

62-F-253M

QUEENSWAY - MAIN ST.

BRIDGE # 25

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REPORT

TO

DELEUW, CATHER & COMPANY OF CANADA LIMITED

ON

SITE INVESTIGATION

PROPOSED QUEENSWAY - MAIN STREET OVERPASS

BRIDGE NO. 25

OTTAWA

ONTARIO

62-F-253 M

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## ABSTRACT

The results of a site investigation carried out for Bridge 25, the Main Street Overpass and the associated retaining wall to the west on the proposed Queensway in Ottawa, Ontario, are reported.

It was found that the site is covered by a thin layer of granular fill and silty sand overlying a stratum of Leda clay at about elevation 216. The Leda clay which is stiff and sensitive extends down to about elevation 160 and is followed by about 30 feet of compact to very dense silt grading to a silty fine sand with depth. A thin layer of very dense sandy silt till underlies the silty fine sand and rests on shale bedrock.

It is recommended that the overpass structure and retaining wall be founded on spread or strip footings in the upper clay, using a maximum allowable net bearing pressure of 3,000 pounds per square foot.

The stability of the approach embankments as proposed is adequate provided the rate of construction is controlled as discussed in the report.

Settlement of the approach embankment, overpass and retaining wall structures due to consolidation of the clay strata underlying the site will take place. Therefore allowance for settlement should be made in the design of the overpass and retaining wall structures. The magnitudes and time rates of settlement are discussed in the report.

It is recommended that the approach embankments be placed one year prior to construction of the overpass structure and final paving of the roadway in order to minimize the effects of differential settlements.

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## INTRODUCTION

H. Q. Golder & Associates Ltd. has been retained by DeLeuw, Cather & Company of Canada Limited, Consulting Engineers, to carry out a site investigation for the proposed Queensway-Main Street overpass structure in Ottawa, Ontario. The purpose of this investigation, which covered an area between about stations 423+00 and 429+00 along the Queensway, was to determine the soil conditions at the site, to provide information for the foundation design of the proposed structure and associated retaining wall and to study the stability of the approach embankments.

We have been supplied with drawing C45-E-S1, a site plan showing the location of the proposed structure and retaining wall.

## PROCEDURE

The field work for this investigation was carried out during the period November 16th to December 1st, 1961 using both mobile power auger and diamond drilling equipment. Two detailed boreholes with adjacent dynamic penetration tests and two additional dynamic penetration tests were put down at the proposed structure location. Both boreholes were taken down to bedrock and the bedrock was proved by core drilling for up to 15 feet in AXT size. Samples of the clay overburden in one borehole were obtained using the N.G.I. piston sampler and in the other by a conventional piston sampler. In addition to the boreholes at the structure site, three shallow bore-

holes with accompanying dynamic penetration tests were put down along the route of the proposed retaining wall.

The locations of all the borings put down in this investigation are shown on Figure 1. Sections of the inferred soil stratigraphy at the structure location and along the proposed retaining wall are given on Figure 2. Detailed logs of each boring are given on the Records of Boreholes.

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

All elevations given in this report were furnished by DeLeuw, Cather & Company of Canada Limited, and are referenced to Geodetic datum.

#### SITE TOPOGRAPHY AND GEOLOGY

The proposed Main Street overpass structure is located within the centre town section of the Ottawa Queensway. The area under investigation extends west from Main Street to a point about 100 feet east of Echo Drive, a total distance of about 600 feet. The Queensway centreline in this locality follows or closely parallels an abandoned railway right of way. The general ground surface at the site is essentially flat at about elevation 222 falling off to about elevation 210 in the vicinity of the Rideau Canal which crosses the area to the west and north-west of the

Queensway centreline at Echo Drive.

Geological information indicates that bedrock at the site is an Ordovician shale of the Billings Formation. The lower portion of the overburden consists of a relatively thin layer of glacial till deposited by the Labradorean Glacier during the Wisconsin stage of glaciation. At the close of this glaciation period, the area in the Ottawa-St. Lawrence River Valleys was invaded by an arm of the ocean known as the Champlain Sea. During this marine environment, thick beds of sands, silts and clays, (Leda Clay), were laid down over the glacial till. These marine deposits were then exposed by the subsequent uplift which occurred after retreat of the glaciers. In localized areas geologically recent deposits of sand and peat cover the Champlain soils.

#### SOIL CONDITIONS

The following main soil strata were encountered by the borings put down at the site.

##### Fill

A layer of fill between 2 and 3.5 feet thick covers the site along the line of the proposed retaining wall and at the northern end of the proposed bridge structure. This fill, which is associated with the existing railway line, is comprised of variable proportions of brown sand in all particle sizes with black cinders and occasionally some silt.

Standard penetration tests carried out in the layer gave

'N' values ranging from 3 to 8 blows per foot. Based on these results together with the dynamic penetration test results, the relative density of the fill corresponds to that of material which is very loose to loose and generally loose.

#### Fine Sand to Silt

At ground surface at the southern end of the bridge structure location and beneath the fill along the proposed retaining wall line west of about station 427+00, a stratum of brown to grey fine sand to silt with some clay was encountered. The stratum ranges from about 3 to 7 feet in thickness with an average of about 5 feet. Typical grain size curves obtained on samples from this stratum are shown on Figure 3. These show that in general the stratum at the bridge structure location is comprised of fine to medium sand with a trace of silt and to the west is comprised of essentially silt with some clay and fine sand. In borehole 7 at about station 423+50 a thin layer or pocket of peat was encountered between the fill and the surface of this stratum.

Standard penetration tests gave 'N' values of 3 to 28 blows per foot, which together with the dynamic penetration test results indicate that the stratum is generally loose to compact.



### Upper Clay

Underlying the sand to silt or fill at an average elevation of about 216, a stratum of generally highly plastic marine clay was encountered. The colour of the stratum is grey throughout, except in about the upper 5 feet where it is grey-brown as a result of weathering and oxidation. This upper clay stratum which extends down to about elevation 193, has a uniform thickness of about 23 feet across the site.

The composition of the clay is relatively homogeneous throughout. A grain size distribution curve obtained from a typical sample is shown on Figure 4. The curve shows that the stratum contains some silt with clay size material predominating. Examination of the samples obtained showed that the clay is generally fissured throughout and occasionally contains shell fragments and traces of black organic matter.

Atterberg limit determinations were carried out on samples from the stratum and these results together with the results of natural water content determinations are plotted on the Records of Boreholes. The liquid limit for the upper clay ranges generally between 60 and 85 with some values up to 90. The plasticity index ranges between limits of about 60 near the surface of the clay to about 35 with depth. The liquidity index, which is the ratio of natural water content minus the plastic limit to the plasticity index, ranges between about 0.5 to 1.2. Plots of plasticity index and liquidity

index versus elevation are shown on Figure 8.

The Atterberg limit test results for the stratum are also plotted on the plasticity chart in Figure 9. Based on the plasticity chart classification the stratum is an inorganic clay of high plasticity.

The activity of the upper clay, which is the ratio of plasticity index to the clay fraction, based on the results of one grain size analysis is about 0.5. This is in the inactive zone for clays.

Fourteen unit weight determinations on samples of the upper clay gave values of 98 to 108 pounds per cubic foot with an average of about 103 pounds per cubic foot.

The undrained shear strength of the clay was measured by in situ vane testing in the field and by unconfined and triaxial tests on samples in the laboratory. The results of these tests are plotted on Figure 10 and also on the Records of Boreholes. Typical stress strain curves for the laboratory tests are shown on Figure 11.

The strength results obtained by the field and laboratory testing, as shown by the shear strength profile on Figure 10, are generally in good agreement. A definite pattern of strength with depth is apparent with the shear strength in the upper desiccated crust above elevation 210 ranging between about 1,000 and 2,500 pounds per square foot. Below elevation 210 the shear strength increases from about

1,000 to 1,500 pounds per square foot at elevation 200 and decreases to about 1,200 pounds per square foot at elevation 193.

The sensitivity of the clay as determined by the field vane measurements ranges from about 7 to 23.

A set of two consolidated undrained triaxial compression tests with pore pressure measurements was carried out on samples of the upper clay. The purpose of these tests was to assess the effective shear strength parameters of the clay. The results obtained are plotted on Figure 21 using the method suggested by Rendulic (1937). The plot indicates that the effective angle of shearing resistance,  $\phi'$ , lies between the limits of  $25^\circ$  and  $30^\circ$ , with the lower limit at about 3 percent axial strain and the upper limit at about 7 percent axial strain. No appreciable cohesion,  $c'$ , is indicated.

A series of the above tests was carried out on samples of the upper clay in the investigation for the Queensway crossings at Rideau Canal and Elgin Street to the west of Echo Drive (our Report 6102-3 dated December, 1961). The results obtained also indicate that  $\phi'$  lies between the limits of  $25^\circ$  and  $30^\circ$  with no appreciable cohesion intercept.

Five consolidation tests were carried out on samples from the upper clay and the resulting pressure-void ratio curves are presented on Figures 12, 13, 16, 17 and 18. The results indicate that the clay is overconsolidated and the

most probable preconsolidation pressure has been computed by means of the Casagrande Construction and is shown on Figure 19.

#### Lower Silty Clay

Underlying the upper clay is a less plastic stratum of grey silty clay which extends down to about elevation 160. The thickness of the stratum is about 33 feet. Although deposited in a similar marine environment as the upper clay, the lower clay has less clay size particles and more silt and fine sand, as shown by the grain size distribution curve on Figure 5. The structure of the lower clay is less homogeneous than that of the upper clay. It is characterized by horizontal stratifications representing a minor variance in the colour and composition. Frequent thin layers of fine sand and silt are present within the stratum as well as shell fragments and organic matter in the forms of a black mottling. The black mottling or spotting is due to the presence of anaerobic bacteria in the silty clay, which disappears upon exposure of the soil to air.

As would be expected the plasticity of the lower clay is lower than that of the upper clay due to the higher silt and fine sand content. Liquid limits obtained ranged between about 20 and 45, with plasticity indices varying from about 5 to 20. The liquidity index for all the samples tested was above unity and reached a value of about 4. The pattern of plasticity index and liquidity index versus elevation is shown on Figure 8.

The lower clay based on the plasticity chart classification on Figure 9 is of generally low plasticity.

Total unit weight determinations on samples from the stratum gave values ranging from 109 to 123 pounds per cubic foot with an average representative value of about 115 pounds per cubic foot. The activity of the lower clay, based on one grain size analysis, is about 0.2 which is in the inactive zone for clays.

The undrained shear strength of the lower clay was also measured by vane tests in the field and by unconfined and triaxial compression tests in the laboratory. As in the case of the upper clay good agreement between the field and laboratory tests was obtained. All the undrained shear strength results obtained for the clay strata at the site are plotted against elevation on Figure 10. Typical stress strain curves from the laboratory tests are shown on Figure 11.

The sensitivity of the lower clay is generally slightly higher than that for the upper clay with the values as determined by the field vane ranging from about 7 to 32.

Consolidated undrained triaxial tests with pore pressure measurements carried out on lower clay samples in the Rideau Canal investigation gave a range for the effective angle of shearing resistance,  $\phi'$ , from about  $25^{\circ}$  to  $30^{\circ}$  with no appreciable effective cohesion,  $c'$ .

Two consolidation tests were carried out on samples of the lower clay in this investigation and the resulting pressure-void ratio curves are presented on Figures 14 and 15. The most probable preconsolidation pressure has been computed using the Casagrande construction and is shown on Figure 19. Also shown on the figure are the results for the upper clay and the results obtained from the Rideau Canal and Elgin Street investigation together with a range of preconsolidation pressures for the Leda clay in the Ottawa area reported by Crawford (1961) plotted against elevation. The lower range of values shown in the figure is not so much representative of the possible variation in preconsolidation pressure at a particular elevation as it is indicative of the difficulty of measuring the preconsolidation load in the laboratory on the sensitive marine clay. Refined sampling and laboratory testing techniques supplemented by data obtained from field studies indicate that the preconsolidation load in the Ottawa area is at least as great as that given by the upper limit shown in Figure 19 (Crawford, 1961). This is generally confirmed by the majority of results obtained in this investigation. It therefore appears reasonable to take the upper limit of the range of preconsolidation pressure on Figure 19 for design purposes.

The consolidation parameters for both the upper and lower clay obtained from the log pressure-void ratio curves are shown on Figure 20. Included in this figure are the results of the Rideau and Elgin investigation. The initial void

ratio ( $e_o$ ), compression index ( $C_c$ ) and recompression index ( $C_{cr}$ ) for each consolidation sample has been plotted against the corresponding plasticity index. The resulting curves present a generally good trend of correlation. As mentioned previously a plot of plasticity index versus elevation for the clay strata at the site is given on Figure 8. The compression index, ( $C_c$ ), is a measure of the slope of the virgin or straight line portion of log pressure-void ratio curve and the recompression index, ( $C_{cr}$ ), is a measure of the slope of the rebound and reload cycle of the curve when the pressure is reduced to about the existing overburden pressure of each sample.

#### Silt to Silty Sand.

The lower clay is underlain by a stratum of silt to silty sand at about elevation 160. The thickness of the stratum increases from about 25 feet at Main Street to about 40 feet at Echo Drive. In general the upper portion of the stratum is comprised of silt with a trace of clay and fine sand grading to a sandy silt and silty fine sand with depth. Occasional thin clayey layers and traces of gravel are present throughout the stratum. Typical grain size distribution curves for the material are given on Figure 6.

Standard penetration tests carried out in the stratum gave 'N' values ranging from 10 to 67 blows per foot. Based on these results together with the pattern of the dynamic

penetration test results, the stratum is compact becoming very dense with depth.

### Sandy Silt Till

A layer of basal till, up to 10 feet in thickness, underlies the silt to silty sand stratum. As shown by the grain size distribution curve on Figure 7, the till is comprised of a well graded composite of silt, sand and subangular gravel with a trace of clay. Although not obtained in the sampler, due to its limited size ( $1\frac{1}{2}$  inches I.D.), the till contains coarse gravel, cobble and boulder sizes. The general predominance of silt and sand sizes allows classification as a sandy silt till, essentially non-plastic.

Based on the standard penetration tests, which gave 'N' values in excess of 50 blows per foot, the till is very dense.

### Bedrock

Underlying the till is a sound dark grey shale bedrock, which was proved in AXT size for a maximum distance of 15 feet in borehole 1. The bedrock was encountered at about elevations 131 and 122 at the north-east and south-west corners of the proposed bridge structure location and slopes off to about elevation 113 at Echo Drive.



### Groundwater

Piezometers were installed in each of the boreholes put down in this investigation and water level readings taken periodically for several months following completion of the field work. The stabilized groundwater levels as observed in the piezometers are given on the Records of Boreholes. From the readings taken, the winter groundwater level across the site is generally at about elevation 215 which is at about the surface of the upper clay.

### DISCUSSION

#### General

The Queensway-Main Street overpass at about station 428+50 has been proposed as a reinforced concrete rigid frame with a free span of about 60 feet. The overpass structure is to be approached on the Queensway by earth fill embankments at a grade of about elevation 240 or about 18 feet above the Main Street roadway level. The western approach embankment to main street is a continuation of the approach embankment from the eastern side of the proposed Rideau Canal bridge at about station 422+50. The maximum height of the embankment will be about 23 feet above existing ground level at station 423+00, decreasing uniformly to about 18 feet at Main Street.

Due to space restrictions along the north side of the Queensway, a concrete retaining wall about 500 feet in length is to be provided to support the embankment fill between the Main

Street and Canal bridges. The south side of the embankment in this area is to be built using standard 2 horizontal to 1 vertical side slopes. The extent of the retaining wall along the north side is shown in plan on Figure 1.

#### Approach Embankments

The stability of the overpass structure and retaining wall, as proposed, will be controlled by the overall stability of the approach embankments. The most critical period for the stability of the embankment at the site will be when the fill is first brought to full height and the pore pressures induced in the underlying clay by the embankment load are highest. Following completion of construction the factor of safety will increase with time as the pore pressures in the clay are allowed to dissipate.

An analysis using the total stress approach (undrained shear strength of the clay) is a valid method for predicting the initial stability of an embankment founded on normally consolidated or lightly overconsolidated clay but does not provide the factor of safety at any given time during or following the construction period. In order to evaluate the factor of safety for these conditions, the stability is analysed using the effective stress approach (drained shear strength parameters of the clay) incorporating pore pressures. However, it is not possible to accurately predict the pore pressures during construction on the basis of laboratory tests alone.

Therefore, the most practical approach is to design the embankment initially on the basis of the undrained shear strength and to control its rate of construction by the effective stress analyses based on the measured field pore pressures.

The most critical section for analysis of stability has been taken through the retaining wall at station 423+00, shown on Figure 22, where the fill is the highest. A circular arc mode of failure has been considered with the arcs extending through the approach fill and into the underlying clay strata.

For embankments constructed at a normal rate, the most reasonable estimate of overall stability has been found empirically to be based on the undrained shear strength,  $c$ , of the clay. A profile of the shear strengths measured in the clay strata at the site is presented on Figure 10. The pattern of shear strength with depth, considering the transition from clay to silty clay at about elevation 193, is characteristic for slightly overconsolidated clays. The shear strengths range generally from a minimum of about 1,000 to 1,500 pounds per square foot.

A summary of the computed factors of safety for various assumed shear strengths is plotted on Figure 22. From these results it may be seen that the most critical mode of failure for the retained embankment is a shallow circle (circle A) which gives a factor of safety of about 1.5 for the minimum shear strength at the site of 1,000 pounds per square foot.

For an average shear strength of 1,200 pounds per square foot, the minimum factor of safety is above 1.5, the normal minimum used in design. Thus it is concluded, based on the undrained strength approach, that the overall stability of the approach embankment along the retaining wall and at the overpass structure is adequate.

The importance of the effective stress stability analysis to the control of construction of the embankment is shown by a series of analyses carried out for assumed pore pressure distributions in the clay strata. The results of these analyses, which are summarized on Figure 23, indicate that if the rate of construction of the embankments is very rapid thus inducing high pore pressures in the underlying clay, the factor of safety for the embankment could be lower than unity. However, comparison of the effective stress analyses with the undrained analyses is complicated by the difficulty in predicting the exact distribution of pore pressure induced by the embankment at any set time. However, recent experience on the construction of an embankment on foundation soil of similar nature to that at the site indicates that for a normal rate of construction the maximum ratio of induced pore pressure to the maximum stress exerted on the foundation soil by the embankment,  $(\Delta u / \sigma_h)$ , is of the order of 0.5. Reference to the chart on Figure 23 where the effective stress analyses are summarized shows that the factor of safety at a pore pressure ratio,  $(\Delta u / \sigma_h)$ , of 0.5 would be

about 1.2 for an effective angle of shearing resistance,  $\phi'$ , of  $25^\circ$  and about 1.5 for  $\phi'$  of  $30^\circ$ . The corresponding factor of safety for the undrained analysis using a shear strength of 1,000 pounds per square foot is about 1.5. This comparison between the two approaches, although not conclusive, does show that there is a general agreement depending on the choice of strength parameters and considering the assumptions of the pore pressure distribution.

### Foundations

It is recommended that the overpass structure and retaining wall be founded on spread or strip footings in the upper portion of the clay stratum. Allowing for an adequate earth cover to prevent frost action and taking the footings below the weathered upper few feet of the clay, the foundation grade across the site would be at about elevation 213. Based on the average design shear strength of 1,200 pounds per square foot for the clay, the maximum allowable net bearing pressure for foundation design is 3,000 pounds per square foot.

The overpass structure abutments and retaining wall should be backfilled with free-draining and non-frost susceptible granular material for at least 6 feet in horizontal extent. For the computation of lateral earth pressures on the rigid frame abutments, it is recommended that a coefficient of earth pressure at rest,  $K_0$ , of 0.5 be employed. With

effective drainage provided by the granular material behind the abutments and retaining wall and with heavy compaction of the embankment fill, the unit weight of the fill material may be taken as 135 pounds per cubic foot.

With the clay foundation soil at the site and the use of a precast concrete crib type retaining wall, as planned, it is considered that sufficient movement of the wall will take place to develop an active earth pressure condition in the embankment fill. For compact to dense granular backfill material the coefficient of active earth pressure,  $K_a$ , is of the order of 0.3.

In the computation of sliding resistance between a rough concrete retaining wall base and the clay foundation soil at the site, a strength value of 1,000 pounds per square foot may be used in design, provided precautions are taken to prevent any softening of the sensitive clay at foundation grade during and following construction.

#### Settlement

The overpass structure and retaining wall together with the approach embankment will experience settlement due to consolidation of the underlying clay strata resulting from application of load over the site. The points which must be considered in the design of the overpass and retaining wall structures and roadway pavement are the probable magnitude

and time rate of settlement. The most important factor affecting settlement is the relationship of the applied loads from construction to the preconsolidation load of the foundation strata, which is the maximum load under which the clays have been consolidated in the past.

The preconsolidation loads have been computed from consolidation tests and are plotted against elevation on Figure 19. Also shown on this figure is a range of preconsolidation pressures for the Ottawa area reported by Crawford (1961) plotted against elevation. As discussed under a previous section of this report "Soil Conditions", the results of this investigation confirm the use of the upper limit of the range of preconsolidation pressure for design purposes.

The computed existing overburden stresses in the clay and the stresses which would be induced beneath the central portion and vertical edge of the embankment, where the fill is highest at station 423+00, are shown on Figure 24. This figure shows that the new stress level under the embankment would be below the line of preconsolidation pressure throughout the clay strata and thus the settlement from the embankment load would be due only to reconsolidation of the clay. The ultimate consolidation settlement based on the consolidation parameters given on Figure 20 is estimated to be about 2 inches under the central portion of the embankment and about  $1\frac{1}{2}$  inches at the retaining wall.

As for the retaining wall, settlement of the overpass structure abutment due to consolidation of the clay will take place due to the composite stresses induced by the approach embankment and structure loading. The stresses induced beneath the centre and edge of the structure abutment have been computed and are shown on Figure 25. Based on these stress distributions the maximum settlement is estimated to be about  $1\frac{1}{2}$  inches under the centre and about 1 inch at the end of each abutment. There will therefore be some differential settlement across the abutment and it is recommended that in design allowance be made for up to 1 inch of tilting between the centre and ends of each abutment and across the span.

The time rate of settlement has also been estimated from the results of the consolidation tests. The coefficient of consolidation,  $c_v$ , on which the estimate is based ranges generally from about 0.1 to 0.3 square feet per day for the stresses which will be induced in the subsoil. The resultant range in time rate of settlement is shown on Figure 26. The best estimate of the probable range is also shown and it is likely that the average rate of settlement would be in this zone. Thus after a period of about  $1\frac{1}{2}$  to 2 years following construction 50 percent of the settlement would have taken place with 90 percent occurring in about 6 to 8 years after construction.

It is understood that paving of this section of the Queensway will take place immediately after completion of the



overpass structure. In order to avoid possible settlement cracking in the pavement, it is recommended that the approach embankments be constructed as soon as practicable. If the embankments can be constructed far enough in advance of the scheduled time for paving, a substantial portion of the consolidation settlements in the clay would be taking place in the period before and during the construction of the overpass structure. An allowance of about one year between the construction of the embankments and paving would mean that about 40 percent of the settlement would have taken place.

The actual time rate of settlement of the embankments can only be ascertained accurately by installing piezometers and settlement gauges before construction and taking regular observations during and following construction. The information obtained from the piezometers would also afford a check on the overall stability as discussed in a preceding section of the report. The piezometers should be installed in minimum sets of two with each piezometer approximately at the third point of the upper clay stratum. Settlement gauges should be placed at the surface of the upper clay.

#### Construction Procedures

It is recommended that the first step in construction be the installation of piezometers and settlement gauges for control. This installation should be completed several weeks prior to commencement of approach embankment construction.

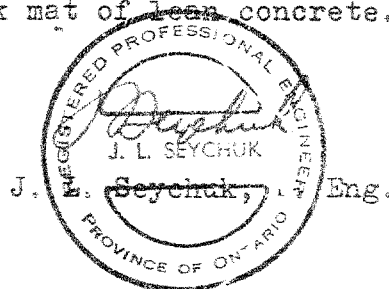
The piezometers and settlement gauges should be installed near the west abutment of the overpass structure.

It is further recommended that the approach embankments be placed at least one year in advance of construction of the overpass structure and final paving operations to minimize the effects of differential settlements.

Prior to placement of the embankment fill all topsoil across the site should be removed. The fill should be placed over the full width of the embankment in uniform lifts each approximately 6 to 9 inches in thickness. In order to minimize rapid build-up in pore pressure in the foundation clay strata, it is recommended that not more than 2 lifts be placed over a 24 hour period.

To prevent softening and remoulding of the sensitive clay at the site due to entrance of surface water and construction operations, it is recommended that the base of all footing excavations, once foundation grade is reached, be immediately covered by a 4 inch thick mat of lean concrete.

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## 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:

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

### SOIL TESTS

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

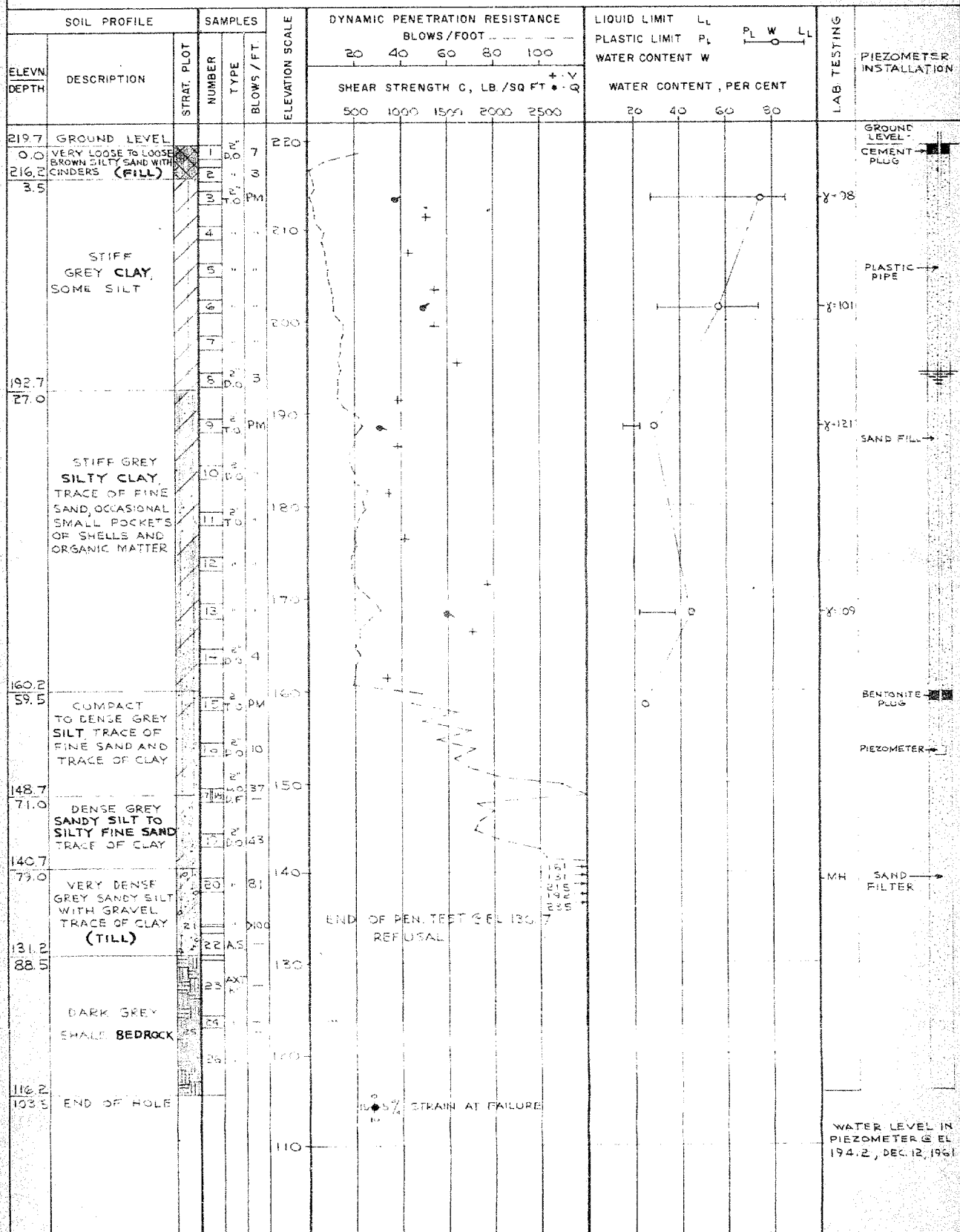
Note: Undrained triaxial tests in which pore pressures are measured are shown as Q' or Q'c.

### SOIL PROPERTIES

$\gamma$ - Total Unit Weight	K - Coefficient of Permeability
$\gamma_d$ - Dry Unit Weight	c - Undrained Shear Strength ( $\frac{1}{2}$ Compressive Strength)
$\gamma_b$ - Submerged Unit Weight	St - Sensitivity
L <sub>L</sub> - Liquid Limit	$\phi'$ - Effective Angle of Shearing Resistance
P <sub>L</sub> - Plastic Limit	c* - Effective Cohesion Intercept
W - Natural Water Content	Cc - Compression Index
G - Specific Gravity	Cv - Coefficient of Consolidation
e - Void Ratio	

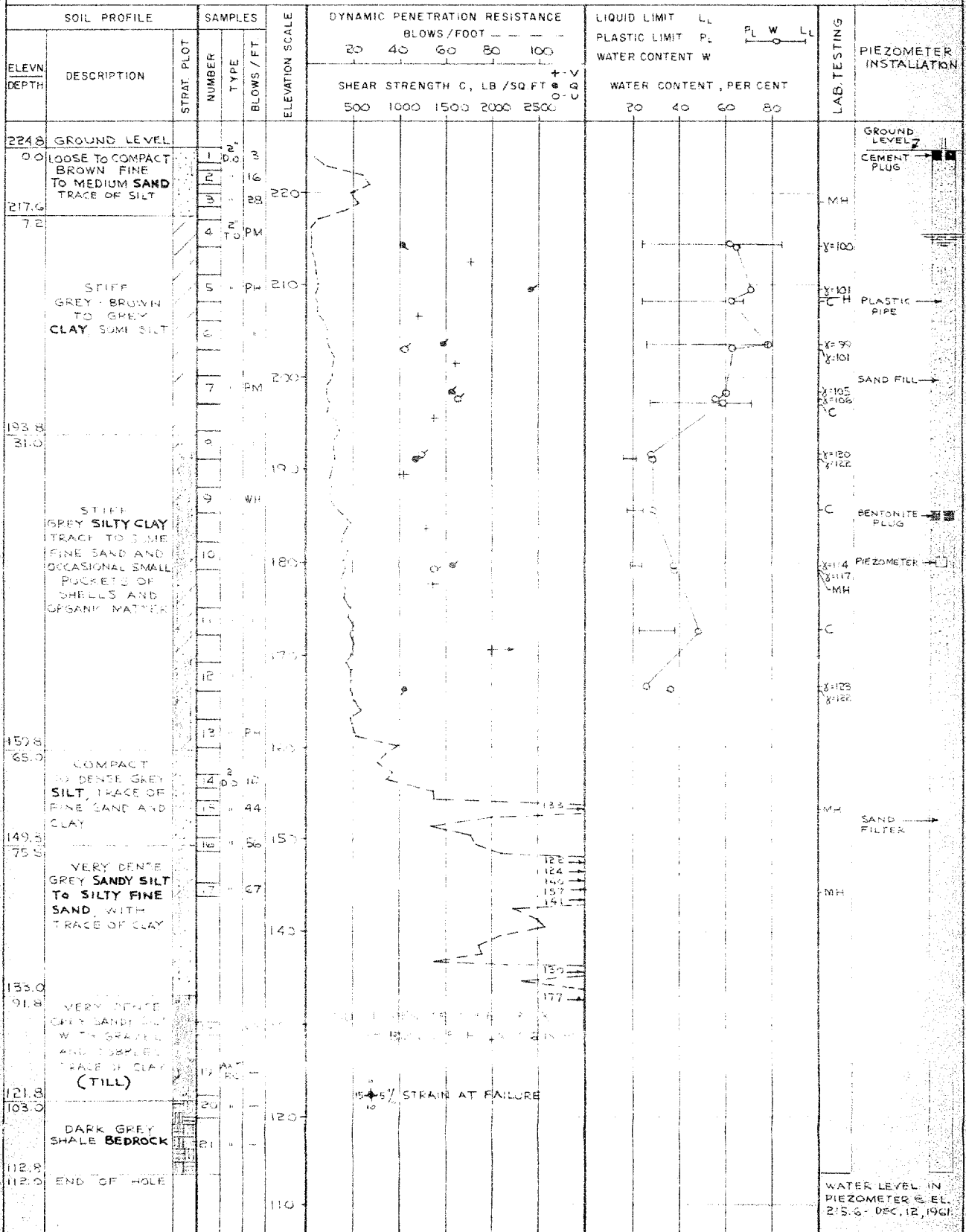
## RECORD OF BOREHOLE 1

LOCATION SEE FIGURE 1 BORING DATE NOV. 16-22, 1961 DATUM GEODETIC  
 BOREHOLE TYPE POWER AUGER & DIAMOND DRILL BOREHOLE DIAMETER 4.5" & BX CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



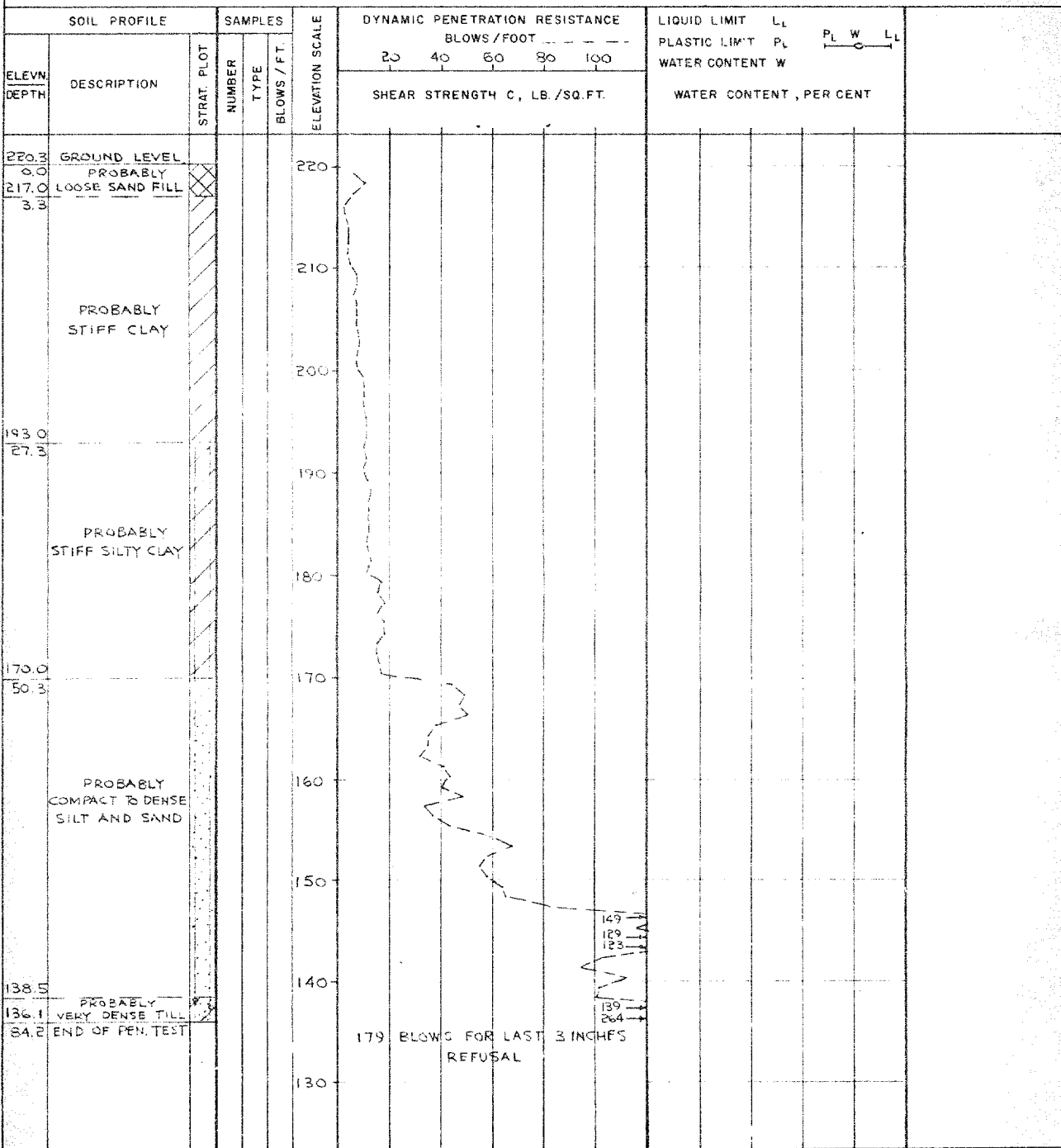
## RECORD OF BOREHOLE 2

LOCATION SEE FIGURE 1 BORING DATE NOV. 20-30, 1961 DATUM GEODETIC  
 BOREHOLE TYPE POWER AUGER & DIAMOND DRILL BOREHOLE DIAMETER 4.5" & BX CASING  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



# RECORD OF BOREHOLE 3

LOCATION SEE FIGURE 1 BORING DATE NOV. 25, 1961 DATUM GEODETIC  
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —  
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



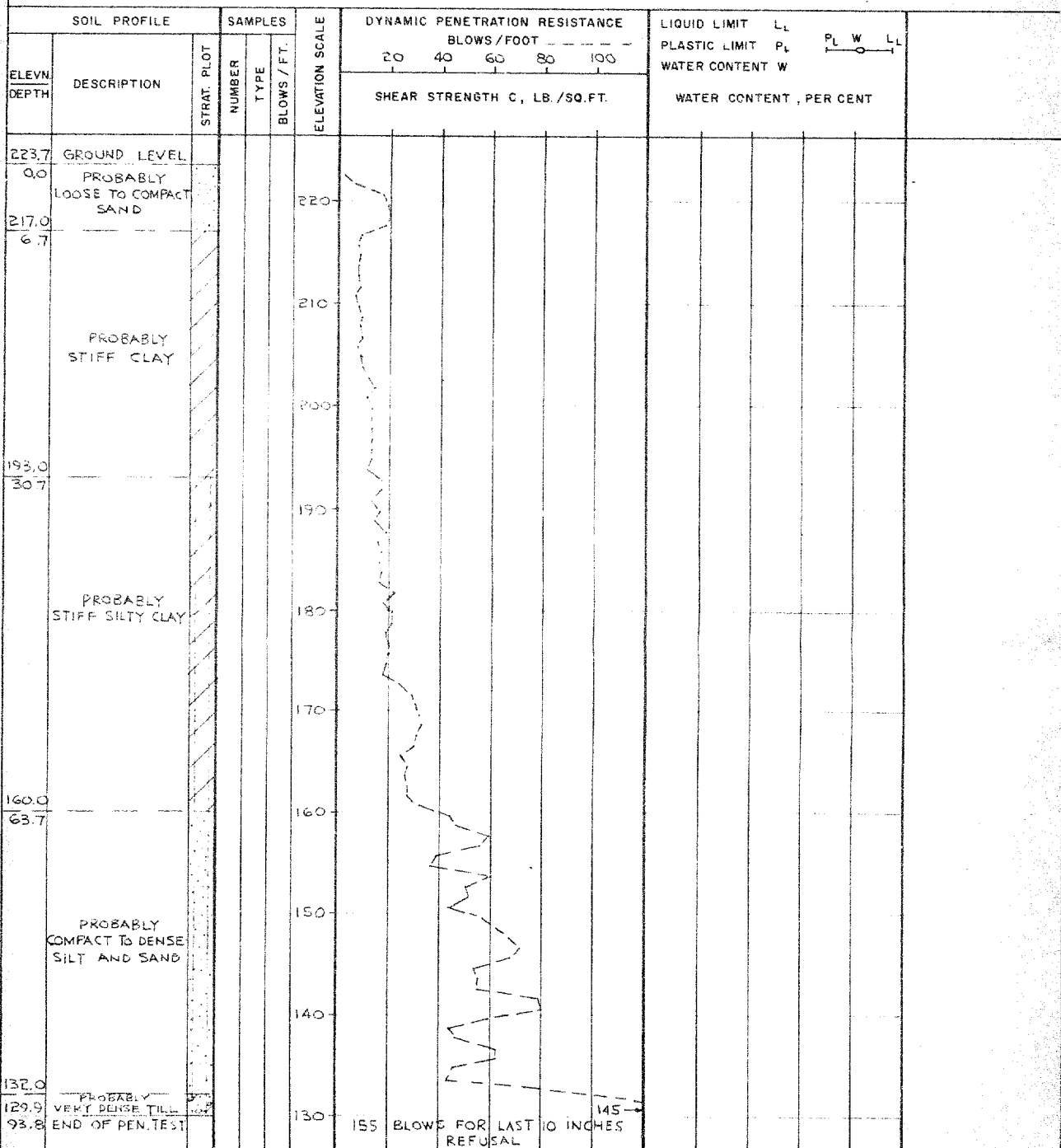
VERTICAL SCALE  
 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.  
 CHECKED *br*

# RECORD OF BOREHOLE 4

LOCATION SEE FIGURE 1 BORING DATE NOV. 27, 1961 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 FEET

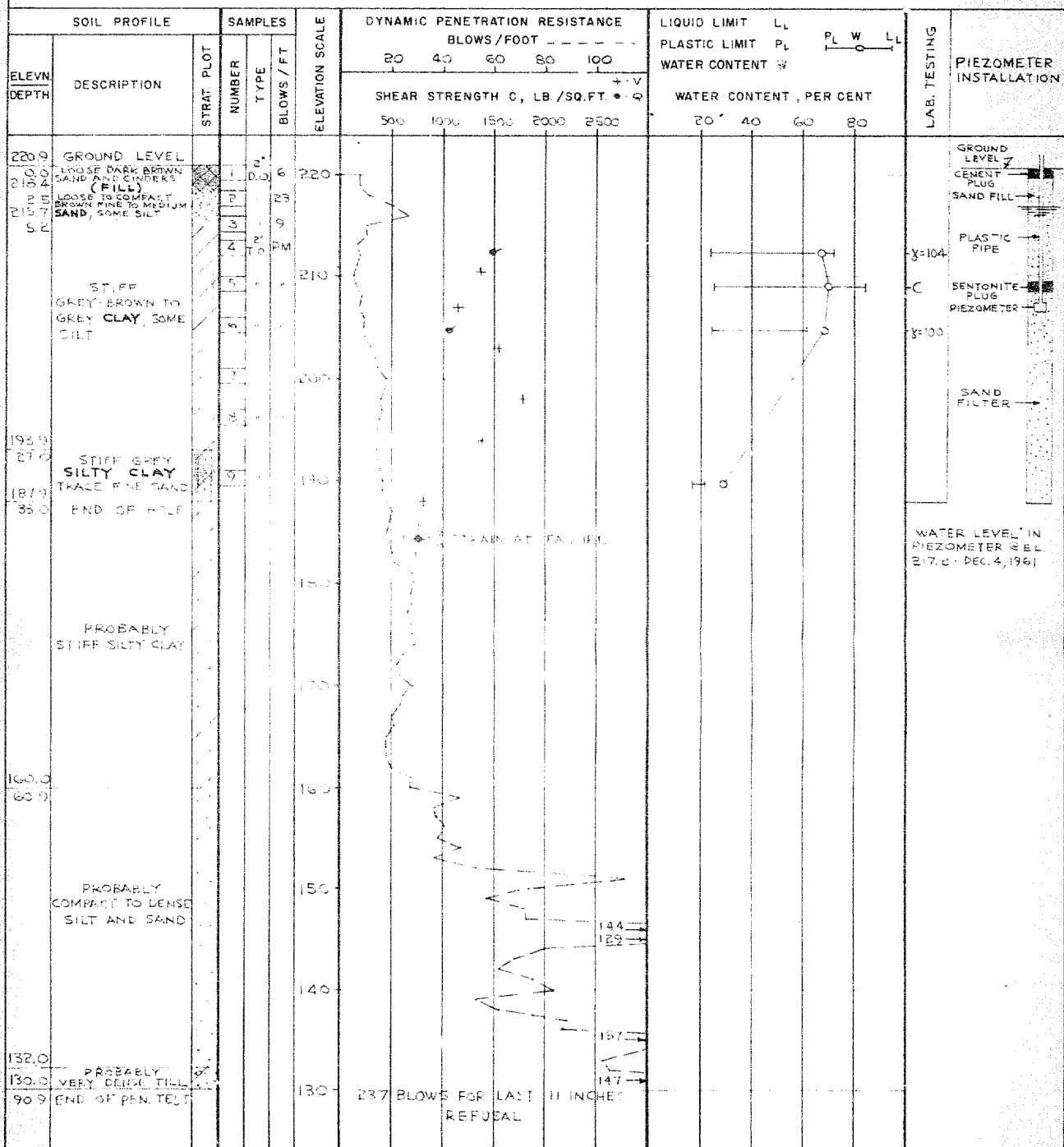
GOLDER & ASSOCIATES

DRAWN J.A.  
 CHECKED *dry*



## RECORD OF BOREHOLE 5

LOCATION	SEE FIGURE 1	BORING DATE	NOV. 21-22, 1961	DATUM	GEODETIC
BOREHOLE TYPE		POWER AUGER BORING		BOREHOLE DIAMETER	
4.5"					
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES		PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES			



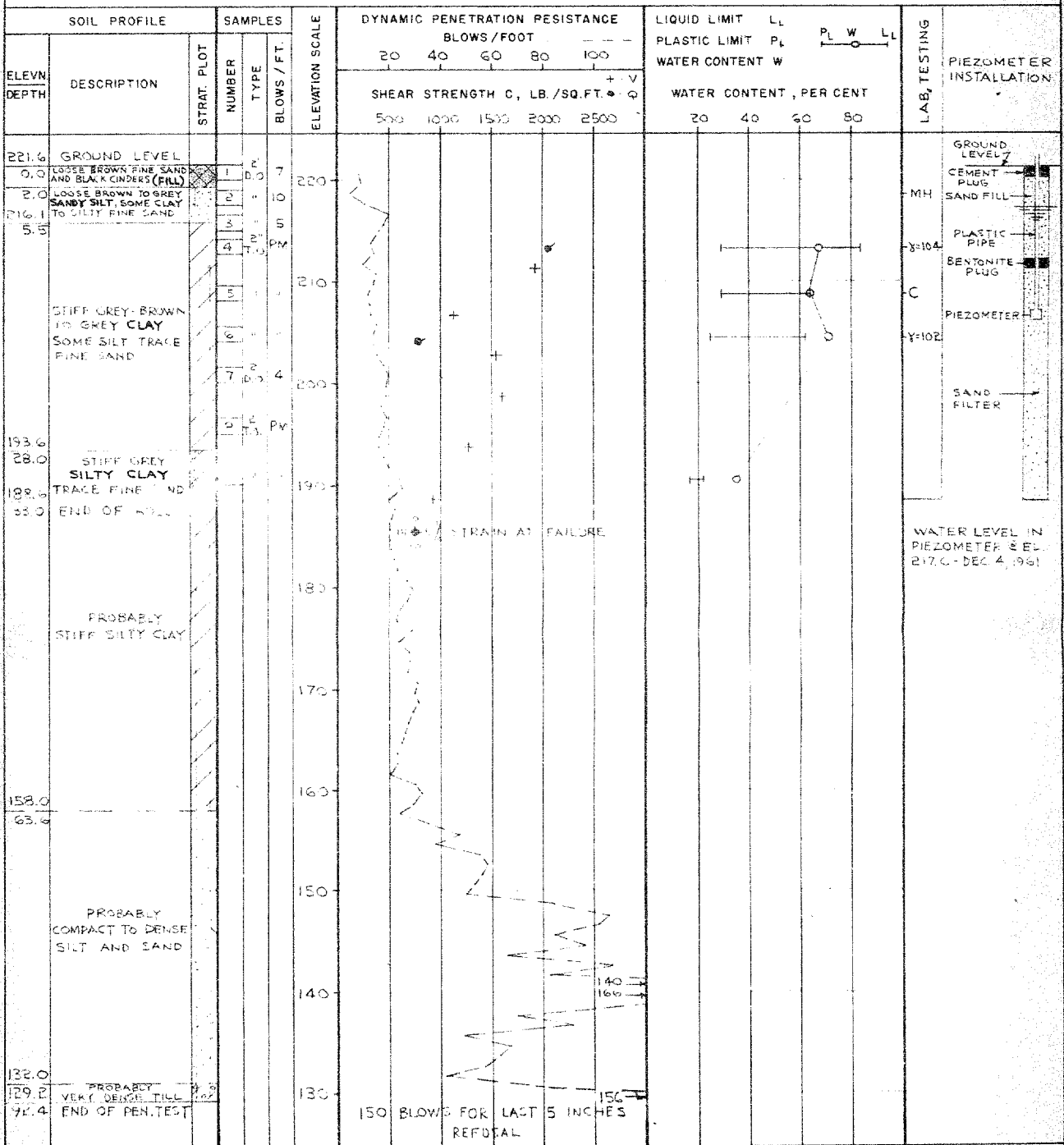
VERTICAL SCALE  
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.  
CHECKED *[Signature]*

# RECORD OF BOREHOLE 6

LOCATION SEE FIGURE 1 BORING DATE NOV. 23, 1961 DATUM GEODETIC  
 BOREHOLE TYPE POWER AUGER BORING BOREHOLE DIAMETER 4.5"  
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE  
 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.  
 CHECKED J.R.

## RECORD OF BOREHOLE 7

GEODETIC

4.5"

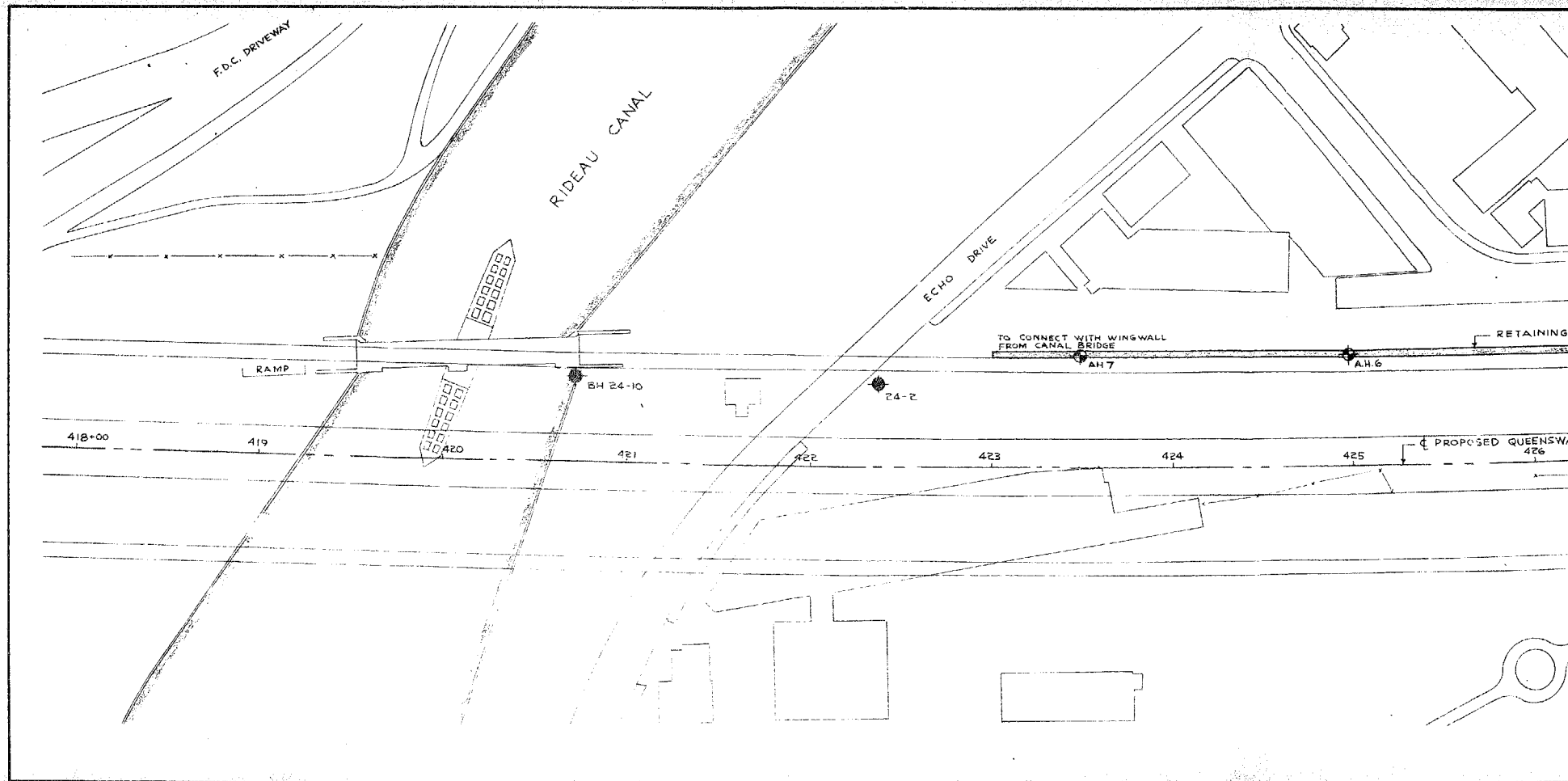
PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

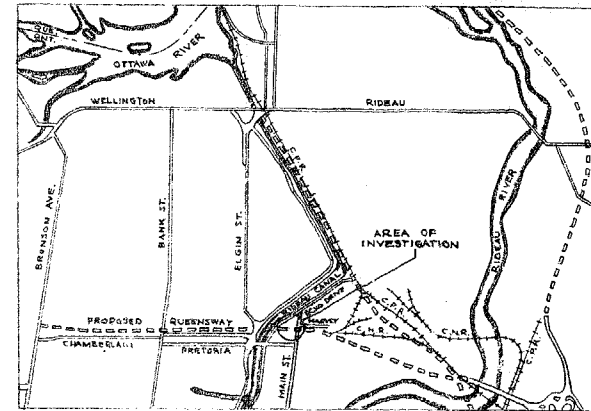
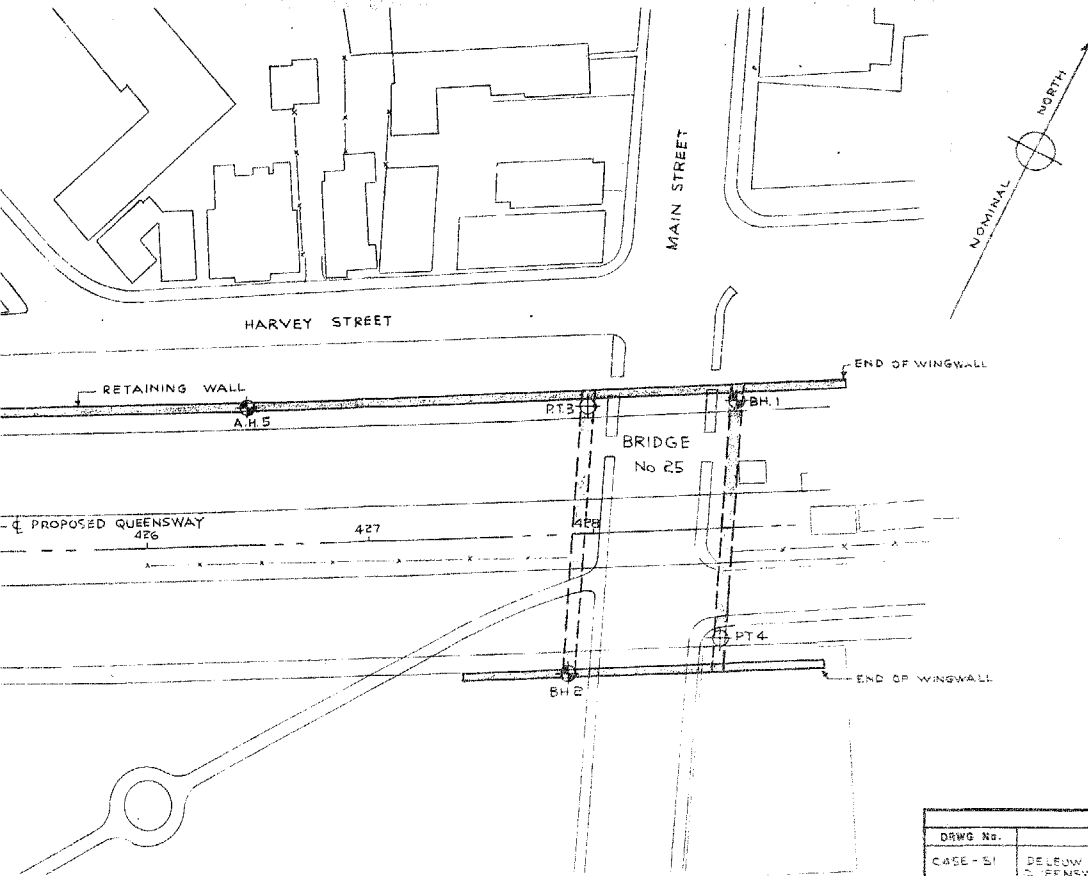
SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT L <sub>L</sub>		LAB. TESTING	STANDPIPE INSTALLATION		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS/FOOT					PLASTIC LIMIT P <sub>L</sub>	WATER CONTENT W				
						20	40	60	80	100					P <sub>L</sub>	W
						SHEAR STRENGTH C, LB./SQ. FT. • q					WATER CONTENT, PER CENT					
						500	1000	1500	2000	2500	20	40	60	80		
222.2	GROUND LEVEL															
219.1	LOOSE BROWN SAND AND BLACK CINDERS (FILL)		1	0.0	8											GROUND LEVEL
215.1	SOFT DARK BROWN PEAT		2	0.0	15											CEMENT PLUG
215.1	LOOSE GREY BROWN SANDY SILT SOME CLAY		3	0.0	9											M.H.
215.1			4	0.0	5											γ=108
215.1			5	0.0	5											SAND FILL
215.1			6	0.0	5											OBSERVATION PIPE
215.1			7	0.0	5											BENTONITE PLUG
215.1			8	0.0	5											γ=132
215.1			9	0.0	5											
215.1			10	0.0	5											
215.1			11	0.0	5											
215.1			12	0.0	5											
215.1			13	0.0	5											
215.1			14	0.0	5											
215.1			15	0.0	5											
215.1			16	0.0	5											
215.1			17	0.0	5											
215.1			18	0.0	5											
215.1			19	0.0	5											
215.1			20	0.0	5											
215.1			21	0.0	5											
215.1			22	0.0	5											
215.1			23	0.0	5											
215.1			24	0.0	5											
215.1			25	0.0	5											
215.1			26	0.0	5											
215.1			27	0.0	5											
215.1			28	0.0	5											
215.1			29	0.0	5											
215.1			30	0.0	5											
215.1			31	0.0	5											
215.1			32	0.0	5											
215.1			33	0.0	5											
215.1			34	0.0	5											
215.1			35	0.0	5											
215.1			36	0.0	5											
215.1			37	0.0	5											
215.1			38	0.0	5											
215.1			39	0.0	5											
215.1			40	0.0	5											
215.1			41	0.0	5											
215.1			42	0.0	5											
215.1			43	0.0	5											
215.1			44	0.0	5											
215.1			45	0.0	5											
215.1			46	0.0	5											
215.1			47	0.0	5											
215.1			48	0.0	5											
215.1			49	0.0	5											
215.1			50	0.0	5											
215.1			51	0.0	5											
215.1			52	0.0	5											
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215.1			54	0.0	5											
215.1			55	0.0	5											
215.1			56	0.0	5											
215.1			57	0.0	5											
215.1			58	0.0	5											
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215.1			62	0.0	5											
215.1			63	0.0	5											
215.1			64	0.0	5											
215.1			65	0.0	5											
215.1			66	0.0	5											
215.1			67	0.0	5											
215.1			68	0.0	5											
215.1			69	0.0	5											
215.1			70	0.0	5											
215.1			71	0.0	5											
215.1			72	0.0	5											
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215.1			83	0.0	5											
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215.1			91	0.0	5											
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215.1			93	0.0	5											
215.1			94	0.0	5											
215.1			95	0.0	5											
215.1			96	0.0	5											
215.1			97	0.0	5											
215.1			98	0.0	5											
215.1			99	0.0	5											
215.1			100	0.0	5											
215.1			101	0.0	5											
215.1			102	0.0	5											
215.1			103	0.0	5											
215.1			104	0.0	5											
215.1			105	0.0	5											
215.1			106	0.0	5											
215.1			107	0.0	5											
215.1			108	0.0	5											
215.1			109	0.0	5											
215.1			110	0.0	5											
215.1			111	0.0	5											
215.1			112	0.0	5											
215.1			113	0.0	5											
215.1			114	0.0	5											
215.1			115	0.0	5											
215.1			116	0.0	5											
215.1			117	0.0	5											
215.1			118	0.0	5											
215.1			119	0.0	5											
215.1			120	0.0	5											
215.1			121	0.0	5											
215.1			122	0.0	5											
215.1			123	0.0	5											
215.1			124	0.0	5											
215.1			125	0.0	5											
215.1			126	0.0	5											
215.1			127	0.0	5											
215.1			128	0.0	5											
215.1			129	0.0	5											
215.1			130	0.0	5											
215.1			131	0.0	5											
215.1			132	0.0	5											
215.1			133	0.0	5											
215.1			134	0.0	5											
215.1			135	0.0	5											
215.1			136	0.0	5											

VERTICAL SCALE  
1 INCH TO 10 FEET

GOLDER & ASSOCIATES




DRAWN J.A.  
CHECKED ?





KEY PLAN  
SCALE: 1" TO 2,200' (APPROX.)

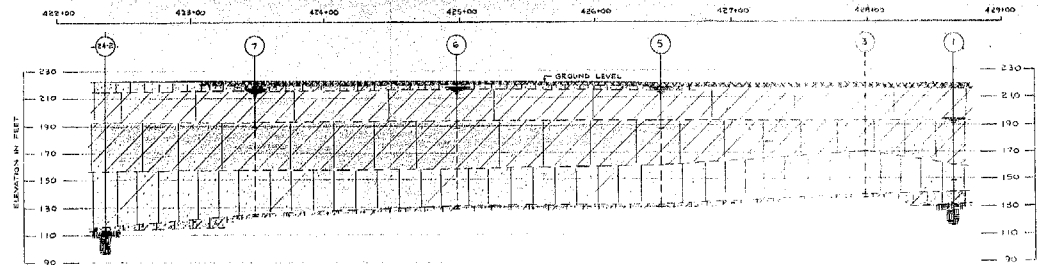
### LEGEND

-  BOREHOLE WITH PENETRATION TEST IN PLAN
-  PENETRATION TEST IN PLAN
-  BOREHOLE IN PLAN (RIDEAU CANAL INVESTIGATION) REPORT No. 6102-3

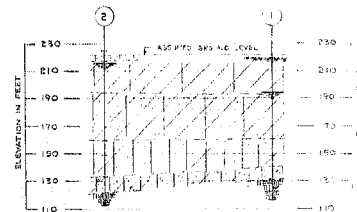
REFERENCE	
DRWG. No.	DESCRIPTION
CASE - 51	DE LEUW, CATHER & COMPANY - OTTAWA QUEENSWAY, BRIDGE NO. 25 - SITE PLAN DATED NOV. 15, 1941.
REPORT G102 - 3	H. G. GOLDER & ASSOCIATES LTD. DATED DEC. 1961.

DE LEUW, CATHER & COMPANY OF CANADA LIMITED	
OTTAWA	ONTARIO
PROPOSED QUEENSWAY-MAIN ST OVERPASS	
OTTAWA	ONTARIO
BORING PLAN	

GOLDER & ASSOCIATES CONSULTING CIVIL ENGINEERS	
DATE: JAN. 22, 1962 SCALE: 1" TO 40'-0"	
MADE J.A.	CHKD. APPD. FIGURE 1



SECTION ALONG PROPOSED RETAINING WALL



SECTION ALONG CENTRE LINE - MAIN ST.

## STRATIGRAPHY

- LOOSE BROWN SAND AND CINDERS (FILL)
- LOOSE TO COMPACT FINE SAND TO SANDY SILT, SOME CLAY
- STIFF GREY-BROWN TO GREY CLAY, SOME SILT
- STIFF GREY SILTY CLAY, TRACE FINE SAND, OCCASIONAL SHELLS AND ORGANIC POCKETS
- COMPACT TO VERY DENSE GREY SILT WITH TRACE FINE SAND AND CLAY BECOMING SANDY SILT TO SILTY FINE SAND WITH TRACE OF CLAY
- VERY DENSE GREY SANDY SILT WITH GRAVEL AND COBBLES, TRACE OF CLAY (TILL)
- DARK GREY SHALE BEDROCK

## LEGEND

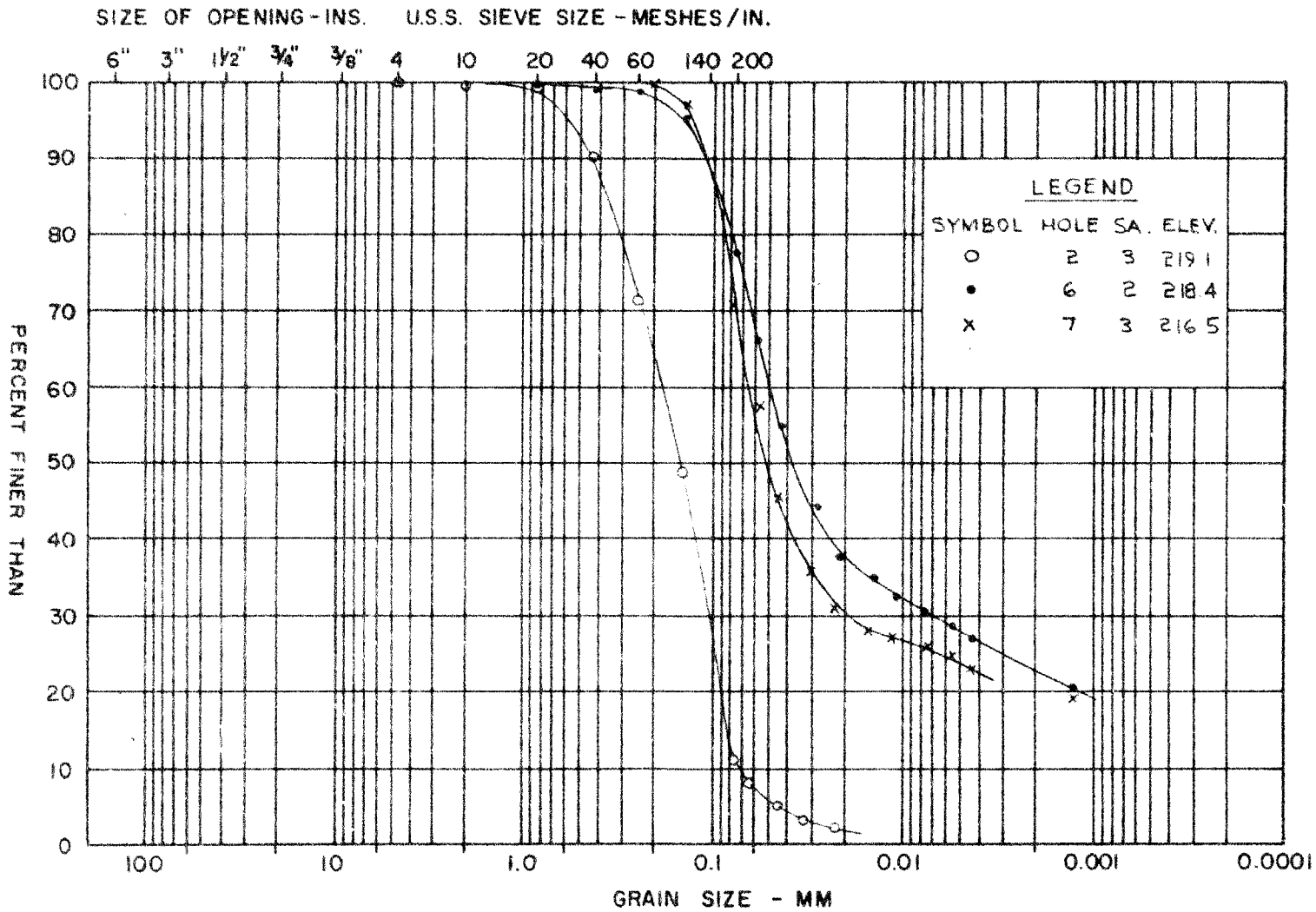
- BOREHOLE AND PENETRATION TEST IN ELEVATION
- PENETRATION TEST IN ELEVATION
- BOREHOLE IN ELEVATION (RIDEAU CANAL INVESTIGATION) REPORT NO. G102-2
- WATER LEVEL IN BOREHOLE, DEC. 1961

REFERENCE	
DRWG. NO.	DESCRIPTION
FIGURE 1	GOLDER & ASSOCIATES - BORING PLAN
REPORT GMS	GOLDER & ASSOCIATES - DATED DEC 1961

DE LEUW CATHIER & COMPANY OF CANADA LIMITED	
OTTAWA PROPOSED QUEENSWAY - MAIN ST. OVERPASS OTTAWA	ONTARIO
SOIL STRATIGRAPHY	

GOLDER & ASSOCIATES CONSULTING CIVIL ENGINEERS	
DATE: JAN. 22, 1962 SCALE: 1" TO 40'-0"	
MADE J.A.	CHKD. J.P.
APPD. J.P.	FIGURE 2

## M.I.T. GRAIN SIZE SCALE

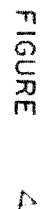


GOLDER &amp; ASSOCIATES

GRAIN SIZE DISTRIBUTION  
UPPER GRANULAR STRATA

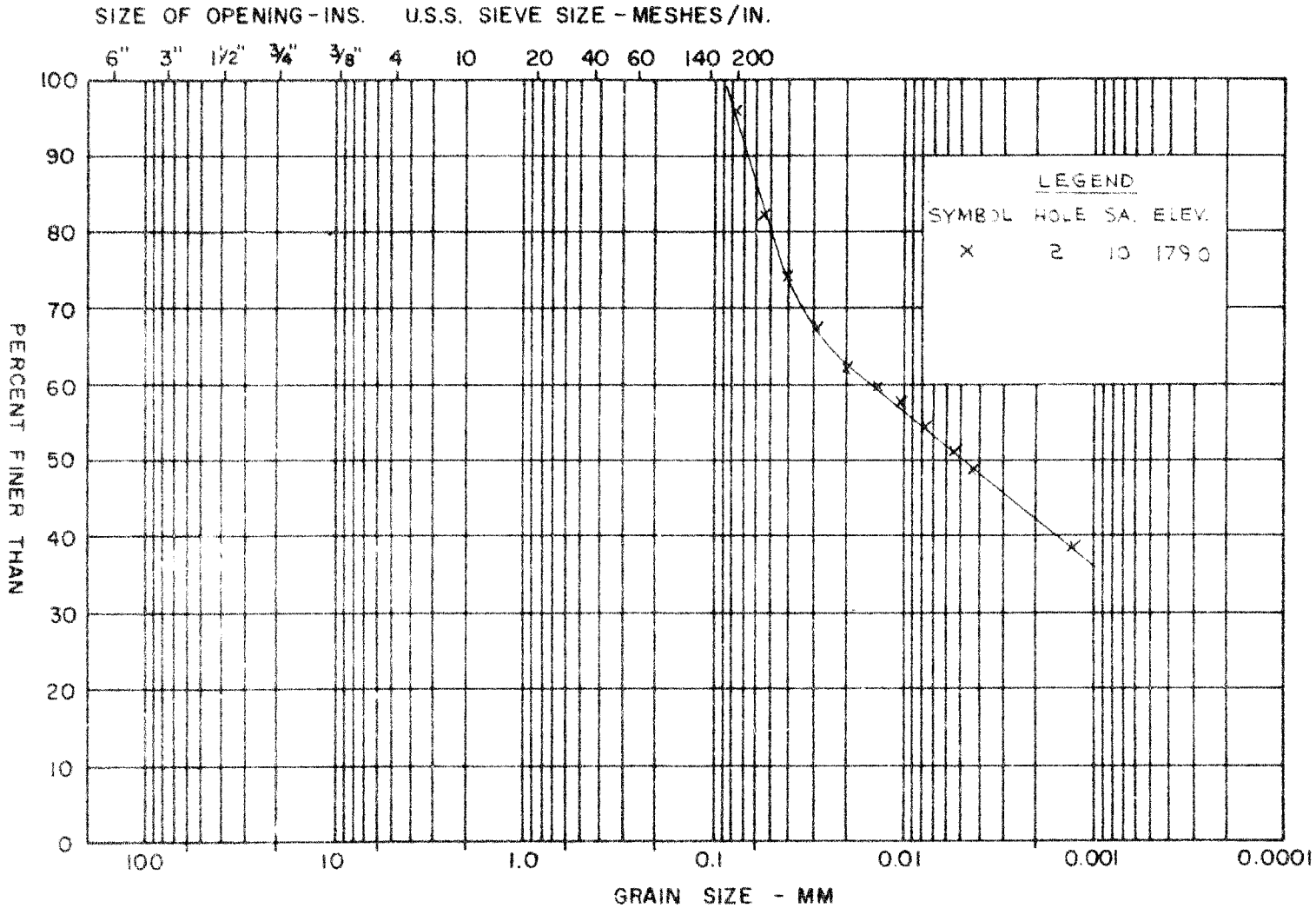
FIGURE 3

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





M.I.T. GRAIN SIZE SCALE

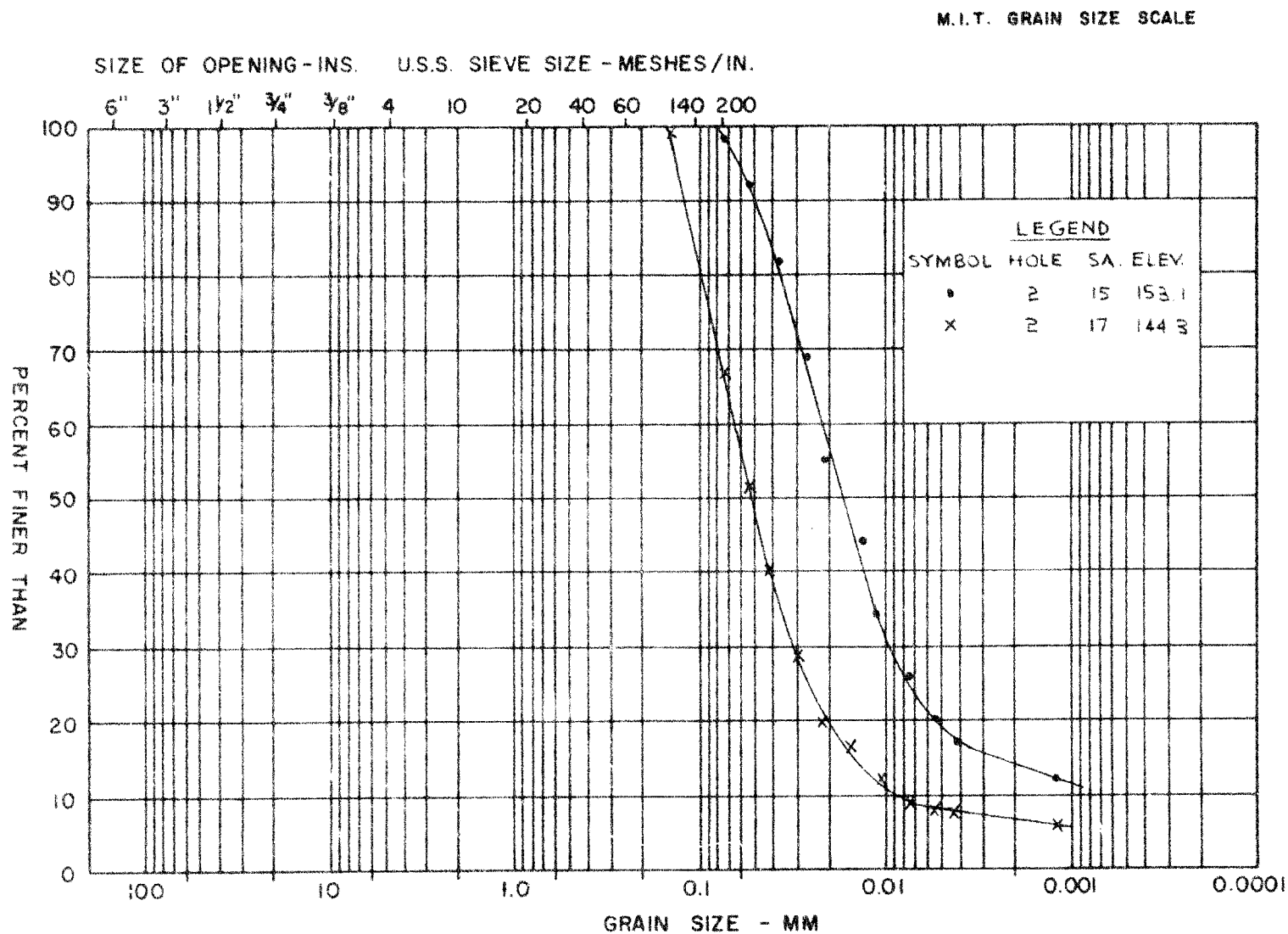


GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION  
LOWER CLAY

FIGURE 15

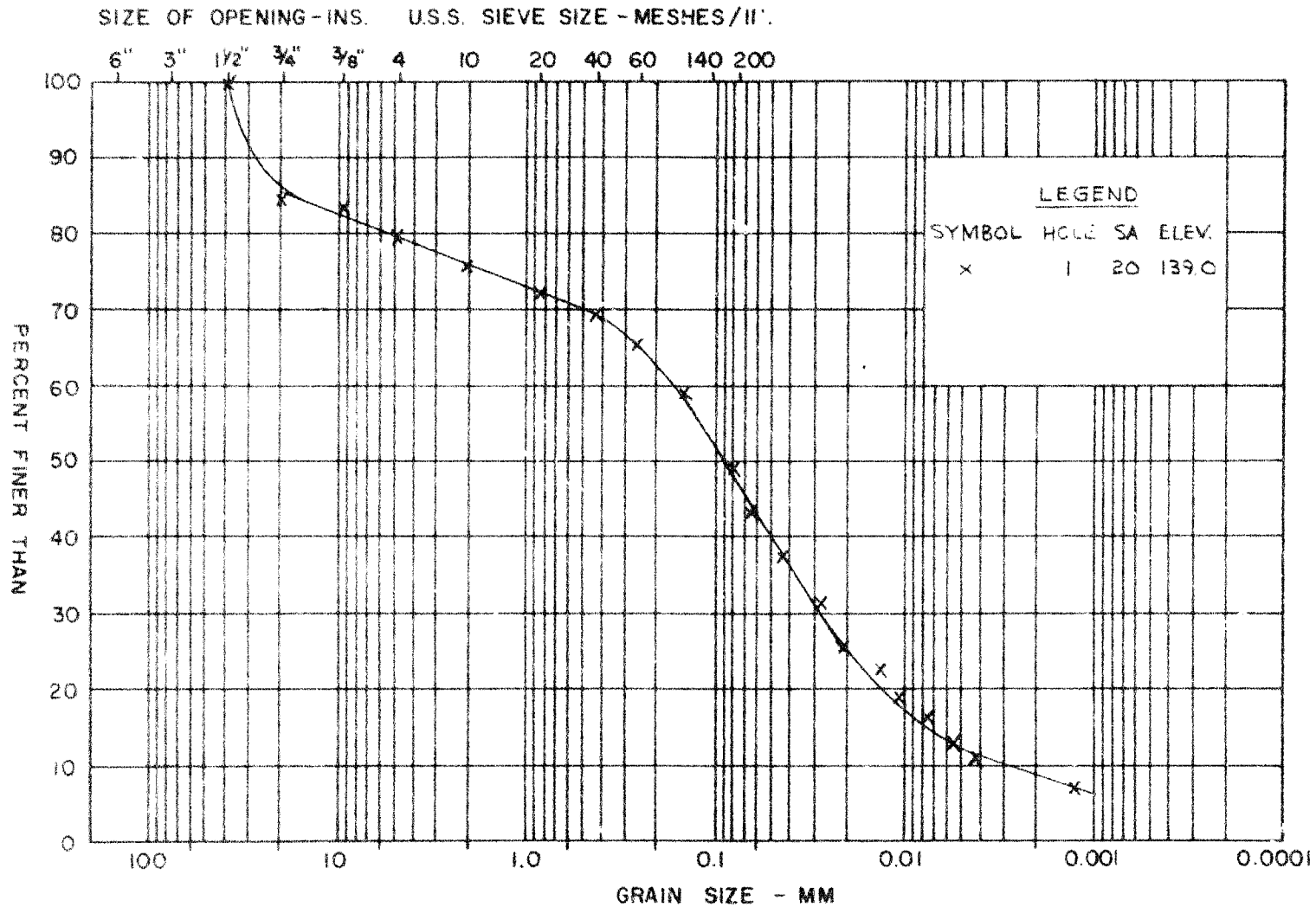
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	



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

GRAIN SIZE DISTRIBUTION  
SILT TO SANDY SILT

## M.I.T. GRAIN SIZE SCALE



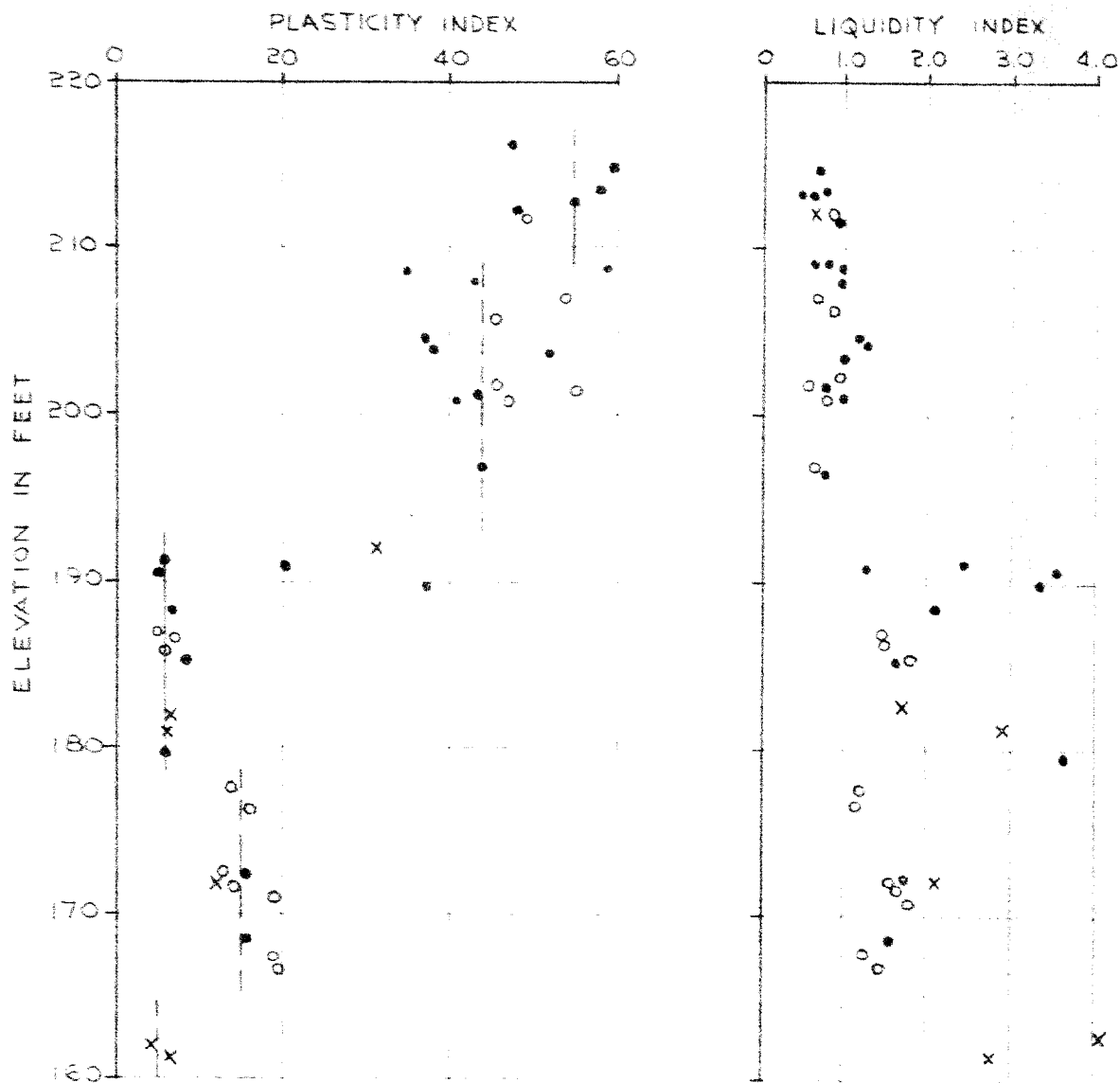
GOLDER &amp; ASSOCIATES

GRAIN SIZE DISTRIBUTION  
TILL

FIGURE 7

# PLASTICITY INDEX & LIQUIDITY INDEX CLAY STRATA

FIGURE 8



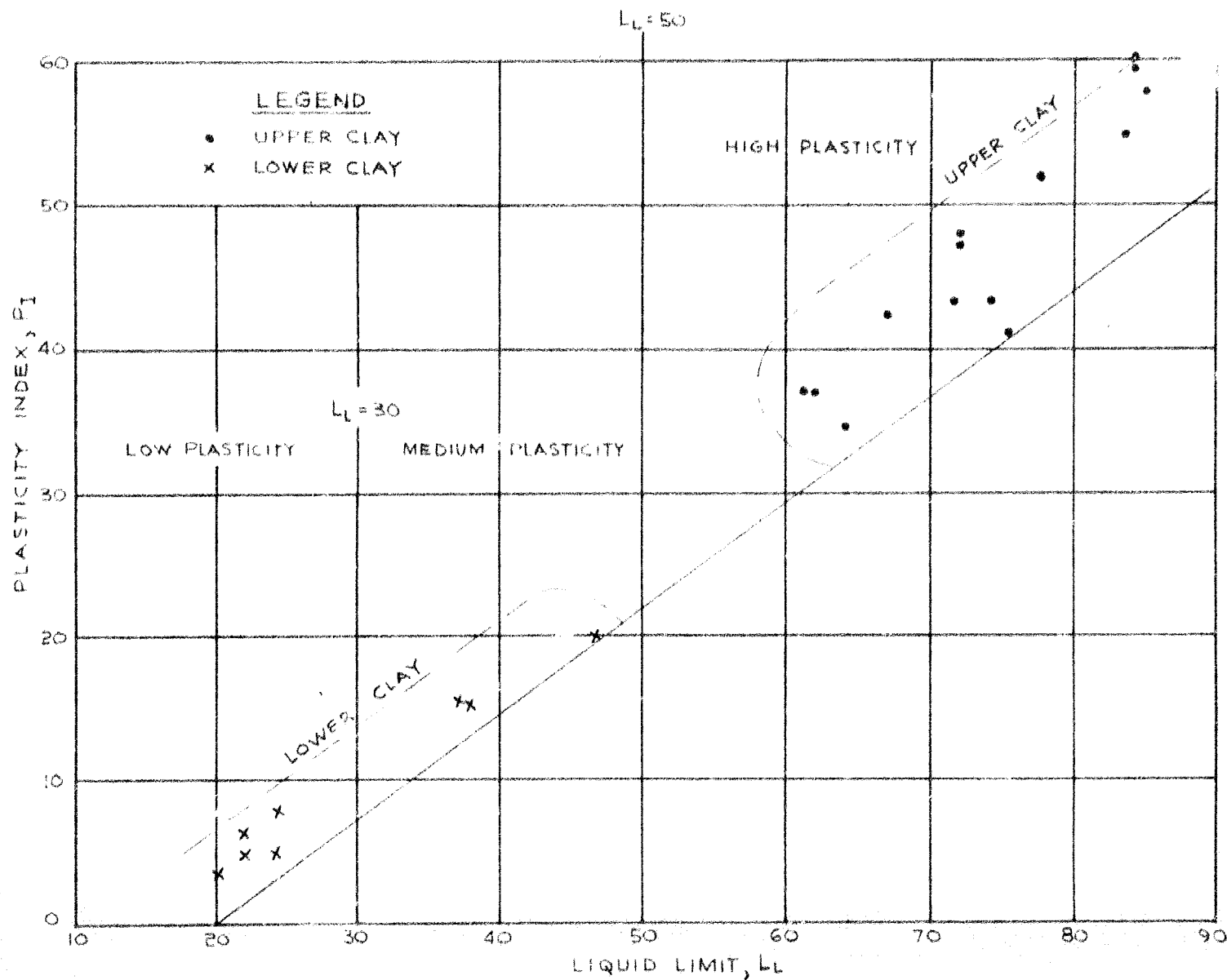
## LEGEND

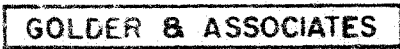
- BOREHOLES 1, 2, 5, 6 AND 7 - PRESENT INVESTIGATION
- BOREHOLE 24-2 - RIDEAU CANAL INVESTIGATION
- x BOREHOLE 24-10 - RIDEAU CANAL INVESTIGATION

PLASTICITY CHART  
CLAY STRATA

FIGURE

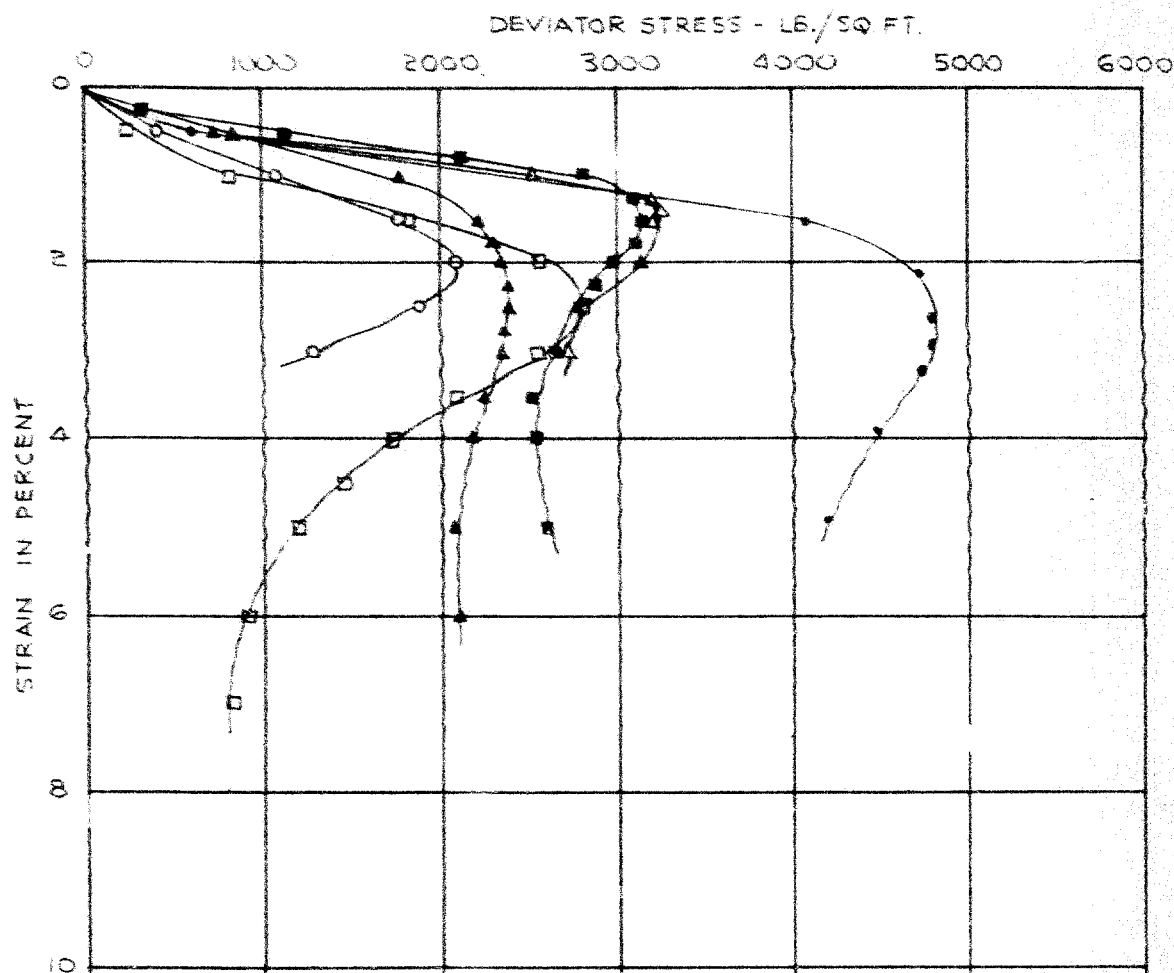
9





# UNDRAINED COMPRESSION TESTS TYPICAL STRESS-STRAIN CURVES CLAY STRATA

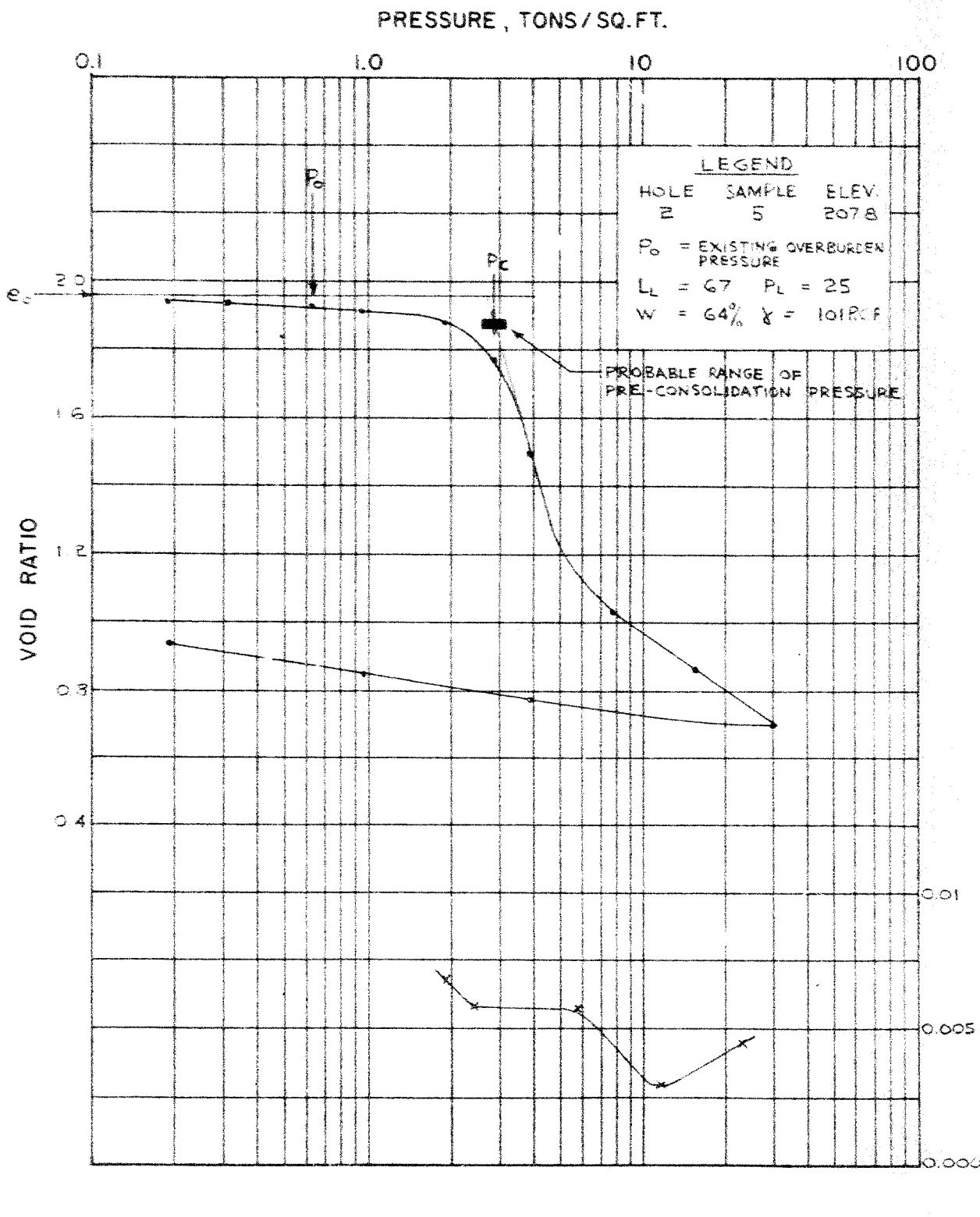
FIGURE 11



LEGEND			
SYMBOL	HOLE	SAMPLE	ELEV.
TRIAXIAL			
●	2	5	209.5
▲	2	8	191.1
■	2	10	179.5
UNCONFINED			
○	2	6	203.0
△	2	7	198.1
□	2	10	179.0

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 12

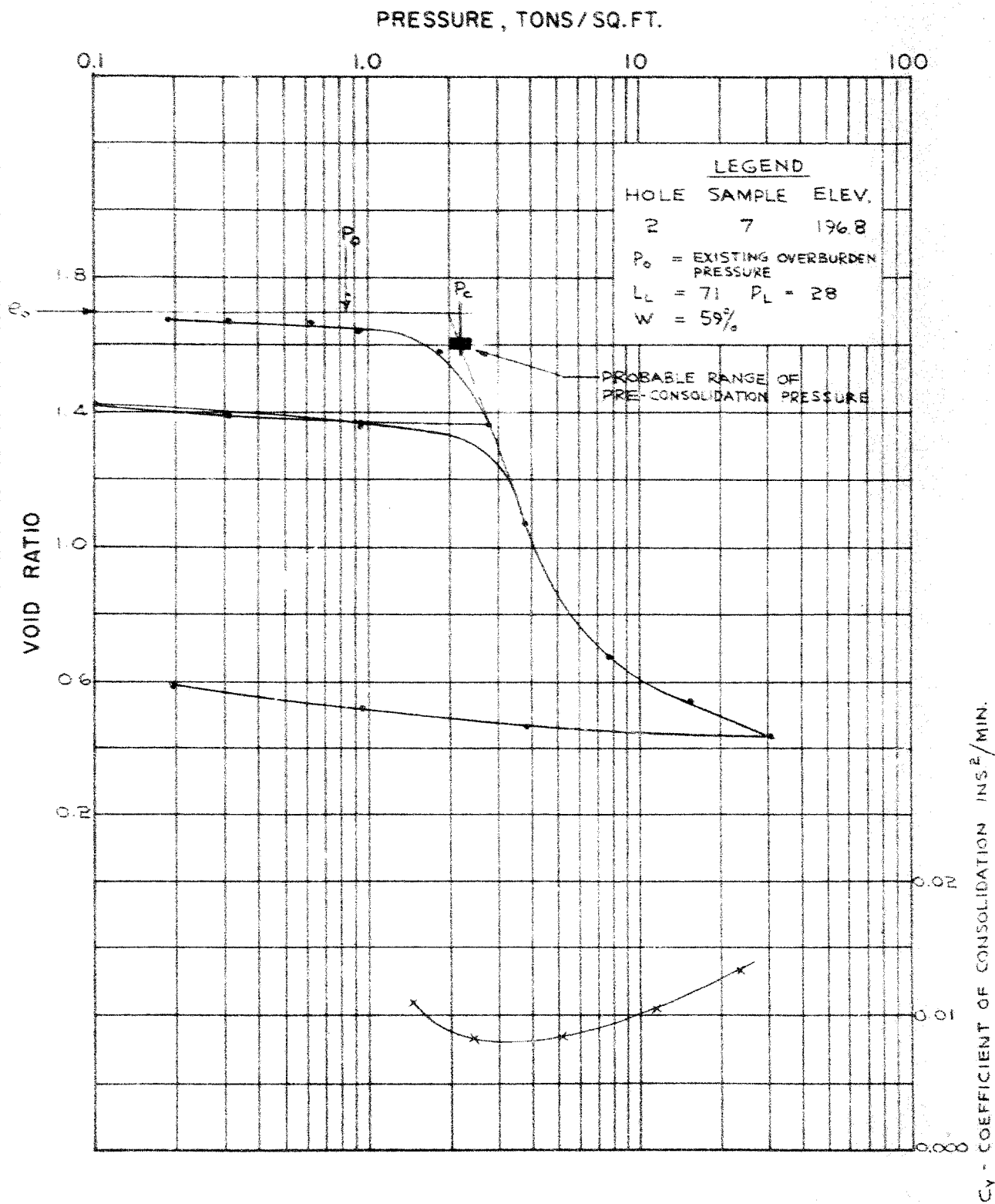


GOLDER & ASSOCIATES



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

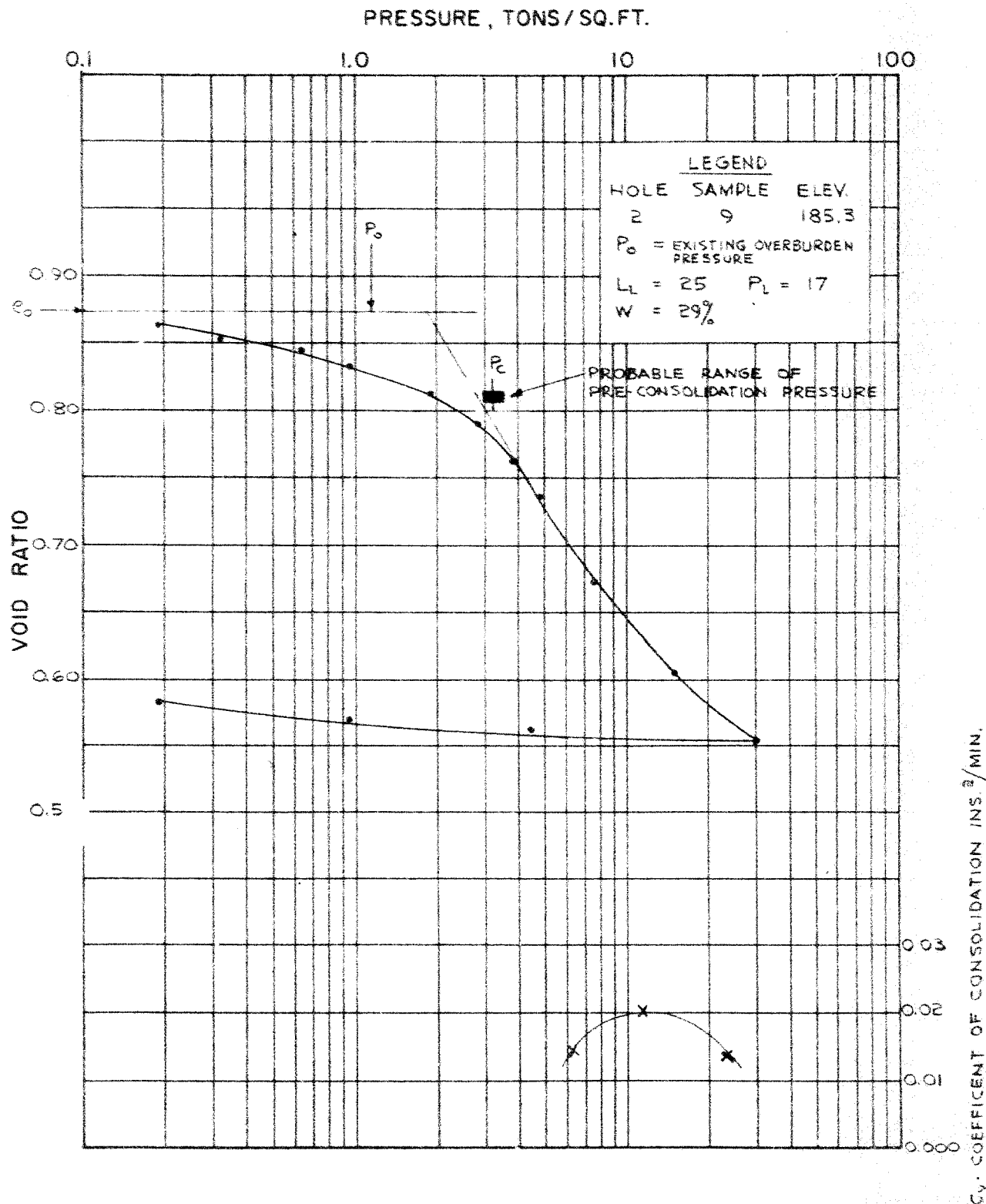
FIGURE 13



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

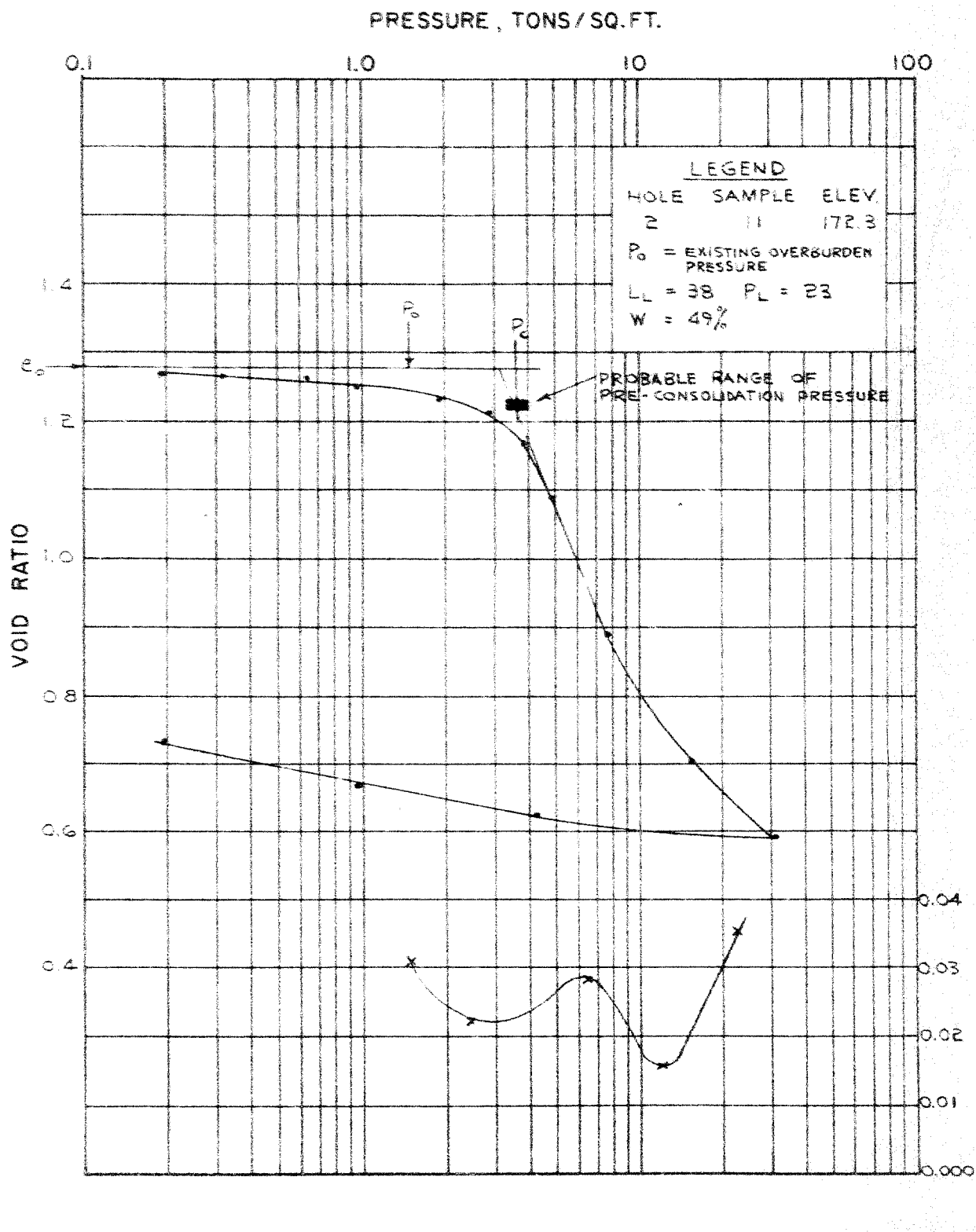
FIGURE 14



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

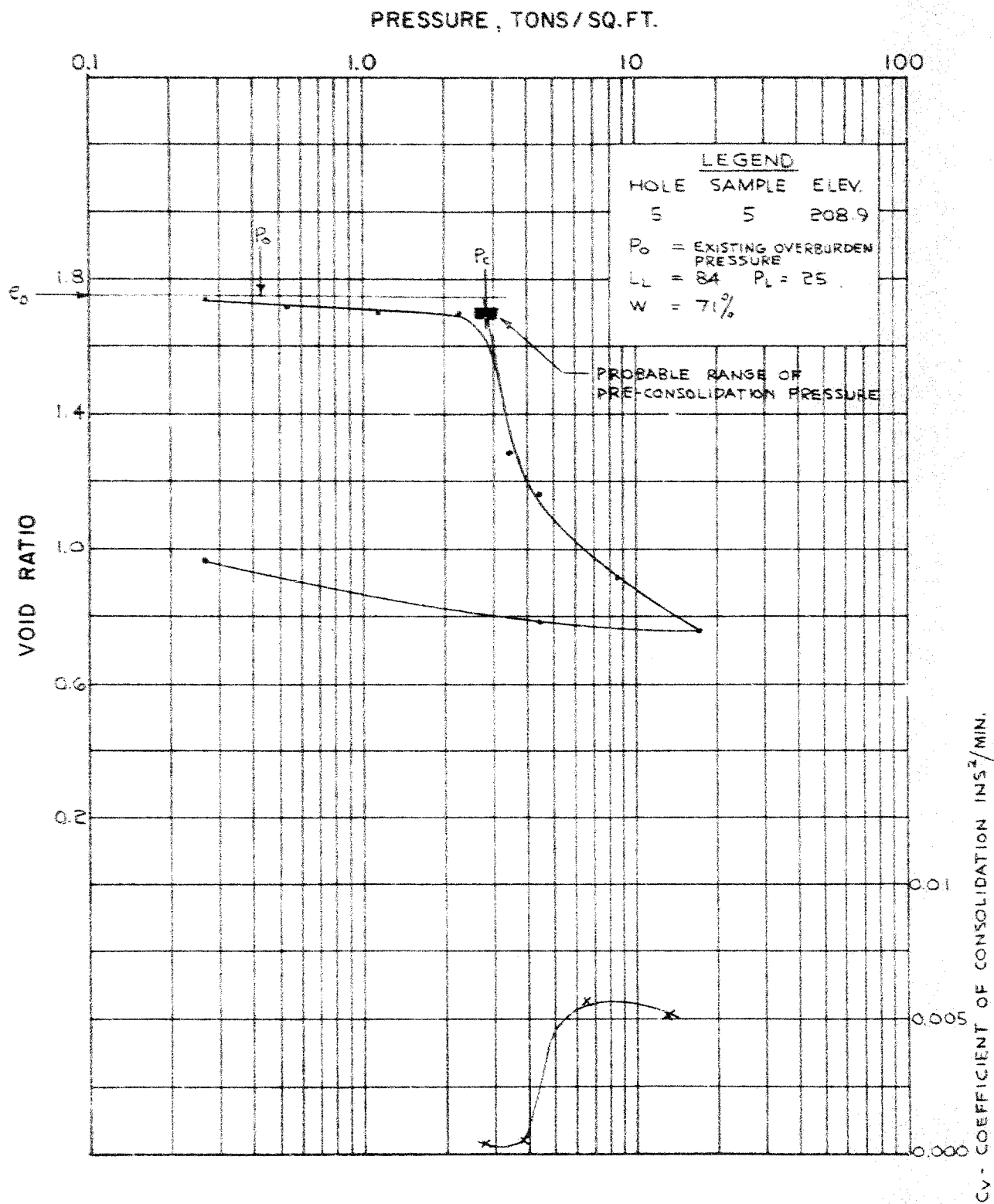
FIGURE 15



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

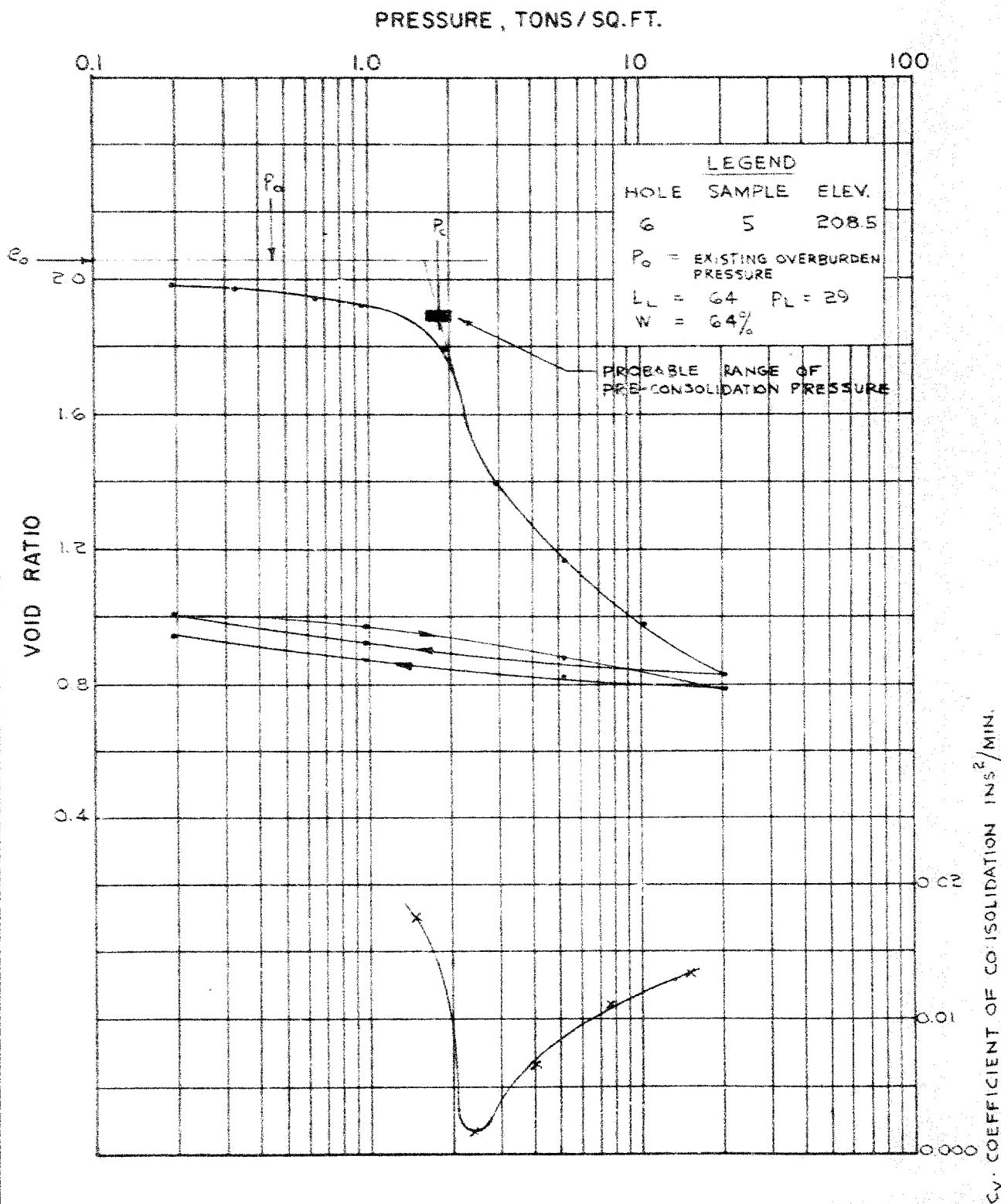
FIGURE 16



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 17

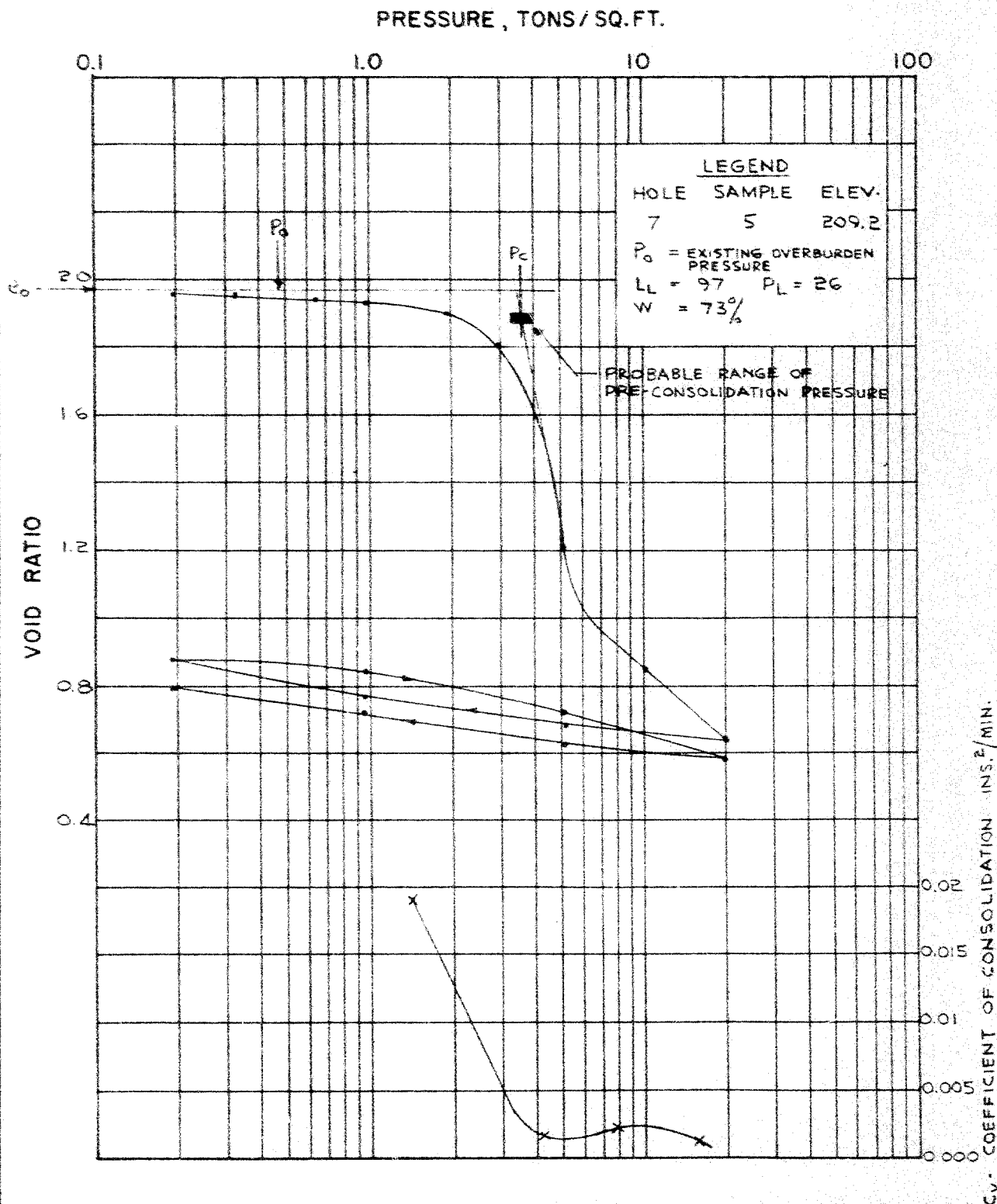


GOLDER & ASSOCIATES

PROJECT No. 6.14.7

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

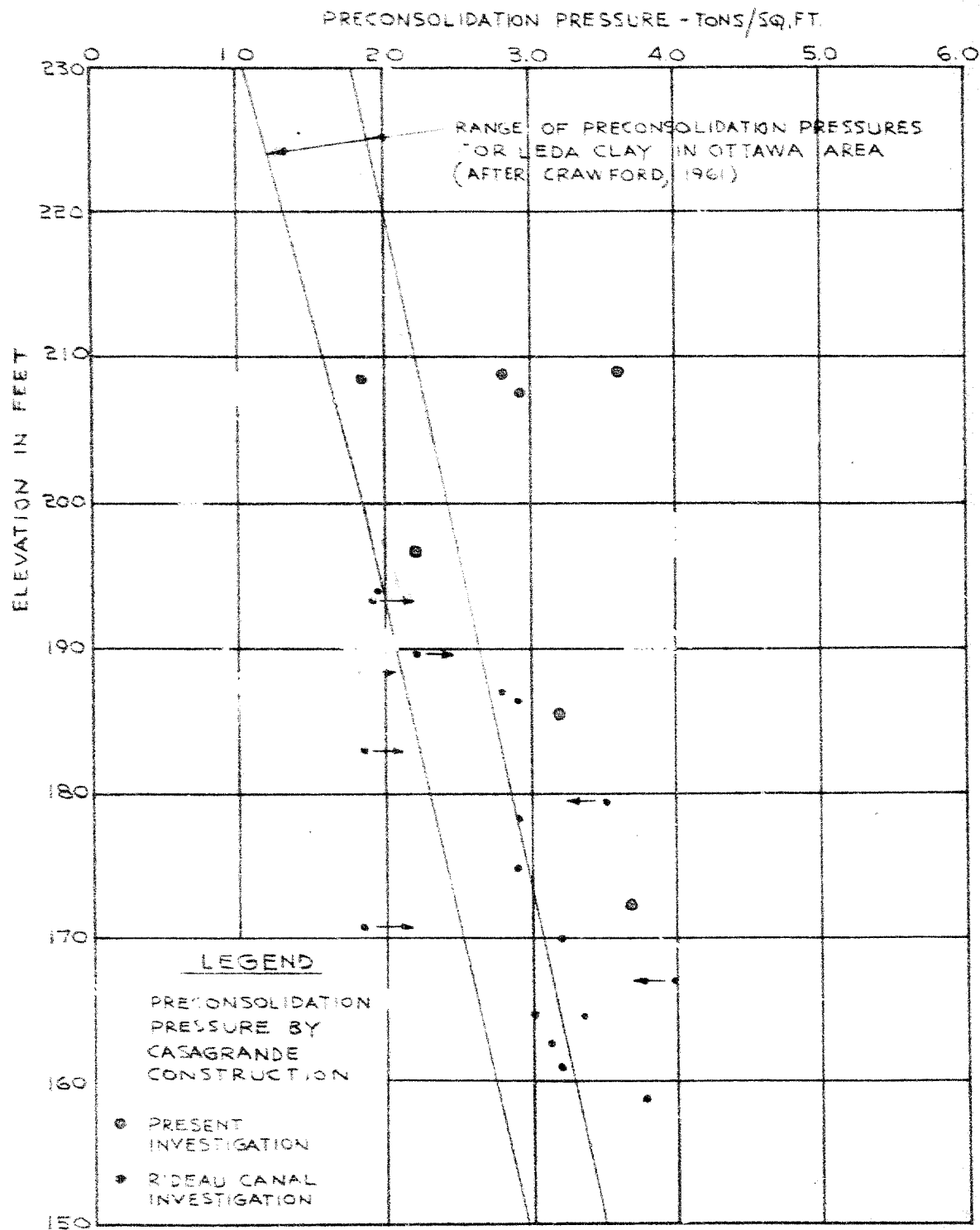
FIGURE 18



GOLDER & ASSOCIATES

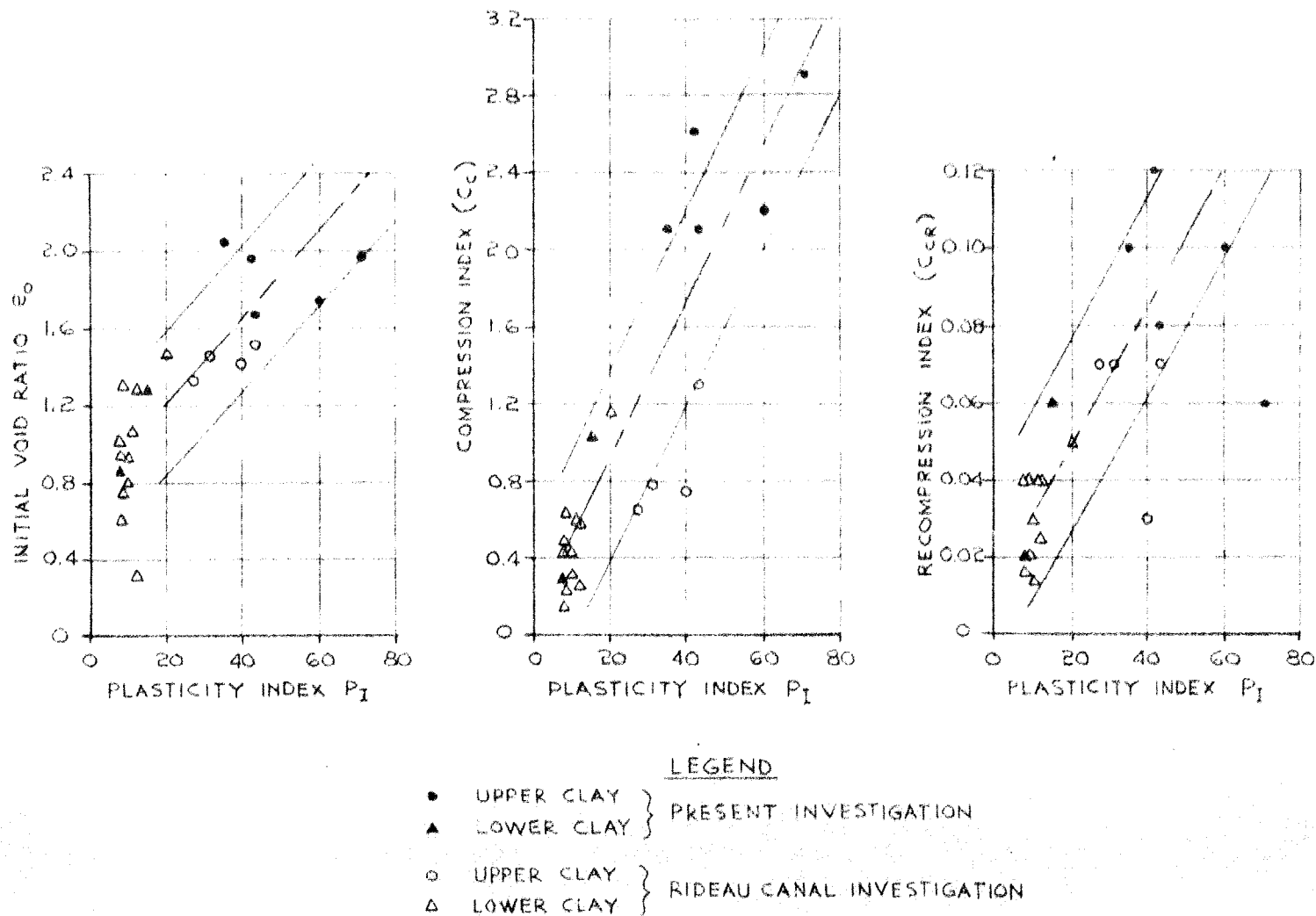
# PRECONSOLIDATION PRESSURE VS ELEVATION CLAY STRATA

FIGURE 19



# CONSOLIDATION PROPERTIES CLAY STRATA

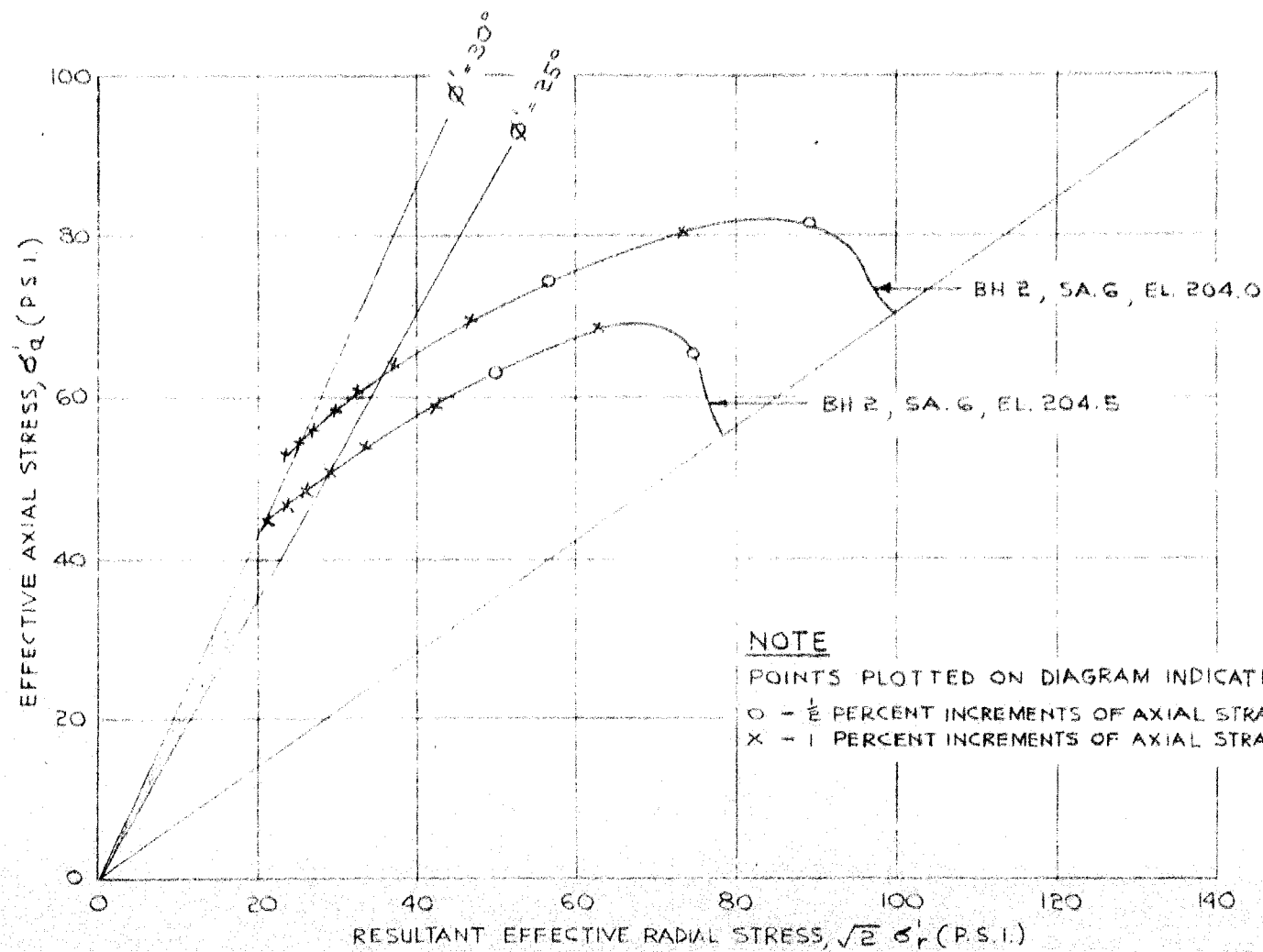
FIGURE 20

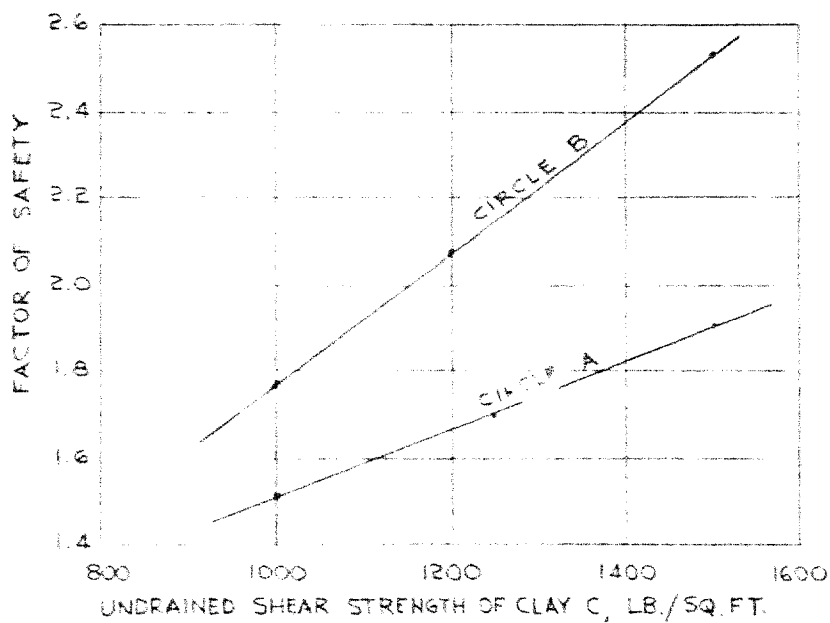
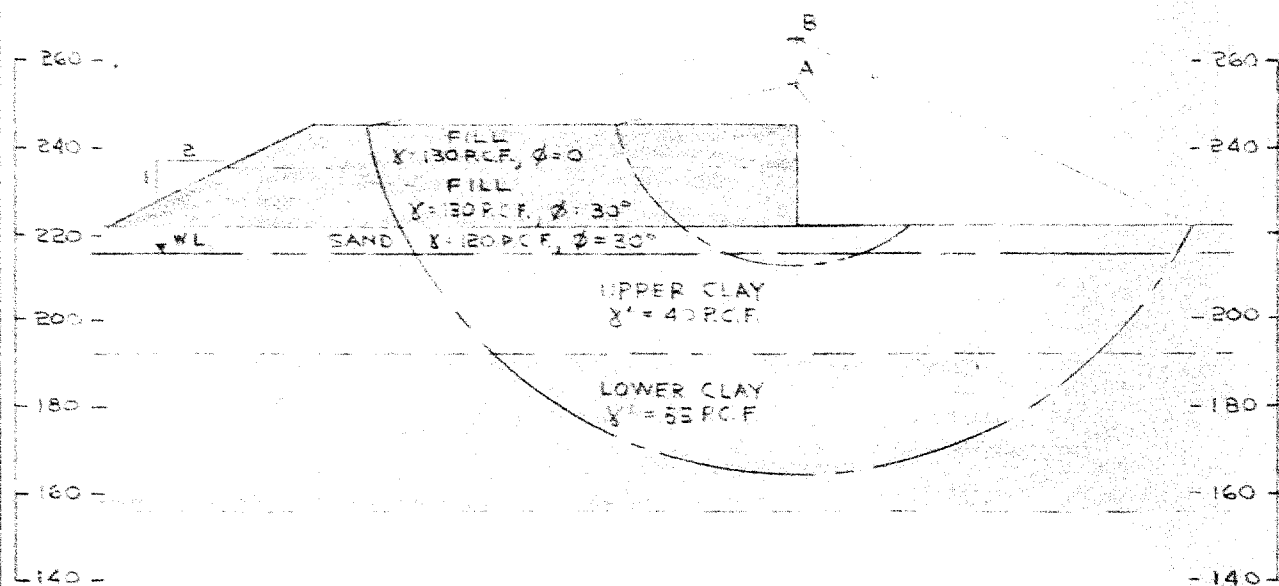




# RENDULIC DIAGRAM CONSOLIDATED UNDRAINED TRIAXIAL TESTS UPPER CLAY

FIGURE 21

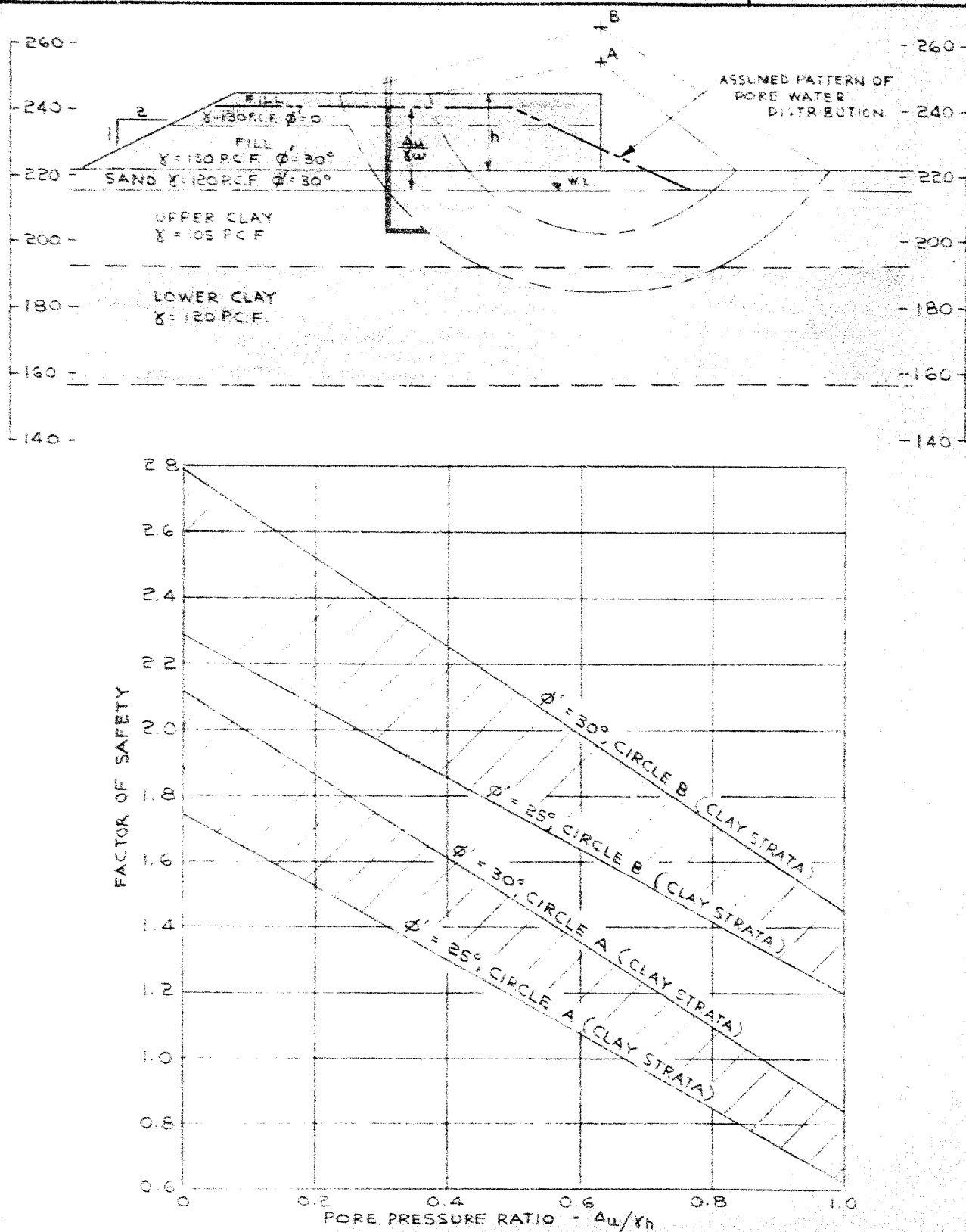




6147

# STABILITY OF EMBANKMENT - STA. 423+00 EFFECTIVE STRESS ANALYSES

FIGURE 23

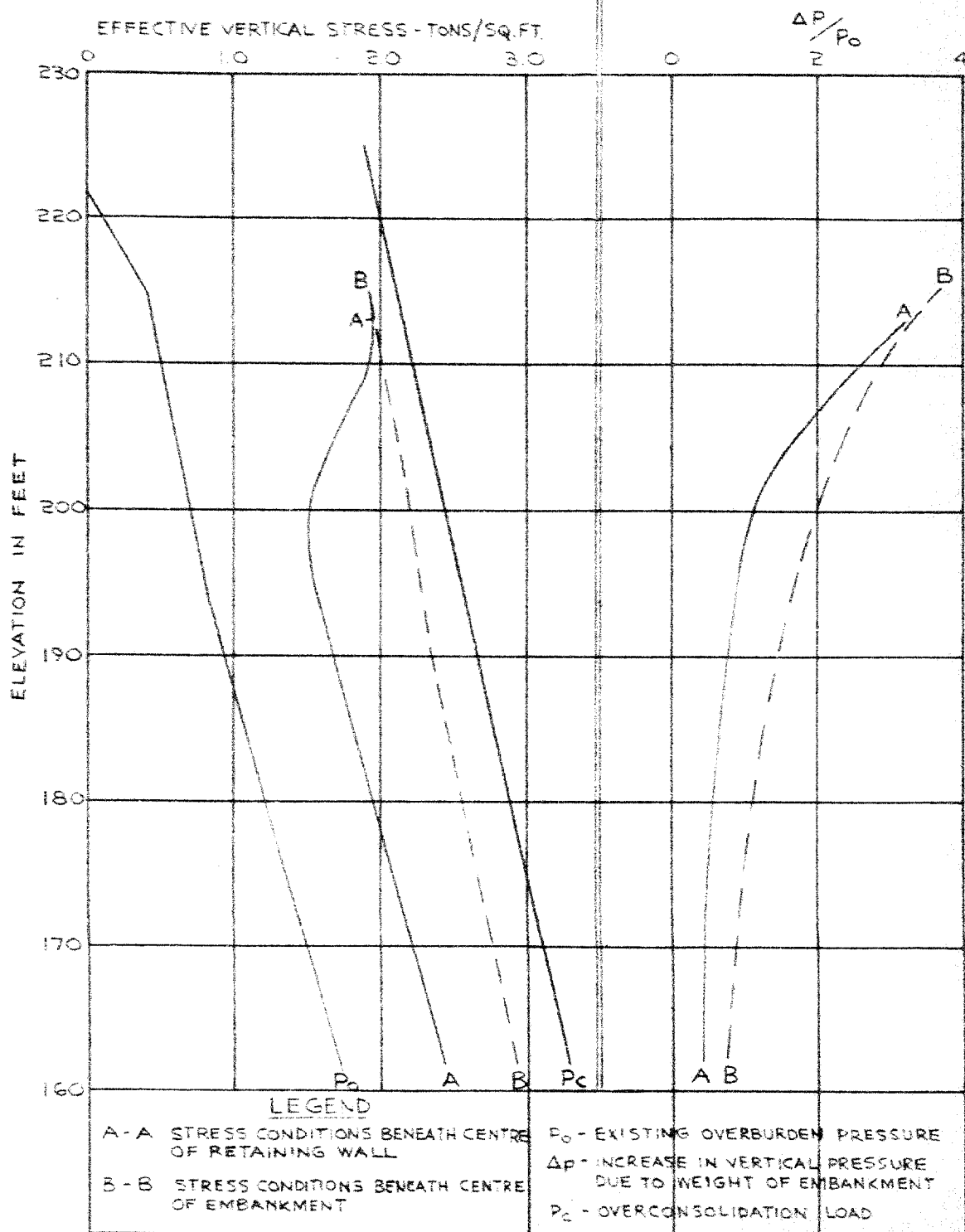


GOLDER & ASSOCIATES

# STRESS DISTRIBUTION - RETAINING WALL

STA. 423+00

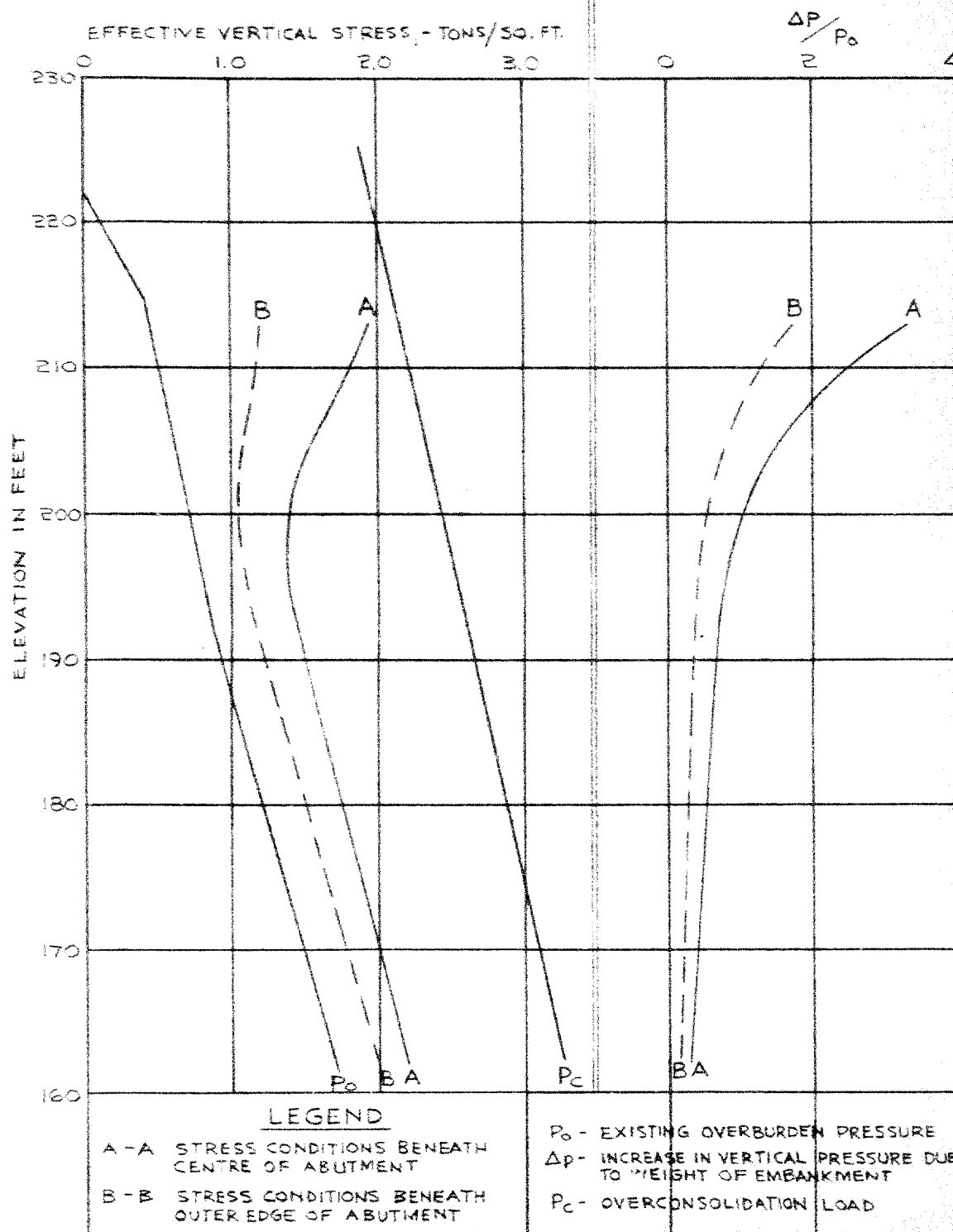
FIGURE 24



GOLDER & ASSOCIATES

## STRESS DISTRIBUTION - MAIN STREET OVERPASS

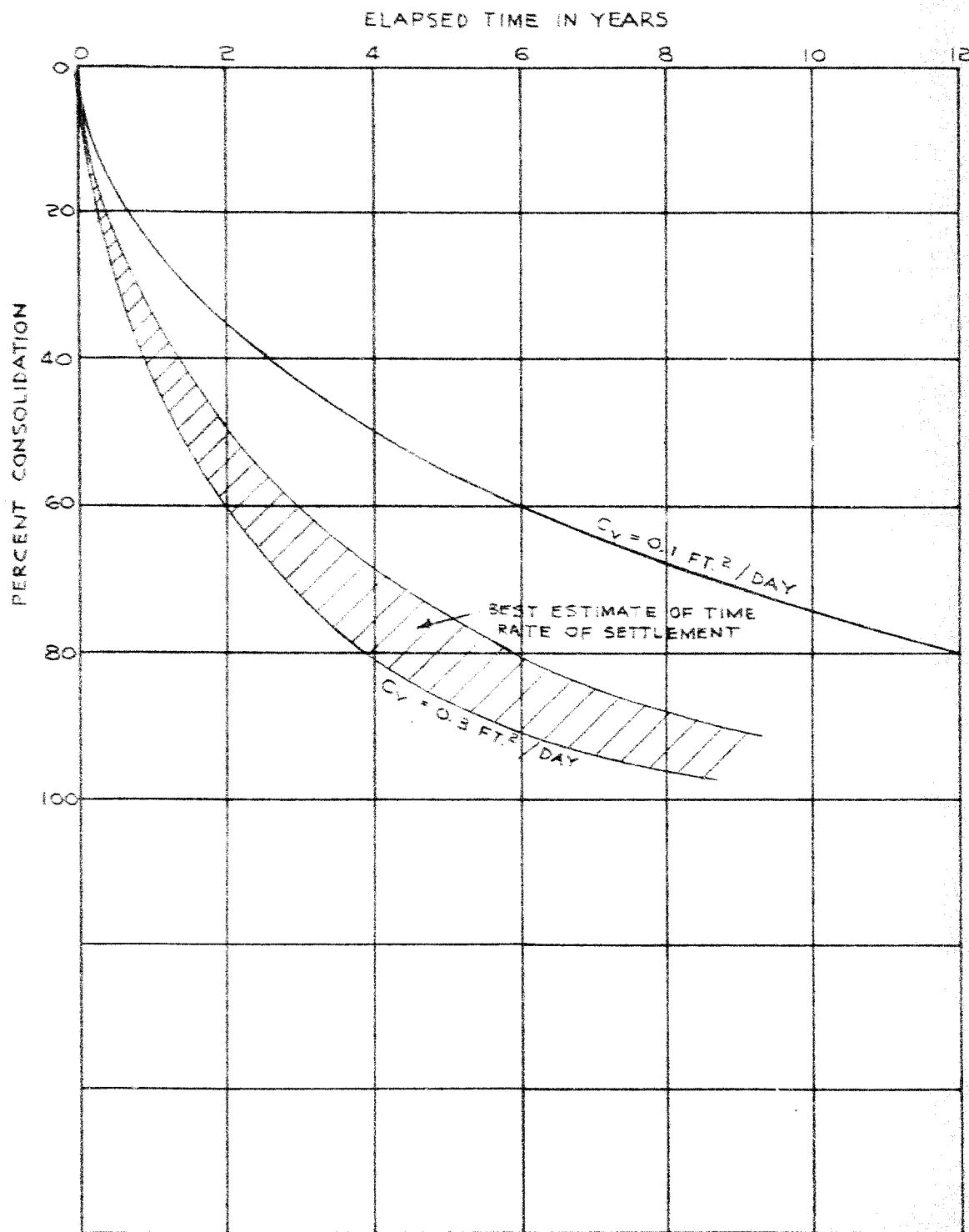
FIGURE 25



PROJECT NO. 8197

# TIME RATE OF SETTLEMENT ESTIMATED FROM RESULTS OF CONSOLIDATION TESTS

FIGURE 26



GOLDER & ASSOCIATES