

Mr. C. E. Robertson,
District Engineer,
Ottawa (District #9).

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

Attention: Mr. W. S. Aitken,
Construction Engr.

June 6, 1967

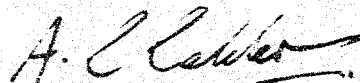
Ray, #401 and Fraser Road Instrumentation
W. J. 66-F-84 -- Contract No. 66-179

At the above mentioned location, we would like to install a vent pipe to the C.I.P. culvert which is now in place.

The details of this vent pipe and the required location are shown on the enclosed drawing.

Would you please make the necessary arrangements to have this work carried out.

ACC/MdeP
Encl.



A. C. Calder,
SENIOR FOUNDATION ENGINEER
For:
N. Davata,
SUPERVISING FOUNDATION ENGINEER

cc: Foundations Files ✓
Gen. Files

Department of Highways Ontario

Copy for the information of

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

Mr. A. McKim,
Bridge Control Engineer,
Administration Building.

Bridge Division,
Downsview, Ontario

June 6, 1967

Fraser Road Underpass, W.P. 107-59, Contract 67-18,
Highway 401, Site No. 67-18, District No. 9

The Foundations Section have just requested that the piles at the ends of the wingwalls (4 piles in all) should be battered backwards (i.e. the tips further from Highway 401 than the tops) at 1 in 10. It might be better to ask the contractor to quote the additional cost and then to give written instructions rather than to change the drawings and wait for a claim. The piles involved are a large bored in place tubes costing \$24.00/ft to drive, a fairly large claim could be involved.

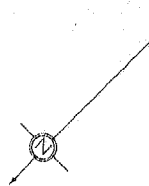
Please discuss this with the District. If you disagree, please give written instructions to change the drawings.

BSR/pr

cc. A. G. Stermac

B. S. Richardson,
Regional Bridge Project Engineer





P22 P23
P21 P20
117' LEFT

P19
P18
97' LEFT

P15 P16
P14 P13
119' LEFT

P12
P11
110' LEFT

P8 P9
P7 P6
110' LEFT

P5 P6 P7 P8
P3 P2 P1
110' LEFT

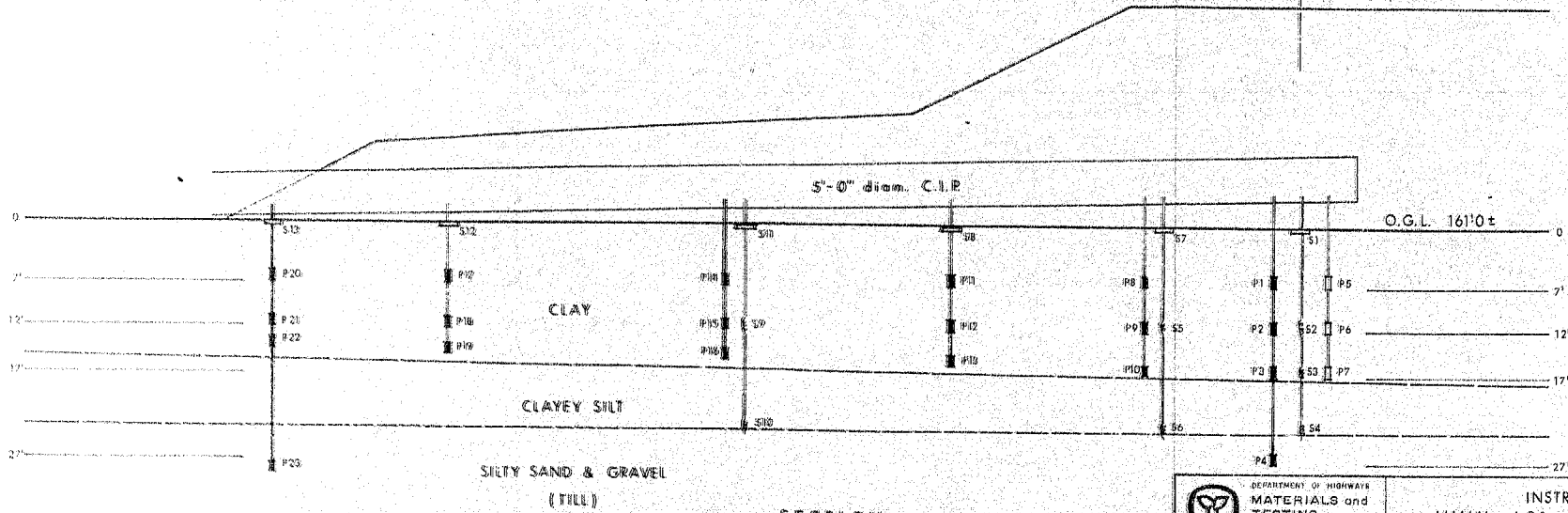
STN. 26 + 68

FRASER ROAD

LEGEND

PLAN	PROFILE	QUANTITY
		SETTLEMENT PLATES 8
		GEONOR PIEZOMETERS 20
		PEAKER PIEZOMETERS 3
		SETTLEMENT AUGERS 7

PLAN SCALE : 1" = 10'



SECTION SCALE : 1" = 10'

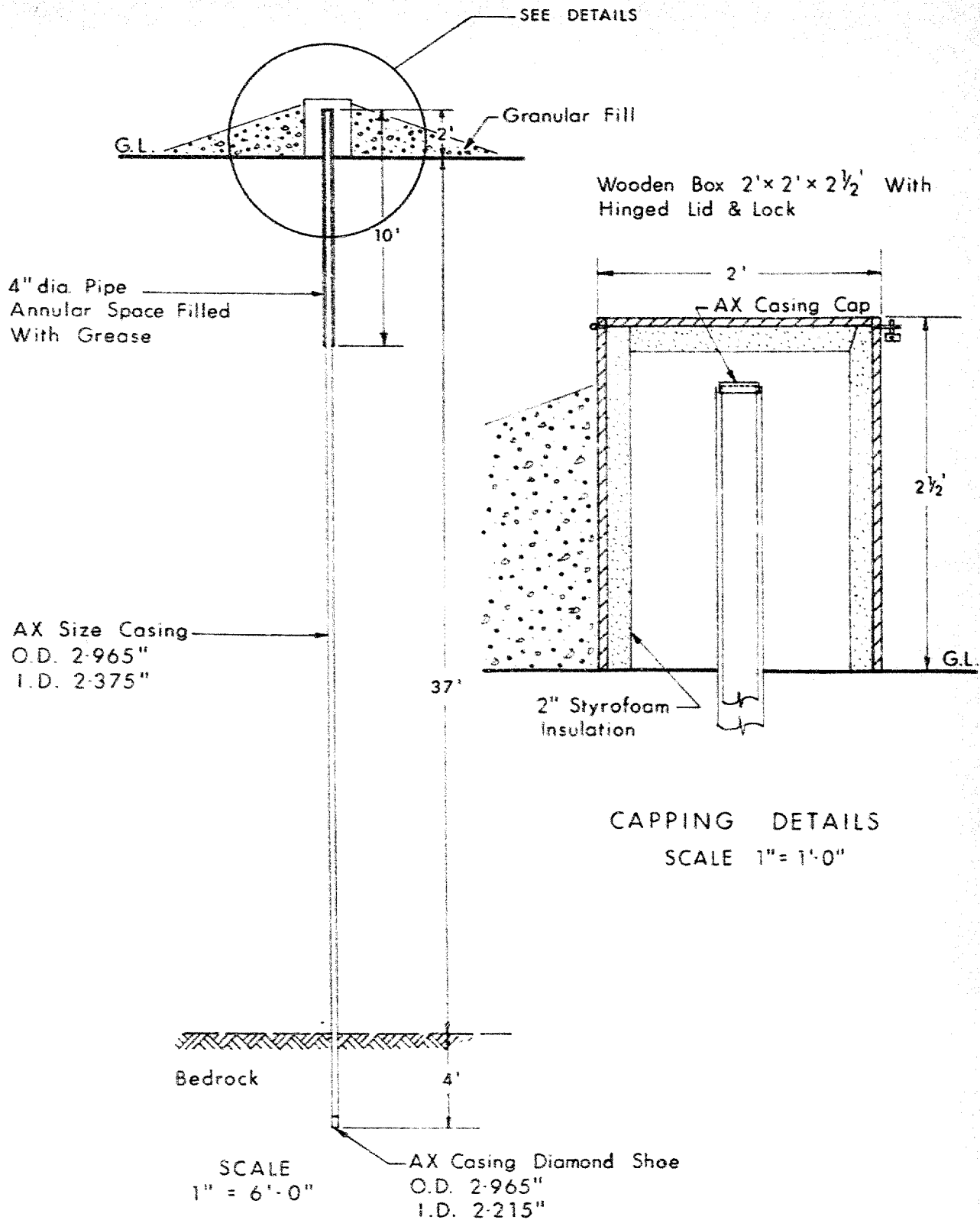


INSTRUMENTATION
HWY. 401 & FRASER ROAD
INSTALLATION DETAILS

DATE: JUNE 1 1967

APPROVED

PROJECT NO. 401-67-01



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

FRASER ROAD & HWY. 401 INSTALLATION DETAILS OF BENCH MARK

DATE 29 SEPT. 1966

APPROVED *M. Swata*

DRAWING NO. 66-F-84 C

Mr. C. B. Robertson,
District Engineer,
District #9 (Ottawa).

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

Attn: Mr. W. S. Aitken,
Construction Engr.

September 16, 1966

-- Instrumentation --
Fraser Road, Highway 401
District #9 (Ottawa)
W.P. 107-59-1

Further to our telephone conversation, we are enclosing a drawing showing the details of the culvert proposed for the above mentioned instrumentation project. We have also detailed on the drawing the various quantities required for placing of the culvert. We feel that 3-ft. square holes should be cut with a torch in the bottom of the culvert at the predetermined distances prior to placing the culvert over the instrumentation area.

The field work for the installation of piezometers, settlement gauges, etc., will commence on September 19, 1966, and our Project Engineer, Mr. R. Magi, will be in charge of this project.

If there are any other points which need clarification, please let us know as soon as possible.

MD/MdeP
Attach.

cc: Mr. J. E. Gruspier

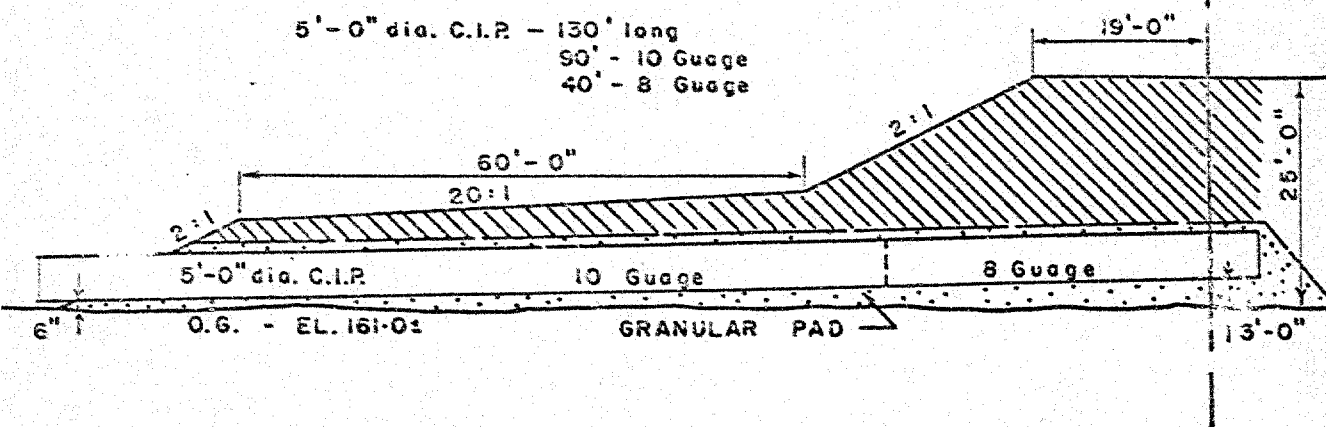
Foundations Office ✓
Gen. Files

M. Devata
M. Devata,
SUPERVISING FOUNDATION ENGR.
For:
A. G. Stermac,
PRINCIPAL FOUNDATION ENGR.

GRANULAR MATERIAL
Estimated at 650 cu. yards.

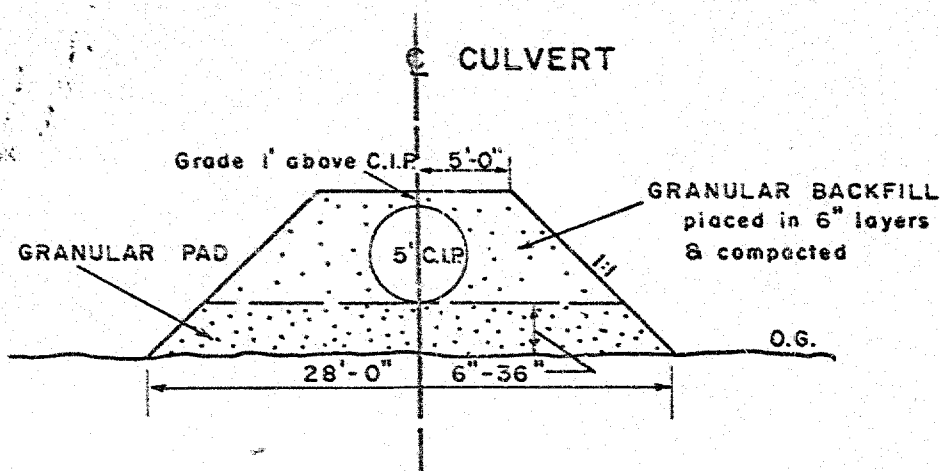
FRASER
ROAD

5'-0" dia. C.I.P. - 130' long
50' - 10 Gauge
40' - 8 Gauge



LONGITUDINAL SECTION
1" = 20'

CROSS-SECTION
1" = 10'



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

HWY. 401 & FRASER ROAD INSTRUMENTATION
C.I.P. PLACEMENT DETAILS

DATE SEPT. 15, 1966

APPROVED *W. J. Thomas*

DRAWING NO. 66-F-84 B

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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

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

W. P. 107 - 59

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS AND FOUNDATIONS

PROPOSED FRASER ROAD UNDERPASS

HIGHWAY 401

GLENGARRY COUNTY

ONTARIO

Distribution:

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

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

January, 1966

65135

*Received Construction
Sept 20/1966*

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ABSTRACT

The results of an investigation to determine the subsurface conditions at the site of the proposed underpass structure to carry the line "A" alignment of Fraser Road over Highway 401 in Charlottenburgh Township near Lancaster, Ontario are reported. Information for the foundation design of the proposed structure and associated roadway approach embankments is also presented.

It was found that the site is underlain by as much as 25 feet of generally firm sensitive marine clay extending down to a 10 foot thick stratum of dense to very dense sandy till. Fairly sound limestone bedrock underlies the till between elevations 120 and 130. The natural groundwater level was found to be at ground surface in the lower lying areas of the site. A slight artesian pressure was encountered in the sandy till stratum with a measured head as high as 2 feet above ground surface.

The sensitive marine clay is the significant subsurface stratum which affects foundation design and controls approach embankment stability. The main design problem at this site is limiting strain and overstressing effects in the sensitive clay resulting from the roadway embankment loading to prevent detrimental lateral movement of the bridge abutments.

Both total and effective stress stability analyses were carried out for the proposed 26 foot high approach embankments and the results of these analyses show that counterbalancing berms some 12 feet high and 80 feet long are required to minimize lateral movements in the clay subsoil beneath the bridge abutment areas to reasonable limits. To resist lateral forces within the subsoil and also negative skin friction forces resulting from consolidation settlement in the clay, it is suggested that a rigid piled foundation be provided for the support of the bridge abutments. With a rigid abutment foundation consideration could be given to reducing the required berm length to about 60 feet. Other possible solutions such as decreasing the height of approach embankments by increasing the slope of the Fraser Road grade line are discussed in this report.

INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by the Department of Highways, Ontario to carry out a subsurface investigation at the site of a proposed underpass to carry revision line "A" of Fraser Road over Highway 401 in Charlottenburgh Township, Ontario. The purpose of this investigation was to determine the subsurface conditions at the site and to provide information for the foundation design of the proposed structure and associated roadway approach embankments.

PROCEDURE

The field work for this investigation was carried out between November 9 and December 3, 1965. During this period a total of 7 boreholes with adjacent dynamic penetration tests and 5 additional dynamic penetration tests, ranging in depth from about 25 to 50 feet, were put down using a skid mounted machine drillrig supplied and operated by the F. E. Johnston Drilling Co. Ltd., Ottawa, Ontario. Following completion of each boring a standpipe or piezometer was installed for groundwater level observation. The field work was supervised throughout by an engineer from our staff.

The locations of the borings and dynamic penetration tests put down during the investigation are shown on Figure 1 located in a pocket following the Records of Boreholes. A detailed log for each boring and dynamic penetration test is given on the Records of Boreholes following the text of this report. A section of the inferred

subsurface stratigraphy along the proposed centerline of Fraser Road is given in Figure 1.

The samples obtained during the investigation were brought to our laboratory for detailed examination and testing. The results of this testing are shown on the Records of Boreholes and on Figures 2 to 15, inclusive.

The elevations given in this report are referred to Geodetic datum and were determined from a bench mark consisting of a nail and washer in a root of a three foot diameter oak tree located 510 feet to the right of station 21+95. The elevation of this bench mark is 165.58 feet. The borehole elevations and locations were supplied to us by the Department of Highways, Ontario.

SITE & GEOLOGY

The site of the proposed underpass to carry revision line "A" of Fraser Road over Highway 401 is located some 2.8 miles west of Lancaster, Ontario in Charlottenburgh Township, Glengarry County, Ontario.

Except for the existing Highway 401 and Fraser Road the site is generally flat and grass covered. In lower lying areas of the site the ground was covered during the period of this investigation by up to 6 inches of water. The grade of the existing Highway 401 is some

6 feet above the surrounding ground surface and the highway consists of 4 paved lanes with median strip and associated gravelled shoulders. Fraser Road is some 4 feet above the surrounding ground surface and has a 20 foot wide gravelled surface.

The site of the proposed underpass structure is located in the physiographic region known as the Lancaster Flats. Based on available geological information it is known that the subsoil consists of rather poorly drained deposits consisting of water-laid materials ranging from clay to very fine sand. The clay in this particular area is of marine origin and was deposited in the upper reaches of the Champlain Sea which covered the St. Lawrence Lowlands in recent geological time. The marine clay, referred to as "Leda" clay, is generally underlain by a granular till deposit which is in turn underlain at a depth of some 25 to 100 feet, by limestone and shale bedrock.

SUBSURFACE CONDITIONS

The detailed stratigraphy encountered in each boring is given on the Records of Boreholes. The stratigraphy along the proposed centerline of Fraser Road has been interpolated from this data and is presented on Figure 1. Following is a summary account of the inferred subsurface conditions at the site.

The borings put down indicate that the existing Highway 401 roadway fill consists of about 5 feet of compact to dense brown silty sand to sand and gravel. Typical grain size distribution curves for

samples of the roadway fill are shown on Figure 2. Up to 3 feet of black silty topsoil was found beneath the roadway fill and across the remainder of the site.

In the southern portion of the site the topsoil is underlain by as much as 2 feet of compact brown to grey sandy silt to silty sand. This surficial deposit varies in composition to a very stiff brown clayey silt in the northern portion of the site. A grain size distribution curve for a sample of the clayey silt is given on Figure 2.

Underlying the shallow surficial deposits at about elevation 158, the borings encountered some 10 to 25 feet of grey sensitive clay containing a trace to some silt throughout. The clay generally has a fissured or chunky structure but the stratum contains zones where no fissuring is evident. The clay does not have any pronounced stratification or layering pattern. Typical grading curves for samples from the sensitive clay stratum are shown on Figure 3.

Based on thirteen Atterberg limit determinations the liquid limit of the clay varies between about 65 and 85 and the corresponding plasticity index is between 50 and 65. The in situ water content of the clay is between about 60 and 90 per cent resulting in a liquidity index generally between about 0.8 and 1.1. The results of the Atterberg limit tests are summarized on Figure 7. This figure indicates that the liquid and plastic limits of the clay are fairly constant

with depth and that there is a trend for an increase in the in situ water content with depth to about elevation 150 below which there is a slight decrease with depth.

The results of fourteen unit weight determinations, which have been plotted against elevation on Figure 7, indicate that the total unit weight of the clay varies between about 95 and 110 lb/cu.ft. with an average value of about 100 lb/cu.ft.

The undrained shear strength of the clay was determined by in situ vane testing in the field and by undrained triaxial compression tests on relatively undisturbed samples in the laboratory. The results of these tests are plotted on the Records of Boreholes and summarized on Figure 7. Typical stress-strain curves for the undrained compression tests are shown on Figure 8. The test results indicate that the failure strain is as low as 1 per cent and that the undrained shear strength decreases with depth from a value of about 1,500 lb/sq.ft. near the surface of the clay to about 450 lb/sq.ft. near the bottom of the stratum. Based on these strength results, together with standard penetration test values which range from about 2 to 6 blows/ft., the consistency of the clay varies from stiff to firm with depth and is generally firm throughout.

Remoulded field vane tests gave values ranging between about 50 and 200 lb/sq.ft. Based on these results the clay has a sensitivity, which is defined as the ratio of undisturbed strength to

remoulded strength, of between about 5 and 15.

A series of three consolidated undrained triaxial compression tests with pore pressure measurements was also carried out in order to determine the effective or drained shear strength parameters of the clay. The results of these tests are plotted on Figure 9 using the method suggested by Rendulic (1937) and also using the conventional Mohr circle plot.

Five consolidation tests were carried out on relatively undisturbed samples of the clay. The results of these tests are presented as pressure-void ratio curves on Figures 10 to 14, inclusive. A summary plot of the preconsolidation pressure estimated from the pressure-void ratio curves is given on Figure 15. This plot indicates that the clay is overconsolidated by about 1.5 tons/sq.ft. in excess of existing overburden pressure near the surface of the stratum and becomes virtually normally consolidated with depth.

The sensitive clay stratum is generally underlain at about elevation 140 by as much as about 5 feet of grey clayey silt with sand and some gravel. This clayey silt layer is not continuous across the site as indicated by boreholes 3 and 5 where it is absent. Typical grain size distribution curves for the clayey silt are shown on Figure 4. Based on three Atterberg limit determinations the clayey silt has an average liquid limit of about 18 and an average plasticity index

of about 7. The in situ water content is about 12 per cent and is mid-way between the liquid and plastic limits.

Based on one field vane test which gave an undrained shear strength value of about 1,000 lb/sq.ft., together with the standard penetration tests which gave "N" values ranging between about 10 and 20 blows/ft., the clayey silt varies between firm and very stiff in consistency and is generally stiff.

Underlying the clayey silt and sensitive clay strata at a depth of between about 25 and 28 feet, the borings encountered some 10 to 15 feet of sandy till. The till consists of grey silty sand and gravel with a trace to some clay and contains some scattered cobbles and boulders particularly in the lower 5 feet of the stratum. Typical grain size distribution curves for samples of the till obtained using 1½ inch I.D. sampling equipment are shown on Figures 5 and 6. Based on the standard penetration test results given on the Records of Boreholes, the till is in a generally dense to very dense state of packing.

The till below about elevations 120 to 130 is underlain by bedrock which was proved by core drilling in AXT size for up to 10 feet in boreholes 1 to 5, inclusive. The bedrock consists of fairly sound grey limestone with interbedded shale layers.

During the boring operations in the upper portion of the

overburden it was noted that the groundwater level was at ground surface in the area of the site outside the existing roadway fills. Ground surface is as low as elevation 161. In advancing the boreholes through the lower portion of the overburden a slight artesian pressure was encountered in the dense sandy till underlying the relatively impervious sensitive clay stratum. The groundwater level was observed to rise as high as elevation 163 in the casing.

A piezometer which was sealed into the till stratum and a standpipe which was placed in the clay stratum were generally installed in each of the borings following their completion. Periodic readings were taken in these installations during the course of the field work. The installation details together with the latest readings obtained are shown on the Records of Boreholes and on Figure 1.

The readings show that the natural groundwater level across the site is between about elevation 161 and 163 corresponding to existing ground surface in the lower portions of the site, and that the artesian water level in the underlying till is as much as 2 feet higher than the surface or upper groundwater level.

DISCUSSION

General

As presently planned the proposed underpass structure is to consist of four simply supported spans with each central span 66

feet long and the end spans each 40 feet in length. It is understood that the grade of Highway 401 is to remain unchanged and that proposed Fraser Road approach embankment grade over Highway 401 is to be at elevation 186. Thus the roadway approach embankments for the Fraser Road will be some 24 to 26 feet above general ground surface. Spill through abutments supported on piles driven to bedrock are to be used.

Statement of Problem

It is understood that in the general vicinity of the present investigation some difficulties with newly built structures have been and still are being experienced by the Department of Highways, Ontario. During the course of this field investigation some of the underpass structures in the area were studied and it was observed that large settlement of most of the roadway approach fills had occurred. In one case (Brookdale Avenue) a fairly extensive rotational distortion of a bridge abutment had taken place. It is understood that this abutment is founded on non-displacement "H" piles driven to bedrock and the height of the roadway approach fill is about 23 feet. A 50 foot long by about 10 foot high stabilizing berm is present at the front of the approach embankment end slope. The reason for the abutment movement is not clearly known but it was probably caused by consolidation settlement of the subsoil due to the embankment loading resulting in negative skin friction on the piles causing them to settle and by overstress of the clay subsoil whereby strain and creep effects produced lateral movement at depth.

Based on the above experience in the same physiographic region as the site presently under consideration, it was decided to give consideration to preventing or minimizing overstress and creep effects within the sensitive clay subsoil in the stability study of the proposed roadway approach embankments.

Stability Analyses

Stability computations for the proposed 26 foot high roadway approach embankments were carried out using the total stress approach (undrained shear strength of the clay). The results of typical computations to determine the factor of safety against a deep seated rotational type failure of the proposed embankment section using this approach are given on Figure 16. Summary plots giving the results of all the total stress stability analyses carried out are given on Figure 17.

Reference to Figure 17 shows that the factor of safety against a deep seated failure of the proposed 26 foot high embankment with 2 horizontal to 1 vertical side slopes is less than unity and of the order of 0.9. This figure also shows that the provision of a counterbalancing berm in front of the proposed embankment section increases the factor of safety. It is significant to note, however, that for the berm heights studied, namely 12 feet and 16 feet, the factor of safety is a function of the length of berm with the berm height having no major effect. Therefore no appreciable advantage is gained by

placing 16 foot high berms rather than 12 foot high berms. Furthermore, with the provision of a 12 foot high berm longer than about 80 feet no increase in factor of safety is obtained for the embankment section since the most critical circle passes through the berm itself. It can thus be concluded from Figure 17 that the maximum factor of safety obtainable with single berm construction is about 1.7. To increase the factor of safety above this level a double berm system is required.

In addition to the total stress stability analysis discussed above effective stress stability analyses were also carried out. The total stress stability approach is a valid method for predicting the initial stability of the embankment. However, the total stress analysis does not take into account the effects of induced pore water pressures in the clay due to embankment loading or unusual groundwater conditions such as artesian pressures within the subsoil. In order to evaluate the factor of safety for these conditions, the stability has to be analysed using the effective stress approach (laboratory drained shear strength parameters of the clay) incorporating pore pressures. However, it is not always possible to accurately predict the pore pressure build up during construction on the basis of laboratory tests alone. Therefore the most practical approach is to design the embankment initially on the basis of both effective and total stress analyses and to control its rate of construction by the effective stress analysis based on measured field pore pressures.

The results of the consolidated undrained triaxial compression tests carried out on samples of the sensitive clay are presented on Figure 9. In the effective stress stability analyses discussed below, the effective cohesion, c' , and the effective angle of shearing resistance, ϕ' , of the clay were taken to be zero and 20° , respectively. These values were chosen on the basis of a strain criterion such that overstressing of the subsoil due to the embankment loading is minimized. For this criterion a limiting strain of between 1 and 2 per cent was selected for the laboratory test results. Reference to Figure 9 further shows that, for an effective stress failure criterion based on maximum deviator stress, $(\sqrt{1}' - \sqrt{3}') \text{ max.}$, an effective angle of shearing resistance, ϕ' , of 24° is obtained taking $c' = 0$.

As mentioned previously it is not possible to accurately estimate the build up of pore pressure within the subsoil due to embankment loading on the basis of laboratory testing alone. Therefore the effective stress stability analyses were carried out using an overall pore pressure parameter, \bar{B} , of zero, 0.5 and 1.0 to cover the range of possible values.

The results of typical stability computations using effective shear strength parameters for the clay are shown on Figure 18. A summary plot of all the effective stress stability computations carried out is presented on Figure 19. This summary plot shows that for the

proposed 26 foot high embankment, the factor of safety varies from 0.4 to about 1.0 as the overall pore pressure parameter, \bar{B} , is varied from 1.0 to zero. Furthermore the factor of safety is increased by the provision of a berm. With a 60 foot long berm, the factor of safety taking full excess pore water pressure ($\bar{B} = 1$) is 0.6 while for the long term case where all excess pore water pressure in the clay is dissipated ($\bar{B} = 0$) the factor of safety is about 1.4.

A direct comparison between the factor of safety obtained by the total and effective stress approaches cannot be made in this case. The total stress analysis gives higher factors of safety due to the fact that the undrained shear strength value used for the clay represents an ultimate or failure condition, while in the effective stress approach the shear strength parameters used do not represent a failure condition but a limited strain condition, as discussed above. It is interesting to note, however, that the effective stress analysis for the long term case taking full excess pore water pressure dissipation gives a factor of safety of about 1.4 for a 60 foot long berm. The total stress analysis factor of safety for this same stabilizing berm size is about 1.5.

For stability analyses based on total stress approach a factor of safety of the order of 1.7 is required if general overstressing of the subsoil is to be minimized. Therefore, a berm length of

the order of 80 feet would be required if excessive movements within the subsoil are to be eliminated. The results of the effective stress stability analyses indicate that for a berm of this length the factor of safety is of the order of 1.5 for the long term case ($\Delta u = 0$). The results also indicate that if the entire loading imposed by the embankment is initially carried by the pore water ($\bar{B} = 1$), the factor of safety for an 80 foot berm is about 0.7. This condition, however, assumes almost instantaneous placing of the embankment, whereas in the field filling operations are relatively slow. Therefore the $\bar{B} = 1$ condition would not be realized and the induced pore pressure in the subsoil, allowing for some consolidation during construction, would correspond to an overall pore pressure coefficient lower than 1.0 and probably of the order of 0.5. For the $\bar{B} = 0.5$ case the factor of safety of a 26 foot high embankment with 80 foot long berms would be slightly greater than unity. During construction of the embankment and associated berm, the induced pore pressure in the subsoil should be measured by piezometers. The readings obtained in the piezometers would form the control governing the rate of embankment construction.

It must be stressed that the factors of safety based on the effective shear strength parameters ($c' = 0$, $\phi' = 20^\circ$) have been calculated on the basis of limited strain within the sensitive clay and do not represent a failure condition for which $\phi' = 24^\circ$ using $c' = 0$ (see Figure 9).

The total settlement beneath the center of the proposed roadway approach embankment due to consolidation within the sensitive clay stratum has been estimated on the basis of laboratory consolidation tests. Since the clay has not been preconsolidated to the pressures to be imposed by the proposed fill, settlement of the embankment is estimated to be large and of the order of 18 to 24 inches. This settlement should take place within 5 to 10 years following construction with the majority of the settlement occurring in the first 2 to 3 years.

Possible Solutions

To prevent significant movement of the proposed bridge abutments due to lateral strain effects within the subsoil caused by the imposed roadway approach embankment loading, the embankments may be constructed as discussed above using an end stabilizing berm some 12 feet high and 80 feet long. For this case the factor of safety based on total stress analyses and using a failure criterion is 1.7 while the long term factor of safety based on effective stress and a limited strain criterion is about 1.5. This solution would however necessitate a considerable increase in the length of the structure.

Since settlement of the proposed roadway approach embankments may be as high as 2 feet, negative skin friction forces will be imposed on the piles supporting the abutments. These forces combined with movement of the subsoil due to strain imposed by the embankment loading will generally tend to displace the piles laterally and, if

they are not firmly seated in bedrock, vertically downward. It is generally conventional to employ non-displacement piles such as "H" piles in sensitive clay subsoil to minimize remoulding and subsequent loss of strength due to driving. Standard "H" pile sections are however flexible and when subjected to combined vertical and lateral forces may tend to distort. This is a possible explanation of the unsatisfactory performance of some of the existing bridge abutments in the general area.

Therefore consideration should be given to founding the proposed bridge abutments on rigid piles such as tube piles filled with reinforced concrete. To prevent excessive disturbance of the sensitive clay the tube piles should be placed in pre-augered holes. Since the negative skin friction forces may be large, the piles should be set firmly in the bedrock to ensure that no settlement of the abutment occurs.

With the provision of a rigid pile foundation capable of withstanding lateral forces due to movement within the subsoil resulting from the imposed embankment loading, it would not be necessary to design the embankment to as high a factor of safety (1.7 total stress) as discussed previously. A factor of safety based on the undrained shear strength of the clay (total stress) of 1.5 would be adequate to prevent failure of the embankment. Thus with provision of a rigid piled abutment foundation reduction of the berm length to

about 60 feet could be considered.

In conjunction with founding the proposed abutments on rigid piles additional strength may be achieved by employing a rigid structure rather than the proposed simply supported spans.

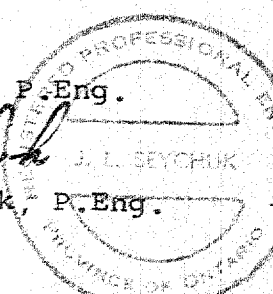
The length of berm required, and hence the length of structure may be further reduced by lowering the grade at which Fraser Road crosses Highway 401. This however is not likely possible due to clearance requirements. However, the slope of the Fraser Road grade line may be increased thus reducing the required height of approach embankments with a consequent reduction in the length of berm.

At a meeting held on January 12, 1966 between Mr. A. G. Stermac of the Department of Highways, Ontario and members of our staff, the results of our study to date on this project were presented and discussed. It was decided at this meeting to present a report covering all of the work carried out to January 12, 1966. Following a study of this report by the Department of Highways, a further meeting is to be arranged to discuss the possible solutions outlined above and to decide on the most practical and economic solution to the problem. Further analyses would be carried out, as required, after the second meeting and a final report presenting the recommended foundation treatment would be prepared at that time.

JBD:JLS:IMB
65135
January 17, 1966

GOLDER & ASSOCIATES

Jor
for J. B. Davis, P. Eng.
J. L. Seychuk
J. L. Seychuk, P. Eng.

A circular professional engineer seal for the Province of Ontario. The outer ring contains the text "REGISTERED PROFESSIONAL ENGINEER" at the top and "PROVINCE OF ONTARIO" at the bottom. The inner circle contains the name "J. L. SEYCHUK" and "P. Eng." below it.

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

τ	= 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 = $-\Delta e / C \log_{10} \sigma'$
c_s	coefficient of consolidation
T_v	time factor = $c_s t / d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion
ϕ'	effective angle of shearing resistance, or friction
c_u	apparent cohesion*
ϕ_u	apparent angle of shearing resistance, or friction
μ	coefficient of friction
S_t	sensitivity

$\left. \begin{array}{l} \text{in terms of effective stress} \\ \tau_f = c' + \sigma' \tan \phi' \end{array} \right\}$

$\left. \begin{array}{l} \text{in terms of total stress} \\ \tau_f = c_u + \sigma \tan \phi_u \end{array} \right\}$

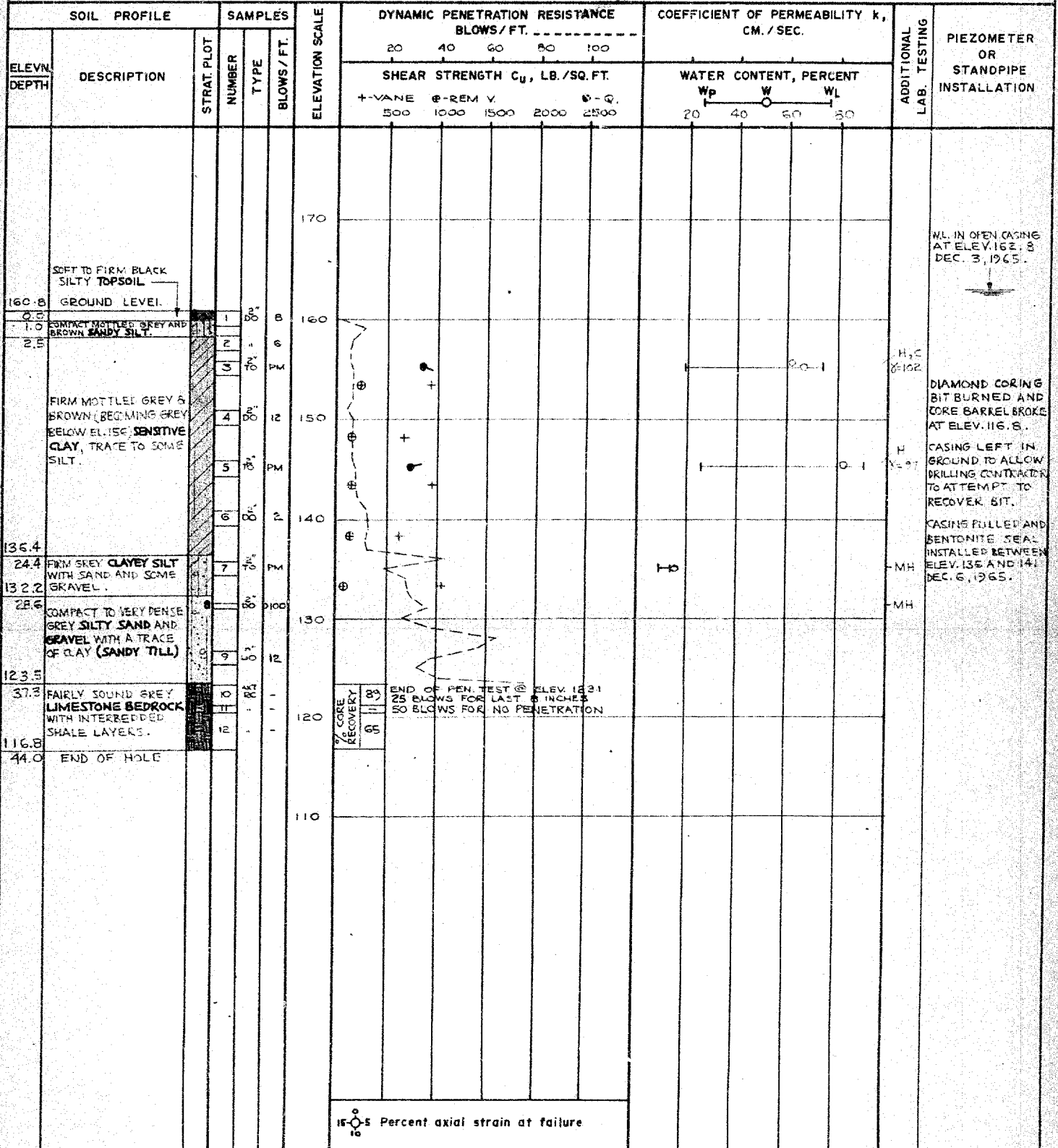
*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

RECORD OF BOREHOLE 1

LOCATION See Figure 1 BORING DATE NOV 10 - 12, 1965. DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, BX CASING

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



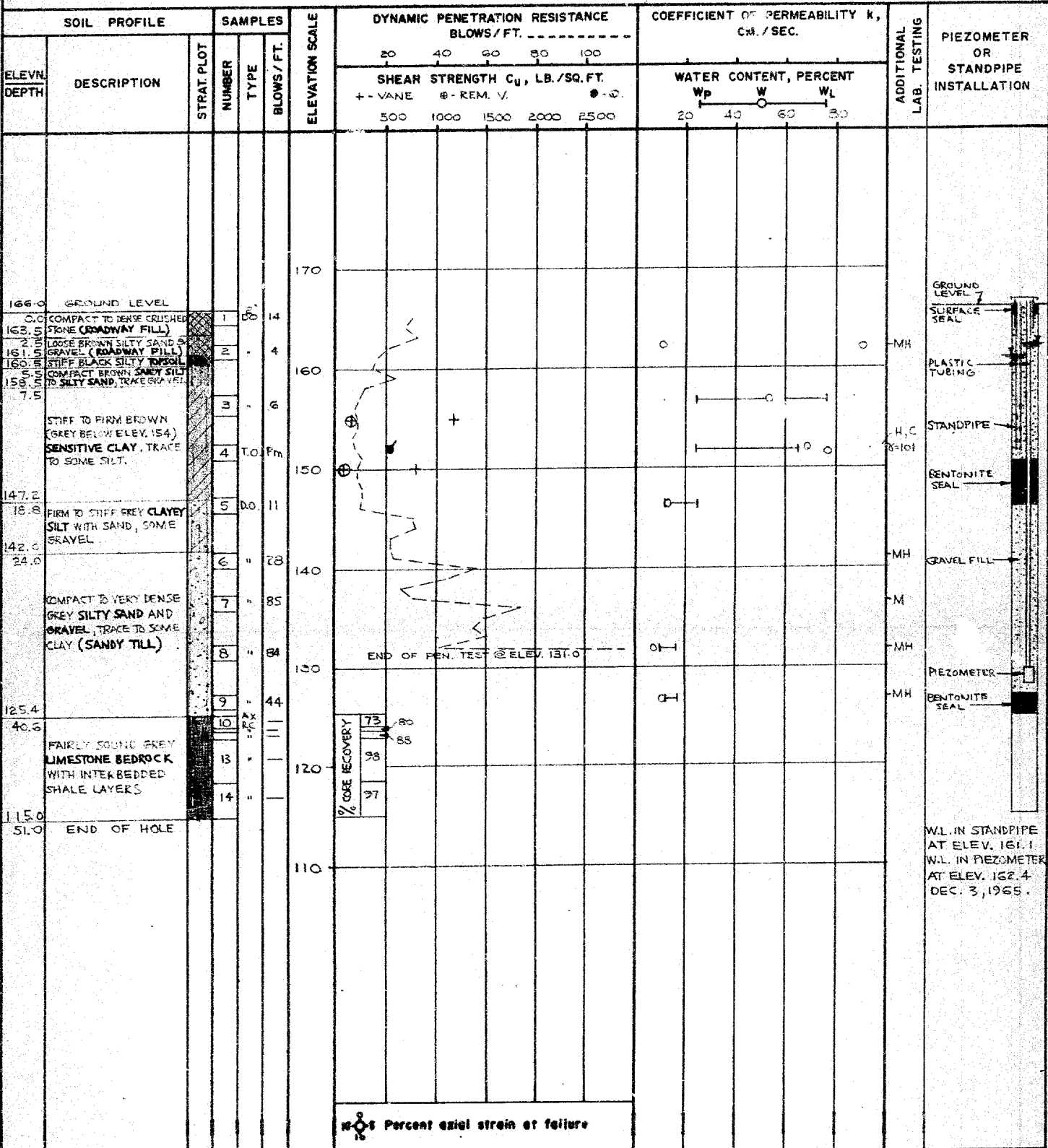
VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN R.H. 1/10
CHECKED JED

RECORD OF BOREHOLE 2

LOCATION See Figure 1 BORING DATE NOV. 13-19, 1945 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX, BX CASING
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN BY *RAH*
CHECKED *ADD*

RECORD OF BOREHOLE 3

LOCATION See Figure 1

BORING DATE DEC. 1-2, 1965

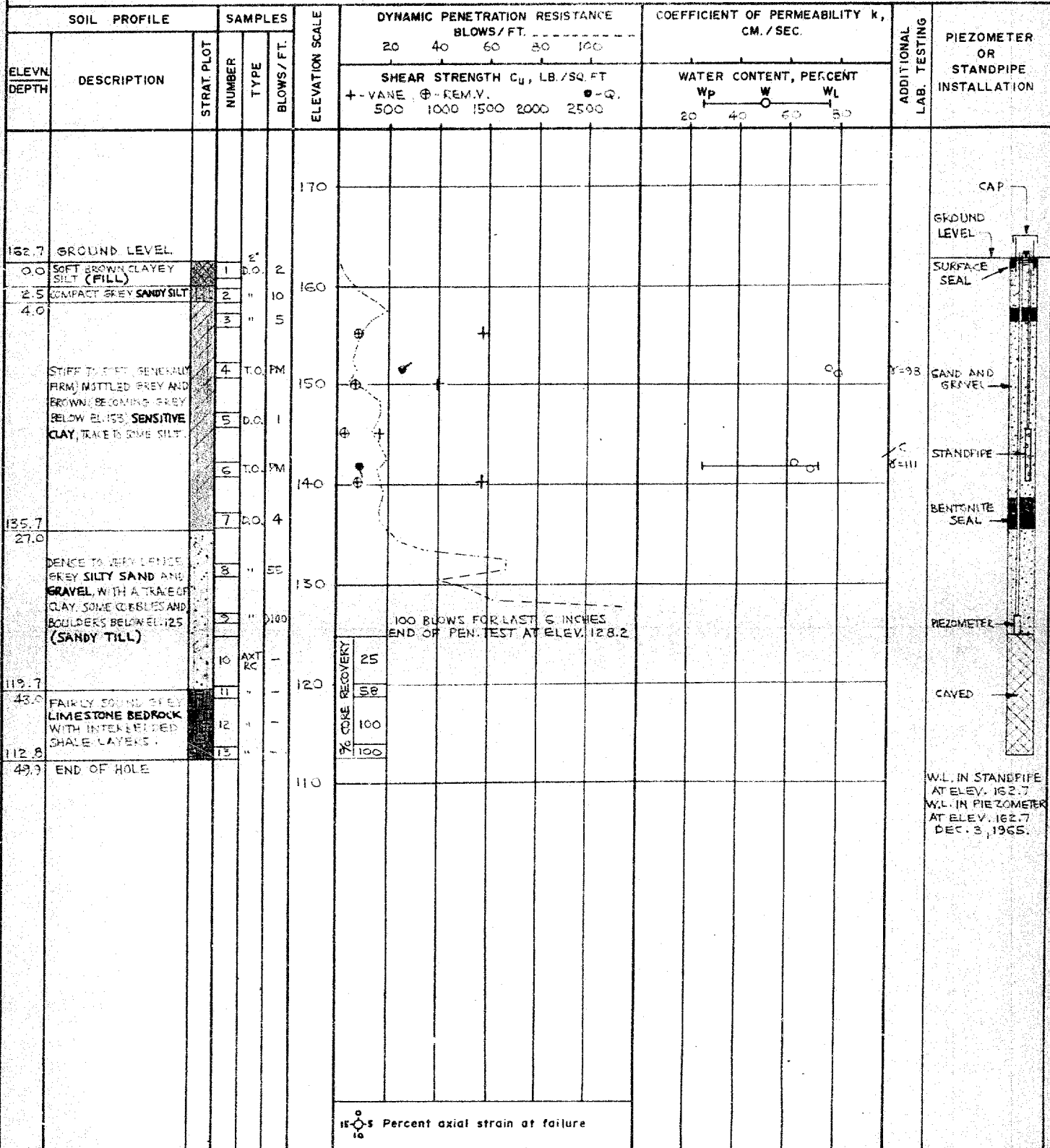
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX, 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 *ma.*
CHECKED *FD*

RECORD OF BOREHOLE 4

LOCATION See Figure 1

BORING DATE NOV. 19-24, 1965

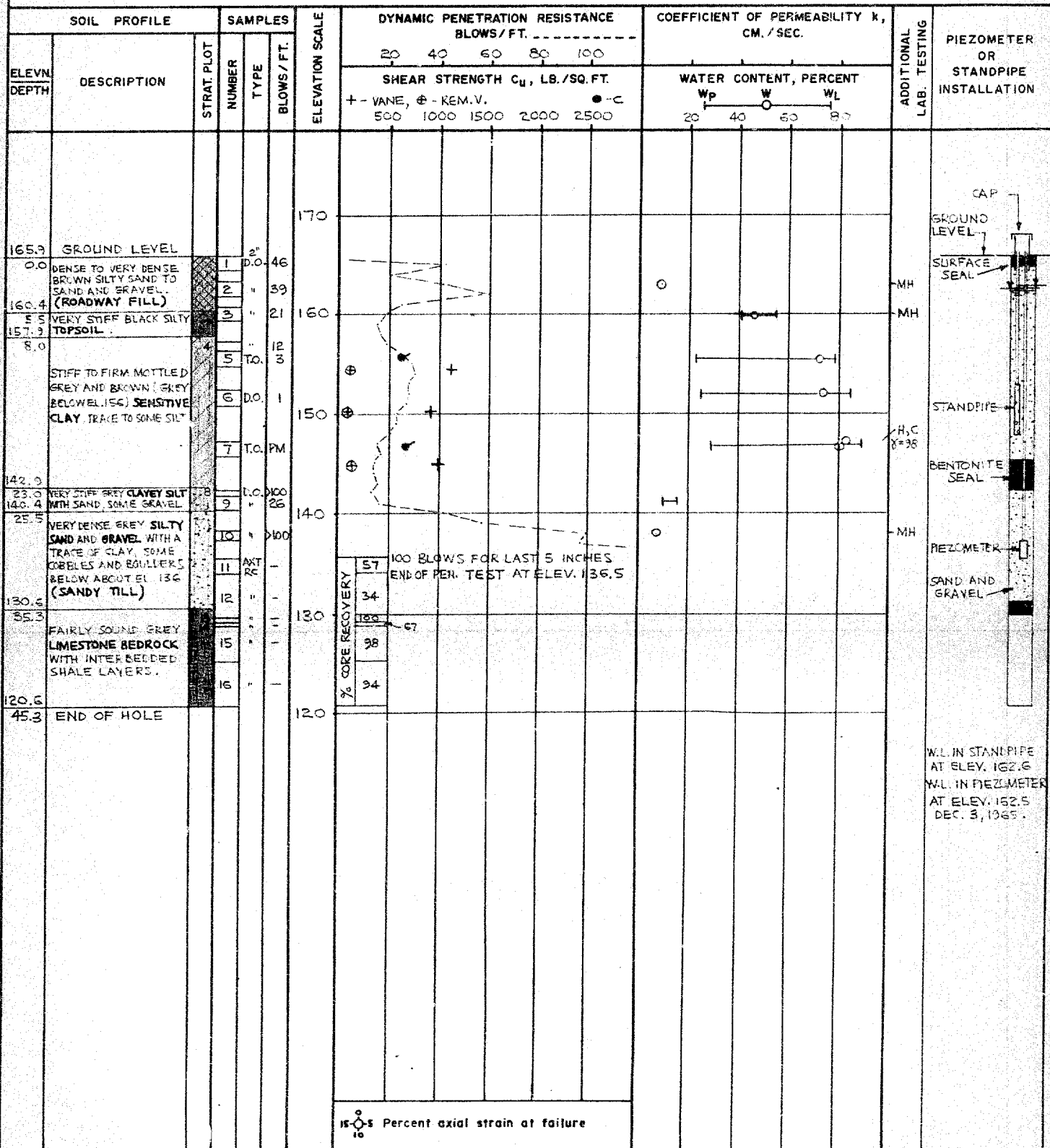
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX, BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



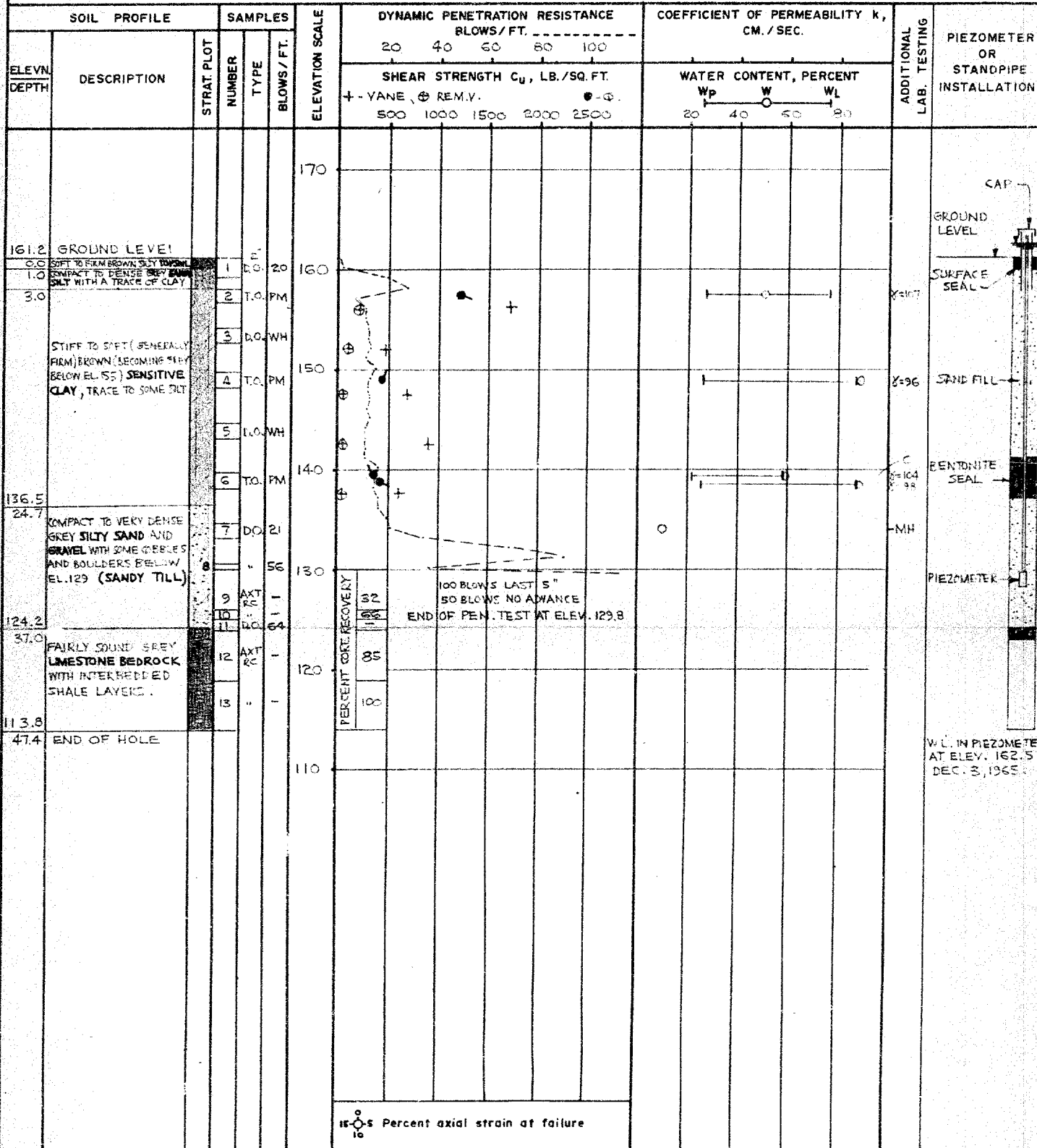
VERTICAL SCALE,
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *MAS*
CHECKED *MAS*

RECORD OF BOREHOLE 5

LOCATION See Figure 1 BORING DATE NOV. 25 - 29, 1965 DATUM GEODETIC
 BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER NX-BX CASING
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.B.D.
 CHECKED J.B.D.

PROJECT NO. 68-097-225-0000

DATUM GEODETIC

BOREHOLE DIAMETER

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

WATER FLOWED
FREELY FROM OPEN
CONE HOLE.
HOLE PLUGGED
BETWEEN ELEV.
138 AND 140.

CHECKED TBD

PEN. TEST RECORD OF BOREHOLE 8

LOCATION See Figure 1

BORING DATE DEC. 1, 1965

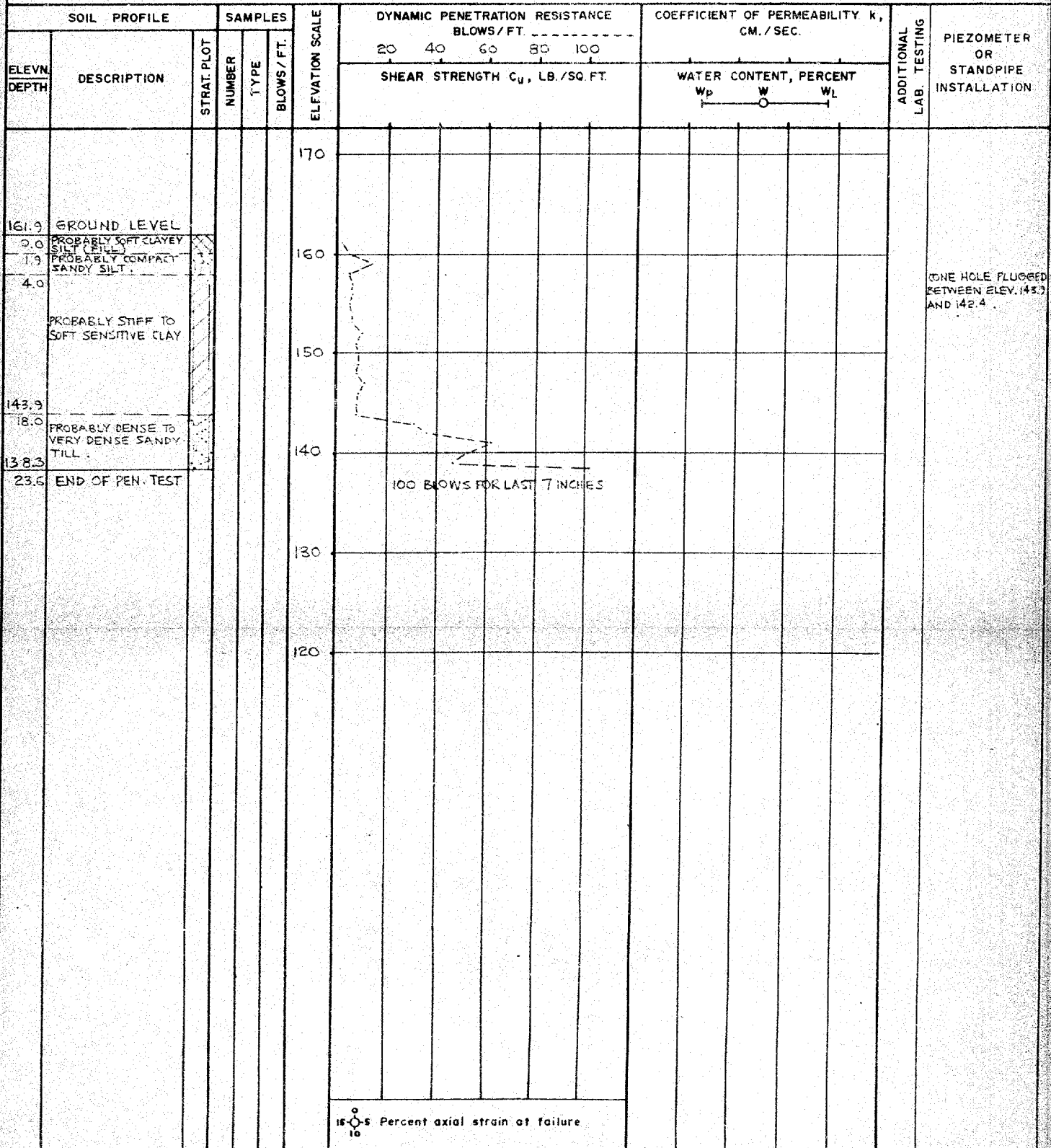
DATUM GEODETIC

BOREHOLE TYPE PENETRATION TEST

BOREHOLE DIAMETER -

SAMPLER HAMMER WEIGHT - LB. DROP - INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

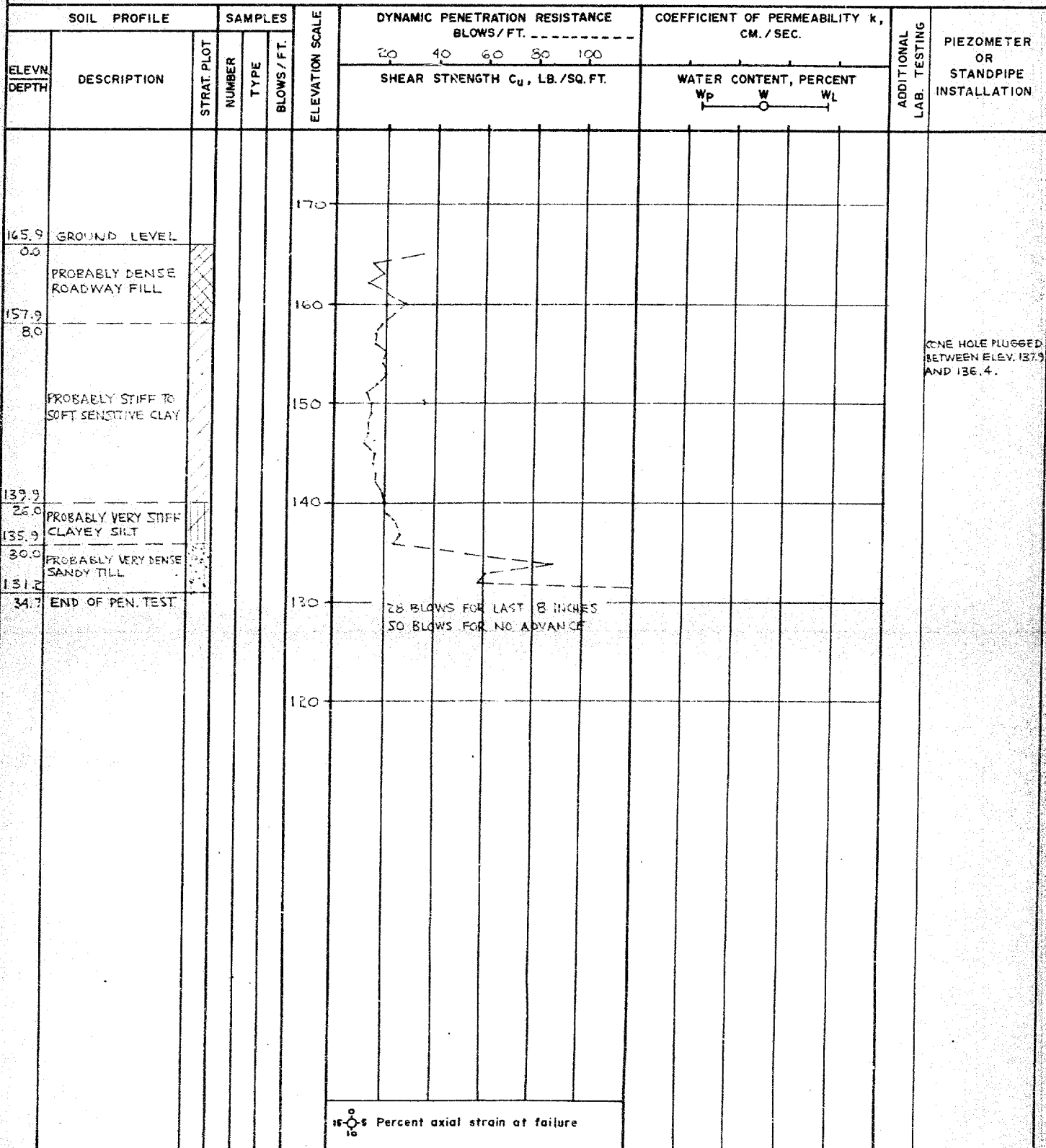
ONE HOLE PLUGGED
BETWEEN ELEV. 143.9
AND 142.4VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN *AW*
CHECKED *TS*

PEN. TEST RECORD OF BOREHOLE 9

LOCATION See Figure 1 BORING DATE NOV. 12, 1965 DATUM GEODETIC
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
 1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN J.A.
 CHECKED

PEN. TEST
RECORD OF BOREHOLE 10

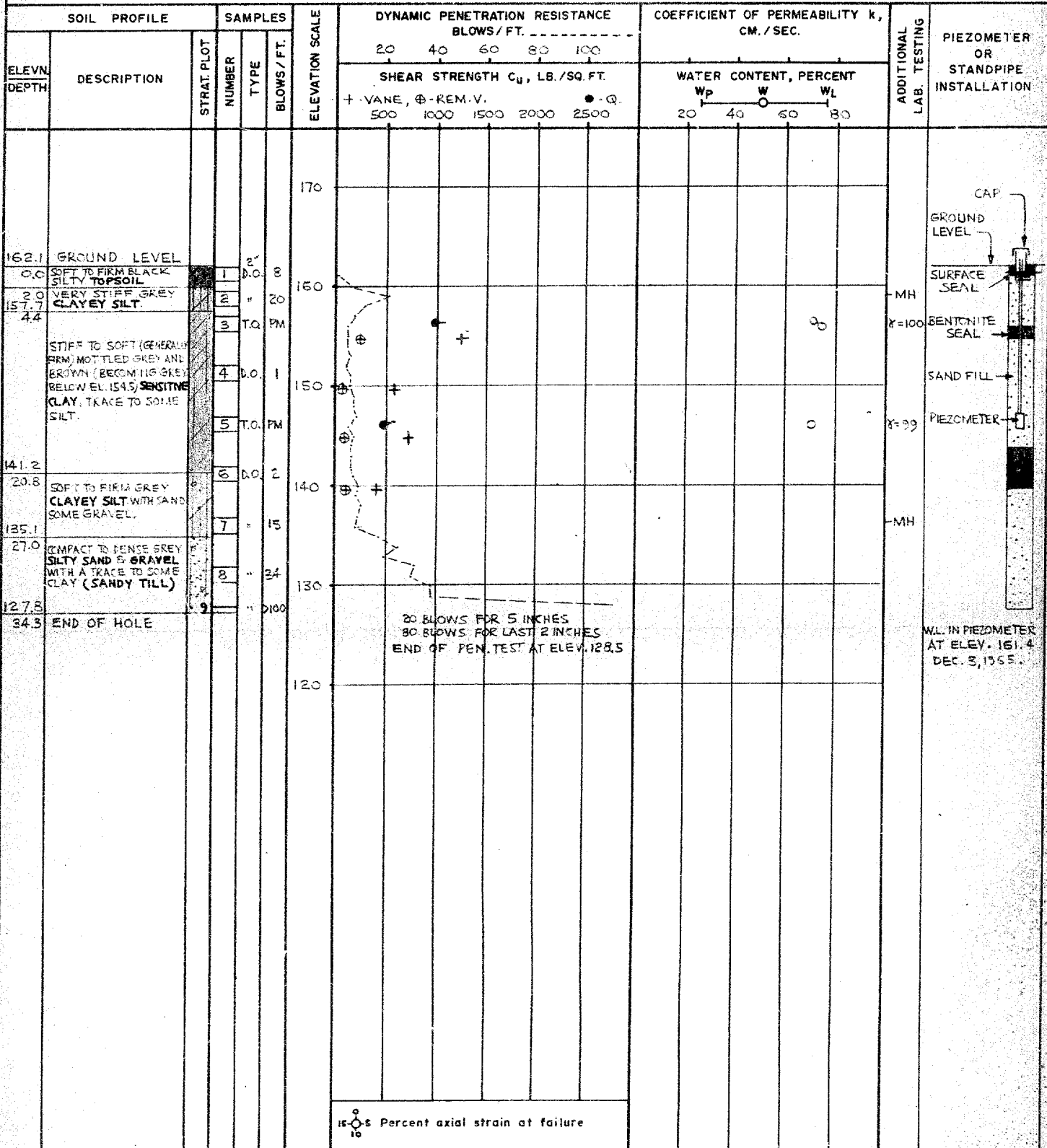
LOCATION See Figure 1 BORING DATE NOV. 25, 1965 DATUM GEODETIC
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —
 SAMPLER HAMMER WEIGHT - LB. DROP - INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE		SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FT.					COEFFICIENT OF PERMEABILITY k_v , CM./SEC.					ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		20	40	60	80	100	WATER CONTENT, PERCENT <div style="display: flex; align-items: center; justify-content: center;"> W_p — W — W_L </div>						
163.7	GROUND LEVEL																
160.0	PROBABLY TOP SOIL																
158.7	PROBABLY COMPACT TO DENSE SANDY SILT																
150.0	PROBABLY STIFF TO SOFT SENSITIVE CLAY																
140.0	PROBABLY COMPACT TO VERY DENSE SANDY TILL																
134.1	END OF PEN. TEST																
129.6																	

CONE HOLE PLUGGED BETWEEN ELEV. 148.7 AND 147.2.

RECORD OF BOREHOLE 11

LOCATION See Figure 1 BORING DATE NOV. 30-DEC. 1965 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"

GOLDER & ASSOCIATES

DRAWN M.W.
 CHECKED J.B.

RECORD OF BOREHOLE 12

LOCATION See Figure 1

BORING DATE DEC. 3, 1965

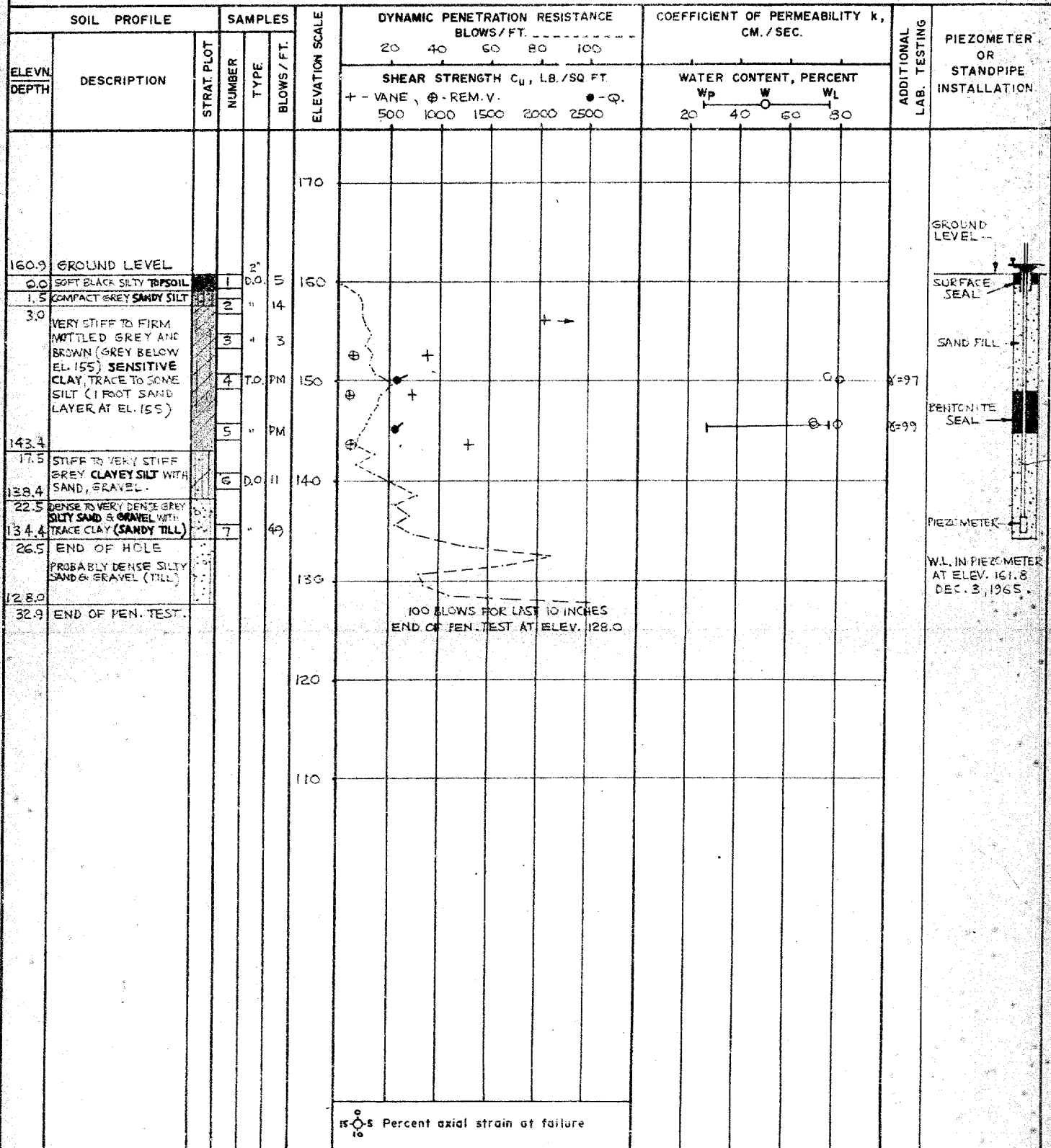
DATUM GEODETIC

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

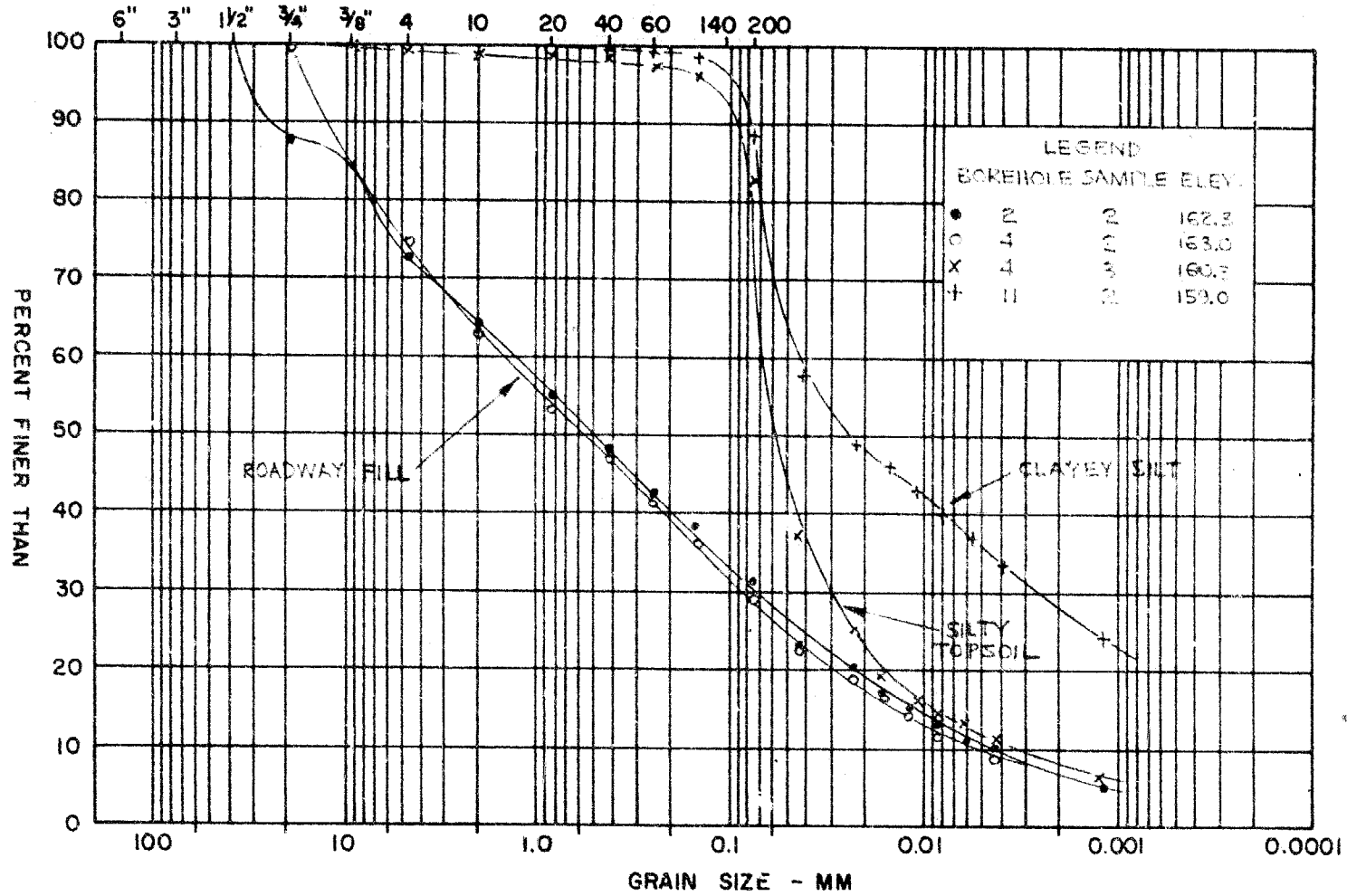
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN *W.W.*CHECKED *T.S.*

M.I.T. GRAIN SIZE SCALE

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



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

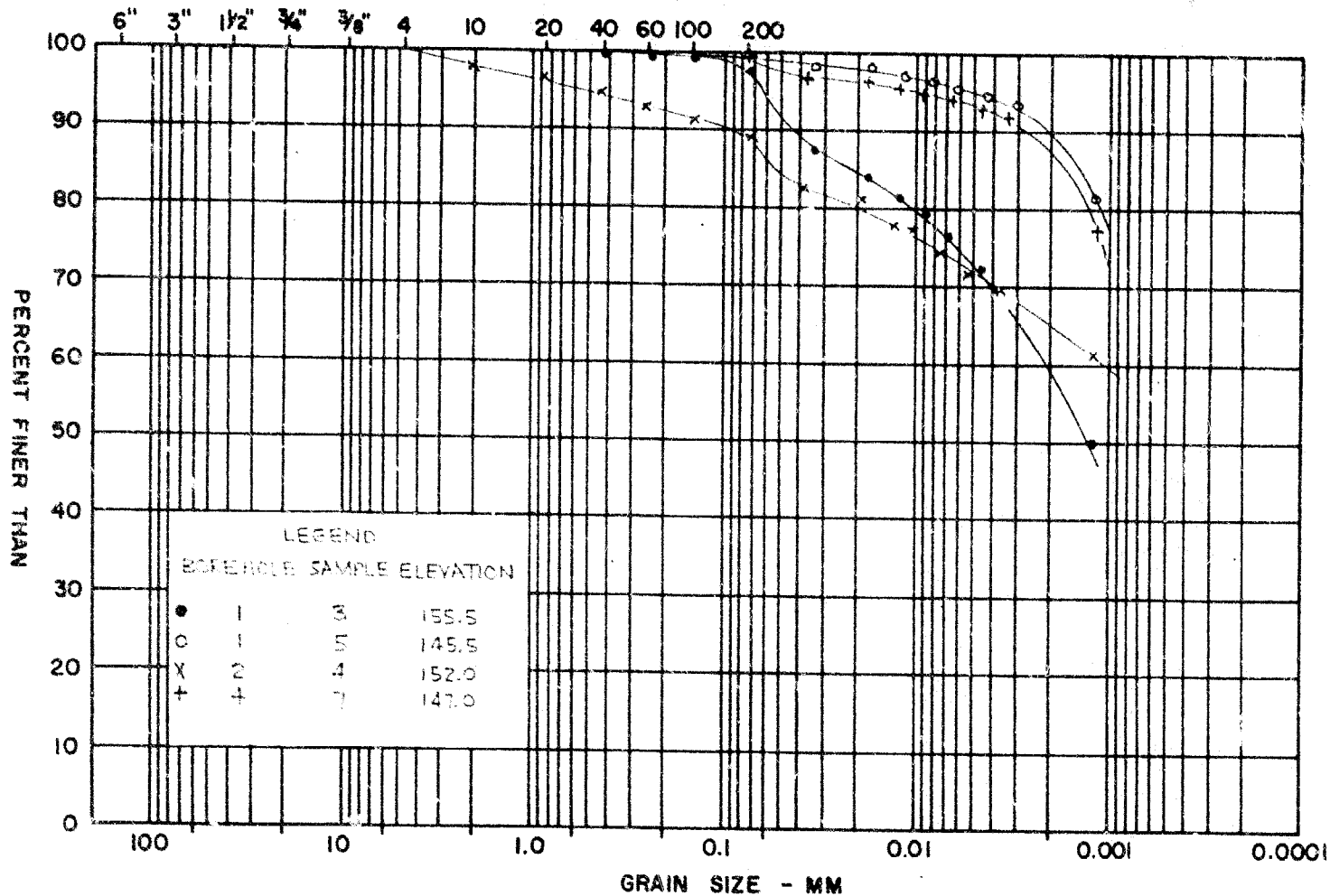
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SURFICIAL DEPOSITS

FIGURE 2

M.I.T. GRAIN SIZE SCALE

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



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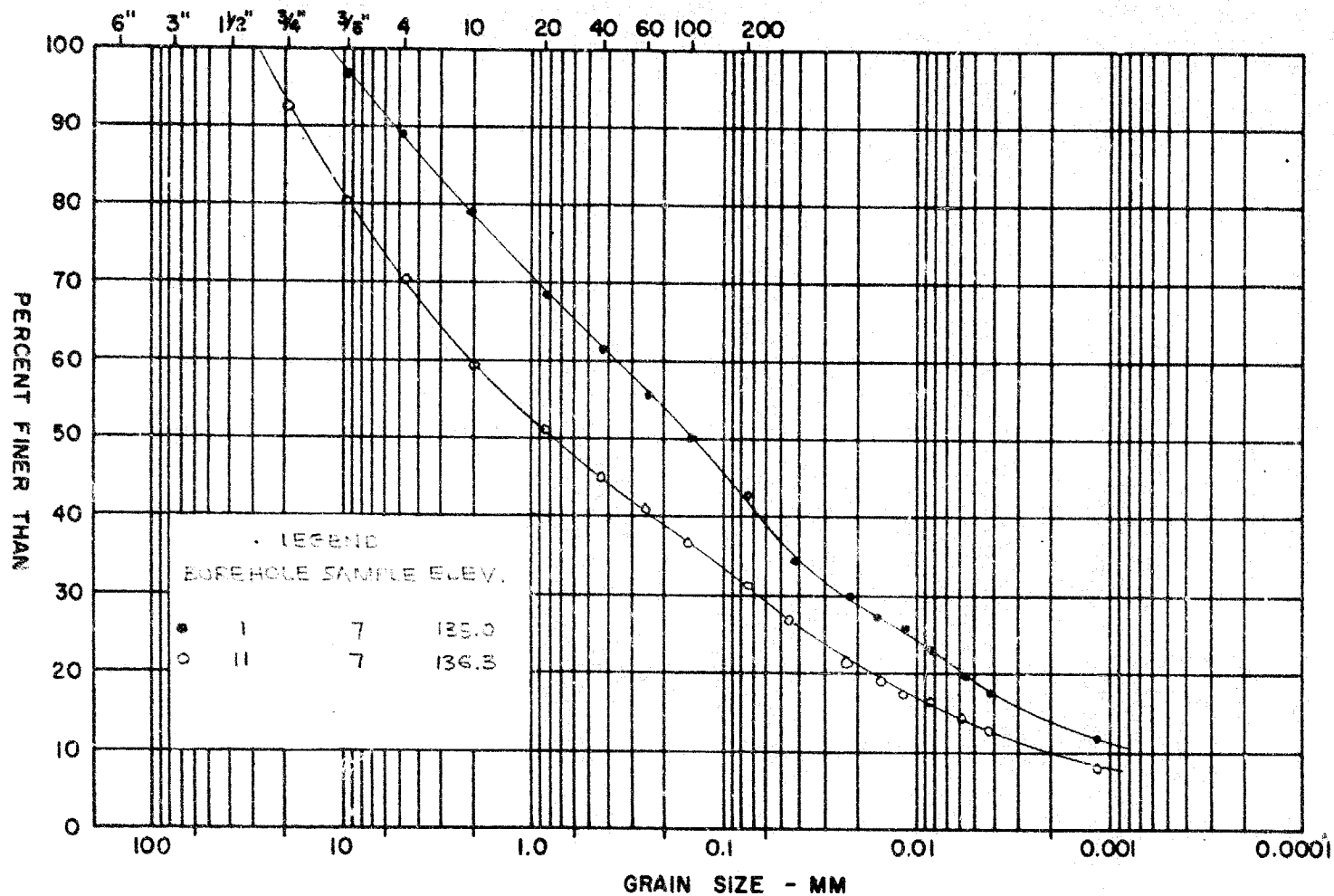
GRAIN SIZE DISTRIBUTION
SENSITIVE CLAY STRATUM

FIGURE

(1)

M.I.T. GRAIN SIZE SCALE

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



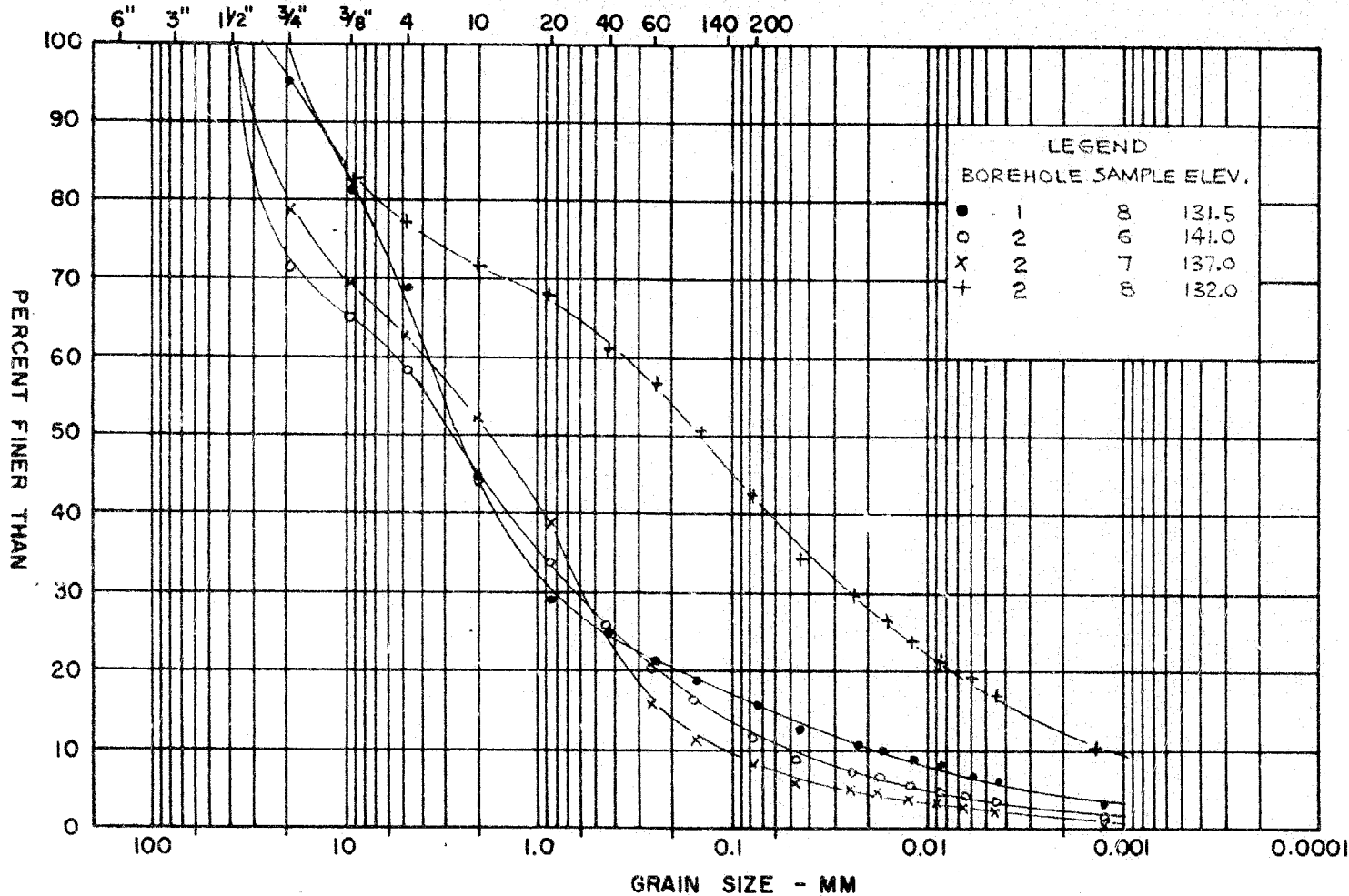
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

GRAIN SIZE DISTRIBUTION
CLAYEY SILT WITH SAND STRATUM

FIGURE

M.I.T. GRAIN SIZE SCALE

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

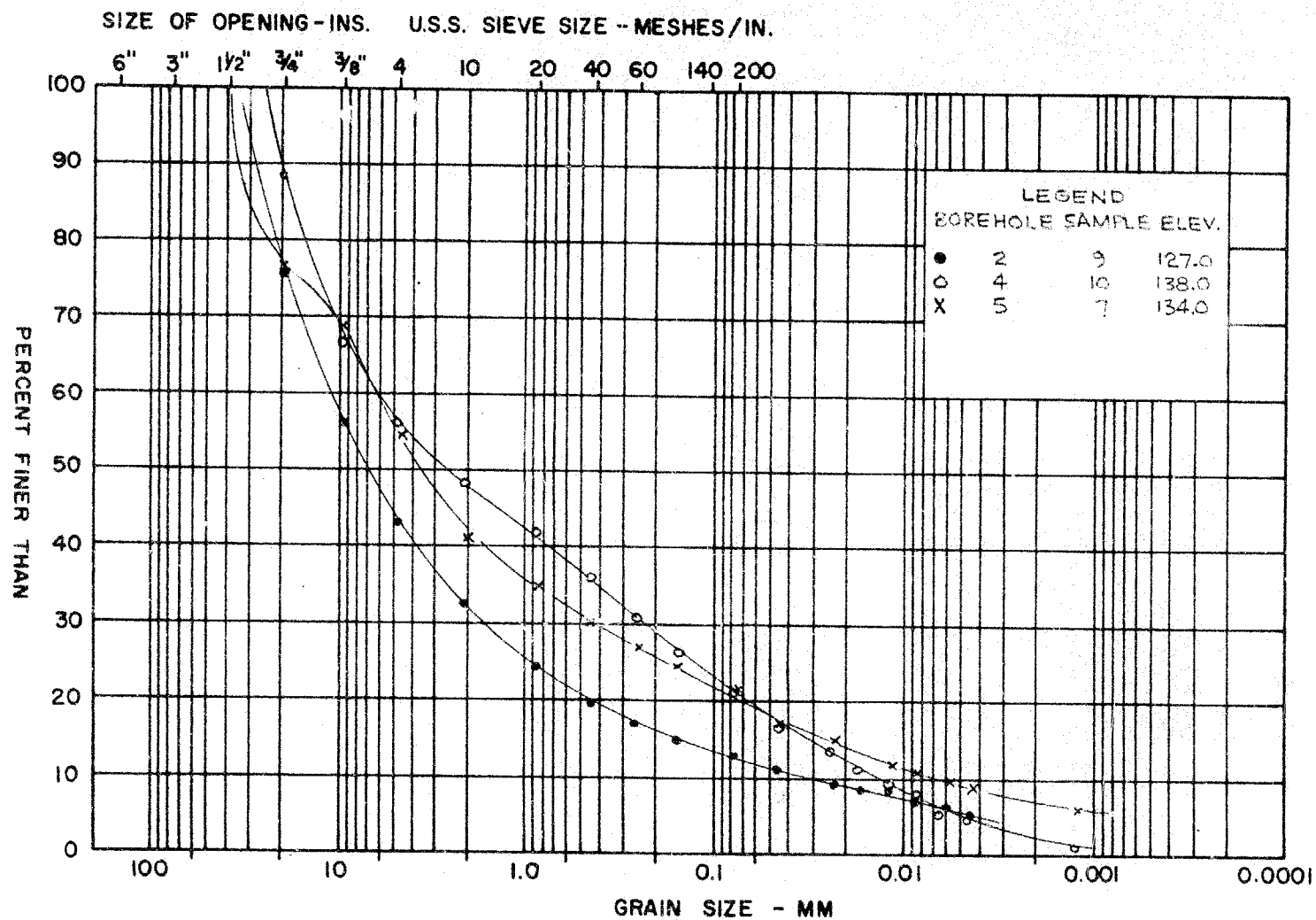


GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
TILL STRATUM

FIGURE 5

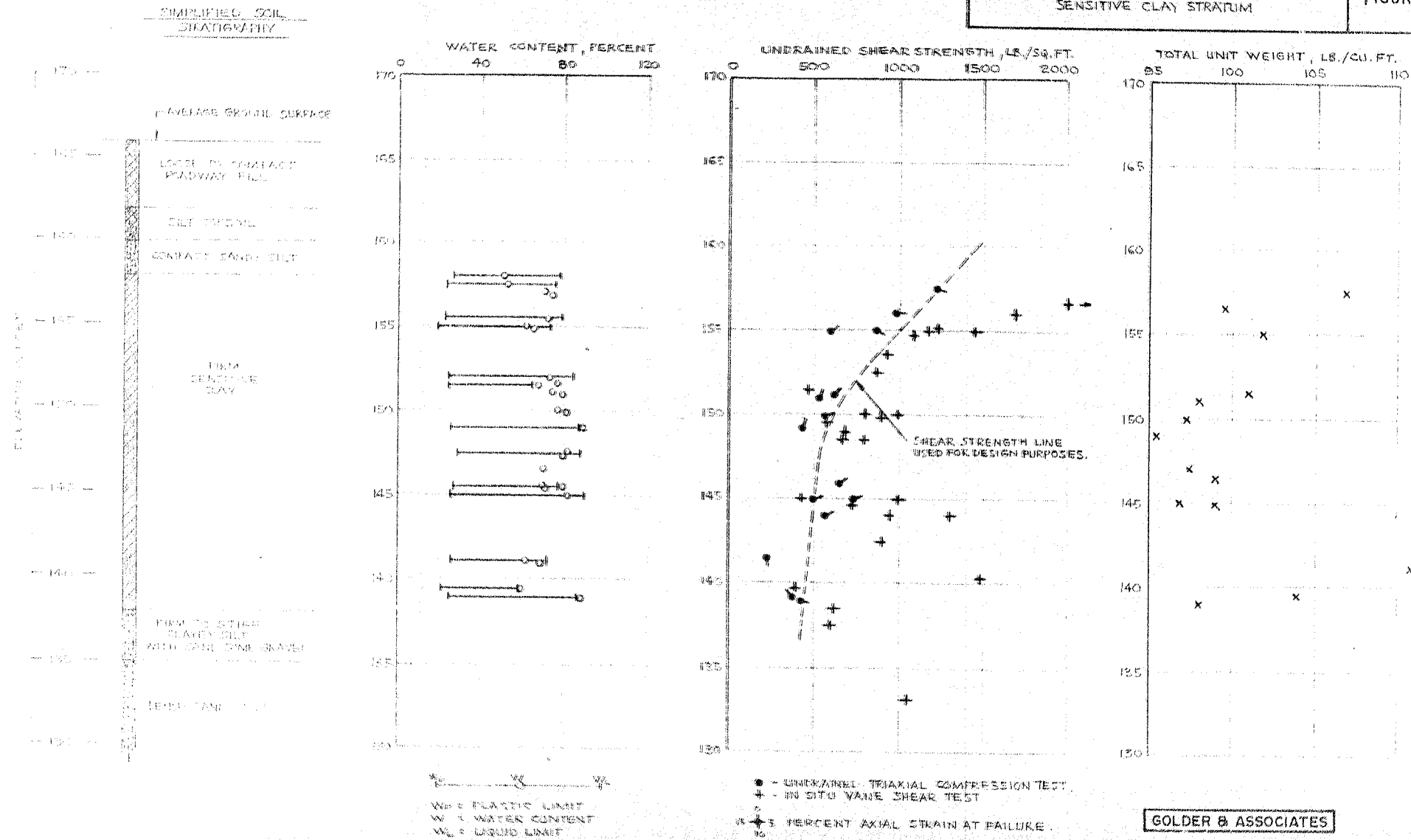
M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

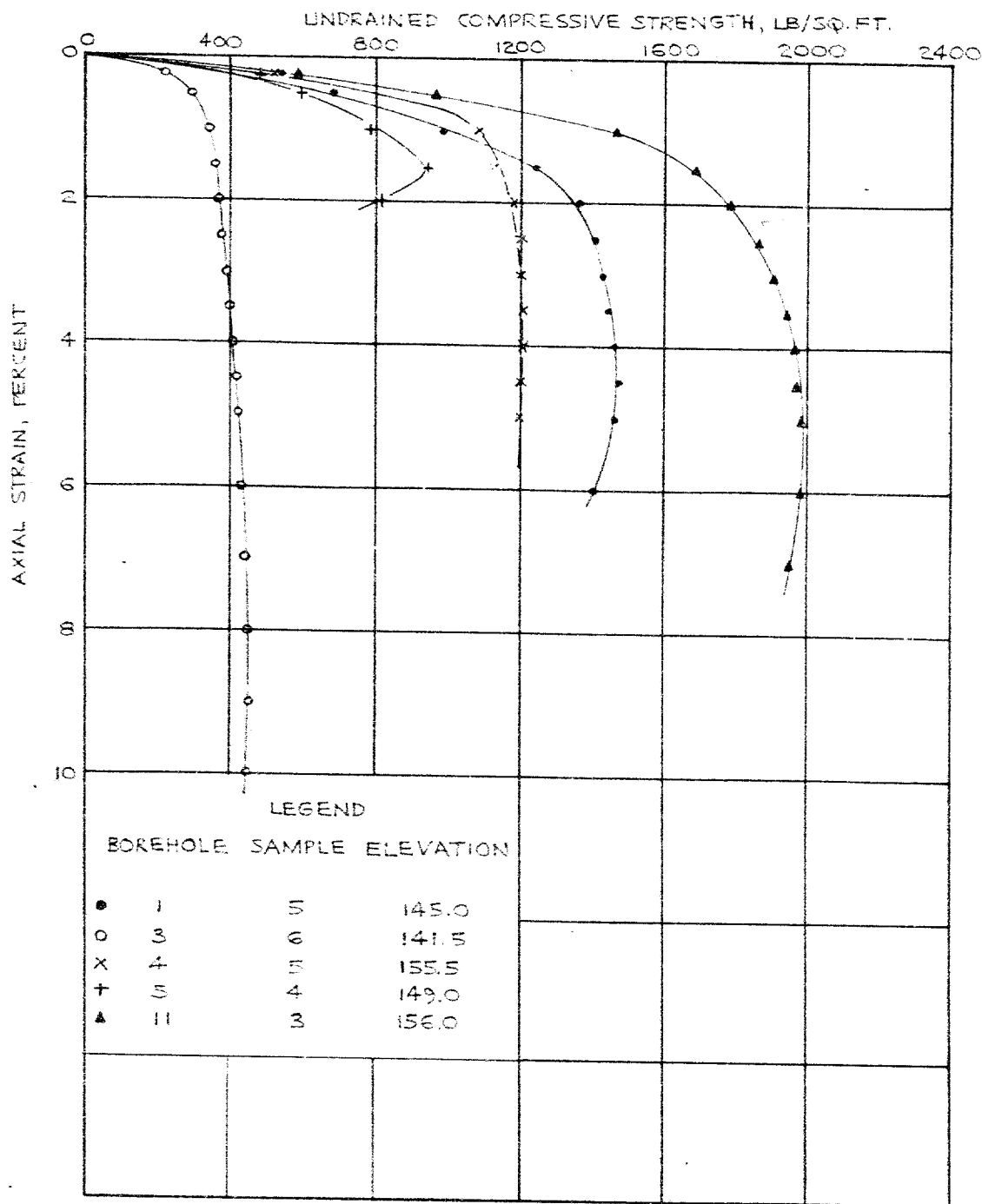
GRAIN SIZE DISTRIBUTION
TILL STRATUM

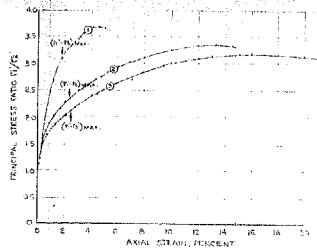
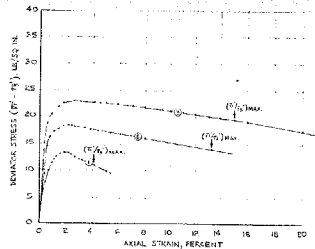
FIGURE 6



UNDRAINED TRIAXIAL COMPRESSION TESTS TYPICAL STRESS- STRAIN CURVES SENSITIVE CLAY STRATUM

FIGURE 8

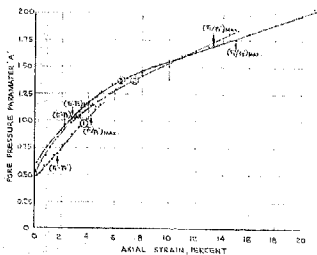
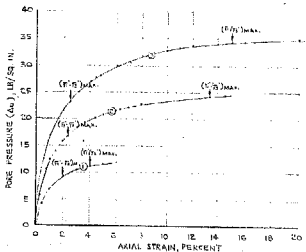




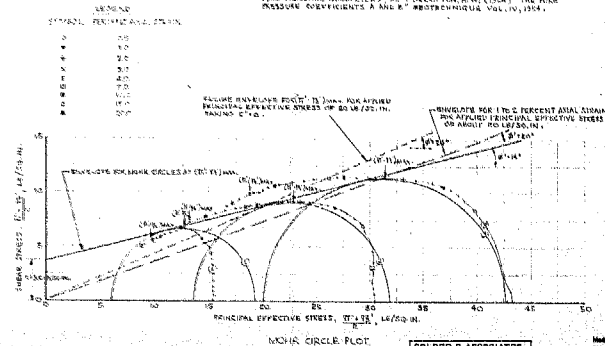
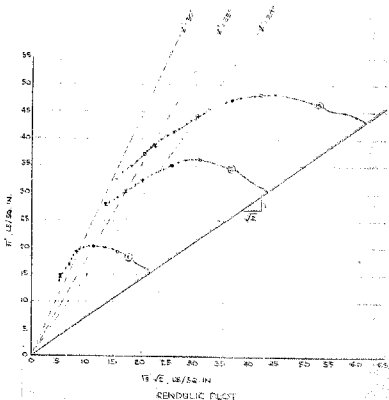
LEGEND					
TEST	HOLE	SAMPLE	ELEVATION	LIQUID LIMIT	PLASTIC LIMIT
1	2	4	151.6	65.0	24.1
2	1	3	144.2	83.0	25.7
3	4	5	146.4	88.0	18.5

TEST	CELL PRESSURE lb/sq. in.	AVERAGE RATIO OF STRAIN $(\epsilon'/\epsilon'')_{max}$, %	W ₁ %	W ₂ %	γ , %
1	10.4	0.6	76	75	1.00
2	30.5	0.5	81	61	0.73
3	45.3	1.0	81	52	0.38

* PORE PRESSURE PARAMETERS, REF. SKEMPTON, A. W. (1954) "THE PORE PRESSURE COEFFICIENTS A AND B" GEOTECHNICAL VOL. IV, 1954.

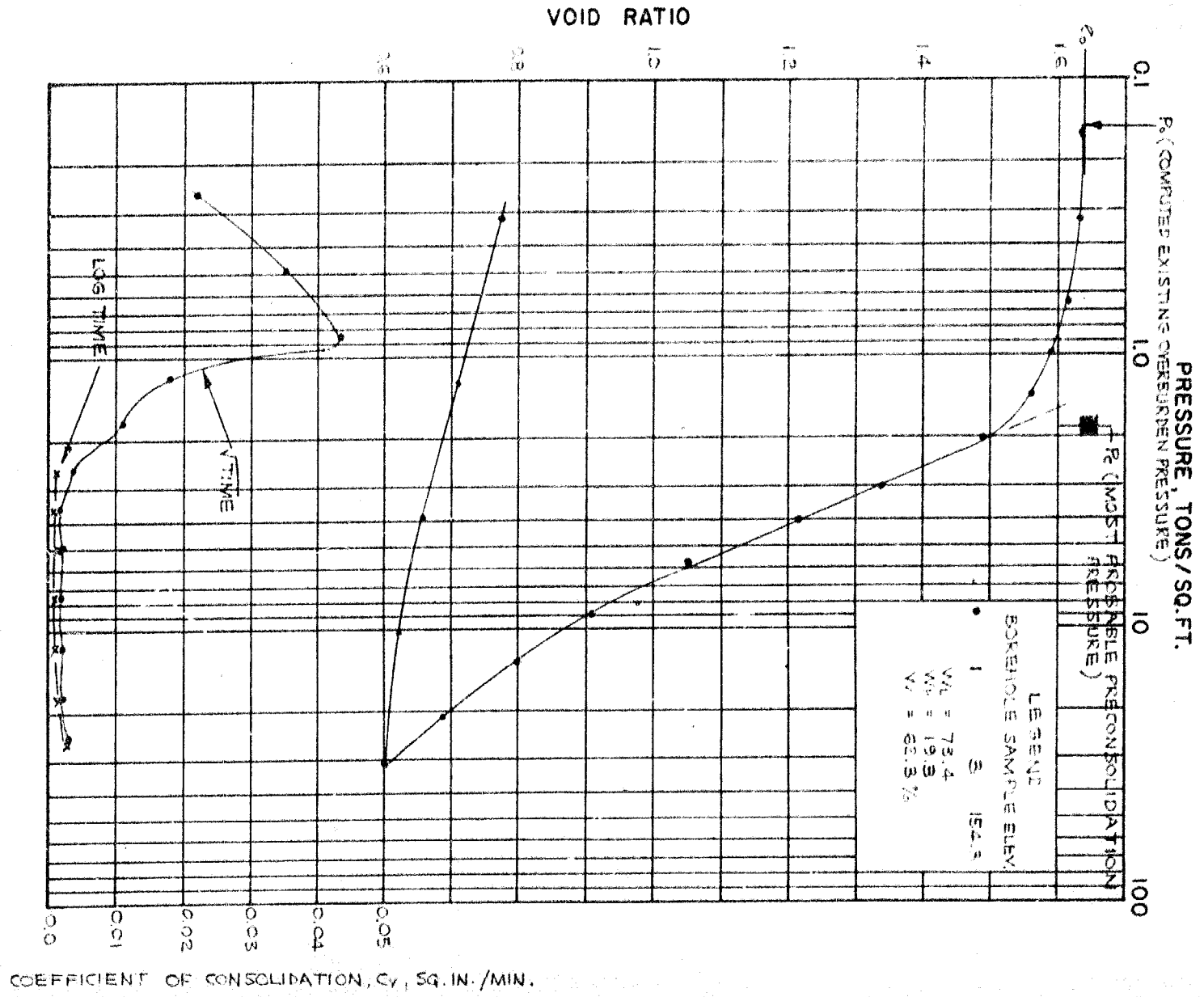


NOTE: PORE PRESSURE PARAMETER 'B' WAS
FOUND TO BE BETWEEN ABOUT 0.35 AND 0.5.



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

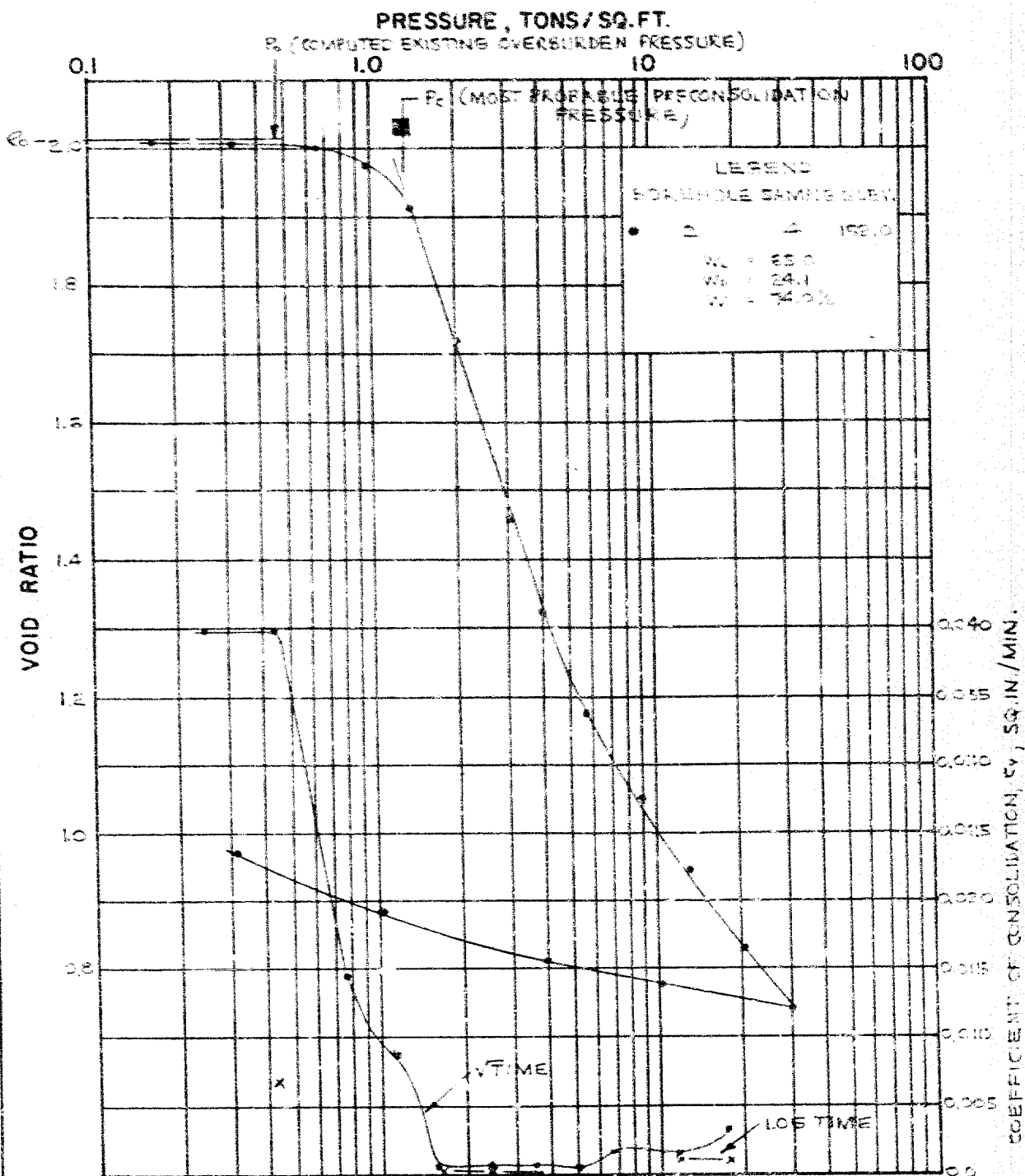
FIGURE 10



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

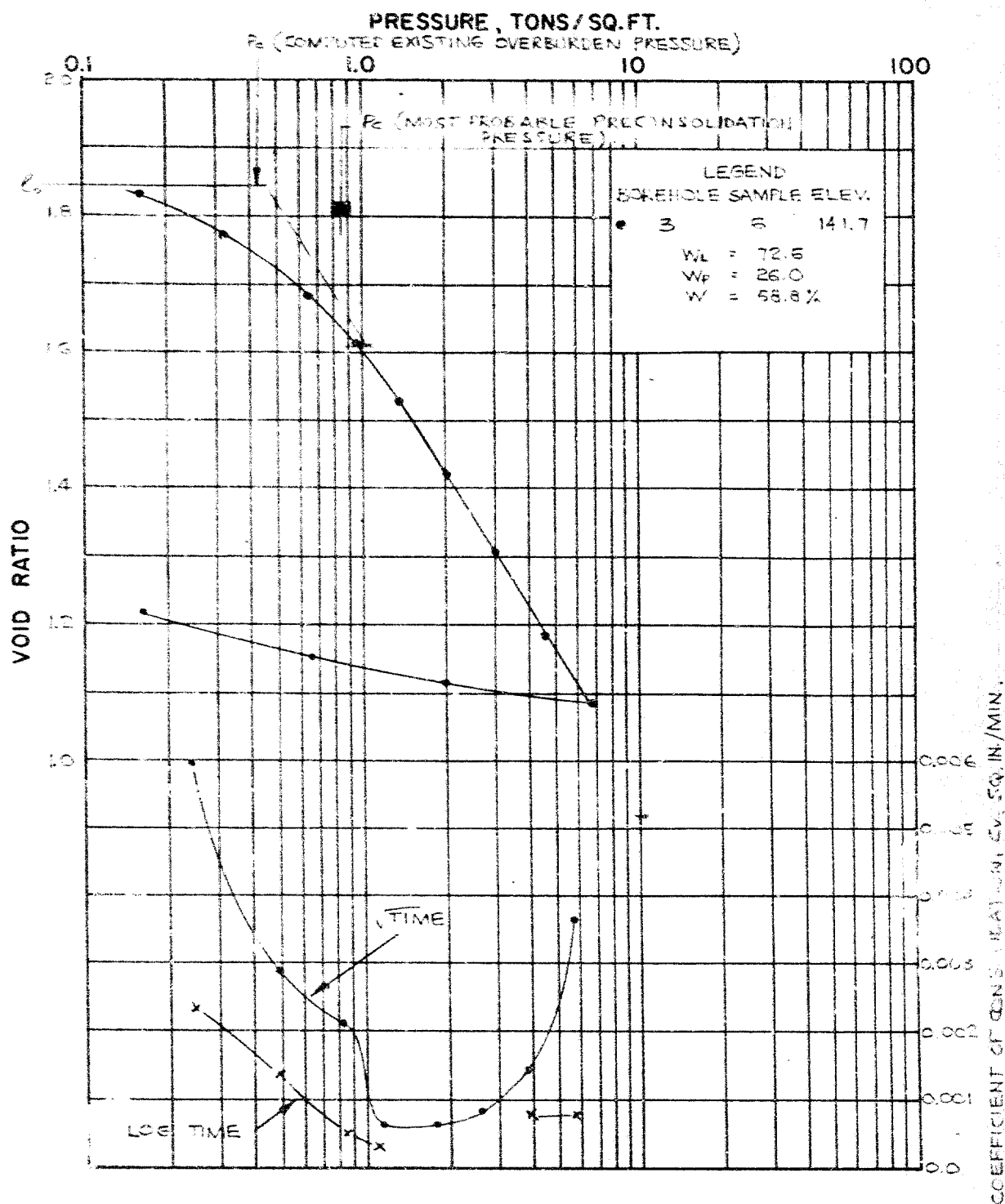
FIGURE 11



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 12

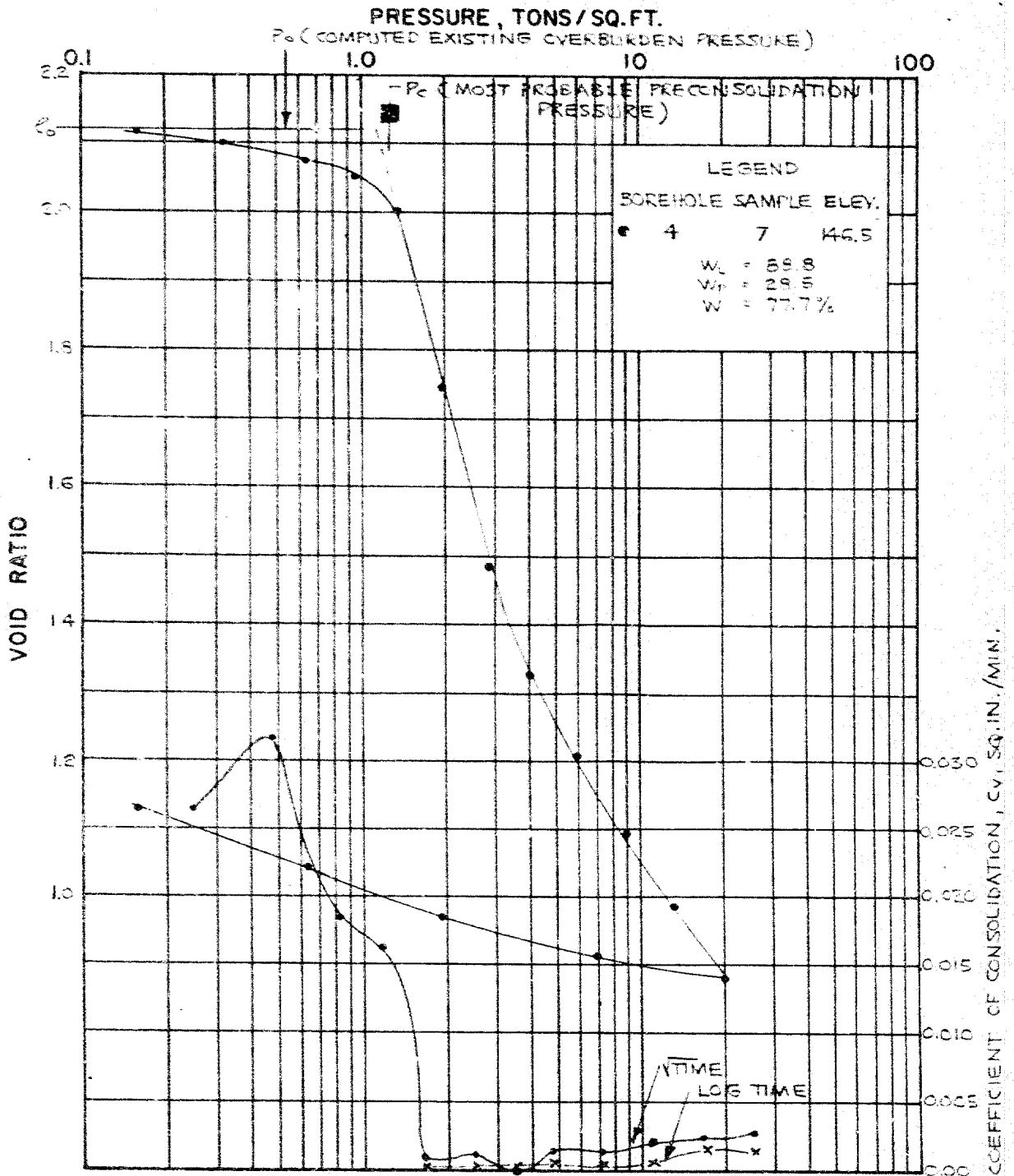


GOLDER & ASSOCIATES

PROJECT NO. 65/35

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

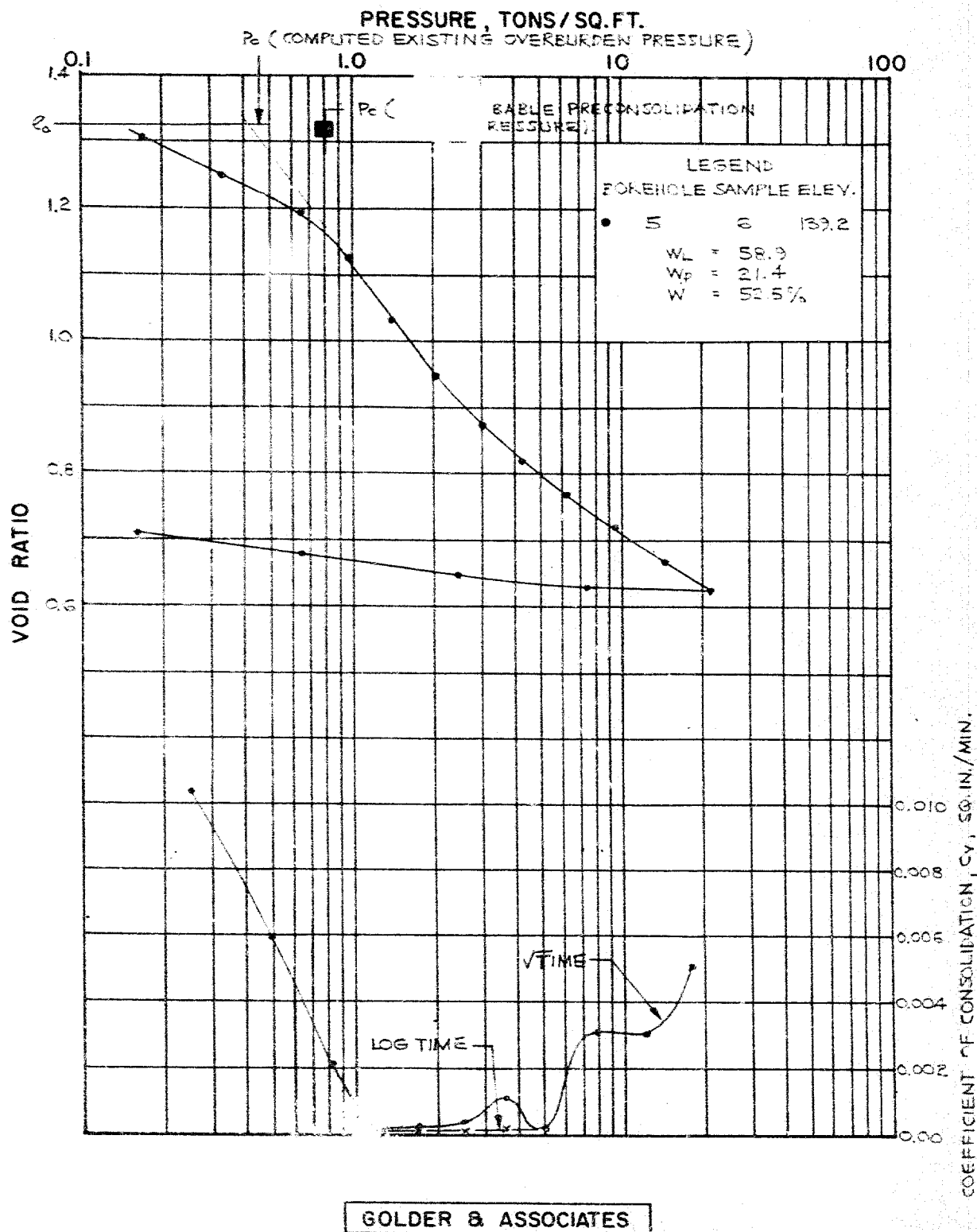
FIGURE 13



GOLDER & ASSOCIATES

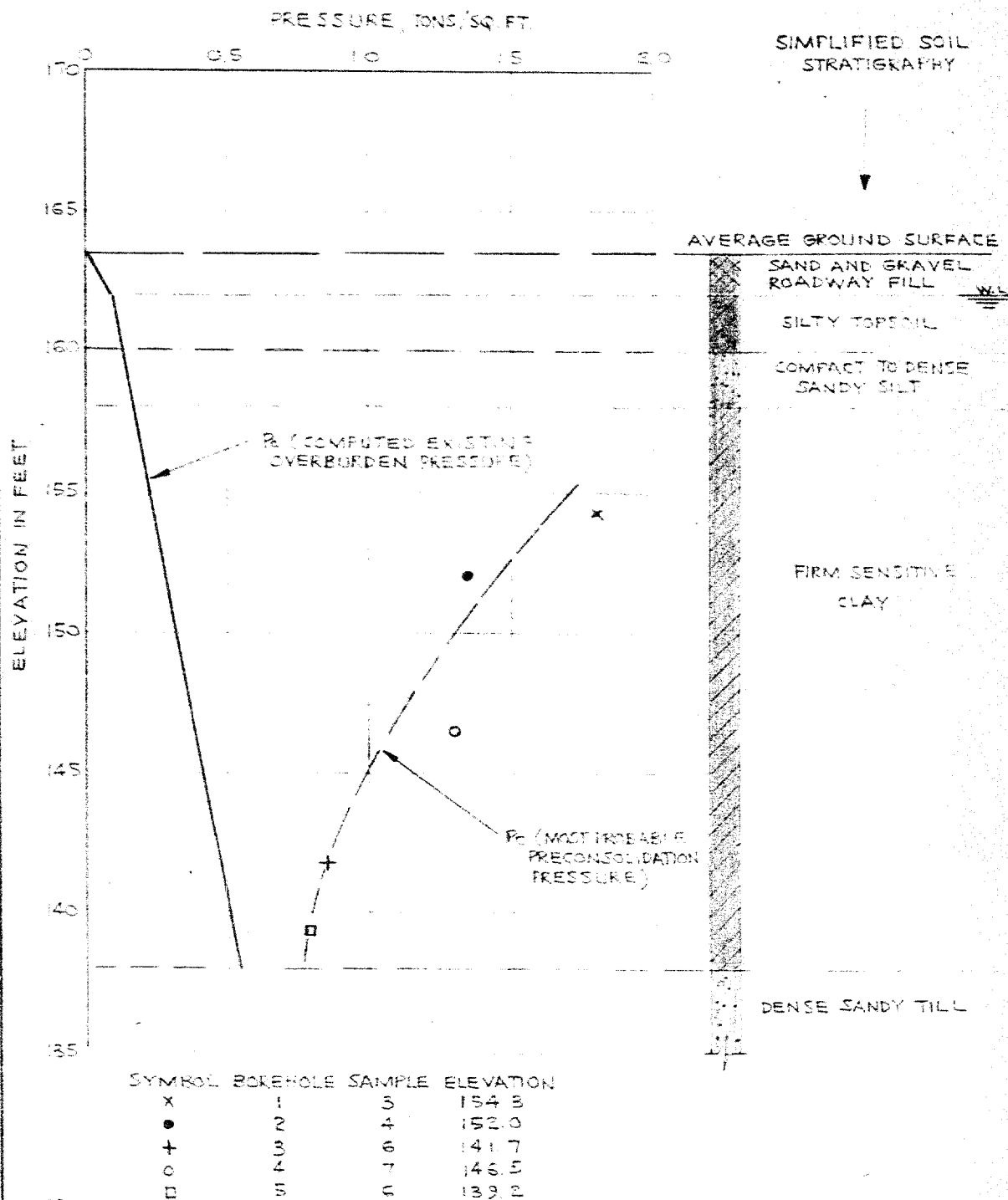
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 14



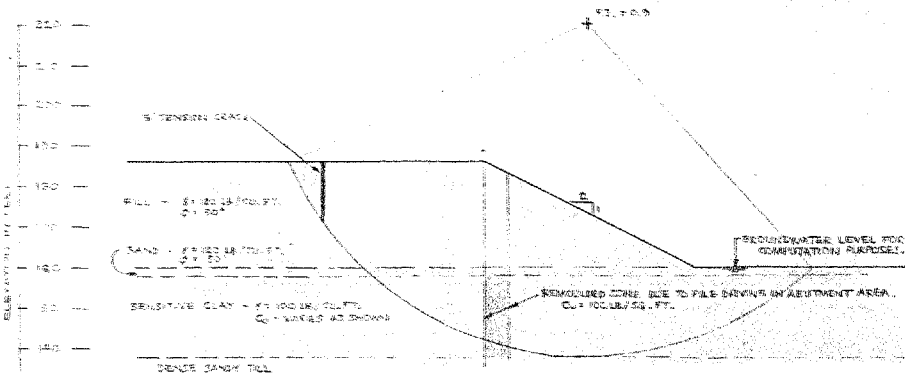
SUMMARY PLOT PRECONSOLIDATION PRESSURE VS ELEVATION SENSITIVE CLAY STRATUM

FIGURE 15

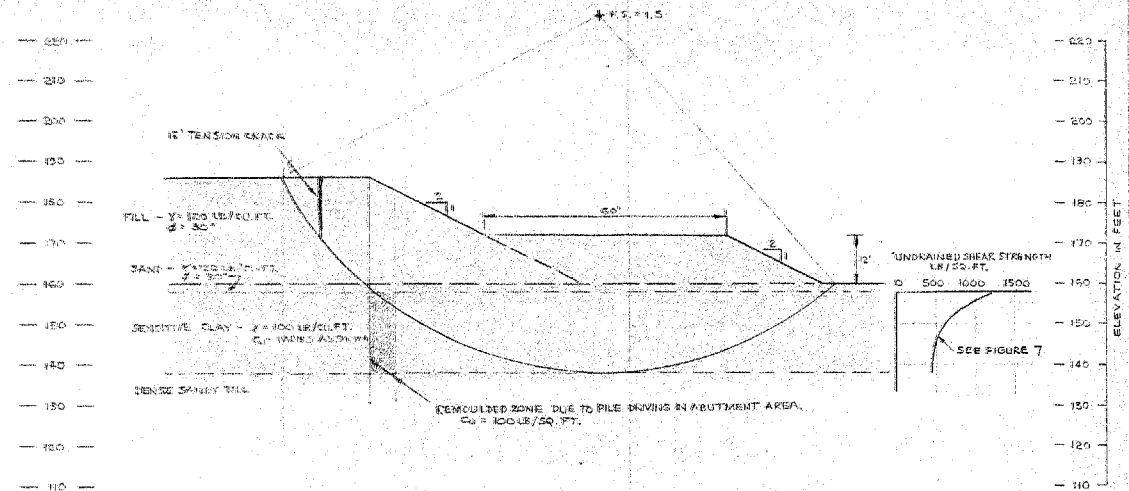


GOLDER & ASSOCIATES

Made *[Signature]*
 Chkd. *[Signature]*
 Appd. *[Signature]*

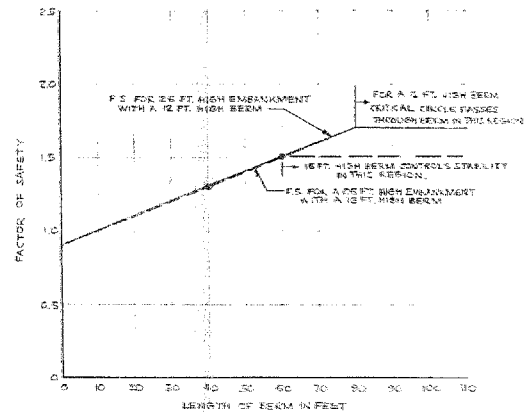
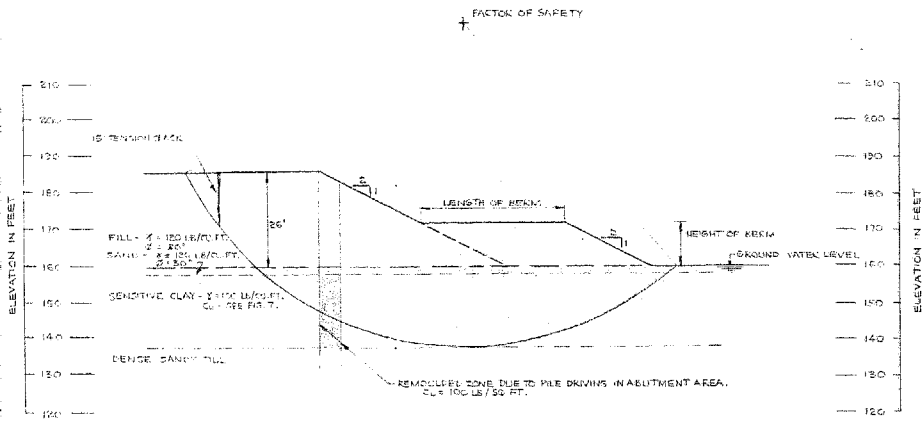
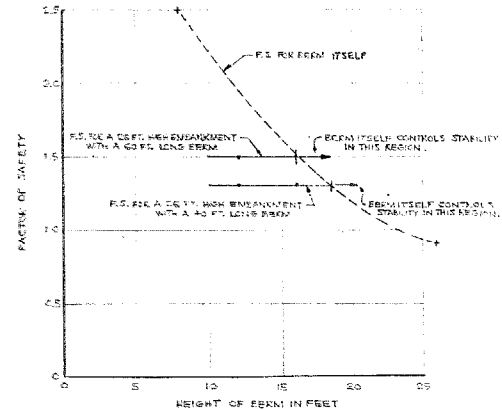


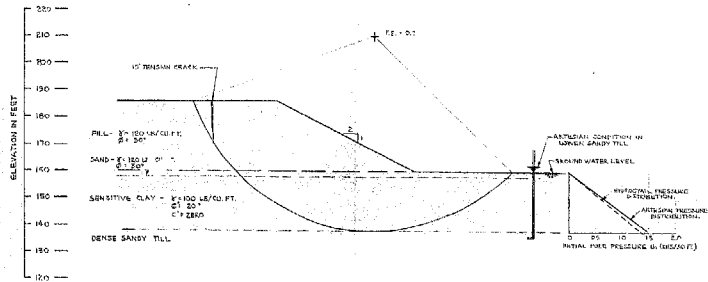
TOTAL STRESS ANALYSIS
26' HIGH EMBANKMENT



TOTAL STRESS ANALYSIS
26' HIGH EMBANKMENT
WITH A 12 FT. HIGH, 60 FT. LONG BERM.

SCALE 1" TO 20'-0"

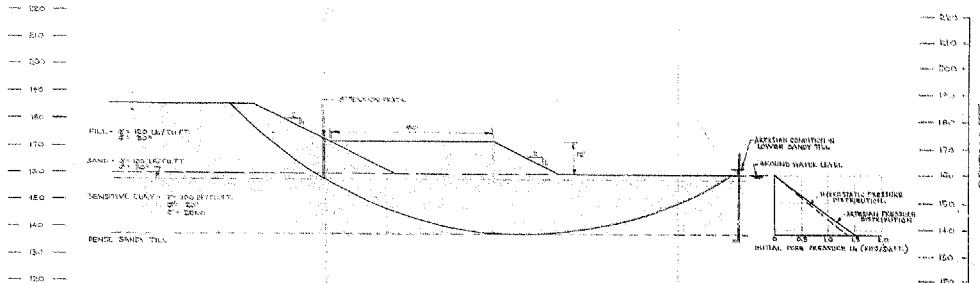
VARIATION OF FACTOR OF SAFETY WITH
LENGTH OF BERM.VARIATION OF FACTOR OF SAFETY WITH
HEIGHT OF BERM.



EFFECTIVE STRESS ANALYSIS
22' HIGH EMBANKMENT

EXCESS PORE PRESSURE (Δu) AT ANY POINT ON THE FAILURE CIRCLE WAS COMPUTED AS:
 $\Delta u = B \Delta \sigma'$
WHERE $\Delta \sigma'$ = EFFECTIVE WEIGHT OF EMBANKMENT DIRECTLY ABOVE POSITION OF CIRCLE UNDER CONSIDERATION.
 $B = 0.5$ (OVERALL PORE PRESSURE PARAMETER)

SCALE 1" TO 10'-0"



EFFECTIVE STRESS ANALYSIS
22' HIGH EMBANKMENT
WITH 12' HIGH, 6' BY 12' CONC. PIER

Drawn JAN. 11, 1966.

GOLDER & ASSOCIATES

Moore, R. L.
Chkd. J. J. G.
Appd. J. J. G.

Hwy. 401 & Keels St.,
Downsview, Ontario.

November 4, 1965

Materials and Testing Division

M. T. Golder and Associates Ltd.,
2444 Bloor Street West,
Toronto, Ontario.

Attention: Mr. J. Seychuk

Re: R.F. 107-59 -- Site No. 31-230,
Fraser Road Underpass,
2.8 Miles West of Jct. Hwy. 2 & 34,
Hwy. 401 -- District 3 (Ottawa).

Dear Sir:

This is to authorize you to carry out a foundation investigation at the above-mentioned site. The plan showing the locations of the proposed crossing and grade, have been handed to your Mr. J. Seychuk, on November 4, 1965.

Preliminary soil surveys carried out by the Department have indicated the presence of a deposit of very soft clay. It is, therefore, believed that some problems regarding the stability of the approach embankments could be encountered. In the same general area, difficulties with newly built structures have been and are still being experienced.

The above is brought to your attention so that you can organize the necessary field exploration work. It is understood that a qualified Soils Engineer will be in charge of the field work at all times.

Ten (10) copies of the completed report should be submitted to the Foundation Section not later than December 17, 1965. Previous requirements as to preliminary borehole information and laboratory testing program, should be followed.

Because the drawings accompanying the Foundation reports, showing the location of borings, the inferred subsoil conditions, etc., are to become contract drawings, you are requested to prepare them in accordance with the U.E.C. standards. To enable

H. Q. Golder & Associates, - 2 -
Attn: Mr. J. Seychuk.

November 4, 1965

you to do this, we are supplying you with sample drawings with all the necessary explanations, together with linen sheets for your drawings. You are also requested to provide us with Cronaflex copies of the drawings.

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

We are attaching Purchase Order J 34797, covering the purchase of any new material required for this work, in order that you may use this as a basis for exemption from the Federal tax for such purchases. The Exemption Certificate is printed thereon.

Yours very truly,

AGS/mdeF
Attach.

a.f.
A. Rutke,
MATERIALS & TESTING ENGINEER

cc: Messrs. S. McCombie
H. S. Piller
L. E. Walker
J. E. Graspier
A. Crowley
Mrs. I. Steinberg
B. Konings
H. Brymanski (2) ✓
Foundations Office
Gen. Files (2)

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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

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

January 18, 1966.

Department of Highways, Ontario,
Materials & Testing Division,
Hwy. 401 and Keele Street,
DOWNSVIEW, Ontario.

Attention: Mr. A. G. Stermac, P.Eng.,
Principal Foundation Engineer.

RE: SUBSURFACE INVESTIGATION,
W.P. 107-59 - SITE NO. 31-230,
PROPOSED FRASER ROAD UNDERPASS,
HIGHWAY 401 - DISTRICT 9 (OTTAWA),
GLEN GARRY COUNTY, ONTARIO.

Dear Sirs:


Ten copies of our report for the above investigation were delivered to you today by messenger. A Cronaflex copy of Figure 1 from the report was also included with the report shipment.

As discussed during a meeting at our offices on January 12, 1965, this report presents all of the work carried out by us to January 12, 1966. Once you have studied our report a further meeting is to be arranged to discuss the possible solutions outlined in our report and to decide on the most practical and economical solution to the problem.

If you have any questions in the meantime, please call us.

Yours truly,

H. Q. GOLDER & ASSOCIATES LTD.,



J. L. Seychuk, P.Eng.

JLS:hdg
65135

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

Attention: Mr. S. MacCombie

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

January 19, 1966

JAN 19 1966

FOUNDATION INVESTIGATION REPORT BY
S. Q. Golder and Associates Limited -
W.P. 107-59 -- Site No. 31-230,
Fraser Road Underpass, 2.8 Miles West
of Jet. Hwy. 2 & 34, Hwy. 401, Twp. of
Charlottenburgh -- District 9 (Ottawa).

Attached, please find the above-mentioned report prepared and submitted by the Consultant, S. Q. Golder and Associates Ltd.

We have reviewed the report and have found that it contains all the necessary factual information, and that all the necessary laboratory work was carried out.

The report deals with the problem of embankment stability in great detail. This is due to the unsatisfactory performance of some bridges in the same area founded on comparable subsoil conditions. The undersigned has discussed with the Consultant, the probable causes of the unsatisfactory behaviour of some of the bridges in the area, and a consensus of opinion was reached that a more rigid type of piled foundation is definitely desirable and that the stability of the approach embankments should be considered also from the strain point of view, rather than only from the failure stress point of view.

It is beyond doubt that counterbalancing berms are needed in order to provide stability of the embankment. The final and exact length of the berms remains yet to be determined. We would, therefore, suggest that a meeting be called at which the entire problem should be discussed, everybody be familiarized with the thinking leading to the design criteria, and everybody be given an opportunity to voice his opinion. At this meeting, representatives of the Bridge Design, Bridge Location, Road Design, and Foundation Section, as well as the representatives of the Consultant, should take part.

AGS/MSF

Attach.

cc: Messrs. B. E. Davis (2)

E. A. Tregaskes

G. A. Farren

A. G. Pillar

L. E. Walker

J. E. Gruebler

A. G. Stersac
A. G. Stersac,
PRINCIPAL FOUNDATION ENGINEER

A. Watt

Foundations Office

Gen. Files

Mr. R. S. Pillar,
Senior Project Design Engr.,
Road Design Division (Kingston)

Foundation Section,
Materials and Testing Division,
Room 107, Lab. Bldg.

February 14, 1966

Your Memo -- Feb. 7, 1966

W.P. 107-59, Underpass at Fraser Road - Hwy. 401,
-- District No. 9 (Ottawa) --

With reference to your memorandum of February 7, 1966, to Mr. W. Wigle, Program Engineer, Program Section, Downsview, we wish to make the following comments:

It is realized by all parties concerned that bridges built in the Lancaster area, which are founded on sensitive clay deposits, have not performed as expected. Large settlements of the approach fills and certain movements of the abutments have necessitated considerable maintenance, and still do. Settlements of the approach fills are understood and were anticipated; however, the abutment movements were not anticipated, and are still not quite well understood. Only certain hypotheses have been put forth to explain these movements, but there is insufficient evidence to prove them. The only fact of which we are all convinced, is that the abutment movements are intimately related to the embankment settlements.

The largest part of the settlement of the approach embankments takes place during the first one to three years after construction. To take advantage of this fact, stage construction is recommended. The longer the staging - i.e., the longer the time between the fill placement and the construction of the bridge, the less maintenance will be required.

Due to the intimate relationship between the fill settlements and the abutment movements, the driving of the piles should be carried out during the second stage. Thus, the influence of the embankment settlements - whatever it may be - on the foundations, is decreased and the possibility of abutment movements becomes rather remote.

The recommendation that pile driving be carried out in Stage II, has also been discussed with Mr. L. E. Walker, District Engineer, Ottawa, and the consultant for the Fraser Road underpass, E. Q. Golder and Associates, and they both concur with our reasoning.

ACS/MEP

A. J. Sternac
A. J. Sternac,
PRINCIPAL FOUNDATION ENGINEER

cc: Messrs. L. E. Walker W. Wigle
 S. McCombie H.Q. Golder & Assoc. Ltd.
 D. Richardson
 C. J. Markiewicz
 Foundations Office
 Gen. Files

MEMORANDUM

To: W. Wigle,
Program Engineer,
Program Section,
Downsview.

From: H.S. Pillar,
Sr. Project Design Engineer,
Road Design, Kingston.

Date: February 7th, 1966.

Attention: D. MacFarlane

Our File Ref.

IN REPLY TO

SUBJECT: W.P. 107-59. U'Pass at Fraser Road - Pwy. #401 - Ottawa District.

A meeting was held on January 27th, 1966, with representatives of the Bridge Division, Road Design Division, and the Foundation Section to discuss anticipated foundation problems at the above-mentioned structure site.

It was the consensus of opinion that 60' earth berms were required and that the approach fills be constructed at least two years in advance of the bridge. Not only would savings be effected on the design of the abutments, but future maintenance costs i.e. asphalt padding, replacement of guiderail, curb and gutter, etc. on the approaches would be greatly reduced. A comparable example would be the structure at Brookdale Avenue where maintenance costs have accumulated to a sizeable sum as a result of the settlements similar to those which we hope to avoid at Fraser Road.

It is our recommendation that the present work project be divided as follows:--

- (1) W.P. 107-59-1 - To include grading and drainage on the approaches (including berms), relocation of service roads in conjunction with a detour for Fraser Road, and placing H piles for the abutments only. The total estimated cost for this work is \$60,000.00 approximately of which \$5,000.00 is allotted to the placing of the steel piles.
- (2) W.P. 107-59-2 - To include granular base, paving, curb and gutter, etc. on the approaches and the complete structure excepting the abutment piles. Minor grading and granular base will also be required in connection with the removal of the detour and completion of the service roads. The total estimated cost for this work is \$110,000.00

At present, this project is scheduled for an early award in 1967, however, I would recommend, providing funds are available, that the approaches be constructed as soon as possible this year and the structure re-scheduled for the summer or fall of 1968. Depending upon the results obtained from instrumentation of the approach fills, it may be possible to award the structure sooner.

In any event, I am presently preparing a property request which will be issued within a few weeks. If you agree, we could have a design prepared for the grading work only by July 1st, 1966.

Any work the Bridge Office desires to include ie: piling, etc. would then have to be completed by June 1st, 1966.

Please advise as soon as possible of your decision in this matter.

R.S. Pillar

R.S. Pillar
Sr. Project Design Engineer.

RSP/ss

C.C. to: L. Walker

A.G. Starnes ✓

A.P. Watt

B. Richardson

S.J. Markiewicz.

R.S. Pillar,
Senior Project Design Engineer,
Road Design Division.
Kingston.

W.G. Wigle,
Program Engineer.

February 25, 1966.


W. P. 107-59, Underpass at Fraser Road
Highway 401 - District 9 - Ottawa

This is in reply to your memorandum of February 7th 1966. The Program Division agrees with your recommendations to split the above work project into two separate contracts.

We have now taken the necessary steps to have the first project, which includes grading and drainage on the approaches, etc., programmed for 1966. The second project which includes the granular base, paving and structure, has been programmed for 1967.

In view of Mr. A.G. Stermac's letter to you on February 14th 1966, we have included the estimated cost of placing steel piles, in the second project.

The appropriate schedule of Pre-Engineering will follow in a few days.


W.G. Wigle,
Program Engineer.

WGW:nf

c.c. A.F. Stermac ✓
c.c. L. Walker
c.c. S.J. Markiewicz
c.c. A.P. Watt

Mr. G. Scott,
Regional Bridge Location Engr.,
Bridge Division, Admin. Bldg.

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

March 15, 1966

Your Memo -- Mar. 8/66

W.P. 107-59, Site 31-230,
Fraser Road Underpass,
2.5 Miles West of Jct. Hwy. 2 & 34,
District #9 (Ottawa) Hwy. 401.

With reference to your memo of March 8, 1966,
concerning Preliminary Plan D 5888-P1, Fraser Road Underpass,
we wish to make the following comments:

It has been decided that stage construction will
be used at this site. The first stage will consist of building
the approach embankments together with the berms, while piles
and the structure itself, will be built in the second stage.

By placing the fill one year earlier, some
detrimental effect of the settlements on the piles will thus
be eliminated. It is essential that the entire fill be placed -
i.e., even at the locations of future abutments. The material
placed at these locations will have to be excavated when piles
are to be driven and abutments built.

Two-foot diameter piles battered (1:4) in two
directions, are presently proposed for the support of the
abutments. In view of the large settlements to be expected,
we are not sure whether this solution is the most appropriate
one. We would like to study this matter further and make our
comments at a later stage. A possible change of type or
arrangement of piles will in no way affect the bridge design
and we feel, therefore, that the final decision on the abutment
piles can be deferred.

AGS/Mief

cc: Foundations Office
Gen. Files

A. G. Sternac
A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

MEMORANDUM

To: Mr. A. Stermac,
Principal Foundation Engineer,
Room 107, Lab. Bldg.

From: Bridge Division,
Downsview, Ontario.

Date: March 8, 1966.

Our File Ref.

In Reply To

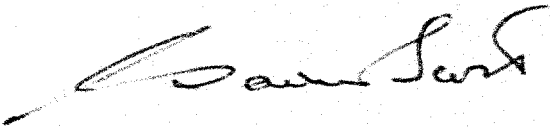
SUBJECT:

W.P. 107-59, Site 31-230,
Fraser Road Underpass,
2.8 miles west of Jct. Hwy. 2 & 34,
District #9, Hwy. 401

We are sending to you herewith one print of preliminary
plan D 5888-P1 for the subject structure.

Would you please let us have your written comments.

GS/ag


G. Scott,
Regional Bridge Location Engineer.

Encl.

Mr. R. S. Pillar,
Sr. Project Design Engineer,
Regional Road Design Office,
Kingston, Ontario.

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

May 5, 1966

Your Memo - April 27/66

W.F. 107-59, Fraser Road Underpass, Hwy. #401

With respect to your memo of April 27, 1966, regarding the above subject, we would like to make the following comments:

Both forward slopes of the approach embankments are shown as 2:1. We would suggest that the upper slope - i.e., the slope of the approach fill above the berm, be 1½:1. This portion of the approach fill will be removed for the placing of the abutment, and it would be beneficial to have as much load there as possible until that time.

Because of the settlements of the fill, consideration has to be given to the compaction of the surcharge. It is believed that possibly between one and two feet of settlement could occur and consequently, part of the surcharge could become part of the approach embankment and should, therefore, be compacted as per standard.

It is our intention to place a number of instruments prior to the placement of the fill, and a three weeks' advance notice would be greatly appreciated.

AGS/MdeP

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office.
Gen. Files.

MEMORANDUM

TO: Mr. T. Stermac,
Principal Foundation Engineer,
Materials and Testing,
Downsview.

FROM: R.S. Pillar,
Sr. Project Design Engineer,
Road Design, Kingston.

DATE: April 27th, 1966.

OUR FILE REF.

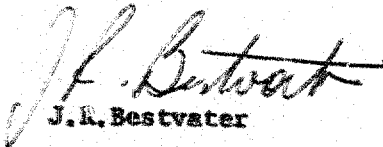
IN REPLY TO

SUBJECT: W.P. 107-59, Fraser Road Underpass, Hwy. #401

Enclosed is a print of a profile No. C-32-17 indicating final grades on which we have shown in red our proposed design for the construction of the berm including the two foot surcharge.

Would you please scrutinize our proposals, and forward your comments in order that they can be incorporated at this stage of the design.

Your immediate concern in this matter is greatly appreciated.


J.R. Bestvater

for:

R.S. Pillar
Sr. Project Design Engineer

JRE/RSP/ss
Encl.

Department of Highways Ontario

Copy for the information of

A.G. STERNAC

L.E. Walker,
District Engineer
Ottawa.

Attention: W. Aitken

*File with report by
H.E. Golden 4/8/66*
R.S. Pillar,
Sr. Project Design Engineer,
Road Design, Kingston.

May 9th, 1966.

W.P. 107-59-1. Fraser Road Underpass, Hwy. #401 - Ottawa District.

Attached is a copy of a memorandum dated April 27th, 1966, from Mr. A.G. Sternac, Principal Foundation Engineer.

I refer you specifically to paragraph 4 where Mr. Sternac has requested 3 weeks advance notice prior to placement of the approach fills. Since this time period will be beyond the design stage, would you please fulfill this request. I might mention that we will incorporate a special provision in the contract advising the contractor that he will be required to safeguard any necessary devices.



R.S. Pillar
Sr. Project Design Engineer

RSP/ss

Att'd.

c.c. to: A.G. Sternac.

MEMORANDUM

*Foundations
Office*

To: Regional Road Design Office -
Kingston.

From: Materials & Testing Division.

Attn: Mr. R. S. Pillar.

DATE: April 5, 1966.

Our File Ref.

IN REPLY TO

SUBJECT: Proposed Fraser Road Underpass,
Hwy. #401, W.P. 107-59.

Spencer to

Stage construction has been proposed and accepted for the above-mentioned structure. The longer the period between approach embankment construction and bridge construction, the more beneficial will be the effects of such a staging. However, because of other reasons it is believed that not more than one year of time is available for staging. During this period a certain amount of settlement will take place. To accommodate for that settlement it is suggested that the fill be built 2 ft. higher. These two feet will also act as a small surcharge. It is believed that the approach fills will settle during the one year interval less than 2 ft. and some of the material will therefore have to be removed prior to the final road grading.

Because of possible failure of stock piles it is recommended that they be built not higher than 18 feet.

It is the intention of this Section to instrument this site prior to fill placement. Settlement plates and piezometers are contemplated. It is hoped that the readings of these instruments will enable a better understanding of the problems connected with the building of approach fills on deposits of sensitive marine clays. We would therefore appreciate to be advised of the time of construction commencement so we could carry out our necessary work well ahead of fill placement.

AGS/tt

cc: Messrs. L. E. Walker
S. McCombie
B. Richardson
S. J. Markiewicz
J. E. Gruspier
Foundations Office,
Gen. Files

A. E. Stermac
A. E. Stermac
PRINCIPAL FOUNDATION ENGINEER

MEMORANDUM

To: Mr. S.J. Markiewicz,
Regional Road Design Engineer,
R.D.O., DOWNSVIEW.

From: M.&T. Division, KINGSTON.

Date: October 13, 1965.

Our File Ref.

IN REPLY TO

SUBJECT: Re: Hwy. 401. W.P. 107-59. Fraser Road Underpass 2.8 Miles West of Lancaster.

Attached is the Soils Section Design report and print of the Soils profile 401J 50-1 for the abovenoted Structure on Hwy. 401 located 2.8 miles west of Lancaster.

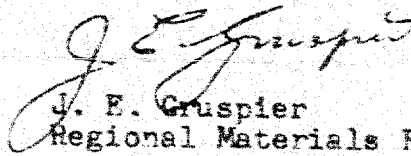
This site is located in a till area where borrow materials of a bouldery nature are likely. It is expected that borrow will be readily available. Some minor grade revisions have been recommended to provide adequate cover over unacceptable subgrade materials.

Granular deposits are generally depleted in the immediate area as are sand deposits. The use of all G.B.C. 'A' is recommended.

A foundation investigation will be required due to the clay subsoil materials encountered. It is possible that the height of the stockpile of granular materials may have to be limited because of the underlying clay but this will have to be determined by the Foundation Section.

Should you have any queries, please contact this office.

JEG:cdr


J. E. Gruspier
Regional Materials Engineer

c.c. D.W. Farren
H.A. Tregaskes
T.C. Muir
W. Wigle
L.E. Walker (2)
S. McCombie
M. Stoyanoff
J.L. Forster
D.A. MacDonald
G.A. Wrong (2)
A.G. Stermac
File

SOILS DESIGN REPORT

Hwy. 401

Fraser Road Underpass
2.8 Miles West of Lancaster

0.57 Miles

V.P. 107-59

Proposed Grading, Granular Base, Paving, & Structure Project

<u>Soils Plan and Profile</u>	<u>Station to Station</u>	<u>Line</u>	<u>Township</u>
401 J 50-1	7+00 to 38+75	'A'	Charlot- tenburgh

GENERAL DATA

This project is located in Charlottenburgh township, approximately 2.8 miles west of Lancaster. It is proposed to pass Fraser Road over Hwy. 401 by means of an underpass to be constructed during the 1966-67 construction program.

DESIGN CRITERIA

A.A.D.T.	- 200
Minimum Vertical Curve	- 500'
Maximum Grade	- 5%
Horizontal Alignment	- 9° Max Curves
Pavement Width	- 20'
Shoulder Width	- 6'
Shoulder Rounding	- 2'

PHYSIOGRAPHY & SOILS DATA

The project is located on the Lancaster Flats Physiographic Region, a lowland area where the till plain has been buried under water-laid deposits leaving exposed only the stony crests of a few drumlins and till ridges. The water-laid materials range from clay to very fine sand.

SOILS INVESTIGATION

A soils investigation was carried out on this project during June of 1965, using a 12" power auger. Borings were

placed to a minimum depth of 4' under the shallow fill sections and to 15' on each side of Hwy. 401 where the high fills are proposed.

The subsoil was found to consist of approximately 2' of moist sandy silt over 4'+ of moist stiff silty clay over 3'+ of moist firm silty clay over wet soft fat clay to depths greater than 15'.

Due to the nature of the subsoil, foundation investigation and stability analysis is required at this site. This has not yet been completed.

EMBANKMENT STABILITY

Throughout this area there have been many other structures built over Hwy. 401 at locations where the foundation soil conditions are similar to those at this site. In some cases berms were required, and in all cases extensive fill settlement adjacent to the structures occurred due to consolidation of the underlying wet clay soil.

At many of the abovementioned sites where extensive fill settlement occurred adjacent to the structures, the concrete gutters are now distorted and useless. It is therefore suggested that gutters not be placed within 50' of the structure at the time of construction.

GRADELINE

To provide an adequate tie-in grade to the county road, the gradeline has been revised from Sta. 7+00 to Sta. 12+00. To provide adequate cover for the unacceptable subgrade soils in the vicinity of Sta. 33+00 and provide a granular lift from Sta. 35+ to Sta. 39+, the gradeline has been revised from Sta. 30+00 to Sta. 41+00.

BORROW MATERIALS

Large quantities of bouldery silty sand till are located on a ridge which crosses the alignment at Sta. 0+00. This is an acceptable earth borrow material and is probably available for use as there is a partially depleted borrow pit located approximately 300' left of Sta. 1+40.

GRANULAR MATERIALS

The crushable gravel sources in the near vicinity of this project were largely depleted during construction of Hwy. 401. However, some crushable gravel still remains in the O'Connor Pit area located approximately 15 miles haul distance from the project. This pit is located 3 miles west of Hwy. 34 and approximately 9 miles north of Lancaster.

RECOMMENDATIONS

1. Type of Granular Contract

Based on the availability of materials, it is recommended that the granular materials on this project consist of G.B.C. Class 'A' only.

2. Depth and Width of Granular Materials

The granular material should be placed full width to a 15" total depth. If the subgrade is constructed of other than the anticipated silty sand till material as indicated under "Borrow Materials" the Materials & Testing Section should be consulted during construction to ascertain whether the 15" granular depth will be adequate over the subgrade as constructed.

3. Structure Backfill

Granular backfill to structures should consist of G.B.C. Class 'A'.

4. Compaction Equipment

It is expected that most of the earth to be compacted on this project will be sandy in nature. For design purposes it is recommended that 80% of the compaction time be allotted for wobble wheel type compaction units and the remainder for sheepsfoot type rollers.

5. Stockpiling Procedures

Due to the soft nature of the clay soils underlying the project site, it is important that any stockpile of granular material required be built on the till ridge in the vicinity of Sta. 0+00.

6. Culvert Types

Due to the nature of the foundation soils any culverts required beneath the proposed fill from Sta. 10+00 to Sta. 35+00 should consist of box type concrete culverts or circular C.I.P. type.

7. Depth and Type of Asphalt Pavement

It is recommended that the pavement consist of the following:

¾" Hot Mix Sand-Asphalt Course

1½" H.L. 3 (Surface Course)

RLS:cdr

Prepared by:

R. L. Smith

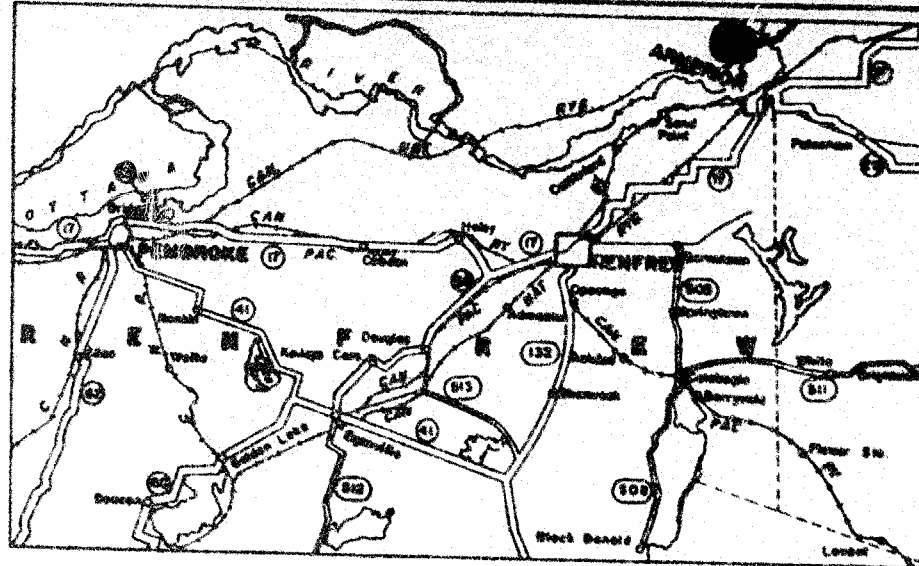
R. L. Smith
Project Soils Supervisor

Reviewed by:

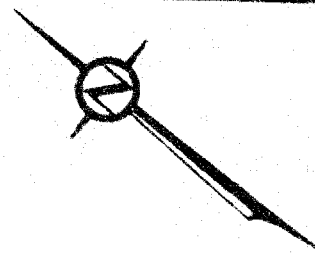
H. A. Meyer

H. A. Meyer
Senior Soils Engineer

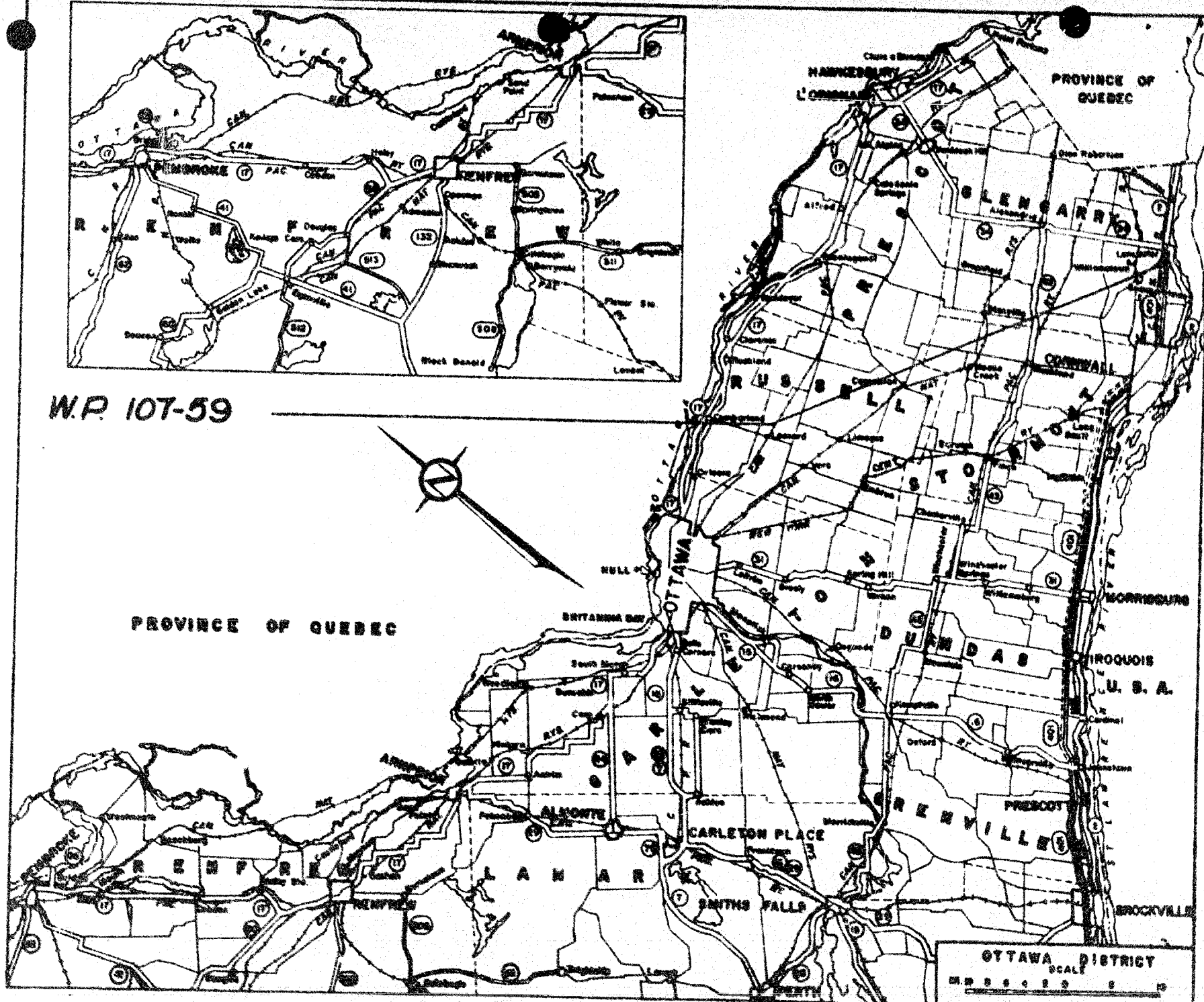
October 13, 1965



W.P. 107-59



PROVINCE OF QUEBEC



Mr. R. S. Pillar,
Sr. Project Design Engineer,
Kingston Regional Office.

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

June 28, 1966

Your Memo -- June 23/66

W.P. 107-59 - Hwy. 401 - Fraser Road Underpass

With respect to your memo of June 23, 1966, regarding the above project, we would like to make the following comment:

The part of the approach embankment where piles are to be driven should not contain any boulders at all. We feel that limiting the size of boulders to 6" maximum could create some serious problems during pile driving.

Since the embankment is going to be built in six-inch layers, the removal of all boulders from a limited area should not represent any problem at all. We would, therefore, strongly suggest that the Special Provisions be altered to incorporate our recommendation.

AGS/MdeF

A. G. Sternac
A. G. Sternac,
PRINCIPAL FOUNDATION ENGINEER

cc: Foundations Office
Gen. Files

MEMORANDUM

To: Mr. A. G. Stermac,
Principal Foundation Engineer,
Lab Building,
DOWNSVIEW, Ontario.

FROM: Mr. R. S. Pillar,
Sr. Project Design Engineer,
KINGSTON, Ontario.

DATE: June 23, 1966

OUR FILE REF.

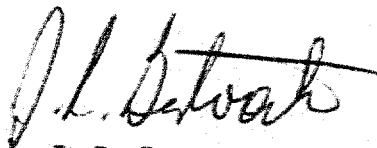
IN REPLY TO:

SUBJECT: Re: W.P. 107-59 - Hwy. #401, Fraser Road Underpass.

Enclosed find one set of our completed Contract Drawings for your information and use.

Special Provisions have been inserted in the Contract as follows:

- (1) Limiting the size of boulders to 6" maximum in the vicinity of the future pile driving areas,
- (2) Providing protection for and allowing access to measuring devices, and
- (3) Limiting the height of granular stockpiles to 18'.



J. R. Bestvater,
For: R. S. Pillar,
SR. PROJECT DESIGN ENGINEER.

Encl.

JRB/RSP/em

SPECIAL PROVISION RE: MEASURING DEVICES

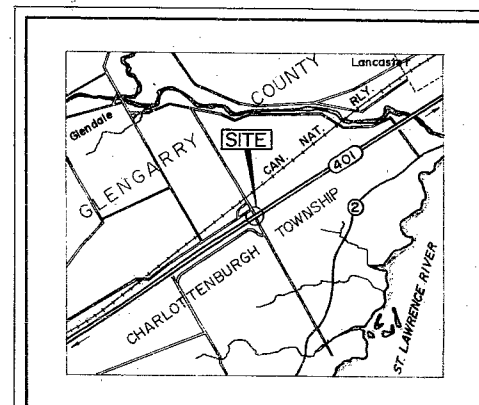
A number of measuring devices (Piezometers, etc.) have been installed by the Department in the proximity of the approach fills.

The Contractor shall:

- A. Provide access to these devices by Department personnel at all times
- B. Provide adequate protection to prevent damage to these devices during his grading operations.

In the event of damage caused to these devices through negligence on the part of the Contractor during operations, all costs arising from repairs or replacement of the devices shall be borne by the Contractor.

#65-F-232
W.P. #107-59
HWY #401
FRASER ROAD



SCALE IN MILES



- | NO. | ELEVATION | STATION | OFFSET |
|-----|-----------|---------|-------------|
| 1 | 160.8 | 26+05 | 19.5' LEFT |
| 2 | 166.1 | 25+69 | 18.5' RIGHT |
| 3 | 162.7 | 24+93 | 19' LEFT |
| 4 | 165.9 | 24+35 | 19.5' RIGHT |
| 5 | 161.2 | 23+93 | 18.5' LEFT |
| 6 | 160.7 | 26+08 | 18.5' RIGHT |
| 7 | 166.0 | 25+65 | 19' LEFT |
| 8 | 161.9 | 24+98 | 19' RIGHT |
| 9 | 165.9 | 24+32 | 19' LEFT |
| 10 | 163.7 | 23+93 | 18.5' RIGHT |
| 11 | 162.1 | 23+00 | 9' LEFT |
| 12 | 160.9 | 27+00 | 9' LINE A |

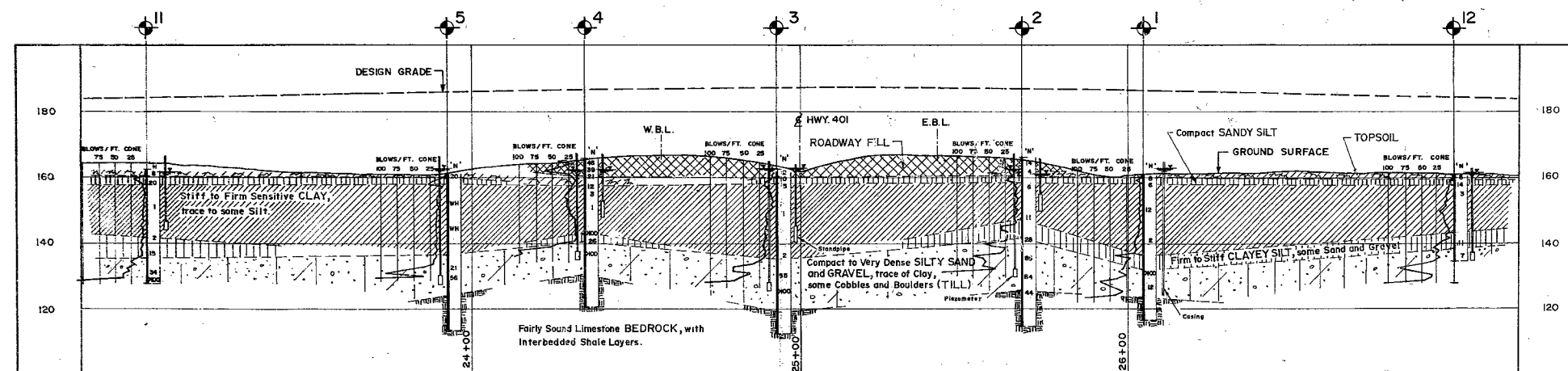
- NOTE -

REVISIONS			
	DATE	BY	DESCRIPTION

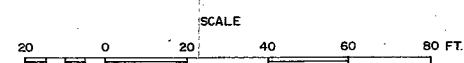
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION - FOUNDATION SECTION

KING'S HIGHWAY NO. 401 DIST. NO. 9
CO. OF GLENGARRY
TWP. OF CHARLOTTENBURGH LOT 2 CON. I

SUBM'D.	CHECKED J.B.D.	W.P. NO. 107 - 59	M.B.T. DRAWING NO.
DRAWN M.W.	CHECKED	JOB NO. 65135	FIGURE 1
DATE DEC. 3, 1965.		SITE NO. 31-230	BRIDGE DRAWING NO.
APPROVED		CONT. NO.	D5888-f



SECTION ALONG PROPOSED REVISION FRASER RD., LINE A

[illegible]