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REPORT

TO

DELEUW, CATHER & COMPANY OF CANADA LIMITED

OF

SITE INVESTIGATION

PROPOSED QUEENSWAY

RIDEAU CANAL BRIDGE - ELGIN STREET OVERPASS

OTTAWA

ONTARIO.

~~WP 952-59~~

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WP 951-59

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ABSTRACT

The results of investigations of soil conditions at Bridge 23, the Elgin Street Overpass, and Bridge 24, the Rideau Canal Bridge, on the proposed Queensway in Ottawa, Ontario, are reported, and recommendations for the design and construction of foundations and approach embankments are made.

It was found that this section of the Queensway is overlain by a thin layer of fill and silty sand followed by about 50 feet of stiff Leda clay and 50 to 60 feet of compact to dense sandy silt. A thin layer of very dense sandy till was found to lie between the silt and the shale bedrock.

It is recommended that the Rideau Canal Bridge be founded on piles driven into the sandy silt, subject to conditions discussed in the report. The Elgin Street Overpass could be founded on spread footings provided that allowance for some settlement is made in design. The stability of the approach embankments was found to be adequate except in the case of the west approach to the Rideau Bridge where it is recommended that the grade of F.D.C. Drive be raised 5 feet in order to maintain a suitable factor of safety.

It is strongly recommended that the approach embankments plus a small surcharge be placed one year in advance of construction of the bridges in order to minimize differential settlements which could possibly be detrimental to the condition of the structures and adjoining rigid pavement. Recommendations are also made regarding procedures for observing settlement magnitudes and rates at key locations in order to provide a basis for scheduling the various stages of construction.

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INTRODUCTION

H. Q. Golder & Associates Ltd. has been retained by DeLeuw, Cather and Company of Canada limited to carry out site investigations in Ottawa, Ontario. These were for Bridge 23, a proposed overpass at the intersection of the Ottawa Queensway and Elgin Street, and for Bridge 24, a structure proposed to carry the Queensway across the Rideau Canal. The objects of the investigations were to determine soil conditions at the sites, to provide criteria for the foundation design of the proposed structures and to study the stability of the approach embankments.

PROCEDURE

The field work at the two sites was carried out in the period from January 25th to April 11th, 1961. Two boreholes with penetration tests were put down to bedrock at the Elgin Street site plus two additional penetration tests. Five boreholes with penetration tests were put down at the Rideau Canal site, three of these to bedrock, plus four additional penetration tests. These boreholes and penetration tests were put down using conventional drilling equipment. In addition, one borehole was put down at each site using a Swedish foil sampler in order to obtain continuous high quality samples of the clays encountered at both sites.

Detailed logs of each borehole and penetration test are given on the Records of Boreholes at the end of this report. The locations of the boreholes are shown on Figure 1 and sections

showing the inferred soil stratigraphy at the sites are given on Figures 2, 3 and 4.

The soil samples obtained during the site investigations 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 employed in this report were furnished by DeLeuw, Cather and Company of Canada Limited and are referred to Geodetic datum. Boreholes have been located with respect to the centerline chainage of the proposed Queensway shown on DeLeuw, Cather & Company Drawings No. III-A-26A and No. IV-A-8 dated July 1, 1959.

SITE TOPOGRAPHY AND GEOLOGY

The area under consideration extends from about 500 feet west of the Rideau Canal to about 200 feet east of the Canal, a total distance of about 800 feet. This section of the Queensway occupies or closely parallels the right of way of an abandoned line of the Canadian National Railways about 300 feet north of the Pretoria Bridge in Ottawa. General ground surface is at about Elevation 220 falling off to about Elevation 210 in the vicinity of the canal. The bottom of the canal is understood to be at about Elevation 200.

Geological information indicates that bedrock at the site is composed of an Ordovician shale of the Billings Formation.

The bedrock is covered by a thin layer of glacial till deposited by the Labrador ice sheet during the Wisconsin stage of glaciation. Following the retreat of the ice sheet, the depressed land surface in the St. Lawrence and Tributary valleys experienced a marine invasion known as the Champlain Sea. During this period of submergence, thick beds of sands, silts and clays were deposited over the glacial till. Geologically recent deposits of sands and peat overlie the Champlain soils in localized areas.

SOIL CONDITIONS

As shown by the section on Figure 2, the soil conditions were found to vary quite regularly along the length of the area under consideration. The following sequence of significant strata was encountered in the various boreholes:

Fill

Fill consisting of sand to clayey sand with occasional cinders was encountered in Boreholes 23-2, 24-3 and 24-9. This fill, which is associated with the existing railway line, was found to be 19 feet thick in Borehole 24-3 and about 5 feet thick in the other two boreholes. Its relative density corresponds to that of material which is loose to compact. A grading curve for a sample of the fill from Borehole 24-, is shown on Figure 5.

Silty Fine Sand

In the remaining boreholes except Borehole 24-1 and underlying the fill in Borehole 24-3 is a stratum of brown

to grey silty fine sand. Typical grading curves for this material are shown on Figure 5. The upper few feet of this stratum in some cases contained some organic matter, and this layer has been distinguished as sandy topsoil on the Records of Boreholes. Taking the two layers together the total thickness of the stratum ranges from 6 to 10 feet, and its condition based on 'N' values of 3 to 19 is loose to compact.

Upper Clay

Underlying the fill or sand, and at ground surface in Borehole 24-1 is a stratum of highly plastic marine clay which is generally grey in colour, but which is weathered to a brownish grey in the upper 5 to 10 feet. The upper boundary of this clay varied with general ground surface, but the lower boundary ranged consistently between Elevations 185 and 190. Thus the thickness of the stratum varies from 7 to 25 feet in the vicinity of the canal and from 30 to 35 feet at Elgin Street.

The clay is relatively homogeneous in structure and composition; typical grain size curves are given in Figure 6 which shows clay size material predominating with some silt and fine sand. Occasional shell fragments and traces of black organic material were noted in a few samples.

The liquid limit of the upper clay ranges generally from 60 to 80 with some values up to 90. The plastic limit

ranges between narrower limits, 25 to 35, and is generally about 30. Natural moisture contents varied between 50 and 70 percent, but were generally about 60 percent. Corresponding liquidity indices ranged from about 0.5 to 1.0. Wet unit weights of 97 to 111 pounds per cubic foot were measured with an average of about 104 pounds per cubic foot.

The undrained shear strength of the clay was measured by vane testing in the field and by undrained triaxial tests in the laboratory. The results of these tests are plotted on the Records of Boreholes and typical stress strain curves for the triaxial tests are shown on Figure 11. The shear strength results vary widely between about 500 and 2800 pounds per square foot, with the field vane results generally higher than corresponding laboratory values. The sensitivity of the clay measured by the field vane ranged from about 6 to 60; data published by the National Research Council (Crawford, 1961) indicates that the average sensitivity of this clay in the Ottawa area is 24. This high sensitivity makes it very difficult to take completely undisturbed samples and accounts for the wide scatter in measured shear strengths. Reference to the shear strength data plotted on the Records of Boreholes shows that, in general, the lower strengths were obtained at relatively high failure strains which is indicative of sample disturbance.. Figure 11 illustrates this point.

A series of consolidated undrained triaxial tests with pore pressures measured was also carried out in order

to determine the effective stress shear strength parameters; the results of these are plotted on Figure 14 using the method suggested by Rendulic (1937). Again there is considerable scatter in the results, but the plot indicates that the effective angle of shearing resistance, ϕ' , lies between the limits of 25° and 30° . No appreciable cohesion, c' , was indicated.

Four consolidation tests were carried out on samples of this clay and the results are shown on Figures 16 to 19 inclusive. As in the case of the strength tests it is evident that the sensitivity of the clay and consequent sample disturbance has affected the results. However, some overconsolidation is indicated by the curves; the most probable degree of overconsolidation is discussed in a later section of this report.

Lower Silty Clay

A stratum of grey silty clay of low plasticity underlies the upper clay. Its lower boundary also varies within narrow limits from Elevation 152 to Elevation 157, except in borehole 24-3 where it rises to Elevation 165. The thickness of the stratum ranges from 25 to 37 feet with an average of 31 feet.

Although related geologically, the lower clay, as indicated in Figure 7, has less clay and more silt and fine

sand than the upper clay. Its structure is also much less homogeneous than that of the upper clay; frequent layers or lenses of fine sand and silt were observed in the samples as well as organic matter in the form of a black mottling and shell fragments. According to Crawford (1961) the black mottling which is due to the presence of anaerobic bacteria in the clay, disappears upon exposure of the samples to air and is probably of no engineering consequence.

It follows from the composition of the lower clay that its plasticity would be lower than that of the upper clay. Liquid limits ranged from 25 to 45 but were generally of the order of 35; plastic limits were generally about 20. Natural moisture contents varied from 30 to 60 with an average of about 40 percent. Liquidity indices were always greater than unity and usually of the order of 2. Wet unit weights ranged from 108 to 130 pounds per cubic foot with an average of about 114 pounds per cubic foot.

As in the case of the upper clay, the undrained shear strength was measured by vane tests in the field and by tri-axial tests in the laboratory. A similar scatter of results was obtained and the same comments made for the upper clay apply here; typical stress-strain curves for this material are shown on Figure 12. All undrained shear strength test results for both clays are plotted against elevation on Figure 13.

A series of consolidated undrained triaxial tests with pore pressures measured were carried out and the results are plotted on Figure 15. Again there is considerable scatter and it is possible to specify only a range of the effective angle of shearing resistance from 25° to 30° .

Eleven consolidation tests were carried out on samples from this stratum and the results are plotted on Figures 20 to 30 inclusive. The sensitivity and consequent disturbance of the samples has again affected the results, but the same degree of overconsolidation noted for the upper clay is indicated here.

Sandy Silt

Beneath the silty clay is a stratum of grey sandy silt ranging from 35 to 62 feet in thickness. Typical grading curves for this material are shown on Figures 8 and 9. Occasional traces of gravel and thin clayey layers were also observed in samples from this stratum.

Standard penetration resistances in the sandy silt ranged from 6 to 87 with an average of 30 blows per foot. About 90 percent of the recorded values were greater than 10 blows per foot and about 50 percent were concentrated between 10 and 30 blows per foot. Therefore, the stratum is generally compact to dense with occasional loose pockets.

Sand

A layer of grey-brown sand with a trace of gravel about 13 feet thick was encountered beneath the sandy silt in Borehole 24-10. It was not encountered in any of the other boreholes. Standard penetration resistances of 44, 58 and 61 blows per foot were measured in the stratum. Allowing for the effect of the gravel it is estimated that the relative density of the stratum is high.

Sandy Till

A thin layer of brown glacial till 2 to 5 feet thick was encountered beneath the sandy silt. As shown in Figure 10 it is composed predominantly of sand with some gravel and a trace to some silt. The stratum is very dense.

Bedrock

Underlying the till is a sound dark grey shale bedrock. It was encountered at an elevation of 114 in Borehole 24-2 at the eastern edge of the site and was found to drop off gradually to the west to Elevation 88.

Groundwater

Observation pipes were installed in three of the boreholes and water levels were recorded periodically during April, May, and June 1961. The observed elevations

of water levels were as follows:

		<u>BH 23-1</u>	<u>BH 24-1</u>	<u>BH 24-4</u>
April	7	-	-	204.1
	11	207.6	202.3	206.0
	19	218.5	200.7	209.9
	26	215.6	202.2	208.2
May	3	215.9	201.7	210.0
	10	Hose Pulled	202.2	209.3
	17	204.6	199.6	210.1
June	1	206.6	201.2	209.5
	7	-	200.7	209.3

It is considered that of the three sets of readings the levels in Borehole 24-1, which fluctuated around Elevation 201, are the more representative of groundwater conditions in the clay. In both Boreholes 23-1 and 24-4 the clay was overlain by granular material which made them difficult to seal against surface water during the spring thaw and, in the case of Borehole 24-4, against seepage from the Rideau Canal. However, in Borehole 24-1 the clay was encountered at ground surface and an effective seal against the entrance of surficial water could be made.

RIDEAU CANAL BRIDGE - DISCUSSION

General

The Rideau Canal Bridge has been proposed as a three span steel plate girder and reinforced concrete structure approached by earthfill embankments at a grade of about Elevation 240 or about 30 feet above general ground level. The locations of the abutments and central piers of the

bridge would be approximately as shown in Figure 1. The existing F.D.C. Drive is to be re-routed under the Queensway through the west approach span at a grade of about Elevation 206. Similarly, Echo Drive will be routed under the east approach span at about Elevation 215. It is understood that the maximum loads which would be transferred by the structure to the underlying soil at foundation level would be 64 kips per lineal foot along the centerline of each pier and 59 kips per lineal foot at the abutments.

Stability of Approach Embankments

The stability of the structure, as proposed, will depend greatly on the stability of the approach embankments. The most critical period for the stability of the embankment will be when the embankment is first brought to full height; in the time following construction the factor of safety will increase as the pore pressures induced in the underlying clay by the fill are allowed to dissipate. An analysis based on total stresses is a valid method for predicting the initial stability of an embankment founded on lightly overconsolidated clay, but does not provide the means for evaluating the factor at any given time during or following the construction period. Furthermore, the possible effects of other concurrent construction procedures such as pile driving on stability cannot be assessed. In order to evaluate the factor of safety for these conditions it is necessary to analyse stability in terms of

the effective stresses within the soil using the effective shear strength parameters. However, it is not generally possible to predict accurately the pore pressures which will develop during construction on the basis of laboratory tests alone. Therefore, the most practical step is to design the embankment initially on the basis of the undrained shear strength and to control its construction by effective stress analyses based on the measured field pore pressures.

The most critical section for analysis of stability has been taken on the west bank of the canal as shown in Figure 31. Two general modes of failure have been considered, one along a deep circular arc extending into the canal (Circle A) and the other along a shallow arc extending only into the F.D.C. Drive (Circle B).

The importance of the effective stress analysis to the control of construction of the west approach embankment is shown by a series of analyses carried out for assumed pore pressure distributions. The results of these analyses, which are summarized on Figures 32 and 33, indicate that, if the embankment were constructed very rapidly, thus inducing high pore pressures in the underlying clay, the factor of safety for the embankment could be lower than unity. Figure 32 also indicates that the super-position of additional pore pressures induced by, for example, pile driving could have a serious effect on stability.

For embankments constructed at a normal rate, the most reasonable estimate of stability has been found empirically to be based on the undrained shear strength, c , of the clay. A summary of all shear strengths measured in the clay and silty clay strata is presented on Figure 13. No characteristic pattern of shear strength with depth is apparent because of the wide scatter in results noted previously. However, with the exception of results from obviously disturbed samples, the shear strengths range generally from 1,000 to 1,400 pounds per square foot with an average of about 1,200 pounds per square foot. The average strength may be assumed for design, although a possible minimum strength of 1,000 pounds per square foot must be kept in mind.

A summary of the computed factors of safety for various assumed shear strengths is plotted for the two modes of failure on Figure 31(a). From this it may be seen that the more critical mode of failure is a shallow circle which gives a factor of safety of about 1.3 for the assumed design strength of 1,200 pounds per square foot. However, it is normal practice to require a minimum factor of safety of 1.5 with respect to the design strength, and in this case, in view of the range of strengths noted above it is recommended that a minimum factor of safety of 1.3 be maintained for a shear strength of 1,000 pounds per square foot.

The most expedient way in which to increase the factor of safety of the slope would be to raise the level of the F.D.C. Drive, thus providing in effect a counterbalancing berm. In Figure 31(b) is shown the effect of raising the grade of F.D.C. Drive on the stability of the critical circle (Circle B). From this it may be seen that with the Drive at an elevation of 211 (i.e. 5 feet above the desired grade) the above criteria would be satisfied. It is recommended, therefore, that this change be incorporated in the final design.

The above analyses did not include the possible effects of the structural loads on the stability of the embankment. In the case of the deep failure arc (Circle A) there would be no significant net change in stability because the pier and abutment loads would tend to cancel each other. However, as indicated in Figure 31(a) the factor of safety in the case of Circle B could be reduced by about 20 percent with the addition of the abutment load; this point must be considered in the selection of the foundations for the structure as discussed below.

Comparison of the effective stress analyses with the results of the undrained analyses is difficult because of the difficulty in predicting the pore pressures which would be induced by the construction of the embankment. However, recent observations of pore pressures induced by the construction of an embankment of similar size on foundation

soil of a similar nature suggest that for a normal rate of construction the maximum ratio of induced pore pressure to the maximum stress exerted on the foundation by the embankment (i.e. maximum $\frac{\Delta u}{\sigma' h}$ as defined in Figure 32) is of the order of 0.5. Entering the charts in Figure 33 for the case where F.D.C. Drive is at the recommended elevation 211, the factor of safety at a pore pressure ratio, $\frac{\Delta u}{\sigma' h}$ of 0.5 would be about 1.4 for an effective angle of shearing resistance, ϕ' of 25° and about 1.5 for ϕ' of 30° . The corresponding factors of safety for the undrained analysis for shear strengths of 1,000 to 1,200 pounds per square foot are about 1.3 to 1.5. Although this comparison is by no means conclusive, the agreement between the two methods is reasonable in view of the assumptions involved.

Settlement of Approach Embankments

The other points which must be considered in the case of the approach embankments are the probable magnitudes and time rates of their settlement due to consolidation of the underlying clay strata. For the embankments under consideration, the most important factor affecting settlement is the relationship of the applied embankment loads to the preconsolidation load, the maximum load under which the clays have been consolidated in the past.

The preconsolidation loads computed from the consolidation tests shown on Figures 16 to 30 inclusive are plotted

against elevation on Figure 34 together with preconsolidation loads computed from tests made by the National Research Council on samples of the clay given to them from Borehole 24-9. Also shown on Figure 34 is a range of preconsolidation loads for the Ottawa area reported by Crawford (1961) plotted against elevation. The range of values shown is not so much representative of the possible variation in preconsolidation load at a particular elevation as it is indicative of the difficulty of measuring the preconsolidation load in the laboratory; the sensitivity of Leda clay and consequent difficulty in obtaining relatively undisturbed samples has been noted previously. Considerable refined laboratory testing supplemented by data obtained from field studies indicates that the preconsolidation load in the Ottawa area is at least as great as that given by the upper limit shown in Figure 34 (See Crawford, 1961). Disregarding data from disturbed samples, the results from the present investigation tend to confirm that use of the upper limit is reasonable for design purposes.

The existing overburden stresses in the clay and the stresses which would be induced beneath the central portion of the embankment are shown on Figure 35. This figure indicates that the new stress level under the embankment would closely follow the line of preconsolidation load throughout the clay strata and that the settlement of the embankment would be due only to reconsolidation in the clay. The value

of ultimate settlement estimated for the embankment based on the compression indices from the reconsolidation portion of the curves in Figures 16 to 30 is approximately 3 inches.

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 0.4 square feet per day; the resultant range in time rate of settlement is shown on Figure 36. The best estimate, based on an average coefficient of consolidation of 0.26 square feet per day, is also shown, and it is likely that the average rate of settlement would range between this and the indicated lower limit. Thus about 50 percent of the settlement would take place in the first year to year and a half after construction with the balance occurring over the following three and one half to five years.

It is understood that this section of the Queensway must be paved immediately upon completion of the structures. In order to avoid cracking of a rigid pavement due to differential settlements, particularly in the vicinity of the structure, it is strongly recommended that the approach embankments be constructed as soon as practicable. If this were done far enough in advance of the scheduled time for paving, a substantial portion of the settlements due to consolidation of the underlying clay would be taking place in the period before and during the construction of the struc-

tures. For example, the allowance of about one year between the construction of the embankments and paving, would mean that about 50 percent of the settlement would have taken place, and the remaining inch or so would be spread over a 3 to 5 year period.

The actual rate of settlement of the embankments can only be ascertained by installing settlement gauges and piezometers before construction and taking regular observations during and following construction. The information from the piezometers would also afford a check on stability as discussed above. The piezometers should be installed in sets of three with one piezometer approximately in the center of each clay stratum and one at the boundary between the two strata. Settlement gauges should be established at the surface of the upper clay stratum.

Foundations

It is recommended that the abutments and piers of the Rideau Canal bridge be founded on piles driven into the compact to dense sandy silt stratum underlying the lower clay. The piers, because of their importance to the structure, should not be so founded that they would be influenced by possible instability in the canal banks or by settlement. Similarly, if the abutments were founded on spread footings in the fill, it has been shown that they would tend to re-

duce the factor of safety of the embankment, perhaps by as much as 20 percent.

The piers should be founded at about Elevation 210 on steel H piles driven at least 15 feet into the sandy silt; this would mean a required pile length of the order of 70 to 80 feet. Steel H piles are recommended in order to minimize displacement and consequent disturbance of the sensitive clays during driving. Assuming that 12 inch sections are employed, an allowable load of 40 tons per pile may be used for design. Final design loads, however, should be based on full-scale load tests, and it is recommended that at least 3 such tests be carried out.

The abutments may also be founded on piles driven into the sandy silt and a load of 40 ton per pile may be assumed for design, provided that the approach embankments are placed far enough in advance of the construction for a substantial portion of the resultant consolidation of the underlying clay to have taken place. If the embankments are not constructed at least a year in advance of construction of the bridge, then negative friction on the piles due to consolidation under the fill might approach the allowable load. This would mean that the piles would have to be driven to refusal in the sandy till or bedrock in order to develop sufficient capacity to carry the structural loads without appreciable settlement at the pile tips.

Recommended Construction Procedure

In view of the above discussion it is recommended that the construction of the Rideau Canal Bridge and associated approach embankments be undertaken in the sequence and in accordance with the procedures outlined below.

It is essential that the first step in construction be the installation of the piezometers and settlement gauges. This installation should be completed several weeks before construction of the approach embankment is commenced.

In accordance with the previous discussion it is recommended that the approach embankments be placed at least one year in advance of construction of the structure. It is further recommended that in the last 100 feet before each abutment the embankments be constructed to a grade which is 3 feet in excess of the final desired grade. This surcharge of 3 feet will effectively increase the rate of settlement in this area and will tend to minimize any secondary consolidation which may take place in the clay. In order to ensure adequate stability of the surcharged embankment, the general ground surface in the vicinity of the embankment should be maintained at a grade of not less than Elevation 214 for the period preceeding final construction of the structure.

The embankment 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 it is recommended that not more than 2 lifts be placed in a 24 hour period.

The pile loading tests may be carried out after the construction of the embankments as soon as the rate of pore pressure dissipation in the clay has been established from the piezometer observations. As noted previously, there should be three tests carried out and it is suggested that one of these be carried out at each pier location and the third at the west abutment. The effects of pile driving on the piezometric readings, particularly at the west abutment, should be carefully noted.

The driving of piles for the main piers may take place as soon as the results of the pile loading tests have been interpreted. However, the driving schedule should be arranged so that the piles for the abutments are not driven until a year has elapsed from the construction of the embankment. Analysis of the observations from piezometers and settlement gauges may allow some revision of this schedule.

Following the driving of the piles construction of the bridge superstructure may proceed normally.

ELGIN STREET OVERPASSGeneral

The Elgin Street overpass has been proposed as a reinforced concrete rigid frame with a free span of about 68 feet. The final grade of the Queensway at this location is about Elevation 240 with Elgin Street to pass under at about Elevation 217. The approach embankments, which are essentially a continuation of the approach embankment to the Rideau Canal Bridge, will be about 20 feet above general ground surface. It is understood that structural loads at each abutment would be about 48 kips per lineal foot.

Approach Embankments

From the analysis of the embankment at the Rideau Canal Bridge it is clear that the stability of the Elgin Street approach embankments, which are lower in height, will be adequate. Therefore, the only point of possible concern here will be the settlement. It is estimated that this would be a maximum of about 2 inches and would take place over the same time period as indicated on Figure 36. If the embankments are constructed one year prior to construction of the structures, as previously recommended, then about 50 percent of the settlement should have taken place before any paving is required.

Foundations

It is recommended that the overpass structure be founded on spread footings in the upper portion of the clay stratum. Allowing for 6 feet of earth cover to prevent frost action, the foundation grade would be at about Elevation 211. Based on a design shear strength of 1,200 pounds per square foot and taking account of the additive effect of the earth cover, the allowable bearing capacity for foundation design is 3,000 pounds per square foot.

Allowance must be made in the design of the structure for some differential settlement due to consolidation of the clay under the combined weight of embankment and structure. The stresses which these would induce beneath the center of the abutment and the end of the abutment are shown on Figure 37. Based on these pressure distributions the maximum estimated settlement under the center of each abutment is about 2 inches and about 1 inch under the ends. Therefore maximum differential settlements in the form of tilting of about 1 inch between the center and the ends of each abutment should be allowed for in design, and about $1\frac{1}{2}$ inches across the span. It is significant to note that preconstruction of the embankments, as recommended, would reduce these differential settlements to about half of the estimated value. A further point to be considered is that, if the embankments were not only preconstructed, but also carried completely across Elgin Street and surcharged by

about 5 feet of fill, the differential settlements in the structure would probably be reduced to a negligible fraction of the maximum estimated. This portion of the embankment could then be removed when necessary to permit construction of the overpass. The estimated time rate of settlement would be that indicated by the faster limit shown on Figure 36.

The abutments should be backfilled with free-draining granular material for at least 6 feet in horizontal extent. For the computation of lateral earth pressures on the abutment 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 abutment and with heavy compaction of the embankment fill, the unit weight of the fill material may be taken as 135 pounds per cubic foot.

Recommended Construction Procedure

It is recommended that the approach embankments be placed at least one year prior to construction of the overpass. As in the case of the Rideau Canal Bridge, piezometers and settlement gauges should be installed near one abutment before filling commences. Comments have been made above regarding surcharge and the extent of fill.

With regard to the foundations for the overpass, it is recommended that a working mat of about 4 inches of lean concrete be poured immediately after the excavation for the footings has been completed. This is to prevent possible softening of the upper part of the sensitive clay due to entrance of surface water and disturbance due to construction operations.

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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

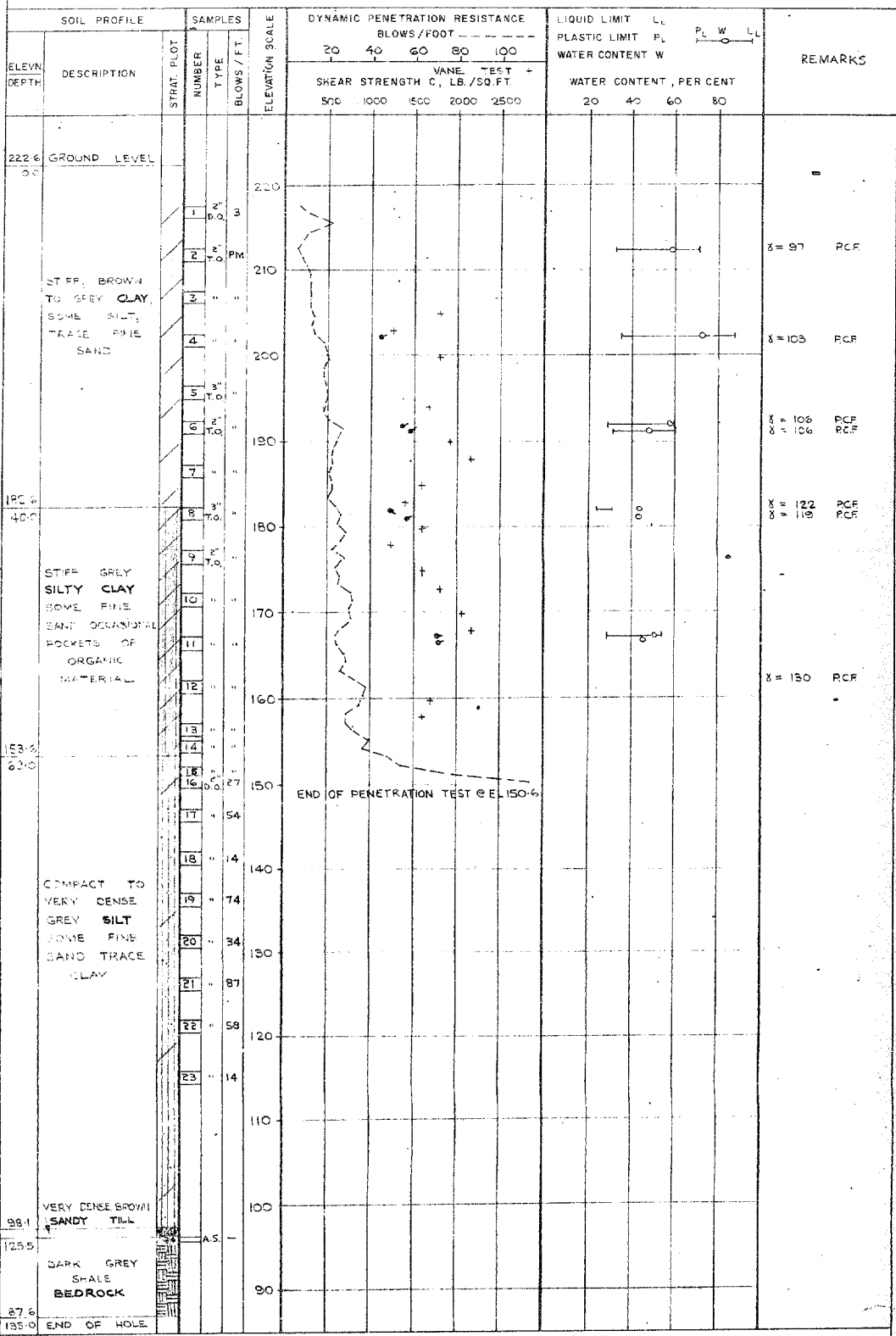
Note: Undrained triaxial tests in which pore pressures are measured are shown as Q' or Q'c.

SOIL PROPERTIES

γ - Total Unit Weight	K - Coefficient of Permeability
γ_d - Dry Unit Weight	c - Undrained Shear Strength (1/2 Compressive Strength)
γ_b - Submerged Unit Weight	St - Sensitivity
L _L - Liquid Limit	ϕ' - 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	

RECORD OF BOREHOLE 23-1

LOCATION SEE FIGURE BORING DATE FEB. 22 - MARCH 1, 1961 DATUM GEODETIC
 BOREHOLE NO. PE POWER AUGER BORING BOREHOLE DIAMETER 4.5"
 SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



(a) Dynamic penetration resistance converted to 4200 inch lb. energy
 (b) Abbreviations listed on page

VERTICAL SCALE
 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
 CHECKED

RECORD OF BOREHOLE 23-2

LOCATION SEE FIGURE

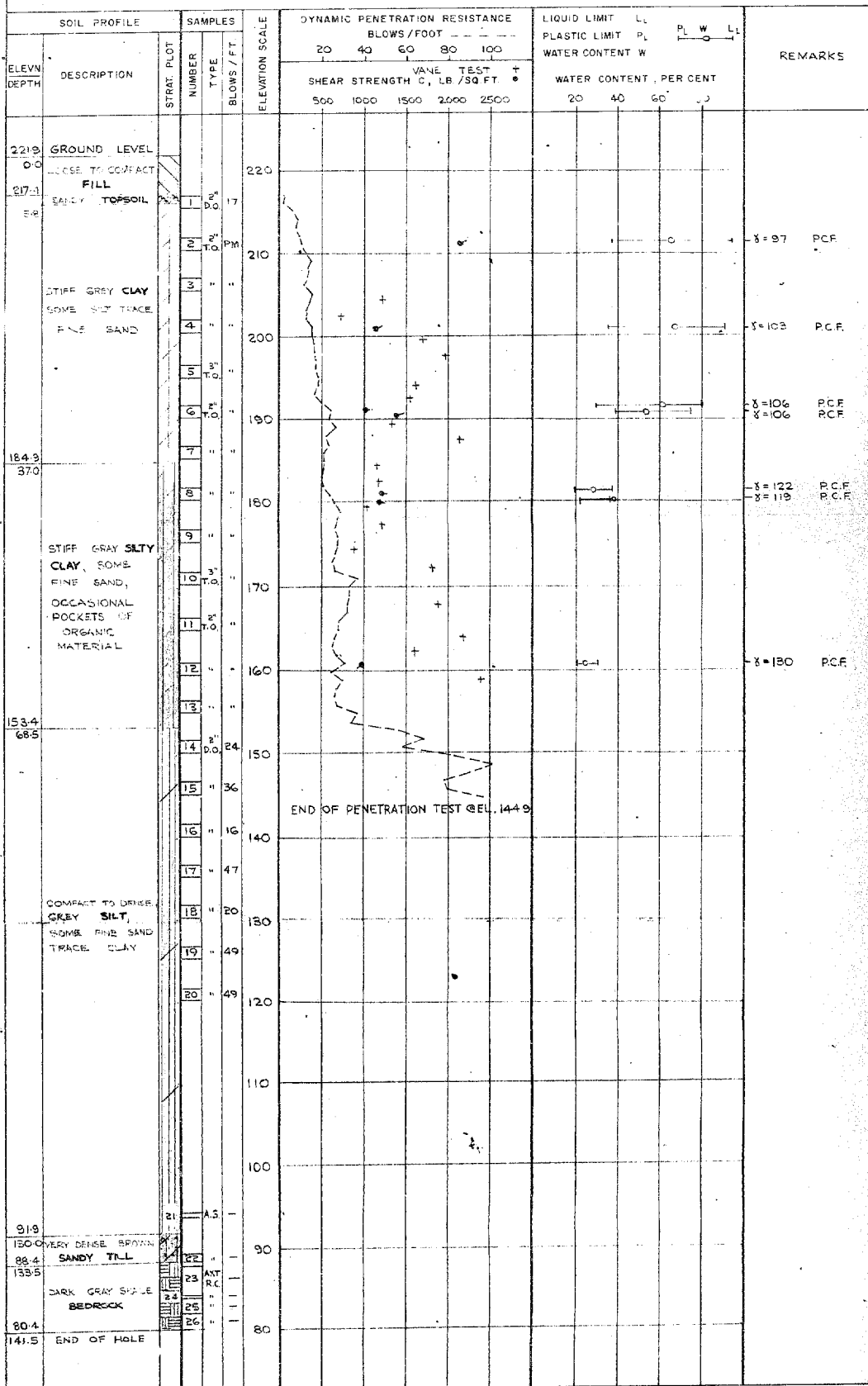
BORING DATE MARCH 2 - MARCH 16 1961 DATUM = GROUND C

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



(a) Dynamic penetration resistance converted to 4200 inch lb. energy

(b) Abbreviations listed on page

VERTICAL SCALE
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED *170*

RECORD OF BOREHOLE 23-3

LOCATION	SEE	FIGURE
----------	-----	--------

BORING DATE MARCH 2, 1961

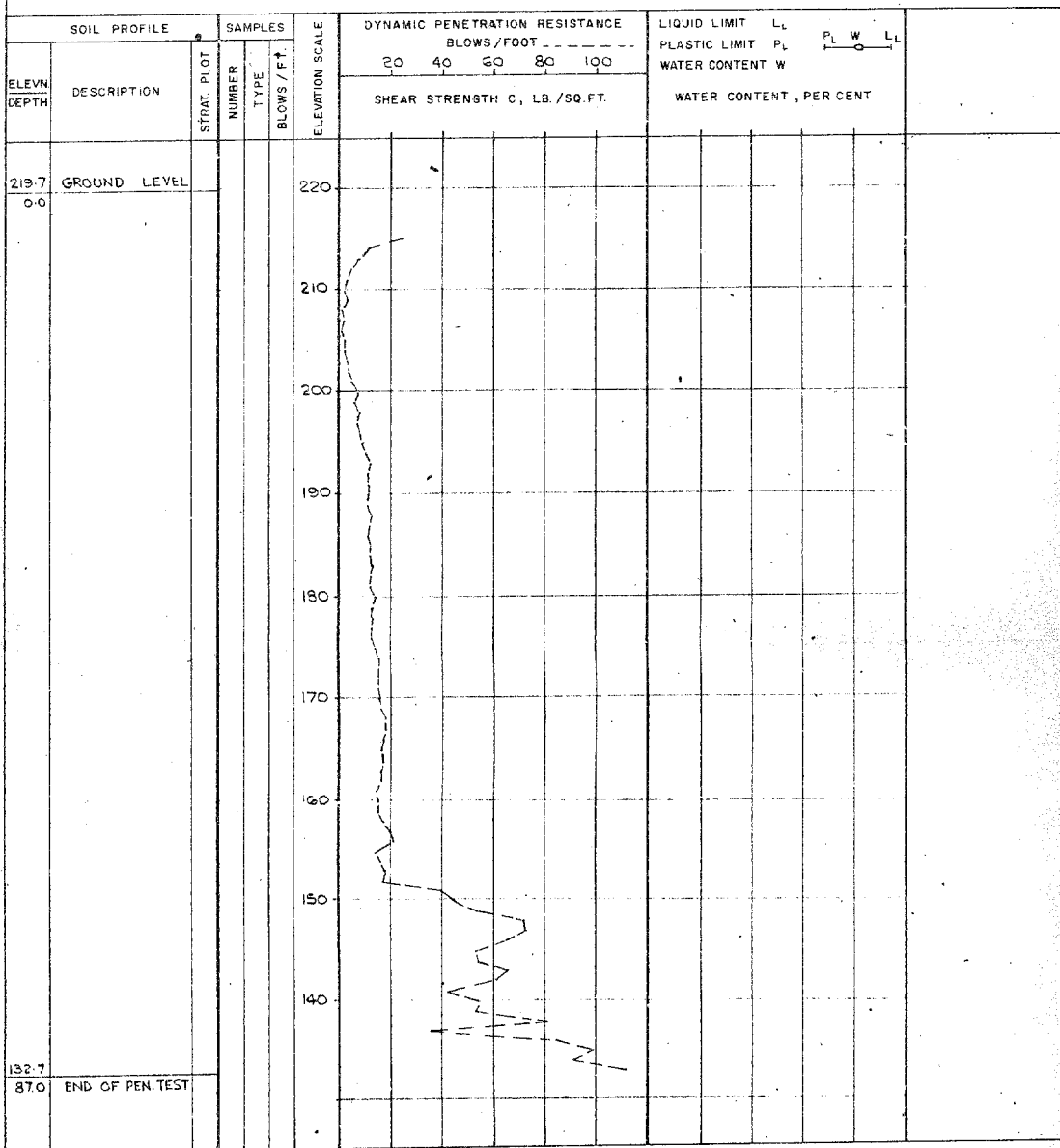
DATUM GEODETIC

BOREHOLE TYPE	PENETRATION	TEST
1	1	1
2	2	2
3	3	3
4	4	4
5	5	5
6	6	6
7	7	7
8	8	8
9	9	9
10	10	10
11	11	11
12	12	12
13	13	13
14	14	14
15	15	15
16	16	16
17	17	17
18	18	18
19	19	19
20	20	20
21	21	21
22	22	22
23	23	23
24	24	24
25	25	25
26	26	26
27	27	27
28	28	28
29	29	29
30	30	30
31	31	31
32	32	32
33	33	33
34	34	34
35	35	35
36	36	36
37	37	37
38	38	38
39	39	39
40	40	40
41	41	41
42	42	42
43	43	43
44	44	44
45	45	45
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47	47	47
48	48	48
49	49	49
50	50	50
51	51	51
52	52	52
53	53	53
54	54	54
55	55	55
56	56	56
57	57	57
58	58	58
59	59	59
60	60	60
61	61	61
62	62	62
63	63	63
64	64	64
65	65	65
66	66	66
67	67	67
68	68	68
69	69	69
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72	72	72
73	73	73
74	74	74
75	75	75
76	76	76
77	77	77
78	78	78
79	79	79
80	80	80
81	81	81
82	82	82
83	83	83
84	84	84
85	85	85
86	86	86
87	87	87
88	88	88
89	89	89
90	90	90
91	91	91
92	92	92
93	93	93
94	94	94
95	95	95
96	96	96
97	97	97
98	98	98
99	99	99
100	100	100

BOREHOLE DIAMETER

SAMPLER HAMMER WEIGHT	LB.	DROP	INCHES
-----------------------	-----	------	--------

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



(g) Dynamic penetration resistance converted to 4200 inch lb. energy

(b) Abbreviations listed on page

VERTICAL SCALE
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED *PK*

RECORD OF BOREHOLE 23-4

LOCATION SEE FIGURE BORING DATE FEB. 22, 1961 DATUM GEODETIC
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER -
 SAMPLER HAMMER WEIGHT - LB DROP - INCHES PEN TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT L _L PLASTIC LIMIT P _L WATER CONTENT W		REMARKS	
ELEVATION DEPTH	DESCRIPTION	STRAT. PLOT NUMBER	TYPE		20	40	60	80		100
222.7 0.0	GROUND LEVEL			220						
				210						
				200						
				190						
				180						
				170						
				160						
				150						
				140						
				130						
				120						
116.7 106.0	END OF PEN. TEST			110						

(a) Dynamic penetration resistance converted to 4200 inch lb energy
 (b) Abbreviations listed on page

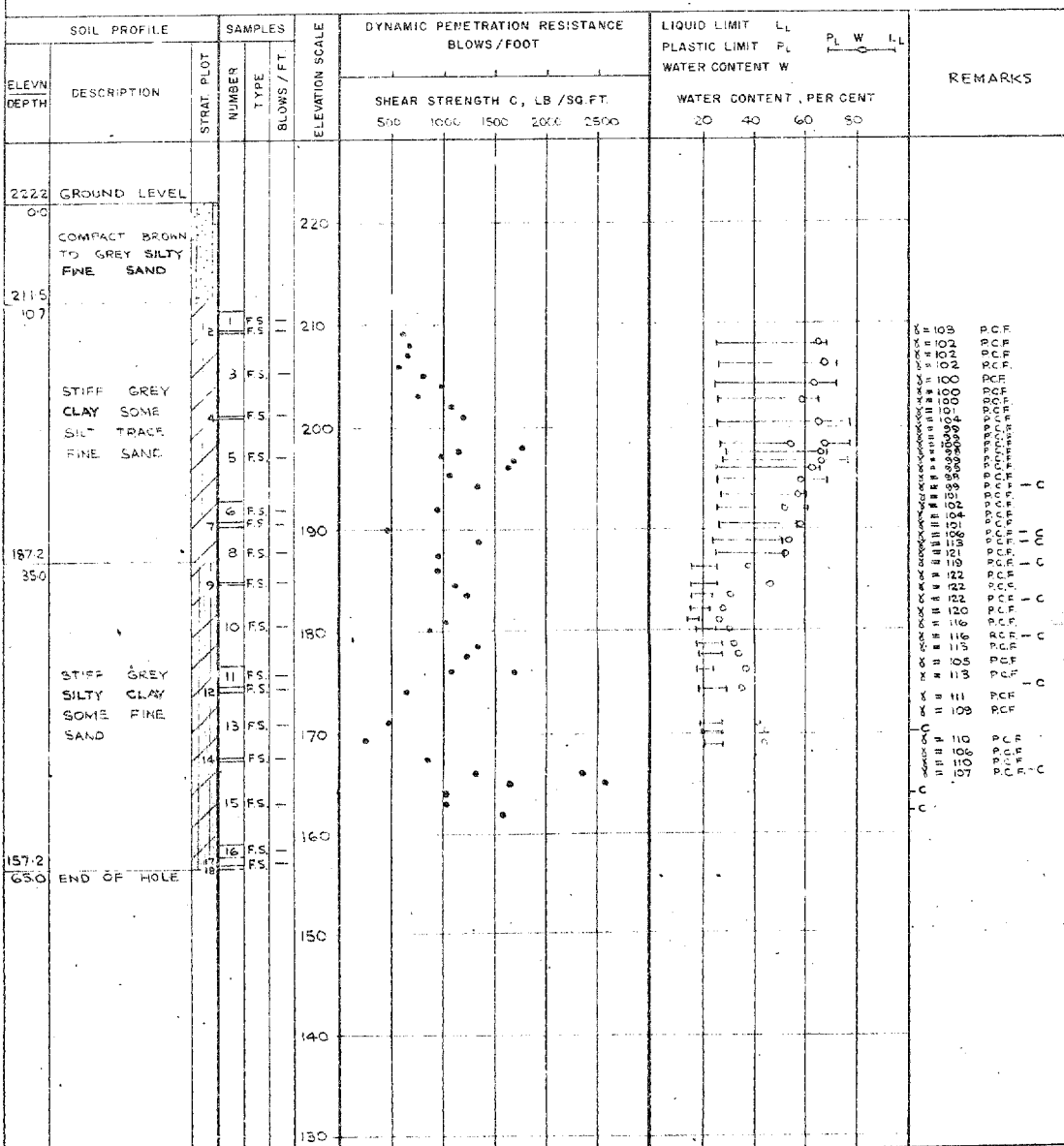
VERTICAL SCALE
 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
 CHECKED *PT*

RECORD OF BOREHOLE 23-5

LOCATION SEE FIGURE BORING DATE FEB 10-11, 1961 DATUM GEODETIC
 BOREHOLE TYPE F.O.L. BOREHOLE DIAMETER
 SAMPLER HAMMER WEIGHT — LB. DROP — INCHES PEN. TEST HAMMER WEIGHT — LB. DROP — INCHES



(a) Dynamic penetration resistance converted to 4200 inch lb. energy

(b) Abbreviations listed on page

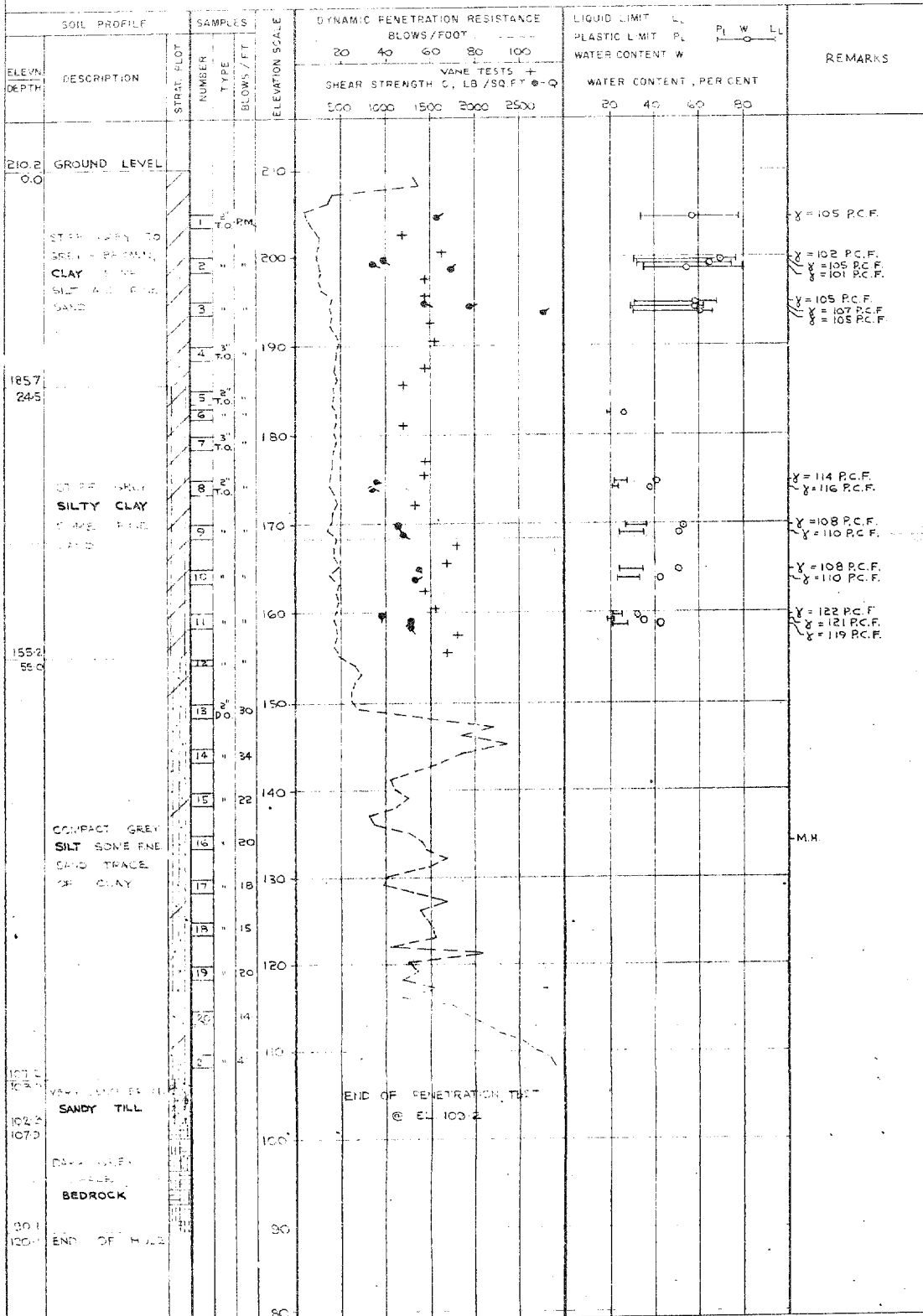
VERTICAL SCALE
 1 INCH TO 10 FEET

DRAWN J.A.
 CHECKED *AL*

GOLDER & ASSOCIATES

RECORD OF BOREHOLE 24-1

LOCATION SEE FIGURE BORING DATE JAN 25 - FEB 2, 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



(a) Dynamic penetration resistance converted to 4200 inch lb. energy
 (b) Abbreviations listed on page

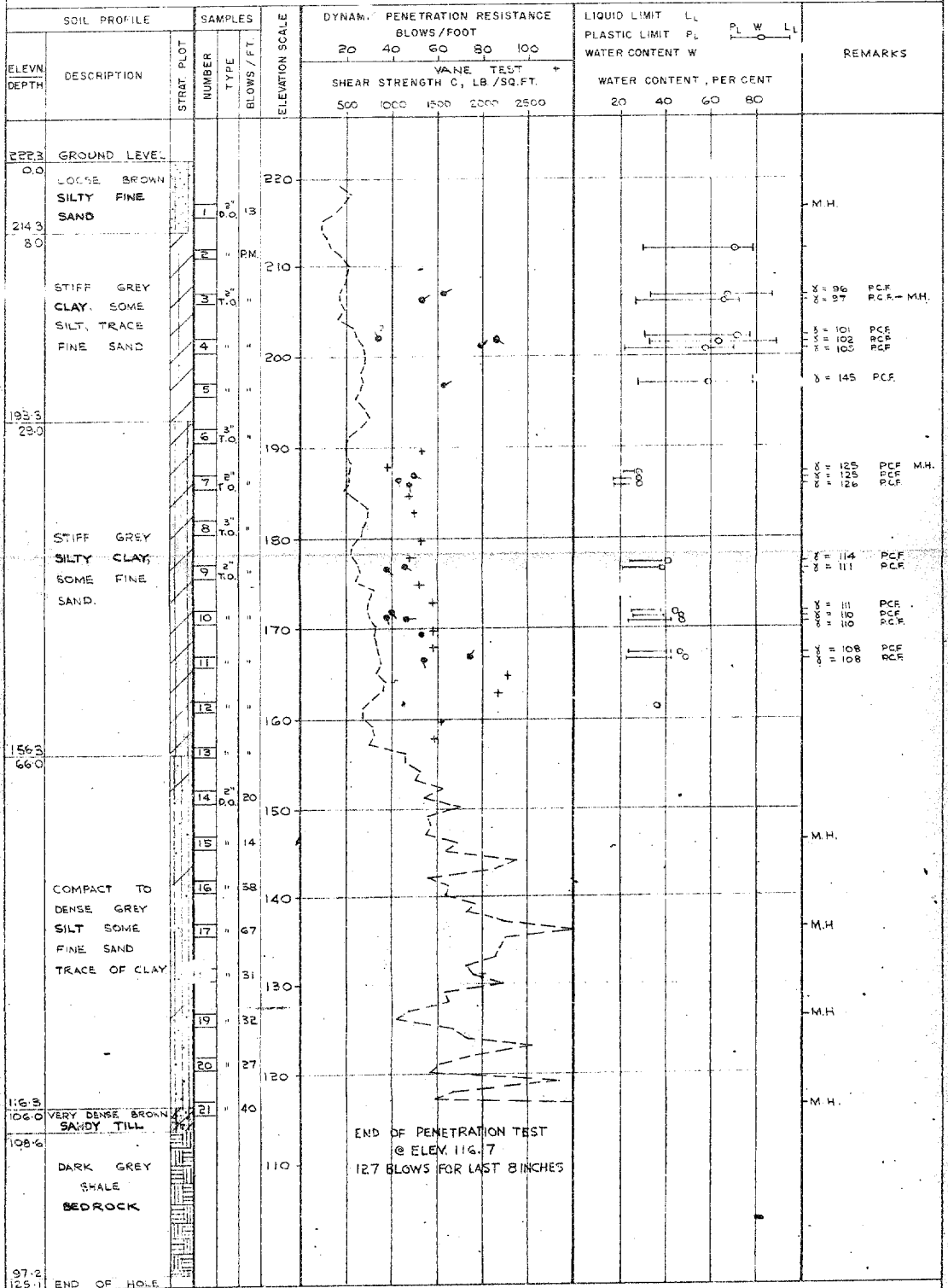
VERTICAL SCALE
 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
 CHECKED

RECORD OF BOREHOLE 24-2

LOCATION SEE FIGURE BORING DATE FEB. 2-7, 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



(a) Dynamic penetration resistance converted to 4200 inch lb. energy

(b) Abbreviations listed on page

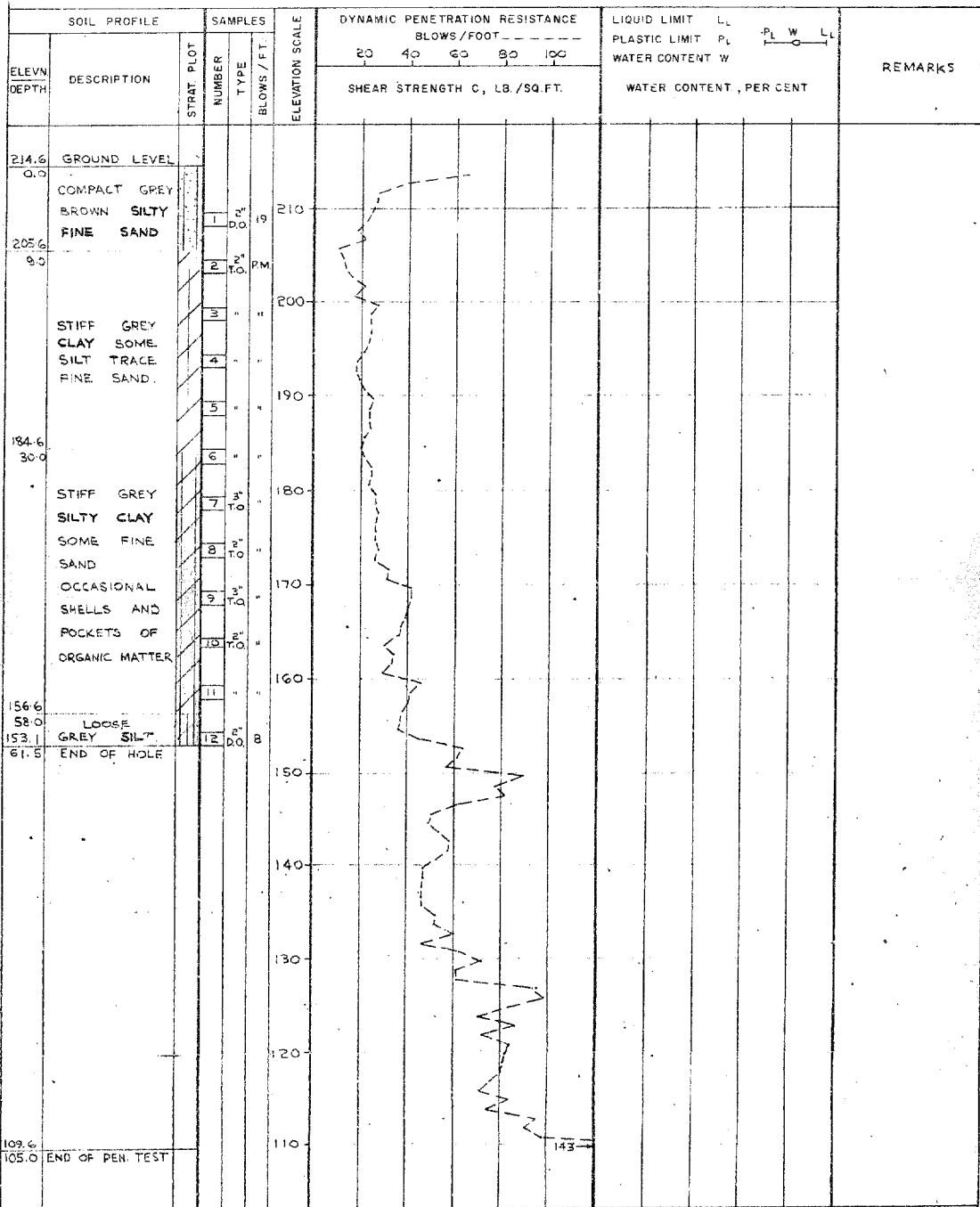
VERTICAL SCALE
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED *MC*

RECORD OF BOREHOLE 24-4

LOCATION SEE FIGURE BORING DATE FEB. 8, 1961 DATUM GEODETTIC
 BOREHOLE TYPE POWER AUGER BORING BOREHOLE DIAMETER 4.5"
 SAMPLER HAMMER WEIGHT LB. DROP INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



(a) Dynamic penetration resistance converted to 4200 inch lb energy

(b) Abbreviations listed on page

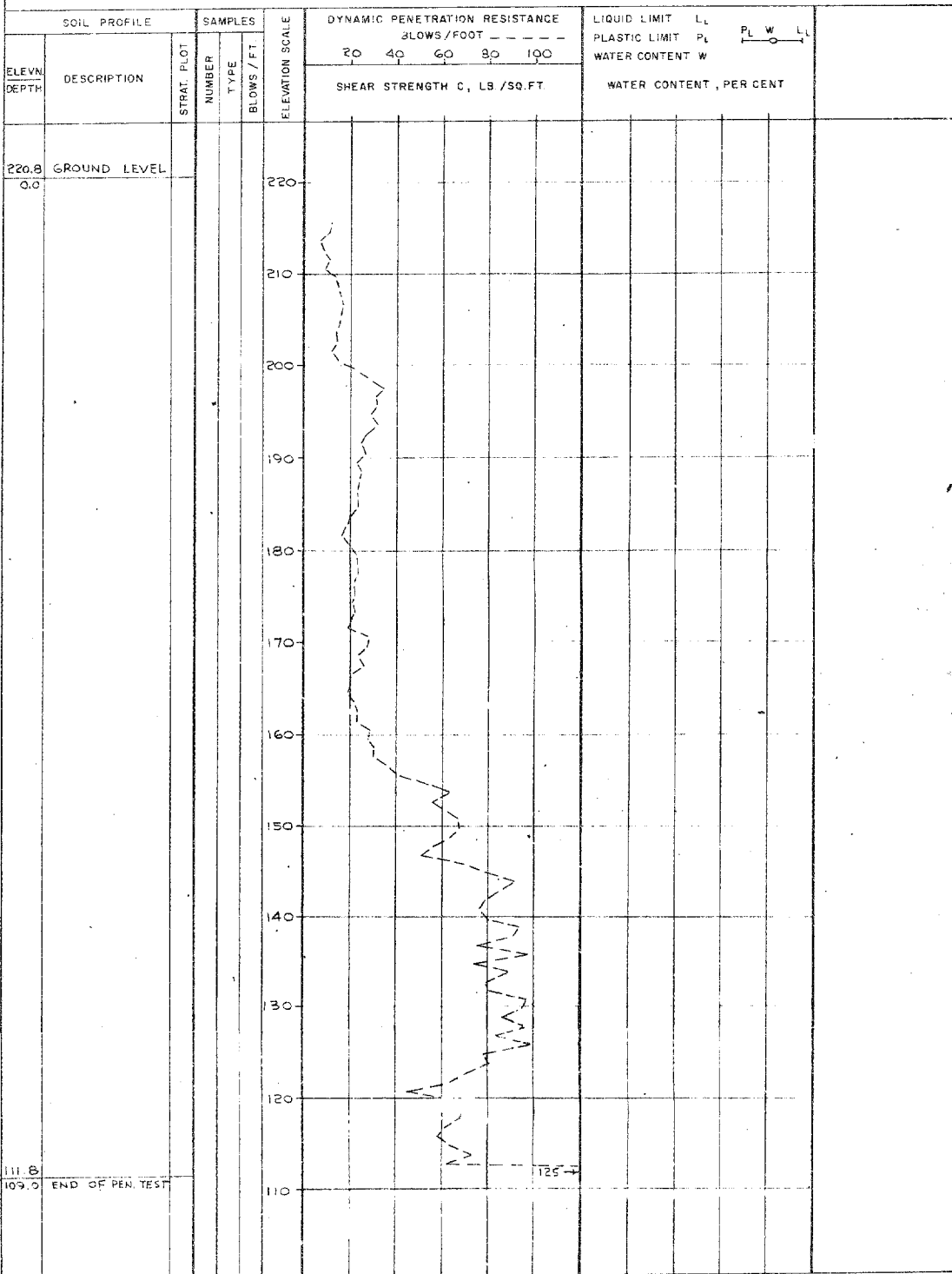
 VERTICAL SCALE
 1 INCH TO 10 FEET

GOLDER & ASSOCIATES

 DRAWN J. A.
 CHECKED *at*

RECORD OF BOREHOLE 24-5

LOCATION SEE FIGURE BORING DATE FEB 13, 1961 DATUM GEODETIC
 BOREHOLE TYPE PENETRATION TEST BOREHOLE DIAMETER —
 SAMPLER HAMMER WEIGHT — LB DROP — INCHES PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



(a) Dynamic penetration resistance converted to 4200 inch lb energy

(b) Abbreviations listed on page

 VERTICAL SCALE
 1 INCH TO 10 FEET

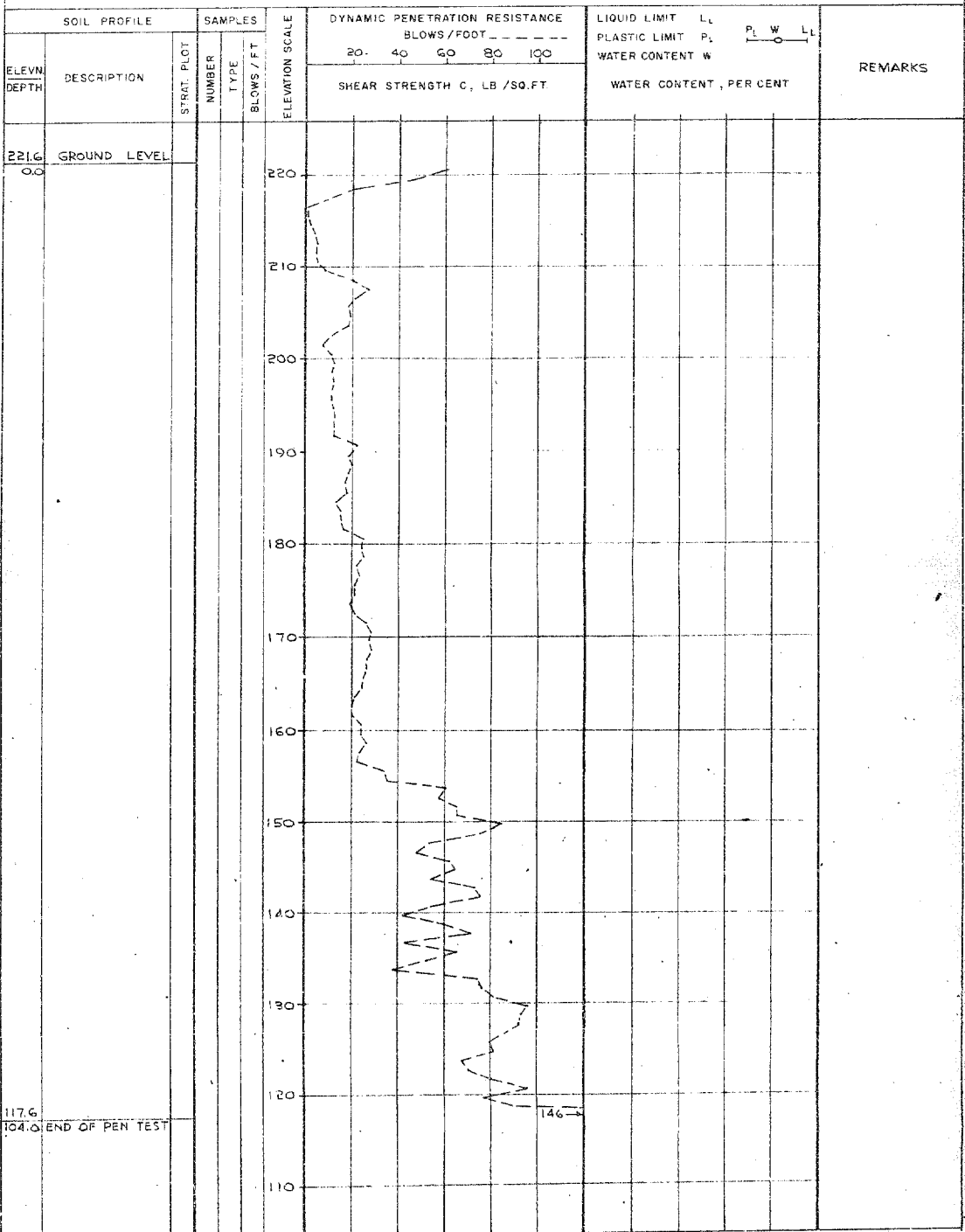
GOLDER & ASSOCIATES

 DRAWN J.A.
 CHECKED *[Signature]*

DATUM GEODETIC

BOREHOLE DIAMETER

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



(a) Dynamic penetration resistance converted to 4200 inch-lb. energy

(:) Abbreviations listed on page

VERTICAL SCALE

1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED *[Signature]*

RECORD OF BOREHOLE 24-7

LOCATION

SEE FIGURE

BORING DATE

FEB. 21 1961

DATUM

GEODETIC

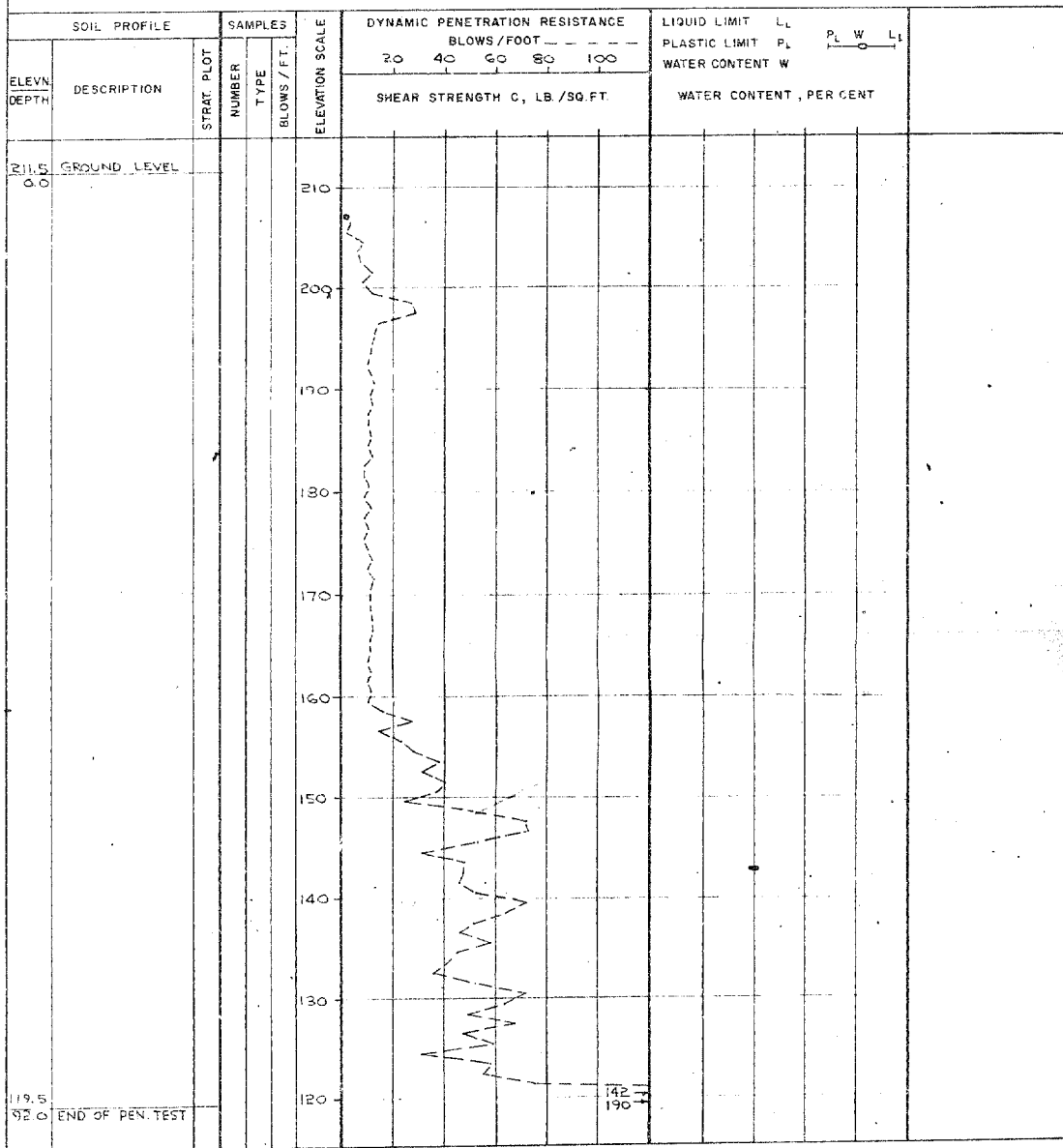
BOREHOLE TYPE

PENETRATION TEST

BOREHOLE DIAMETER

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN TEST HAMMER WEIGHT — LB. DROP — INCHES



(a) Dynamic penetration resistance converted to 4200 inch lb. energy

(b) Abbreviations listed on page

DRAWN J.A.

CHECKED *[initials]*

VERTICAL SCALE

1 INCH TO 10 FEET

GOLDER & ASSOCIATES

RECORD OF BOREHOLE 24-8

LOCATION SEE FIGURE.

BORING DATE

DATUM GEDDETIIC

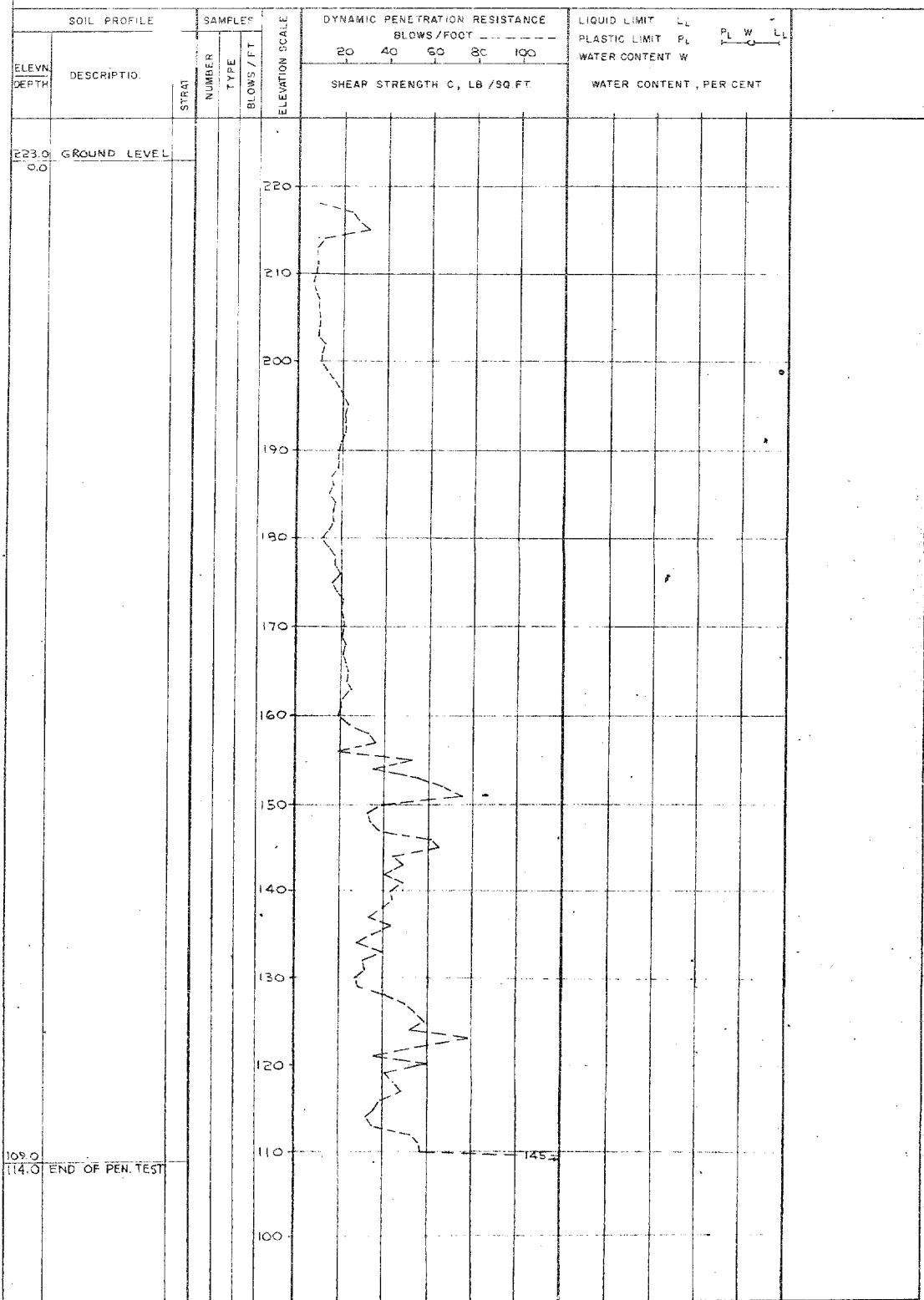
BOREHOLE TYPE

PENETRATION TEST

BOREHOLE DIAMETER

SAMPLER HAMMER WEIGHT --- LB. DROP --- INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



(a) Dynamic penetration resistance converted to 4200 inch lb. energy

(b) Abbreviations listed on page

VERTICAL SCALE

1 INCH TO 10 FEET

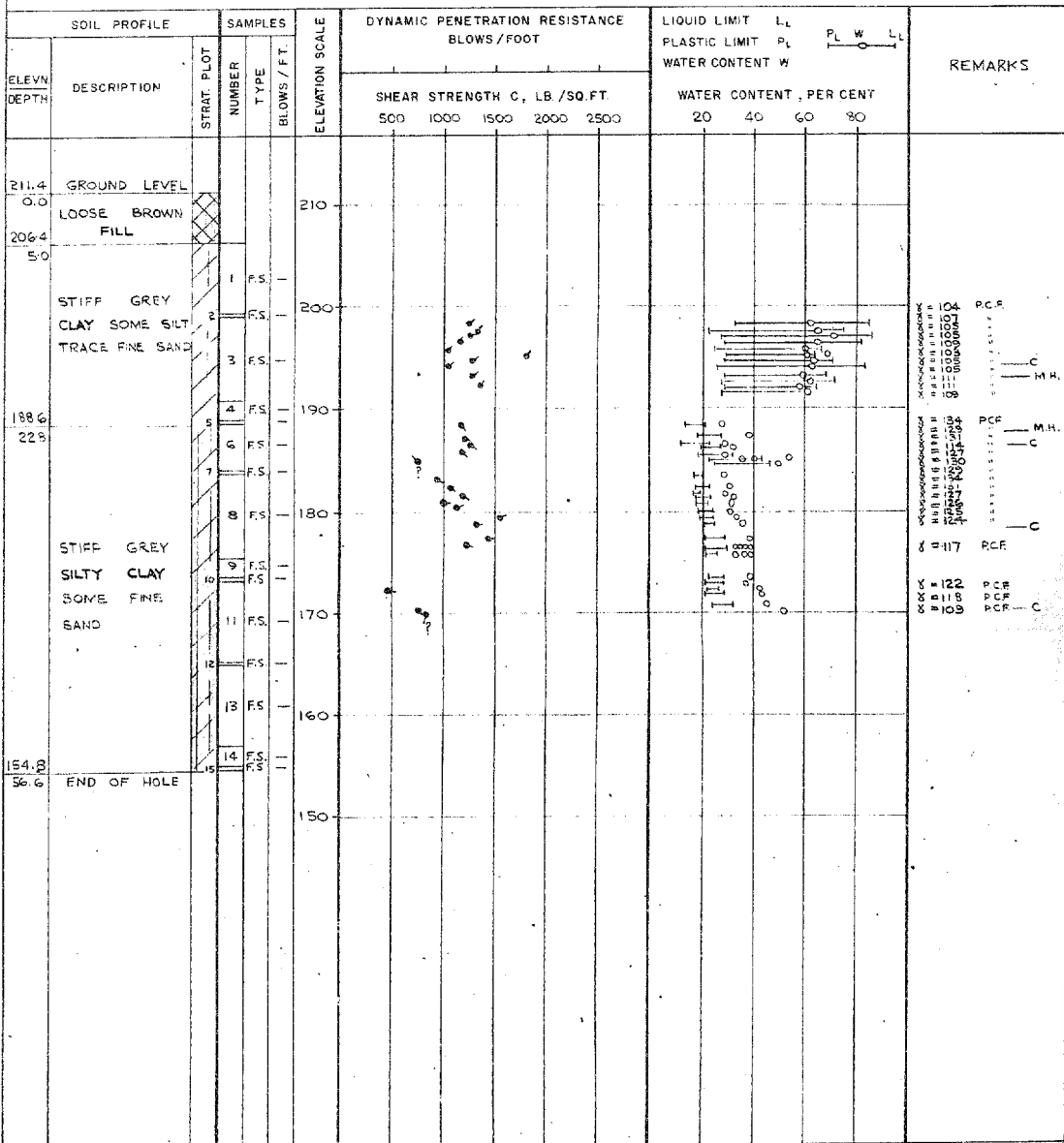
GOLDER & ASSOCIATES

DRAWN J.A.

CHECKED

RECORD OF BOREHOLE 24-9

LOCATION SEE FIGURE BORING DATE FEB. 9, 1961 DATUM GEODETTIC
 BOREHOLE TYPE FOIL BOREHOLE DIAMETER
 SAMPLER HAMMER WEIGHT LB. DROP INCHES PEN. TEST HAMMER WEIGHT LB. DROP INCHES



RECORD OF BOREHOLE 24-10

LOCATION SEE PAGE

BORING DATE 17-22 MARCH 1962

DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER 8X CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT LB. DROP INCHES

SOIL PROFILE		SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT					LIQUID LIMIT L _L PLASTIC LIMIT P _L WATER CONTENT W				REMARKS						
ELEVATION DEPTH	DESCRIPTION	STRAT. PLAT	NUMBER		TYPE	BLOWS/FT.	VANE TEST +					SHEAR STRENGTH C, LB./SQ.FT.				WATER CONTENT, PER CENT				
							500	1000	1500	2000	2500	20	40		60	80				
207.5 0.0	GROUND LEVEL																			
193.0 8.5	LOOSE BROWN SILTY SAND TRACE GRAVEL		1	DO	3	200														
			2	DO	18															
	STIFF GREY CLAY SOME SILT		3	PM															γ = 110 PCF	
187.7 12.8			4	PM																
			5	PM															γ = 120 PCF X = 119 PCF	
	STIFF GREY SILTY CLAY		6	PM		180														
	TRACE OF FINE SAND		7	PM															γ = 116 PCF	
	OCCASIONAL SHELLS AND POCKETS OF BLACK ORGANIC MATERIAL		8	PM		170														
			9	PM															γ = 122 PCF	
			10	PM		160														
151.2 56.3			11	PM		150														
			12	DO	24															
			13		33															
	LOOSE TO COMPACT GREY SILT SOME FINE SAND TRACE CLAY		14		8	140														
			15		12														M.H.	
			16		21	130														
			17		10	120														
116.2 91.3			18		43															
	DENSE GREY BROWN SAND TRACE GRAVEL		19		44														M.H.	
			20		61	110														
103.5 104.2 100.0 107.5			21		58															
	VERY DENSE BROWN SANDY TILL		22		80	100													M.H.	
	DARK GREY SHAPE BEDROCK		23	EXT RC																
90.0 117.5	END OF HOLE		24			90														

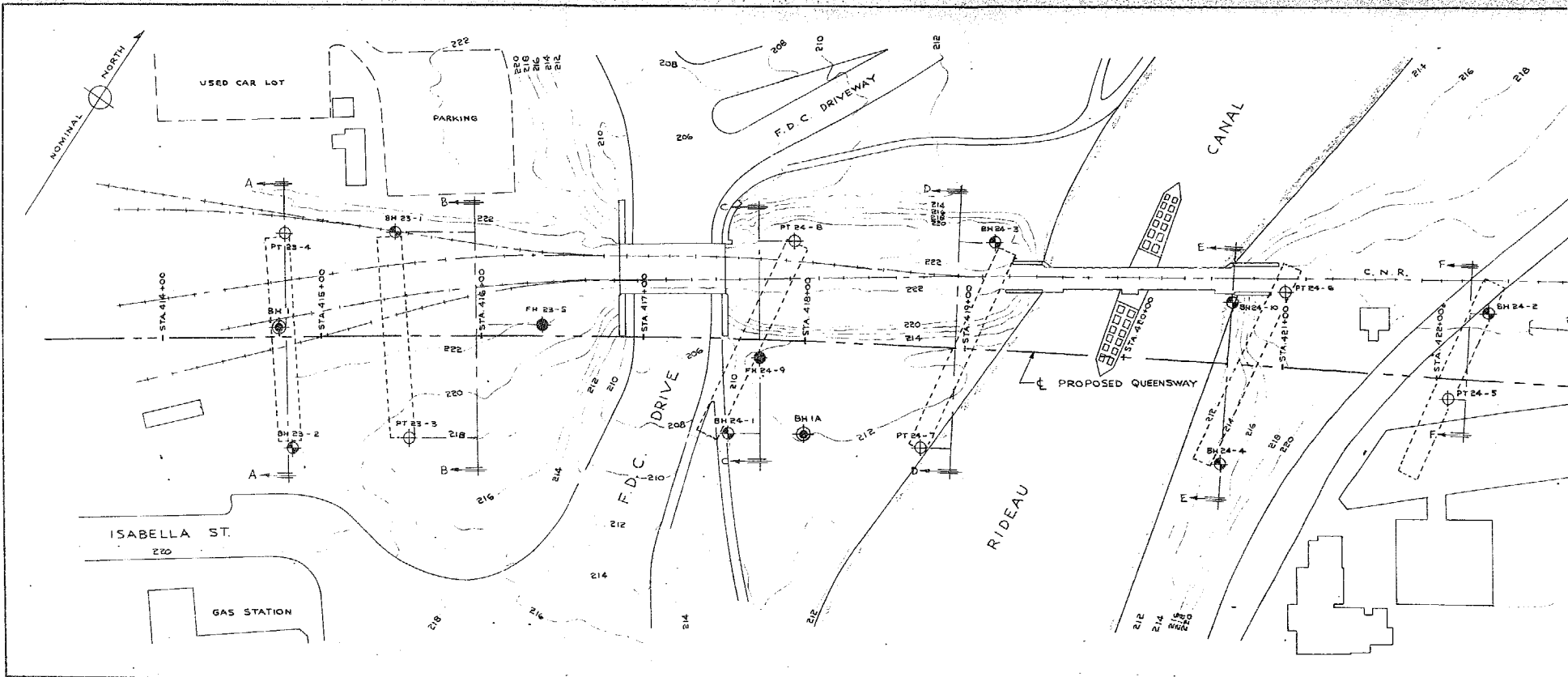
(a) Dynamic penetration resistance converted to 4200 inch lb. energy

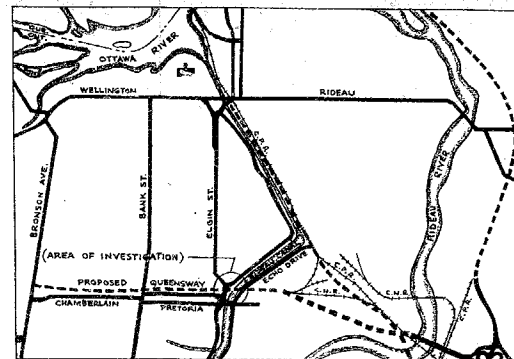
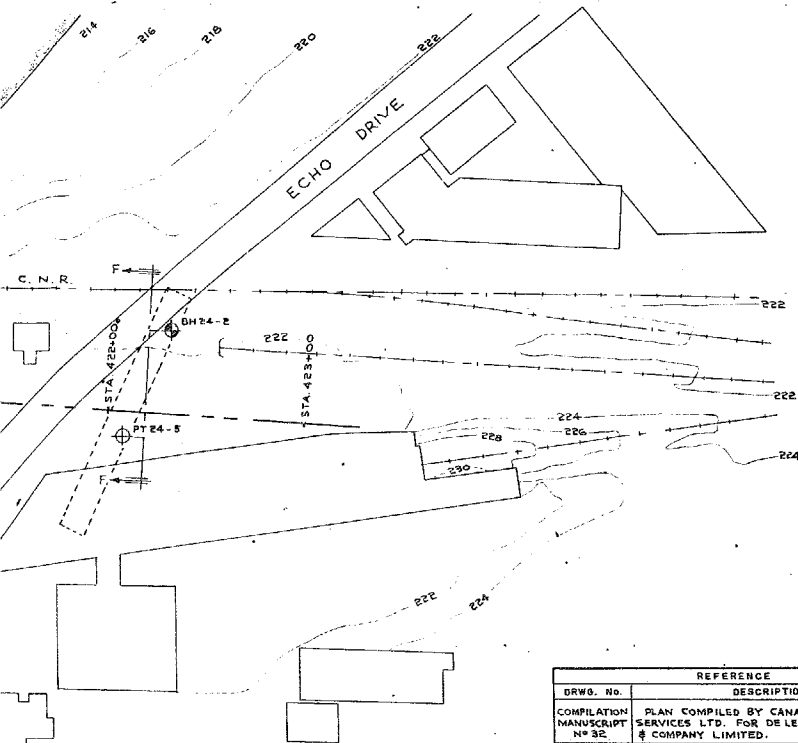
(b) Abbreviations listed on page

VERTICAL SCALE
1 INCH TO 10 FEET

GOLDER & ASSOCIATES

DRAWN J.A.
CHECKED J.A.





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

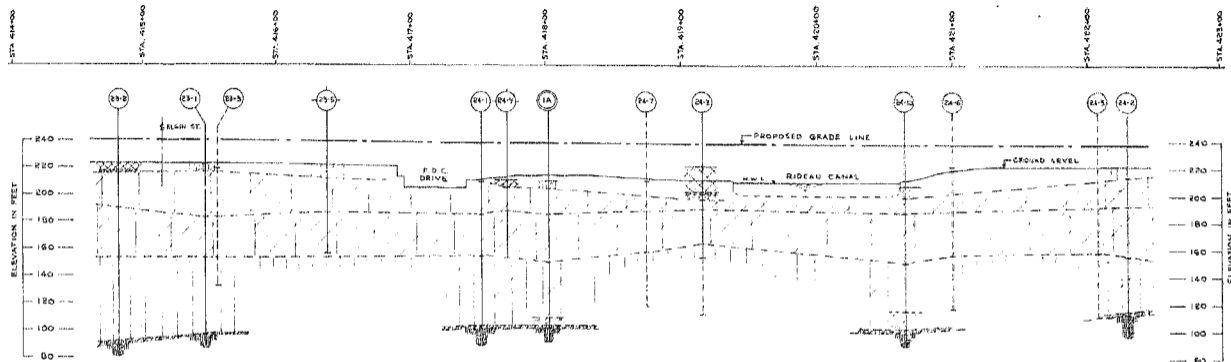
LEGEND

- BOREHOLE WITH PENETRATION TEST IN PLAN
- SWEDISH FOIL HOLE IN PLAN
- PENETRATION TEST IN PLAN
- BOREHOLE BY OTHERS IN PLAN (PREVIOUS INVESTIGATION)
- APPROX. LOCATIONS - PROPOSED PIERS AND ABUTMENTS

REFERENCE	
DRWG. No.	DESCRIPTION
COMPILATION MANUSCRIPT No 32	PLAN COMPILED BY CANADIAN AERO SERVICES LTD. FOR DE LEUW CATHER & COMPANY LIMITED.
REPORT No's SF- 510 SF- 511	BY MCROSTIE & ASSOCIATES LTD. TO DE LEUW CATHER & COMPANY LIMITED. DATED JANUARY, 1961.

DE LEUW CATHER & COMPANY
OF CANADA LIMITED
OTTAWA ONTARIO
PROPOSED QUEENSWAY
ELGIN STREET AND RIDEAU CANAL STRUCTURES
OTTAWA ONTARIO
BORING PLAN

GOLDER & ASSOCIATES
CONSULTING CIVIL ENGINEERS
DATE: OCT 4, 1961 SCALE: 1" TO 40'
MADE CHKD. APPD.
J.A. AHS V.H.
FIGURE 1



SCHEMATIC SECTION ALONG CENTRE LINE OF PROPOSED QUEENSWAY
SCALE: 1" TO 40'-0"

STRATIGRAPHY

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

LEGEND

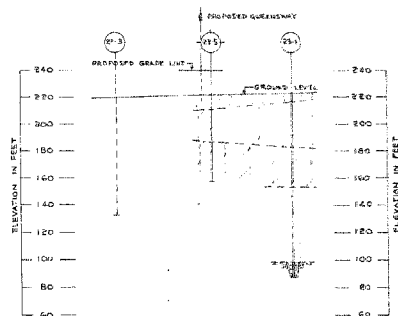
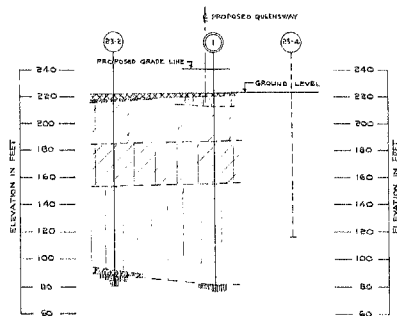
- BOREHOLE IN ELEVATION
- SWEDISH FOIL HOLE IN ELEVATION
- PENETRATION TEST IN ELEVATION
- BOREHOLE IN ELEVATION (PREVIOUS INVESTIGATION)

REFERENCE	
ORIG. NO.	DESCRIPTION
III - A - 24A	DEPARTMENT OF HIGHWAYS, ONTARIO OTTAWA QUEENSWAY - STA. 405+00 TO STA. 417+00, R/W & UTILITIES.
IV - A - B	DEPARTMENT OF HIGHWAYS, ONTARIO OTTAWA QUEENSWAY - STA. 417+00 TO STA. 423+00, R/W & UTILITIES.

DE LEUW CATHÉ & COMPANY
OF CANADA LIMITED
OTTAWA ONTARIO
PROPOSED QUEENSWAY
ELGIN STREET AND RIDEAU CANAL STRUCTURES
OTTAWA ONTARIO
GENERAL SOIL STRATIGRAPHY

GOLDER & ASSOCIATES
CONSULTING CIVIL ENGINEERS
DATE: OCT 9, 1961 SCALE: 1" TO 40'
MADE: J.A. CHD: R.J. APP: R.J. FIGURE 2

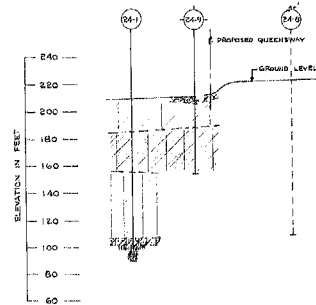
STRATIGRAPHY



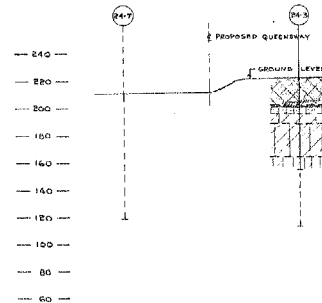
- TOPSOIL
- VERY LOOSE TO LOOSE CONTACT BROWN TO GREY SILTY FINE SAND
- LAYER OF CONTACT FILL
- GREY SILTY CLAY WITH TRACE OF FINE SAND
- SILTY CLAY, SOME FINE SAND, OCCASIONAL SHELLS AND ORGANIC POCKETS
- LIGHT TO MEDIUM GREY SILT, SOME FINE SAND, TRACE OF CLAY
- VERY DENSE TO MEDIUM SANDY TILL
- DARK GREY SHALE BEDROCK

- BOREHOLE IN ELEVATION
- SWEDISH FOIL HOLE IN ELEVATION
- PENETRATION TEST IN ELEVATION
- BOREHOLE IN ELEVATION (PREVIOUS INVESTIGATION)

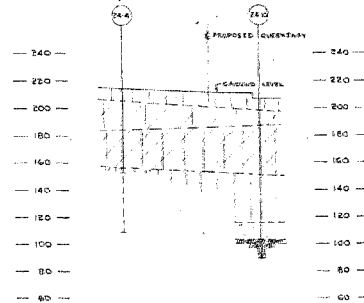
REFERENCE		DE LEUW CATHER & COMPANY OF CANADA LIMITED		GOLDER & ASSOCIATES CONSULTING CIVIL ENGINEERS	
DRWS. NO.	DESCRIPTION	OTTAWA	ONTARIO	DATE: OCT. 19, 1961	SCALE: 1" TO 40'-0"
FIGURES 1 AND 2	GOLDER & ASSOCIATES - BORING PLAN AND GENERAL SOIL STRATIGRAPHY	OTTAWA	ONTARIO	MADE: J.A.	CHAB. 7/10/61
PROPOSED QUEENSWAY ELGIN STREET OVERPASS SOIL STRATIGRAPHY				FIGURE 3	



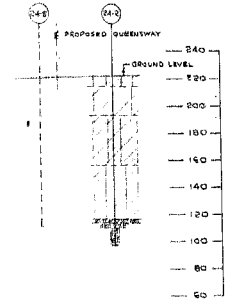
SECTION C-C



SECTION D-D



SECTION E-E



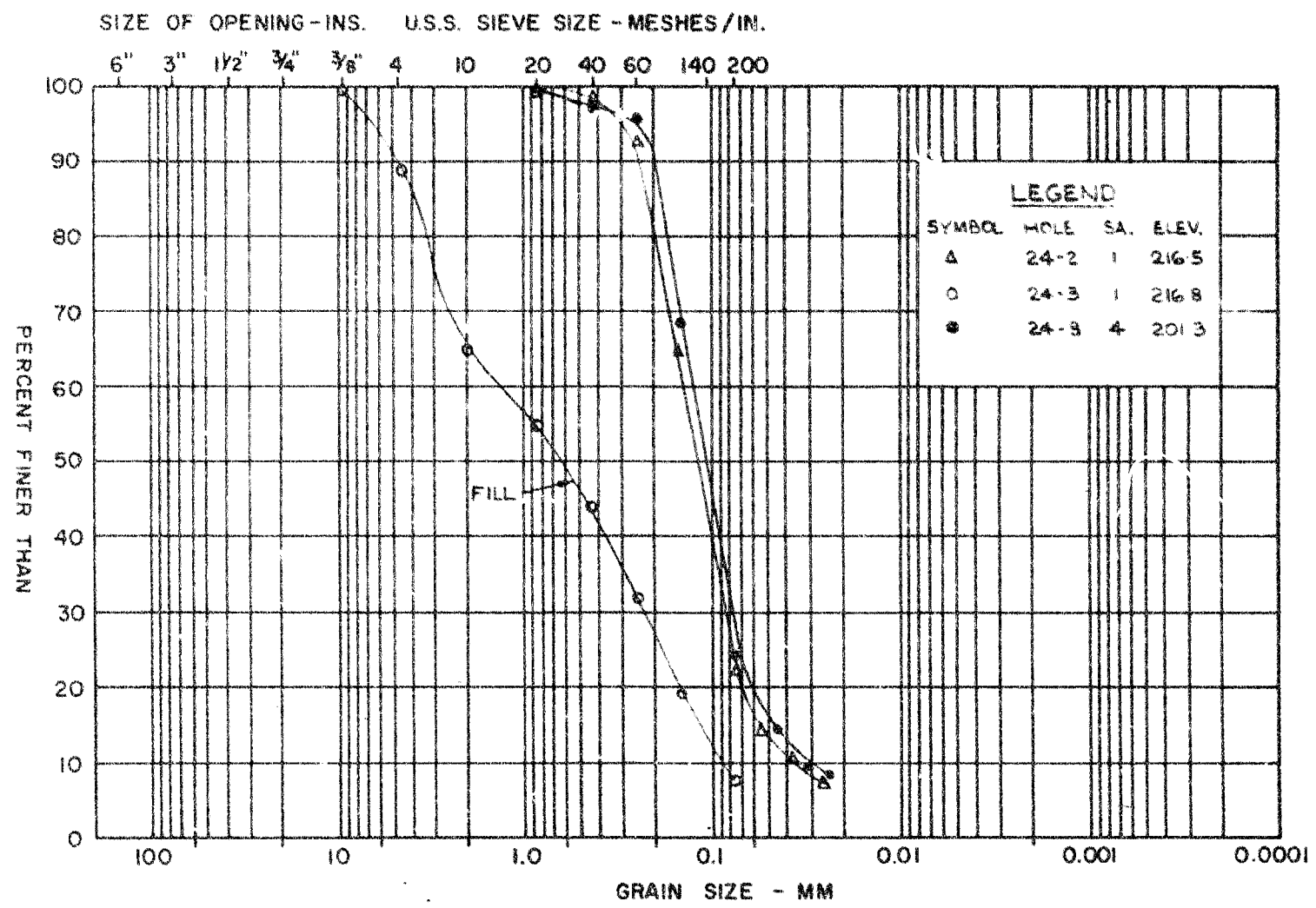
SECTION F-F

- STRATIGRAPHY**
- TOPSOIL
 - VERY LOOSE TO COMPACT BROWN TO GREY SILTY FINE SAND
 - LOOSE TO COMPACT FILL
 - STIFF GREY-BROWN TO GREY CLAY SILT, TRACE OF FINE SAND
 - STIFF GREY SILTY CLAY SOME FINE SAND, OCCASIONAL SHELLS AND DEBRIS POCKETS
 - LOOSE TO COMPACT GREY SILT SOME FINE SAND, TRACE OF CLAY
 - LOOSE GREY-BROWN SAND, TRACE OF GRAVEL
 - VERY DENSE BROWN SANDY TILL
 - DARK GREY SHALE BEDROCK

- LEGEND**
- BOREHOLE IN ELEVATION
 - SWEDISH SOIL HOLE IN ELEVATION
 - PENETRATION TEST IN ELEVATION

REFERENCE		DE LEUW CATHER & COMPANY		GOLDER & ASSOCIATES	
DRWG. NO.	DESCRIPTION	OTTAWA	ONTARIO	OTTAWA	ONTARIO
FIGURES 1 AND 2	GOLDER & ASSOCIATES - BORING PLAN AND GENERAL SOIL STRATIGRAPHY.	OTTAWA	ONTARIO	OTTAWA	ONTARIO
		PROPOSED QUEENSWAY RIDEAU CANAL BRIDGE		DATE: OCT. 18, 1961 SCALE: 1" TO 40'-0"	
		SOIL STRATIGRAPHY		MADE BY: J.A. DATE: 10/18/61 APPD. J.M. DATE: 10/18/61	
				FIGURE 4	

M.I.T. GRAIN SIZE SCALE



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

GOLDER & ASSOCIATES

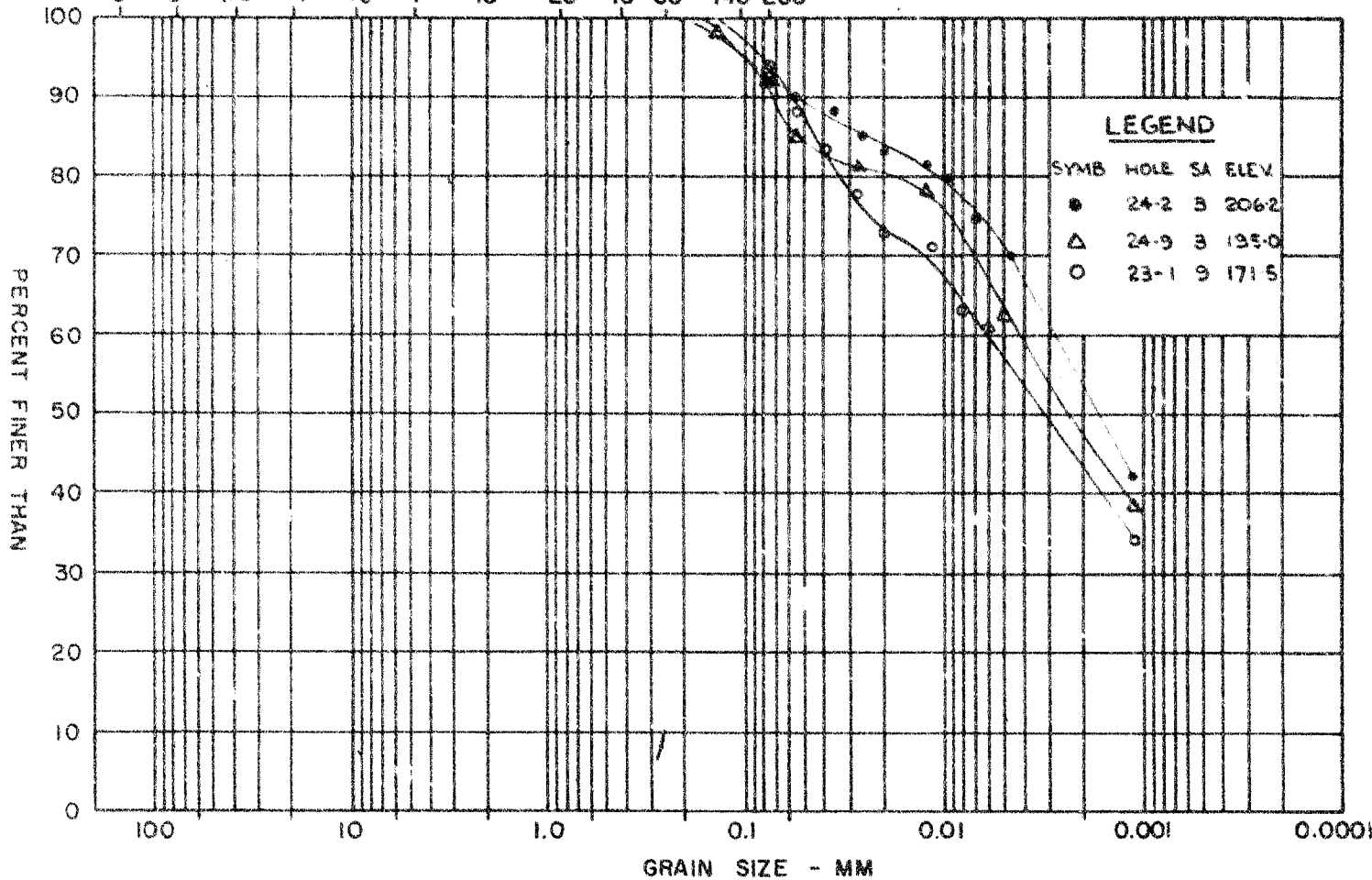
GRAIN SIZE DISTRIBUTION

FIGURE 15

M.I.T. GRAIN SIZE SCALE

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

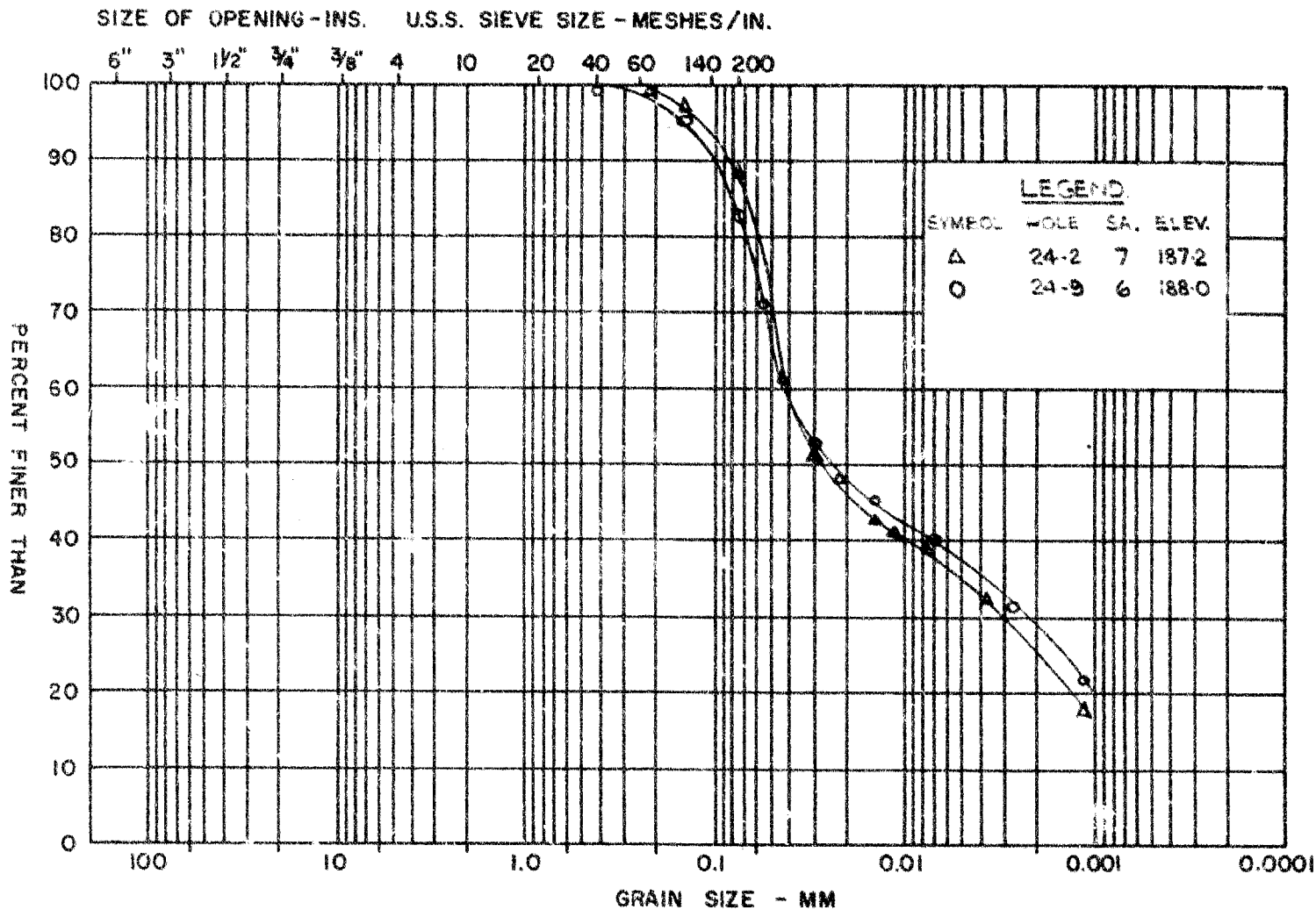
6" 3" 1 1/2" 3/4" 3/8" 4 10 20 40 60 140 200



GRAIN SIZE DISTRIBUTION
UPPER CLAY

FIGURE

M.I.T. GRAIN SIZE SCALE



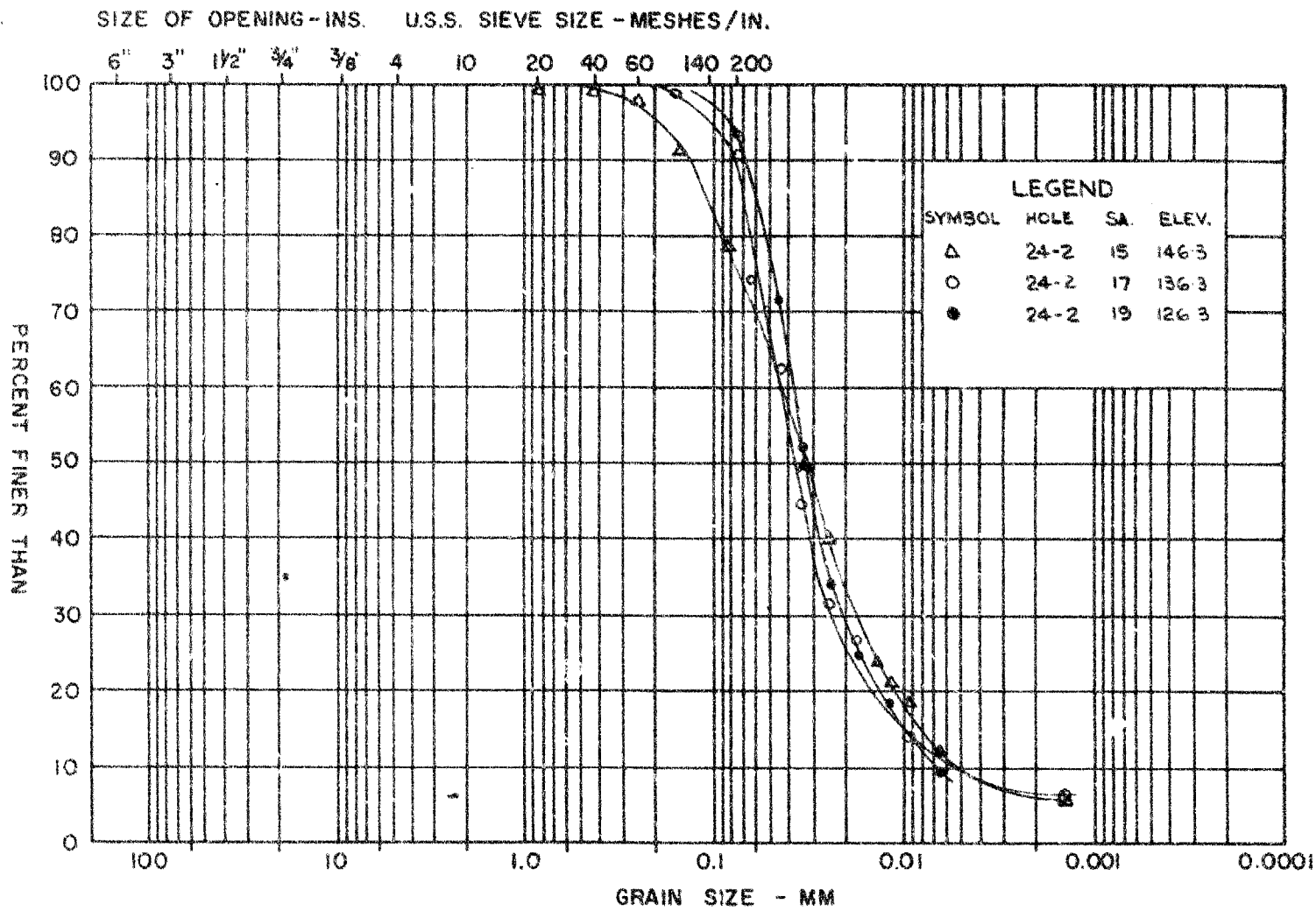
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
LOWER CLAY

FIGURE 7

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
SANDY SILT

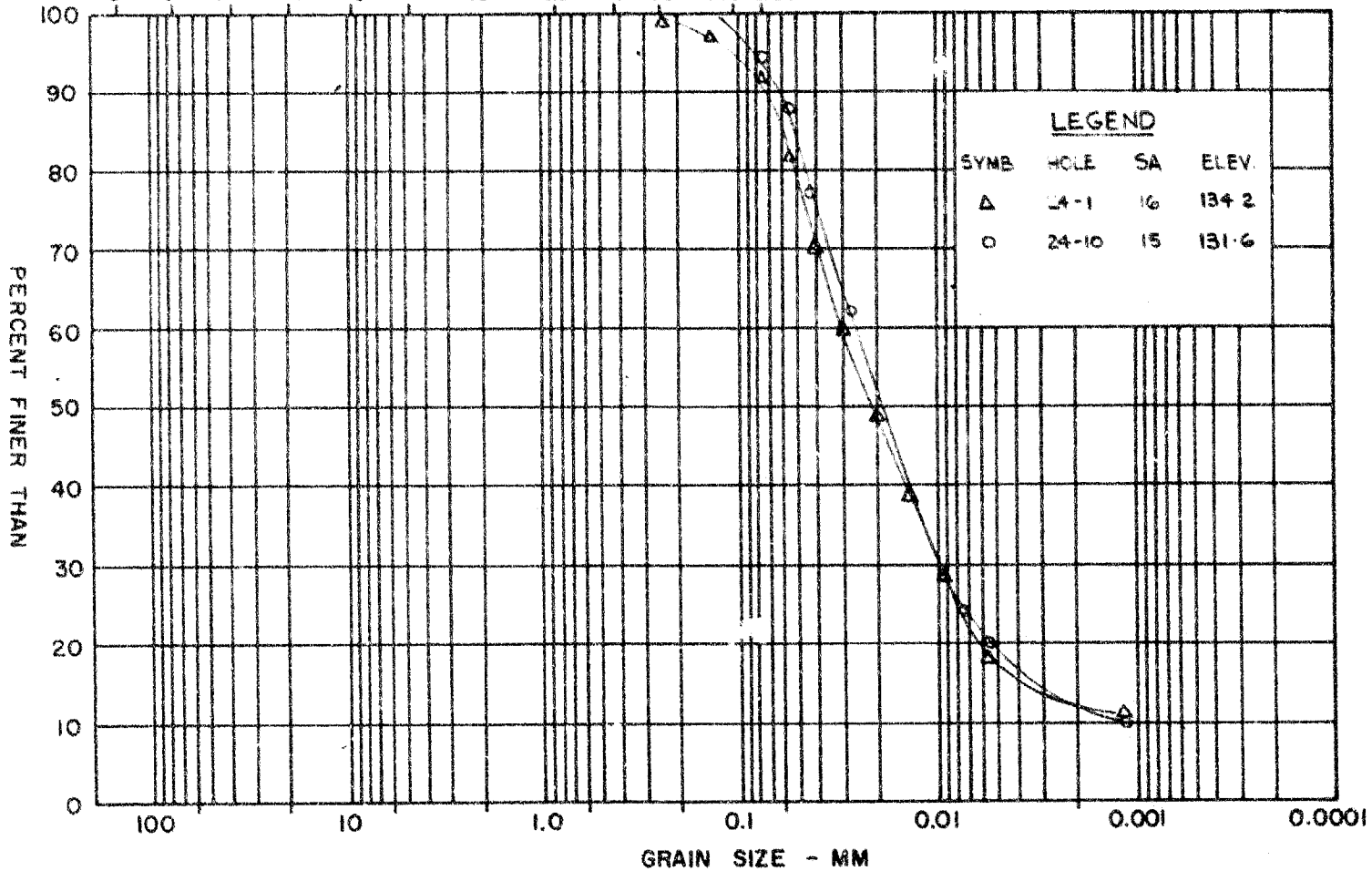
FIGURE 8

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

M.I.T. GRAIN SIZE SCALE

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

6" 3" 1 1/2" 3/4" 3/8" 4 10 20 40 60 140 200

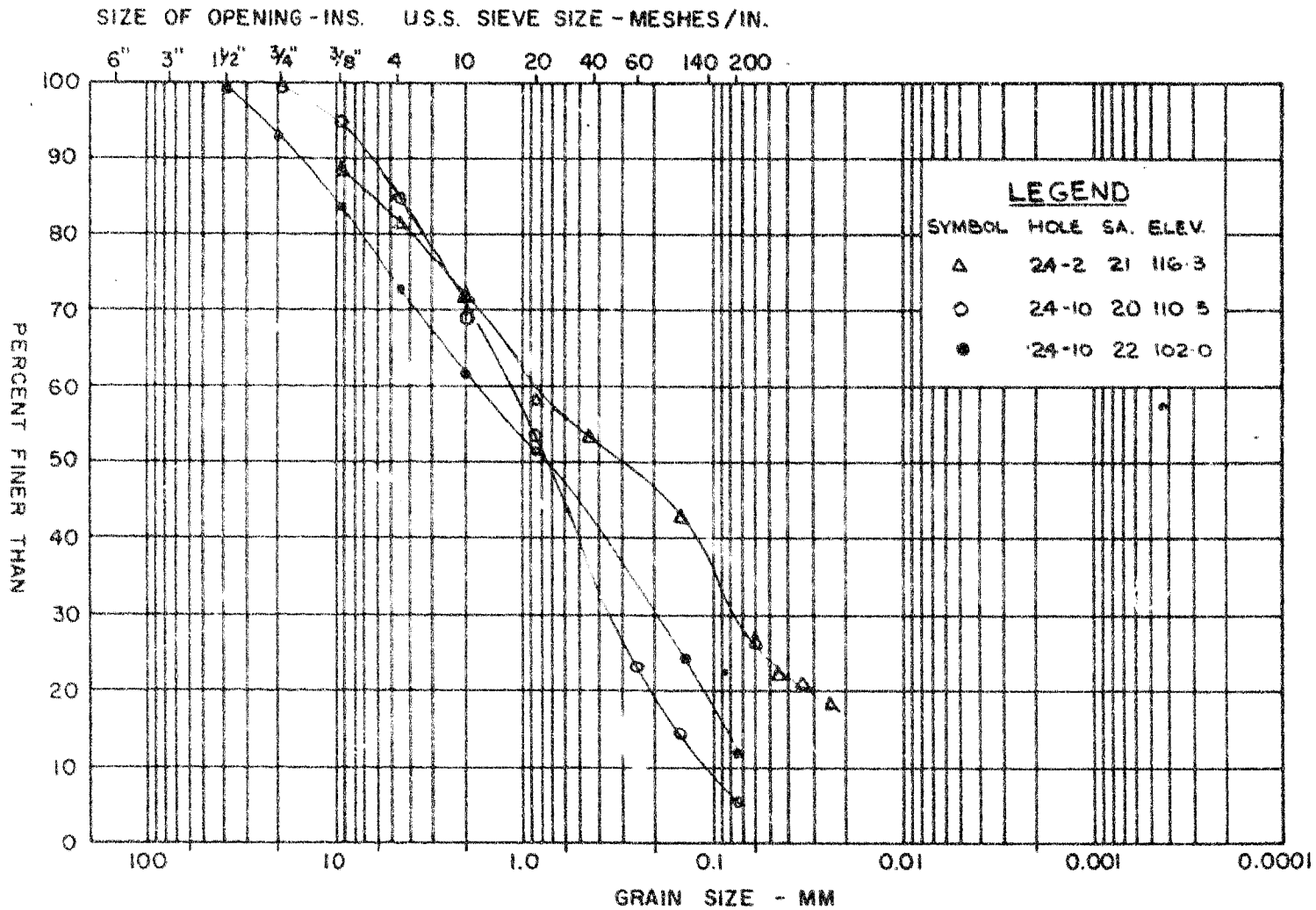


GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION
SANDY SILT

FIGURE 9

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

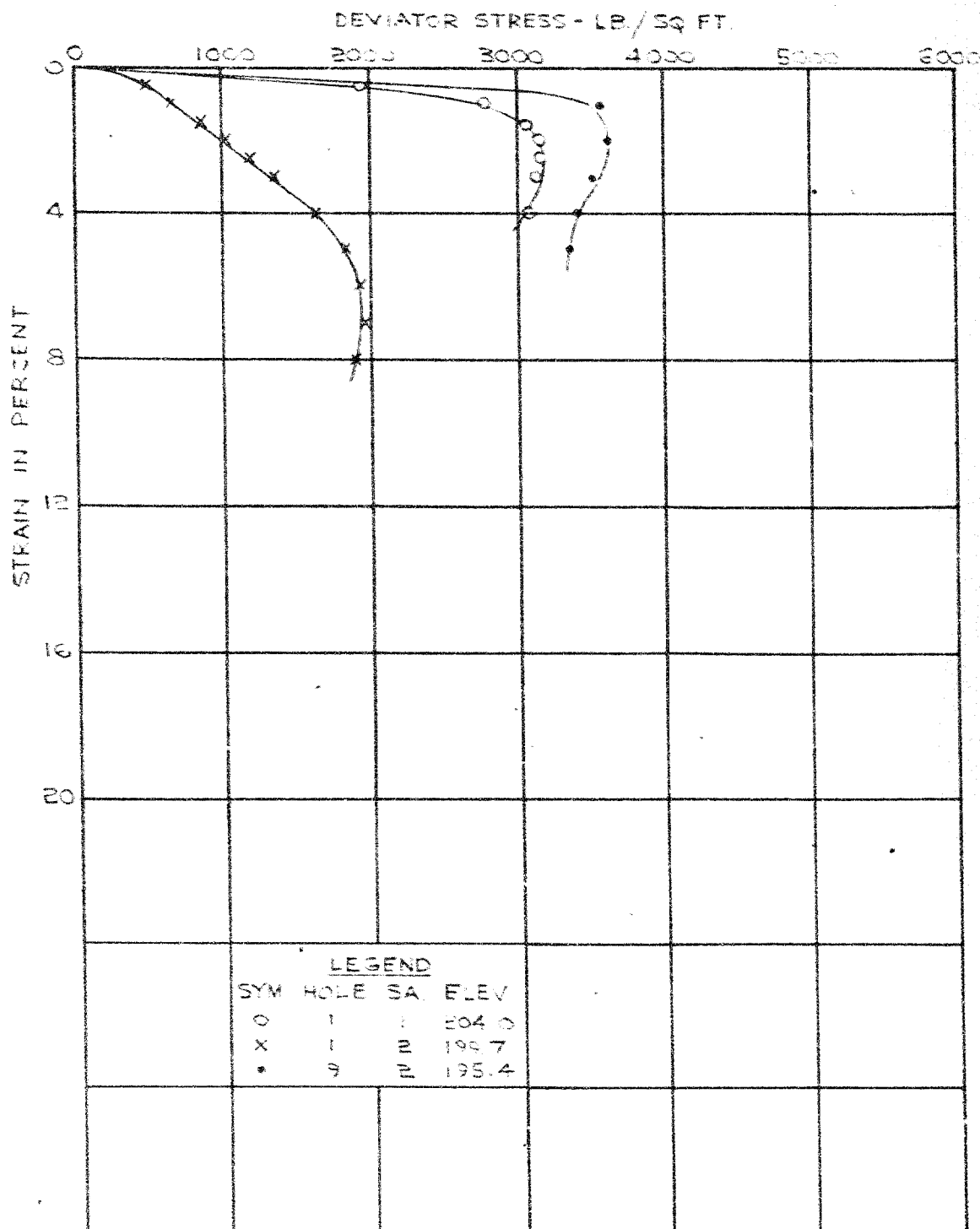
GRAIN SIZE DISTRIBUTION
SANDY TILL

FIGURE 10

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

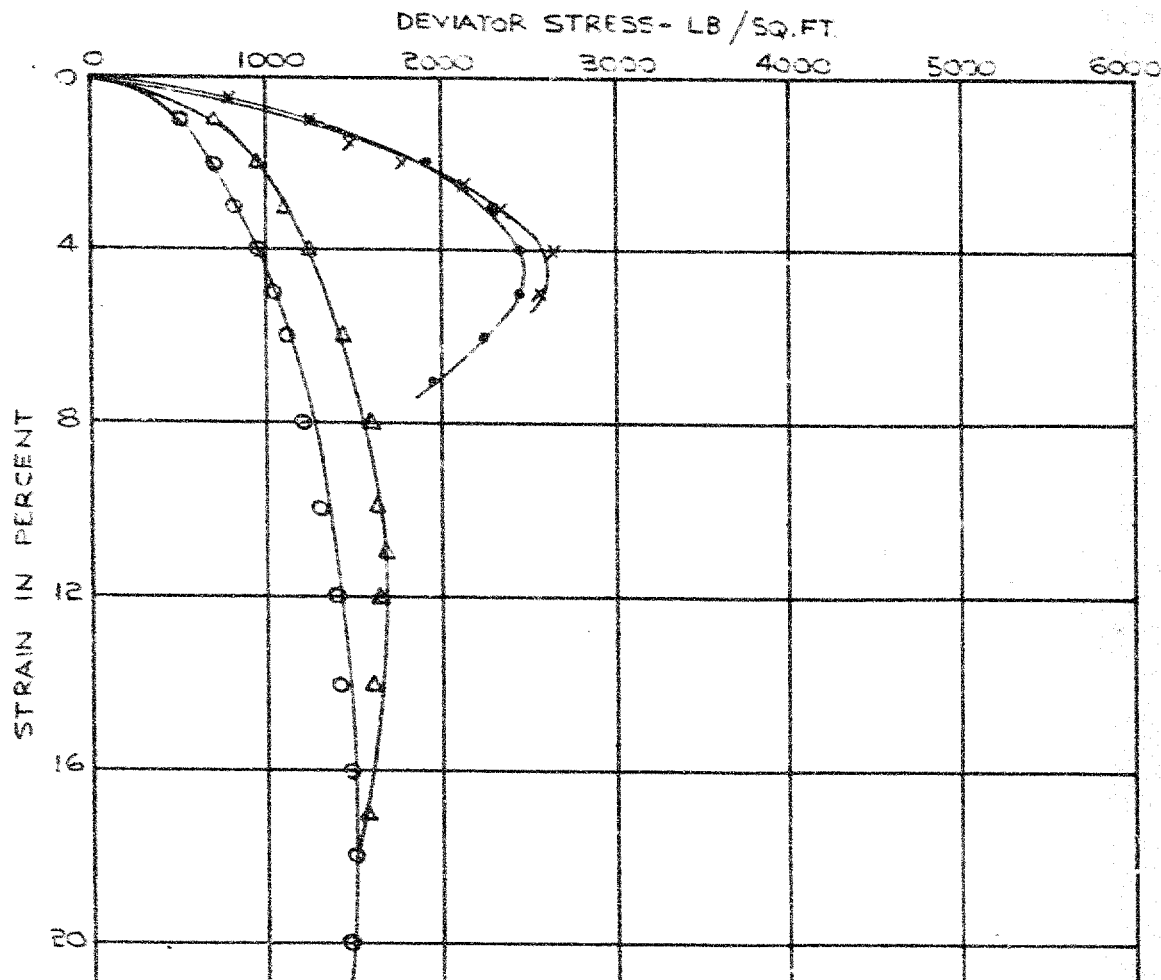
UNDRAINED TRIAXIAL COMPRESSION TESTS TYPICAL STRESS-STRAIN CURVES - STIFF GREY CLAY

FIGURE 11



UNDRAINED TRIAXIAL COMPRESSION TESTS TYPICAL STRESS-STRAIN CURVES - STIFF GREY SILTY CLAY

FIGURE 12

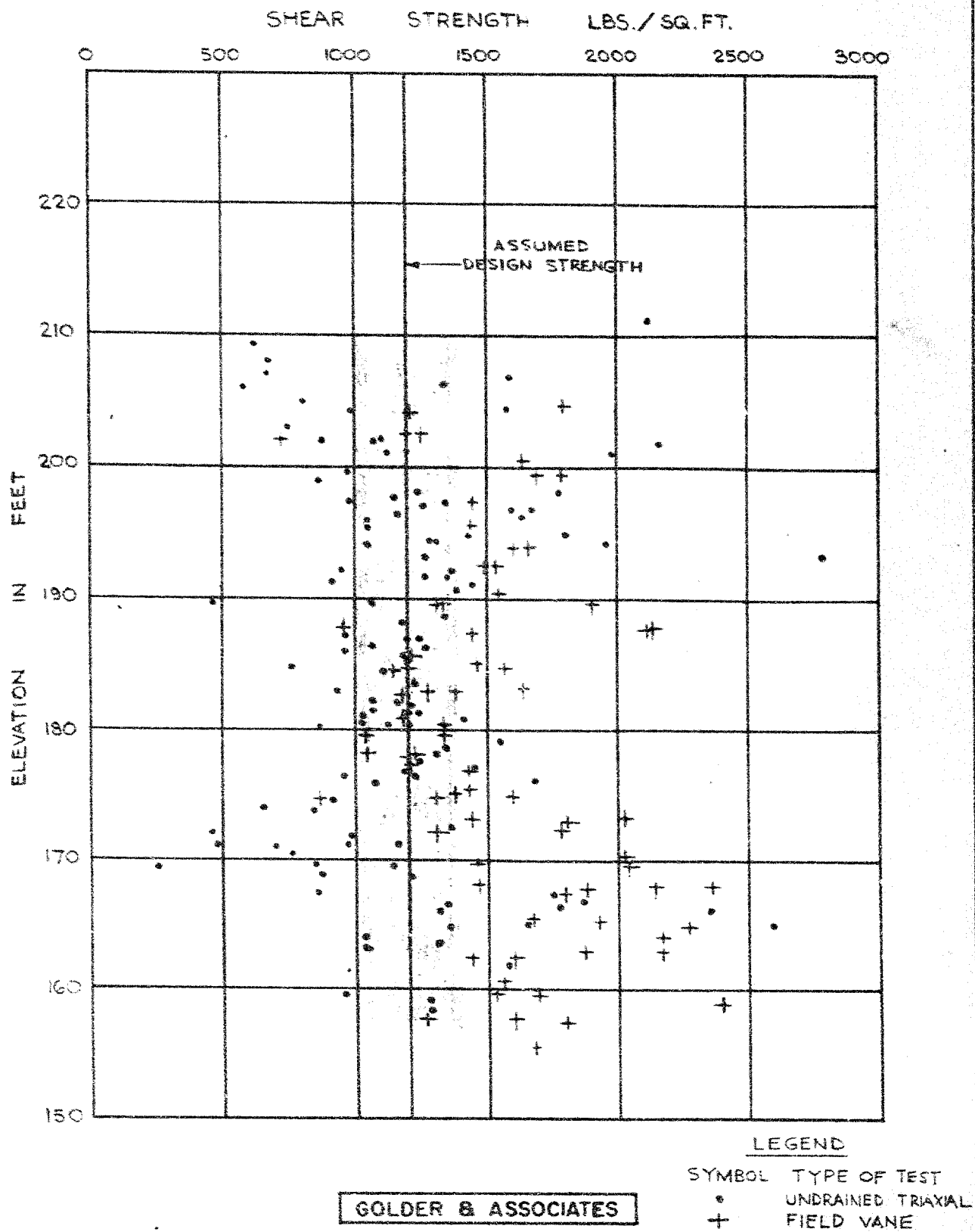


LEGEND

SYM.	HOLE	SA	ELEV.
X	1	9	164.2
O	9	3	185.4
•	4	176.4	
Δ	5	163.4	

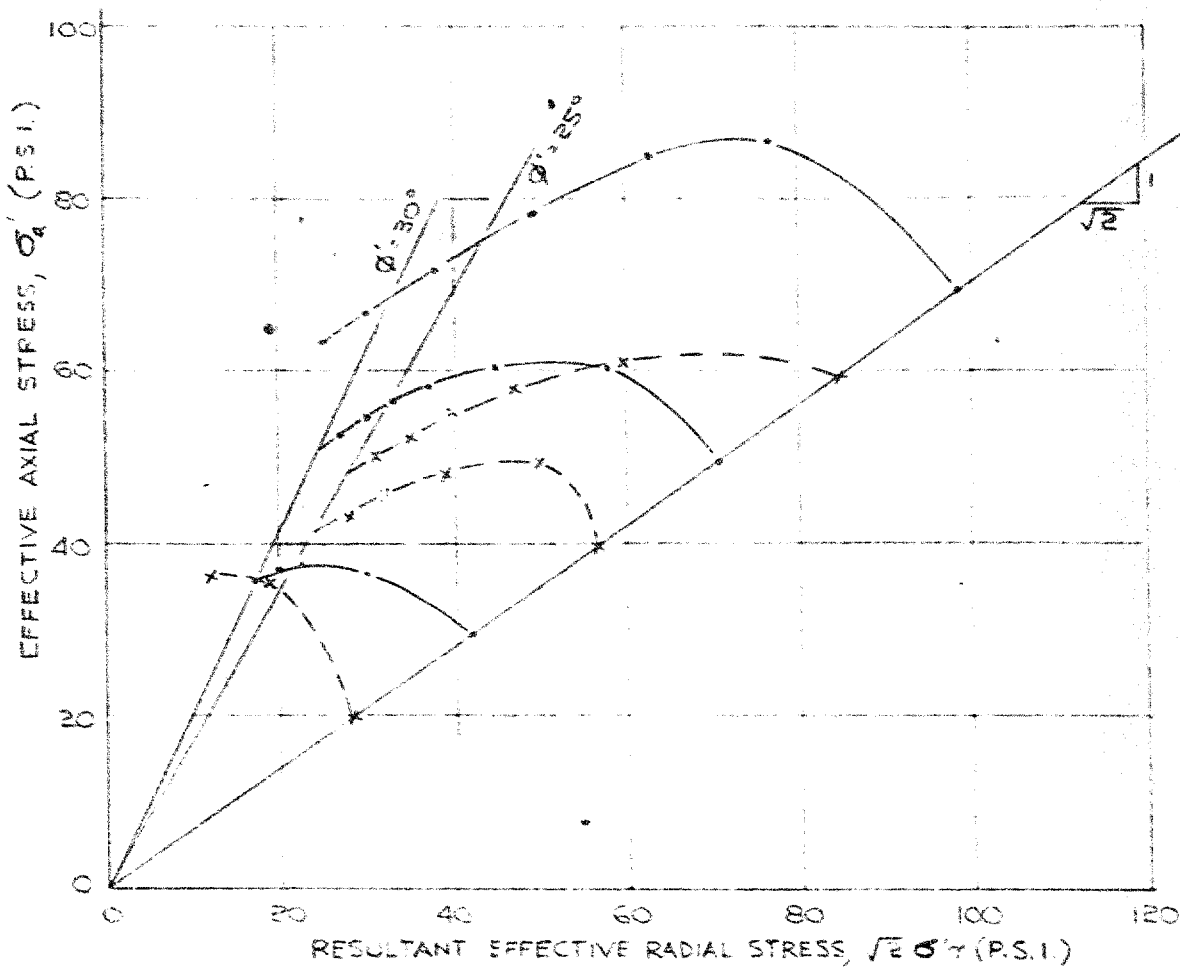
SUMMARY
SHEAR STRENGTH VS. ELEVATION

FIGURE 13



RENDULIC DIAGRAM CONSOLIDATED UNDRAINED TRIAXIAL TESTS STIFF GREY CLAY

FIGURE 14



LEGEND

SYMBOL	HOLE	DEPTH
x	5	21' to 22'
.	5	22' to 29'

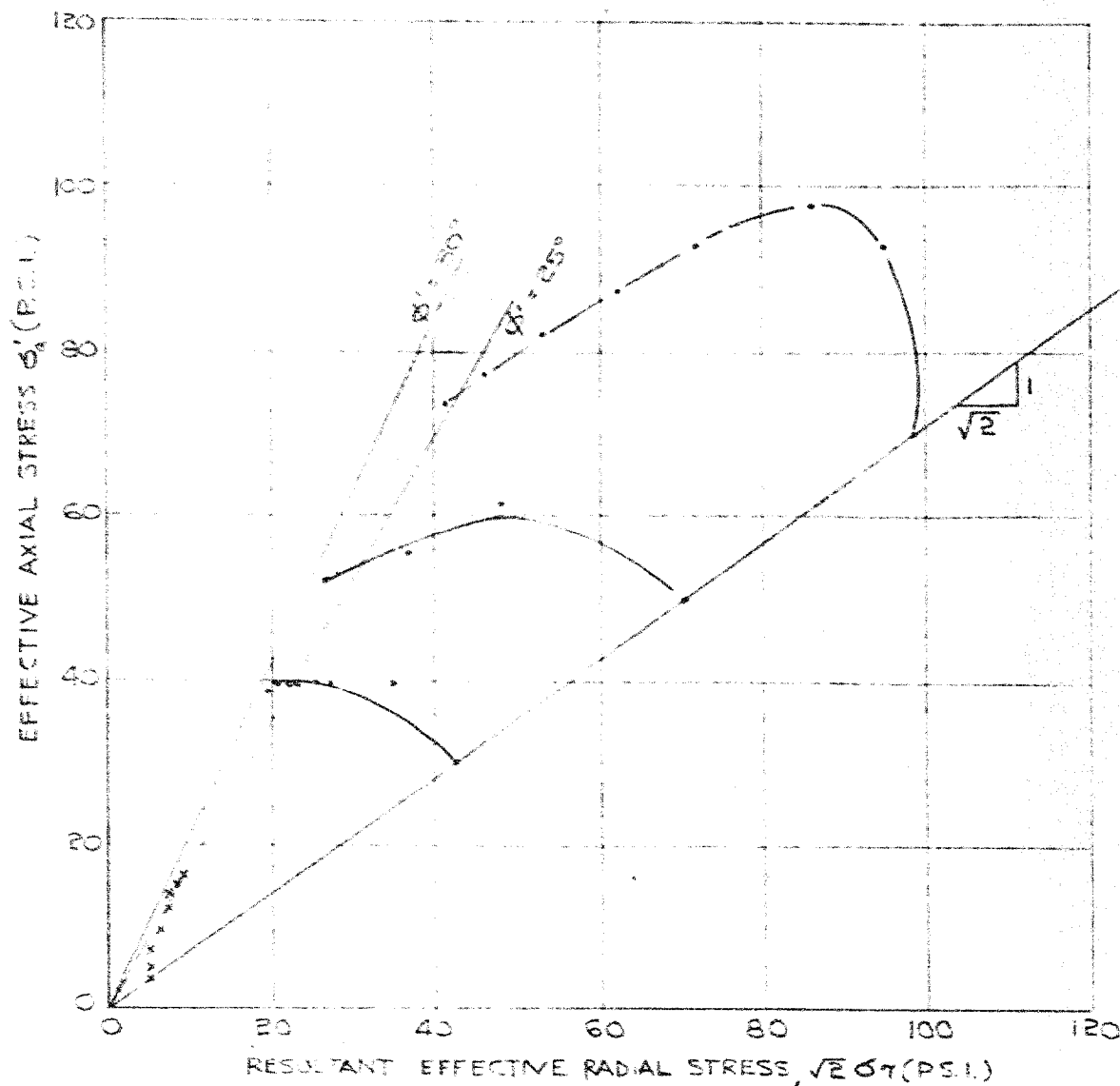
NOTE

POINTS ON DIAGRAM ARE PLOTTED FOR $\frac{1}{2}$ PERCENT INCREMENTS OF AXIAL STRAIN

GOLDER & ASSOCIATES

RENDULIC DIAGRAM CONSOLIDATED UNDRAINED TRIAXIAL TESTS STIFF GREY SILTY CLAY

FIGURE 15



LEGEND

SYMBOL	HOLE	DEPTH
x	5	39' To 40'
•	5	50' To 51'

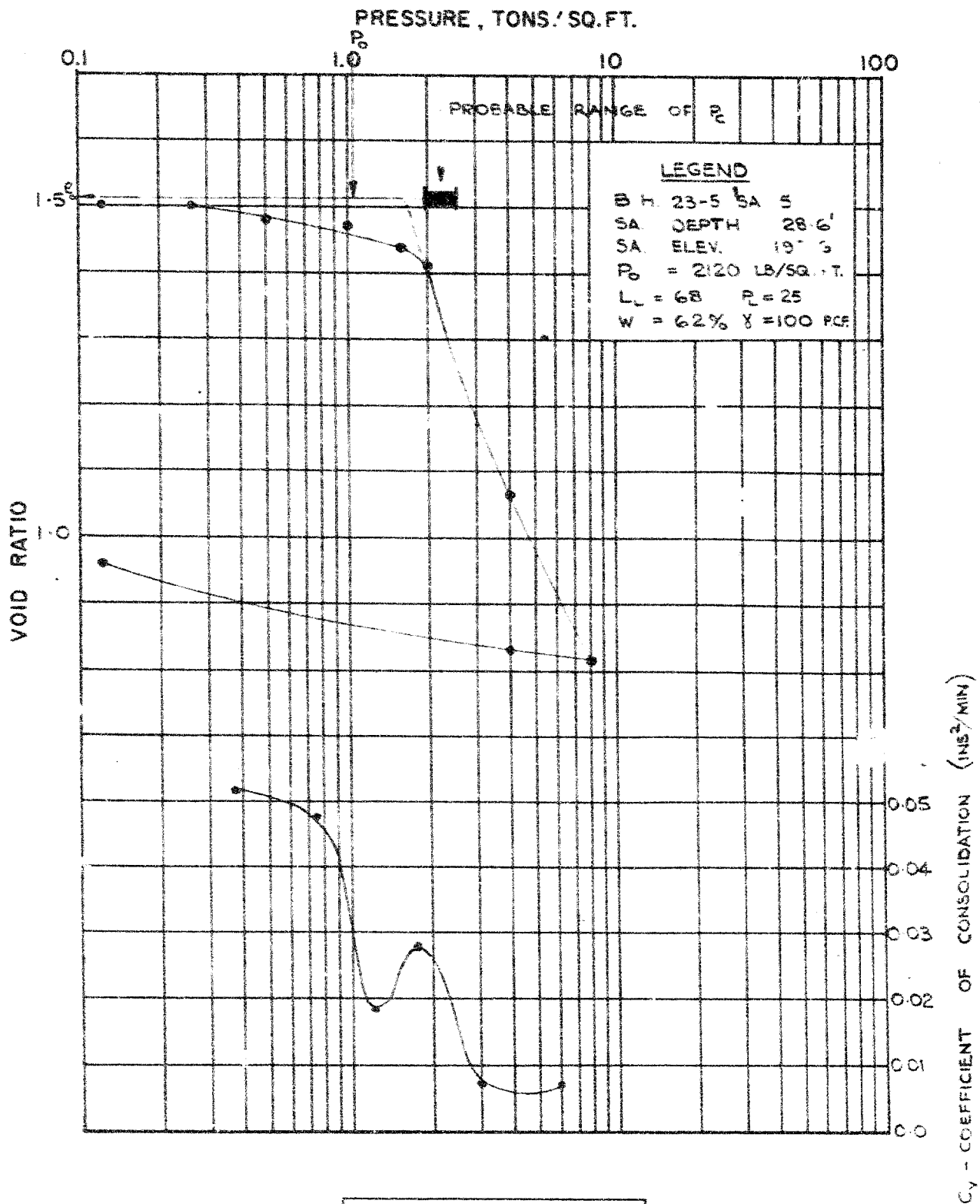
NOTE

POINTS ON DIAGRAM ARE PLOTTED FOR $\frac{1}{2}$ PERCENT INCREMENTS OF AXIAL STRAIN

GOLDER & ASSOCIATES

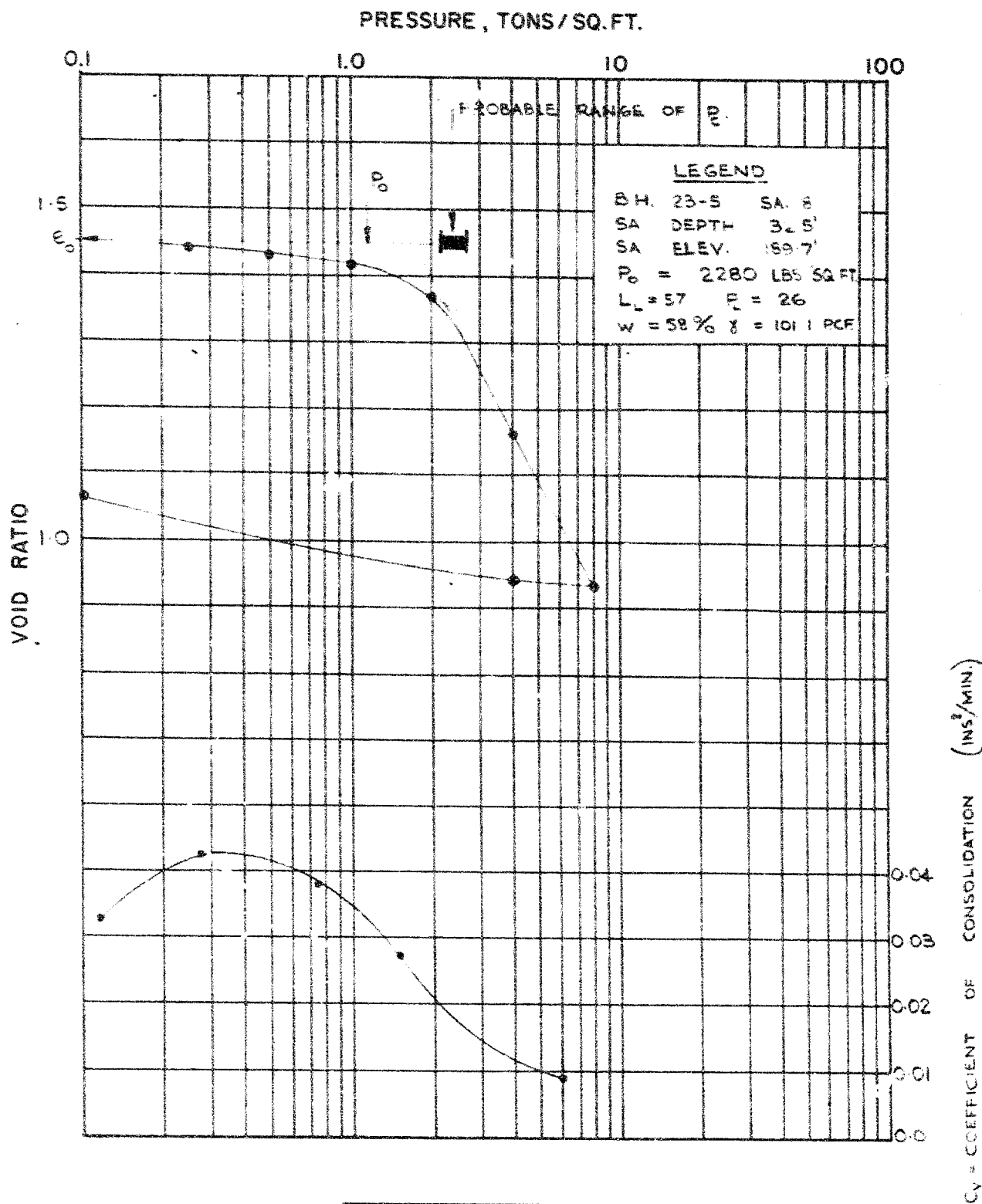
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 16



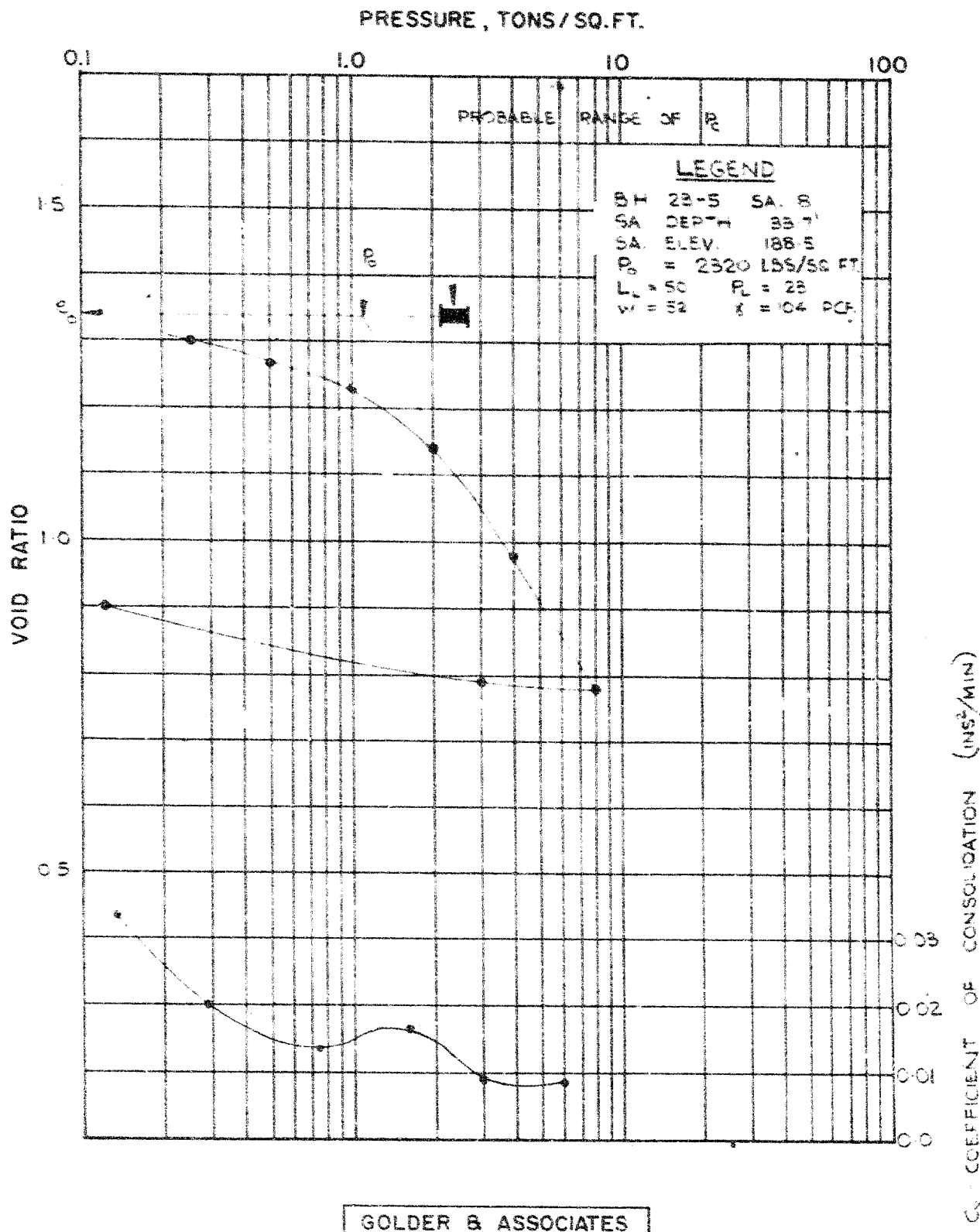
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 17



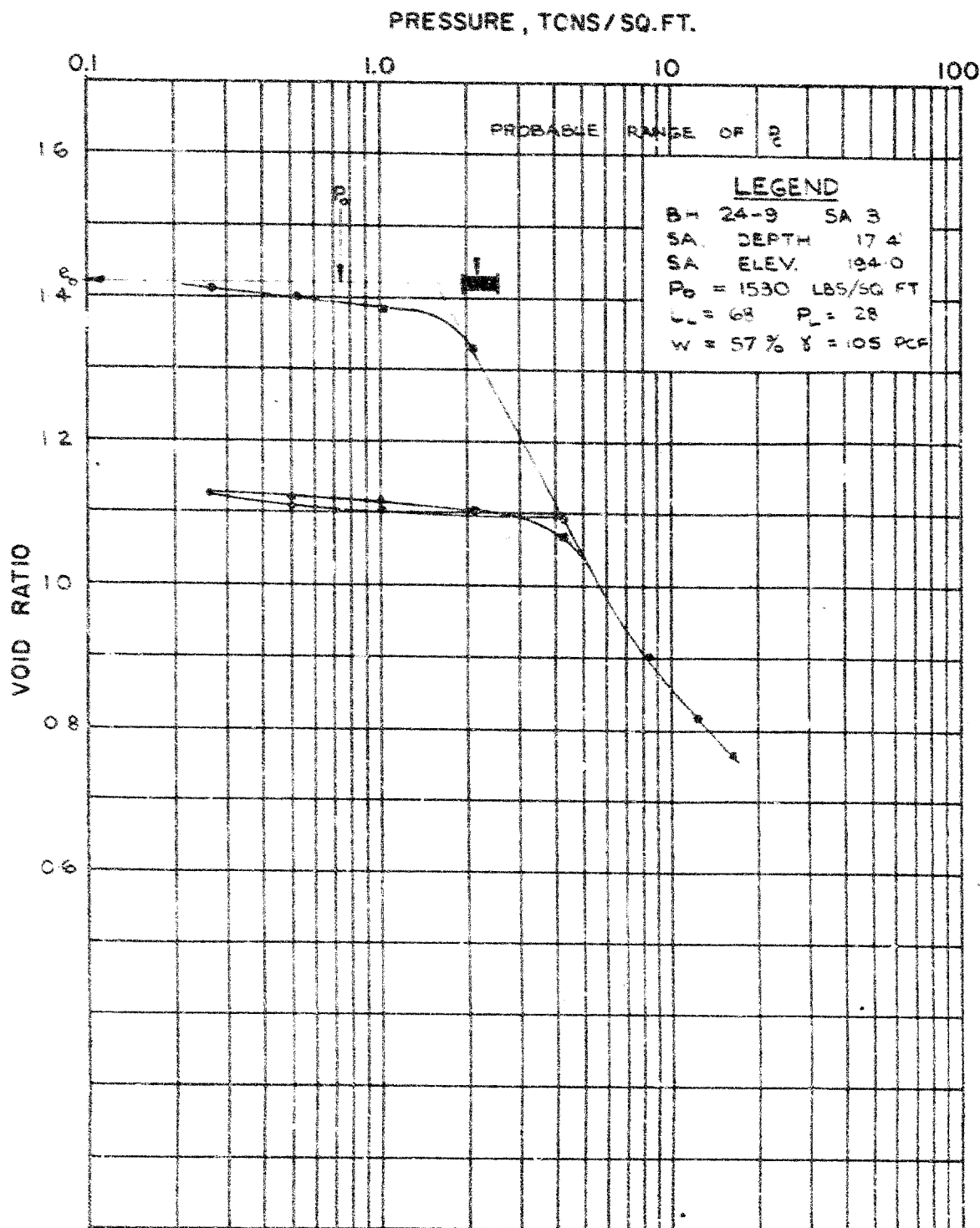
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 18



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 19

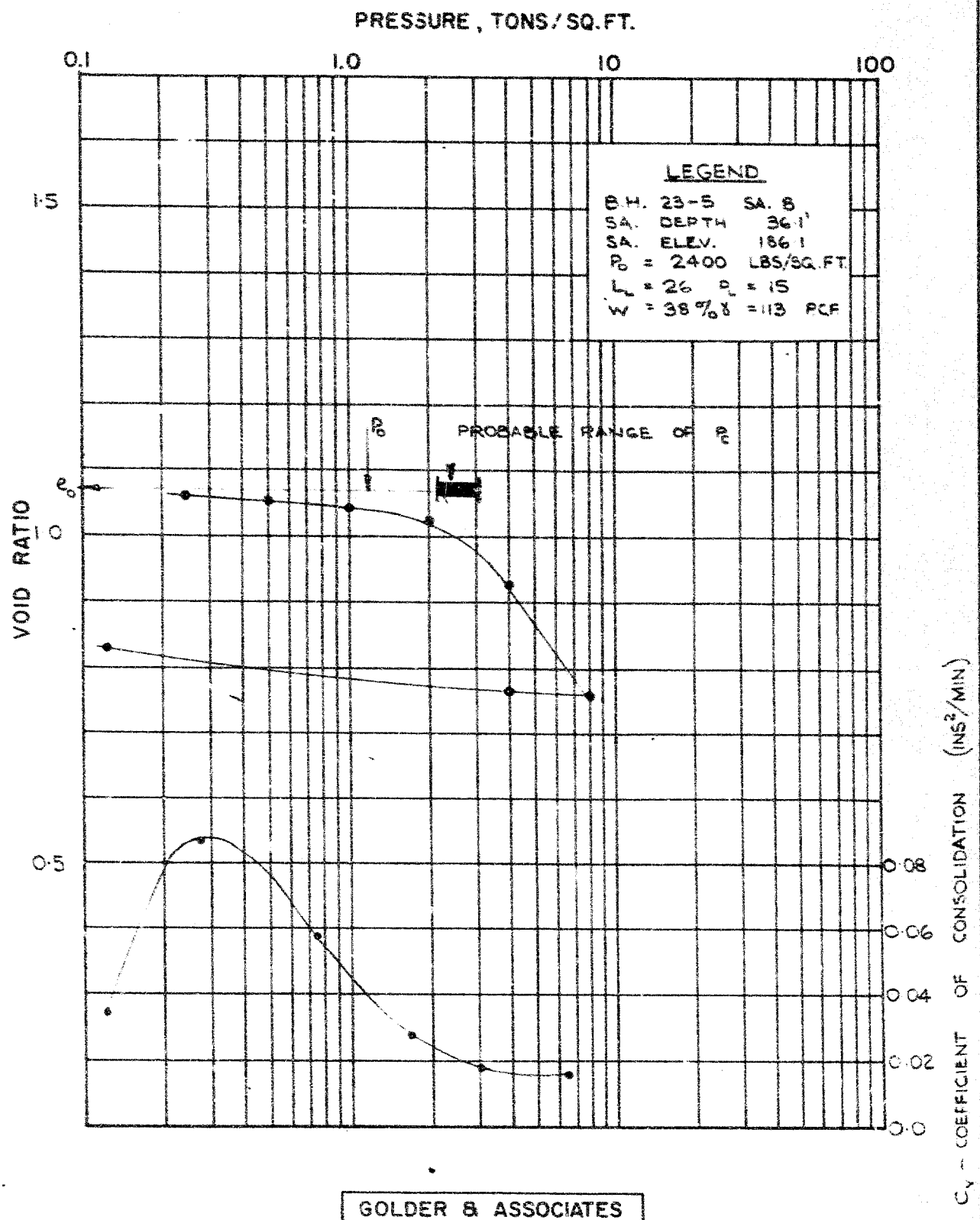


GOLDER & ASSOCIATES

PROJECT No. 6102

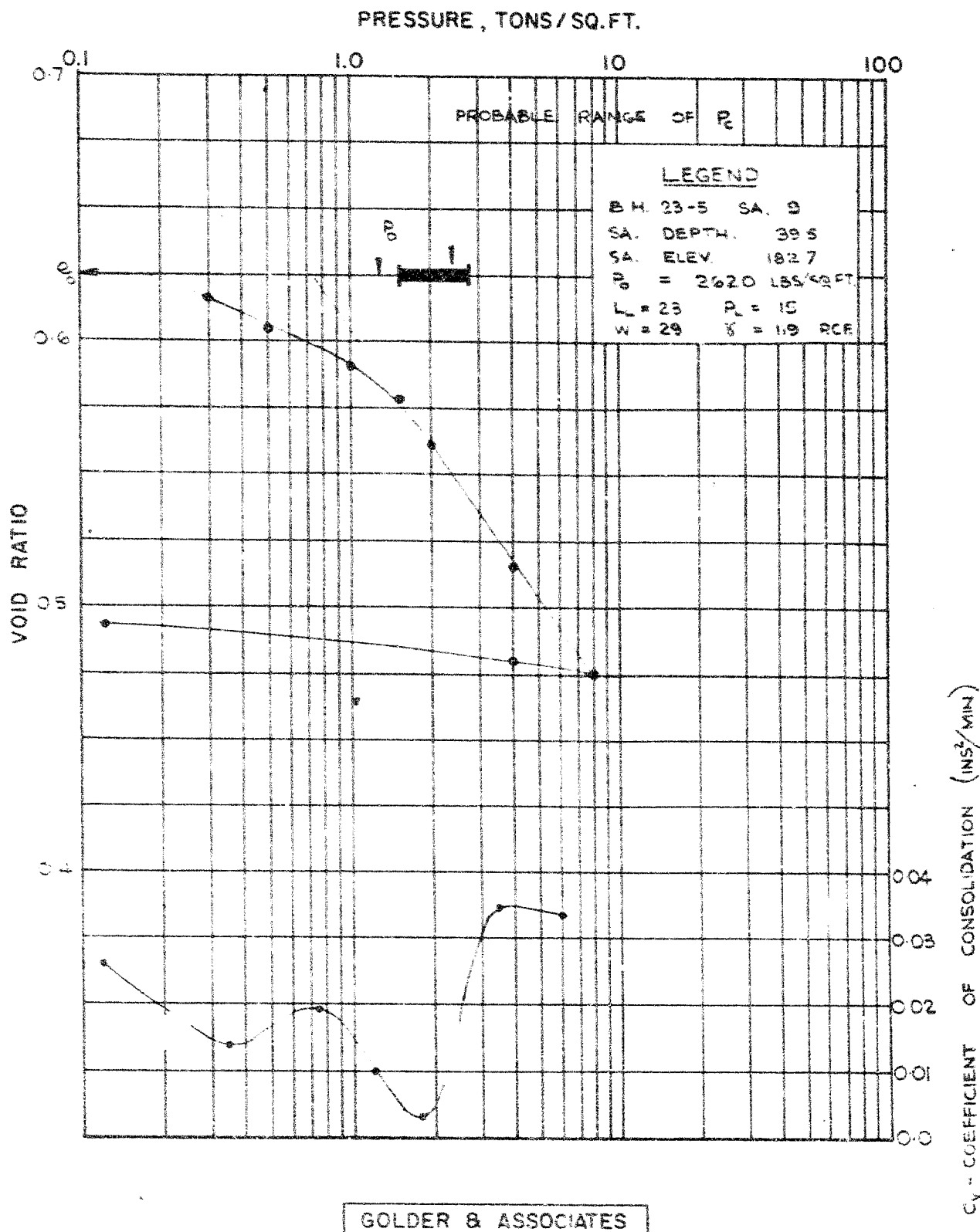
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 20



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

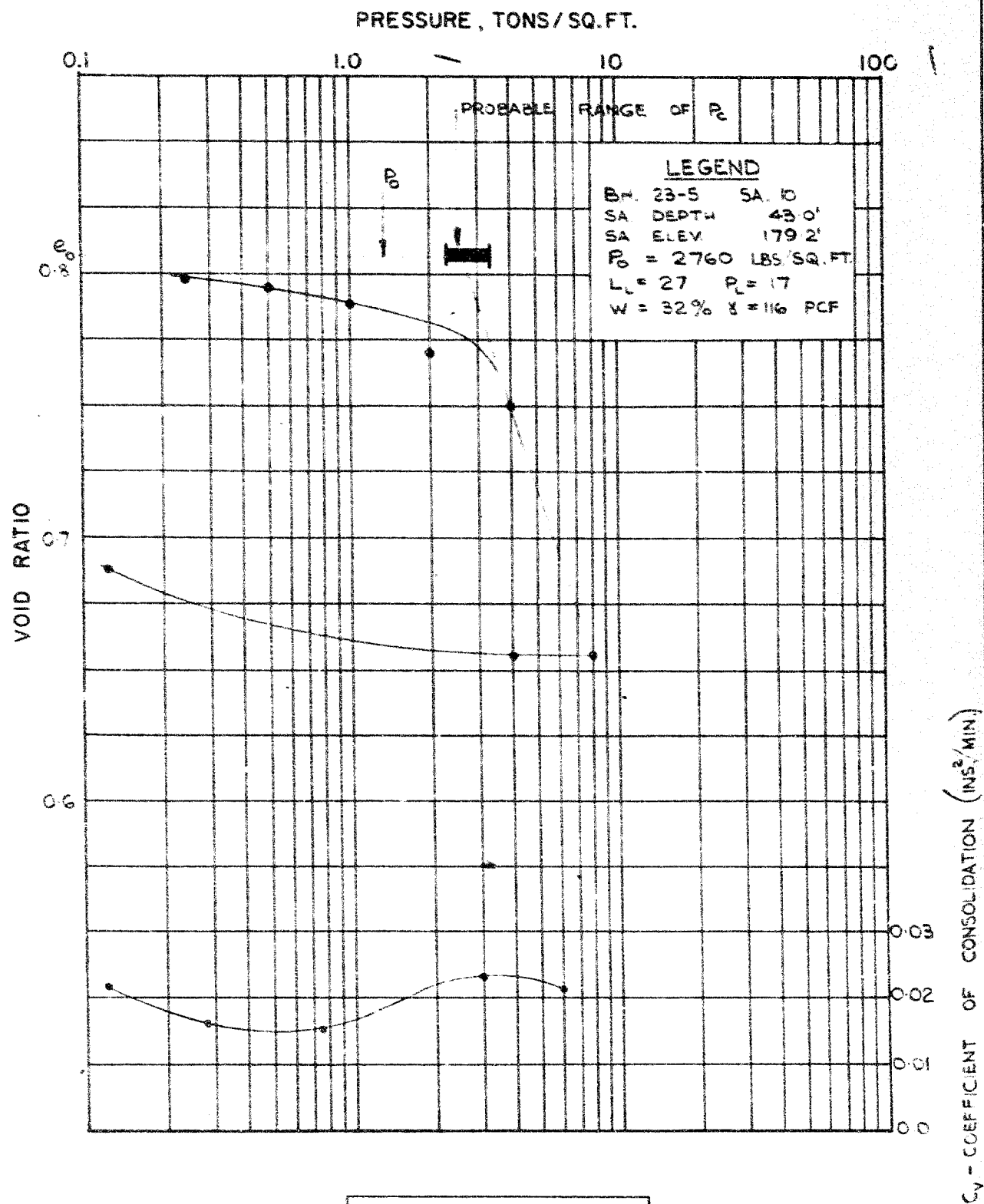
FIGURE 21



PROJECT No. 6103

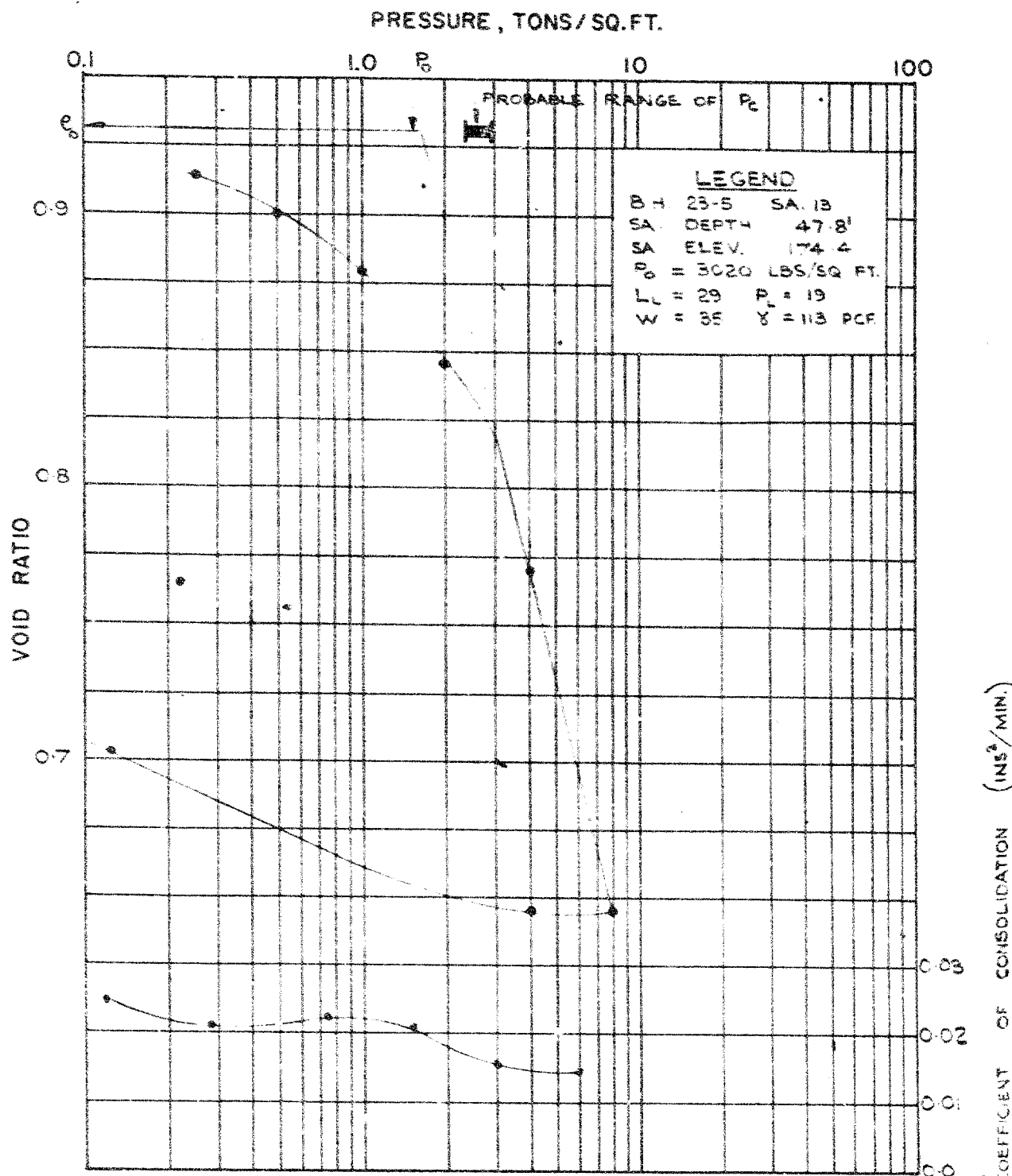
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 22



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

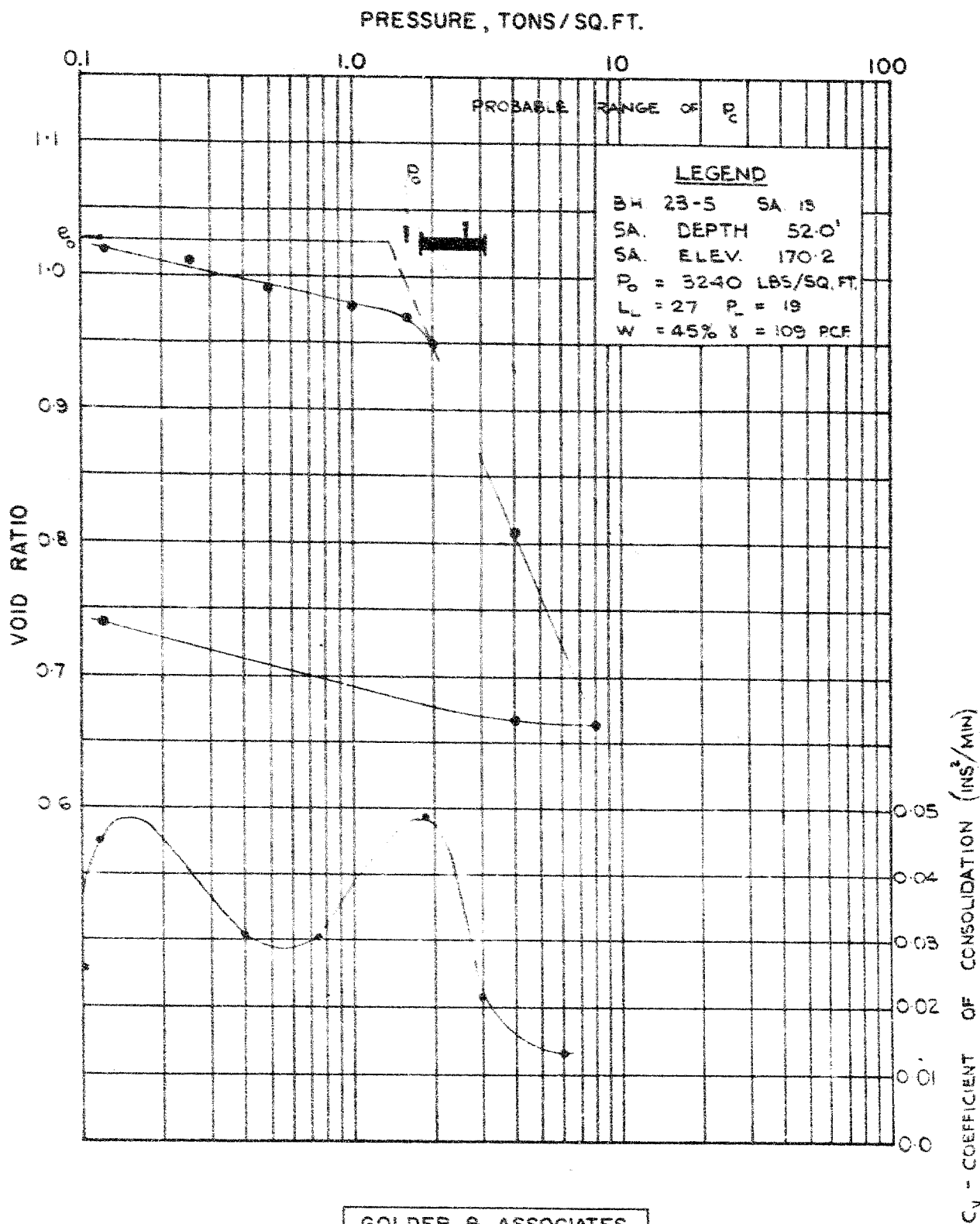
FIGURE 23



GOLDER & ASSOCIATES

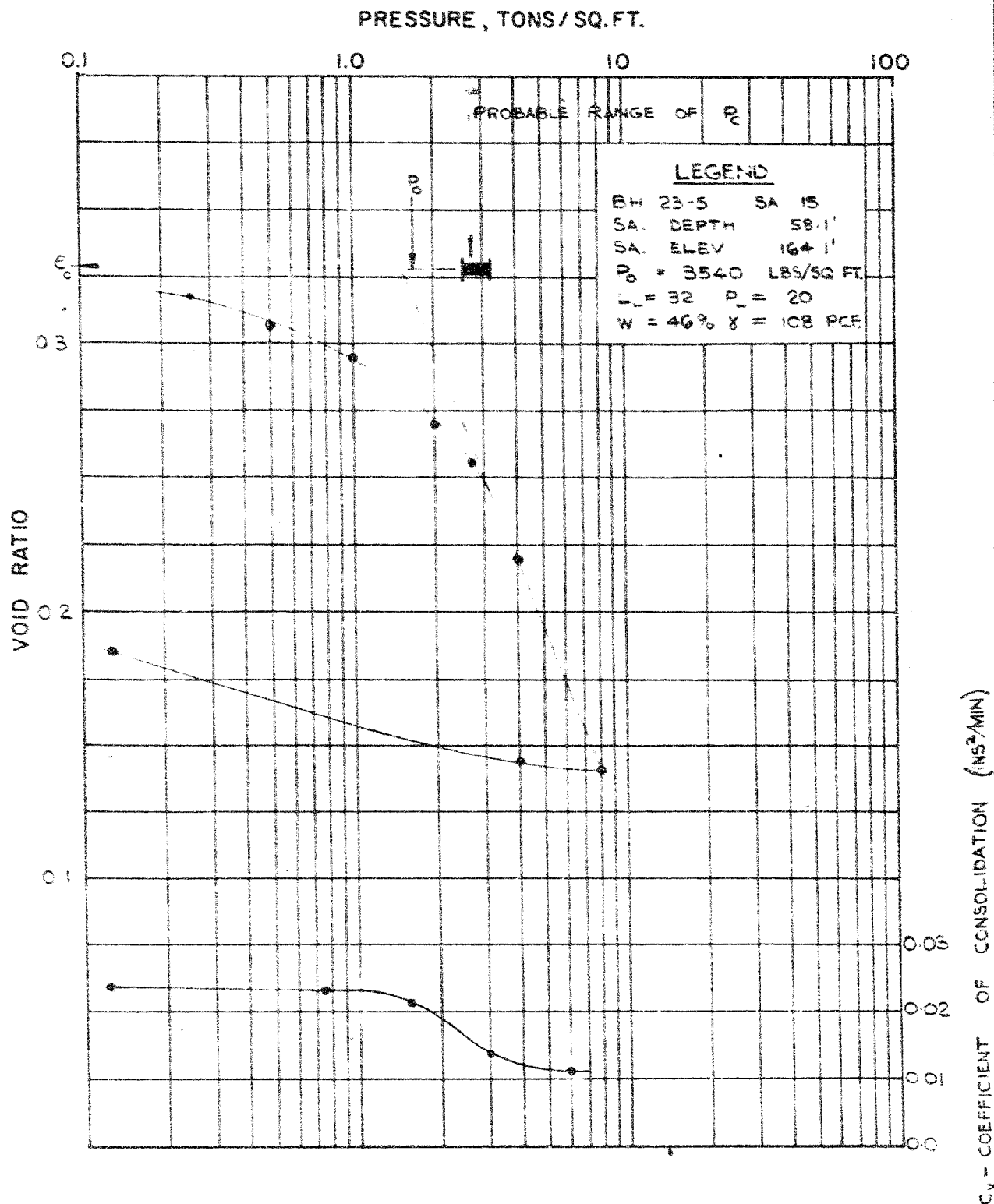
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 24



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

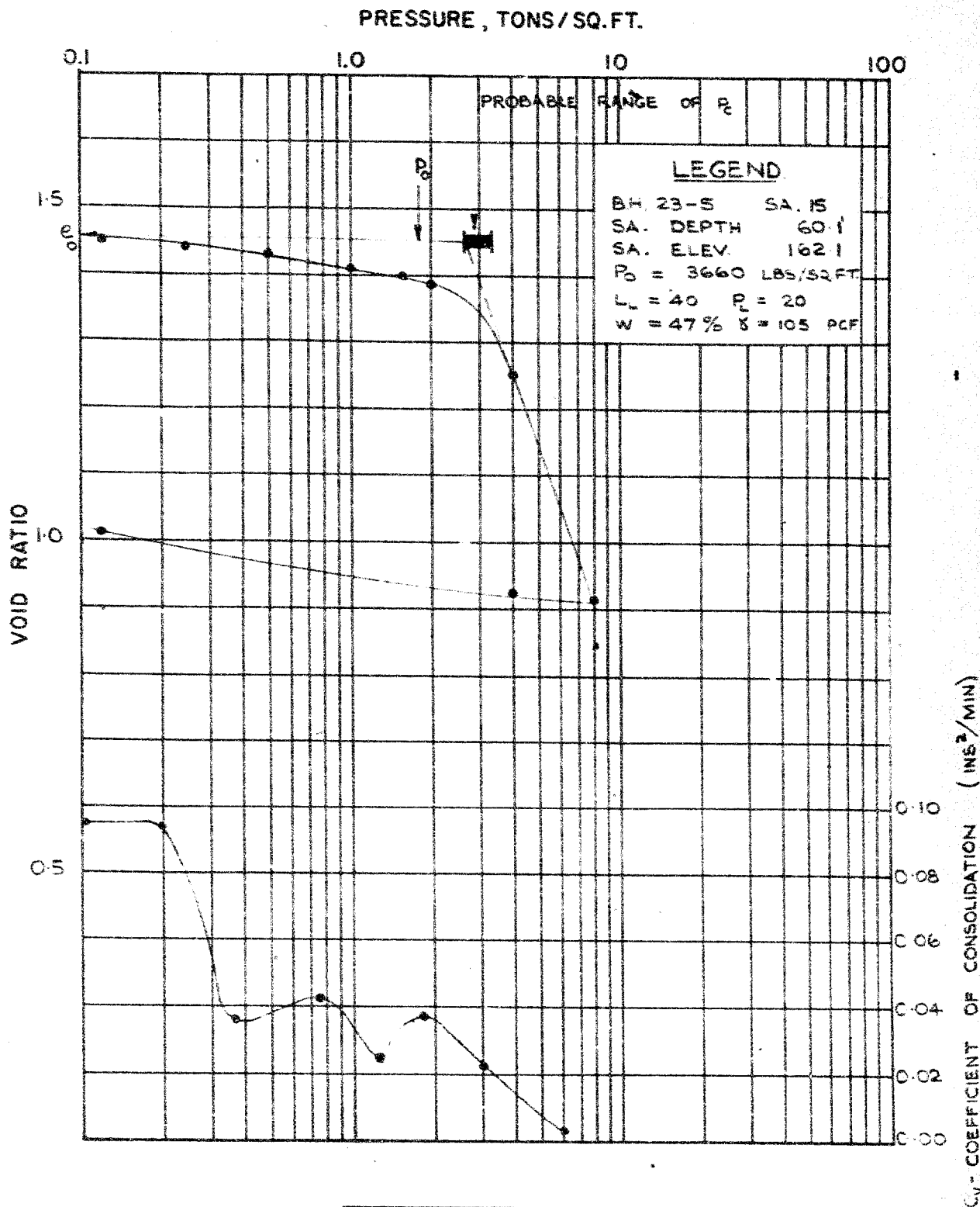
FIGURE 26



GOLDER & ASSOCIATES

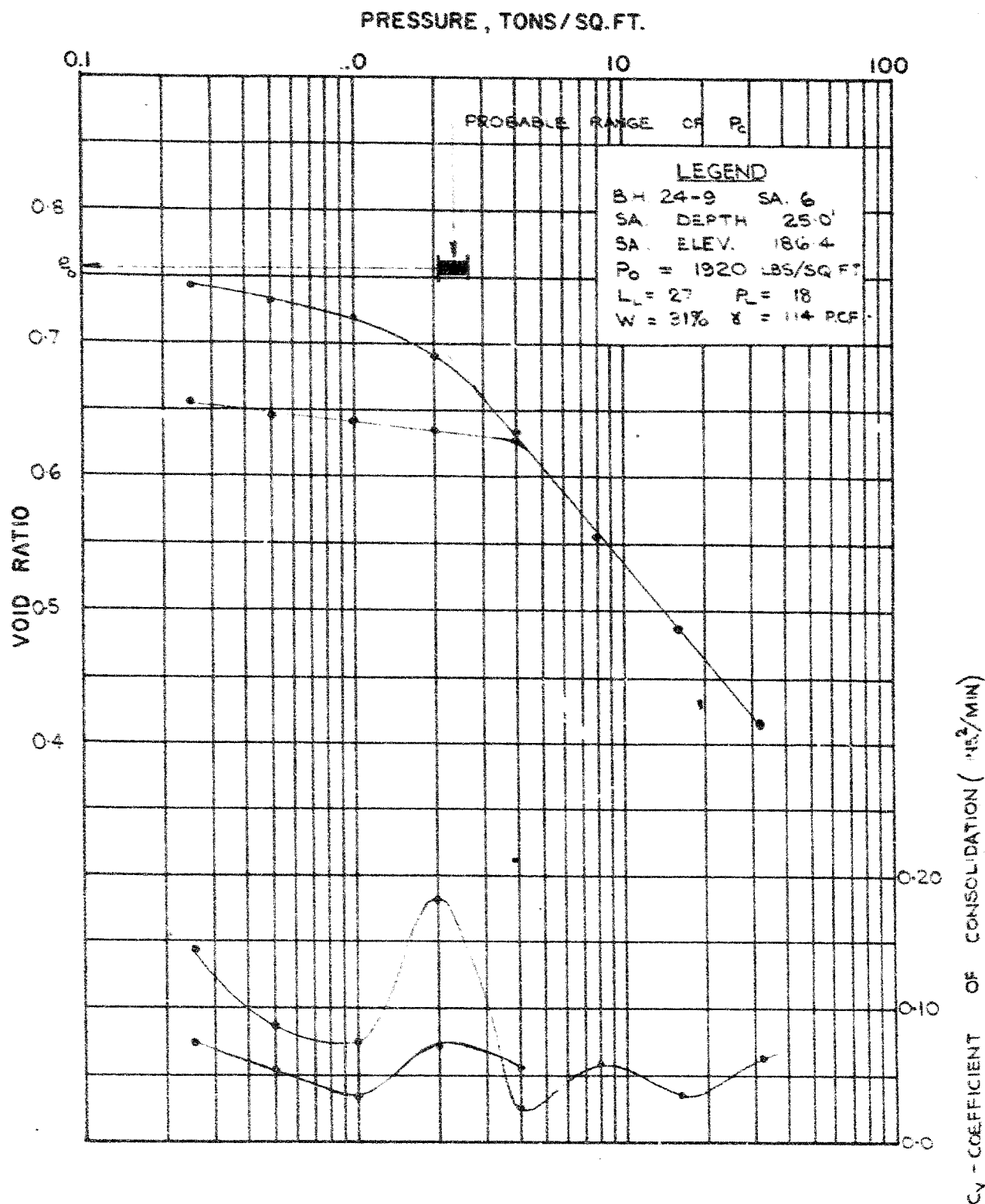
VOID RATIO - PRESSURE CURVES CONSOLIDATION, TEST

FIGURE 27



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

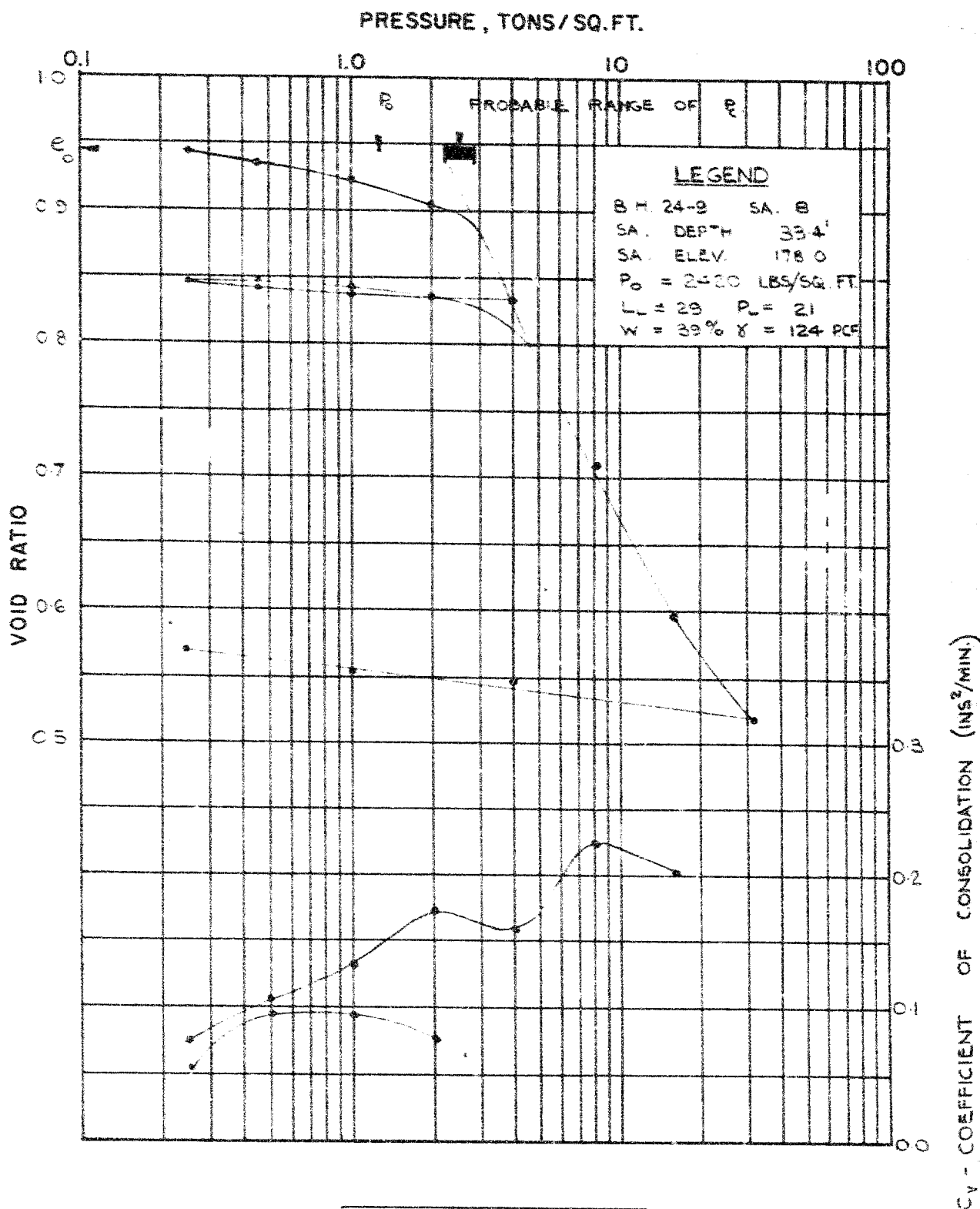
FIGURE 28



GOLDER & ASSOCIATES

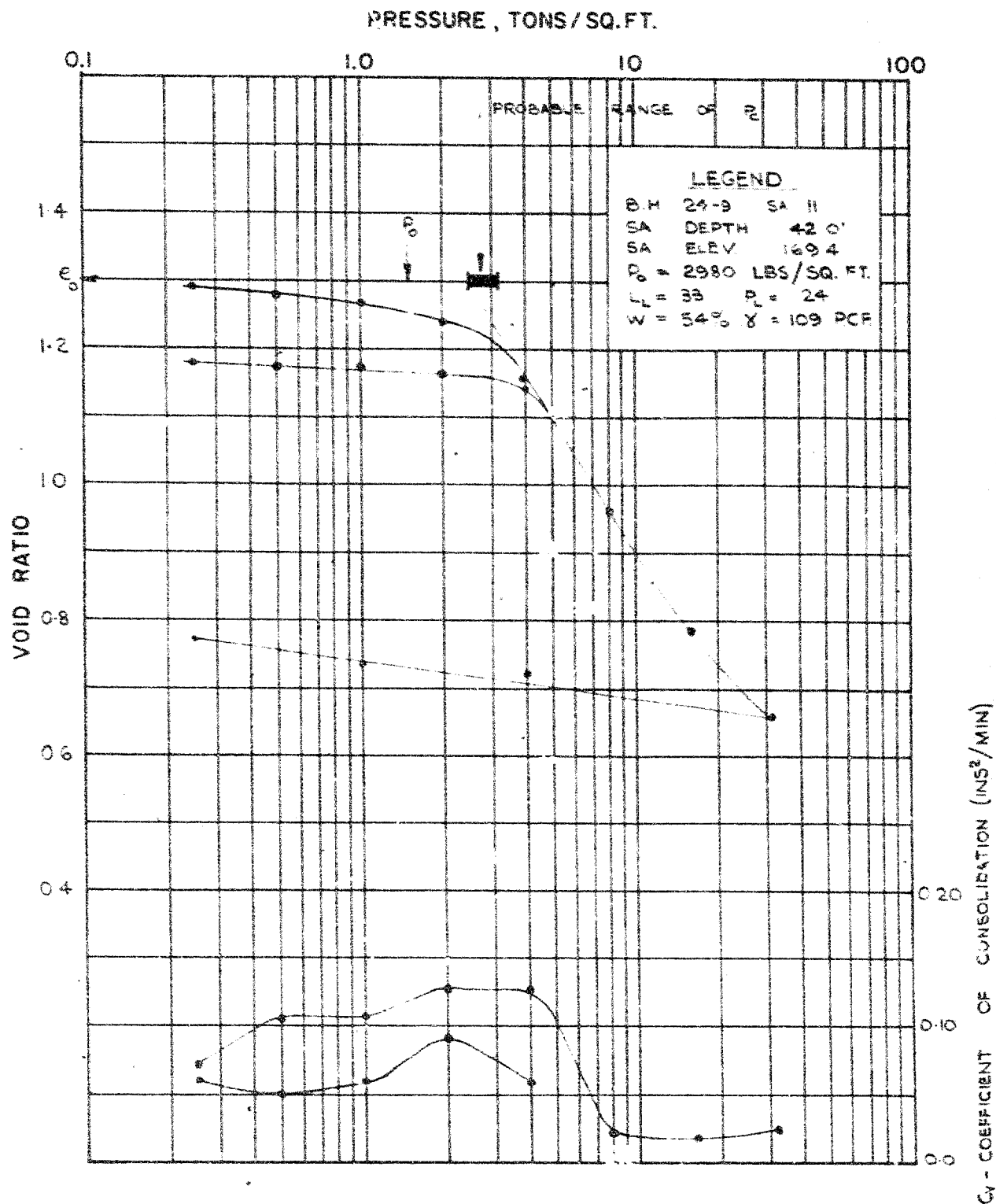
VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 9

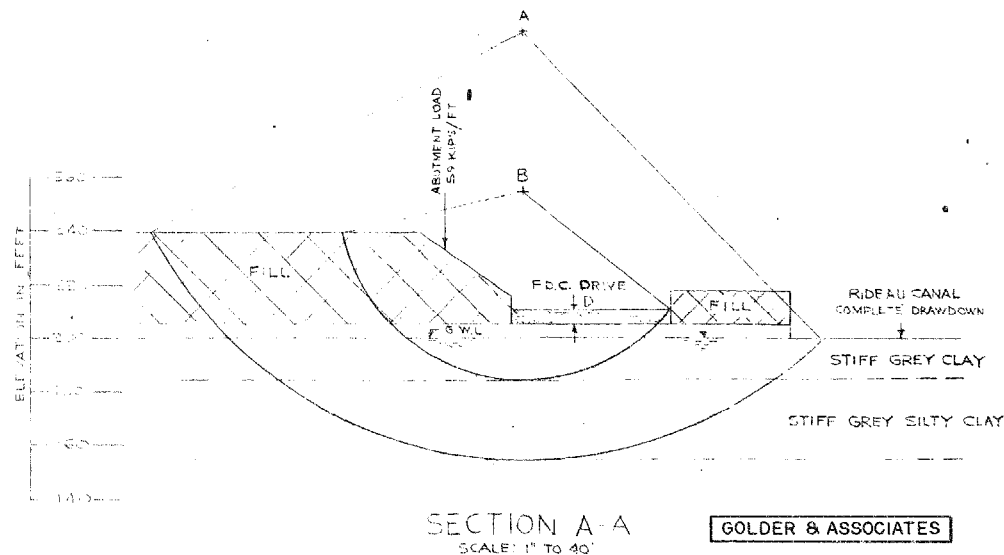
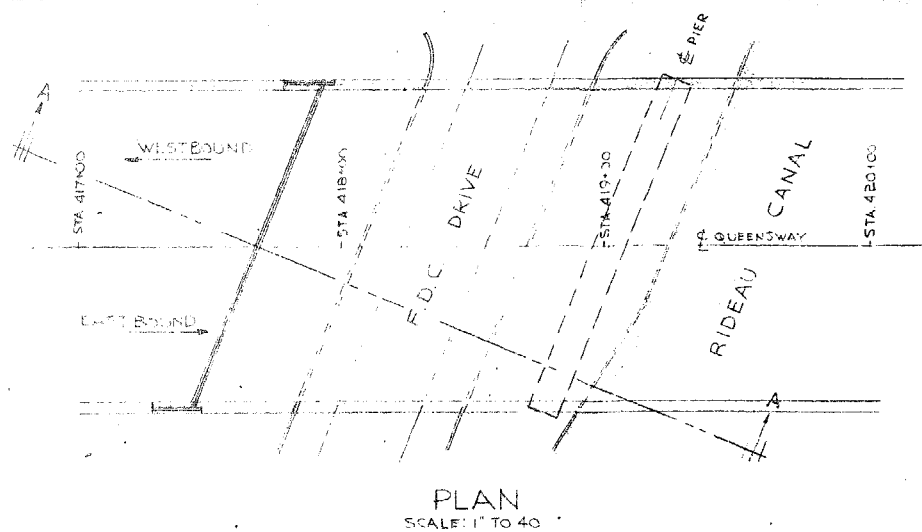


VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

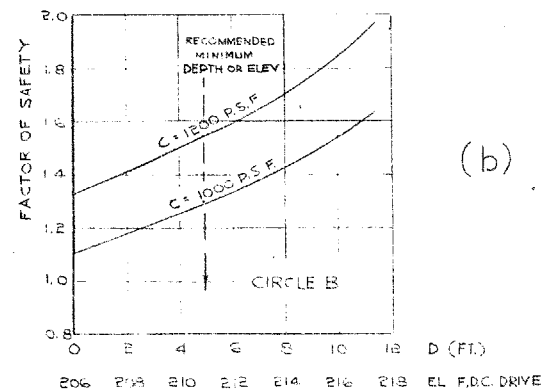
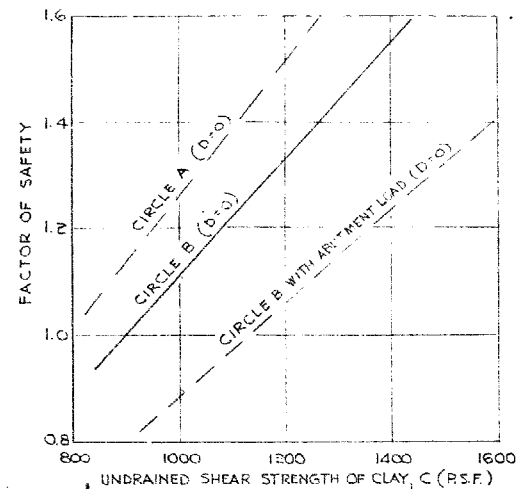
FIGURE 30

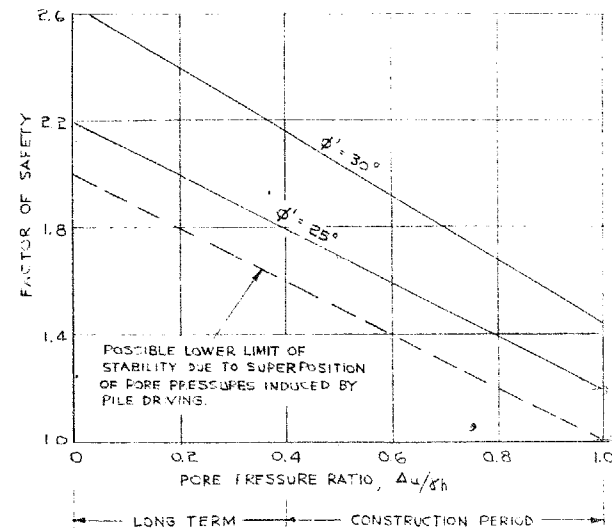
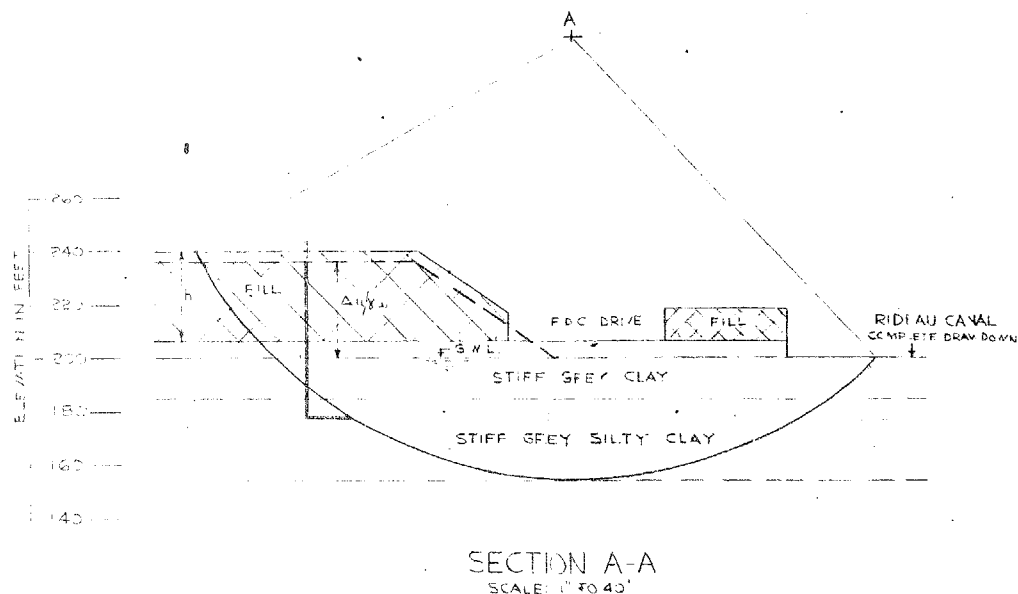


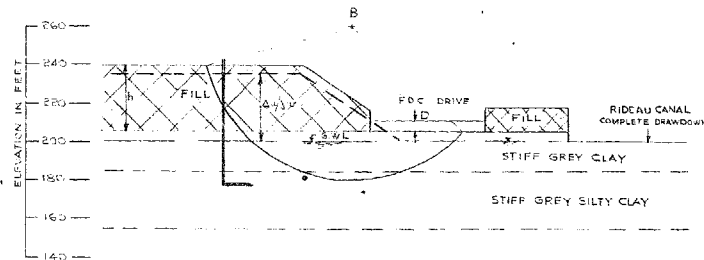
GOLDER & ASSOCIATES



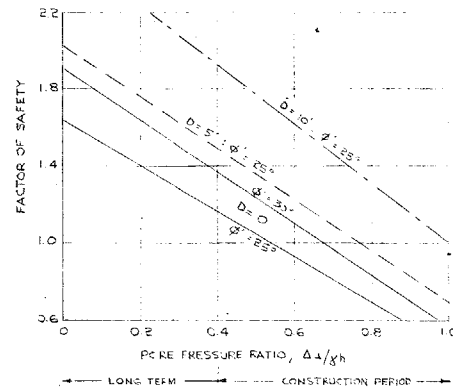
GOLDER & ASSOCIATES



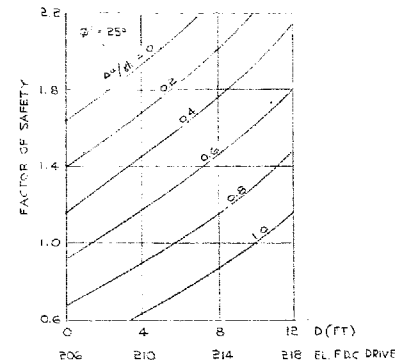




SECTION A-A'
SCALE: 1" TO 43'



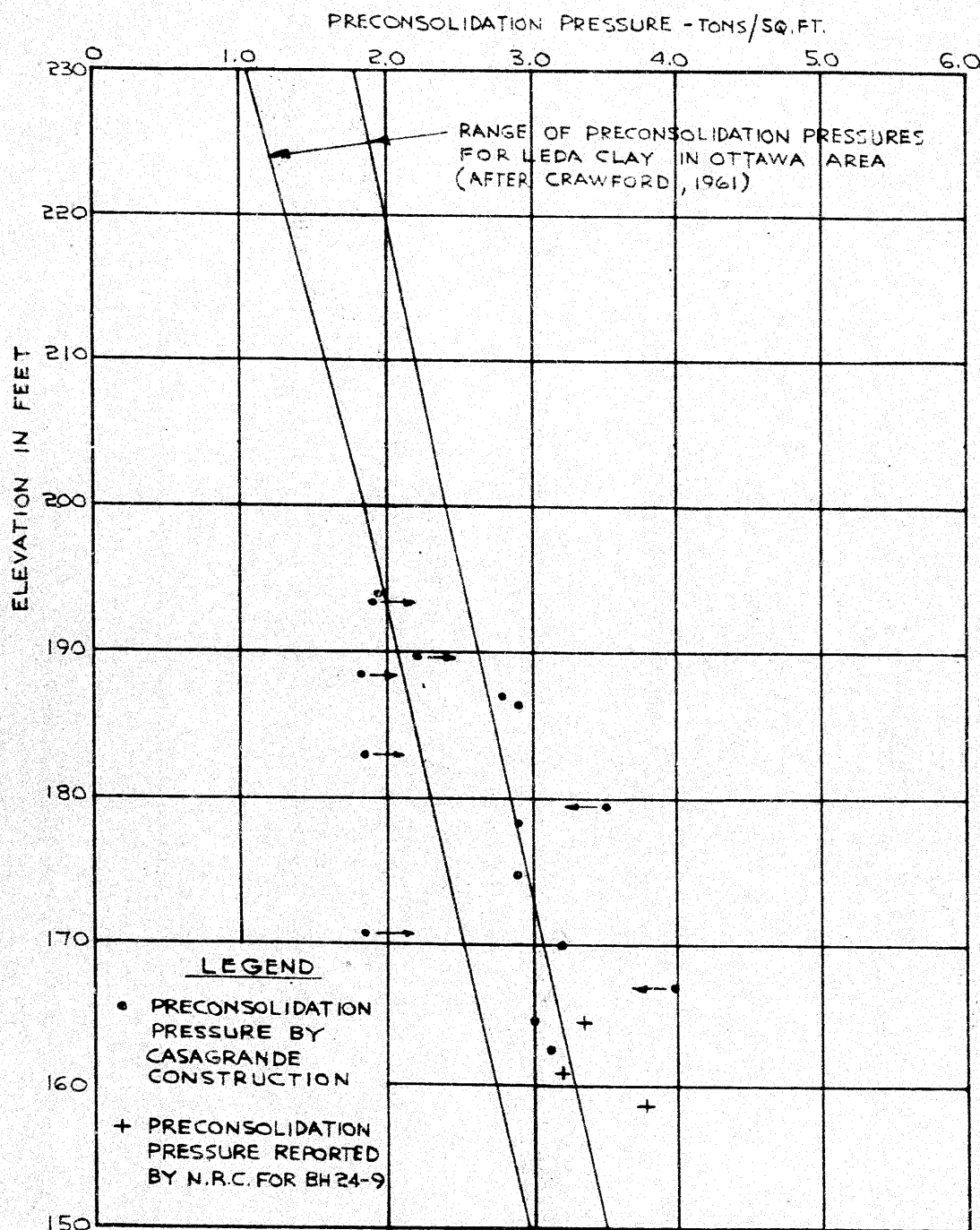
(a)



(b)

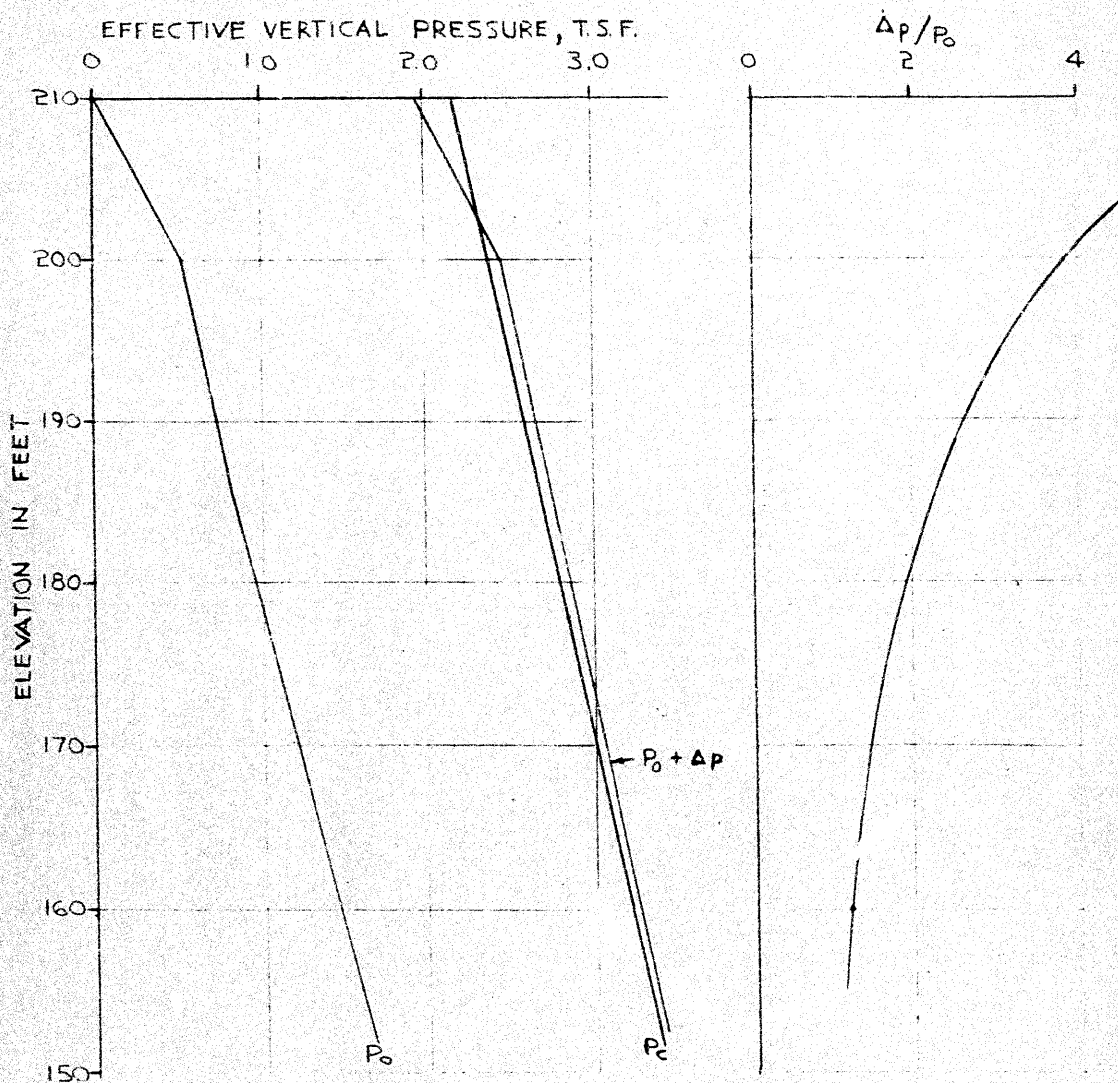
PRECONSOLIDATION PRESSURE VS ELEVATION CLAY AND SILTY CLAY STRATA

FIGURE 34



STRESS CONDITIONS - WEST ABUTMENT RIDEAU CANAL BRIDGE

FIGURE 35

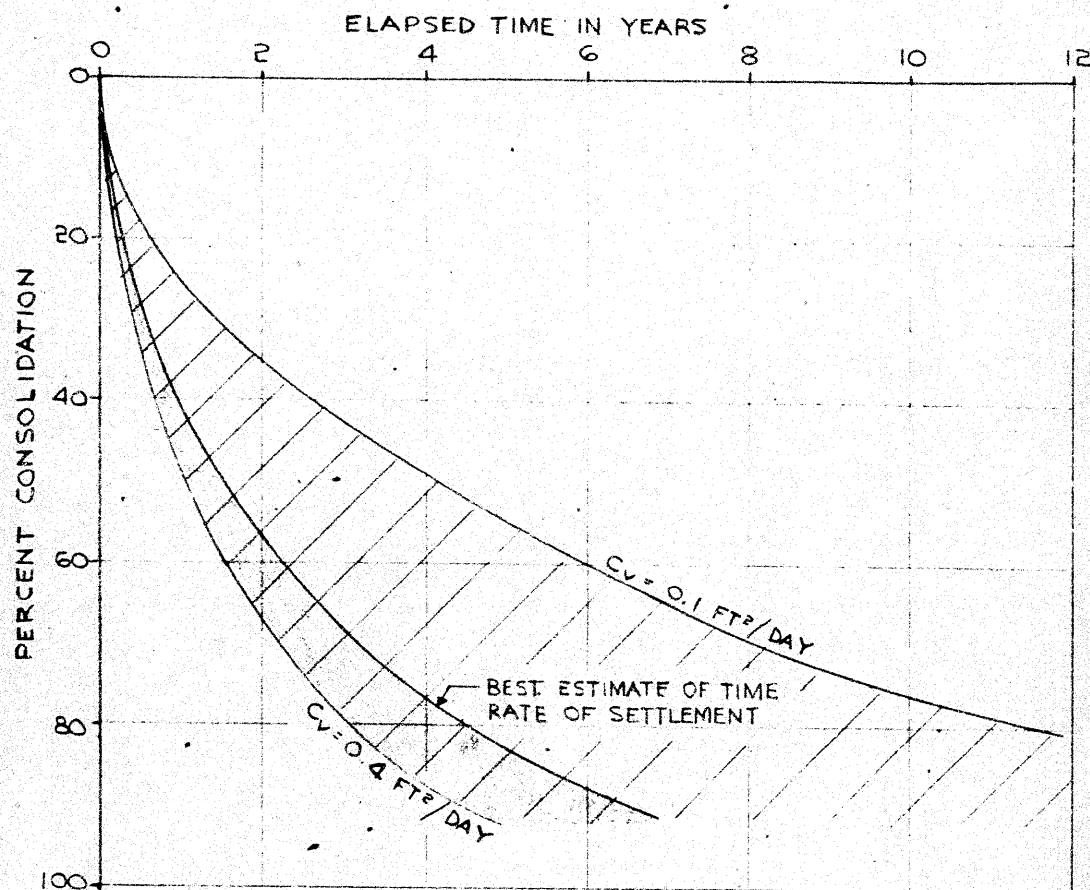


LEGEND

- P_0 - EXISTING OVERBURDEN PRESSURE
- ΔP - INCREASE IN VERTICAL PRESSURE DUE TO WEIGHT OF EMBANKMENT
- P_c - OVERCONSOLIDATION LOAD

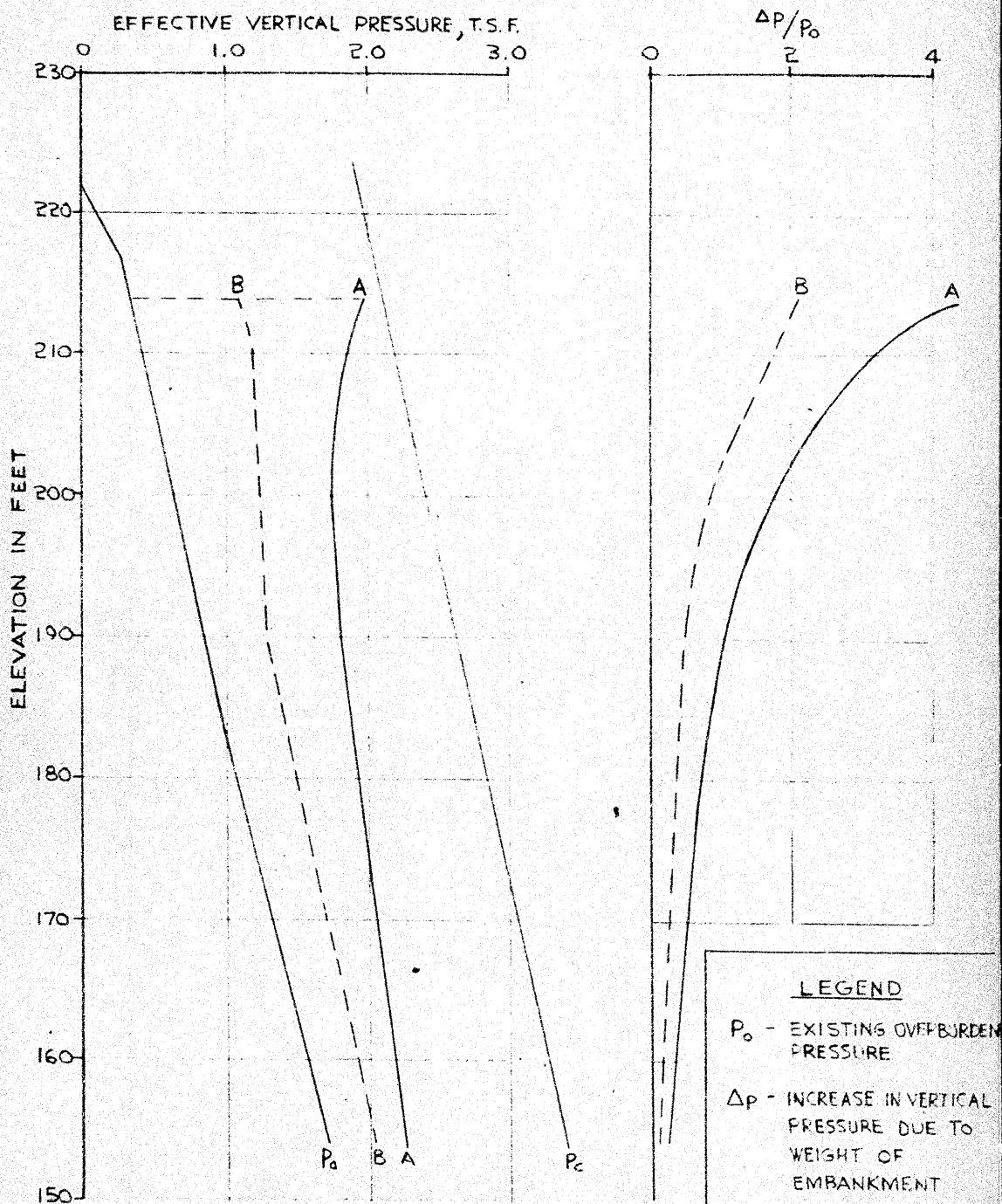
TIME RATE OF SETTLEMENT PREDICTED FROM CONSOLIDATION TESTS

FIGURE 35



STRESS CONDITIONS ELGIN STREET OVERPASS

FIGURE 37



A - STRESS CONDITIONS BENEATH CENTRE OF ABUTMENT
B - STRESS CONDITIONS BENEATH OUTER EDGE OF ABUTMENT

GOLDER & ASSOCIATES

LEGEND

P_o - EXISTING OVERBURDEN PRESSURE

Δp - INCREASE IN VERTICAL PRESSURE DUE TO WEIGHT OF EMBANKMENT

P_c - OVERCONSOLIDATION LOAD



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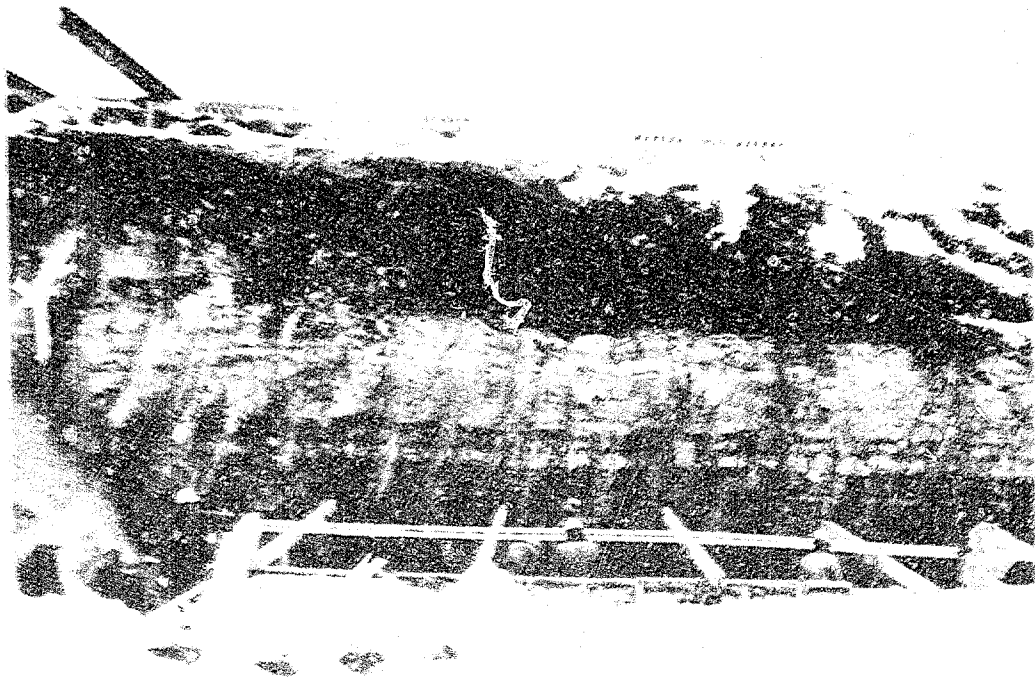


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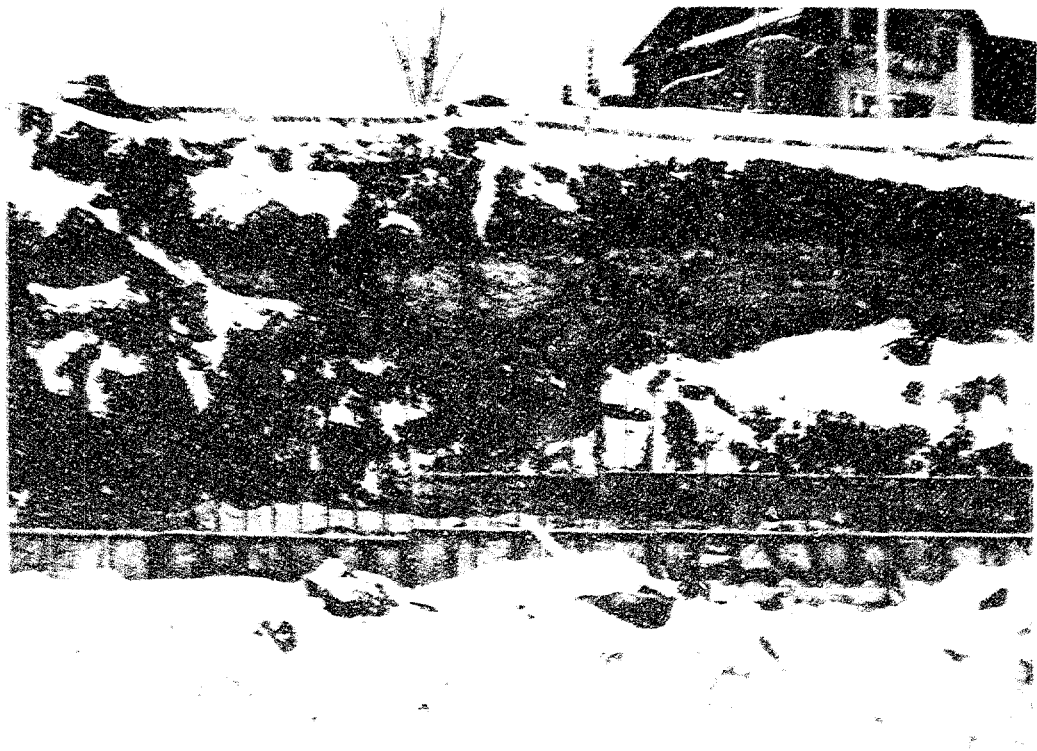
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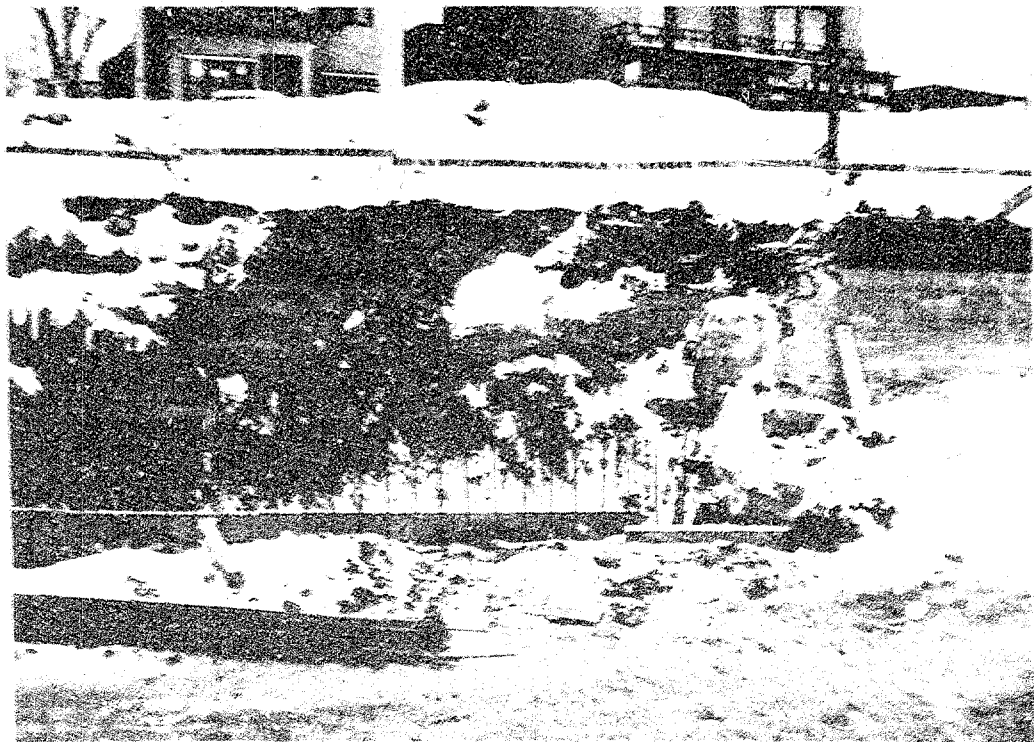


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Jan 8/69

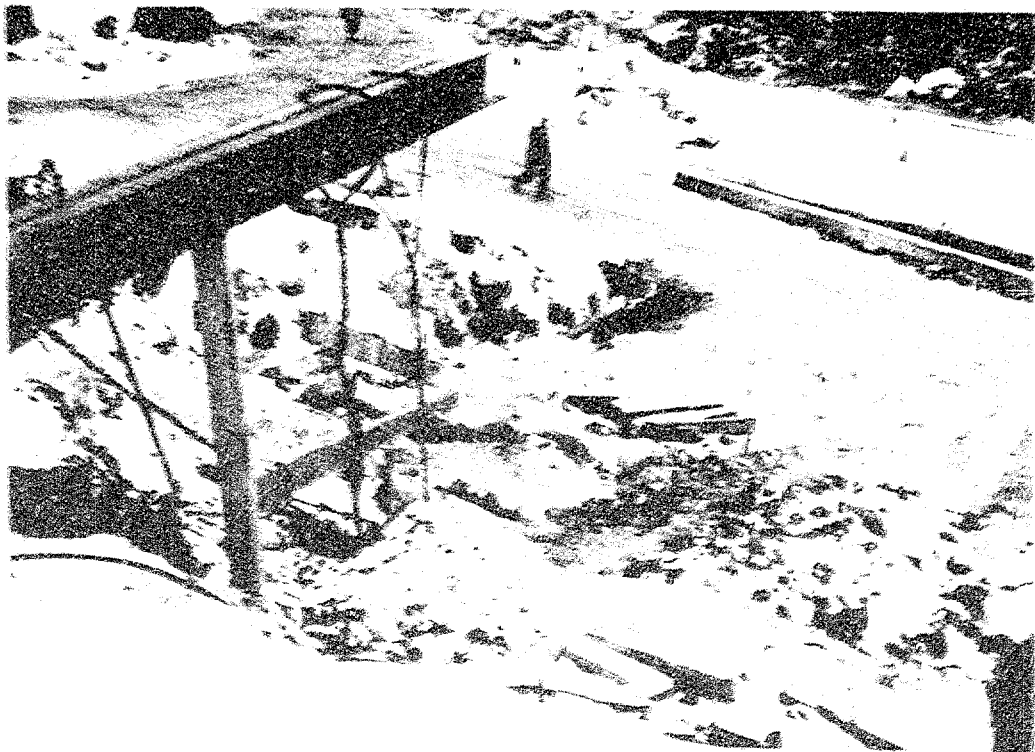
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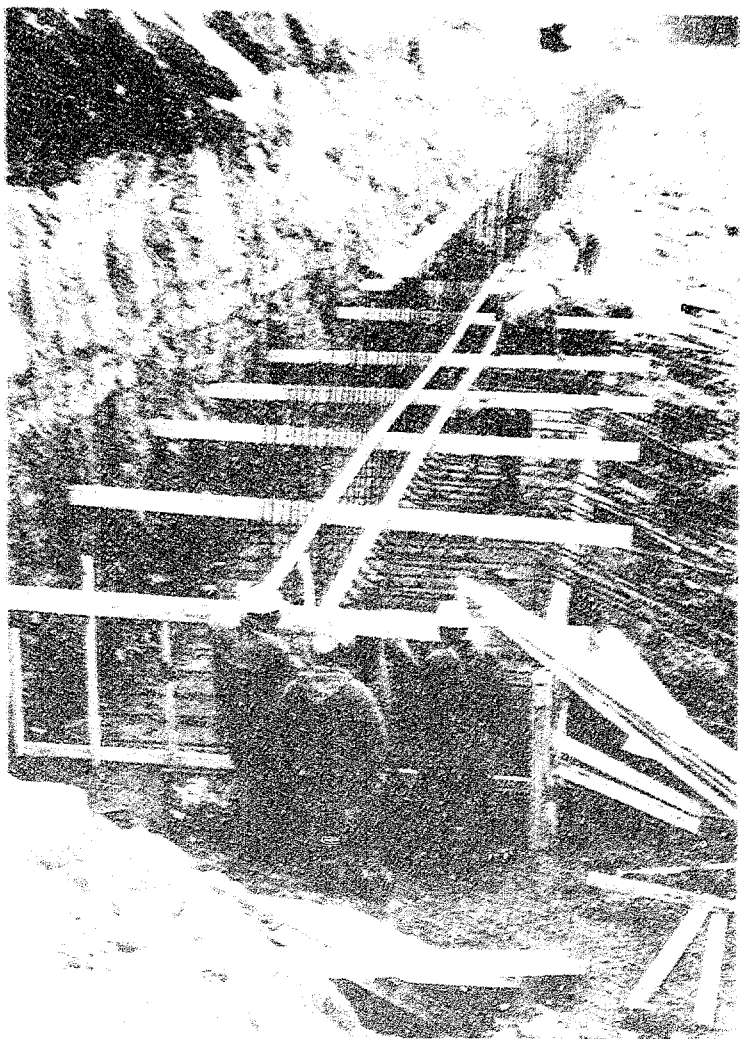


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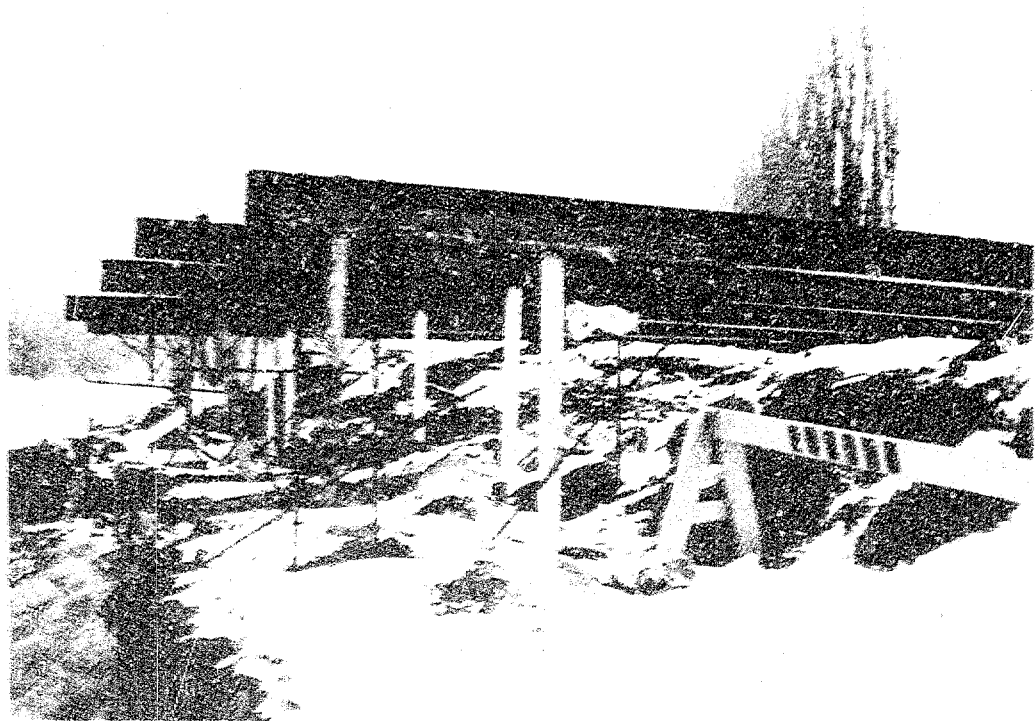


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W.P. #951-59

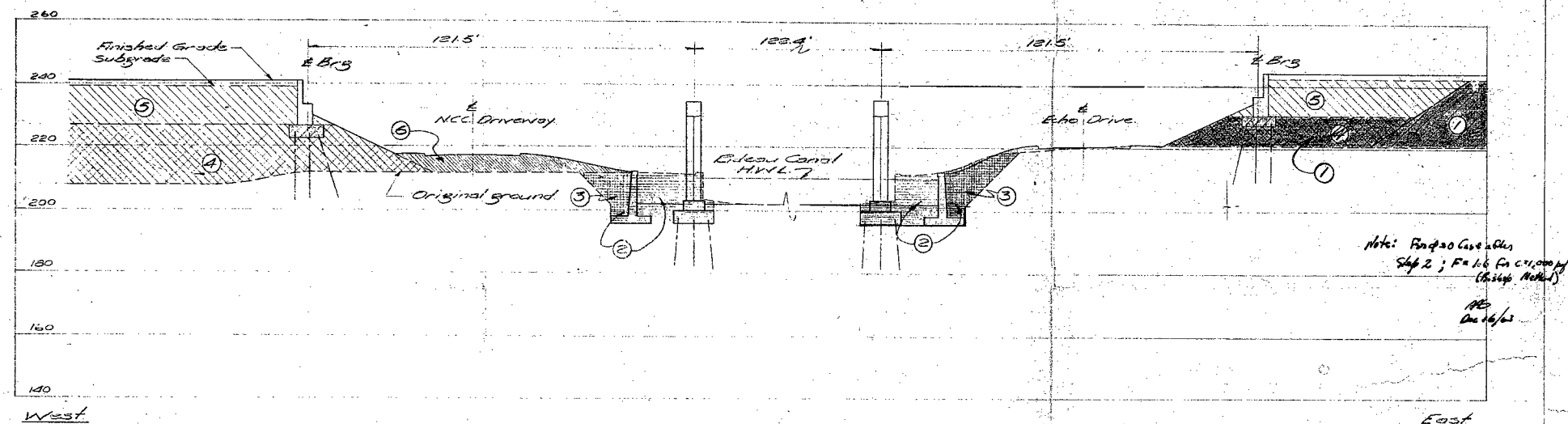
#952-59

QUEENSWAY

RIDEAU CANAL

BRIDGE-ELGIN

ST. OVERPASS



- TENTATIVE CONSTRUCTION SEQUENCE
1. Place fill ①
 2. Excavate ②
 3. Drive piles for piers & east abutment
 4. Construct piers & Canal walls.
 5. Place backfill ③
 6. Place fill ④
 7. Drive piles for and construct west abutment
 8. Place backfill ⑤
 9. Place backfill ⑥
- Construct east abutment
- Note: Better procedure

LONGITUDINAL SECTION
(Normal to Abutments and Piers)
Scale: 1" = 30'

This drawing to be
used for soils
investigation
purposes only.

Additional revisions - reviewed
from A.G.S. Dec 12/63

Date	Revisions	BY

OTTAWA QUEENSWAY
BRIDGE No 24
AT ERIE CANAL

DESIGNED BY G.S.S.	DATE 5/9/68	REVISIONARY C45D-P28
CHECKED BY N.W.	DATE 11-20	