CC: FOUNDATIONS

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section,
(Foundations Office).

August 16, 1961.

FOUNDATION INVESTIGATION REPORT
by: H.Q. Golder & Assoc., Ltd.

Re: Soil Conditions --
Proposed Welland Canal Tunnel,
Welland, Ontario, District #14.

Attention: Mr. S. McCambie.

We have reviewed the report for the above mentioned structure, submitted by the Consultant, H.Q. Golder & Associates.

The report contains all the required factual information which is very well presented, and we believe will be adequate for your further design work. Should there be any additional information that you might require, please feel free to call on our Office.

A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF
Attach.
cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
I. C. Campbell
J. C. Thatcher
T. J. Kovich
J. Roy
E. R. Saint
J. E. Gruspier
F. Norman
Sir Alexander Gibb, Underwood, McLellan, Consulting Engineers.
Foundation Engineering Corporation (FENCO)
Foundations Office
Gen. Files.

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A.G. Stermac,
Principal Foundation Engineer.
Attention: Mr. K. Selby

From: Chemical Section,
Materials & Research Division.

Date: June 6, 1963.

Our File Ref. 11-15-1

In reply to

Subject: Soil Samples

At your request five soil samples were analyzed for their moisture and sulphate contents. The test results are shown together with the ratings of sulphate attack as listed in the "Concrete Manual" (1956, sixth edition, page 12).

Sample No.	% Moisture	% Sulphate (SO_4)*	Rating of Sulphate Attack
61F11-BH8B S#2 30'	15.2	0.1038	positive
61F11-BH8B S#3 40'	15.4	0.2295	considerable
61F11-BH8B S#9-D70'	17.1	0.4000	considerable
61-F11-BH9B S#2 12'	18.9	0.0316	negligible
61-F11-BH9B S#5 42'	24.7	0.2242	considerable

* Calculated on dry sample weight.

R. Sterk,
Chemical Engineer.

RS/hc

cc. Files.

27
12
9.0
324.0
2.0
230
18.40
30
5400

DEPARTMENT OF HIGHWAYS ONTARIO

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R. Sterk,
Chemical Engineer.

RS/hc

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R. Sterk,
Chemical Engineer.

RS/hc

cc. Files.

cc: Foundations Office (Rm. 110)

Mr. W. R. Bennett,
Principal Materials Engr.,
Materials Section.

Attention Mr. P. Smith

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section.

June 10, 1963

Welland Canal Tunnel - District #4
(Chemical Tests on Soil Samples) W.P. 130-61

We are enclosing, for your information, the results of chemical tests to determine sulphate contents, carried out on soil samples from the site of the proposed Welland Canal Tunnel.

The elevations at which the various samples were obtained, are as follows:

B.H. 8B	S #2	El. 555.0
	S #3	El. 545.0
	S #9	El. 515.0
B.H. 9B	S #2	El. 572.0
	S #5	El. 542.0

Ground elevation varies from a high of about 590.0 to a low of about 550.0 (canal bed), and the proposed tunnel will be constructed down to about El. 510 at the lowest point.

It would seem, from the results obtained, that some precautions are necessary, as the stratum appears to contain corrosive chemicals throughout most of its depth.

K. G. Selby

KGS/MdeF
Encl.

cc: Foundations Office
Gen. Files

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

Mr. B. Davis,
Bridge Design Engineer.

P. Smith.
Materials & Research Div.

June 12th, 1963.

130-61

Proposed Tunnel at Welland - W.P. ~~274-62~~

I arranged for some of the core samples taken at the site of the tunnel under the canal at Welland to be analysed to see if there were sulphates in the soil such that attack on the concrete might occur if precautions were not taken. The attached report indicates that this might happen. I think, pending later evidence to the contrary, we should include for the use of sulphate resisting cement in all concrete likely to be in contact with the ground or ground water. I am not sufficiently acquainted with the project to know precisely what this involves, but, if you wish, I would be happy to further discuss the details with you.

P. Smith,
Sr. Materials Engineer
(Concrete).

PS/jf
Att.

c.c. K.Selby, Foundation Engineer.

Please keep this aspect in mind if any more cores are taken or ground water levels explored.

130-61

Geotechnical file

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. A.G. Stermac, Principal Foundation Engineer.

From: Chemical Section, Materials & Testing Division.

Date: January 18, 1967.

Our File Ref. 11-7-5

In reply to

Subject: Sulphate Water Samples

At the request of Mr. M. Devata, Supervising Foundation Engineer, twelve samples of water were tested for their sulphate contents. The tests were made with respect to attack of concrete by sulphate containing waters.

The test results are listed in the Appendix together with the corresponding rating of relative degree of sulphate attack on concrete. These ratings were taken from the "Concrete Manual" (6th edition, 1956, Page 12, Table 2).

The test results showed that all water samples contained sulphate and that their aggressiveness with respect to concrete should be rated as positive, considerable, and severe.

A.C. Suter,
Principal Chemical Engineer.

Per.

R. Sterk,
Chemical Engineer.

RS/c

cc. Mr. M. Devata,
Files.

APPENDIX

SULPHATE CONTENTS OF WATER SAMPLES

PROJECT NO. 66134

Bore Hole No.	T-1	T-1	T-1	T-2	T-3	T-3	T-3
Depth (ft)	Water Level	35	54	18	10	34	35
ppm Sulphate (SO_4)	3095	4715	1865	1041	168.5	244	195.5
Rating	severe	severe	consider-able	consid-erable	positive	positive	positive

Bore Hole No.	T-7A	T-7A	T-7A	T-8	T-8
Depth (ft)	10	14	22	31	40
ppm Sulphate (SO_3)	640	565	562	202	208
Rating	positive	positive	positive	positive	positive

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
V. MILLIGAN

2446A BLOOR ST. W.
TORONTO 9
RO. 7-9201

REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

SOIL CONDITIONS

PROPOSED WELLAND CANAL TUNNEL

WELLAND, ONTARIO.

Distribution;

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

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

July, 1961

6108

INDEX

ABSTRACT

The results of the review of a detailed site investigation, the purpose of which was to determine the detailed soil conditions at the site of a proposed tunnel in Welland, Ontario are reported. It was found that following a minor thickness of topsoil, the site is underlain by about 85 to 100 feet of very stiff to firm reddish-brown silty clay, followed by a thin layer of glacial till and then dolomite bedrock.

The upper portion of the silty clay deposit is heavily desiccated and undrained shear strengths of up to 2000 pounds per square foot were measured; below this the shear strengths are relatively constant with depth with an average value of about 1000 pounds per square foot. This average value of shear strength at lower depths is somewhat higher than that measured in the previous investigation. The higher strength value obtained is attributed to detailed sampling techniques and the use of vane shear apparatus which was carefully checked to correlate with the remoulded shear strength determined in the laboratory. The lower portion of the silty clay is stratified and contains layers of dense silt up to 4 inches in thickness.

Ground water levels established in piezometers located on both sides of the Welland Canal gave a fairly constant piezometric elevation of between elevation 570 to 580.

INTRODUCTION

H. Q. Golder & Associates Limited has been retained by the Department of Highways, Ontario, to prepare a report on the soil conditions at the site of the proposed Welland Canal Tunnel in Welland, Ontario. This report discusses the soil conditions in detail and is a supplement to various interim reports and recommendations given to the Foundation of Canada Engineering Corporation Limited, concerning soil mechanics problems in the design of the proposed tunnel and approaches.

The results of the soil investigation and laboratory testing as they became available were discussed with personnel of the Department of Highways, Ontario and of The Foundation of Canada Engineering Corporation Limited. In particular, various discussions were held with Mr. P. J. Thompson of The Foundation of Canada Engineering Corporation Limited concerning soil mechanics problems relating to the design of the proposed tunnel and approach cuts. These discussions are summarized in Appendix II to this report. It will be noted that the effective shear strength parameters estimated for the clay in the Appendix are in error in view of the measured shear strength parameters now available. However, the computations are in essence similar when we consider that for these notes in Appendix II a value of ϕ' equal to 20° and a value of cohesion intercept, c', equal to 350 pounds per square foot was assumed, whereas the present laboratory testing indicates that ϕ' is now 25° and c' is zero.

Essentially the reduction of c' from 350 pounds per square foot to zero reduces the factor of safety somewhat but this is counter-balanced to a degree by the increase in the estimated value of Ø from 20° to 25°. The notes in Appendix II will serve to complete the scope of our work on this project.

PROCEDURE

A site investigation was carried out by the Department of Highways, Ontario, and supervised by Mr. K. Selby of their staff. Throughout the course of the investigation periodic discussions were held with Mr. Selby to review the progress and the data obtained from the borings. Recommendations were also made covering the sampling methods to be adopted, the spacing of borings, the installation of piezometers, in situ permeability tests and the like.

The investigation was carried out during the period February 23rd to March 27th, 1961. The borings and in situ tests were so spaced as to supplement information obtained from a preliminary investigation carried out by H. Q. Golder & Associates Ltd., in October, 1960.

A total of 15 boreholes was put down, these being boreholes 4 to 18 inclusive, 3 boreholes having been put down in a previous investigation. At the location of borehole 4, 3 holes were actually drilled in close proximity, one hole being used for vane testing and the other two for detailed undisturbed sampling. At the locations of boreholes 5, 8 and 9, two holes were put down in close proximity, one hole being used for vane testing and the other also for detailed sampling. Boreholes 6, 7, 10, 11, 12, 13, 14, 16 and 18 were drilled by a mobile power auger down to the upper contact of the dense till stratum which underlies the clay overburden at the site. This contact was readily observed during drilling operations and was confirmed where necessary by a sample taken at this depth. Examination was also made of the auger tips after completion of each hole in order to ensure that the till stratum had been penetrated. Borehole 15 was put down in order to carry out in situ permeability tests in the overburden at various elevations. In borehole 9, an in situ falling head permeability test was carried out in the dense till stratum.

A total of 9 piezometers was installed. These piezometers are numbered P1A to P9A inclusive. Piezometers P4A and P9A were established in bedrock, the others at various elevations in the overburden. Bedrock was cored in AXT size in boreholes 4, 5, 8 and 9.

The locations of the boreholes are shown on Figure 1 together with sections showing the inferred soil stratigraphy and ground surface profiles. Detailed logs of each borehole are given on the Records of Boreholes.

The samples obtained during the investigation were brought to the laboratory of the Department of Highways, Ontario, for testing. The testing program was discussed and various recommendations were made by us. These recommendations were approved by Mr. L. G. Soderman, D.H.O., and the testing program completed in the D.H.O. laboratories. The results of this testing are shown on the Records of Boreholes and on the figures. Representative samples from each of the soil strata defined by the investigation are presently being stored in the laboratories of the Department of Highways, Ontario.

All elevations in the report are referred to Geodetic datum and were established by survey crews of the Foundation of Canada Engineering Corporation Limited.

SITE TOPOGRAPHY AND GEOLOGY

The site of the proposed Welland Canal Tunnel is located in the physiographic region known as the Haldimand clay plain which is in this area relatively flat. The area is known to be underlain by a thickness, approximately 120 feet, of stratified silt and clay deposits of glacial Lake Warren. The silt and clay are underlain by a thin layer of glacial drift followed by shales and dolomites of the Paleozoic era.

SOIL CONDITIONS

General

Ground surface at the site is almost horizontal. The principal soil stratum at the site is a very stiff to firm red brown silty clay containing thin layers of of red and grey clay and occasional small gravel particles up to 1 inch in size. Occasional pockets or small lenses of silt were also encountered. The upper portion of this clay is highly desiccated. The approximate thickness of the desiccated crust is generally about 15 to 17 feet. The average thickness of the clay is about 78 feet. The desiccated crust of the clay was encountered immediately below a thin layer of topsoil in all the boreholes.

In the majority of samples obtained some faint indications of stratification were apparent, implying that the individual layers are of similar composition. The details of stratification noticed in the samples are presented in Appendix I which is a description of all samples obtained in the investigation. This detailed description was made by Mr. K. Selby. The fact that the layers or stratifications noted in samples are of similar composition is evidenced by Figure 2 which shows a plot of Plasticity Index measured for various layers against Liquid Limit. The general range of this figure is within the normal range for glacial lake clays. However, at a depth of about 75 to 79 feet, in the lower half of the stratum, zones of definitely layered material were encountered. In addition to the brown to reddish silty clay which forms the bulk of the stratum, these layers which range from 1/16 to 4 inches in thickness are composed of red or grey silty clay with occasional seams of silty fine sand and relatively pure silt. In local areas the silt portion of the stratified deposit predominates.

Figures 3 and 4 show the grain size distribution for various layers in the deposit. In Figure 4 it may be noted that the grading for samples in borehole 5 at an elevation of 509 to 507 indicates a pure silt. This is considered to be typical of the grading of the silt layers at this depth. Figure 5 shows the grading for a typical portion of the reddish brown clay with a liquid limit of about 36 and a plastic limit at about 20.

From the number of Atterberg limits carried out on samples from the stratum the liquid limits may be taken to be within the range of from 22 to 60, with an average of 36 and the plasticity indices to range from about 3 to 33, with an average of about 16. The higher liquid limits of the order of 50 to 60 with the plasticity indices of the order of 30 or above are generally representative of the red and grey layers discussed above. It may be noted that the highest plasticity indices were measured in the lower portion of the stratum. The liquid limits of the order of 30 to 40 are considered representative of the brown silty clay. The limits from 40 to 50 are indicative of thinly layered samples where it was not possible to separate individual layers.

The water content ranged between 9 and 46 percent with an average of 30 percent. The wet unit weight ranged between 101 to 130 pounds per cubic foot with an average of 121 pounds per cubic foot. These values are sensibly in agreement with values measured in the previous investigation and for similar clay deposits in the local area. They are typical of glacial lake clay.

The Atterberg limit results are plotted together with the natural water contents on the records of boreholes. It may be seen that generally in the upper 20 to 30 feet of the stratum the natural water contents are very close to the plastic limit, and below these depths there is some trend of increase in moisture content with increasing depth. This has been illustrated graphically by a plot of liquidity index versus elevation for borehole 1 in our previous report 6022 in December, 1960.

The activity of the material is generally about 0.5; however, certain anomalies appear in comparing the plasticity index obtained for specific samples with their grain size distribution. In some cases particularly where the grain size distribution curves are obtained for relatively highly plastic materials it is considered that the high silt percentage shown on the curves may be due to some flocculation during testing.

Dispersion of the particles in these samples would be quite difficult due to the probable presence of iron oxides as cementing agents. It is considered therefore, that the material composing the various layers of this silty clay deposit is generally inactive.

The red brown silty clay and lower layered silty clay are underlain at a depth of about 90 to 95 feet by a thin stratum of clay, silt, sand and gravel which has been described as a glacial till. This stratum contains occasional boulders up to about 1 foot in diameter. The maximum thickness of the stratum recorded is about 11 feet. The consistency of the till, based on the measured shear strength in a previous investigation and the difficulty encountered in penetrating the stratum, is estimated to be very stiff to hard.

The till is underlain by a dark grey dolomite of the Guelph formation.

Compressibility Of The Clay

To study the compressibility of the clay deposits and thus to determine the possible movements of the tunnel units as a result of swelling of the clay on relief of pressure due to the weight of overburden on excavation and movement due to rebound consolidation of the clay when the final tunnel load is applied, a number of consolidation tests were carried out. The results of these consolidation tests are plotted on Figures 6 to 14 inclusive. These consolidation tests show that in general the clay is probably over-consolidated by about 2,000 pounds per square foot in excess of overburden pressure throughout its full depth save in the upper desiccated portion of the clay where the degree of preconsolidation induced by desiccation could be much larger.

From the general shape of the pressure-void ratio curves it is apparent that the samples exhibit some degree of disturbance. It is therefore not possible to compute exact values for the compression index, C_c. For computation purposes it is considered best to use the actual plot of void ratio versus the logarithm of pressure as shown on the figures.

To simulate the possible movements due to swelling and rebound the consolidation tests were subjected to various load cycles. In general the cycles were commenced from overburden pressure as computed from the results of the boreholes. The pressure was then reduced to about 0.1 tons per square foot and the sample reloaded to a pressure slightly in excess of existing overburden pressure. The resulting rebound curve is generally quite uniform in shape, the average value of the recompression index C_{cr} is estimated to be about .05 to .08. This value may be used in design to estimate movement due to swelling and rebound as the values of the swelling index C_{cs} and the values for C_{cr} will be essentially similar.

Computations of swelling and rebound have been presented to Foundation of Canada Engineering Corporation Limited and are summarized in Appendix II at the rear of the report.

Further evidence that the clay is slightly overconsolidated is given by the pattern of water content and corresponding liquidity index versus depth.

The modulus of elasticity of the clay as determined from the results of undrained shear triaxial tests in the laboratory was found to range between 140 tons per square foot and 200 tons per square foot. These values are determined from typical stress strain curves which are plotted on Figure 15. The value used in the computations to estimate the elastic rebound or movement of the silty clay under load was 140 tons per square foot

Shear Strength Of The Clay

The shear strength of the clay as measured in the previous investigation ranged between 6,000 pounds per square foot in the desiccated portion above elevation 550 to about 750 pounds per square foot below elevation

550. There was no appreciable increase in shear strength with depth.* Several undrained shear strength measurements lower than 750 pounds per square foot were recorded. It was considered that these samples exhibited disturbance in sampling with a consequent reduction in the measured laboratory shear strength.

In this investigation great attention was paid to sampling procedures in order to obtain as undisturbed samples as possible. Consequently, samples from depths in excess of 50 feet were obtained in auger boreholes which were carefully augered to this depth, sampling then being carried out continuously below this depth using thin walled Shelby tube samplers of low area-ratio.

The shear strength of the clay was also measured in situ using vane shear apparatus. It is felt that the shear strength measured by the vane apparatus might be influenced by the presence of gravel particles within the silty clay deposits. To carry out a vane test individual boreholes were put down where vane testing only was carried out.

It was found that of a total of thirty-three quick triaxial undrained shear strength tests carried out in the laboratory the highest value recorded was 1940 pounds per square foot and the lowest was 300 pounds per square foot. The higher values were invariably measured in the upper portion of the stratum where some desiccation had taken place. The average shear strength measured in the laboratory was about 1100 pounds per square foot. In comparison the results of in situ vane shear tests, of which thirty-nine were carried out, gave a high value for the shear strength of 2000 pounds per square foot and a low value of 920 pounds per square foot with an average of about 1400 pounds per square foot. It may be observed from the pattern of shear strength with depth plotted on the borehole records that there is in some boreholes, a considerable difference between the in situ vane shear strength and the shear strength as measured in the laboratory.

To check the validity of the vane shear measurements remoulded vane tests were carried out in the field and the results of these were compared with remoulded triaxial tests. It may be noted that the highest value of remoulded shear strength obtained on laboratory samples was 900 pounds per square foot and the lowest 300 pounds per square foot with an average of about 500 pounds per square foot. In comparison the highest value of remoulded vane shear strength was 960 pounds per square foot and the lowest 200 pounds per square foot with an average value of about 575 pounds per square foot. Thus, there is fairly good agreement between the remoulded shear strength as measured by the vane in the field and the triaxial tests in the labotatory. Consequently, if we calibrate vane shear strength as being a function of the remoulded shear strength as measured in the field or as measured in the laboratory, it may be inferred that the calibration of the vane is satisfactory. Therefore it may be concluded that the strength as measured by the vane apparatus is more indicative of the in situ shear strength of the material than triaxial tests on the soil samples obtained. This may be corroborated by the fact that the low shear strengths measured in the laboratory were generally at a very high strain in the sample. A plot of measured laboratory shear strength, C, versus the strain at failure in percent is shown on Figure 16. It may be seen that at strains to failure above 8 percent the shear strength is invariably quite low while for strains to failure less than 8 percent the shear strengths are in good agreement with the values as measured by the vane apparatus.

Based on the above it is inferred that the undrained shear strength throughout the clay stratum is generally uniform and has an average value of about 1000 pounds per square foot throughout its full depth; in the desiccated crust however, this value is quite conservative and the in situ undrained shear strength could be as high as 2000 pounds per square foot or higher.

In order to examine the long term stability of approach cuts it is necessary to determine the effective shear strength parameters of the silty clay stratum. It is also necessary to estimate the pore water conditions both during and following construction of the approach cuts. To determine the effective shear strength characteristics of the silty clay a series of undrained triaxial compression tests with pore pressure measurements were carried out. The results of these tests are plotted on Figures 17 to 20 inclusive. The results obtained indicate that the effective angle of shearing resistance, ϕ', for the silty clay ranges between 21° and 30° with an average value of about 25°. The tests in general show that the value of effective cohesion, c', is approximately zero. Only one series of tests gave a definite cohesion intercept, this being approximately 100 pounds per square foot.

The above results are in general agreement with similar results on clays in the Niagara area. In this area where overconsolidation is quite low the value of the cohesion intercept above zero is quite doubtful and for the purposes of design it should be considered to be equal to zero. The value of the effective angle of shearing resistance, ϕ', in general is approximately 25° and we consider that this value may be used in the analysis of stability of the approach cuts.

GROUND WATER CONDITIONS

During the initial course of the site investigation it had been thought that the lower stratified portion of the silty clay stratum was under some artesian head. It was also considered that the silt layers which are up to 4 inches in thickness were loose. However, further detailed sampling and boring indicated that the silt only became loose when subjected to an unbalanced hydrostatic head in the borehole and when boreholes were completely filled with water prior to sampling the silt or stratified deposit was found to be dense in sampling. The undrained shear strength of such samples measured in the laboratory was high.

To establish the groundwater conditions in the silt layers and stratified clay deposit, various piezometers were installed, nine in total being put down at elevations ranging from 534 to 478. A plot of the measured water level in the piezometric observation tubes versus elevation of the piezometer tip is given on Figure 21. These elevations of water level were measured over a period of two months, checked at successive intervals thereafter. It may be seen that within the limits of accuracy of the water level measurements the piezometric level across the site is sensibly constant between about elevation 570 and 580. For design purposes a piezometric level of 580 may be used in computations.

REGISTERED PROFESSIONAL ENGINEER
V. MILLIGAN
PROVINCE OF ONTARIO

V. Milligan, P. Eng.

VM/jb
6108
July, 1961.

LIST OF STANDARD ABBREVIATIONS

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

SAMPLE TYPES

A.S. – Auger Sample
C.S. – Chunk Sample
D.O. – Drive Open
D.S. – Denison Type Sample
F.S. – Foil Sample
R.C. – Rock Core
S.T. – Slotted Tube
T.O. – Thin-walled, Open
T.P. – Thin-walled, Piston
W.S. – Wash Sample

PENETRATION RESISTANCES

Dynamic Penetration Resistance – The energy required to drive a 2 inch diameter, 60 degree cone attached to the end of the drilling rods into the ground: expressed in blows per foot, where each blow represents 4,200 inch-pounds of energy.

Standard Penetration Resistance, N – The number of blows by a 140 pound hammer dropped 30 inches required to drive a 2 inch drive open sampler one foot into the ground.

Sampler advanced by static weight – weight, hammer – Wh
Sampler advanced by pressure – pressure,hydraulic – Ph
Sampler advanced by pressure – pressure,manual – Pm

SOIL DESCRIPTION

The standard terminology for the descriptions of the relative density of cohesionless soils and the consistency of cohesive soils is as follows:

Relative Density	N, Blows/ft.	Consistency	c, lb/sq. ft.
Very Loose	0 to 4	Very Soft	Less than 250
Loose	4 to 10	Soft	250 to 500
Compact	10 to 30	Firm	500 to 1,000
Dense	30 to 50	Stiff	1,000 to 2,000
Very Dense	over 50	Very Stiff	2,000 to 4,000
		Hard	over 4,000

SOIL TESTS

C – Consolidation Test
H – Hydrometer Analysis
M – Sieve Analysis
MH – Combined Analysis, Sieve and Hydrometer
Q – Undrained Triaxial
Qc – Consolidated Undrained Triaxial
S – Drained Triaxial
U – Unconfined Compression
V – Field Vane Test

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

SOIL PROPERTIES

γ – Total Unit Weight
γ_d – Dry Unit Weight
γ_b – Submerged Unit Weight
L_L – Liquid Limit
P_L – Plastic Limit
W – Natural Water Content
G – Specific Gravity
e – Void Ratio
K – Coefficient of Permeability
c – Undrained Shear Strength (½ Compressive Strength)
St – Sensitivity
Ø' – Effective Angle of Shearing Resistance
c' – Effective Cohesion Intercept
Cc – Compression Index
Cv – Coefficient of Consolidation

PROJECT No. 6108

RECORD OF BOREHOLE

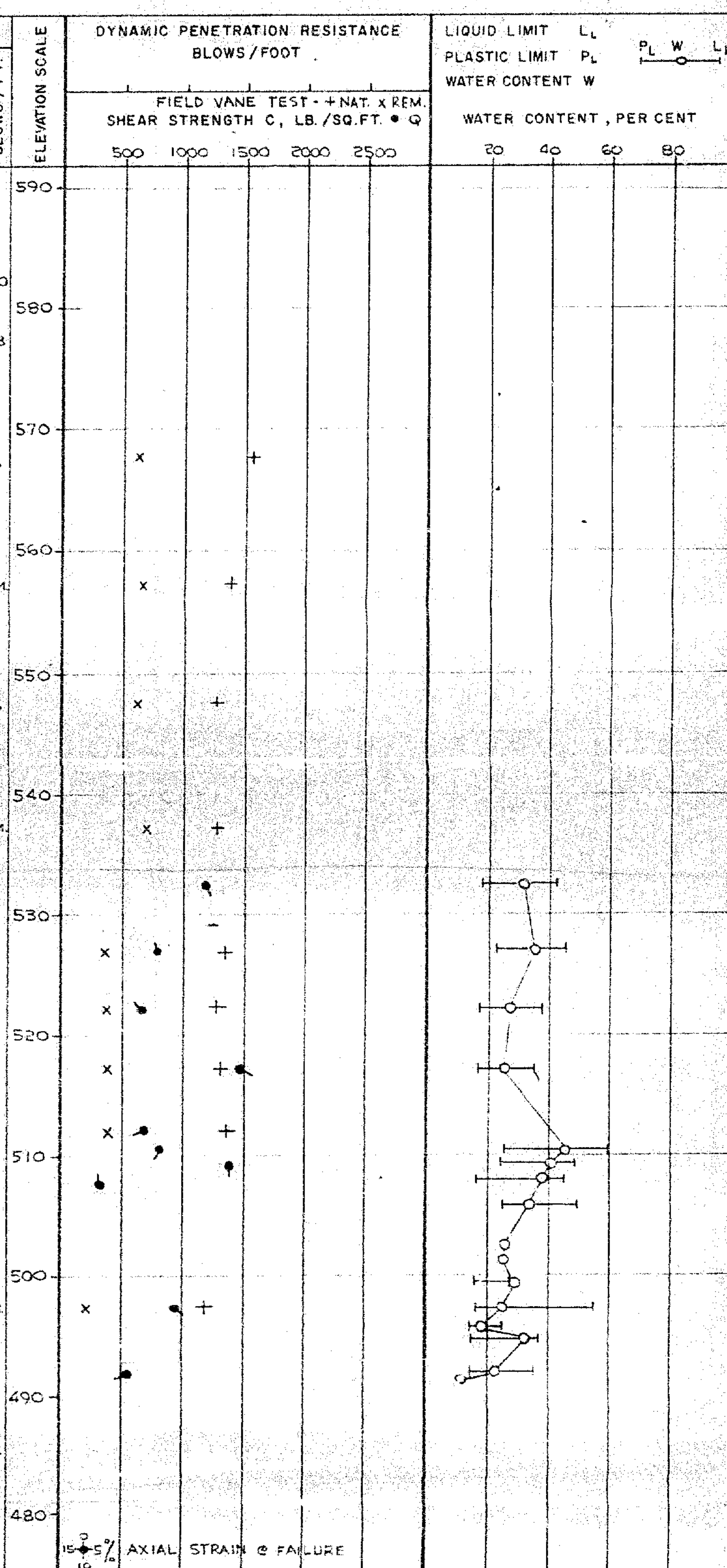
LOCATION SEE FIG. 1 BORING DATE FEB. 24-27, 1961 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER 3"

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

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION
587.0	GROUND LEVEL
586.0 / 1.0	TOPSOIL
	VERY STIFF TO FIRM RED-BROWN **SILTY CLAY** WITH STONES UP TO 1" DIAMETER AND CONTAINING MANY SMALL SILT POCKETS OCCASIONAL LAYERS OF GREY CLAY ⅛" TO ¼" THICK AND RED CLAY ⅛" TO 2" THICK. (UPPER 16' DESICCATED)
508.0 / 79.0	FIRM TO STIFF RED-BROWN **SILTY CLAY** CONTAINING ⅛" TO 4" LAYERS OF BROWN SILT, SPACED 1" TO 6" APART. ALSO FREQUENT LAYERS OF GREY CLAY ⅛" TO ½" THICK AND RED CLAY ⅛" TO 3" THICK.
491.7 / 95.3	VERY STIFF **SILTY CLAY** WITH FINE TO COARSE SAND AND GRAVEL. (TILL)
487.5 / 99.5	LIMESTONE DOLOMITE **BEDROCK**
482.5 / 104.5	END OF HOLE

SAMPLE NUMBER	TYPE	BLOWS / FT.
1	2" D.O.	20
2	"	38
3	"	8
4	2" T.O.	P.M.
5	D.O.	16
6	T.O.	P.M.
7	"	"
8	"	"
9	"	"
10	"	"
11	"	"
12	"	"
13	"	"
14	"	"
15	"	"
16	"	"
17	"	"
18	D.O.	11
19	"	9
20	T.O.	P.M.
21	"	"
22	"	"
23	"	"
24	AXT R.C.	—



REMARKS

PIEZOMETRIC W.L. @ EL. 574.0 - APRIL 4, 1961.

γ = 130 P.C.F.

γ = 122 P.C.F.

γ = 118 P.C.F.

C
γ = 121 P.C.F.
MH

γ = 127 P.C.F.
MH

MH
C
γ = 111 P.C.F. MH
γ = 111 P.C.F. MH
γ = 119 P.C.F. MH
C

MH

MH
γ = 128 P.C.F.

γ = 125 P.C.F.

VERTICAL SCALE
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED [illegible]

PROJECT No.

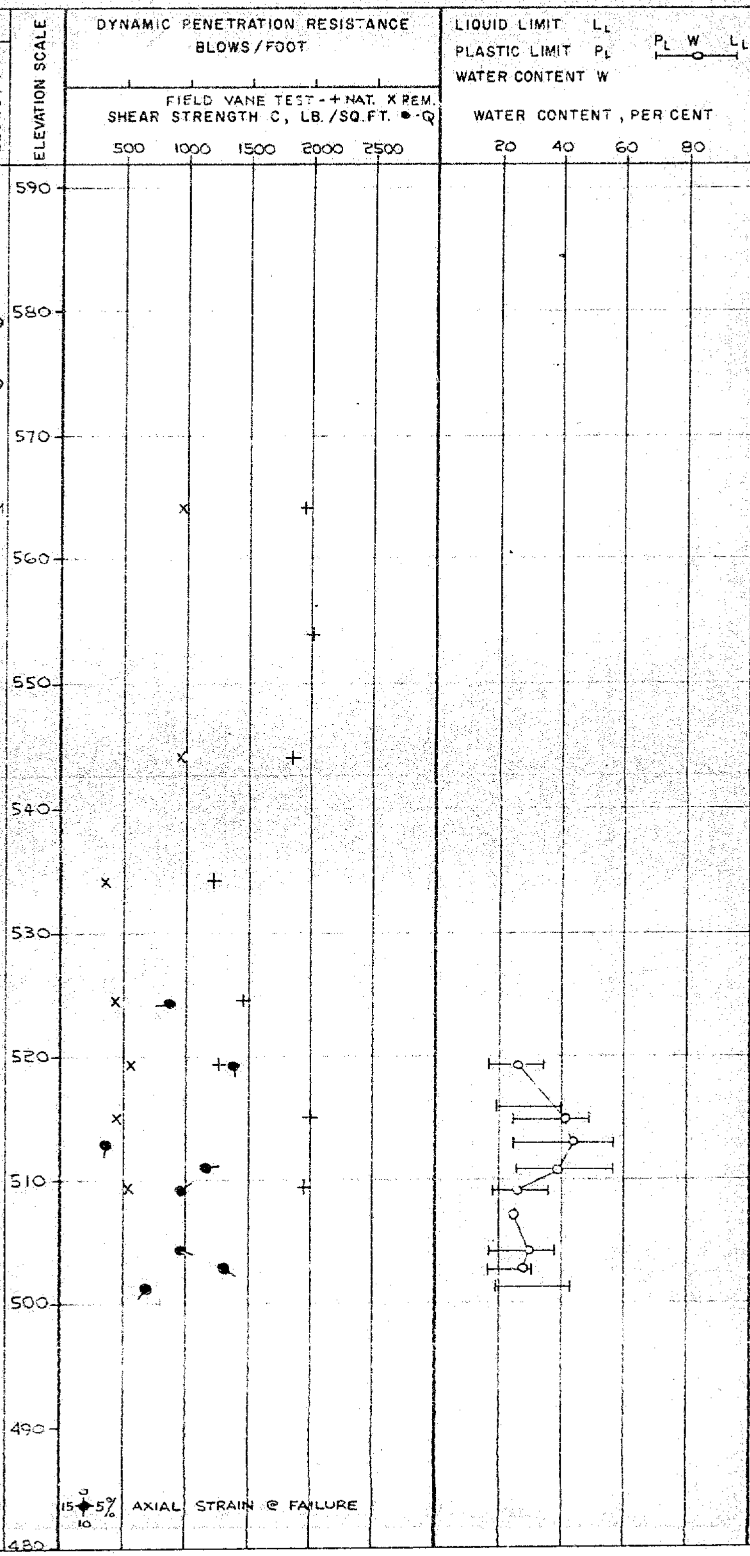
RECORD OF BOREHOLE 5

LOCATION SEE FIG. 1 BORING DATE MARCH 6-8, 1961 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER 3"

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

ELEVN. / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	BLOWS / FT.	REMARKS
585.0	GROUND LEVEL				
584.0 / 1.0	TOPSOIL				PIEZOMETRIC W.L. @ EL. 575.3 - APRIL 4, 1961.
	VERY STIFF TO FIRM RED-BROWN **SILTY CLAY** WITH SOME STONES UP TO 1" DIAMETER AND CONTAINING FREQUENT VERY SMALL SILT POCKETS, OCCASIONAL LAYERS OF GREY CLAY ¼" TO ½" THICK AND RED CLAY ⅛" TO 2" THICK. (UPPER 16'-0" DESICCATED)	1	2" D.O.	39	
		2	"	29	
		3	2" T.O.	PM	
		4	"	"	
		5	"	"	
		6	"	"	
		7	"	"	γ= 120 P.C.F.
		8	"	"	C, γ= 126 P.C.F., MH
		9	"	"	
		10	"	"	MH, C
		11	"	"	MH
		12	"	"	γ= 112 P.C.F., C
		13	"	"	γ= 118 P.C.F. MH
510.0 / 75.0	FIRM TO STIFF RED-BROWN **SILTY CLAY** CONTAINING UP TO 4" THICK SILT LAYERS SPACED 1' TO 6' APART, ALSO LAYERS OF RED AND GREY CLAY ⅛" TO 2" THICK, OCCASIONAL STONES	14	"	"	γ= 124 P.C.F. MH
		15	"	"	MH
		16	"	"	γ= 125 P.C.F.
		17	"	"	γ= 127 P.C.F.
		18	"	"	
500.5 / 84.5	VERY STIFF **CLAYEY SILT** CONTAINING FINE TO COARSE SAND AND GRAVEL **(TILL)**	19	2" D.O.	25	
489.3 / 95.7	LIMESTONE DOLOMITE **BEDROCK**	20	AXT R.C.	—	
485.4 / 99.6	END OF HOLE				



VERTICAL SCALE 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED Y.M.

RECORD OF BOREHOLE 8

LOCATION: SEE FIG. 1 — BORING DATE: MARCH 13-20, 1961 — DATUM: GEODETIC

BOREHOLE TYPE: WASH BORING — BOREHOLE DIAMETER: 3"

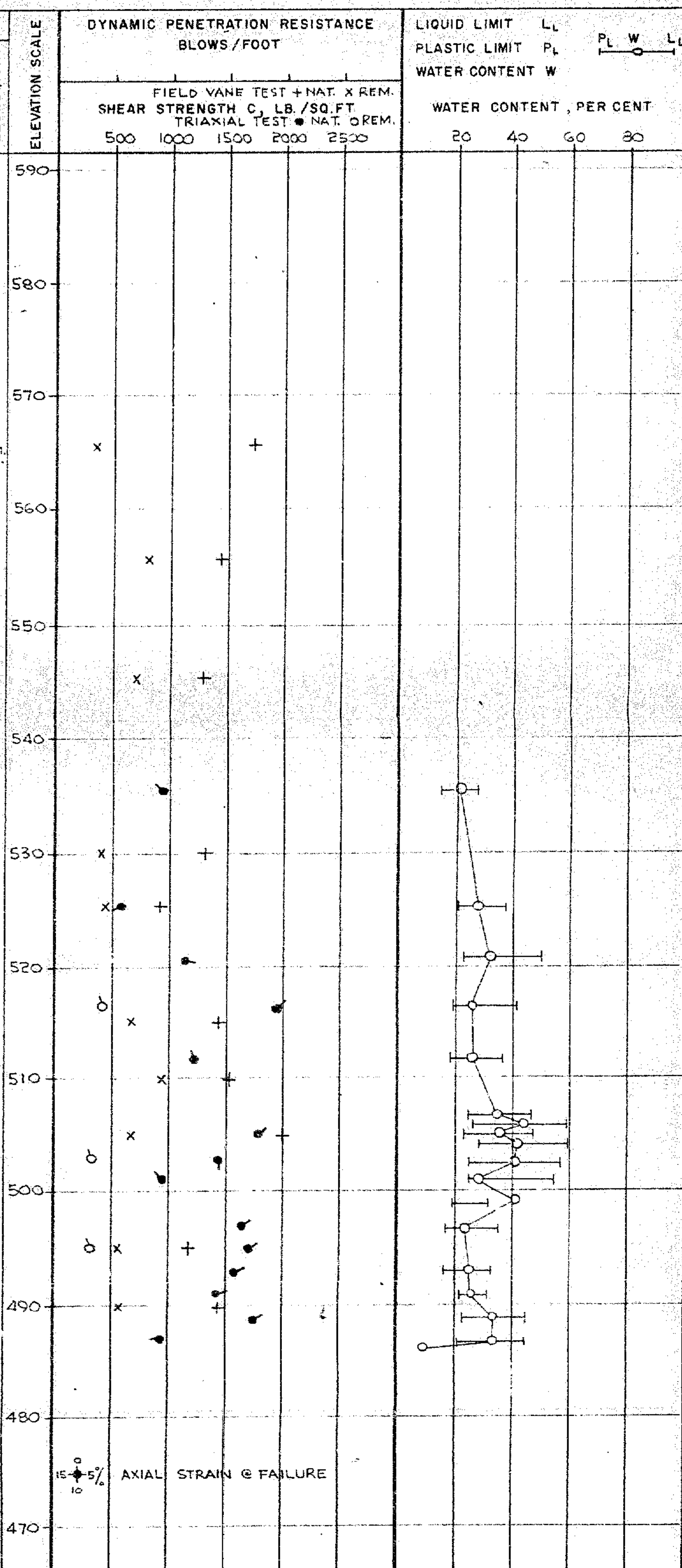
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES — PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES

SOIL PROFILE

ELEVN. / DEPTH	DESCRIPTION
585.0	GROUND LEVEL
584.0 / 1.0	TOPSOIL
506.0 / 79.0	VERY STIFF TO STIFF RED-BROWN **SILTY CLAY** CONTAINING OCCASIONAL THIN LAYERS OF RED CLAY AND GREY CLAY, ALSO OCCASIONAL SMALL STONES UP TO 1" DIAMETER AND FREQUENT VERY SMALL POCKETS OF SILT. (UPPER 16'-0" DESICCATED)
496.0 / 89.0	BROWN SILT LAYERS UP TO 4" THICK SPACED 1" TO 6" APART PARTED BY ½" TO 2" LAYERS OF RED AND GREY CLAY, ALSO ½" TO 6" WITH LAYERS OF STIFF RED-BROWN CLAY
486.5 / 98.5	MEDIUM STIFF RED-BROWN **SILTY CLAY** CONTAINING LAYERS OF RED CLAY AND GREY CLAY ½" TO 2" THICK ALSO OCCASIONAL SMALL STONES UP TO 1" DIAMETER
483.5 / 101.5	HARD CLAYEY SILT SAND & GRAVEL (TILL)
478.5 / 106.5	LIMESTONE DOLOMITE **BEDROCK**
	END OF HOLE

SAMPLES

NUMBER	TYPE	BLOWS / FT.
1	2" T.O.	P.M.
2	"	"
3	"	"
4	"	"
5	"	"
6	"	"
7	"	"
8	"	"
9	"	"
10	"	"
11	"	"
12	"	"
13	"	"
14	"	"
15	"	"
16	"	"
17	"	"
18	"	"
19	"	"
20	"	"
21	"	"
22	"	"
23	D.O.	91
24	AXT R.C.	—



REMARKS

PIEZOMETRIC W.L. @ EL. 571.2 - APRIL 4, 1961.

γ=128 P.C.F.
γ=111 P.C.F.
γ=116 P.C.F.
γ=125 P.C.F.
C
γ=127 P.C.F. MH
C
γ=114 P.C.F.
γ=117 P.C.F.
γ=123 P.C.F.
γ=123 P.C.F.
γ=123 P.C.F.
γ=127 P.C.F.
γ=127 P.C.F.
γ=127 P.C.F. C
γ=116 P.C.F.
γ=121 P.C.F.

VERTICAL SCALE 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED T.H.

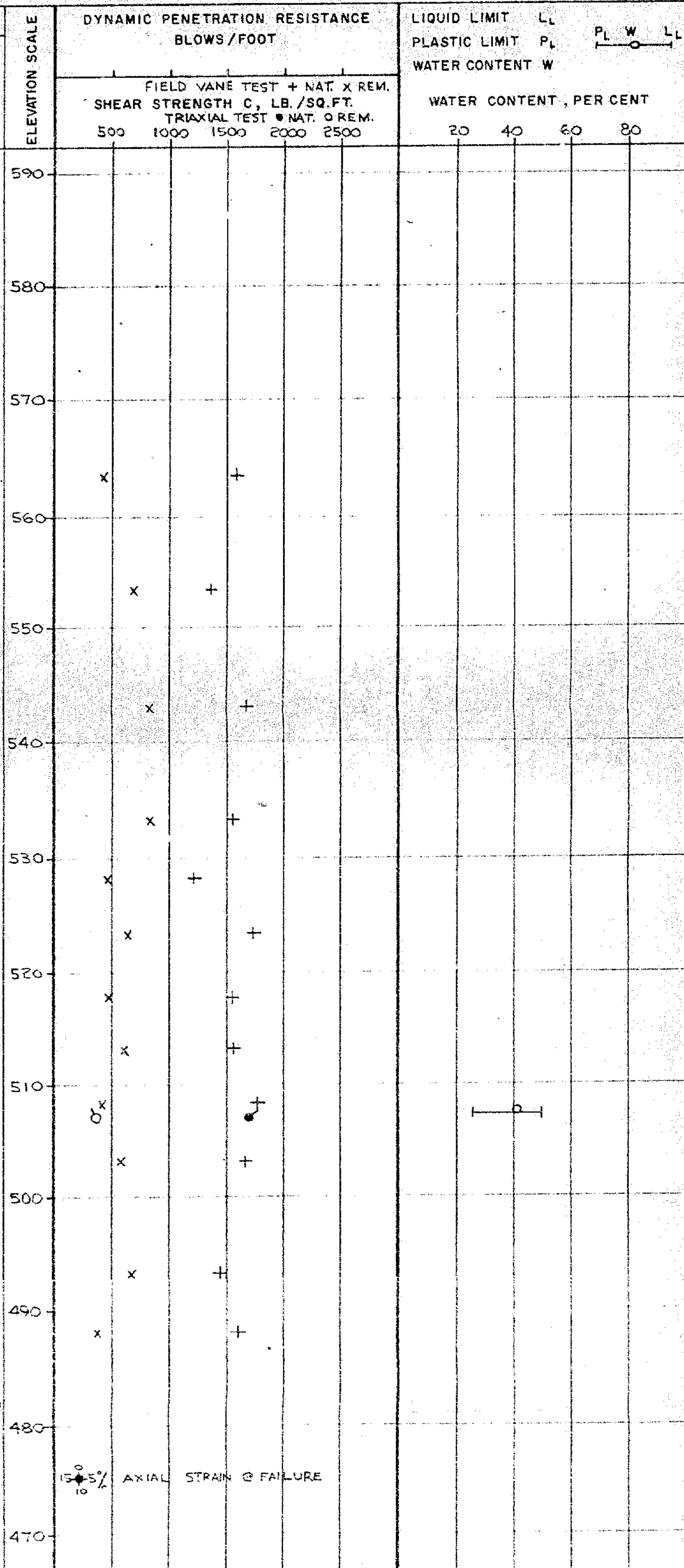
RECORD OF BOREHOLE 9

LOCATION SEE FIG. 1 BORING DATE MARCH 16-17, 1961 DATUM GEODETIC

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER 3"

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

ELEVN. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FT.	REMARKS
584.0	GROUND LEVEL				
583.0 / 1.0	TOPSOIL				
	VERY STIFF TO STIFF RED-BROWN SILTY CLAY, OCCASIONAL LAYERS OF RED AND GREY CLAY ⅛" TO 2" THICK, ALSO OCCASIONAL SMALL STONES UP TO 1" DIAMETER AND NUMEROUS VERY SMALL SILT POCKETS (UPPER 16'-0" DESICCATED)	1	2" D.O.	13	PIEZOMETRIC W.L. @ EL. 571.2 APRIL 4, 1961.
		2	"	35	
		3	2" T.O.	P.M.	Q_c'
		4	"	"	
		5	"	"	Q_c'
		6	"	"	Q_c'
		7	"	"	
		8	"	"	
		9	"	"	Q_c'
		10	"	"	
		11	"	"	
		12	"	"	
		13	"	"	
		14	"	"	γ = 117 P.C.F.
506.5 / 77.5	SILT LAYERS PRESENT BELOW EL. 506.5 PROBABLY UP TO 4" THICK				
482.5	HARD CLAYEY SILT	15	D.O.	>100	K = 5×10⁻⁴ CMS/SEC (FROM FIELD MEASUREMENT IN B.H.)
480.5 / 103.5	SAND & GRAVEL (TILL)				
477.7 / 106.3	LIMESTONE DOLOMITE BEDROCK	16	AXT R.C.	—	
	END OF HOLE				



VERTICAL SCALE 1 INCH TO 10 FEET

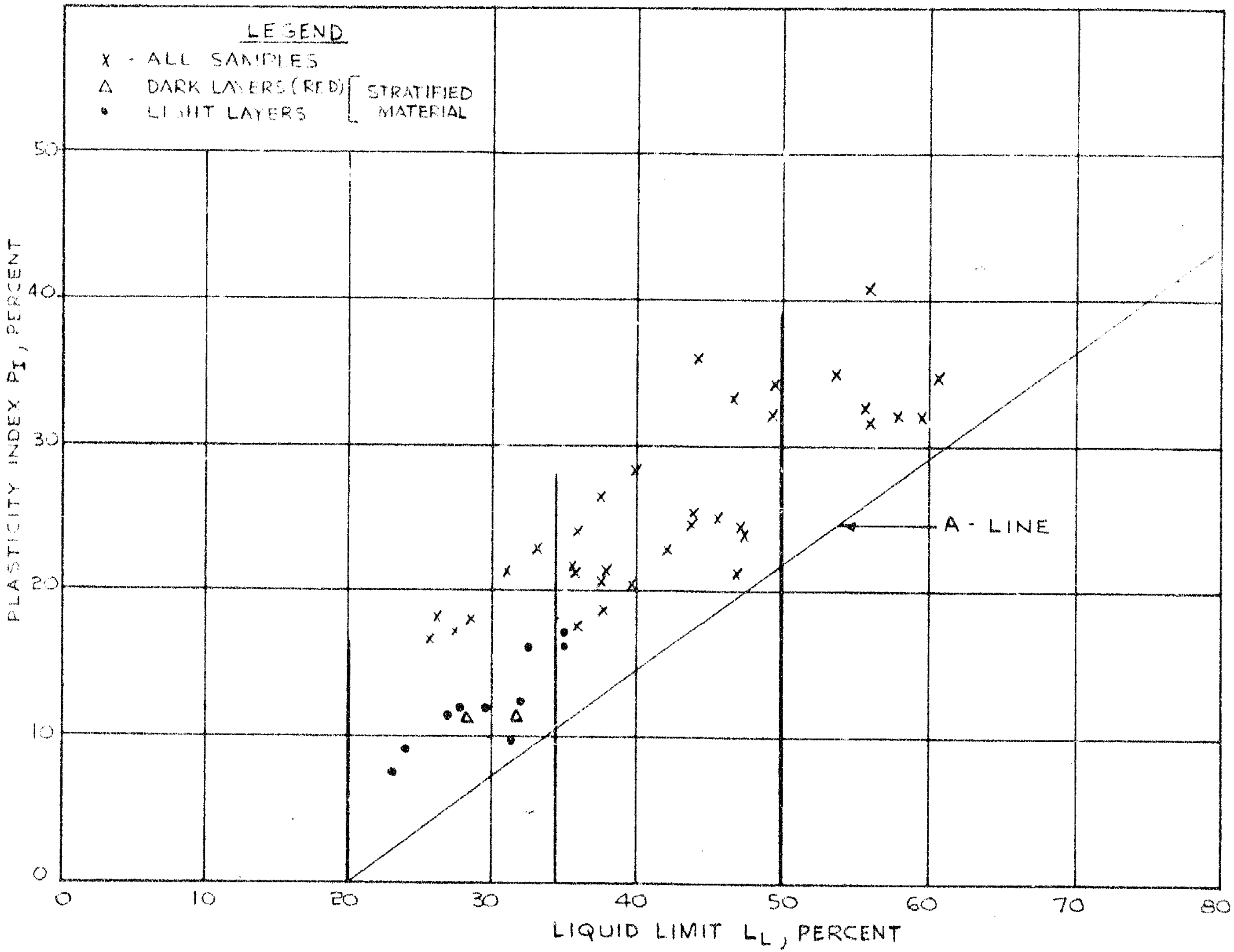
GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED

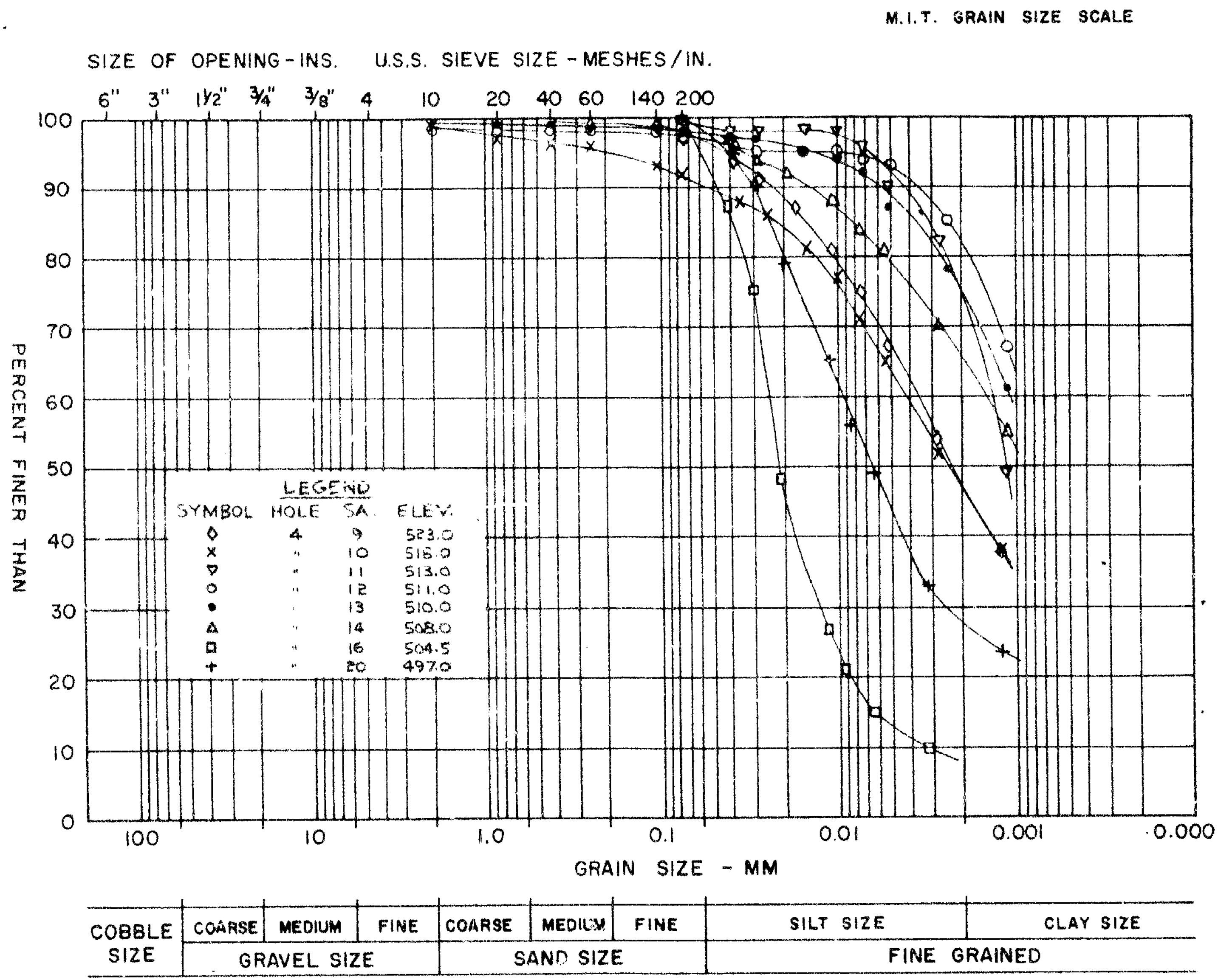
PROJECT No. 6108

GOLDER & ASSOCIATES



PLASTICITY CHART

FIGURE 2



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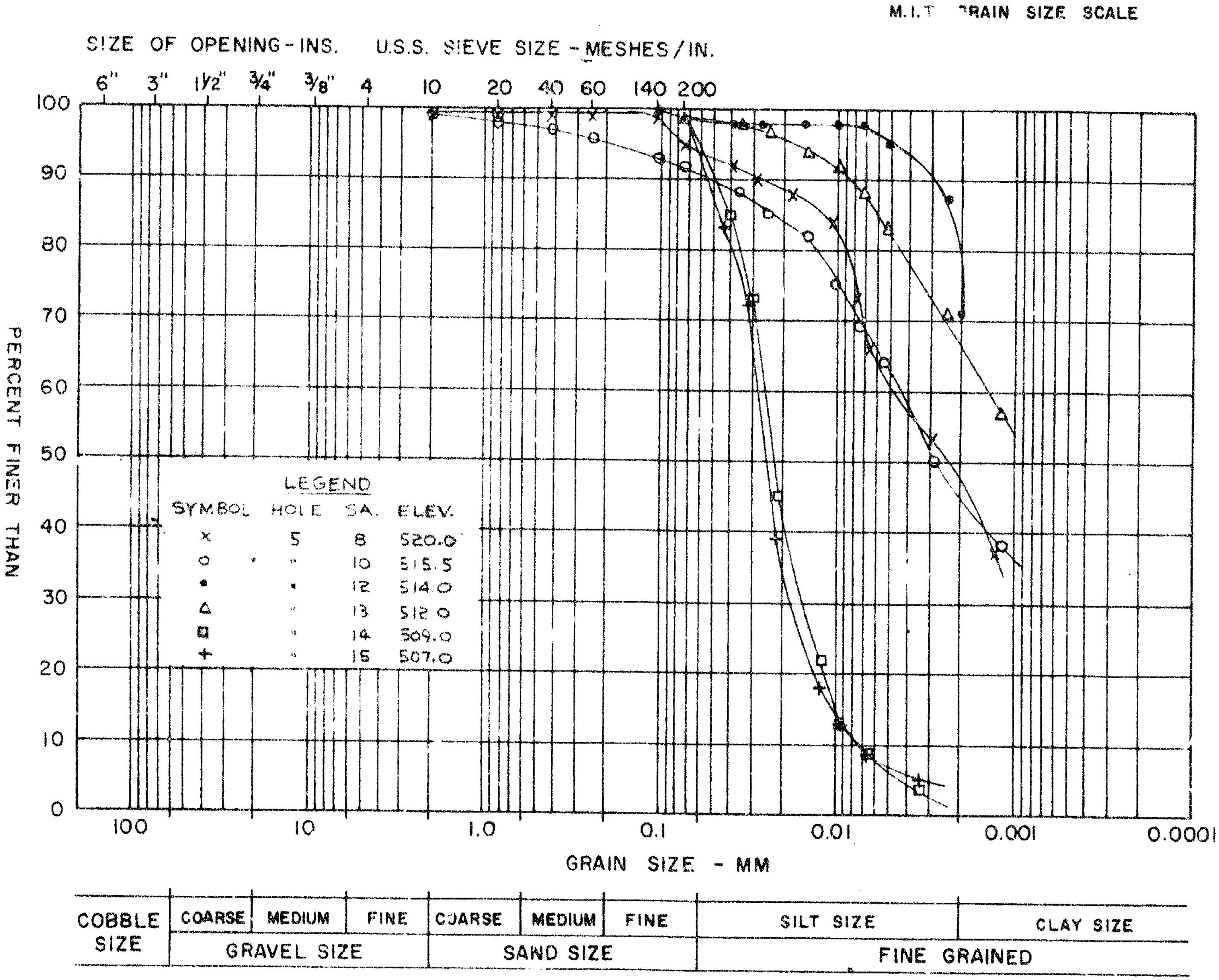
GRAIN SIZE DISTRIBUTION

FIGURE 3



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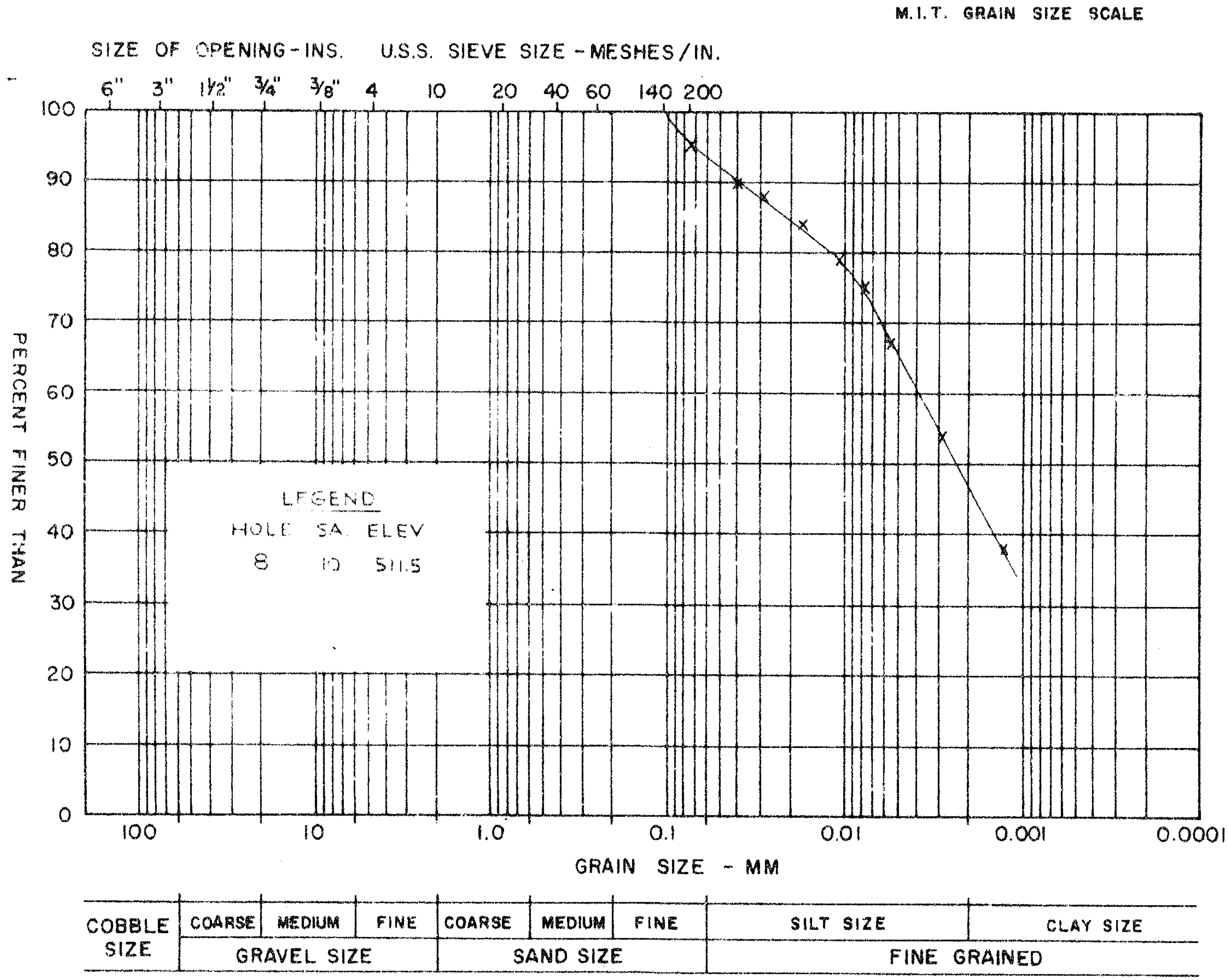
GRAIN SIZE DISTRIBUTION

FIGURE 4

PROJECT No. 6128



GOLDER & ASSOCIATES

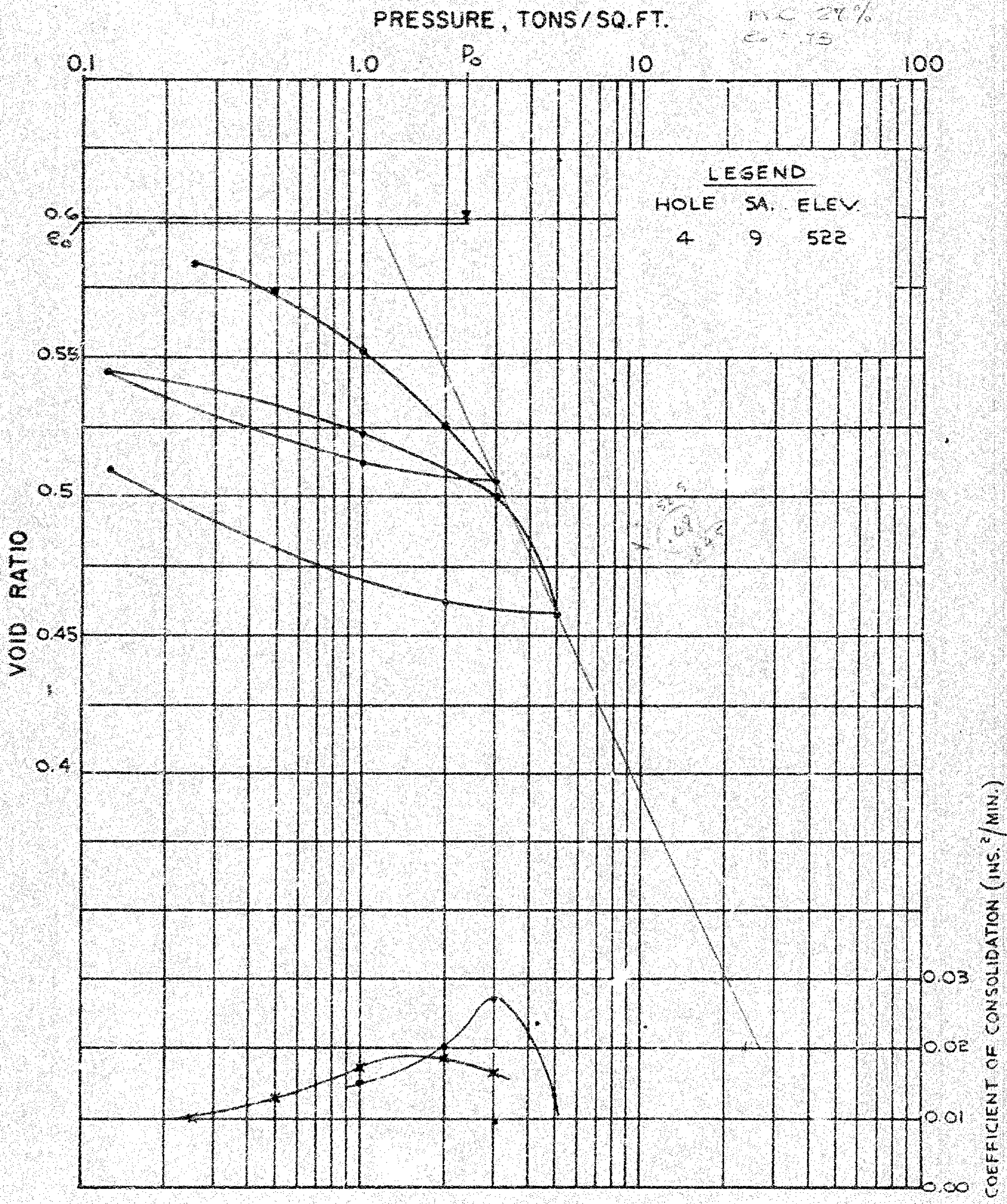
GRAIN SIZE DISTRIBUTION

FIGURE 5

PROJECT No. 6108

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

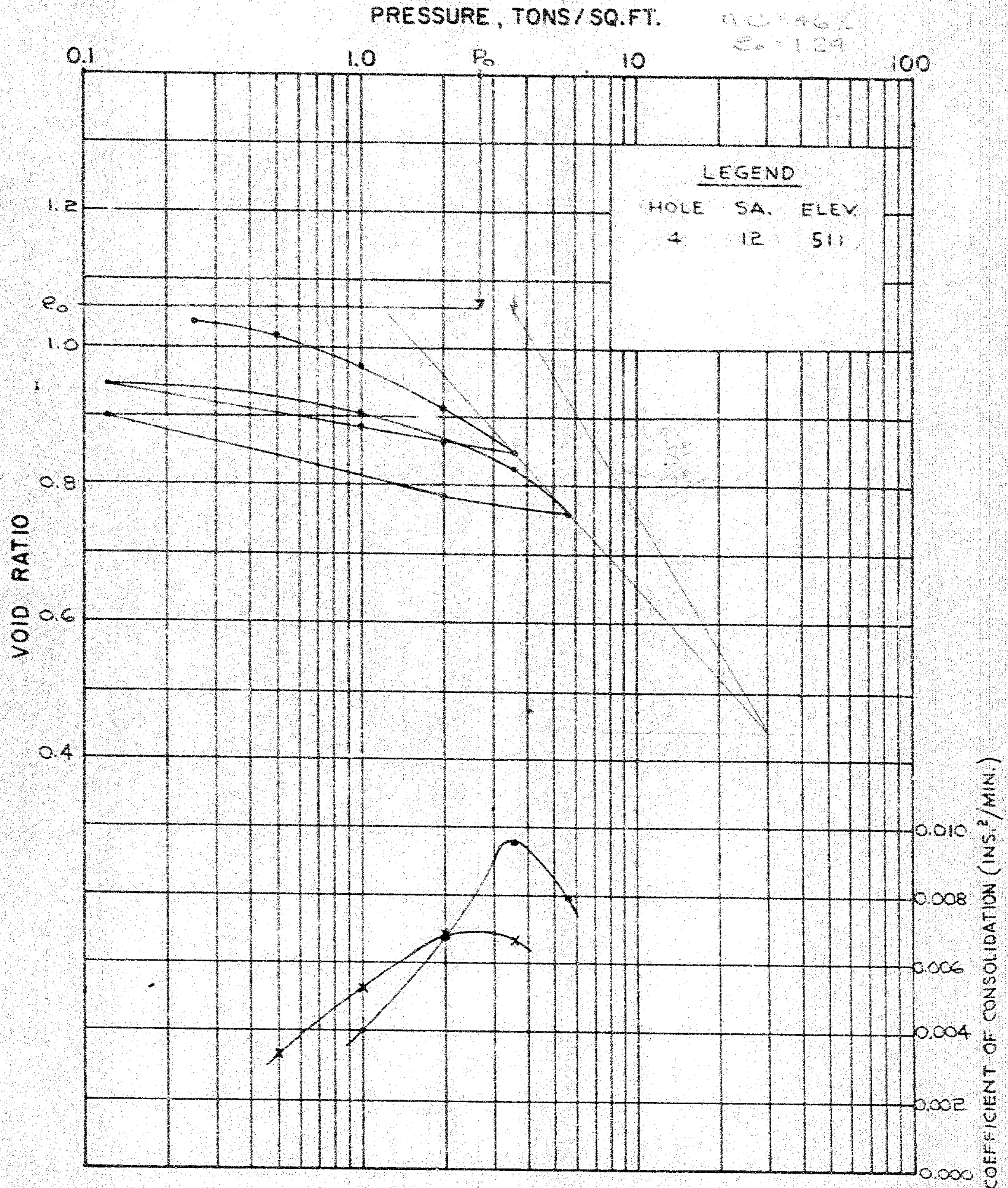
FIGURE 6



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

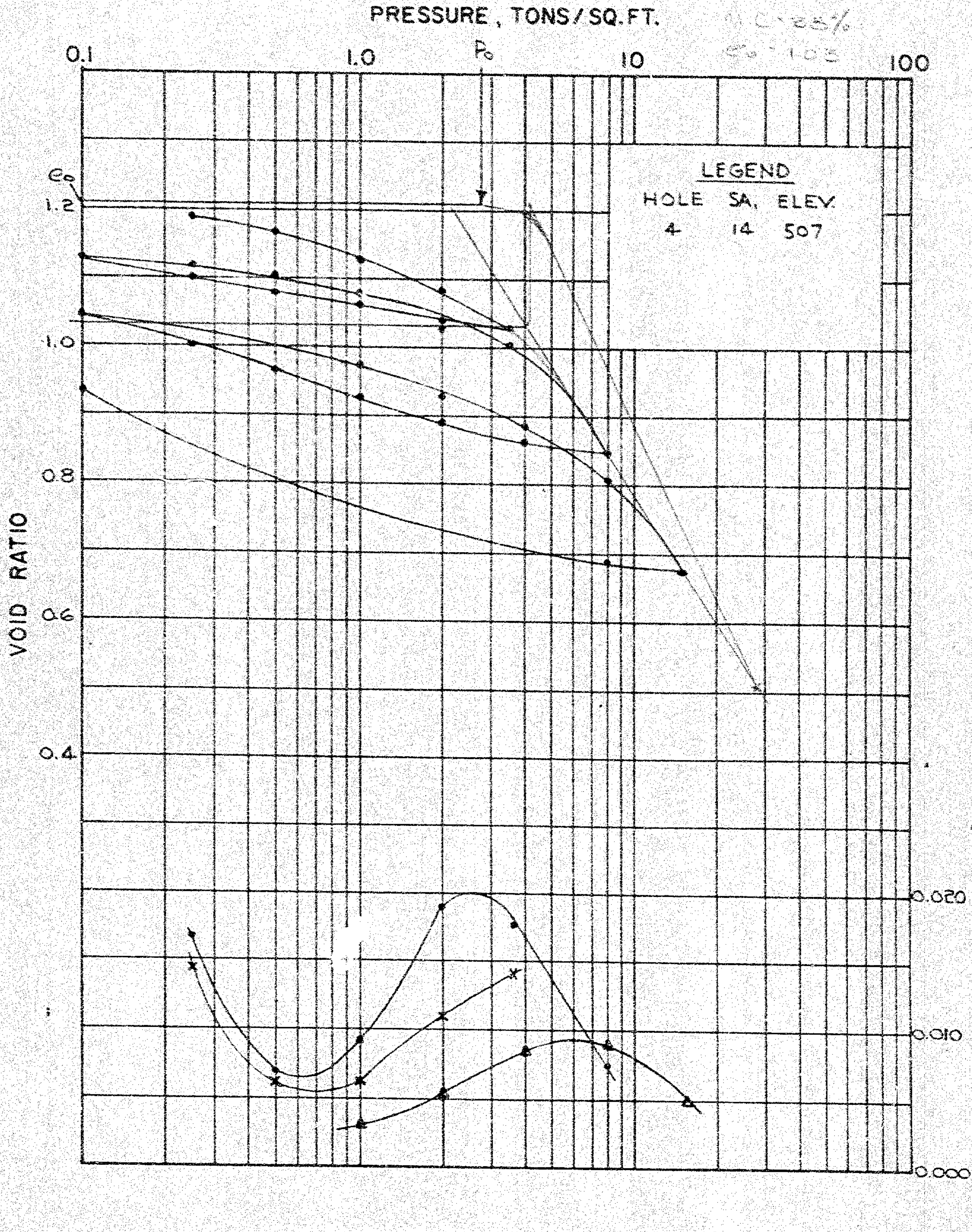
FIGURE 7



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 8



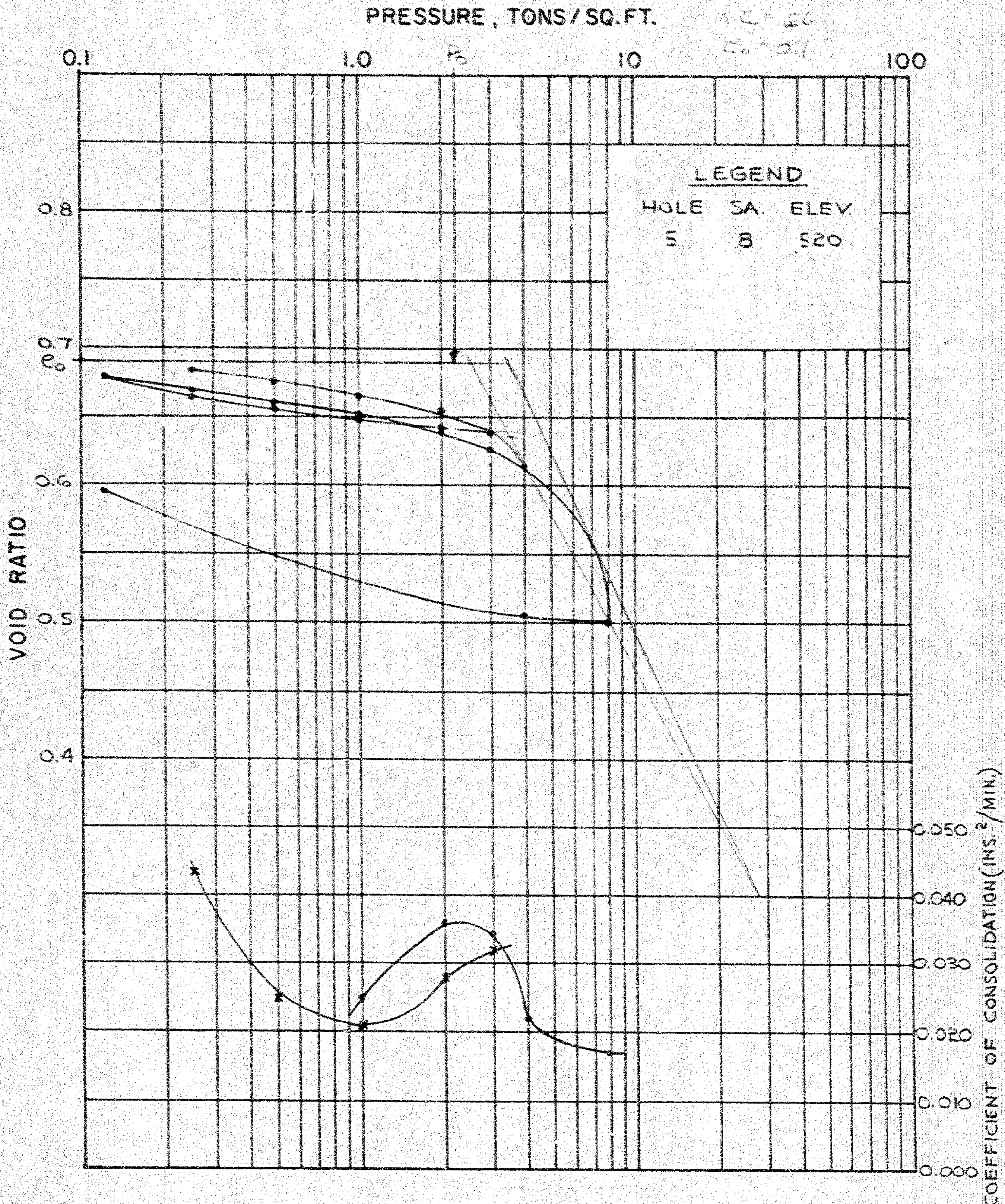
GOLDER & ASSOCIATES

PROJECT No. 6108

PROJECT No. ......9108......

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 9

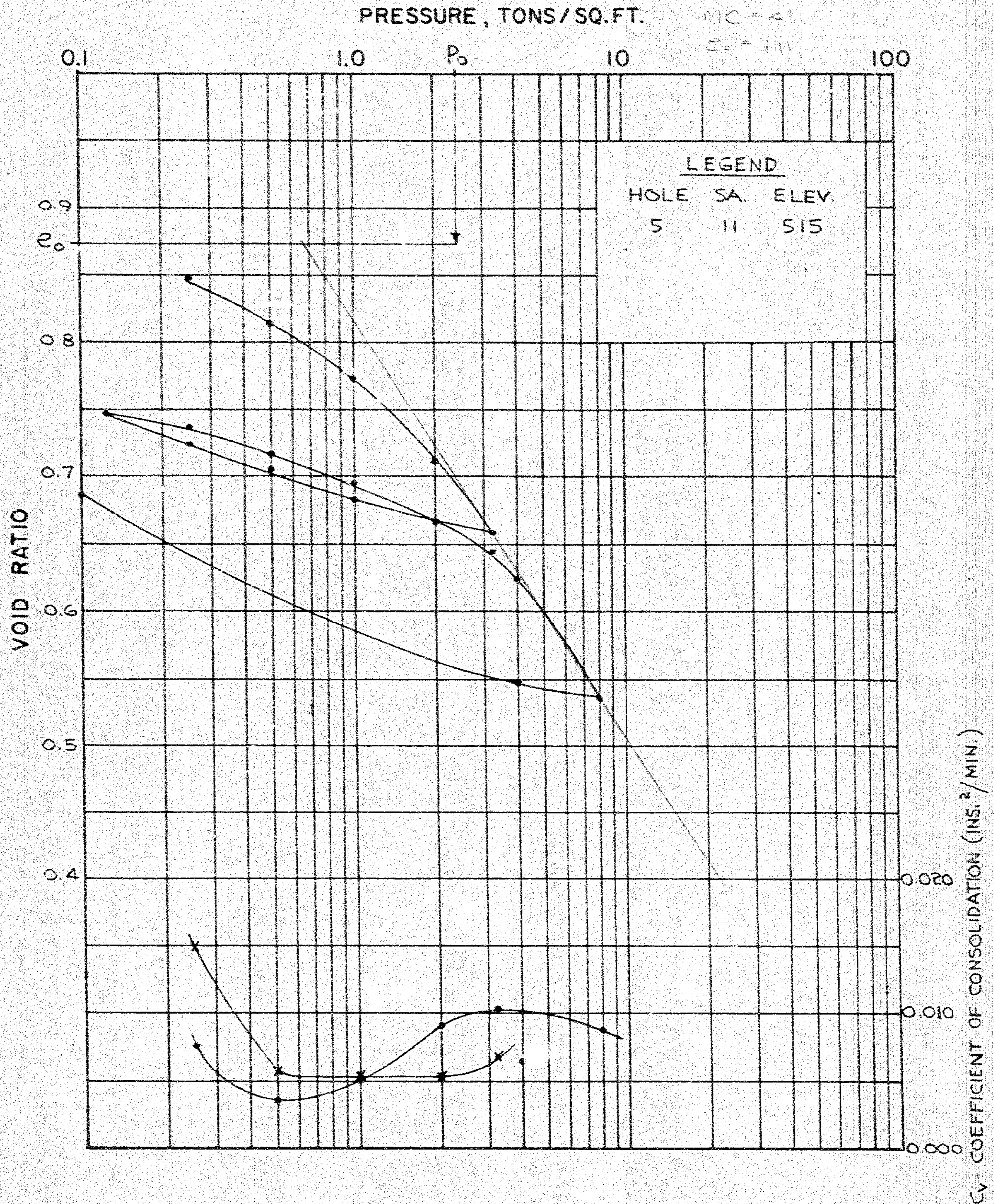


GOLDER & ASSOCIATES

PROJECT No. 6108

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 10

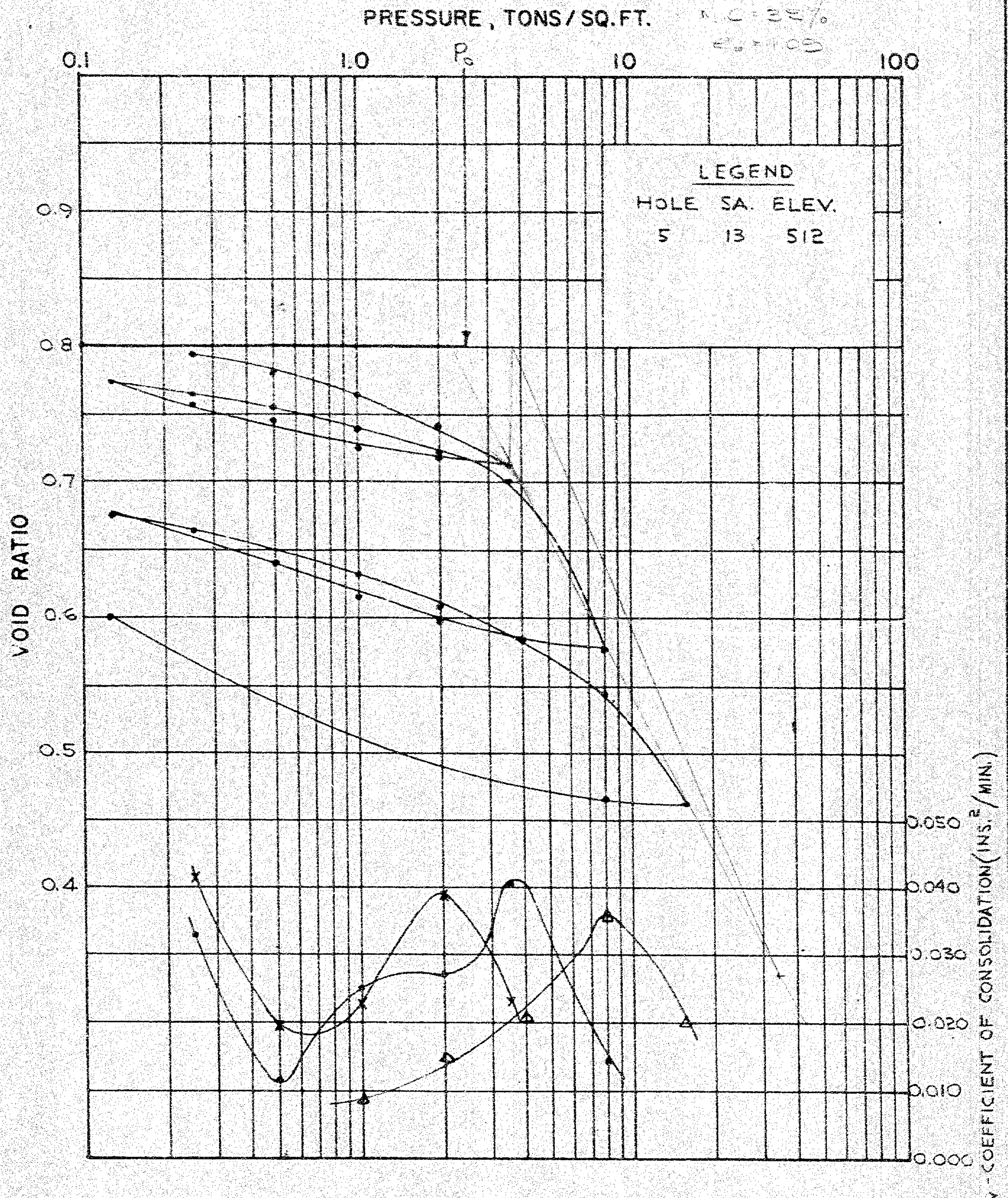


GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 11

PROJECT No. 6108

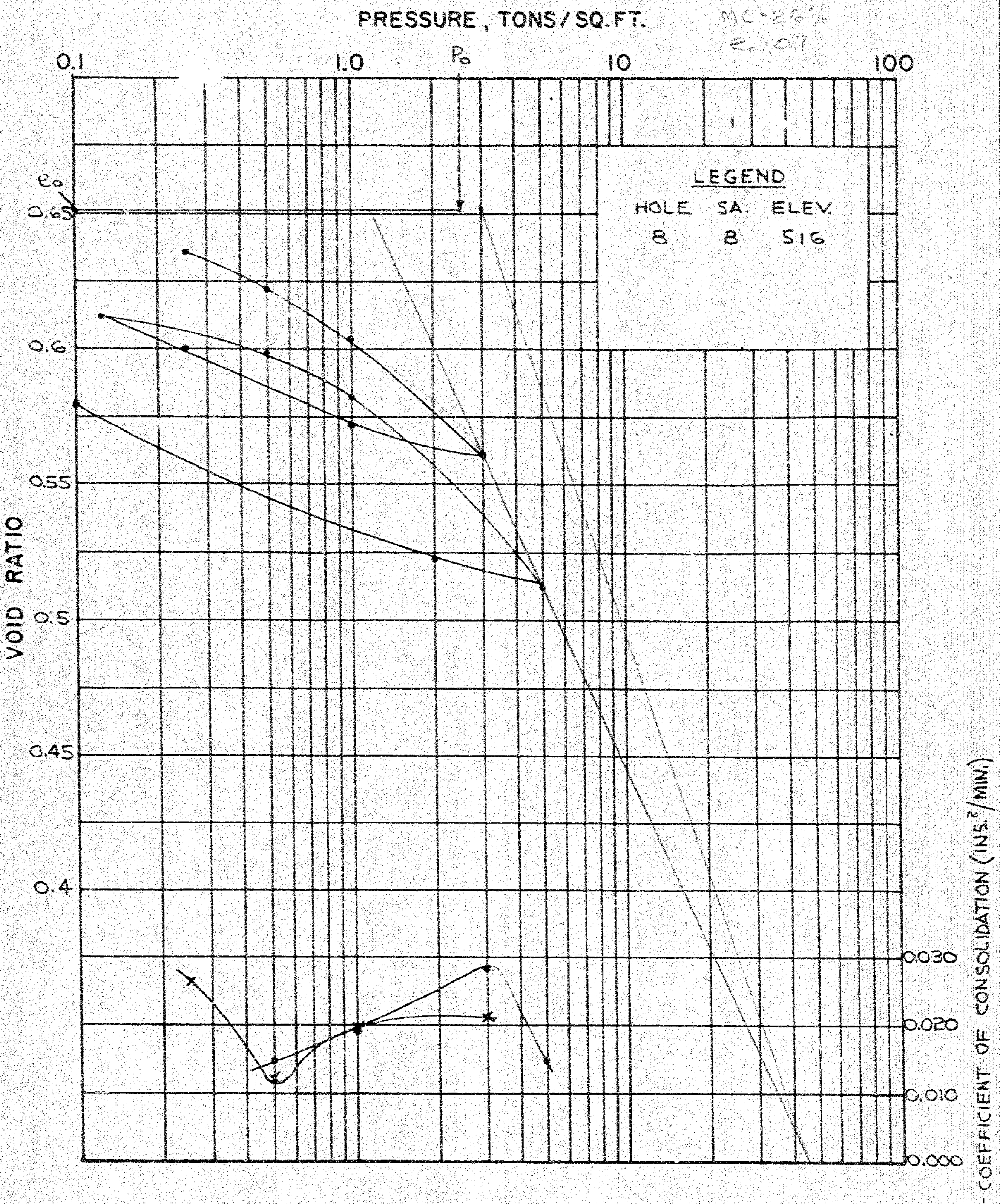


GOLDER & ASSOCIATES

PROJECT No. 61.08

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 12

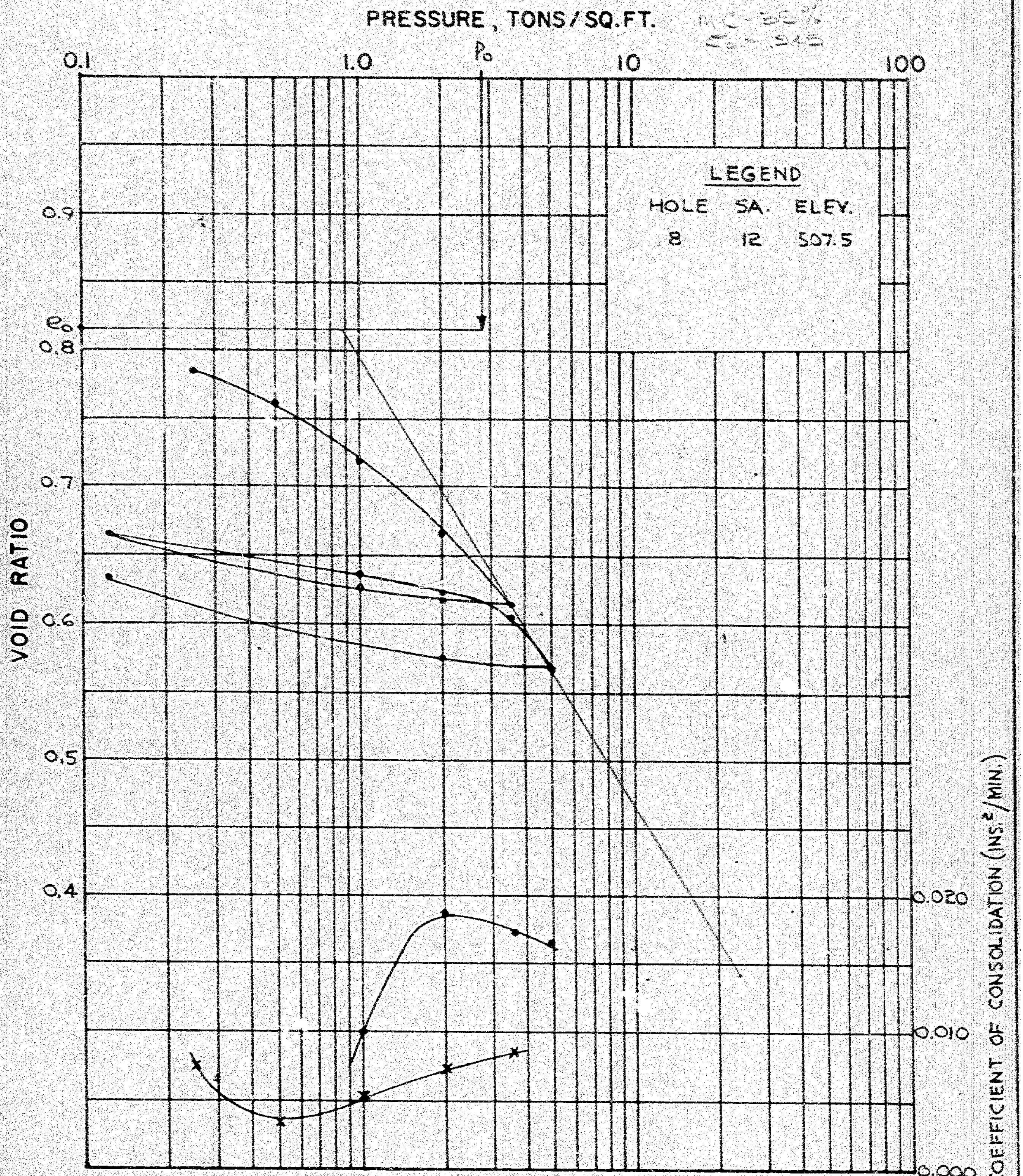


GOLDER & ASSOCIATES

PROJECT No. 6108

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 13

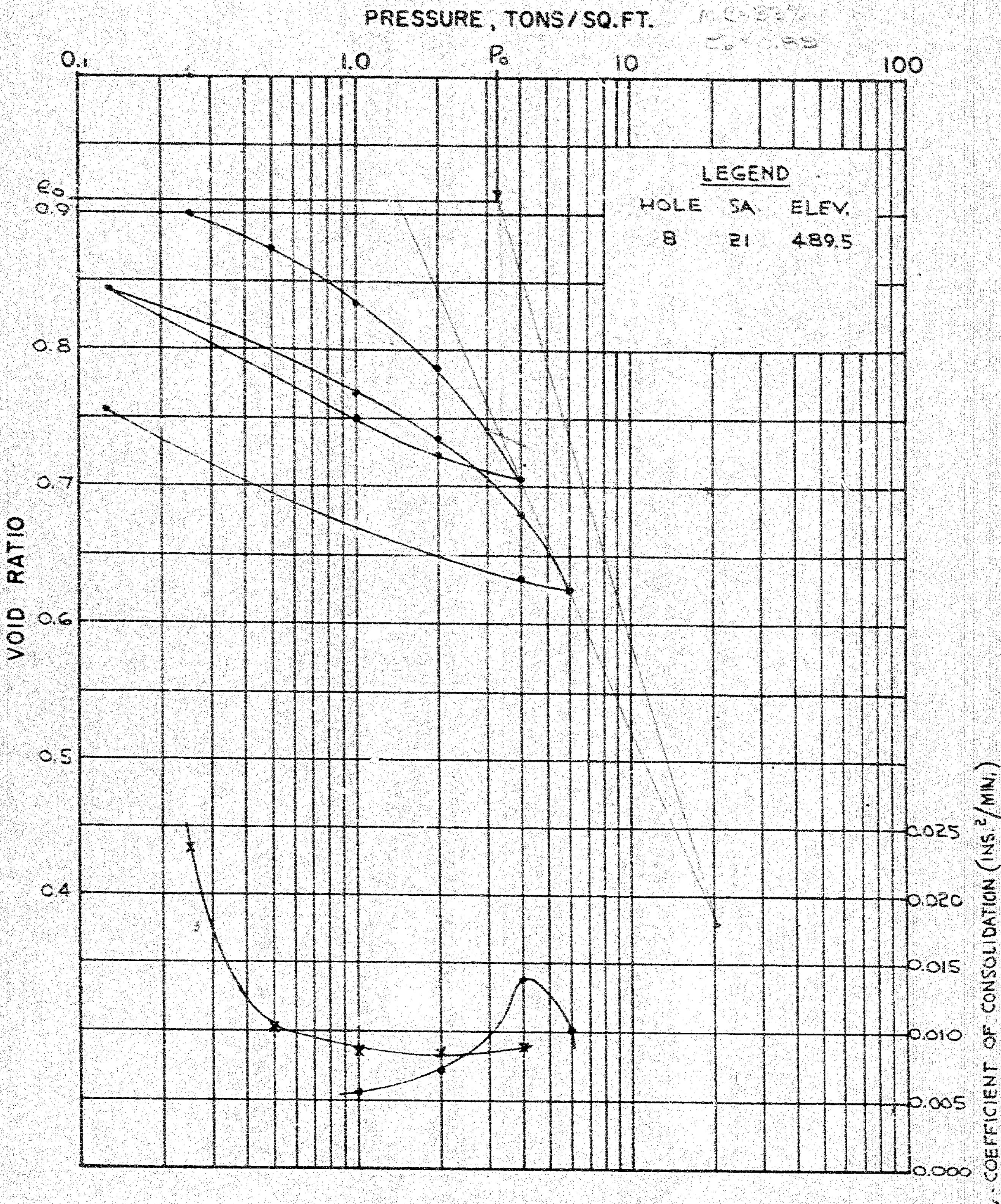


GOLDER & ASSOCIATES

PROJECT No. 6108

VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 14



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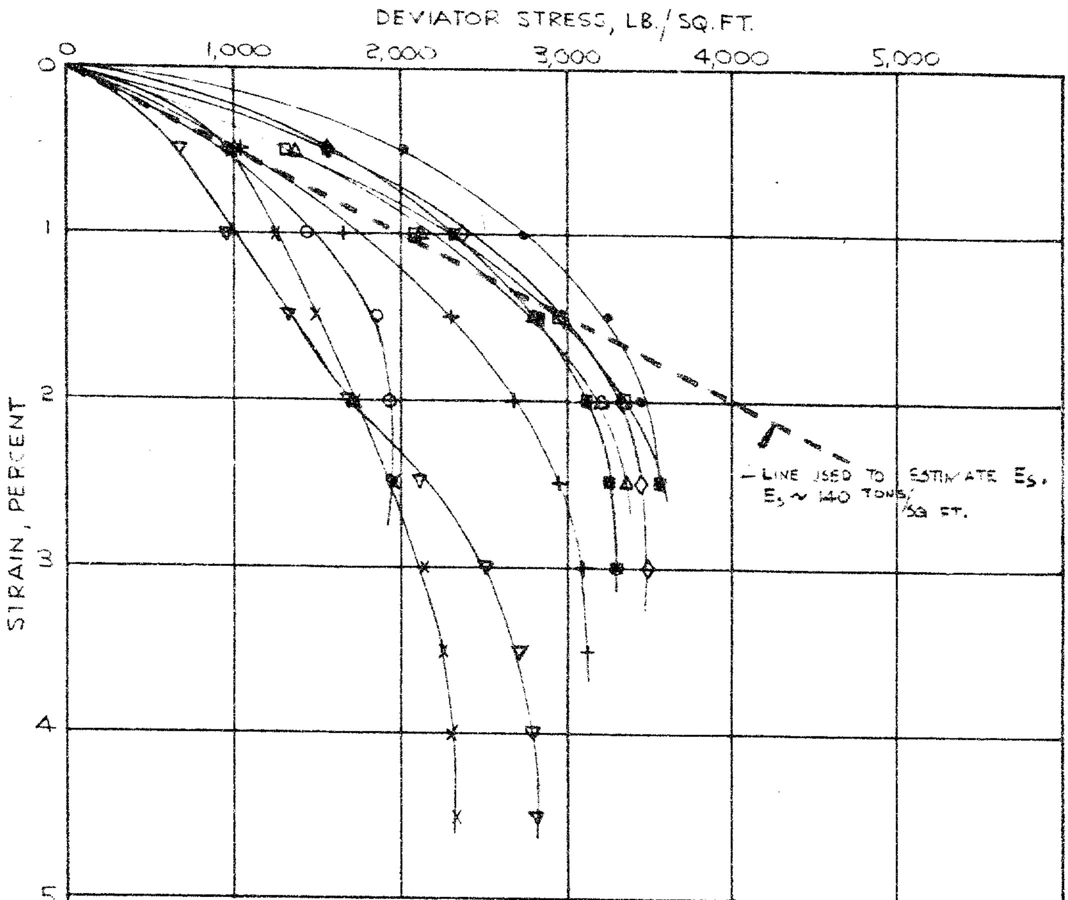
PROJECT No. 6108

UNDRAINED TRIAXIAL COMPRESSION TESTS

TYPICAL STRESS-STRAIN CURVES

FIGURE 15



LEGEND

SYMBOL	HOLE	SAMPLE	ELEV.
×	5	13	512.0
○	5	14	510.0
•	8	13	505.5
△	8	17	497.7
□	8	18	495.0
+	8	19	492.7
▽	8	20	490.7
◇	8	21	490.0
■	9	14	507.3

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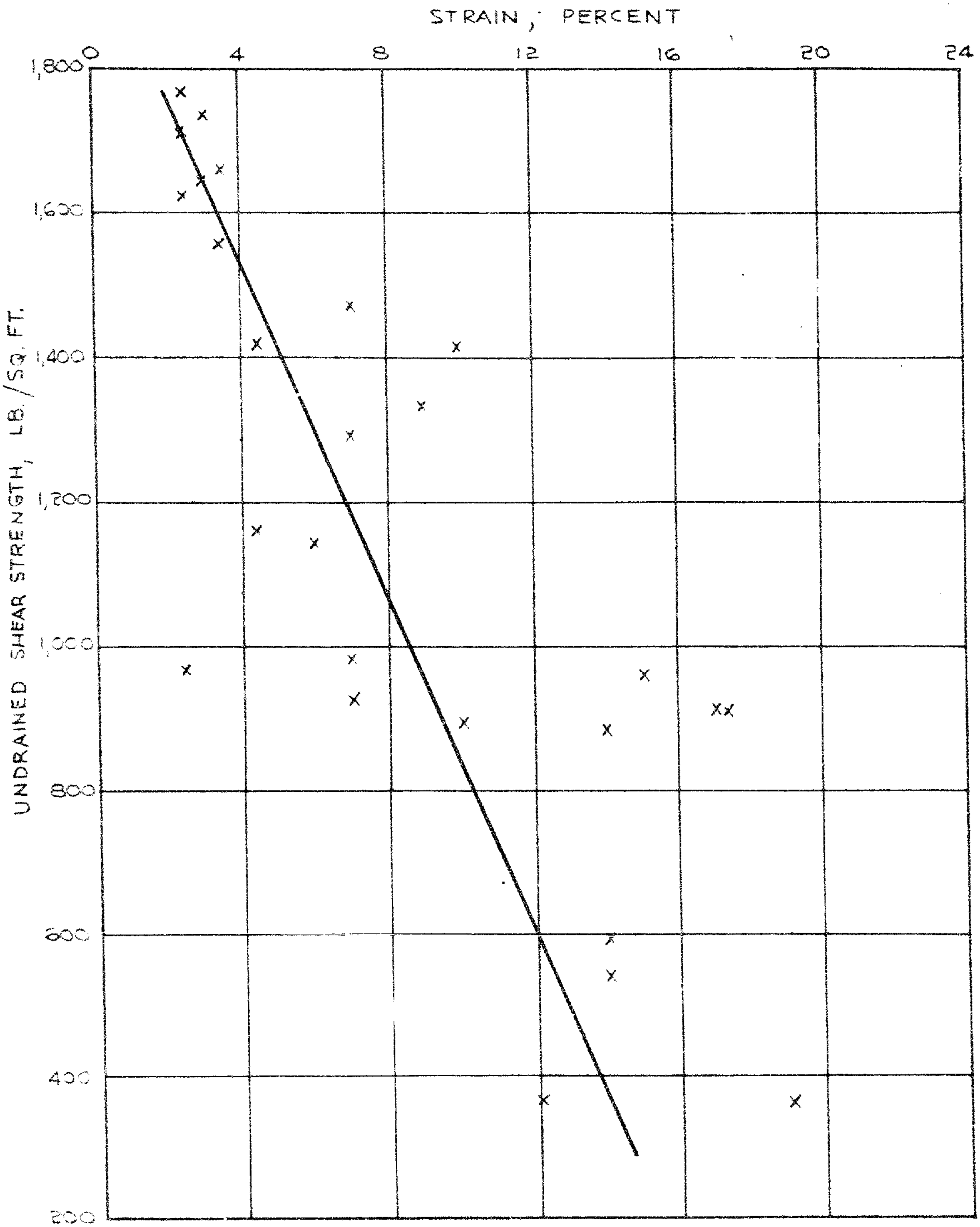
PROJECT No. 6108

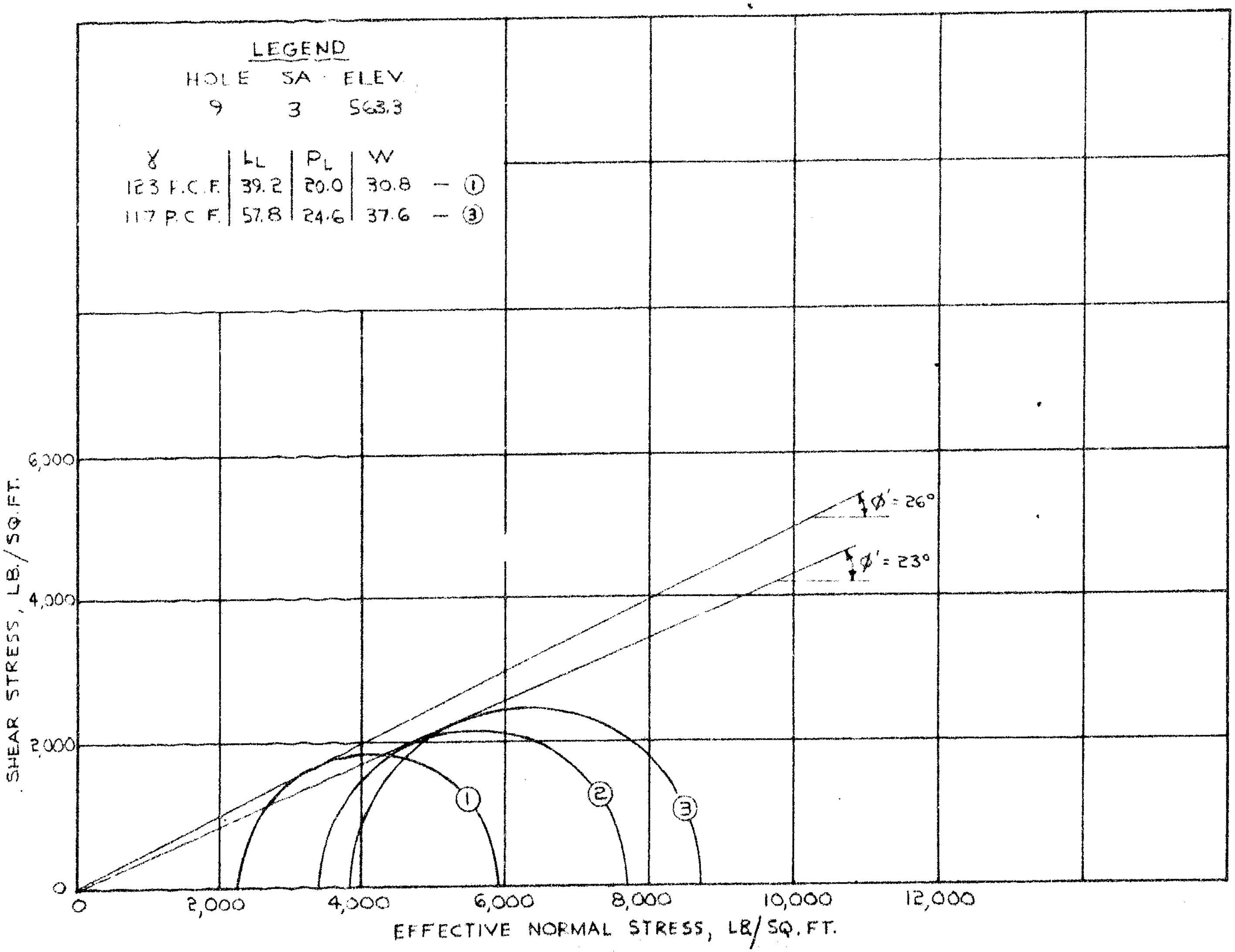
UNDRAINED SHEAR STRENGTH, C VERSUS STRAIN AT FAILURE

FROM UNDRAINED TRIAXIAL TESTS

FIGURE 16



GOLDER & ASSOCIATES



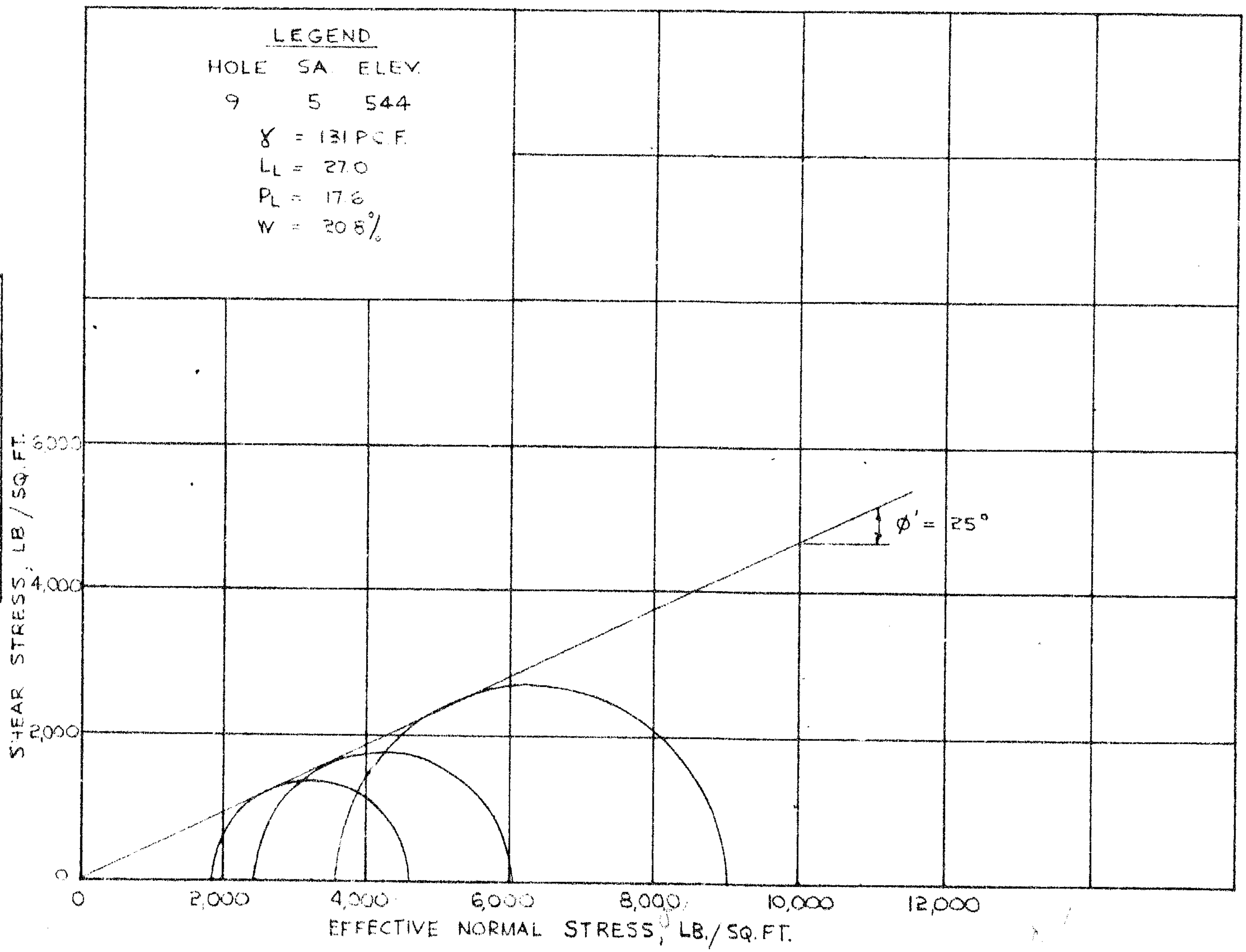
GOLDER & ASSOCIATES

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS

WITH PORE PRESSURE MEASUREMENTS

FIGURE 17

PROJECT No. 6108

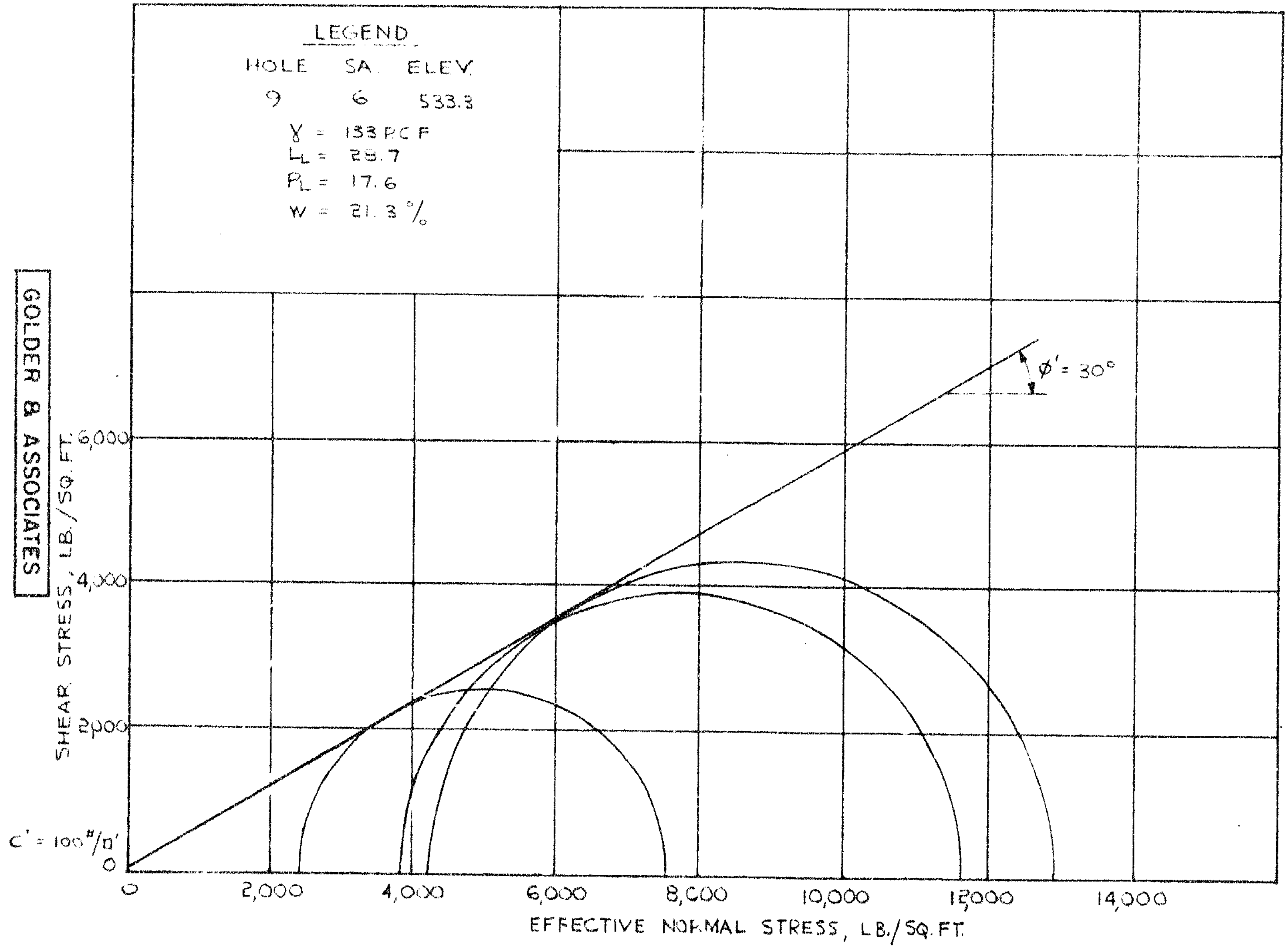


GOLDER & ASSOCIATES

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS
WITH PORE PRESSURE MEASUREMENTS

FIGURE 18

PROJECT No. 6108



GOLDER & ASSOCIATES

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS
WITH PORE PRESSURE MEASUREMENTS

FIGURE 19

PROJECT No. 6108

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS

WITH PORE PRESSURE MEASUREMENTS

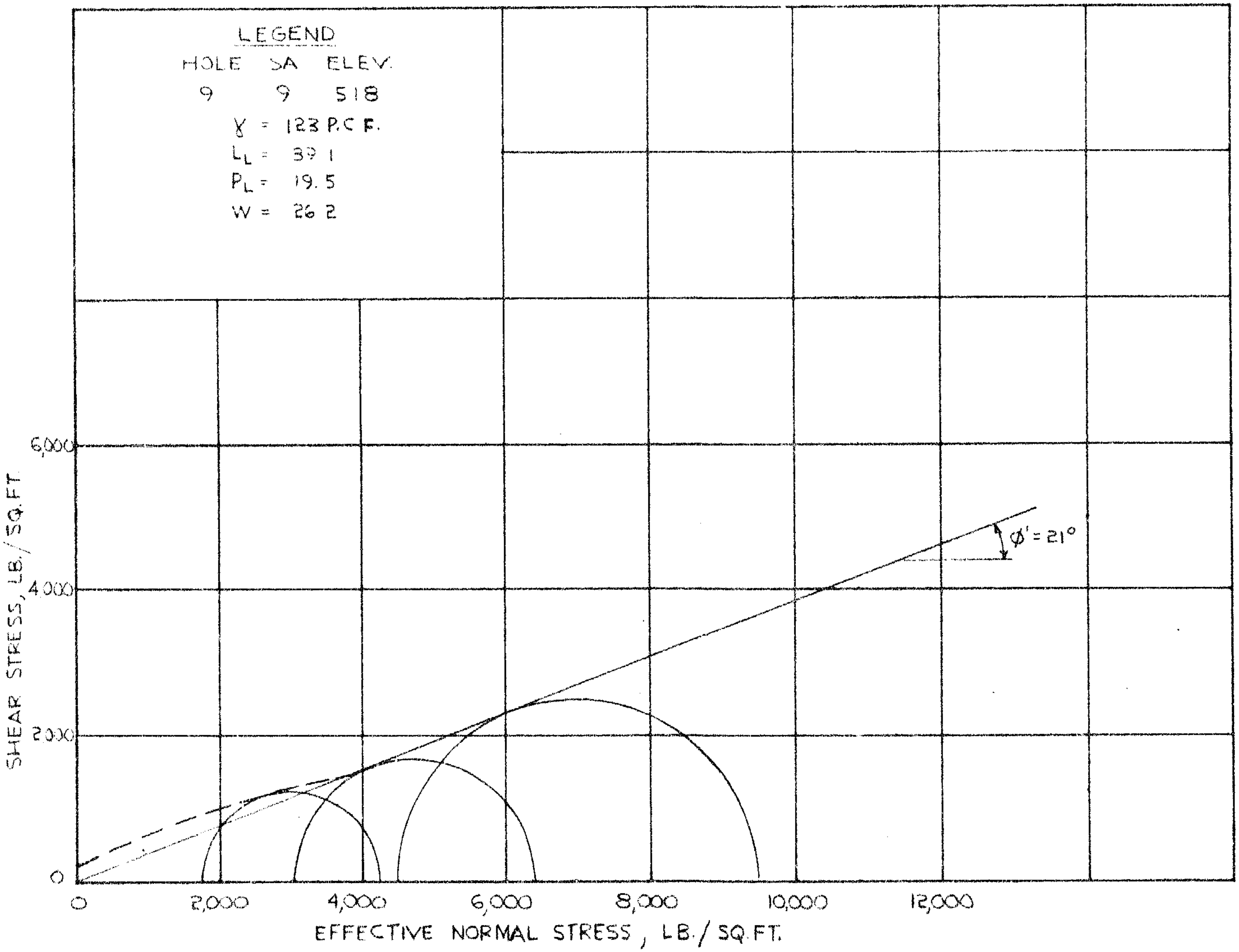


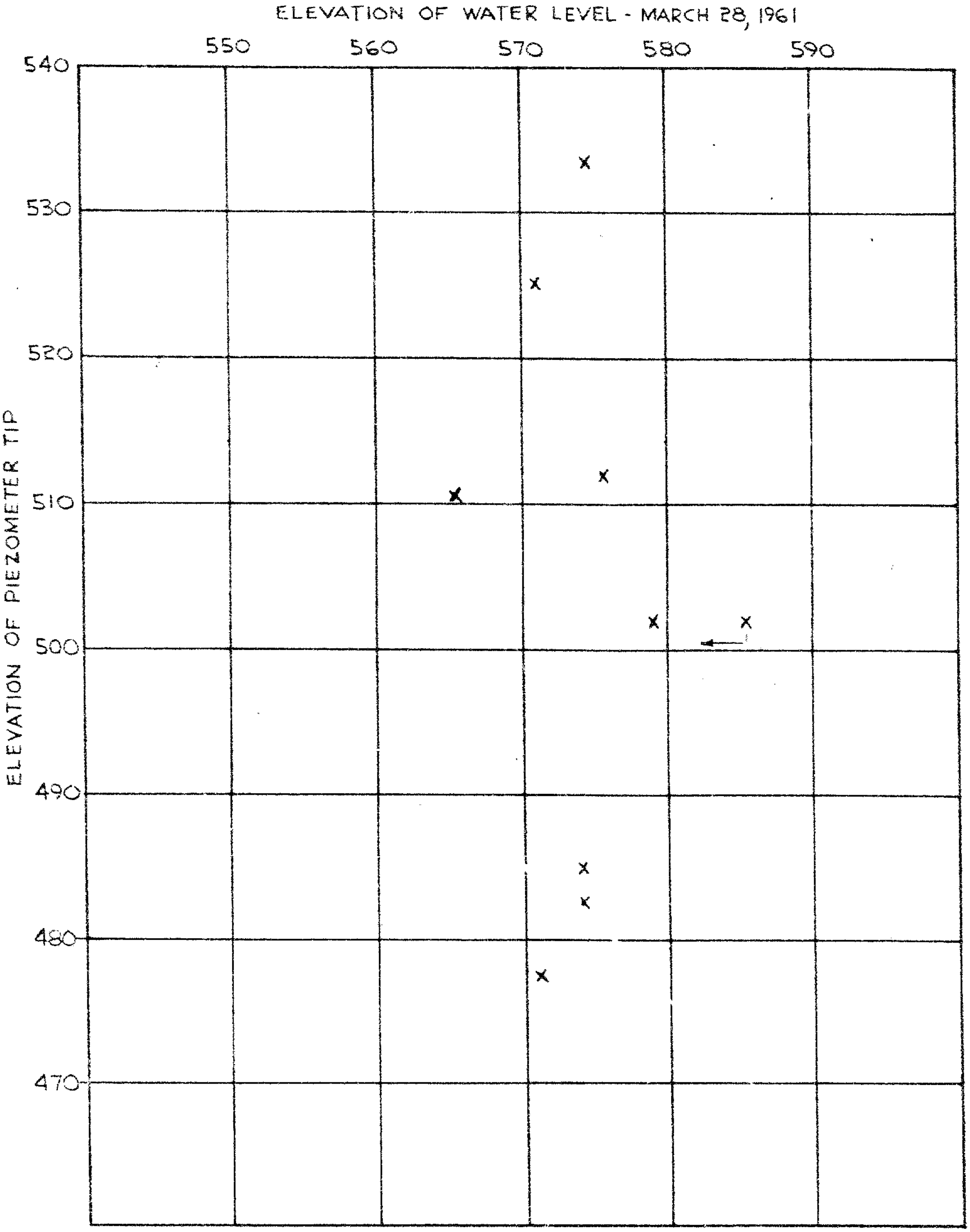
FIGURE 20

GOLDER & ASSOCIATES

PROJECT No. ....

WATER LEVEL MEASUREMENTS IN PIEZOMETERS 1A TO 9A

FIGURE 21



GOLDER & ASSOCIATES

APPENDIX I

DESCRIPTION OF INDIVIDUAL SAMPLES

BOREHOLES 4, 5 and 8

APPENDIX I

DESCRIPTION OF INDIVIDUAL SAMPLES*

Borehole No. 4

Depth in Feet	Description
3.0-4.5	Stiff brown oxidized silty clay cont. 1/8" layers of red clay spaced ½" - 1½" apart.
9.0-10.5	V. stiff brown oxidized silty clay.
19.0-20.5	Medium stiff red-brown silty clay cont. 1/8" bands of red clay spaced ½" apart.
29.0-30.5	Medium stiff red-brown silty clay cont. occasional small stones and tiny pockets of gray silt.
39.0-40.5	As above.
49.0-51.5	Medium stiff red brown silty clay with ½" bands of red clay & 1/4" bands of green clay spaced 1"-2" apart.
54.0-55.0	Medium stiff red silty clay cont. 1/4" layers of grey clay and 1/4" layers of brown silt at varied slopes of stratification.
55.0-55.5	Medium stiff red brown silty clay.
59.0-60.5	Medium stiff red brown silty clay traces of green shale.
64.0-65.5	Medium stiff red brown silty clay - traces of green shale.
69.0-70.5	Medium stiff red brown silty clay cont. occasional pieces of fine gravel.
76.0-77.5	Medium stiff red-brown silty clay cont. 2" layers of red clay and 1/4" layers of grey clay.
77.5-79.0	Medium stiff red-brown silty clay cont. 2"-2½" layers of red clay and 1/4"-½" layers of grey clay.
79.0-80.5	Medium stiff red-brown silty clay cont. 1/4" layers of grey clay and ½"-1" layers of brown silt.

*(Descriptions made by Mr. K. Selby, D.H.O.)

Borehole No. 4 - Cont'd 2

81.0-81.5	Brown silt.
81.5-82.5	Red brown silty clay with thin 1/4"-1" silt seams- occasional small pockets of grey clay.
82.5-84.0	4" layers of brown silt parted by a 1/4" layer of grey clay and a 1½" layer of red clay.
84.0-85.0	As above.
85.0-87.0	As above.
77.0-89.0	4" layers of brown silt parted by thin 1/4" layers of grey clay and 1½" layer of red clay.
89.0-90.5	Medium stiff red-brown silty clay cont. 1/4" -3" layers of brown silt.
90.5-92.0	Medium stiff red brown silty clay cont. thin (1/8"-1/4") silt seams brown in colour 1"-2" apart: also occasional grey clay layer.
92.0-92.8	Medium stiff red brown silty clay cont. thin seams 1/4" and pockets (small) of brown silt 1"-2" apart.
92.8-93.0	Layer of red clay and grey clay underlain by ½" layer of fine sand (grey).
93.0-93.5	Red brown clay cont. occasional v. small silt pockets - medium stiff.
94.0-95.3	Red brown silty clay cont. traces of grey silty clay - medium stiff.
95.3-95.5	Stiff grey silty clay (brown gray in colour) cont. fine - coarse sand and fine gravel (Till).

Borehole No.5

5.0-6.5	Stiff brown oxidized silty clay cont. 1/8" bands of red clay spaced 3"-4" apart.
10.0-11.5	Stiff brown oxidized silty clay.
20.0-21.5	Medium stiff red brown silty clay with occasional stones and small pockets of silt.
30.0-31.5	As above.
40.0-41.5	Medium stiff red brown silty clay cont. small gravel occasional stones only up to 3/4".
50.0-51.5	Medium stiff red brown silty clay cont. ½" layers of grey clay 4"-5" apart.

60.0-61.5	Medium stiff red brown silty clay - some v. fine gravel.
65.0-66.5	Medium stiff red brown silty clay cont. occasional particles of fine gravel.
66.5-68.0	As above.
69.5-70.0	Medium stiff red-brown silty clay with occasional stones - up to 3/4".
70.0-70.5	Medium stiff red-brown silty clay.
70.5-71.0	Medium stiff red-brown silty clay.
71.0-72.5	Medium stiff red brown silty clay cont. ½" layers of grey clay and ½" layer of red clay 4"-5" apart.
73.0-74.5	1"-2" layers of red-brown silty clay (medium stiff) with 1/4"-1" layers of red clay with 1/4" layers of grey clay. Grey layers parted by red layers.
75.0-76.0	4" layers of brown silt underlain by ½" layers of grey clay and red brown clay.
76.0-76.5	Medium stiff red-brown silty clay underlain by 1" of brown silt.
77.0-78.0	Brown silt underlain by 1/8" layer of red clay and ½" layer of grey clay.
78.0-79.0	Brown silt layers 1"-3" parted by 1/4" layers of red brown silty clay.
80.0-81.3	Medium stiff red-brown silty clay layers ½"-1" thick parted by brown silt layers 1/16"-2" thick.
81.3-81.5	1" layer of red clay and 1 3/4" layer of dark brown silty clay parted by 1/4" layer of grey clay.
81.5-81.7	Brown silt.
81.7-82.4	Red-brown silty clay cont. numerous very thin silt seams. (Med. stiff).
82.4-82.6	1½" layer of red clay underlain by ½" grey clay.
82.6-83.0	Red brown silty clay medium stiff.
83.0-84.5	Red brown silty clay cont. frequent silt seams - very thin to 1" thick. Also v. small pockets of silt. (At 84.0' 5/8" layer of red clay underlain by 1/4" layer of grey clay). Occasional stones (small). Tip of tube cont. 1/4" of silty sand.

19.0-20.5	Medium stiff red brown silty clay cont. occasional small stones and very small pockets of silt.
29.0-30.5	Medium stiff red brown silty clay cont. occasional small stones and very small pockets of silt.
39.0-40.5	As above.
49.0-50.5	As above.
59.0-60.5	Medium stiff red brown silty clay cont. occasional small stones and small silt pockets. ½" layer of red clay underlain by 1/4" layer of grey clay spaced 6" apart.
64.0-65.5	Medium stiff red brown silty clay cont. occasional small stones and small silt pockets.
66.0-67.5	As above.
68.0-69.5	Medium stiff red brown silty clay with occasional stones and small pockets of silt.
70.0-71.5	As above.
73.0-74.5	As above.
76.0-77.5	Lost sample.
77.5-79.0	Soft-medium red brown silty clay. 3" layer of grey clay at bottom.
79.5-81.0	Medium stiff 1½"-2" layers of red clay with 1/4"-2" layers of grey clay also ½" layers of red brown clay. 1/4" layers of grey silt spaced 2" to 3" apart.
81.5-83.0	Medium stiff red clay layers 1"-3" thick with grey clay layers 1/4" - 1" thick also red brown layers 1"-2" thick.
83.5-85.0	Medium stiff red brown silty clay cont. ½" layers of red clay spaced 4" apart also 1/4" layers of silt spaced 3"-6" apart.
85.5-86.0	Brown silt.
86.0-86.5	Red clay med-stiff cont. 1/4" layer of grey clay.
86.5-87.0	Medium stiff red brown silty clay.
87.5-89.0	1"-3" layers of brown silt spaced 2"-4" apart cont. layers of medium stiff red brown clay and 1/4" layers of grey clay.

Borehole No. 8 - Cont'd 5

89.5-91.0	Medium stiff red brown silty clay cont. 1" layer of grey clay and 1" layer of red clay.
91.5-92.5	2" layer of red clay. 2½" layer of red, brown and grey mottled clay.
92.5-93.0	Red brown silty clay medium stiff cont. occasional small stones.
93.5-95.0	Medium stiff red-brown silty clay occasional small stones and small pockets of silt.
95.5-97.0	As above.
97.5-98.3	Medium stiff red brown silty clay.
98.3-98.6	1/4" seam of fine sand underlain by sandy till.
98.6-100.1	Hard brown clayey silt with fine - coarse sand and gravel - glacial till.

APPENDIX II

NOTES CONCERNING WELLAND CANAL TUNNEL

AND APPROACHES

APPENDIX II

NOTES CONCERNING

WELLAND CANAL TUNNEL AND APPROACHES

(Dated April 26th, 1961 and presented on the basis of the laboratory data then available.)

Further to the meeting on April 20, 1961, and further discussion on April 24, 1961, between Mr. P. J. Thompson of Foundation of Canada Engineering Corporation Limited and the writer, these notes will confirm points of discussion and our preliminary findings concerning soil conditions and foundation design for the proposed Welland Canal Tunnel at Welland, Ontario.

General outlines of the proposed tunnel scheme are shown in Fenco drawings 2301-5T-9 and -10 dated April 1961. These drawings have been given to us. The tunnel will consist of three precast reinforced concrete units about 70 feet by 32 feet in section and 267 feet long placed in trench excavation below the present Welland Canal bottom and with open approaches some 550 feet in length in reinforced concrete trough sections. The maximum grade in the tunnel or approaches will be 5.5 percent.

The main points covered by the discussion were:-

1. Soil Conditions.
2. Proposed tunnel grade.
3. Movement of tunnel units and approach trough sections due to swelling and elastic rebound at the bottom of excavated trenches or approach cuts.

4. The stability of the approach cuts at various stages of construction.

These points are discussed separately below.

(i) SOIL CONDITIONS

Ground surface at the site is generally horizontal. The soil conditions consist of a very stiff to firm, red brown silty clay containing thin layers of red and grey clay and occasional small gravel particles up to 1 inch in size. Occasional pockets or small lenses of silt were also encountered. The upper portion of this clay is highly desiccated. The approximate thickness of the desiccated crust is generally about 16 feet. The average thickness of the clay may be taken to be about 79 feet.

Below this depth it is underlain by a stratified deposit consisting generally of brown silt layers up to 4 inches in thickness separated by layers approximately 2 inches in thickness of red and grey clay. The overall thickness of this stratified deposit is about 20 feet. In local areas the silt portion of the stratified deposit predominates. The overall consistency of this deposit is generally stiff or where silt is predominant it may be considered as dense.

The stratified deposit is underlain by till ranging in thickness from 2 feet to about 10 feet. The till consists of sand and gravel in a silt matrix. This till is extremely dense. It is underlain by limestone or dolomitic bedrock at an overall depth of about 100 feet.

The preliminary borehole logs which have been prepared by D.H.O. show the undrained shear strength as measured by in situ vane tests and unconfined compression tests to be extremely high in the upper portion of the silty clay deposit. Below this the strength is generally uniform between about 1000 to 1500 lbs/sq.ft. This undrained shear strength is quite constant across the site. It may be observed, however, that the quick triaxial or unconfined compression tests invariably show a lower strength than that measured by the vane. A study of this point has been made and it has been found that where the strain at failure of the triaxial test samples is high, namely over 10 percent, then the strength is quite low in comparison to the vane tests. However, where the strain of failure is less than 10 percent, there is good correlation between laboratory compression tests and the in situ results. It is inferred that the high strains measured in certain samples are due to unavoidable remoulding in sampling.

It is proposed to use for design an undrained shear strength of the order of 1000 to 1500 lbs/sq.ft. throughout the full depth of the clay deposit save in the desiccated crust where the shear strength is of the order of 2000 lbs/sq.ft. or greater.

It has been initially thought that the stratified deposit was under some artesian head. It was also postulated that the silt was extremely loose. However these postulates were found to be invalid as further careful sampling carried out at the site showed that the silt only became loose when subjected to a large unbalanced hydrostatic head in the borehole. When the borehole was completely filled with water prior to sampling at the elevation of the stratified deposit then the silt or stratified deposit was found to be extremely dense and the undrained shear strength was high. It is stressed however that the stratified layer is susceptible to piping when subject to an unbalanced hydrostatic head.

(ii) PROPOSED TUNNEL GRADE

The proposed tunnel grade has been discussed with Mr. Thompson and it is understood that the lowest grade during construction will be at about elevation 506. This lowest point is at the centre of the canal. There are no borings in this area. However as this portion will be excavated in the wet by trench methods then there is little danger of piping at the bottom of the excavation. The lowest grade to which the approach cuts will be lowered during construction is about elevation 519. It may be noted that this is sensibly above the stratified deposit susceptible to piping. Consequently we feel that provided due care is taken there should be only minor construction difficulties due to piping at the base of excavations.

(iii) MOVEMENT DUE TO SWELLING AND REBOUND

From the limited number of consolidation and swelling tests now available computations have been made to determine the possible movement of tunnel units or approach trough sections due to swelling of the clay on release of the overburden pressure. It is estimated that the maximum net movement due to swelling should generally be less than 3 inches. This movement however, will require time for complete swelling and then re-loading. During the limited construction period of about 4 to 6 months it can be estimated that differential movement due to swelling and re-loading of the clay should be relatively minor.

Further computations have been carried out to assess the elastic rebound of the clay on relief of the overburden pressure. The total elastic rebound has been computed from the modulus of elasticity, E, of the clay as determined from triaxial compression tests. E is in the range of 150 to 200 tons per square foot. Computations indicate that the elastic rebound at the base of excavations should generally be of the order of 1½ inches or less. In general, therefore, the total movements due to possible swelling plus elastic rebound would be in the order of 3 - 4½ inches. As the units are over 250 feet in length (L) they should be relatively flexible. Taking L/1000 as the limit of flexibility then the differential movement which may be accommodated is about 4 inches.

(iv) STABILITY OF APPROACH CUTS

We understand that initially a cutting some 40 feet in depth will be made for the approach trough sections. Side slopes of the cutting will be two horizontal to one vertical. Steel sheet piling will then be driven to refusal in the lower till stratum from the bottom of this cutting. The cutting will then be deepened within the confines of the sheet piling to about elevation 519. During excavation sheeting will be strutted or tied back. The excavation will be flooded during this deepening and while flooded a tremie concrete seal will be poured at the bottom of the flooded excavation. The top level of the tremie concrete plug will be at or about elevation 523.

The stability has been initially examined as illustrated on Figure 1, attached. In this analysis cuts of 40 feet and 60 feet at side slopes of 2 to 1 have been examined. This does not apply exactly to the final excavation but it enables us to define the general overall stability of the cuttings.

It may be seen that the cutting 40 feet deep will have a factor of safety F greater than 1.5 for the undrained case. This we can typify as during and shortly following the construction period. However with time, F will drop as illustrated on the figure. As the cutting will be for a large part in the desiccated crust the shear strength parameters of the clay have been taken as $c' = 350$ lbs/sq.ft. and $\phi' = 20^\circ$. Using the effective shear strength parameters and considering the probable ground water level which will take place when the cutting has been open for some time, F is computed to be slightly greater than 1.1. We consider that in this preliminary stage of design it can be assumed that a 40 foot cutting will be satisfactory.

When the cutting is lowered, however, the drop in F is most significant. There is no doubt that with a computed value of F of about 0.6 the cutting will be unstable unless support is provided at the base of the excavation. It is proposed by Fenco to provide lateral restraint at the base of the deep cutting through the use of steel sheet piling driven to the till stratum. In this case resistance to sliding will be offered by the fixity of the sheeting in the till and the rigidity of the sheet piling against bending. For this case, then, the problem is one of the long term stability of a strutted excavation. This has been examined using the general method adopted by D.J. Henkel, 1956, considering the effective stress parameters* of the clay and the assumed ground water level. The general approach is illustrated on figure 2. Preliminary computations indicate that stability is possible provided that:

a. The sheeting is adequately strutted or braced above elevation 519,

b. Fixity is obtained for the toe of the sheet piling in the till,

c. Sheeting is capable of resisting part of the earth pressure through bending. Otherwise the piles will fail by bending and the bottom of the approach cut will heave.

The most critical stage, as far as bending in the sheet piling is concerned, is when the excavation has been lowered to elevation 519, tremie concrete poured to elevation 523, and the excavation pumped dry. Preliminary computations have been carried out for this stage. It has been assumed that

*For this case, c' is taken to be 150 psf and Ø' to be 20°.

the sheeting has been adequately supported at about elevation 540 and at elevation 520 by the tremie concrete seal. (We understand support will be provided at elevation 540 by rock anchor ties). It is estimated that a penetration of 10 feet in the till should provide a reasonable degree of fixity. (The till is about 10 feet thick at the deepest point of the approach cuts, DHO. Dwg. 61-F-11C).

The preliminary computations show that the tremie concrete and rock anchor ties may have to resist a load of about 50 to 60 kips per lineal foot of cutting. The bending moment on the buried portion of the sheet piling could be that induced by a load of about 50 kips per lineal foot of cutting. This would give a maximum bending moment, assuming fixity in till, of about 100 foot kips in the sheet piles which is approximately within the capacity of Z.P.38 piles.

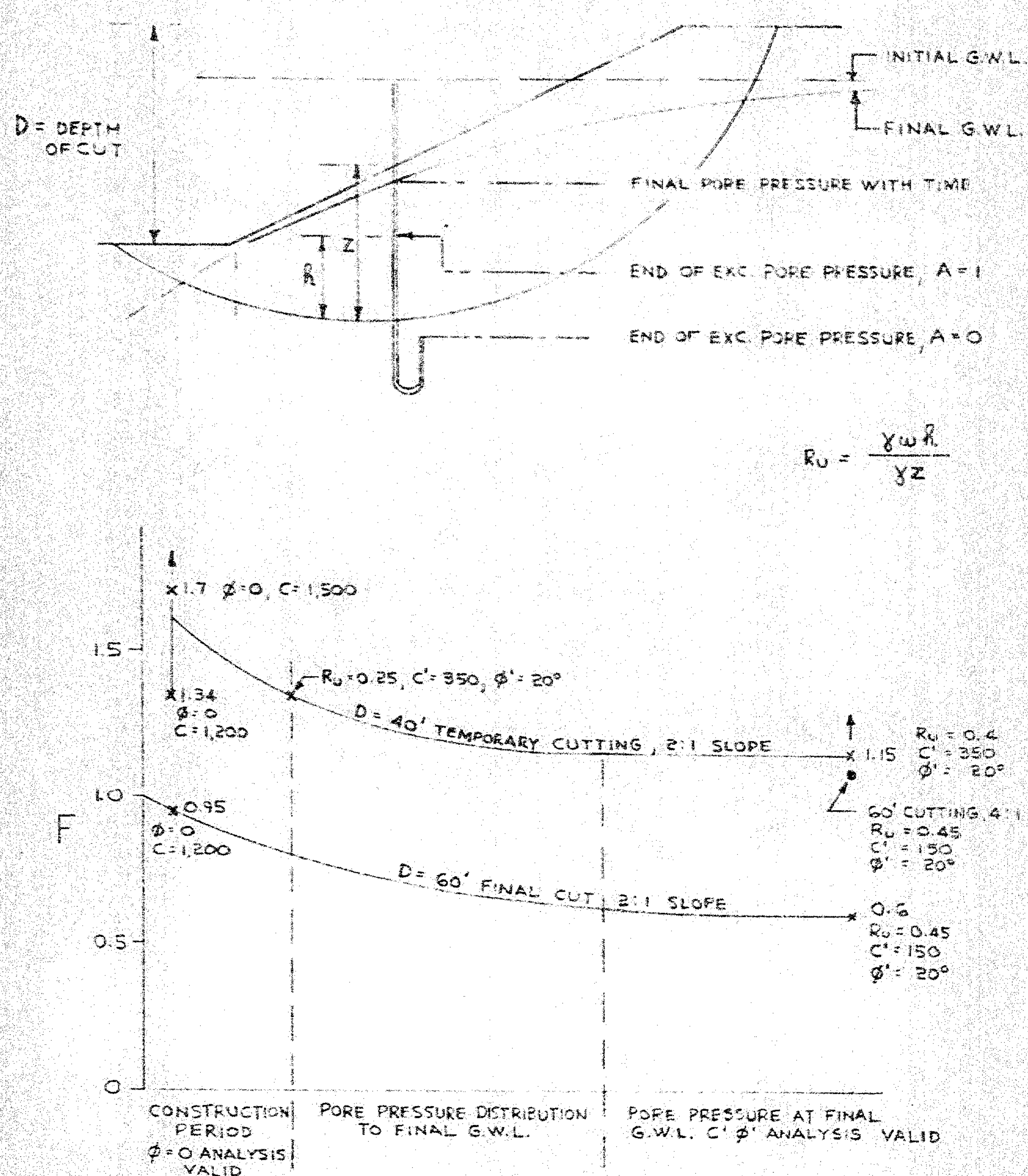
Consequently, although various stages of construction loading have to be checked in detail, the scheme proposed appears to be feasible and warrants further study. We understand that this is now being carried out by you. When laboratory testing is complete and the shear strength parameters of the clay defined, our final conclusions will be presented.

VM/jb
6108

V. Milligan, P. Eng.

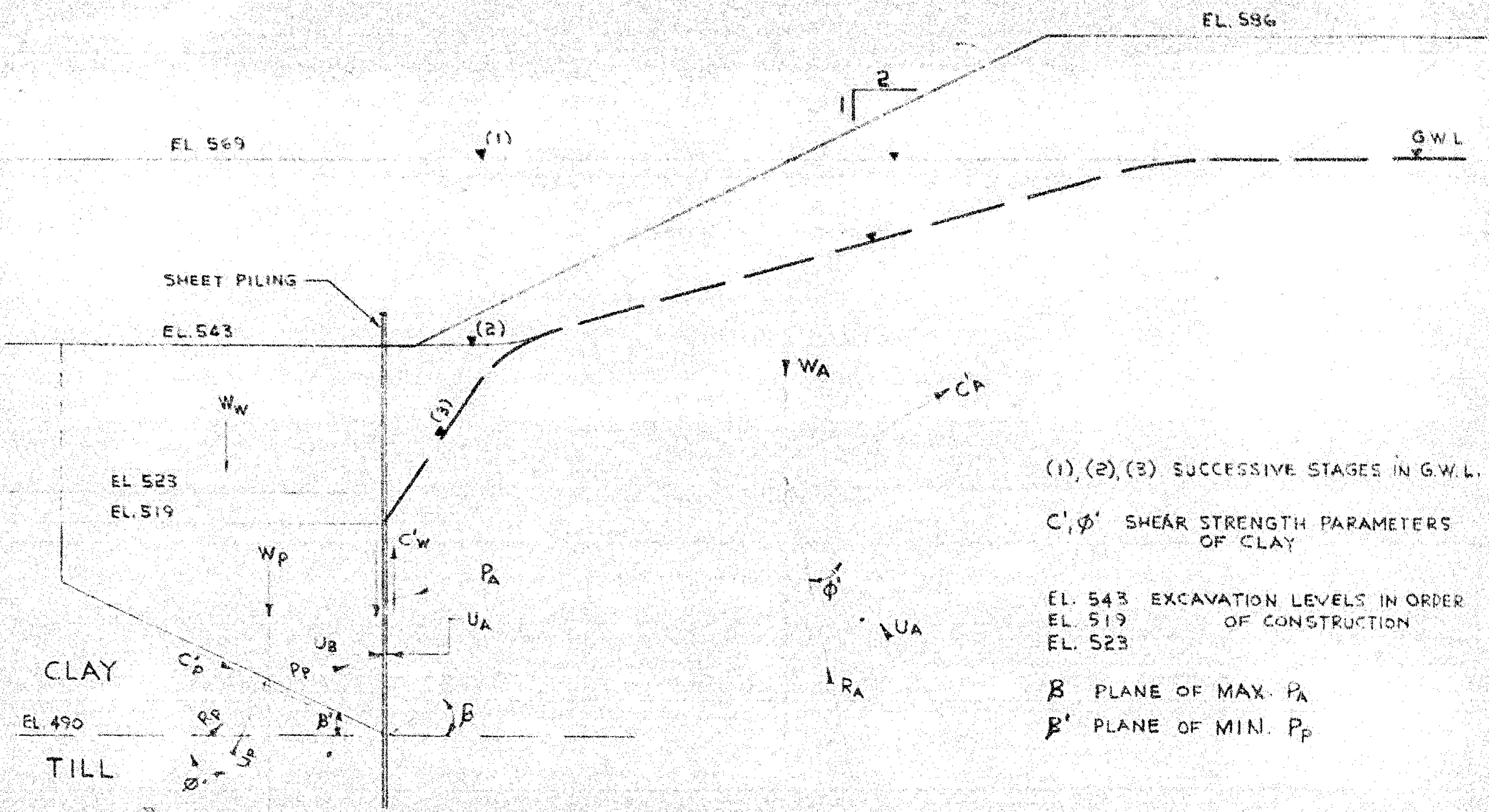
SUMMARY — STABILITY ANALYSES

FIGURE 1



GOLDER & ASSOCIATES

EARTH PRESSURE DISTRIBUTION - CONSTRUCTION STAGES



GOLDER & ASSOCIATES

FIGURE 2

Mr. A. M. Toye,
Bridge Engineer.
Materials and Research Section.

December 20, 1960.
FOUNDATION INVESTIGATION REPORT
by: H. Q. Golder & Associates, Ltd.

Attention: Mr. S. McCombie.

~~W.P. 139-61~~
W.P. 130-61

Re: Proposed Welland Canal Tunnel, Welland, Ontario, District #4.

Submitted herewith, for your information, is the report on soil conditions at the above noted tunnel location.

The details presented in this report have been discussed with the Consultants, H. Q. Golder & Associates, and we are in agreement with the facts presented.

The Consultants retained to study the feasibility of the tunnel at this location, Sir Alexander Gibb & Partners, have discussed these test results on soil conditions in detail, with Dr. Golder & Associates, and have based their preliminary studies on the soil conditions presented in this report.

L. G. Soderman
L. G. Soderman,
PRINCIPAL FOUNDATION ENGINEER

LGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
D. G. Lumsay
I. C. Campbell
R. E. Richardson
T. J. Kovich

Foundations Office
Gen. Files.

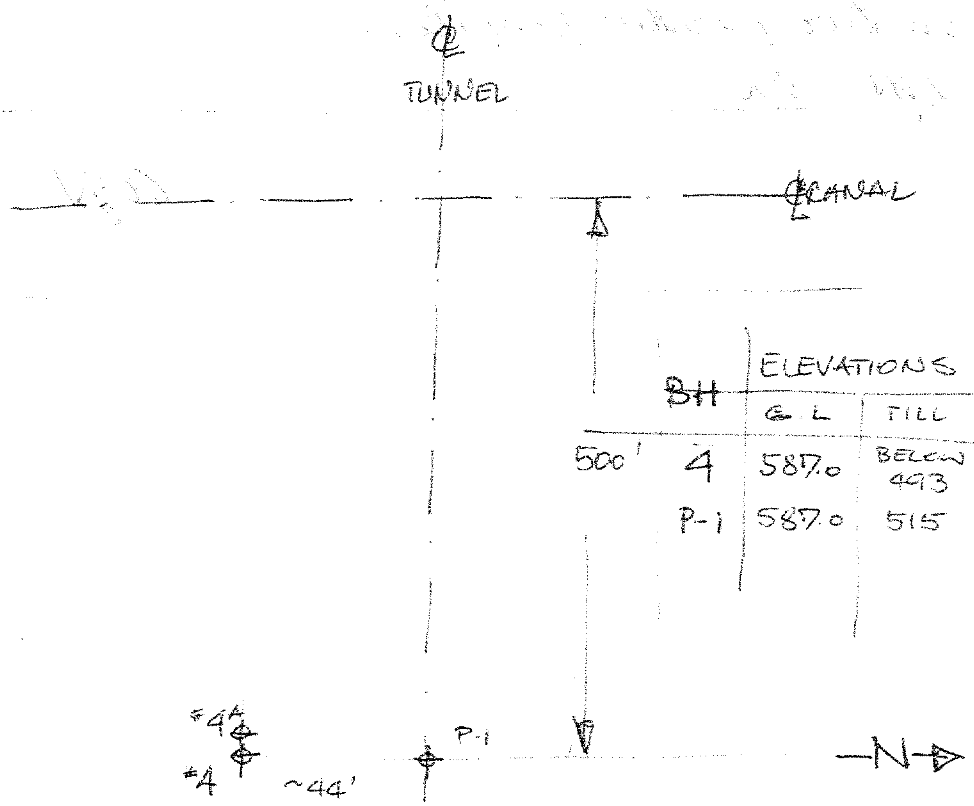
OVER

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION
FORM OB-ML-30A
FEB. 1961

MADE BY ______ DATE March 1, 1961 JOB No. ______
CHECKED BY ______ DATE ______ SECTION ______
SHEET No. ______

Field Visit March 1st 1961

Welland Tunnel Project



BH	ELEVATIONS G.L	TILL
4	587.0	BELOW 493
P-1	587.0	515

BOREHOLE 4A - VANE TESTS ONLY

DEPTH	C lb/sq ft
20.0	1,580
30.0	1,400
40.0	1,980
50.0	1,280
60.0	1,360
65.0	1,280
70.0	1,300
75.0	1,360
80.0	400
85.0	80

PROBLEM

could the BH north of ℄ of the tunnel be moved closer to the ℄ of the canal. Reason: chosen location is on the lawn of a house. New location ~50' towards ℄ of canal is on waste land.

Larry OK.

Because of the different till elevations it is suggested that cone tests be carried out on two parallel profiles.

Lgs. OK.

AGT

H. Q. GOLDER & ASSOCIATES LTD.
CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
V. MILLIGAN

2446A BLOOR ST. W.
TORONTO 9
RO. 7-9201

REPORT
TO
DEPARTMENT OF HIGHWAYS, ONTARIO
ON
PROPOSED WELLAND CANAL TUNNEL
WELLAND, ONTARIO

Distribution:

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

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

December, 1960

6022

ABSTRACT

The results of a preliminary investigation, the purpose of which was to determine soil conditions at the site of a proposed tunnel in Welland, Ontario, are reported. It was found that, following minor thicknesses of topsoil and fill, the site was underlain by about 80 to 90 feet of hard to firm silty clay followed by a thin layer of glacial till and then dolomite bedrock. The hard portion of the silty clay stratum is in the upper 20 to 30 feet, where undrained shear strengths range up to about 6,000 pounds per square foot; below this the shear strengths are relatively constant with an average of about 750 pounds per square foot. Groundwater levels established at the borehole locations ranged from 3 to 7 feet above normal canal level.

INDEX

INTRODUCTION

H. Q. Golder and Associates Ltd. have been retained by the Department of Highways, Ontario under the terms of a letter of authorization dated October 7th, 1960 to carry out a preliminary investigation at the site of a proposed tunnel in Welland, Ontario. The purpose of the investigation was to determine the soil and groundwater conditions at the site.

PROCEDURE

The field work for the investigation was commenced on October 11th, and was completed on October 28th, 1960. Three boreholes were put down in BX size to depths of 93 to 123 feet using a standard skid-mounted machine drillrig. Soil samples were taken with standard 2-inch drive open and thin-walled samplers, and bedrock was cored in AX size in two holes, Piezometers for the determination of groundwater conditions were installed successfully in two boreholes, but one was destroyed by children before observations could be recorded. The locations of the boreholes together with the inferred soil stratigraphy at the site are shown on Drawing 1. Detailed logs of each borehole are given on the Records of Boreholes.

Following the field work, the soil samples were returned to the laboratory for testing. The results of the testing are plotted on the Records of Boreholes and on the enclosed figures. Samples remaining after testing will be stored until May 1st, 1961 at which time you will be notified regarding their disposal.

All elevations used in the report were supplied by the City of Welland and are referred to the Welland Canal Datum.

SITE TOPOGRAPHY AND GEOLOGY

The site is located in the physiographic region known as the Haldimand Clay Plain, which is, in this area, relatively flat. The area is known to be underlain by stratified silt and clay deposits of glacial Lake Warren which are of the order of 100 feet thick. The silt and clay are underlain by a thin layer of glacial drift followed by shales and dolomites of the Paleozoic era.

SOIL CONDITIONS

The following soil strata have been determined from the results of the investigation:

Fill

A stratum of brown to reddish-brown fill was encountered in Borehole 3 on the east bank of the canal beneath a thin layer of topsoil. This fill, which may have been placed during the construction of the canal, was about 17 feet thick and was composed primarily of silty clay with some sand, gravel and organic matter. Two water contents of about 20 per cent were measured on samples from the stratum.

Penetration of the stratum was difficult with standard

Fill (continued)

penetration resistances ranging from 15 to greater than 100 blows per foot. Based on these and visual observation of the samples, the consistency of the stratum is estimated to be hard.

A thin layer of cinder fill was also encountered in Borehole 2.

Silty Clay

Beneath the topsoil in Borehole 1 and immediately underlying the fill in Boreholes 2 and 3, was a stratum of brown to reddish-brown silty clay extending to depths of 92 to 96 feet. Included in the matrix of the silty clay throughout its depth were small amounts of granular material ranging in size from sand to sub-angular pebbles of 1½ inch diameter.

In the majority of samples only faint indications of stratification were apparent, implying that the individual layers are of very similar composition. However, occasional zones of definitely layered material were encountered, usually in the lower half of the stratum. In addition to the brown to reddish silty clay, which forms the bulk of the stratum, these layers, which ranged from 1/16 to 1 inch in thickness, were composed of red or grey silty clay. Occasional thin seams of silty sand were also encountered; their thickness never exceeded ½ inch and was usually less than 1/16.

Silty Clay (continued)

Samples from the upper 20 to 30 feet of the stratum frequently exhibited thin fissures containing grey silt which is indicative of weathering and/or desiccation.

A total of 50 sets of Atterberg limits were carried out on samples from this stratum. The liquid limits ranged from about 20 to 60 with an average of 36, and plasticity indices ranged from about 3 to 33, with an average of about 16. The higher liquid limits of the order of 50 to 60 with plasticity indices of the order of 30 are generally representative of the red and grey layers discussed above. The liquid limits of the order of 30 to 40 are representative of the brown silty clay, with the limits from 40 to 50 indicative of thinly layered samples where it was not practical to separate the layers.

The limits are plotted, together with natural water contents, on the Records of Boreholes. These show that in the upper 20 to 30 feet of the stratum, the natural moisture contents are very close to the plastic limit, and that below these depths there is a definite trend of increase in moisture content. This is illustrated more graphically by a plot of liquidity index versus elevation for Borehole 1, given as Figure 1. Also, in the lower part of the stratum, the average liquidity index is about 0.6. Based on accumulated experience

Silty Clay (continued)

with the clays of Southern Ontario, it is considered that this may be indicative of a slight over-consolidation in the stratum.

Another observation which is confirmed by the plotted limits and water contents is that in the lower 10 to 15 feet of the stratum, the material becomes very silty. This is indicated by the low limits and the sharp reduction in the natural water contents.

A total of 46 wet unit weights were measured on samples from this stratum. These ranged from 107 to 134 pounds per cubic foot with an average of 125 pounds per cubic foot.

Grain size distributions were determined on samples of the clay and are shown on Figures 2, 3 and 4. These indicate the clay to be generally very silty. In some cases, particularly where these curves were obtained for relatively highly plastic materials, it is considered that the high silt percentage may be due to flocculation; during testing, dispersion of some samples was found to be difficult, probably due to the presence of iron oxides as cementing agents.

A total of 47 undrained triaxial compression tests were carried out on samples of the clay. The results of these are plotted on the Records of Boreholes, and are summarized on Figure 6, which is a plot of undrained shear strength versus

Silty Clay (continued)

elevation. Typical stress-strain curves are shown on Figure 5. Again, the influence of desiccation or weathering in the upper portion of the stratum is indicated by the fact that above elevation 550 the shear strengths are generally high, ranging up to about 6,000 pounds per square foot. Below elevation 550 the shear strengths average about 750 pounds per square foot with no appreciable increase with depth. About 10 strengths lower than 750 pounds per square foot were recorded, but it is considered that most of these samples were disturbed.

From the results of the strength tests, the consistency of the clay stratum varies from hard at the top to firm below elevation 550.

Clayey Silt, Sand, and Gravel

A stratum of brown clayey silt, sand, and gravel, which is probably a glacial till, was found to underlie the silty clay in Boreholes 1 and 3. A boulder of about 1 foot diameter was also encountered within the stratum in Borehole 1. The thickness of the stratum was about 11 feet in Borehole 1 and 5 feet in Borehole 3.

One liquid limit of 16 and a corresponding plastic limit of 12 were obtained from a sample in this stratum. A grain size distribution curve is shown on Figure 4 (BH1,Sa48).

Clayey Silt, Sand, and Gravel (continued)

Two wet unit weights of 136 and 148 pounds per square foot were measured.

Two triaxial compression tests were carried out and shear strengths of 1,150 and 5,190 pounds per square foot were obtained.

The consistency of the till, based on the above shear strengths and the difficulty encountered in penetrating the stratum, is estimated to be hard.

Dolomite Bedrock

Bedrock was proven for 10 feet in Borehole 1 and about 22 feet in Borehole 3. It was found to be composed of a dark grey dolomite of the Guelph formation.

Groundwater Conditions

Water levels were measured in the boreholes at the time of the investigation, and these together with observations made in a piezometer installed in Borehole 3 are considered to be a valid indication of groundwater conditions at the site. These observations indicate that groundwater levels ranged from about 572 at Boreholes 1 and 3 on the banks of the canal to about 576 at Borehole 2. Canal level was at 569.5 during the investigation.

A. A. Gass, P. Eng.

AAG:IMB
6022
December, 1960

V. Milligan, P. Eng.

GOLDER & ASSOCIATES

LIST OF STANDARD ABBREVIATIONS

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

SAMPLE TYPES

A.S. – Auger Sample	R.C. – Rock Core
C.S. – Chunk Sample	S.T. – Slotted Tube
D.O. – Drive Open	T.O. – Thin-walled, Open
D.S. – Denison Type Sample	T.P. – Thin-walled, Piston
F.S. – Foil Sample	W.S. – Wash Sample

PENETRATION RESISTANCES

Dynamic Penetration Resistance – The energy required to drive a 2 inch diameter, 60 degree cone attached to the end of the drilling rods into the ground: expressed in blows per foot, where each blow represents 4,200 inch-pounds of energy.

Standard Penetration Resistance, N – The number of blows by a 140 pound hammer dropped 30 inches required to drive a 2 inch drive open sampler one foot into the ground.

Sampler advanced by static weight	– weight, hammer	– Wh
Sampler advanced by pressure	– pressure, hydraulic	– Ph
Sampler advanced by pressure	– pressure, manual	– Pm

SOIL DESCRIPTION

The standard terminology for the descriptions of the relative density of cohesionless soils and the consistency of cohesive soils is as follows:

Relative Density	N, Blows/ft.	Consistency	c, lb/sq. ft.
Very Loose	0 to 4	Very Soft	Less than 250
Loose	4 to 10	Soft	250 to 500
Compact	10 to 30	Firm	500 to 1,000
Dense	30 to 50	Stiff	1,000 to 2,000
Very Dense	over 50	Very Stiff	2,000 to 4,000
		Hard	over 4,000

SOIL TESTS

C – Consolidation Test	Q – Undrained Triaxial
H – Hydrometer Analysis	Qc – Consolidated Undrained Triaxial
M – Sieve Analysis	S – Drained Triaxial
MH – Combined Analysis, Sieve and Hydrometer	U – Unconfined Compression
	V – Field Vane Test

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

SOIL PROPERTIES

γ – Total Unit Weight	K – Coefficient of Permeability
γd – Dry Unit Weight	c – Undrained Shear Strength (½ Compressive Strength)
γb – Submerged Unit Weight	St – Sensitivity
L_L – Liquid Limit	$\emptyset'$ – Effective Angle of Shearing Resistance
P_L – Plastic Limit	c' – Effective Cohesion Intercept
W – Natural Water Content	Cc – Compression Index
G – Specific Gravity	Cv – Coefficient of Consolidation
e – Void Ratio	

GOLDER & ASSOCIATES

PROJECT No. 6022

RECORD OF BOREHOLE 1

LOCATION SEE DRWG. No 1 BORING DATE OCT. 11 - 17, 1960 DATUM WELLAND CANAL

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER BX CASING

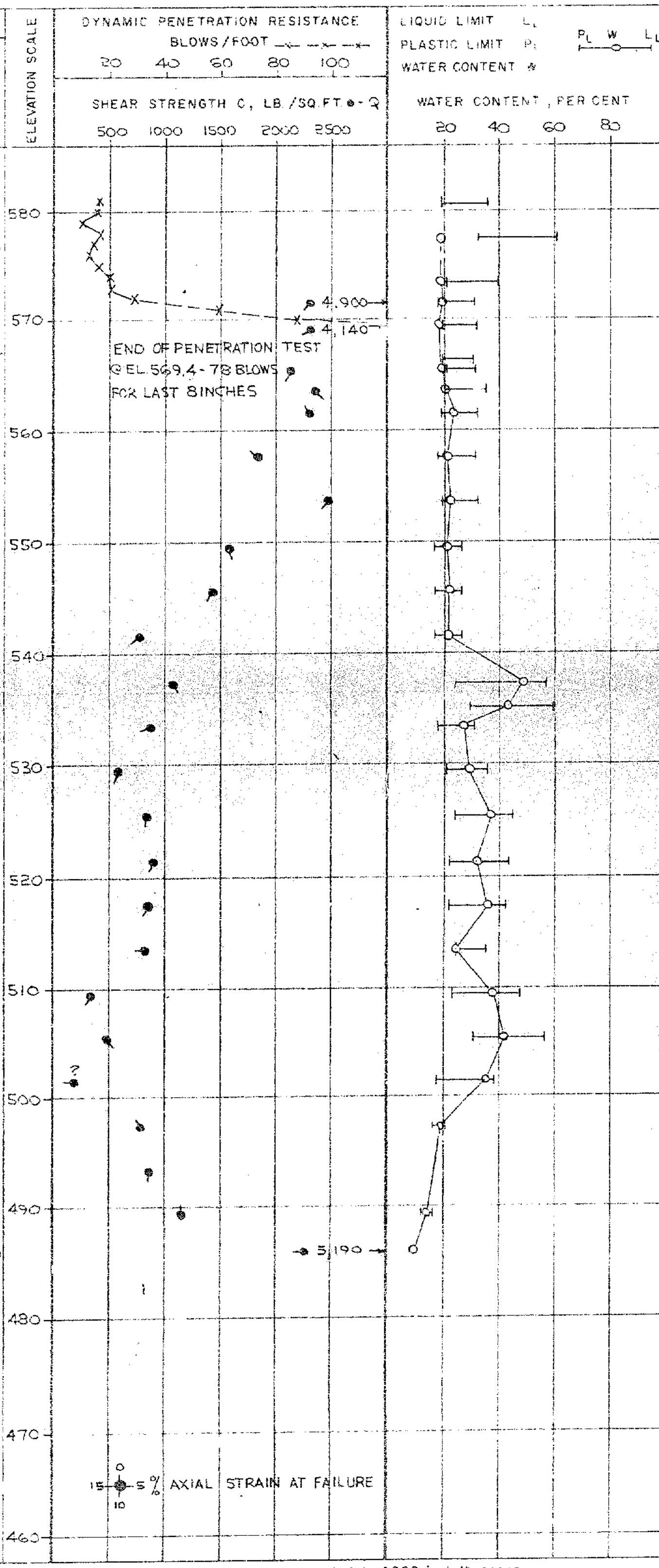
SAMPLER HAMMER WEIGHT 140 LB DROP 30 INCHES PEN TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE

ELEVN. / DEPTH	DESCRIPTION
582.0	GROUND LEVEL
0.5	TOPSOIL
	HARD TO FIRM BROWN TO REDDISH - BROWN SILTY CLAY WITH PEBBLES UP TO 1½" DIAMETER; OCCASIONAL RED OR GREY CLAY LAYERS 1/16" TO 3/4" THICK; BECOMING VERY SILTY BELOW 85' DEPTH.
502.0 / 80.0	
489.7 / 92.3	HARD BROWN CLAYEY SILT, SAND, GRAVEL AND BOULDERS
479.0 / 103.0	DARK GREY DOLOMITE BEDROCK
469.0 / 113.0	END OF HOLE

SAMPLES

NUMBER	TYPE	BLOWS / FT
1	2" D.O.	19
2	T.O.	26
3	D.O.	60
4	T.O.	47
5	"	45
6	"	34
7	D.O.	
8	T.O.	27
9	"	32
10	"	34
11	"	31
12	"	27
13	"	24
14	"	41
15	"	PM
16	"	"
17	"	"
18	"	"
19	"	"
20	"	"
21	"	"
22	"	"
23	"	"
24	"	"
25	"	"
26	"	"
27	"	"
28	"	"
29	"	"
30	"	"
31	"	"
32	"	"
33	"	"
34	"	"
35	"	"
36	"	"
37	"	"
38	"	"
39	"	"
40	"	"
41	"	"
42	"	"
43	"	"
44	"	"
45	"	22
46	"	80
47	"	77
48	D.O.	100
50	AXT R.C.	—
51	AXT R.C.	—
52	AXT R.C.	—



Dynamic penetration resistance converted to 4200 inch lb. energy

REMARKS

GROUND W.L. IN HOLE @ EL. 572, OCT. 15, 1960 @ 8.00 A.M. HOLE @ 91' CASED TO 34'

H
γ = 134 P.C.F.
γ = 134 P.C.F.
γ = 127 P.C.F.
γ = 127 P.C.F.
γ = 129 P.C.F.
γ = 129 P.C.F.
H
γ = 130 P.C.F.
γ = 131 P.C.F.
γ = 130 P.C.F.
γ = 130 P.C.F.
H (2)
γ = 107 P.C.F.
H
γ = 123 P.C.F.
H
γ = 121 P.C.F.
γ = 115 P.C.F.
H
γ = 119 P.C.F.
γ = 118 P.C.F.
γ = 123 P.C.F.
H
γ = 112 P.C.F.
H
γ = 110 P.C.F.
γ = 126 P.C.F.
γ = 122 P.C.F.
H
γ = 125 P.C.F.
γ = 136 P.C.F.
γ = 148 P.C.F.
MH

VERTICAL SCALE
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED

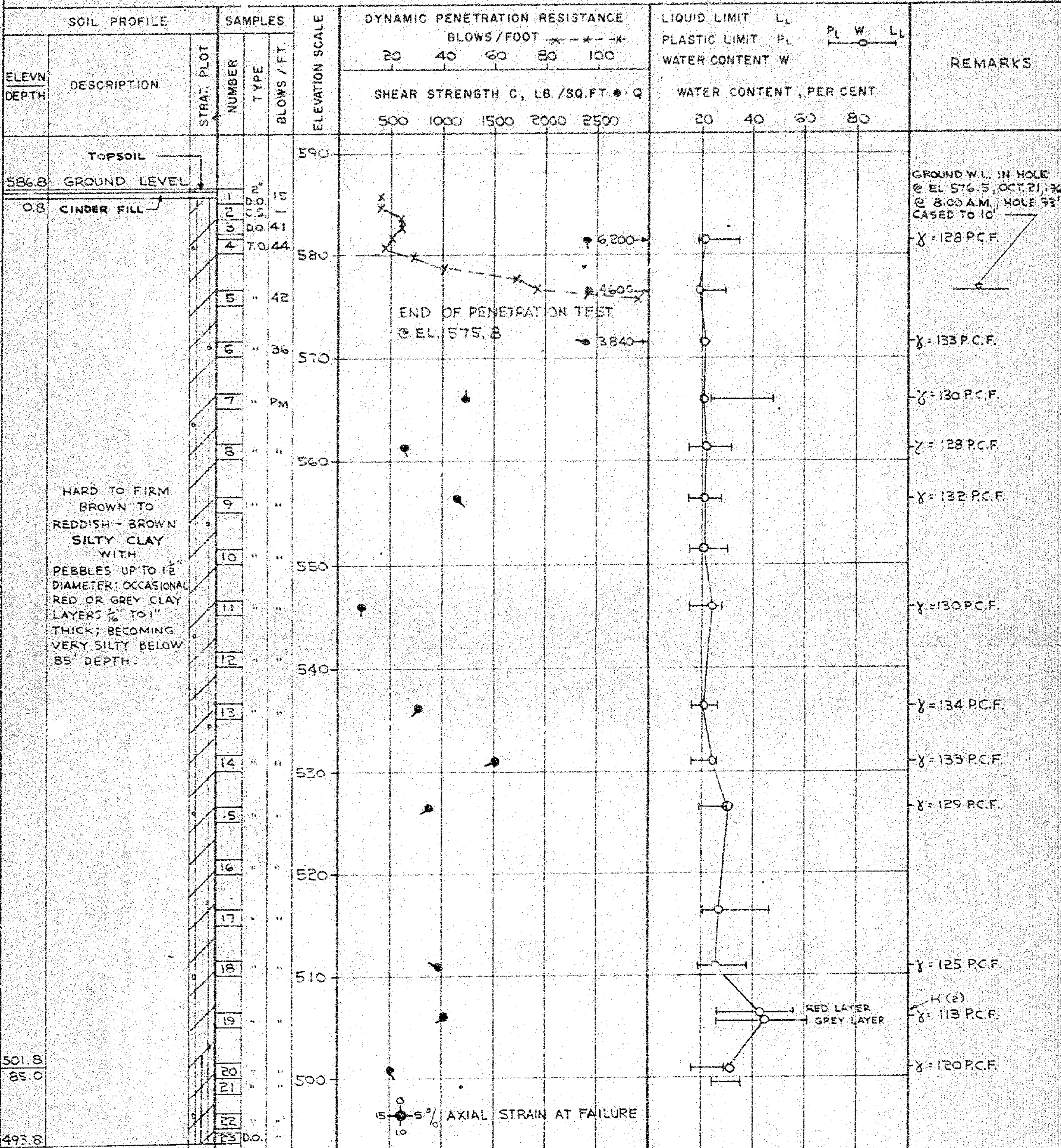
PROJECT No. 6022

RECORD OF BOREHOLE 2

LOCATION SEE DRWG. No. 1 BORING DATE OCT. 18 - 21, 1960 DATUM WELLAND CANAL

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER BX CASING

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



Dynamic penetration resistance converted to 4200 inch lb energy

VERTICAL SCALE
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED [illegible]

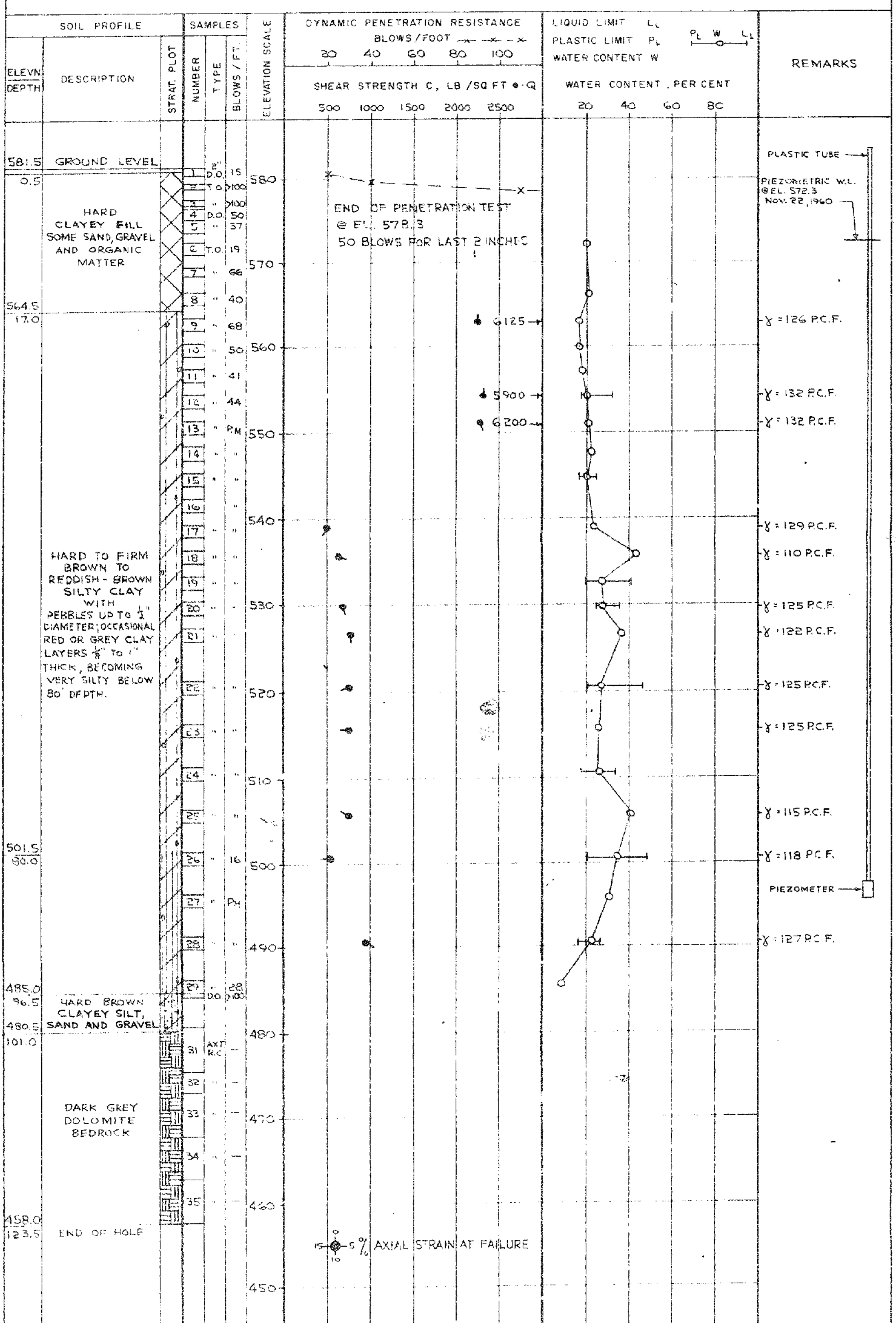
PROJECT No. 6055

RECORD OF BOREHOLE 3

LOCATION SEE DRWG. No. 1 BORING DATE OCT. 21-27, 1960 DATUM WEL. AND CANAL

BOREHOLE TYPE WASH BORING BOREHOLE DIAMETER BX CASING

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



Dynamic penetration resistance converted to 4200 inch lb energy

VERTICAL SCALE 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

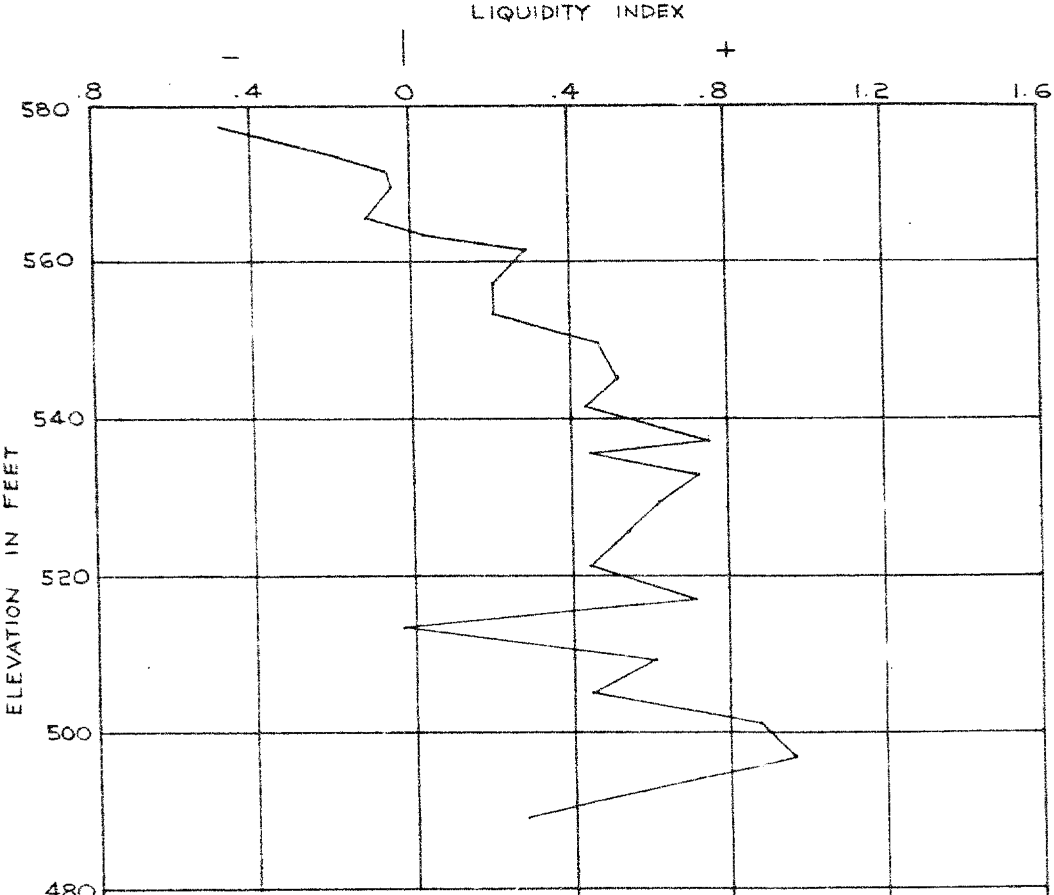
DRAWN J.A.

CHECKED

LIQUIDITY INDEX VS ELEVATION

BOREHOLE 1

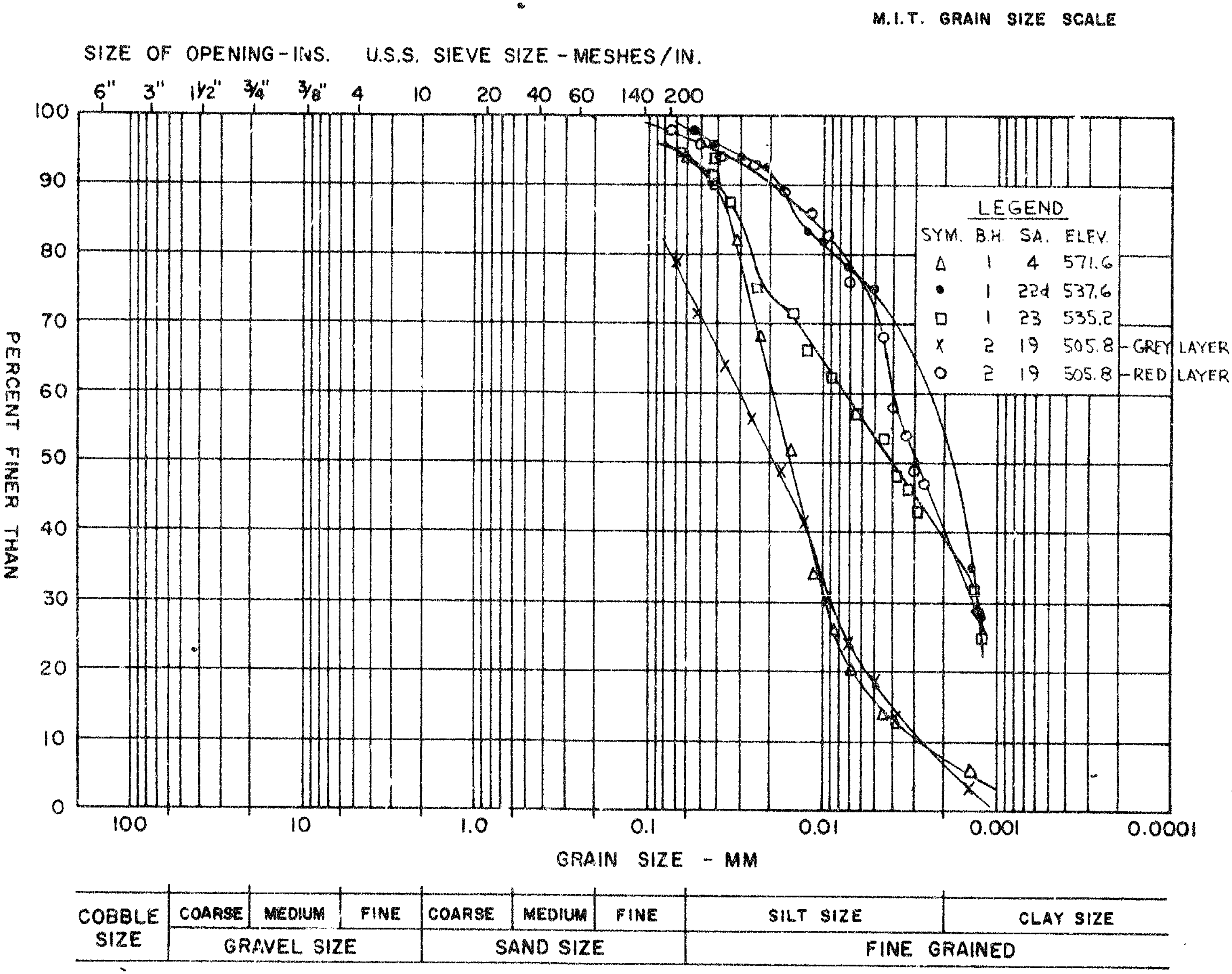
FIGURE 1



LIQUIDITY INDEX = $\frac{W - P_L}{L_L - P_L}$

GOLDER & ASSOCIATES

PROJECT No. 6022

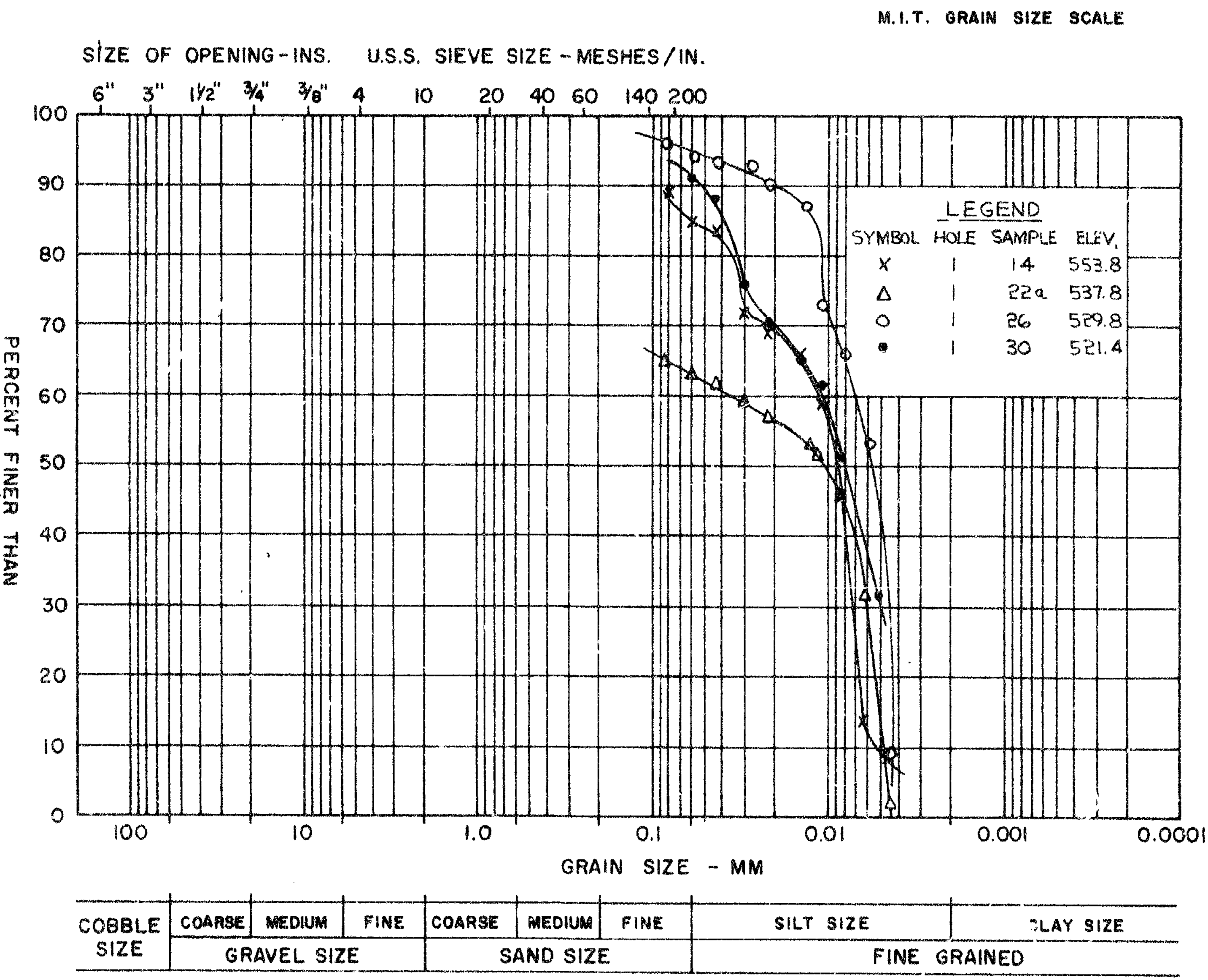


GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

FIGURE 2

PROJECT No. 6022

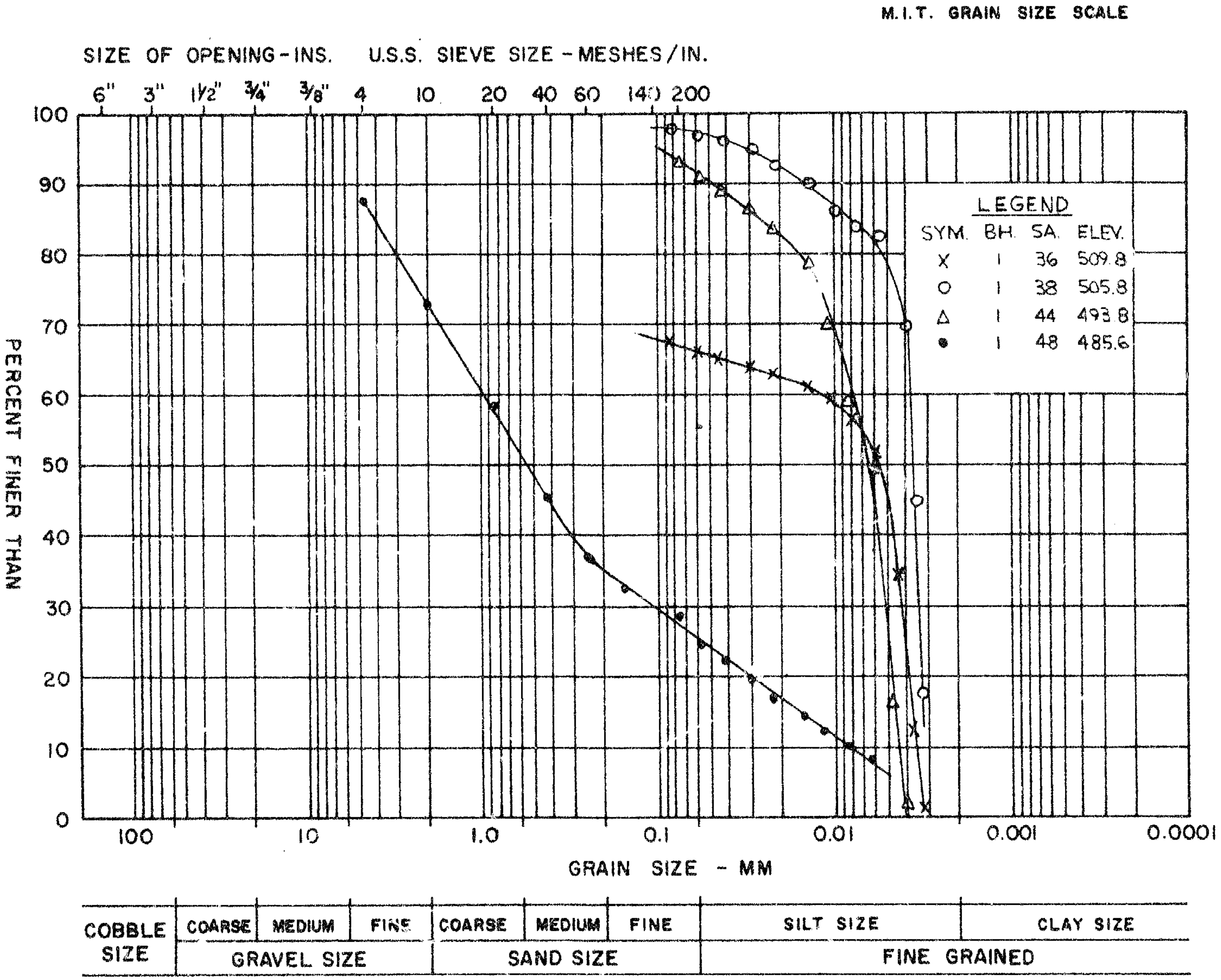


GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

FIGURE 3

PROJECT No. 6022



GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

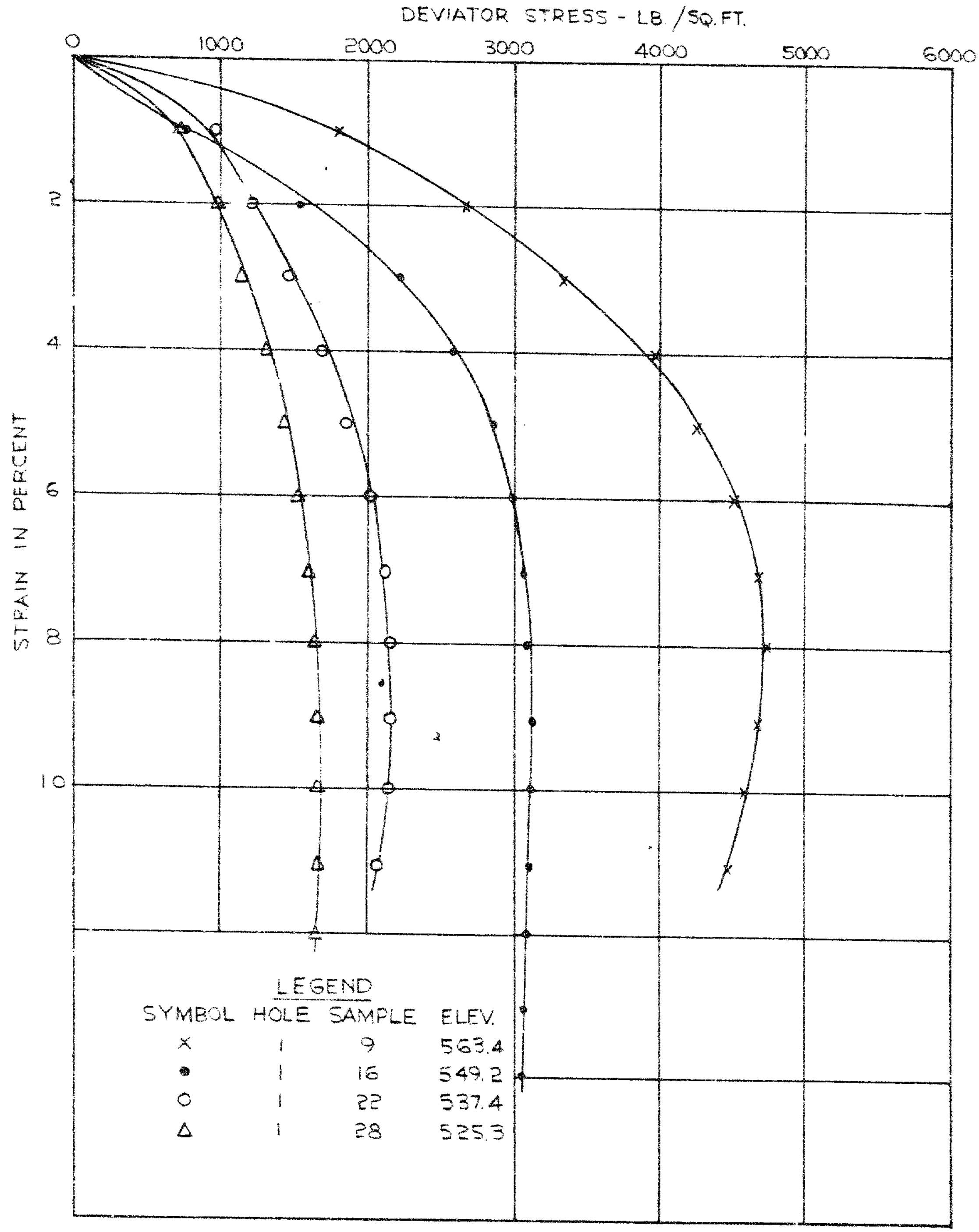
FIGURE 4

PROJECT No. 6022

UNDRAINED TRIAXIAL COMPRESSION TESTS

TYPICAL STRESS-STRAIN CURVES

FIGURE 5

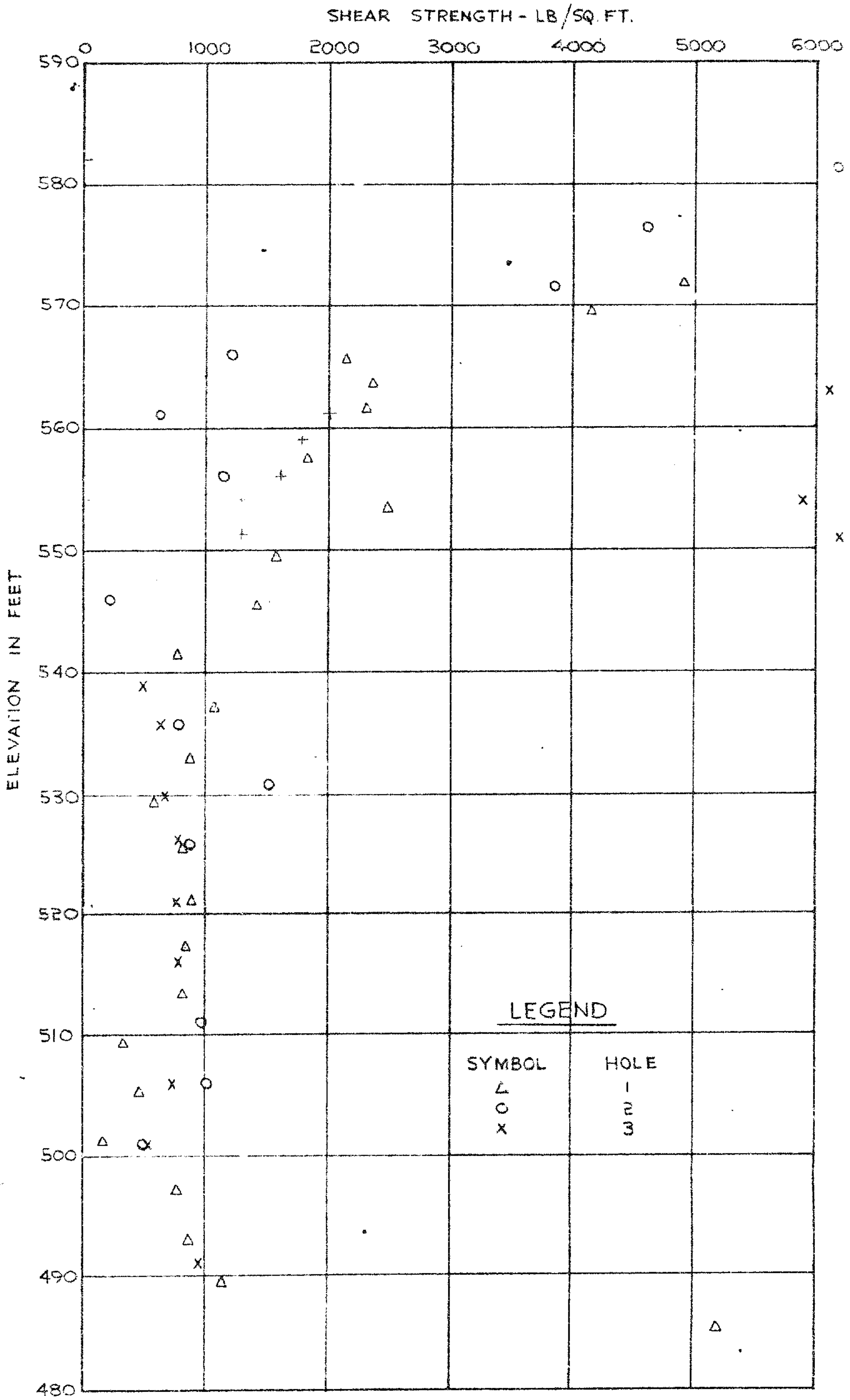


GOLDER & ASSOCIATES

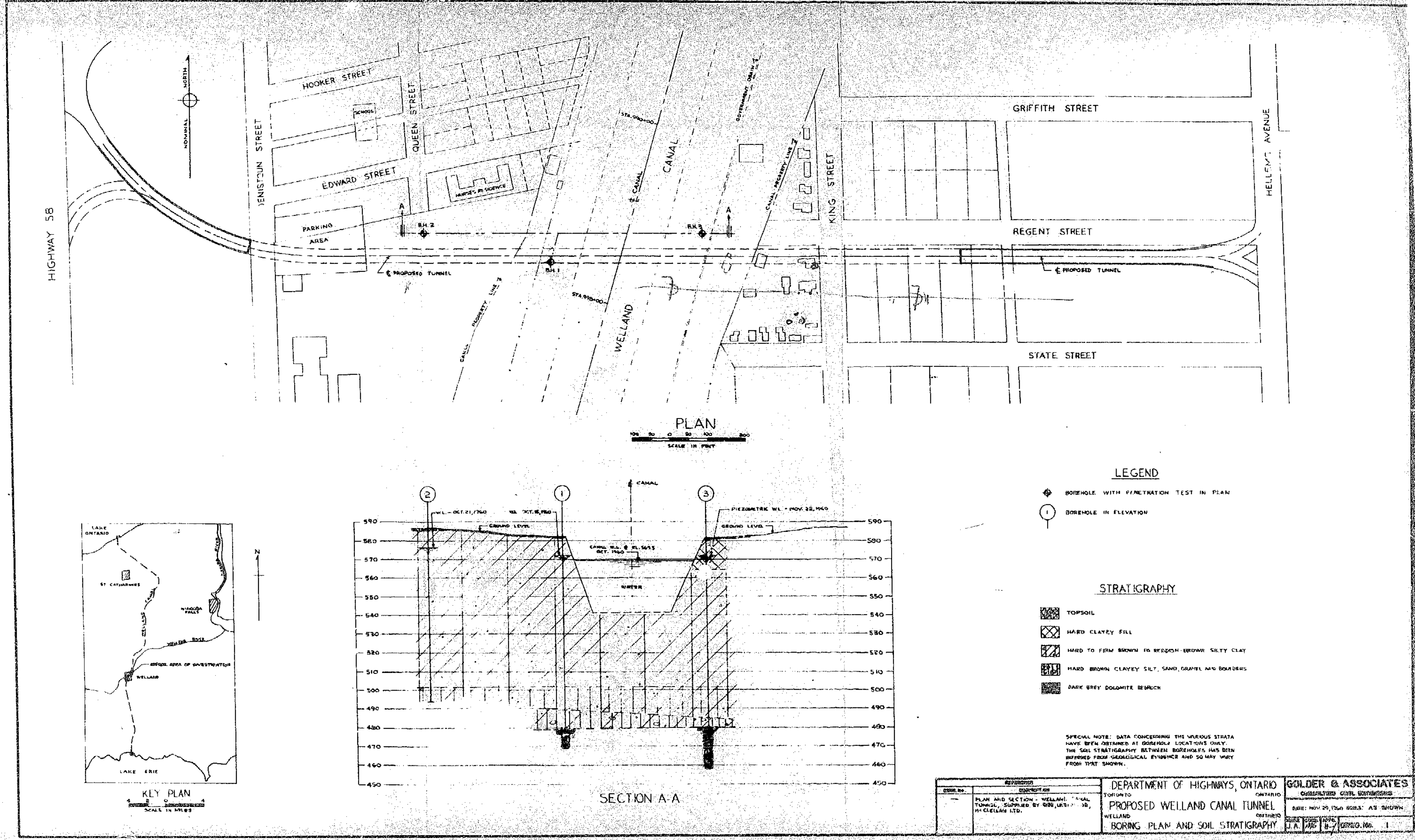
PROJECT NO 6022

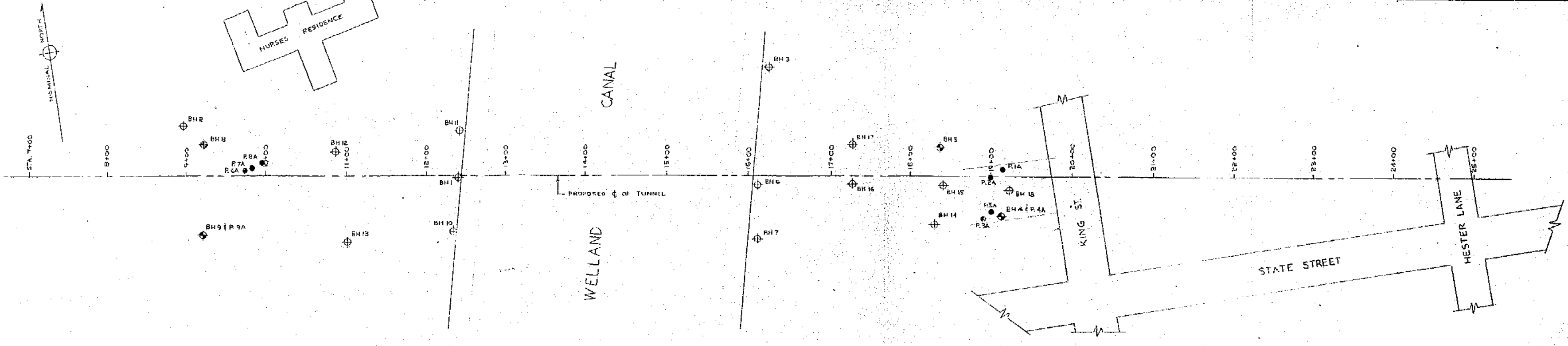
UNDRAINED TRIAXIAL COMPRESSION TESTS
SHEAR STRENGTH VS ELEVATION

FIGURE 6

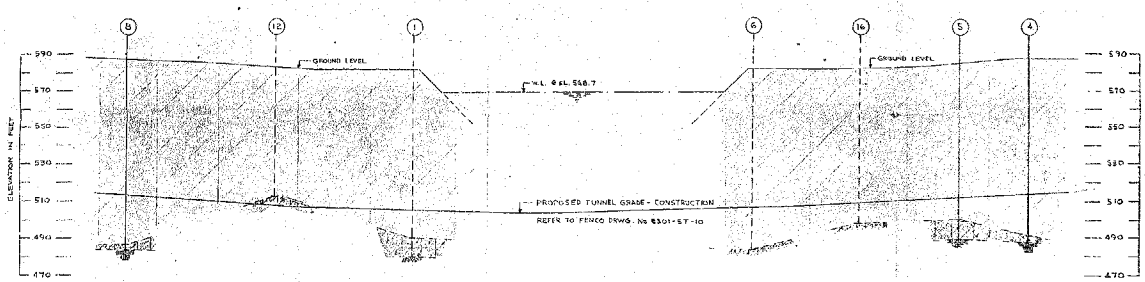


GOLDER & ASSOCIATES

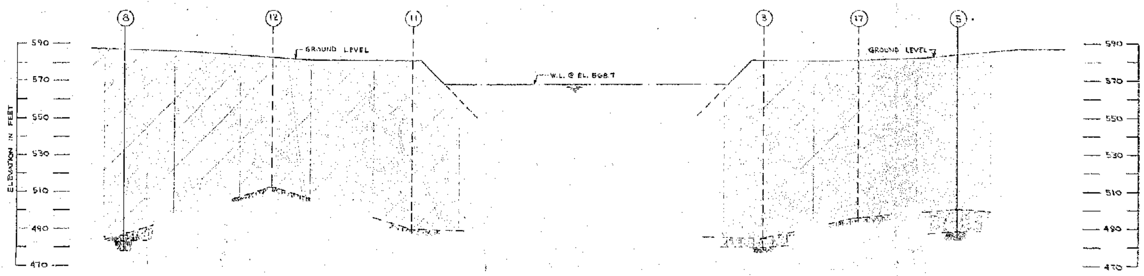

HOOKER STREET
SCHOOL
QUEEN STREET
JENISTOUN STREET
EDWARD STREET
NURSES RESIDENCE
NOMINAL NORTH
PARKING AREA
HIGHWAY 58
B.H. 2
B.H. 3
B.H. 1
A
℄ PROPOSED TUNNEL
CANAL PROPERTY LINE
WELLAND CANAL
℄ CANAL
GOVERNMENT
KING STREET
GRIFFITH STREET
REGENT STREET
STATE STREET
HELLEMS AVENUE
PLAN
SCALE IN FEET
℄ CANAL
GROUND LEVEL
WATER
PIEZOMETRIC W.L. - NOV. 22, 1960
SECTION A-A
590
580
570
560
550
540
530
520
510
500
490
480
470
460
450
LAKE ONTARIO
ST. CATHARINES
WELLAND CANAL
NIAGARA FALLS
WELLAND RIVER
WELLAND
LAKE ERIE
KEY PLAN
SCALE IN MILES
LEGEND
BOREHOLE WITH PENETRATION TEST IN PLAN
BOREHOLE IN ELEVATION
STRATIGRAPHY
TOPSOIL
HARD CLAYEY FILL
HARD TO FIRM BROWN TO REDDISH-BROWN SILTY CLAY
HARD BROWN CLAYEY SILT, SAND, GRAVEL AND BOULDERS
DARK GREY DOLOMITE BEDROCK
SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.
PLAN AND SECTION - WELLAND CANAL TUNNEL, SUPPLIED BY
McCLELLAN LTD.
DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO
PROPOSED WELLAND CANAL TUNNEL
WELLAND ONTARIO
BORING PLAN AND SOIL STRATIGRAPHY
GOLDER & ASSOCIATES
SCALE: AS SHOWN
J.A.
DWG. No. 1



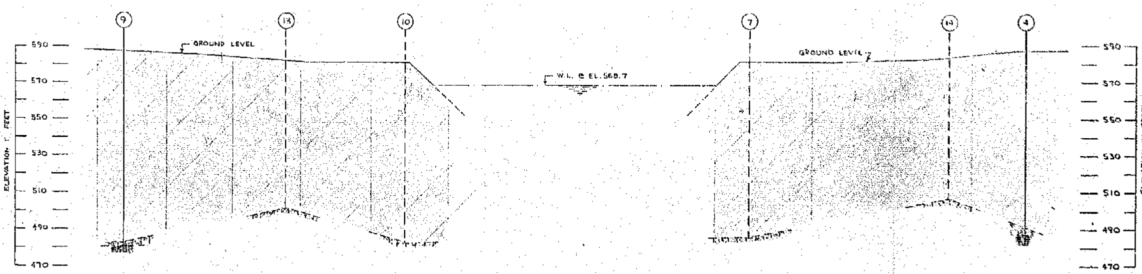
PLAN
SCALE: 1" TO 30'-0"



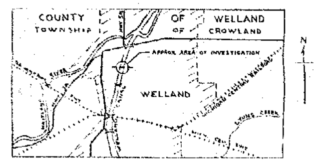
SCHEMATIC SECTION ALONG PROPOSED ¢
SCALE: 1" TO 30'-0"



SCHEMATIC SECTION NORTH OF PROPOSED ¢
SCALE: 1" TO 30'-0"



SCHEMATIC SECTION SOUTH OF PROPOSED ¢
SCALE: 1" TO 30'-0"



KEY PLAN
SCALE: 1" to 0.8 MILES

LEGEND

DETAILED BOREHOLE IN PLAN (4) IN ELEVATION

BOREHOLE TO ESTABLISH CLAY/TILL CONTACT IN PLAN (1) IN ELEVATION

P.1A PIEZOMETER IN PLAN

STRATIGRAPHY

VERY STIFF TO MEDIUM STIFF SILTY CLAY

CLAYEY SILT, SAND AND GRAVEL (TILL)

LIMESTONE DOLOMITE BEDROCK

SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

REFERENCE	
DRWG. No.	DESCRIPTION
	DEPARTMENT OF HIGHWAYS, ONTARIO PROPOSED TUNNEL UNDER WELLAND CANAL
61-F-11A	BORING PLAN - DATED APRIL 12, 1961
61-F-11B	PROFILE ALONG TUNNEL ¢ - DATED APRIL 12, 1961
61-F-11C	SECTIONS SHOWING SOIL STRATIGRAPHY - DATED APRIL 15, 1961

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO
PROPOSED WELLAND CANAL TUNNEL
WELLAND ONTARIO
BORING PLAN AND SOIL STRATIGRAPHY

GOLDER & ASSOCIATES
CONSULTING CIVIL ENGINEERS

DATE: MAY 26, 1961 SCALE: 1" TO 30'-0"

MADE J.A. CHKD. APPD. FIGURE 1