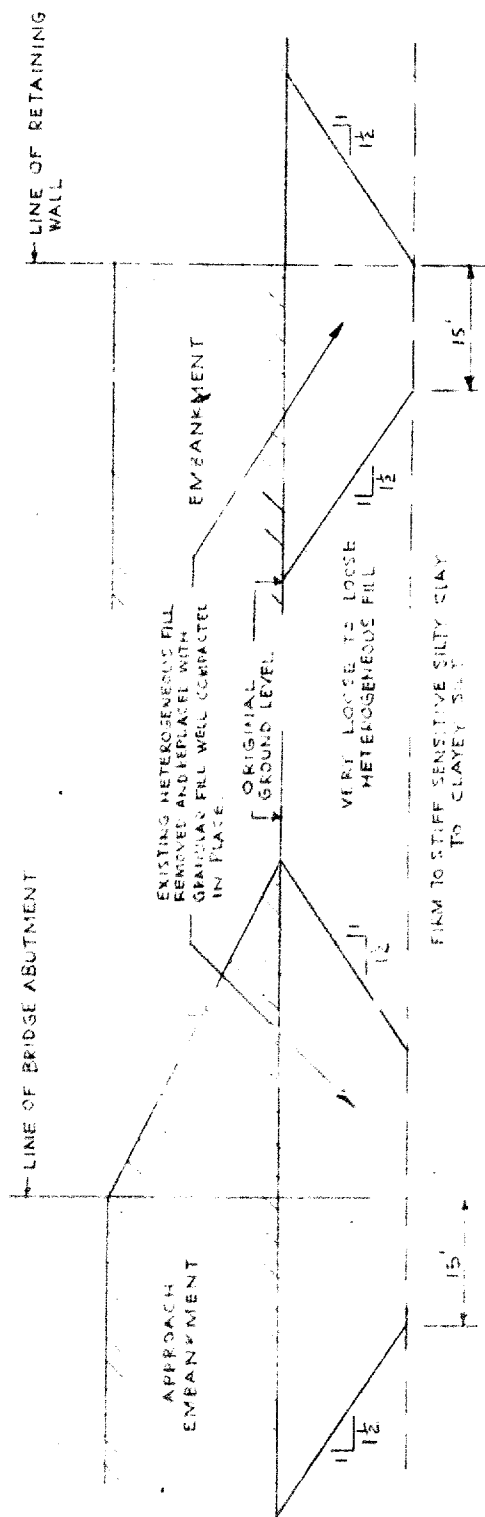


GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
TILL

FIGURE 6

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



(b)
RETAINING WALL AREA

(a)
BRIDGE ABUTMENT AREA

NOT TO SCALE

GOLDER & ASSOCIATES

Made J. A.
Chkd. OK
Appd. [Signature]

MEMORANDUM

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

FROM: Bridge Division,
Downsview, Ontario.

DATE: May 8, 1964.

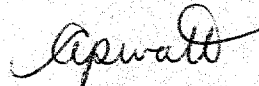
OUR FILE REF.

IN REPLY TO

SUBJECT: W.P. 954-59,
Nicholas Street Bridge # 38,
Ottawa Queensway,
District # 9.

Attached please find one copy of the Preliminary
Bridge Plan D-5474-P1 for the above structure.

Would you kindly review the bridge foundations
proposed and inform the Bridge Office if they are
satisfactory.



APW/kd

A.P. Watt,
Regional Bridge Location Engineer.

*No comments. The designer seems to have
combined with the recommendations of the
soils report*

May 12, 1964

Agtermae

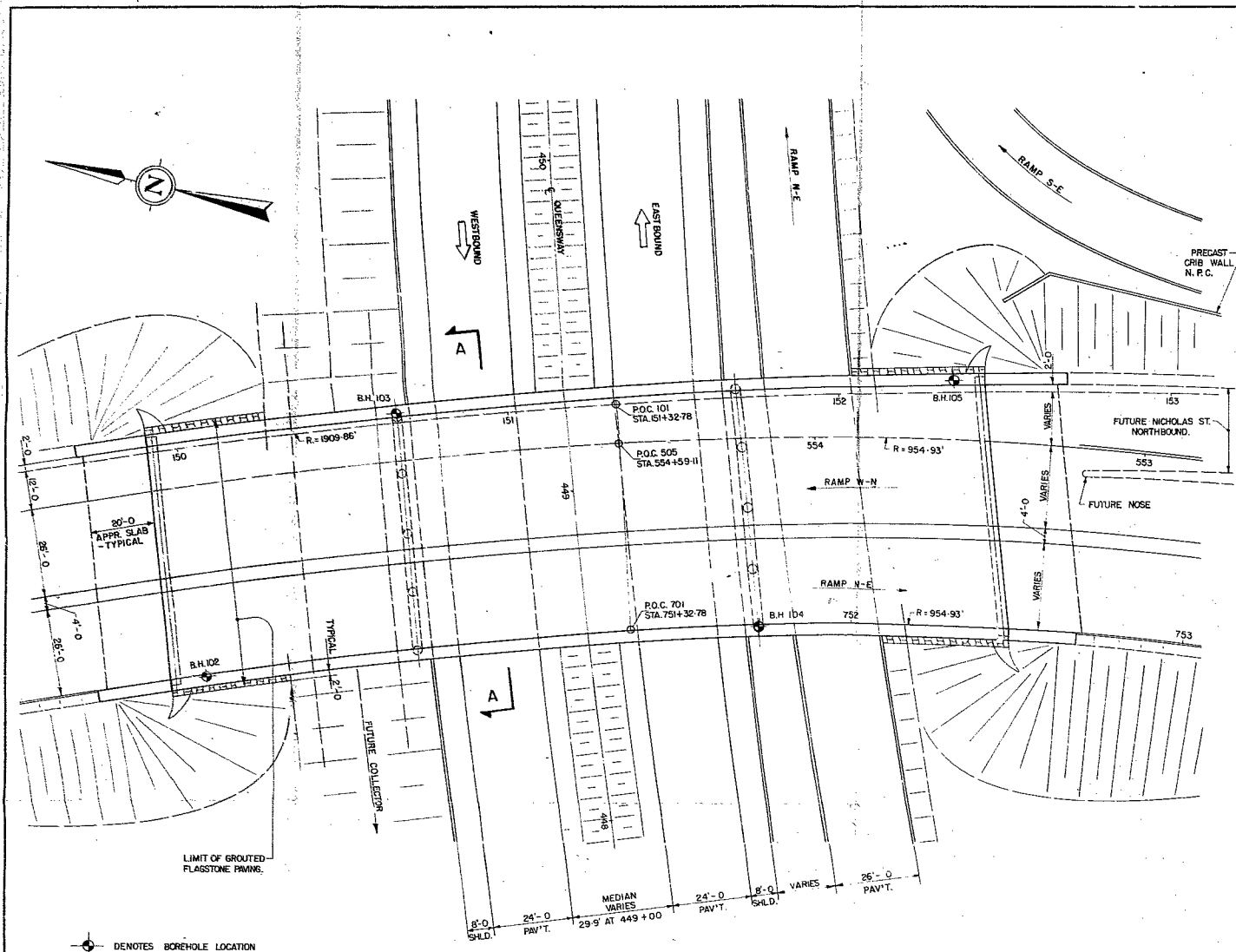
#64-F-226C

W.P.#954-59

OTTAWA

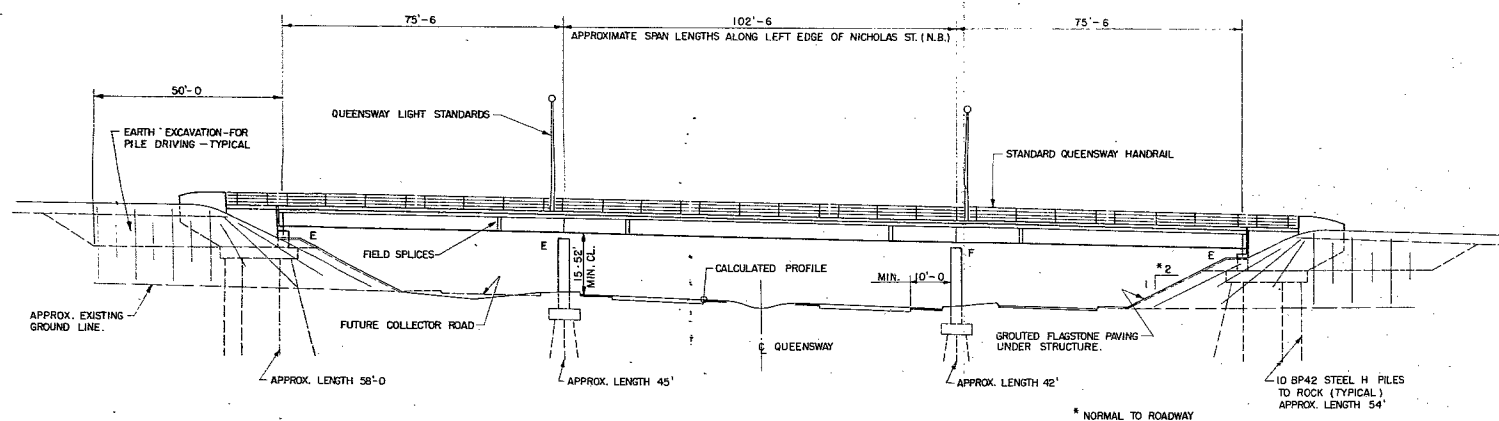
QUEENSWAY

BRIDGE #38



BRIDGE PLAN

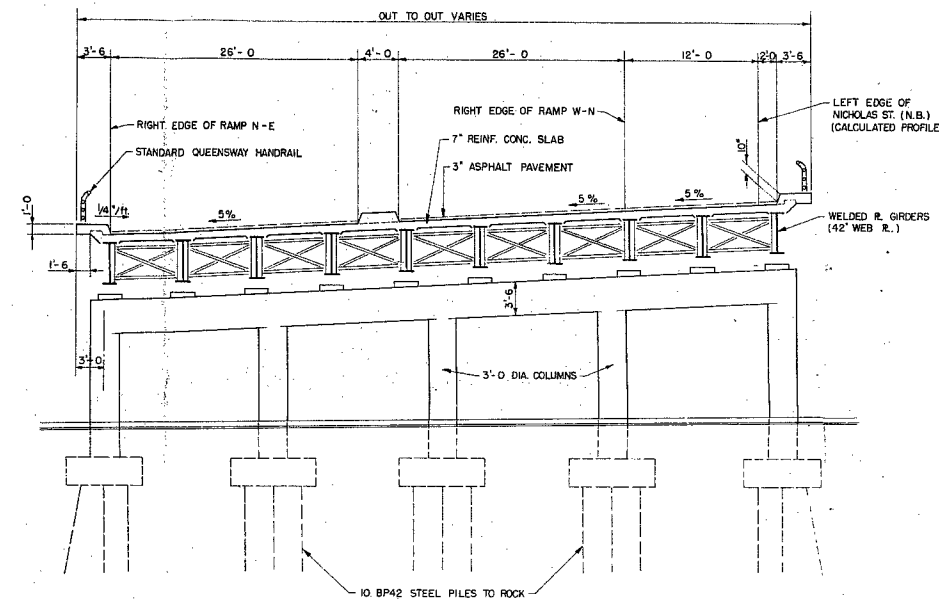
SCALE: 1" = 20'-0"



WEST ELEVATION

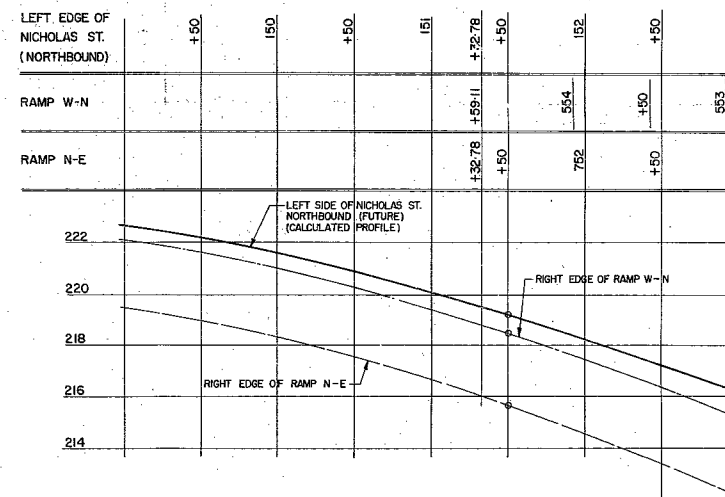
SCALE: 1" = 20'-0"

NOTE:
SUBEXCAVATION UNDER APPROACH EMBANKMENTS
AND CONSTRUCTION OF EMBANKMENTS IS INCLUDED
IN THE WORK OF THE GRADING CONTRACT - W.P. No. 949-59-3.



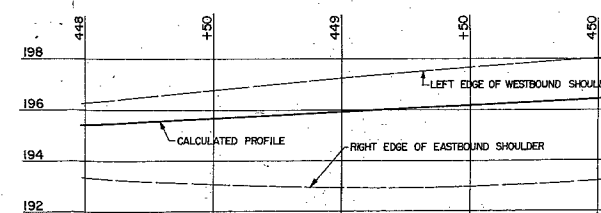
SECTION 'A-A'

SCALE: 1" = 8'-0"



NICHOLAS STREET PROFILE

SCALE: HORIZ. 1" = 50'-0"
VERT. 1" = 3'-0"



QUEENSWAY PROFILE

SCALE: HORIZ. 1" = 30'-0"
VERT. 1" = 3'-0"

	B.H. 102	B.H. 103	B.H. 104	B.H. 105
205				
200				
195				
190	HETERO-GENEUS FILL	HETERO-GENEUS FILL	HETERO-GENEUS FILL	HETERO-GENEUS FILL
185		FILL		SILTY SAND
180	SILTY CLAY	SILTY CLAY	SILTY CLAY	SILTY CLAY
175				
170	SILT	SILT	SILT	SILT TO SANDY SILT
165				
160	TILL	TILL	TILL	TILL
155				
150				
145	BEDROCK	BEDROCK	BEDROCK	BEDROCK
140				
135				

BOREHOLE LOG

SOILS REPORT BA1797

NOTES:

- DESIGN SPECIFICATIONS:
A.A.S.H.O. STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES 1961 AND D.H.Q. BRIDGE DIVISION TENTATIVE STANDARDS AND MEMORANDUMS.
- LIVE LOAD:
H20-S16-44
- CONCRETE:
3000 P.S.I. AT 28 DAYS.
- STRUCTURAL STEEL:
C.S.A. SPECIFICATIONS G40-8, GRADE A.
- SUPERSTRUCTURE:
CONTINUOUS WELDED PLATE GIRDERS WITH REINFORCED CONCRETE DECK SLAB - COMPOSITE ACTION IN POSITIVE MOMENT ONLY. FASCIA GIRDERS TO BE CURVED IN PLAN; INTERIOR GIRDERS TO BE STRAIGHT BETWEEN BEND POINTS LOCATED AT FIELD SPLICES.
- FOUNDATIONS:
ABUTMENTS AND PIERS SUPPORTED ON STEEL BEARING PILES - 10 BP42 WITH POINT REINFORCEMENT. ALLOWABLE LOAD 50 TONS.
- PRELIMINARY ESTIMATE OF COST:
\$247,000

No.	Revisions	By	Date
DEPARTMENT OF HIGHWAYS OF ONTARIO			
OTTAWA QUEENSWAY LIMITED-ACCESS HIGHWAY OTTAWA CANADA			
BRIDGE No. 38 AT NICHOLAS ST. (N.B.) PRELIMINARY BRIDGE PLAN			
DE LEUW CATHAR & CO. OF CANADA LIMITED Consulting Engineers		DEPT. OF HIGHWAYS OF ONTARIO Director of Planning & Design	
Designed by: G.S.S.		Date: APRIL, 1964	
Drawn by: E.T. - A.G.Y.		DWG. No. D5474-PI	
Checked by: L.J.M.		Scale: AS SHOWN	
CONTRACT NUMBERS		Sheet 1 of 1	
WORKS PROJECT No. 954-59		DISTRICT No. 9	

C456-PI