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Copy for: Mr. A. V. Toys, Bridge Engr.

ATTN: MR. S. McCOMBIE.

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

H. Q. GOLDER
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2444 BLOOR STREET WEST
TORONTO 9, ONTARIO
767-9281
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PLEASE NOTE IN YOUR COPIES (2)
OF THIS REPORT.

January 4, 1963

A. G. Stermac
Principal Fdn. Engr.

Department of Highways, Ontario,
Materials and Research Section,
Parliament Buildings,
TORONTO 2.

30M14-142
EXHIBIT No.

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


RE: SITE INVESTIGATION
PROPOSED LESLIE STREET INTERCHANGE,
TORONTO, ONTARIO.

Dear Sirs:

There is a typographical error in our report to you on the above project (Report 6205, dated October, 1962). Starting at line 6 on page eleven the text should read "The results of these tests are given on Figures 26 to 35 as plots of void ratio, e_0 , and coefficient of consolidation, c_v , versus log pressure. The laboratory compression index, C_c , ranged from about 0.05 to 0.27. An average value of"

Yours faithfully,

H. Q. GOLDER & ASSOCIATES LTD.



N. R. McCammon, P. Eng.

McC/jb
6205

DEC 11 1962

Mr. A. M. Toye,
Bridge Engineer,
Bridge Division.

Attention: Mr. S. McCombie.

Mr. A. G. Stermac,
Principal Foundation Engr.,
Foundation Section,
Materials & Research Division.

November 26, 1962.

FOUNDATION INVESTIGATION REPORT BY -
H. Q. Golder & Associates, Limited,
Proposed Alterations - Leslie Street
and Highway 401 Interchange, Dist. #6.

Attached, we are forwarding to you the
above-mentioned report submitted by the consulting firm of
H. Q. Golder and Associates, Ltd.

We have briefly reviewed the report and
believe that the information and recommendations contained
within, will be sufficient and adequate for your further
design work. Should there, however, be any problems that you
would like to discuss, or additional information, please feel
free to contact our Office.

AGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2) ✓
H. A. Tregaskes
H. D. McMillan (3)
G. K. Hunter (2)
C. Fraser
T. J. Kovich
J. Roy
J. E. Gruspier
E. R. Saint
F. Norman
A. Watt
Foundations Office
Gen. Files.

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

To be plotted on 30M74 Map

From CNR TO DON RIV.

BA 1542

H. Q. GOLDER & ASSOCIATES LTD.

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REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

<i>20M14-142</i>
CLERKS NO.

ON

SOIL CONDITIONS AND ENGINEERING STUDY

PROPOSED ALTERATIONS

LESLIE STREET AND HIGHWAY 401 INTERCHANGE

TORONTO

ONTARIO

WP. 150-61

Distribution:

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

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

October, 1962

6205

ABSTRACT

The results of an investigation of the soil conditions at the site of the proposed alterations to the Leslie Street and Highway 401 Interchange are presented and detailed recommendations are made for the design and construction of foundations and embankments.

The site is underlain by a compact surface sand followed by a variable thickness of soft silty clay, then a dense silty sand and till overlying bedrock.

Because of the low shearing strength of the soft clay at the site it will be necessary for embankments over 25 feet in height to be constructed using berms. The proposed bridge structures should be founded on piles and bearing in the dense silty sand or till and provision should be made to minimize the effects of differential settlement between the embankments and adjacent structures.

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INTRODUCTION

H. Q. Golder & Associates Ltd. were retained by the Department of Highways, Ontario, to carry out a soil investigation for the proposed new Leslie Street Interchange on Highway 401. This was authorized in a letter dated February 6, 1962. The purpose of the investigation was to examine the soil conditions at the site in relation to the widening of the existing embankments and bridges and the relocation of the access roads.

PROCEDURE

The field work was carried out in two phases. An initial investigation consisting of 10 borings took place between April 24, 1962 and June 1, 1962. A further 2 borings were put down between July 26, 1962 and August 1, 1962. In the early stages of the investigation two standard machine drillrigs were used, but from May 2, 1962 the investigation proceeded with only one. Each boring was accompanied by a dynamic penetration test. The borings were advanced using NX and BX casing to depths of as much as 91 feet with samples being taken at approximately five foot intervals.

A plan of the existing interchange and the proposed changes is shown on Figure 1. The locations of all borings put down during the investigation are shown on this figure, together with the locations of borings put down in previous investigations by The Foundation Company of Canada Limited (reports dated 1953) and Geocon Limited (report dated 1960).

All soil samples obtained during the investigation were returned to our laboratory for examination and testing. The results of the laboratory testing are plotted on the Records of Boreholes and on the figures.

All elevations used in this report are referred to Geodetic datum. They were obtained with reference to B.M. No.T-108 located on a concrete culvert under Highway 401, 1,300 feet easterly from Bayview Avenue. The Geodetic elevation of this benchmark is given as 532.143.

The results of previous investigations carried out for the original construction of the interchange have been used to supplement the data obtained from our field work. This information was used to plot the inferred soil stratigraphy at the site which is shown in Figures 3, 4, 5, 6 and 7.

DESCRIPTION OF THE SITE

The site of the proposed new interchange extends along Highway 401 from Station 125+00 to 150+00. The existing interchange consists of an embankment leading up to a six span bridge over the C.N.R. line and Leslie Street; another embankment extends from this structure to a bridge over the Don River. The original interchange was constructed in 1953 as part of the Toronto Bypass, Highway 401. A failure of the northern slope of the western approach embankment to the C.N.R. and Leslie Street Overpass, occurred a day after the maximum height of about 32 feet had been attained. The slopes were subsequently stabilized by the addition of berms 50 feet wide.

Similar berms were added to all other slopes over 25 feet in height constructed at the site.

The original ground surface sloped down to the Don River from an easterly and westerly direction. The construction of the original interchange has changed the topography so as to form the access roads to and from Highway 401.

SOIL CONDITIONS AND GEOLOGY

The inferred soil stratigraphy across the sections marked on the general site plan Figure 1, is shown in Figures 3, 4, 5, 6 and 7. Essentially, the soil conditions along the centreline of Highway 401 from Station 125+00 and 150+00 consist of very dense fill which forms the existing embankments. This is underlain by a loose to compact deposit of fine sand and a soft compressible silty clay. Below these deposits there are variable dense sand, silt and till deposits.

In preglacial times the Don River flowed in a valley which was located in almost the same position as the existing valley. The area has been covered with varying deposits of till during glaciation. During the advance of the Wisconsin glacier, minor fluctuations of the ice front resulted in a complex assortment of silts and sands being laid down over the till. These silts and sands are predominantly fluvial and glacial outwash with some layers of till included, and have been compacted by subsequent minor advances of the ice.

The shore of glacial Lake Iroquois is known to have risen as high as elevation 433 in the Don River valley (Watt, 1955). During this period of high water level a faintly stratified lacustrine soft silty clay stratum was probably laid down. The upper elevation of this deposit is inferred to be about 433. The soft clay stratum contains a large number of gravel sized particles probably deposited at the same time as the clay. Above this deposit there is a stratified silty clay of different geological age containing fewer gravel size particles and probably formed in an ice-dammed lake whose shore was at least at elevation 465 or higher. The degree of stratification of the clay deposits is much more pronounced in the upper clay stratum above elevation 433. Stratifications consist of irregular alternating layers of silty clay and clayey silt varying from 1/4 inch to 3/4 inch in thickness. The irregular pattern of stratification is further illustrated by the range in liquid limit from about 14 to 56 with a corresponding range in plasticity index from 5 to 35. The liquidity index* measured for individual layers also showed an irregular pattern generally from 0.2 to 1.0 in the upper clay above elevation 433 while two values in excess of 1.0 were measured for layers in the lower clay stratum.

*The liquidity index is defined as the difference between the natural moisture content and the plastic limit divided by the difference between the liquid limit and the plastic limit.

STABILITY OF EMBANKMENTS

The shearing strength of soft clay beneath an embankment is largely controlled by the pore pressures set up by the weight of embankment fill. The excess pore pressure, Δ_u , in an element of clay beneath the fill has been expressed as:

$$\Delta_u = B[\Delta \sigma'_3 + A(\Delta \sigma'_1 - \Delta \sigma'_3)] \quad (\text{Skempton, 1954})$$

where A and B are pore pressure parameters measured in triaxial tests. Beneath the embankment Δ_u will be positive, since B is generally 1 for saturated clays and A relatively close to 1 for most normally consolidated clays. The factor of safety, F, given by an effective stress analysis (Bishop, 1955) will be at a minimum at or near the end of construction, after which F will increase as the excess pore pressures dissipate. However, the use of the effective stress method of stability analyses for the end of construction case means that pore pressures must be predicted; this, in turn, involves an assumption about the stress distribution beneath the embankment. A recent example (Lambe, 1951) has given good agreement between measured pore pressure and those predicted from elastic theory (Jurgenson, 1934). Other factors could decrease the value of F, for example, pore pressures resulting from the driving of piles adjacent to or through the embankment. We cannot, at this time make an accurate prediction of pore pressures resulting from pile driving and the magnitude of such pore pressures can only be determined from field measurements.

To avoid assumptions about the distribution of stress beneath the embankment and hence the prediction of pore pressures

resulting from this stress distribution, it can be alternatively assumed that little or no dissipation of pore pressure will take place during construction. Ignoring the factor of pile driving, a total stress or $\phi=0$ stability analysis can be directly applied. (Skempton, Golder 1948).

The fact that an embankment failure took place during construction indicates that the stability of embankments at the site is critical for the end of construction case and that the measured undrained shear strength of the silty clay may be used in a total stress analysis.

The undrained shear strength of the clay has been measured on these separate occasions; in 1953 during the investigation at the site by the Foundation Company of Canada Limited (reports dated September 30th and October 16th, 1953), in 1959 by Geocon Ltd (report dated April 8th, 1960), and during the present investigation. The undrained shear strengths measured during the 1953 and 1962 investigations are summarized on Figures 13 and 14. From Figure 13 it may be seen that the undrained shear strengths obtained in the present (1962) investigation are slightly but not markedly higher than the 1953 investigation. There is considerable scatter in the results but an average value of the undrained shear strength would be about 700 pounds per square foot. There is virtually no difference in the undrained strength of the clay below the existing embankments and away from the influence of these embankments (Figure 13).

A total stress stability analysis was carried out for the section of the embankment that failed in 1953 (Figure 36). The computations indicate that an average undrained shear strength of about 680 pounds per square foot would have to be mobilized around the failure circle for an assumed factor of safety of one at failure. This compares favourably with the measured value on Figure 14 and with an analysis carried out in 1953, (Foundation Company of Canada Limited, September 30th, 1953) where a factor of safety of 0.9 was calculated assuming the undrained shear strength of the clay to be 600 pounds per square foot.

The effective shear strength parameters of the clay were determined by undrained triaxial compression tests with pore pressure measurements and also by a drained triaxial compression test. The results of these tests are plotted on Figure 15 to 25. The measured values of the effective angle of shearing resistance, ϕ' , ranged from about 22 to 29 degrees except for one value of 33 degrees obtained on a more sandy sample of the clay. The cohesion intercept, c' , was approximately zero, as would be expected for a normally consolidated or lightly over-consolidated material. The value of A is dependent on strain, but at maximum deviator stress, $(\sigma'_1 - \sigma'_3)_{\max.}$, ranged between about 0.5 and 1.0 in the undrained pore pressure tests. (It has been assumed that maximum $(\sigma'_1 - \sigma'_3)$ represents the criterion of failure for an undrained shear rupture of the clay following embankment loading.)

An effective stress stability analysis was carried out for the section of embankment which failed in 1953. The details of this analysis are shown on Figure 38. The pore pressure distribution along the failure surface, which is reasonably well defined, was obtained by estimating the major and minor principal stresses (σ_1, σ_3) beneath the embankment on the basis of elastic theory (Jurgenson, 1934) and using an average value of A at failure of 0.8. A value of F equal to unity was obtained assuming an average value of ϕ' of about 26 degrees for the silty clay. This confirms the average strength criteria for the clay determined in the laboratory and checks with the results of the $\phi=0$ analyses. Calculation also showed that at 50 percent consolidation of the silty clay, F would increase to about 1.5 and to almost 2.0 for 100 percent consolidation. These results summarized on Figure 38 define the long term stability of the embankment ignoring the effects of pile driving which is discussed later.

Using the $\phi=0$ method of analysis, the factor of safety for different heights of embankment at varying side slopes has been studied. The results are plotted on Figure 37. It should be noted that an average clay thickness of 36 feet has been used in the analyses. Over a large part of the site the thickness of the clay is much less; consequently the computed values of F are on the conservative side.

The factor of safety normally used in the design of embankments is 1.5, where the factor of safety is dependent upon

measured values of the undrained shear strength of the foundation soil. However, if the design of the embankment is based on the average undrained shear strength calculated from an embankment failure which took place during construction, we may assume that a lower factor of safety can be adopted for design with reasonable confidence. On this assumption the embankments may be designed to an average factor of safety of 1.2.

The most critical section of the embankments is the western approach to the C.N.R. Overpass, Station 124+00 to 132+00. The northern slope of this part of the embankment failed in 1953. York Farms Limited have subsequently built a warehouse to the north of the highway and apparently cut away the toe of the embankment to provide a driveway and a parking lot. For an average undrained shear strength of 680 to 700 pounds per square foot for the clay, the factor of safety of the embankment must now be close to unity.

The present embankment could safely be widened by duplicating the present slope of 2 horizontal to 1 vertical and retaining the berms on all slopes over 25 feet in height above the existing ground level. However, due to the limitations of space the embankment cannot be built within the existing property. Therefore, in order to prevent the northern slope of this embankment butting against the York Farms building the design of this section will have to be modified. It is therefore recommended that the northern berms between stations 124+00 and 128+00 be constructed using dense granular fill and the upper part of the embankment, above the level of the berms, be completed using a light-weight fill with a maximum dry

density of not greater than 90 pounds per cubic foot. The minimum factor of safety of this section of the embankment is computed to be 1.15. The details of this design are shown in Figure 36.

The remainder of the embankment between Stations 135+00 and 145+00, will be divided into two sections by the relocation of Leslie Street. The section from the C.N.R. Overpass to Leslie Street is less than the critical 25 feet in height and should be widened using an identical slope of 2 horizontal to 1 vertical. The section of the embankment from Leslie Street to the Don River has a maximum height of 35 feet. Analyses were carried out to check the stability of the northern slope; the details are presented in Figure 39. It can be seen that with a berm 125 feet wide and using an undrained shear strength of 680 pounds per square foot the minimum factor of safety against failure is about 1.2.

The end of each embankment should be designed in the same manner as the embankments. Where the embankments are over 25 feet in height, end berms should be provided. While the stability at the ends of embankment is generally some 15 percent greater than for the side slopes, due to edge effects, it is recommended that the criterion for F of not less than 1.2 be adopted in order to minimize possible lateral movement of the abutments. Side approach embankments and ramps to the proposed Highway 401 embankment are all less than the critical height of 25 feet and therefore will be adequately stable at a 2 horizontal to 1 vertical slope.

SETTLEMENT OF EMBANKMENTS

Settlement of the proposed embankments and ramps will occur due to consolidation of the clayey strata under the additional weight of the earthfill. Consolidation tests were carried out on samples of the silty clay to determine its settlement characteristics. The results of these tests are given on Figures 26 to 35 as plots of void ratio, e_0 , and coefficient of consolidation, C_v , ranged from about 0.05 to 0.27. An average value of C_c of about 0.2 has been assumed for settlement computations. These computations were carried out for various thicknesses of clayey strata and for various embankment configuration and the results of the computations are summarized on Figures 40, 41 and 42.

In the consolidation tests carried out in the laboratory no lateral strains are permitted and hence the pore pressures set up in a saturated sample are equal to the applied pressures prior to dissipation. In the field lateral strains will generally occur in the compressible stratum and this fact necessitates modification of computed settlements based on the laboratory testing. A semi-empirical method of correction has been proposed by Skempton and Bjerrum (1957) where the computed settlement is multiplied by a factor, μ , (a function of the pore pressure parameter A ,) to give the estimated probable settlement. Laboratory undrained triaxial compression tests with pore pressure measurements gave values of A at maximum deviator stress ranging from about 0.5 to 1.0. From Skempton and Bjerrum (1957) and further discussion by Wood (1959), a μ value of about 0.7 is computed. The probable settlement values

given on Figures 40, 41 and 42 correspond to this correction factor.

The time rate of settlement of the proposed embankments due to consolidation of the clayey strata has been computed from the measured laboratory coefficient of consolidation, C_v , which generally ranged from about 1.4×10^{-2} to 3.0×10^{-2} square inches per minute. It has been assumed that single drainage of the clay will take place. This is conservative and it may be noted that if double drainage does in fact exist the time to reach 50 or 90 percent consolidation of the clay would be about one quarter the time given on Figure 41 for the single drainage condition. From Figure 41, 90 percent consolidation of a 30 feet thick clay stratum should occur in about 10 years. Two piezometers installed in the clay stratum below the berm at the site of the 1953 embankment failure, and recorded on the Record of Borehole 112, indicate that the excess pore pressures in this area have dissipated at this time, that is 9 to 10 years after construction. Thus the dotted curves on Figure 41 represent the probable maximum time required for consolidation.

BRIDGE STRUCTURES

The bridges which carry Highway 401 over the C.N.R. line and over the Don River will have to be widened while a completely new bridge will have to be constructed where it will cross the new location of Leslie Street. Separate bridges will be required for the access roads at the C.N.R. crossing. The widened bridges will have to accommodate an extra three lanes of traffic in each direction while the new bridge over Leslie Street will carry the full width of the Highway.

Bridge over the C.N.R. Line: The piers of the present bridge are founded on steel 'H' piles, 65 to 70 feet long which penetrate into the dense sand and till while the abutments rest on timber piles 30 feet long which penetrate into the compact surface sand. The abutments and the piers are continuous across the complete width of the bridge but the deck has a construction joint along the centreline of the bridge. Two pile load tests were carried out on the steel 'H' piles during the original construction (Report of Department of Highways, Ontario Project F-54-22. "Pile Load Tests of Leslie Street and 401 Highway", dated April 22, 1955) of the bridge and it was found that the maximum net and gross settlements of the two piles tested were 0.17 inches and 0.35 inches, respectively, the maximum test load being 100 tons (Report of D.H.O. Project F-54-22, April 22, 1955. No pile driving records for these piles are given in the report but it is stated that the piles had been driven to penetration resistances equivalent to design loads of approximately 40 tons using the Boston Pile driving formula and 80 tons using Engineering News Record formula).

The piers and the abutments of the bridges which will carry the extra lanes over the C.N.R. line should be founded on 12 inch, 53 pound steel 'H' piles which are driven into the dense underlying sands and till. The length of the piles should be such that their tips are not higher than elevation 390 and should be driven to a final resistance of at least 10 blows per inch using a hammer delivering about 18,000 foot-pound of energy. This final resistance should be obtained for the last two feet of driving.

Under these conditions the safe design load of the piles may be 60 tons per pile.

Since both the abutments and the piers of the extra spans will be founded on steel 'H' piles driven to practical refusal in the dense strata underlying the silty clay the new structure should be free from any noticeable settlement. However the approach embankments will settle under their own weight and there will be some differential settlement between the abutment and structure. To minimize this differential settlement which, due to end effects, we would estimate to be about half the settlement values given on Figures 40 and 41 the embankment should be constructed as far in advance of the structure as practicable. The new structure should be completely independent of the existing structure because there may be still some settlement taking place under the abutments of the original bridge which were founded on short timber piles in the sand overlying the clay.

The bridges for the access roads should be founded on piles similar to the main structure and using the same design criteria.

Bridge over Leslie Street: Leslie Street is to be relocated to the east of its present position. Part of the existing embankment will have to be cut away and a new bridge constructed to carry the complete width of the highway across Leslie Street. During the construction of the bridge the highway will have to be kept operational at its present capacity. This necessitates that the sections of the bridge which carry the additional lanes must be constructed first.

The embankment will therefore have to be widened to its full width in the early stages of the job. This is recommended so that as much settlement as possible will have taken place before the final surface is put on the pavement.

The piers and the abutments of the new bridge should be founded on 12 inch, 53 pound steel 'H' piles driven to refusal in the dense underlying sand and till with their tips not higher than elevation 380. The safe design capacity of the piles may be 60 tons providing they are driven to a resistance of 10 blows per inch using a hammer delivering at least 18,000 foot-pound of energy. This final resistance should be attained for the last two feet of driving.

Bridge over Don River: Highway 401 at the moment crosses the Don River on two separate bridges. The abutments are founded on steel 'H' piles, 80 to 90 feet long, driven into the dense sand and till but the piers are founded on shorter 22 inch diameter Franki piles whose tips are in a dense sand immediately below the soft clay at an elevation of about 380. Two of the Franki piles were load tested to 200 tons during the construction and showed a maximum gross settlement of 0.461 inches and a maximum net settlement of 0.273 inches. (Recent Test on Franki Pile for Don River - 401 Bridge, Engineering and Contract Record, August 1955).

The foundations of the addition to this bridge should, where possible, be similar to the original design. The abutments may be founded on 14 inch 73 pound steel 'H' piles driven into the dense sand and till to a final resistance of 10 blows per inch using a

hammer delivering at least 18,000 foot-pound of energy. This final resistance should be obtained for the last two feet of driving. The safe design load of the pile may then be 75 tons. The piers may be founded on 22 inch diameter Franki piles whose tips bear, at approximately elevation 380, in the upper dense sand. The safe design load for these piles should be 100 tons.

As with other portions of the site the embankment should be widened early in the construction programme so that as much settlement as possible of the embankments will have taken place before the construction of the bridge structure. This measure should limit differential settlements between embankment and structure.

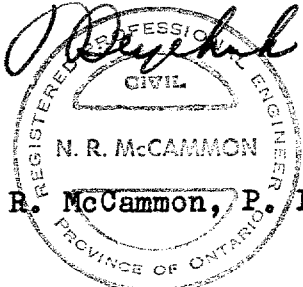
PILE DRIVING

As discussed previously under "Stability of Embankments", the stability of the embankments at any given time is dependent upon the pore pressures in the underlying clay. Since the embankments will have to be constructed first, the pore pressures in the soft clay will have increased to some extent prior to the driving of the piles for the bridge abutments. The driving of the 'H' piles will further increase the pore pressures in the clayey strata in the vicinity of the piles (Bjerrum and Johannessen, 1960). We recommend that piezometers be installed at the abutment locations prior to pile driving to monitor the excess pore pressures set up by pile driving.

STAGE CONSTRUCTION

Figure 13 shows that the increase in the undrained shear strength of the clayey strata below the existing interchange embankments due to consolidation of these strata from 1953 to date is very minor. Thus, from practical evidence the use of stage construction cannot be recommended as a means of increasing the shear resistance of the clay to enable it to sustain a greater height of fill without berms than the 25 feet height already recommended in this report.

McC/jb
6205

for

N. R. McCammon, P. Eng.

October, 1962

for
H. G. Soderman
V. Milligan, P. Eng.

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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
γ_b - Submerged Unit Weight	($\frac{1}{2}$ Compressive Strength)
L _L - Liquid Limit	St - Sensitivity
P _L - Plastic Limit	ϕ' - Effective Angle of Shearing Resistance
W - Natural Water Content	c' - Effective Cohesion Intercept
G - Specific Gravity	Cc - Compression Index
e - Void Ratio	Cv - Coefficient of Consolidation

RECORD OF BOREHOLE 101

LOCATION SEE FIGURE 1

BORING DATE APRIL 24, 25, 26, 1962

DATUM GEODETIC

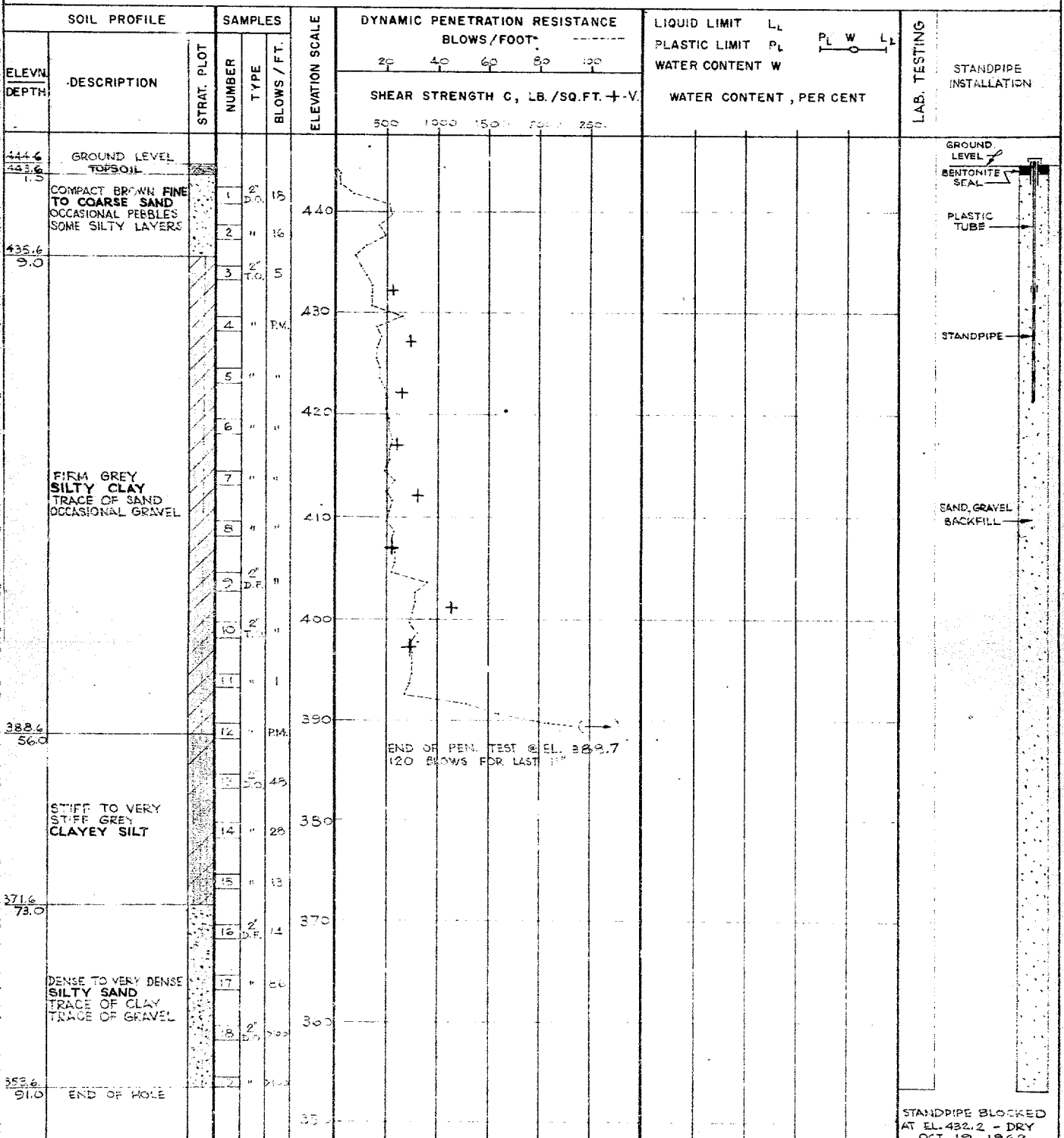
BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10' 0"

GOLDER & ASSOCIATES

DRAWN P.C.

CHECKED *[Signature]*

RECORD OF BOREHOLE 102

LOCATION SEE FIGURE 1

BORING DATE APRIL 26, 27, 30, 1962

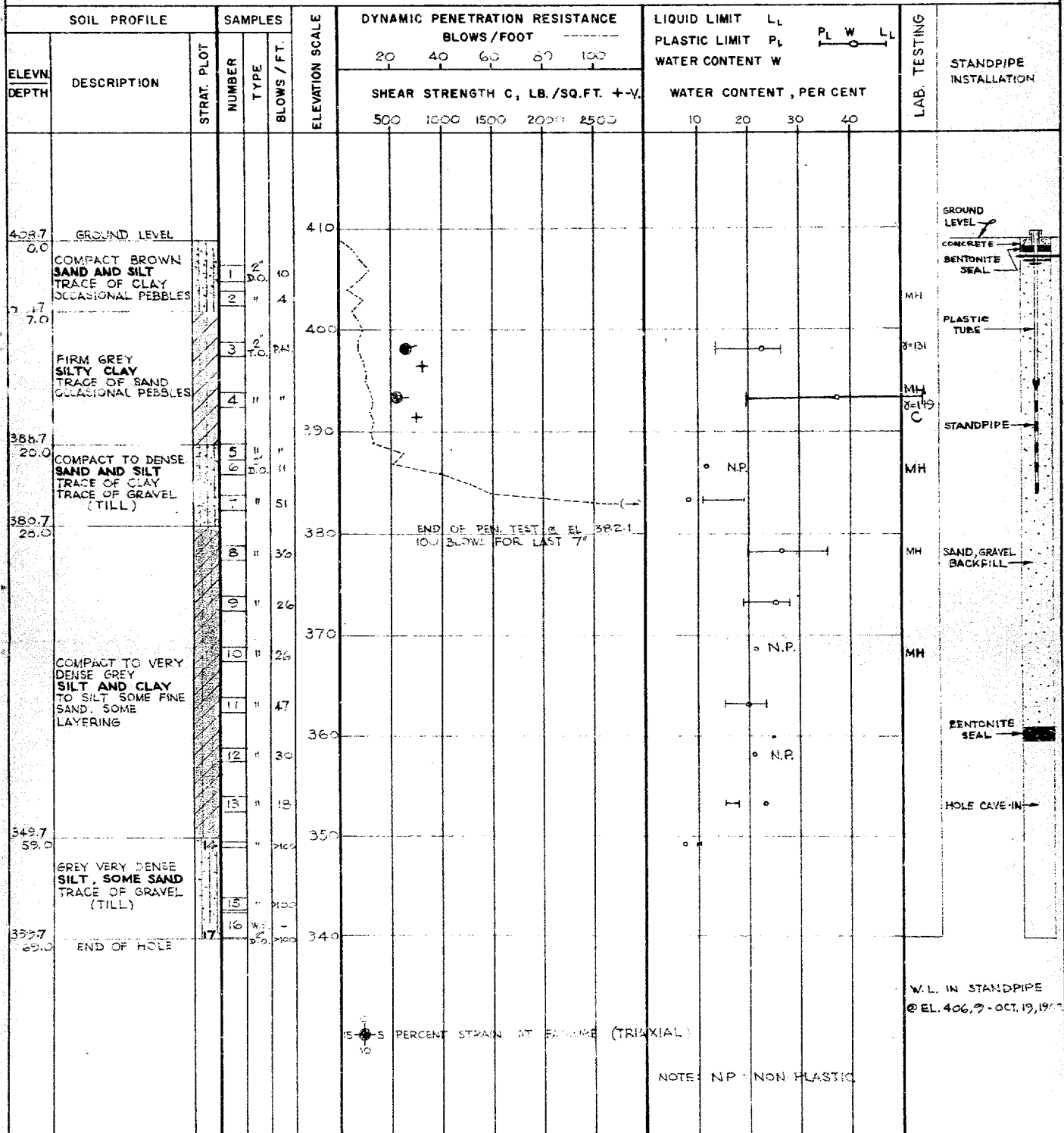
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN P.C.

CHECKED

RECORD OF BOREHOLE 103

LOCATION SEE FIGURE 1

BORING DATE APRIL 27, 30, MAY 1, 2, 1962

DATUM GEODETIC

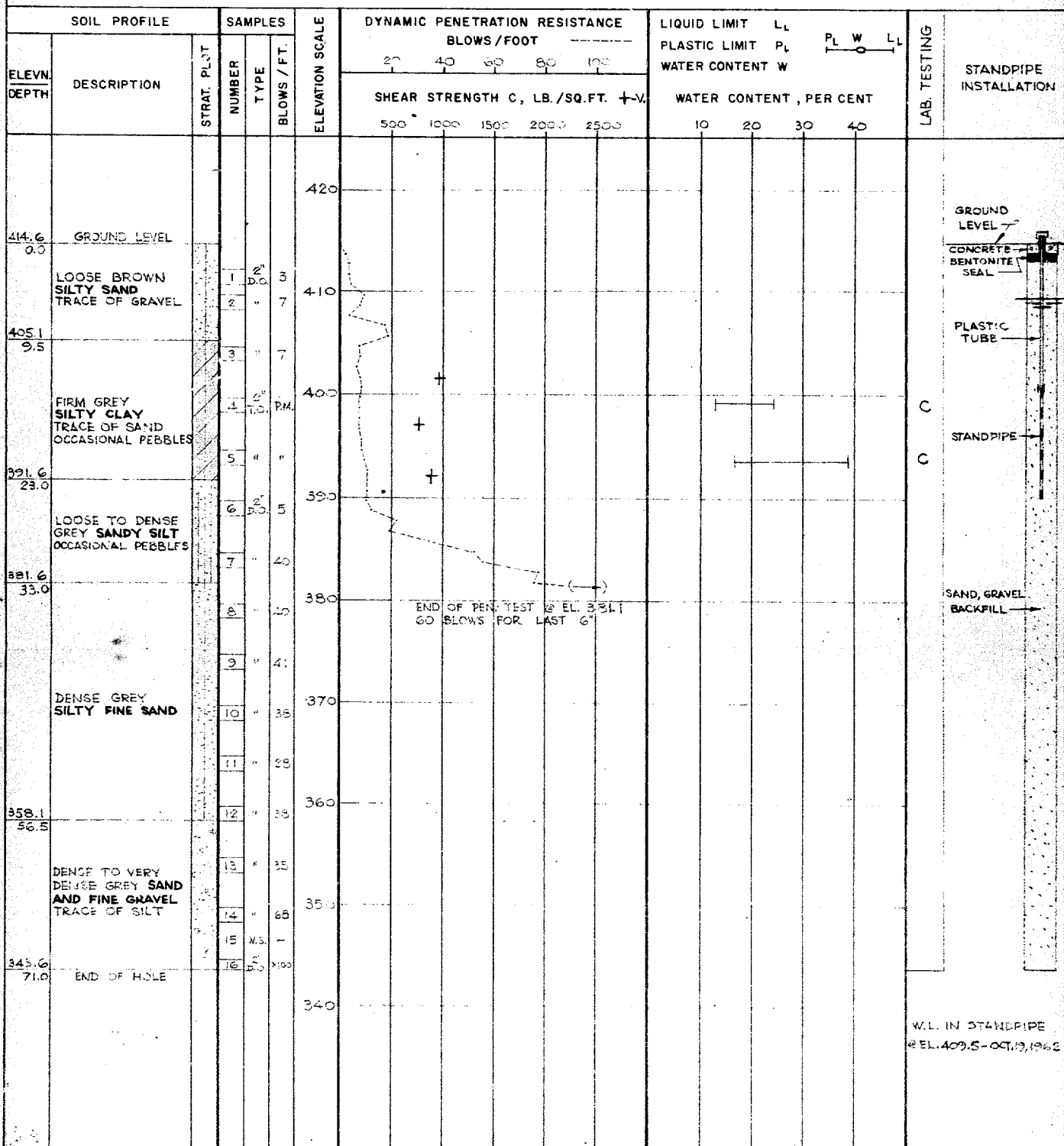
BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER NX & BX CASING


SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN P. C.
CHECKED 

RECORD OF BOREHOLE 104

LOCATION SEE FIGURE 1

BORING DATE APRIL 30, MAY 1, 2, 3, 1962 DATUM GEODETIC

BOREHOLE TYPE

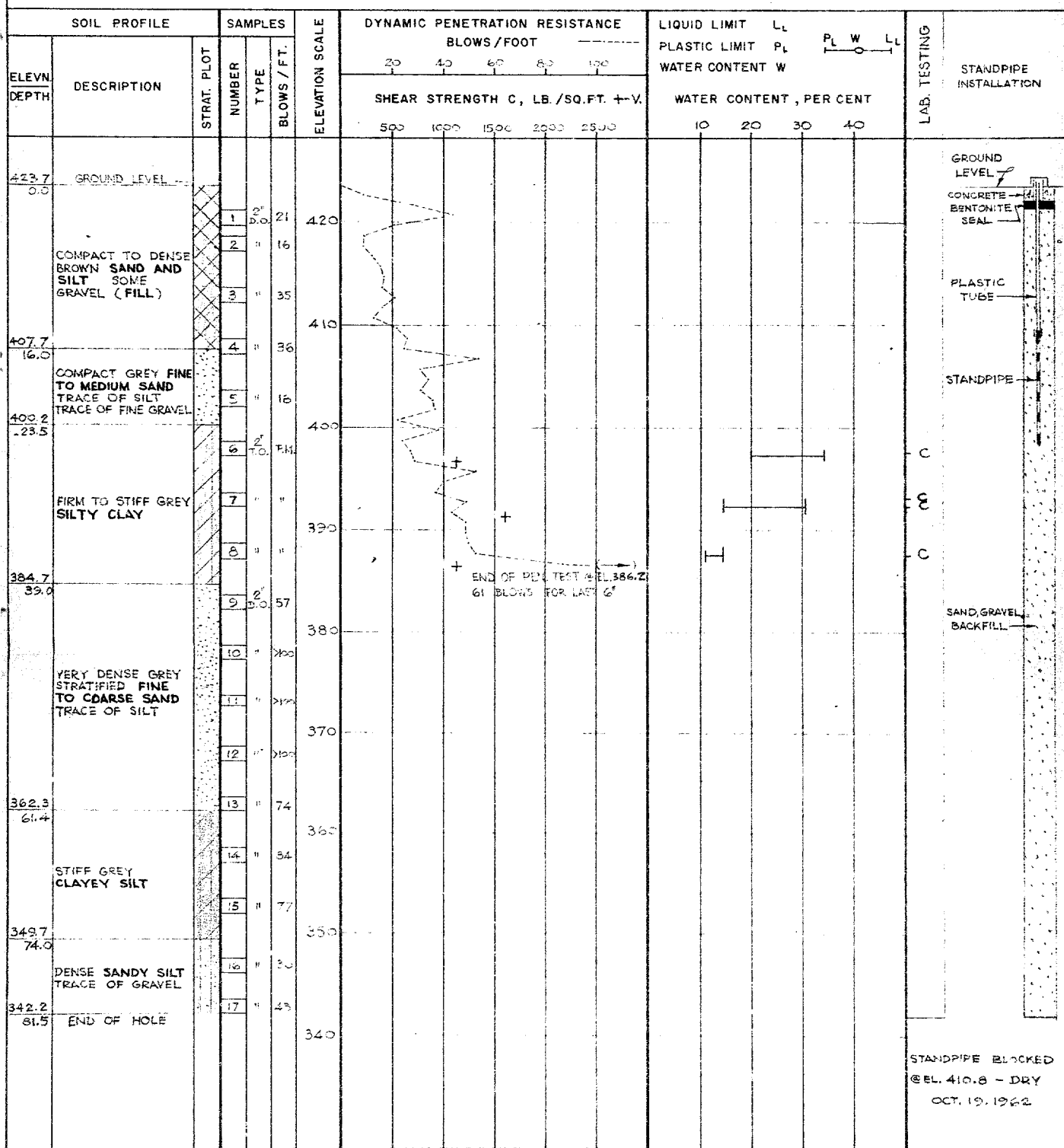
WASH BORING

BOREHOLE DIAMETER

NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN P. C.
CHECKED *[Signature]*

STANDPIPE BLOCKED
@ EL. 410.8 - DRY
OCT. 19, 1962

RECORD OF BOREHOLE 105

LOCATION SEE FIGURE 1

BORING DATE MAY 29, 30, 31, JUNE 1, 1962 DATUM GEODETIC

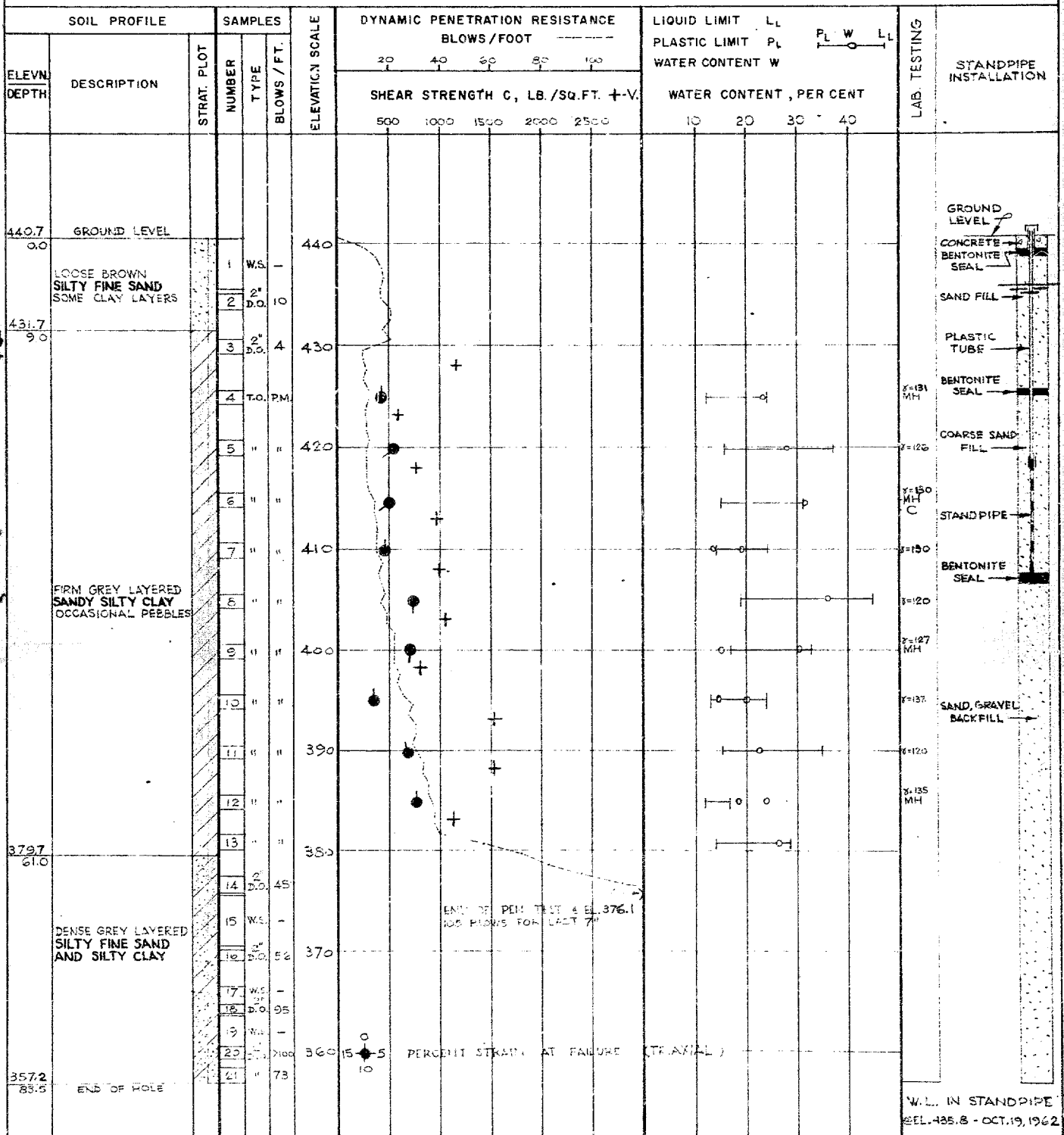
BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN P.C.

CHECKED *[Signature]*

RECORD OF BOREHOLE 106

LOCATION SEE FIGURE 1

BORING DATE MAY 4, 7, 8, 9, 1962 DATUM GEODETIC

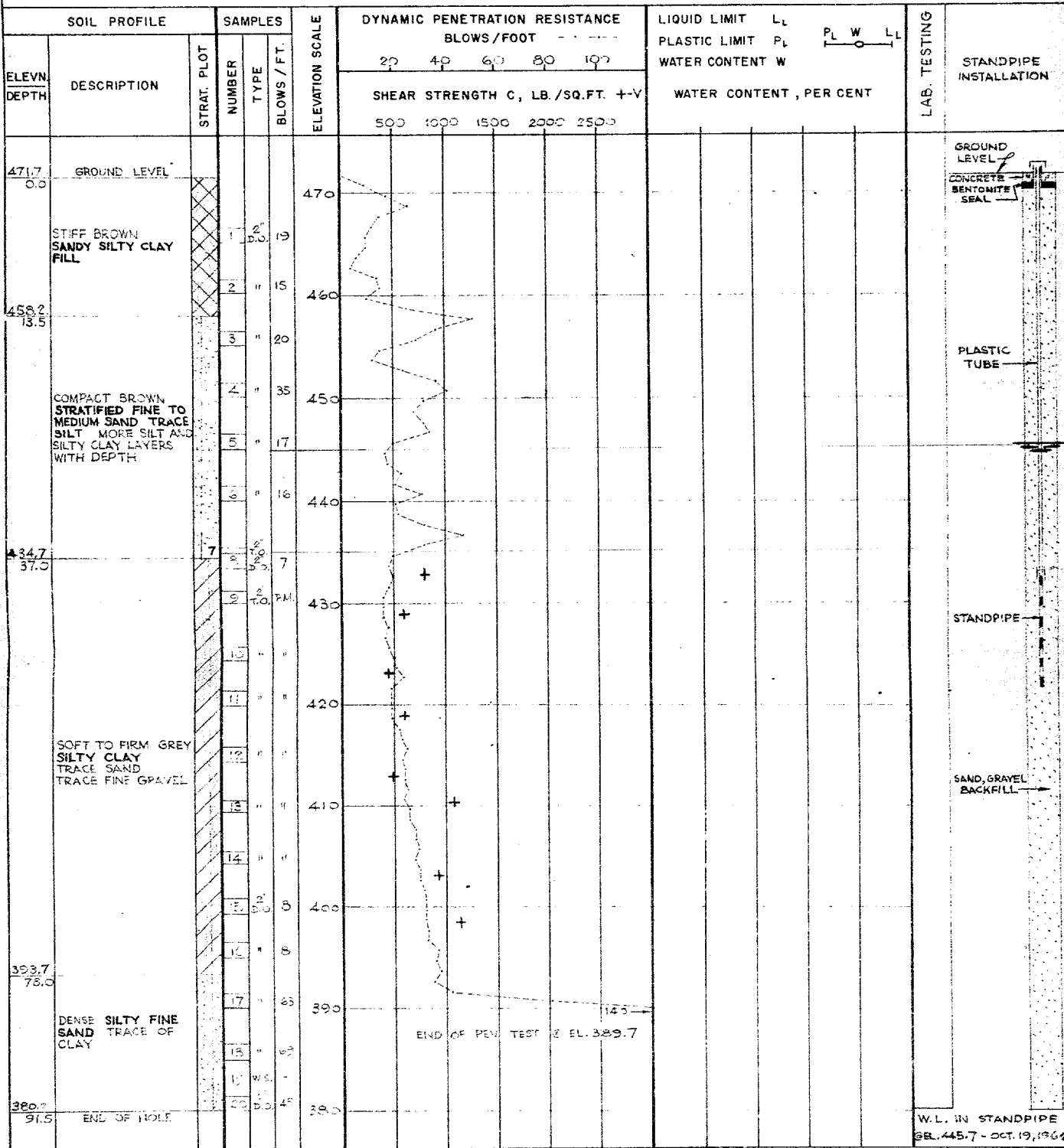
BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10' 0"

GOLDER & ASSOCIATES

DRAWN P.C.

CHECKED

RECORD OF BOREHOLE 107

LOCATION SEE FIGURE 1

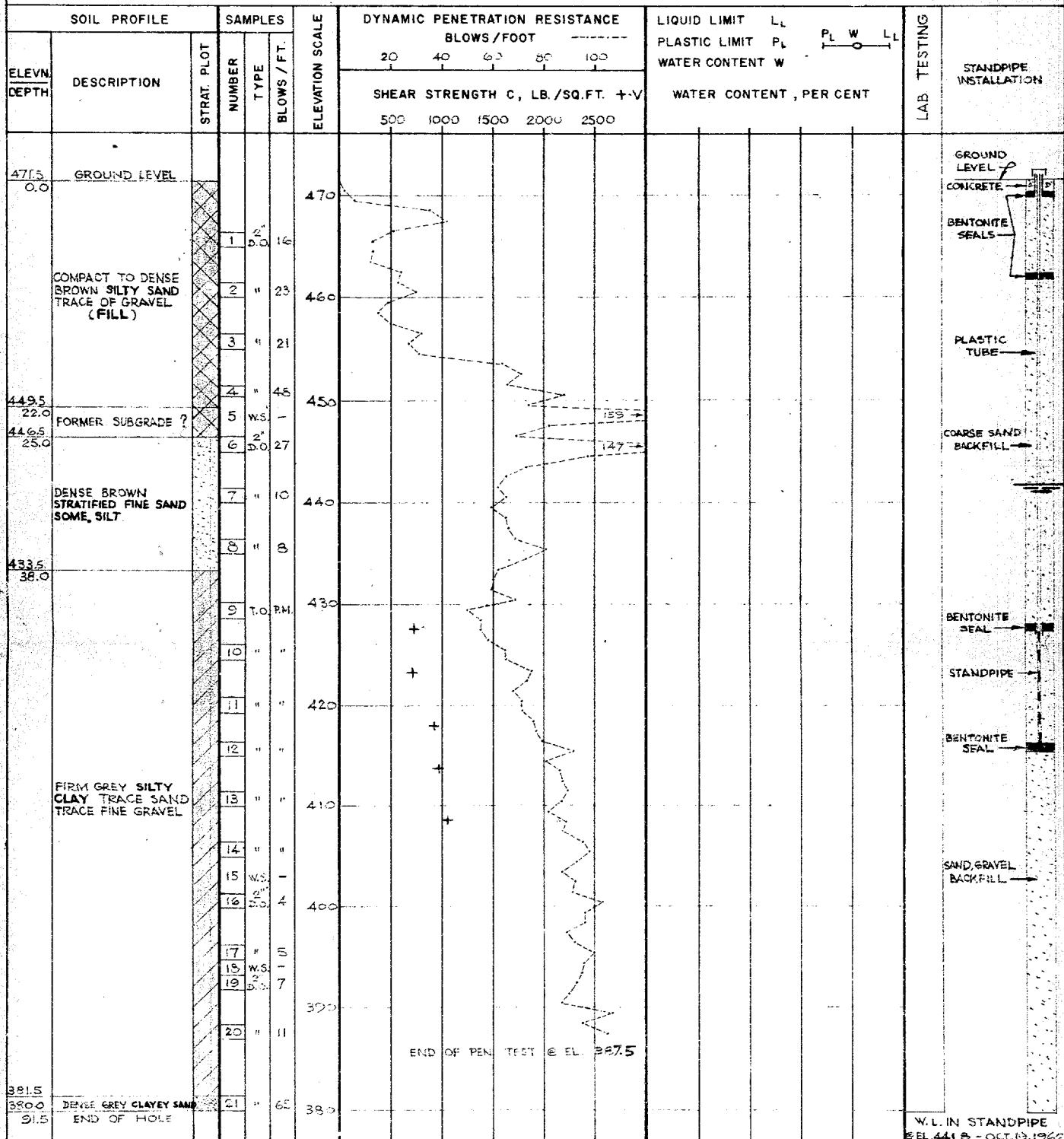
BORING DATE MAY 9, 10, 11, 14, 15, 1962 DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN P. C.

CHECKED *[Signature]*

RECORD OF BOREHOLE 108

LOCATION SEE FIGURE 1

BORING DATE MAY 16, 17, 18, 1962 DATUM GEODETIC

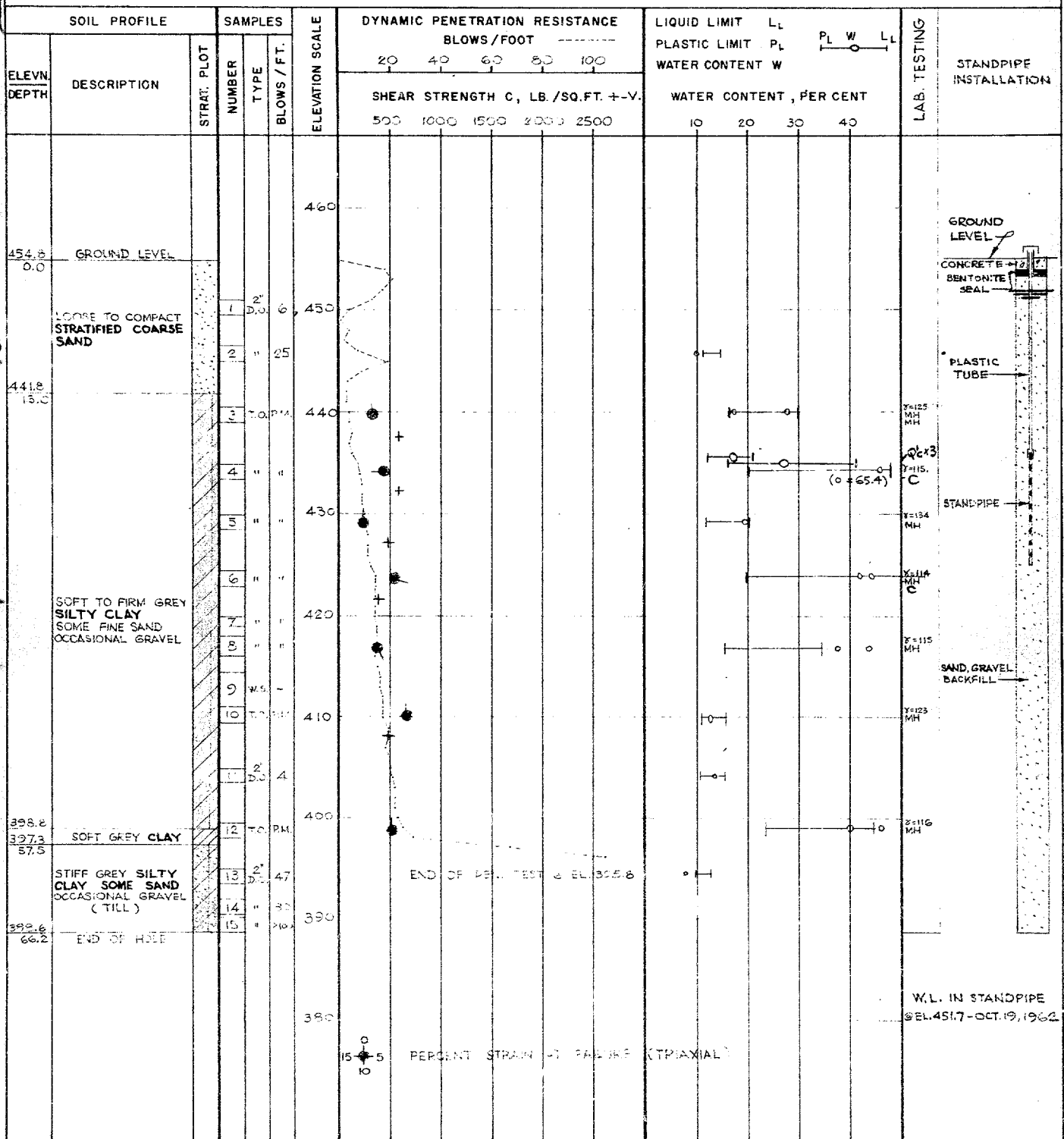
BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10' - 0"

GOLDER & ASSOCIATES

DRAWN P.C.

CHECKED *[Signature]*

RECORD OF BOREHOLE 109

LOCATION SEE FIGURE 1

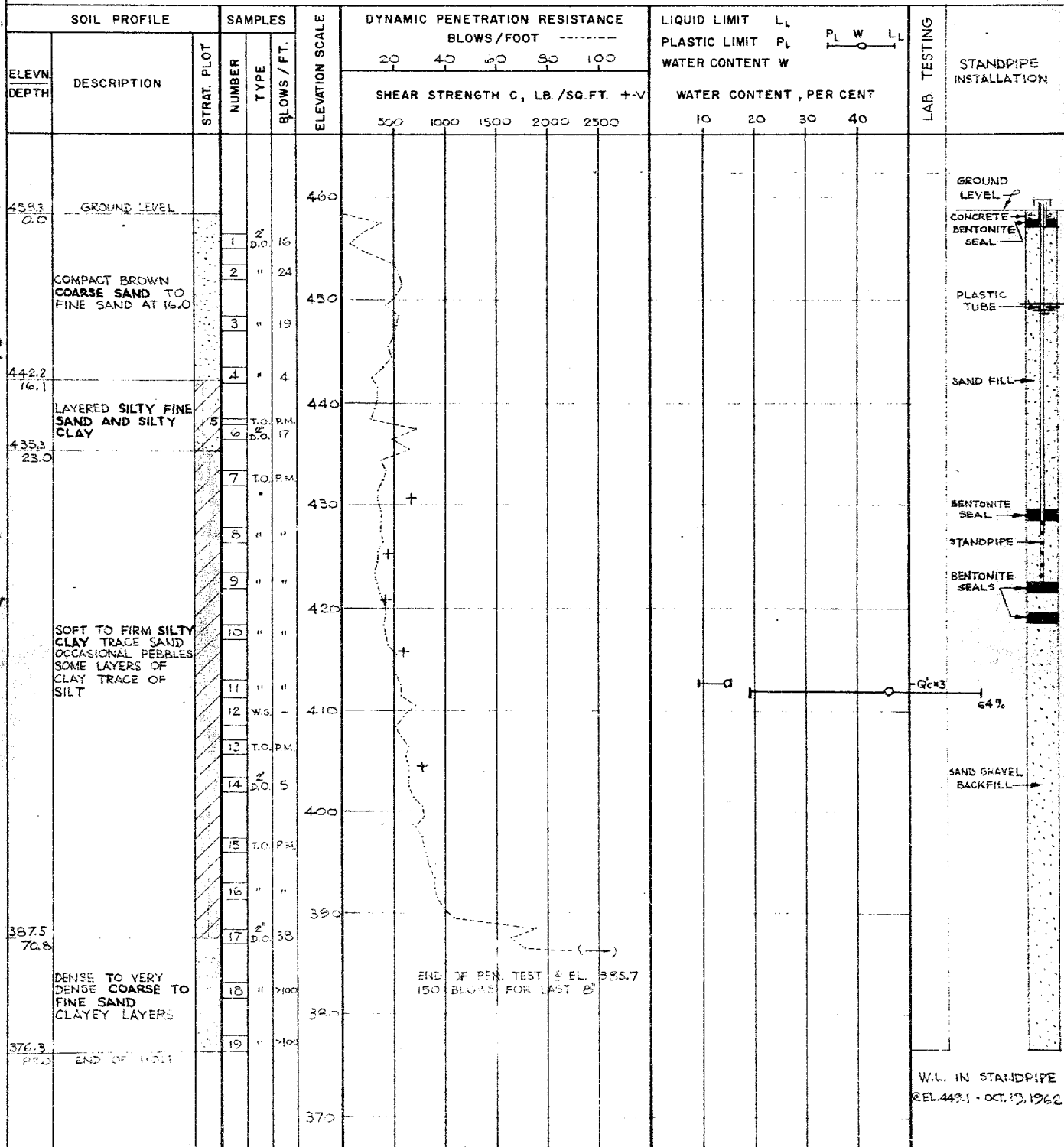
BORING DATE MAY 18, 22, 23, 1962 DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10'

GOLDER & ASSOCIATES

DRAWN P.C.

CHECKED P.C.

RECORD OF BOREHOLE 110

LOCATION SEE FIGURE 1

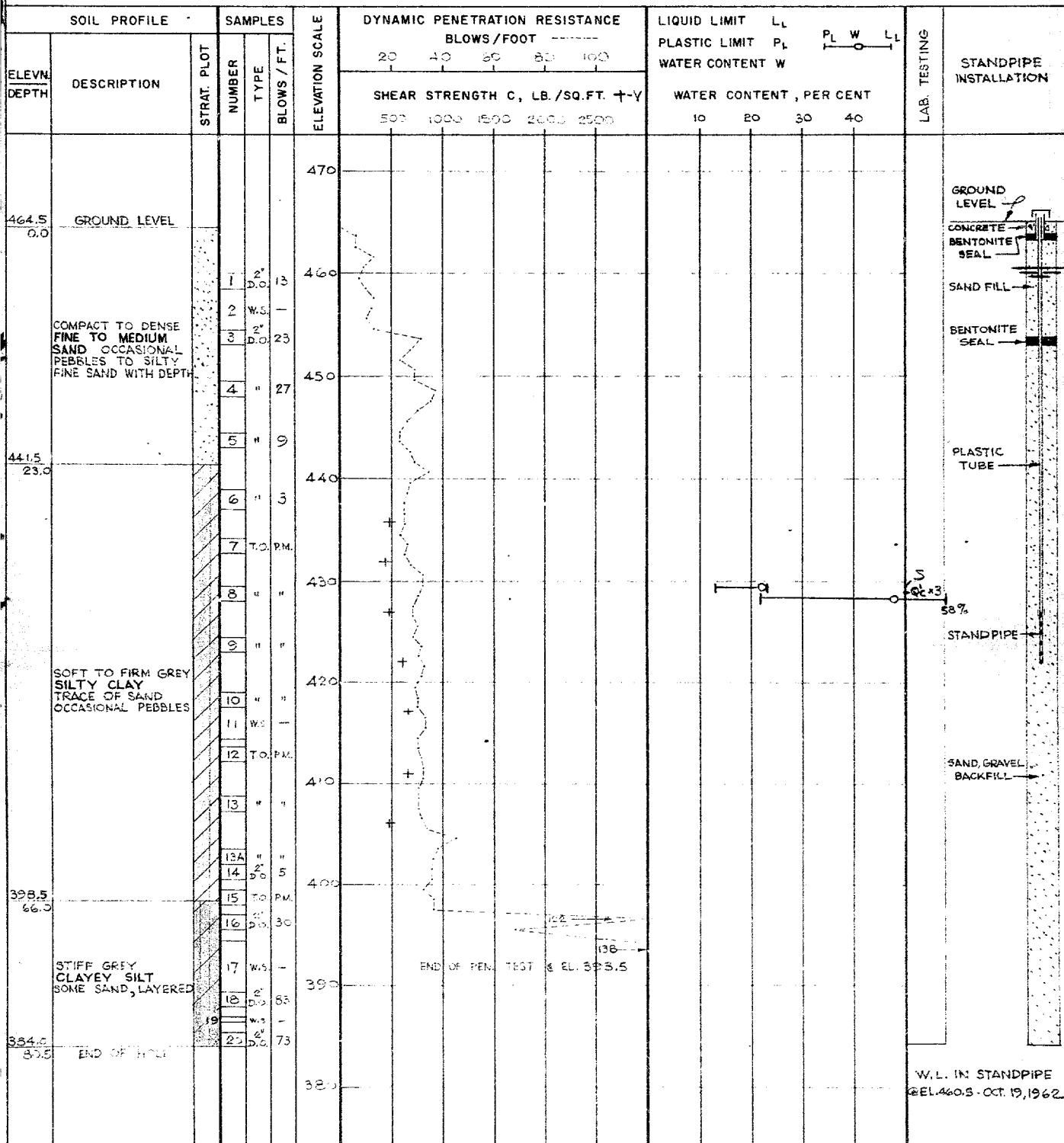
BORING DATE MAY 24, 25, 28, 1962 DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX & BX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES



VERTICAL SCALE

1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN P.C.

CHECKED E.D.

RECORD OF BOREHOLE III

LOCATION SEE FIGURE 1

BORING DATE JULY 26, 27, 1962

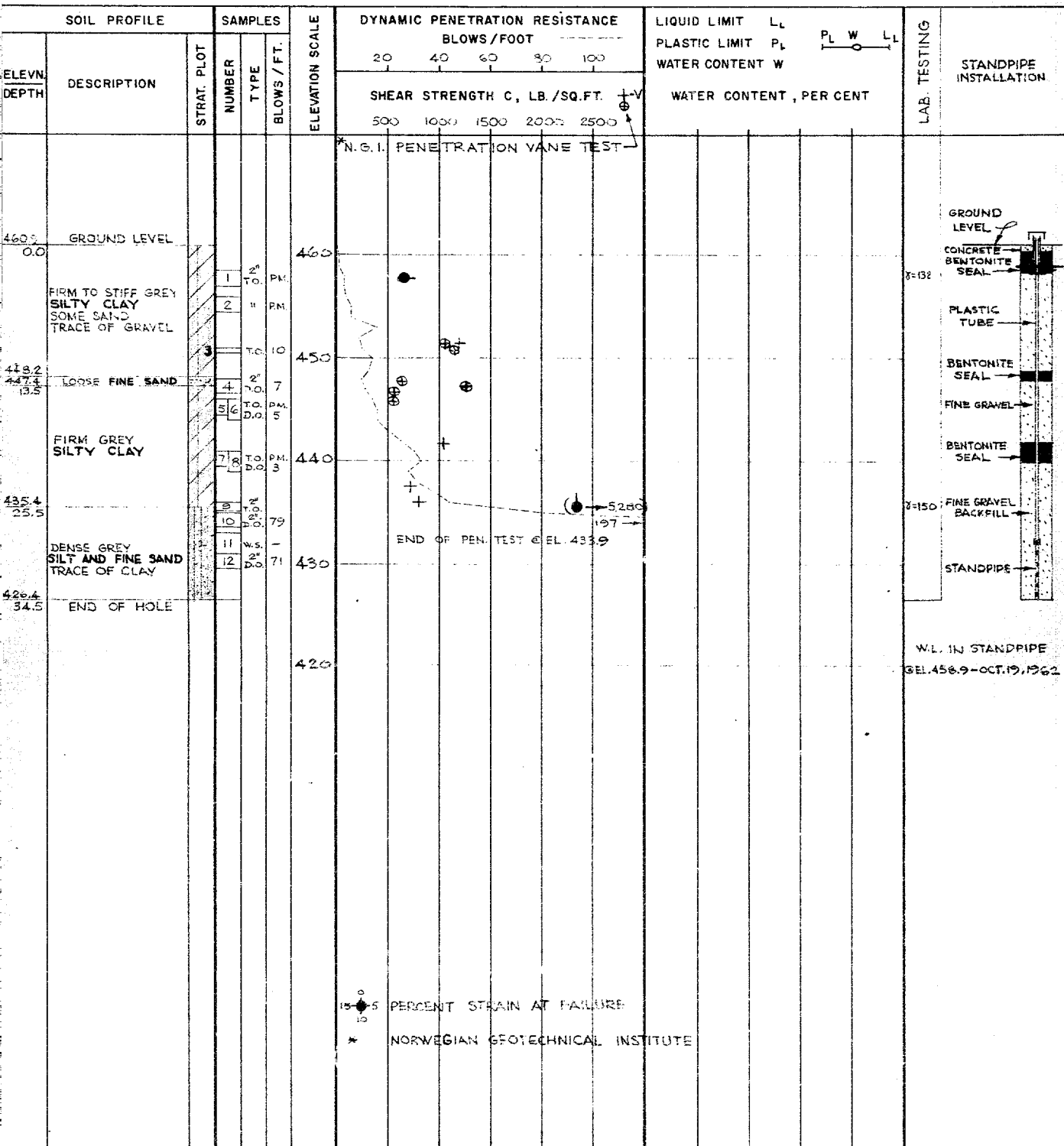
DATUM GEODETIC

BOREHOLE TYPE WASH BORING

BOREHOLE DIAMETER NX CASING

SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN P. C.
CHECKED

RECORD OF BOREHOLE 112

LOCATION SEE FIGURE 1

BORING DATE JULY 30, 31, AUG. 1, 1962

DATUM GEODETIC

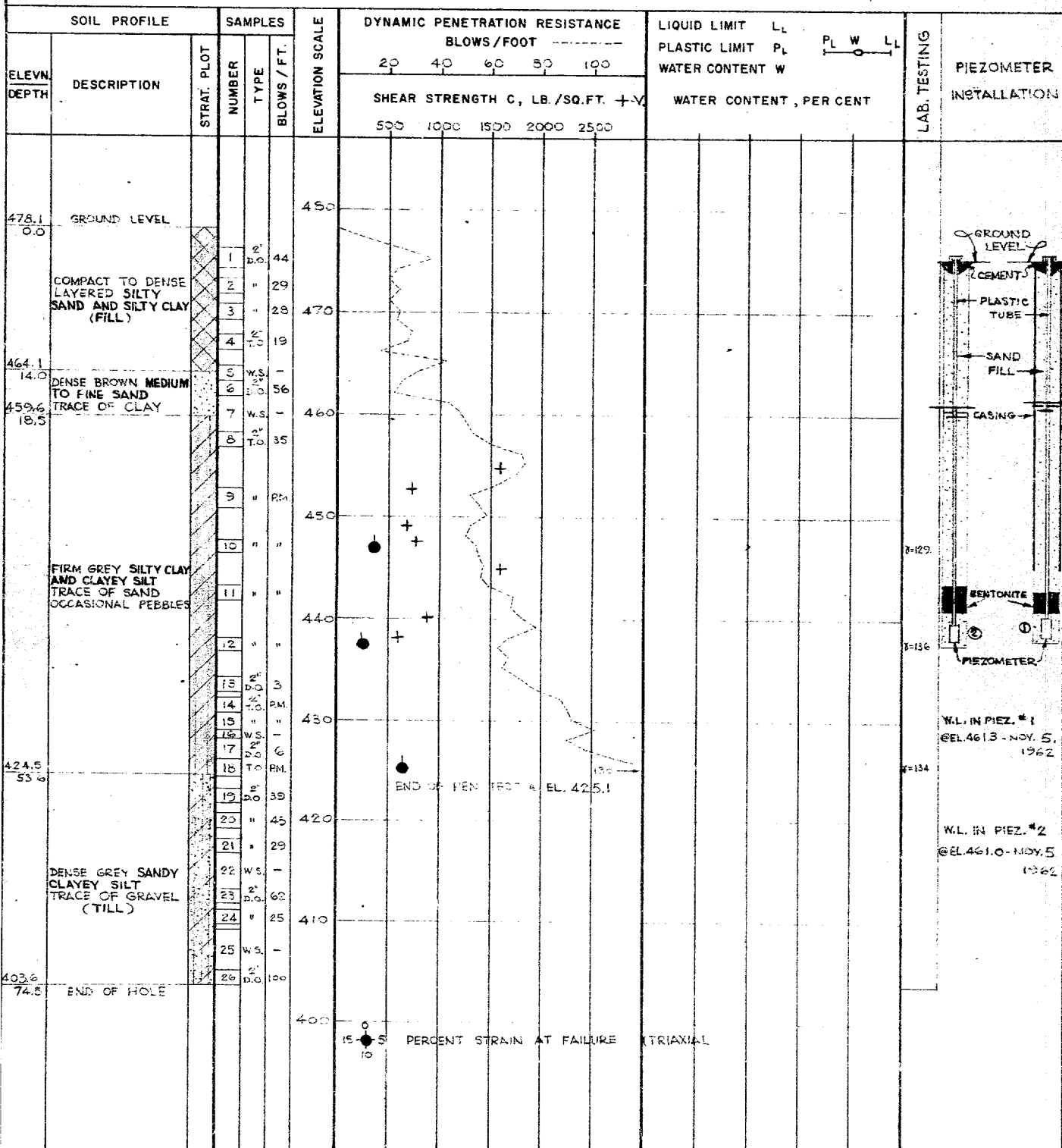
BOREHOLE TYPE

WASH BORING

BOREHOLE DIAMETER NX & BX CASING

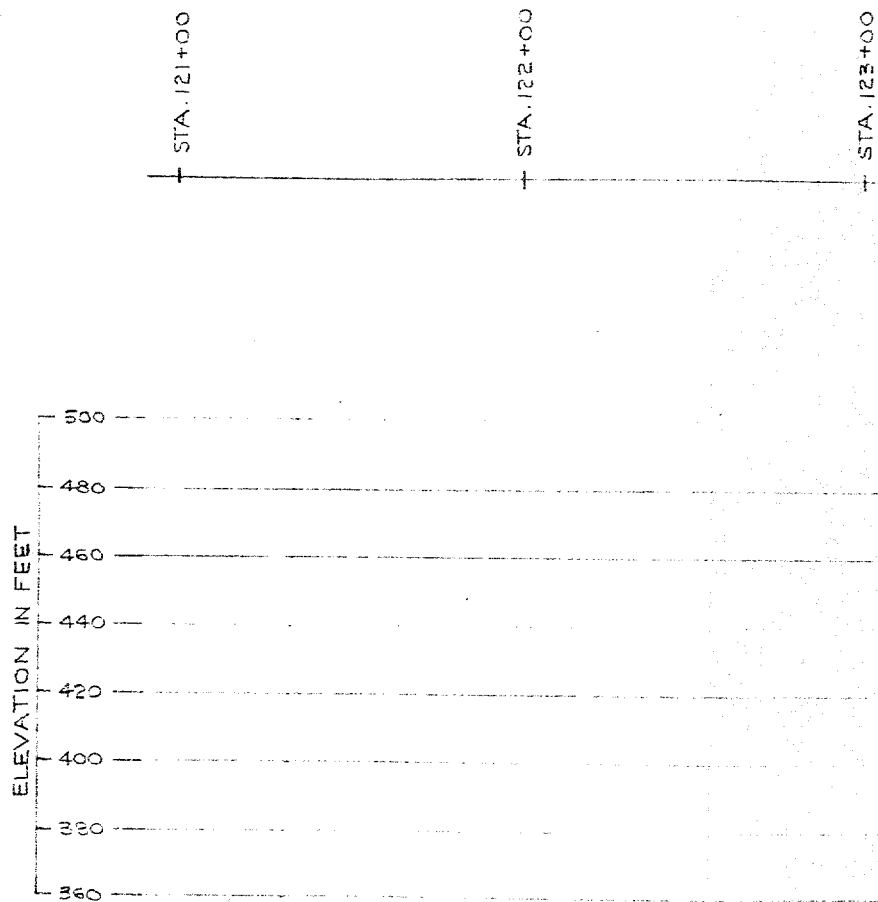
SAMPLER HAMMER WEIGHT 140 LB. DROP 30 INCHES

PEN. TEST HAMMER WEIGHT 140 LB. DROP 30 INCHES

VERTICAL SCALE
1 INCH TO 10'-0"

GOLDER & ASSOCIATES

DRAWN P.C.
CHECKED 50



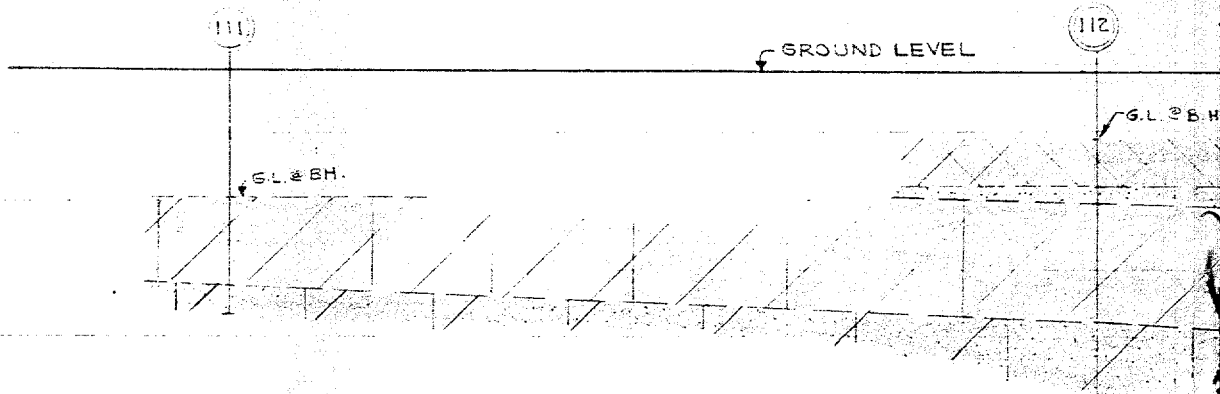
— STA. 124+00

— STA. 125+00

— STA. 126+00

— STA. 127+00

CHAINAGE ALONG CEN



— STA. 127+00

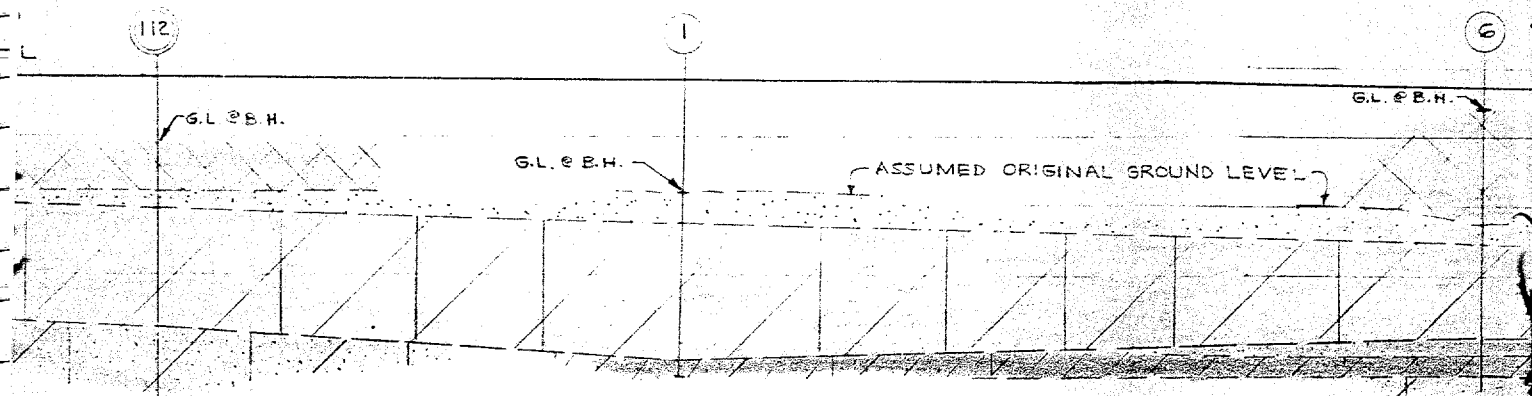
— STA. 128+00

— STA. 129+00

— STA. 130+00

— STA. 131+00

AGE ALONG CENTRELINE HIGHWAY 401



LONGITUDINAL SECTION ALONG CENTRELINE HIGHWAY 401
STATION 121+00 TO 136+00

STA. 131+00

STA. 132+00

STA. 133+00

STA. 134+00

6

8

12

GL. & B.H.

C.N.R. TRACKS

LEG

112

BOREHOLE IN ELEVATION

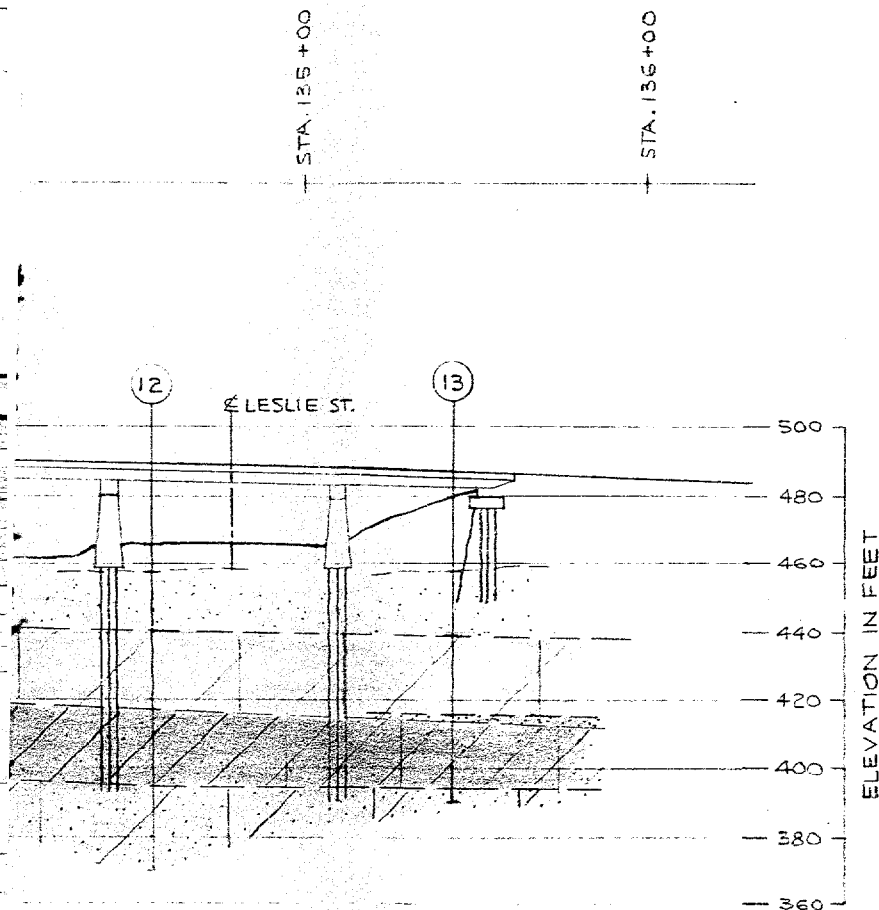
1

BOREHOLE IN ELEVATION

BY F. C.

BY GEO

STRATIG



COMPACT TO DENSE BROWN



COMPACT TO DENSE BROWN M



SOFT TO FIRM GREY SILTY CLAY



LOOSE GREY SAND



FIRM GREY SILTY CLAY WITH



DENSE TO VERY DENSE COARSE
OF SILTY CLAY

NOTE: STRATIGRAPHY DESCRIPTION
OF PRESENT INVESTIGATION

SPECIAL NOTE: DATA CON-
STRATA HAVE BEEN OBTAIN-
ED ONLY. THE SOIL ST-
BOREHOLE HAS BEEN INFER-
EVIDENCE AND SO MAY VARY

REFERENCE	
DWG. NO.	DESCRIPTION
17-B-72	PLAN OF HIGHWAY 401 STA. 112+66.7 TO STA. 267+91.31 COMPILED FROM AERIAL PHOTOGRAPH, DATED SEPT. 1961 SCALE 1" TO 50'

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO

PROPOSED ALTERATIONS
LESLIE STREET AND HIGHWAY 401 INTERCH
TORONTO

SOIL STRATIGRAPHY

LEGEND



BOREHOLE IN ELEVATION (PRESENT INVESTIGATION)



BOREHOLE IN ELEVATION (PREVIOUS INVESTIGATION)

BY F. C. C. LTD., 1953

BY GEOCON LTD., 1959

STRATIGRAPHY

ELEVATION IN FEET

500
480
460
440
420
400
380
360



COMPACT TO DENSE BROWN SILTY & CLAYEY SAND FILL.



COMPACT TO DENSE BROWN MEDIUM TO FINE SAND, TRACE OF CLAY.



SOFT TO FIRM GREY SILTY CLAY, TRACE OF SAND AND GRAVEL.



LOOSE GREY SAND



FIRM GREY SILTY CLAY WITH SAND, GRAVEL AND PEBBLES.



DENSE TO VERY DENSE COARSE TO FINE SAND, AND LAYERS OF SILTY CLAY. (TILL)

NOTE: STRATIGRAPHY DESCRIPTIONS BASED PRIMARILY ON RESULTS OF PRESENT INVESTIGATION, 1962.

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

30M14-142

GEOCRE No.

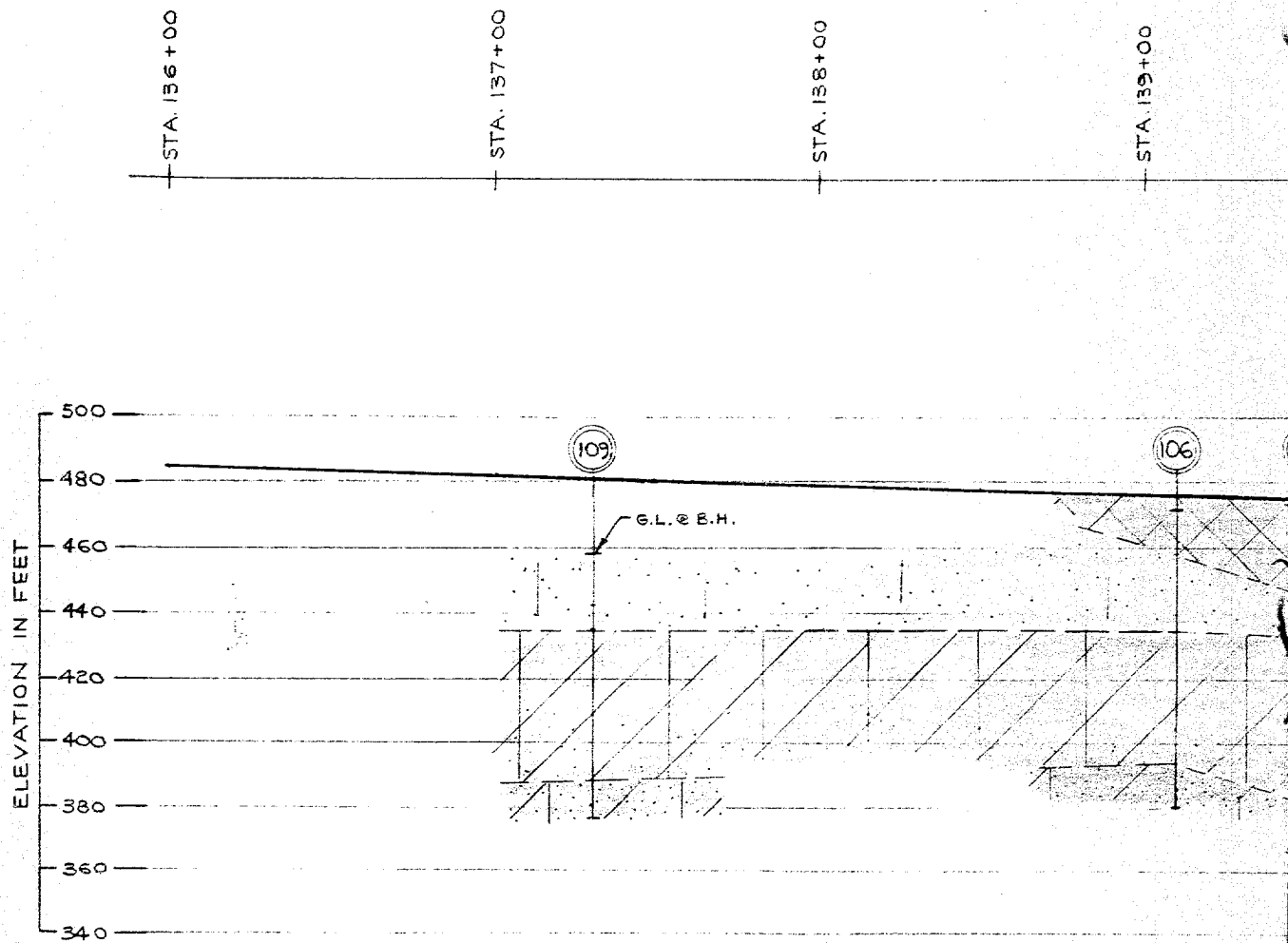
ON
401 STA. 112+66.7
EMPILED FROM
PH, DATED SEPT, 1961
1" TO 50'

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO
PROPOSED ALTERATIONS
LESLIE STREET AND HIGHWAY 401 INTERCHANGE
TORONTO ONTARIO
SOIL STRATIGRAPHY

GOLDER & ASSOCIATES
CONSULTING CIVIL ENGINEERS

DATE: AUG. 13/62 SCALE: 1" TO 40'-0"

MADE M.W. CHKD. APPD. FIGURE 3



STA. 139+00

STA. 140+00

STA. 141+00

STA. 142+00

STA. 143+00

CHAINAGE ALONG CENTRELINE HIGH

106

107

GROUND LEVEL

G.L. & B.H.

LONGITUDINAL

STA. 143+00

STA. 144+00

STA. 145+00

STA. 145+00

NTRELINE HIGHWAY 401

105

17

18

104

19

G.L. @ B.H.

G.L. @ B.H.

ASSUMED ORIGINAL
GROUND LEVEL

G.L. @ B.H.
G.L. @ B.H.

G.L. @ B.H.

G.L. @ B.H.

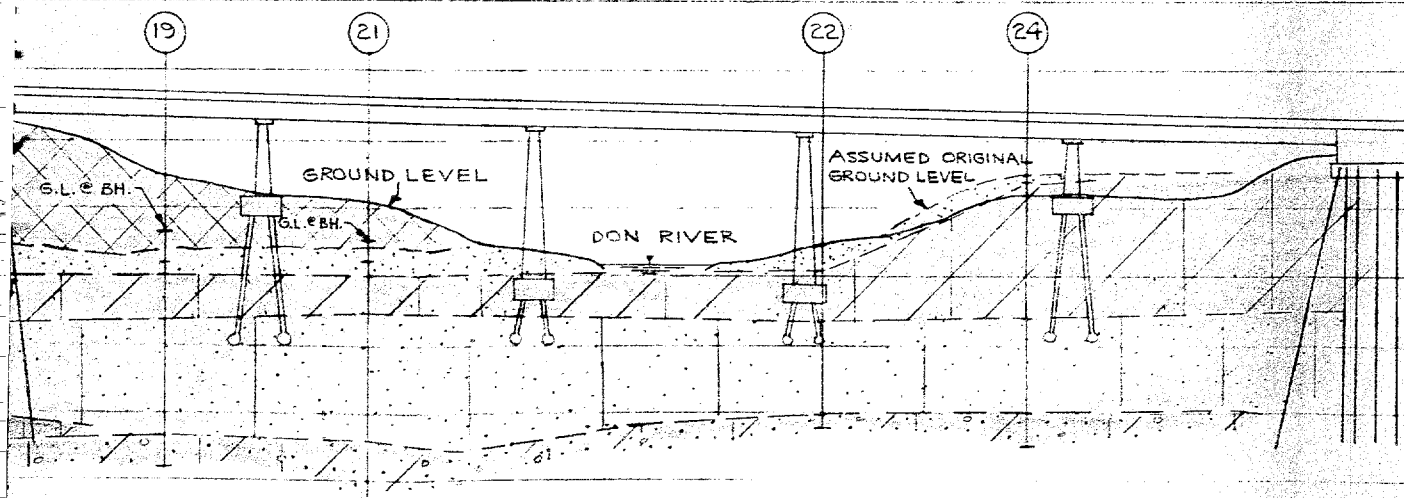
ONGITUDINAL SECTION ALONG CENTRELINE HIGHWAY 401
STATION 136+00 TO 151+00

— STA. 147+00

— STA. 148+00

— STA. 149+00

— STA. 150+00



NOTE: STRATIGRAPHY DESCRIPTIONS BASED PRIMARILY
ON RESULTS OF PRESENT INVESTIGATION, 1962.

DWG. NO.	
17-B-72	PLAN OF TO STA. AERIAL

— STA. 151+00

LEGEND



BOREHOLE IN ELEVATION (PRESENT INVENTORY)



BOREHOLE IN ELEVATION (PREVIOUS INVENTORY)
BY F.C.C. LTD., 1953
BY GEOCON LTD, 1959

STRATIGRAPHY



COMPACT TO DENSE BROWN SILTY CLAYEY SAND



LOOSE TO DENSE BROWN STRATIFIED FINE TO MEDIUM GRAIN SAND WITH TRACE OF SILT.



SOFT TO FIRM GREY SILTY CLAY TRACE OF SAND GRAVEL.



DENSE TO VERY DENSE SILTY FINE SAND WITH OCCASIONAL CLAY LAYERS.



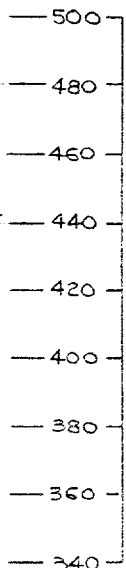
DENSE TO VERY DENSE SILTY COARSE TO FINE SAND LAYERS OF SILTY CLAY. (TILL)



STIFF GREY CLAYEY SILT.

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

ELEVATION IN FEET



REFERENCE

DWG. NO: DESCRIPTION

17-B-72 PLAN OF HIGHWAY 401, STA. 112+66.7 TO STA. 267+91.31 COMPILED FROM AERIAL PHOTOGRAPH, DATED SEPT. 1961 SCALE 1" TO 50'

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO

PROPOSED ALTERATIONS
LESLIE STREET AND HIGHWAY 401 INTERCHANGE
TORONTO ONTARIO

SOIL STRATIGRAPHY

GOLDER &
CONSULTING

DATE: AUG. 13/62

MADE M.W. CHKD. APPD.

LEGEND



BOREHOLE IN ELEVATION (PRESENT INVESTIGATION)

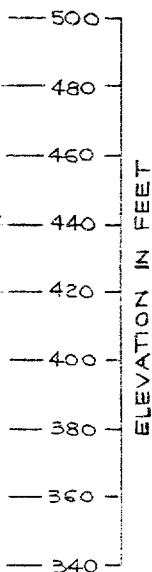


BOREHOLE IN ELEVATION (PREVIOUS INVESTIGATION)

BY R.C.C. LTD., 1953

BY GEOCON LTD, 1959

STRATIGRAPHY



COMPACT TO DENSE BROWN SILTY CLAYEY SAND FILL.



LOOSE TO DENSE BROWN STRATIFIED FINE TO MEDIUM SAND, TRACE OF SILT.



SOFT TO FIRM GREY SILTY CLAY, TRACE OF SAND AND FINE GRAVEL.



DENSE TO VERY DENSE SILTY FINE SAND WITH OCCASIONAL SAND AND CLAY LAYERS.



DENSE TO VERY DENSE SILTY COARSE TO FINE SAND, AND LAYERS OF SILTY CLAY. (TILL)



STIFF GREY CLAYEY SILT.

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

30M14-142

GEOCRES No.

ON
401, STA. 112+66.7
MAILED FROM
TH, DATED SEPT. 1961
"To 50"

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO
PROPOSED ALTERATIONS
LESLIE STREET AND HIGHWAY 401 INTERCHANGE
TORONTO ONTARIO

SOIL STRATIGRAPHY

GOLDER & ASSOCIATES
CONSULTING CIVIL ENGINEERS

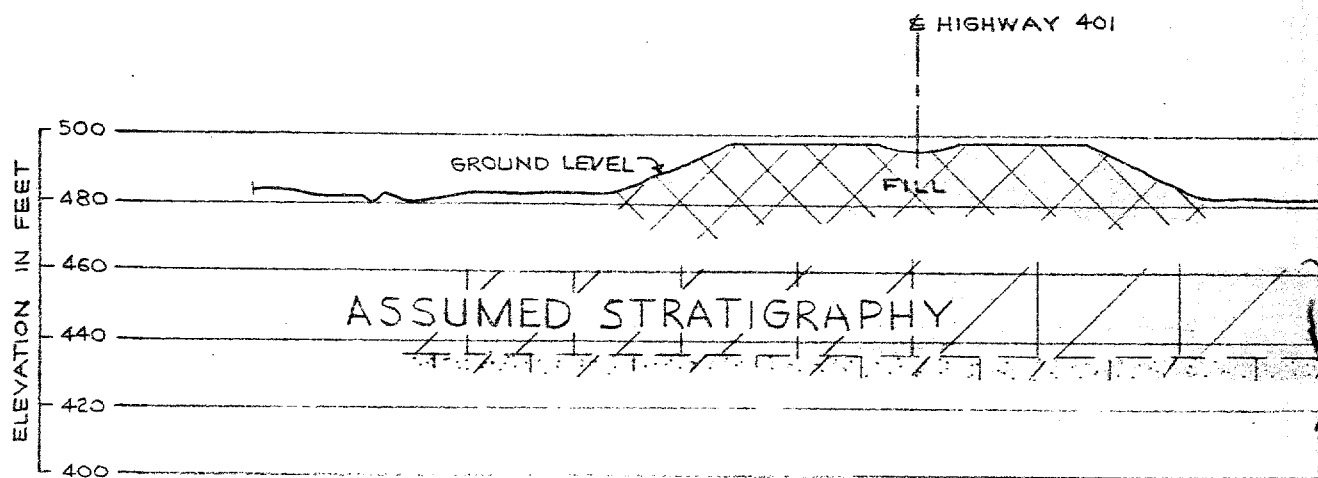
DATE: AUG. 13 / 62 SCALE: 1" TO 40'-0"

MADE
M.W.

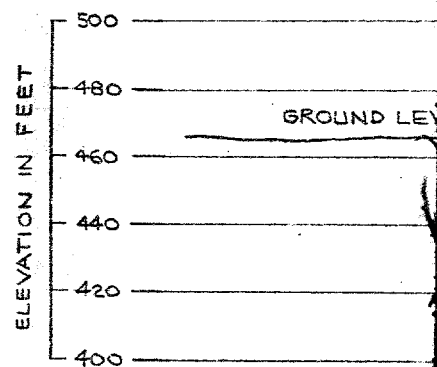
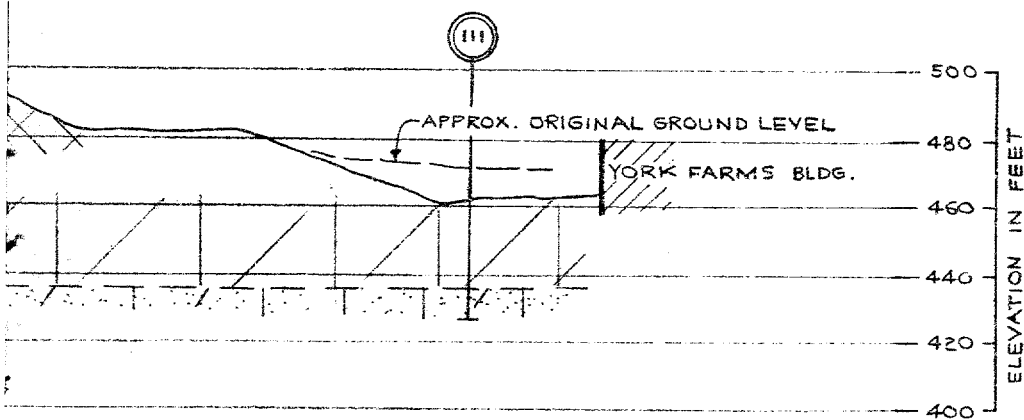
CHKD.
11-16

APPD.
11-16

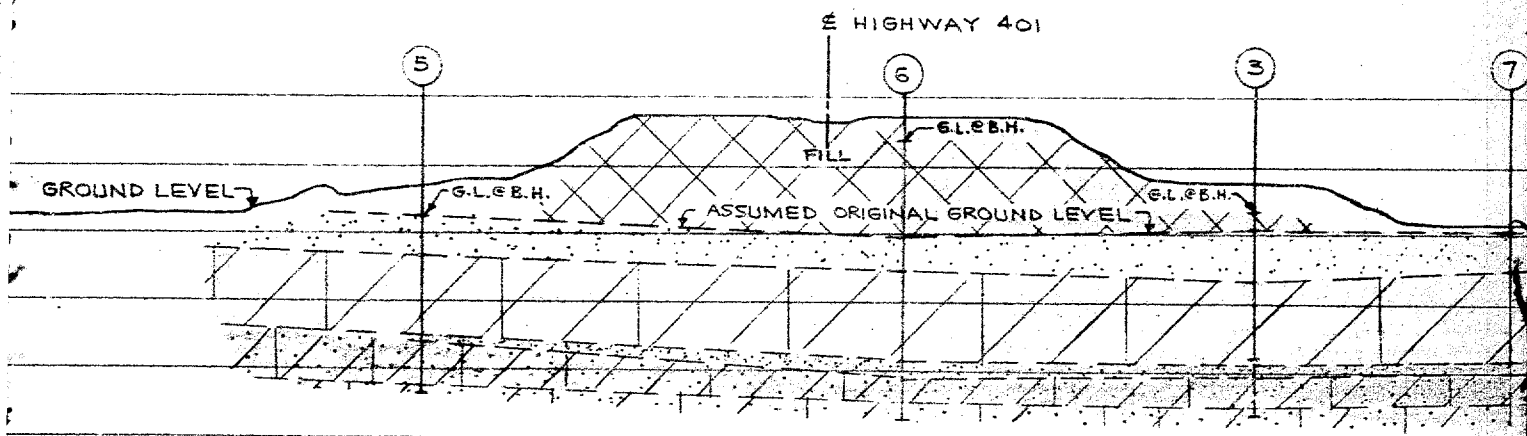
FIGURE 4



SECTION AT STATION 12



ATION 124+65 (HWY. 401)



SECTION AT STATION 131+00 (HWY. 401)

LEGEND



BOREHOLE IN ELEVATION



BOREHOLE IN ELEVATION

BY F.C.C.

BY GEOC

STRATIGRAPHY



COMPACT TO DENSE BROWN ME
TRACE OF CL



FIRM GREY SILTY CLAY, TRACE



LOOSE GREY SAND



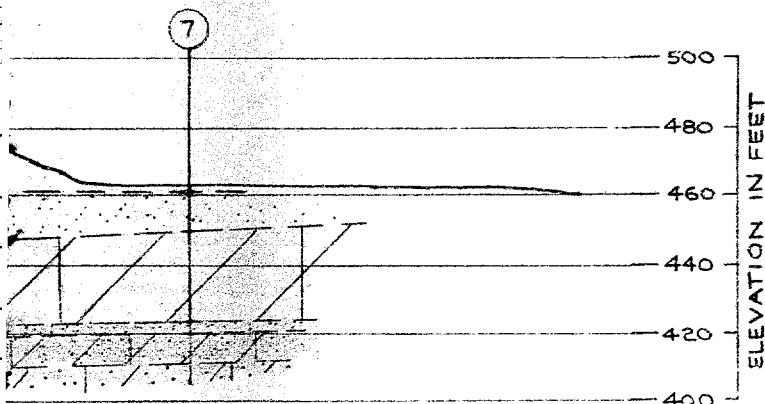
FIRM GREY SILTY CLAY WITH SAND, GA



DENSE TO VERY DENSE COARSE TO F
OF SILTY CLAY (C

NOTE: STRATIGRAPHY DESCRIPTIONS
OF PRESENT INVESTIGATION,

SPECIAL NOTE: DATA CONCERNING THE
STRATA HAVE BEEN OBTAINED AT BOREH
TIONS ONLY. THE SOIL STRATIGRAPHY
BOREHOLE HAS BEEN INFERRED FROM G
EVIDENCE AND SO MAY VARY FROM THAT



401)

REFERENCE		DEPARTMENT OF HIGHWAYS, ONTARIO	
DWG. NO.	DESCRIPTION	TORONTO	ONTARIO
17-8-72	PLAN OF HIGHWAY 401, STA. 112+66.7 TO STA. 267+91.31 COMPILED FROM AERIAL PHOTOGRAPH, DATED SEPT. 1961 SCALE 1" TO 50'	PROPOSED ALTERATIONS LESLIE STREET AND HIGHWAY 401 INTERCHANGE TORONTO	ONTARIO
		SOIL STRATIGRAPHY	

LEGEND



BOREHOLE IN ELEVATION (PRESENT INVESTIGATION)



BOREHOLE IN ELEVATION (PREVIOUS INVESTIGATION)
BY F. C. C. LTD., 1953
BY GEOCON LTD., 1959

STRATIGRAPHY



COMPACT TO DENSE BROWN MEDIUM TO FINE SAND,
TRACE OF CLAY.



FIRM GREY SILTY CLAY, TRACE OF SAND AND GRAVEL.



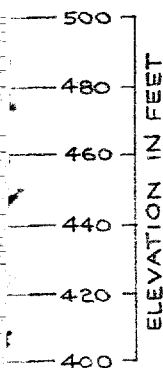
LOOSE GREY SAND



FIRM GREY SILTY CLAY WITH SAND, GRAVEL AND PEBBLES.



DENSE TO VERY DENSE COARSE TO FINE SAND, AND LAYERS
OF SILTY CLAY (FILL)

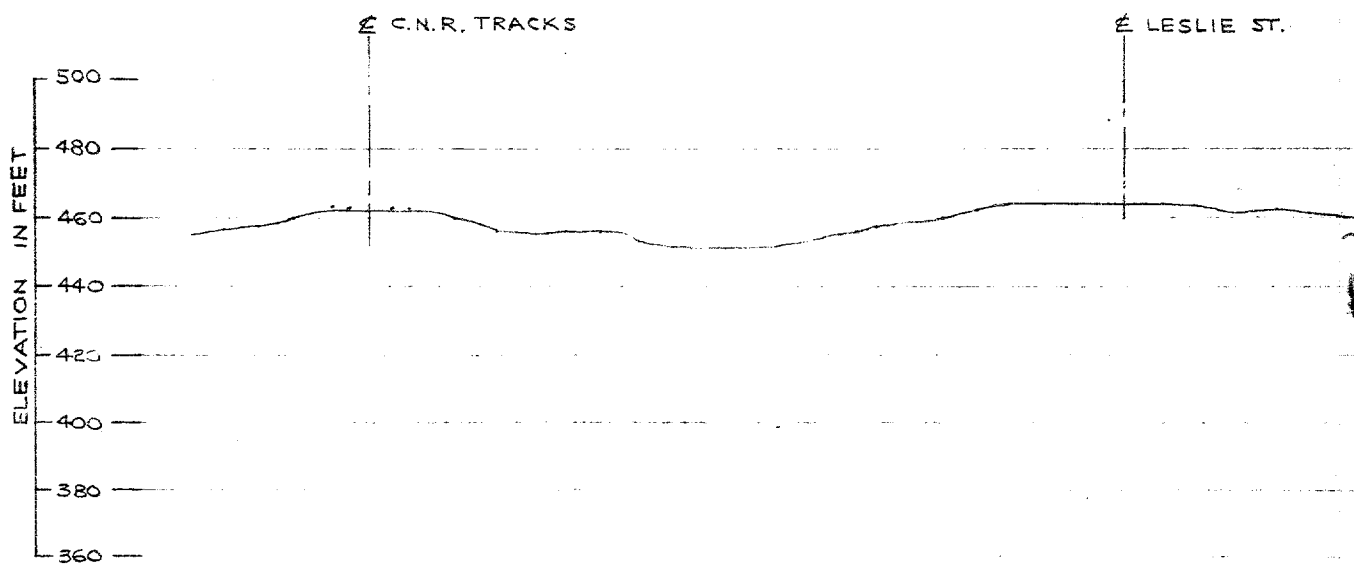


NOTE: STRATIGRAPHY DESCRIPTIONS BASED PRIMARILY ON RESULTS
OF PRESENT INVESTIGATION, 1962.

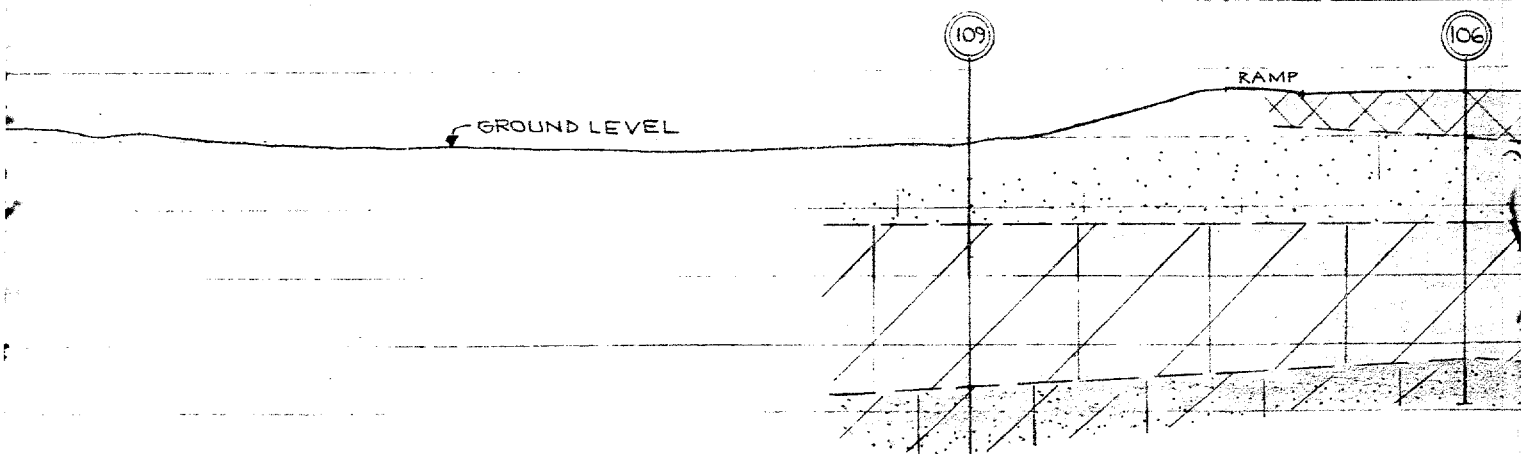
SPECIAL NOTE: DATA CONCERNING THE VARIOUS
STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCA-
TIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN
BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL
EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

30 M14-142

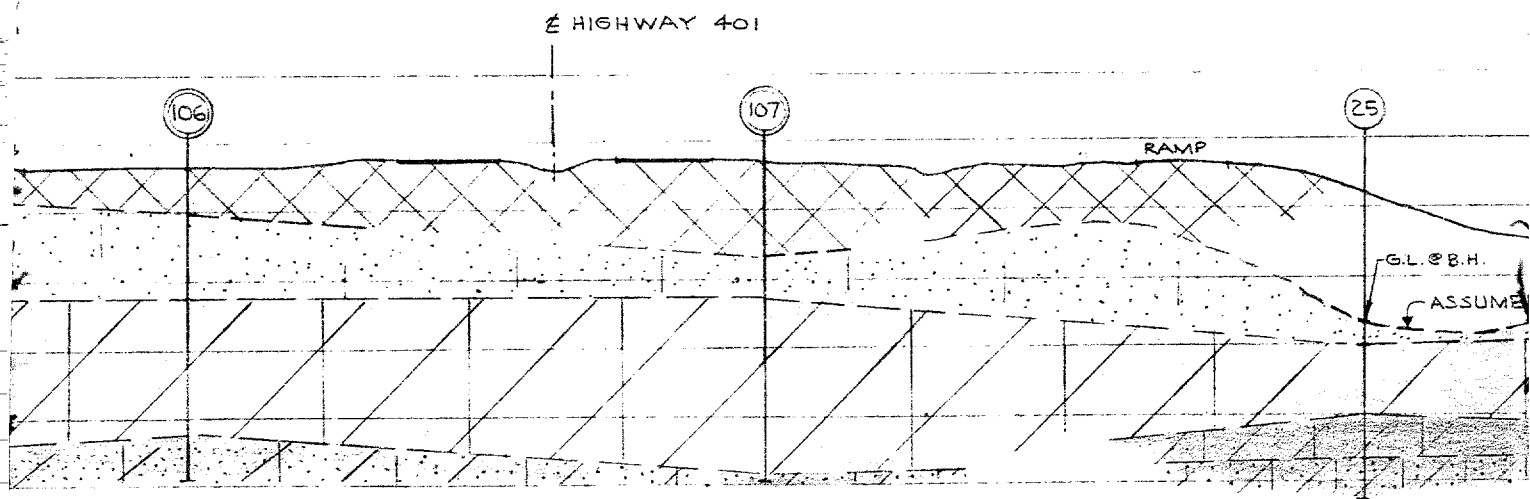
ON 401, STA. 112+66.7 EMPILED FROM PH, DATED SEPT. 1961 "To 50"	DEPARTMENT OF HIGHWAYS, ONTARIO TORONTO	GOLDER & ASSOCIATES CONSULTING CIVIL ENGINEERS				
	PROPOSED ALTERATIONS LESLIE STREET AND HIGHWAY 401 INTERCHANGE TORONTO	DATE: AUG. 13/62 SCALE: 1" TO 40'-0"				
	SOIL STRATIGRAPHY	<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 15%;">MADE M.W.</td> <td style="width: 15%;">CHKD. </td> <td style="width: 15%;">APPD. </td> <td style="width: 55%;">FIGURE 5</td> </tr> </table>	MADE M.W.	CHKD. 	APPD. 	FIGURE 5
MADE M.W.	CHKD. 	APPD. 	FIGURE 5			



LESLIE ST.

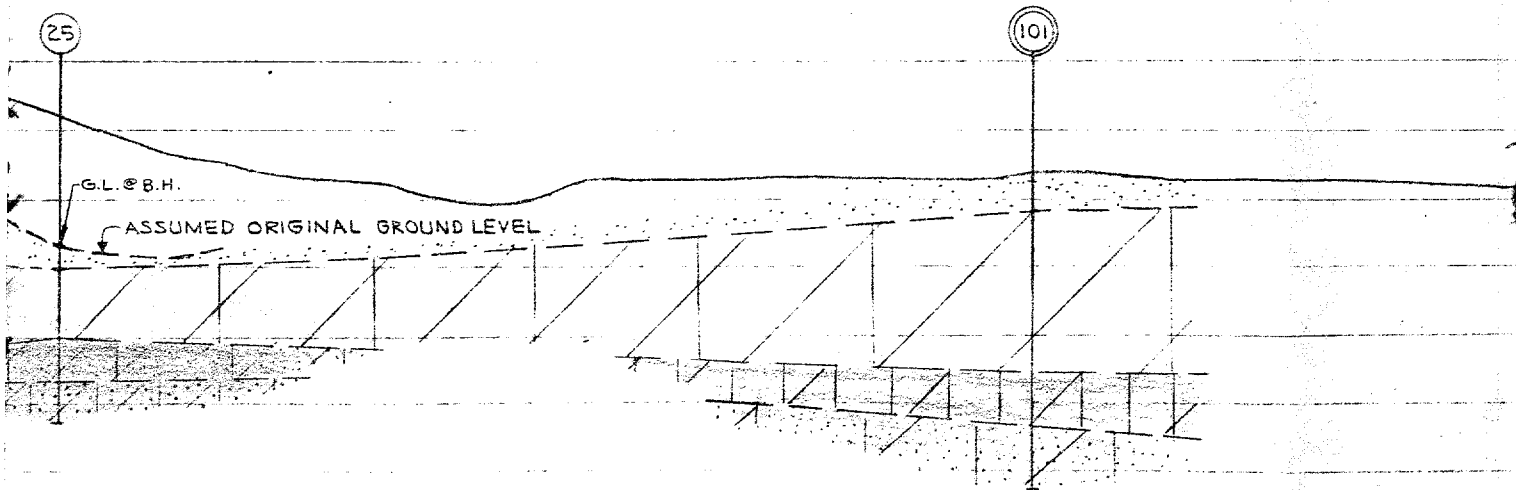


DIAGONAL SEC



GONAL SECTION THROUGH STATION 139+50 (HWY 401)

REFER TO FIGURE 1



REVISIONS	
DWG. NO.	DESCRIPTION
17-B-72	PLAN OF TO STA. 22 AERIAL P

LEGEND

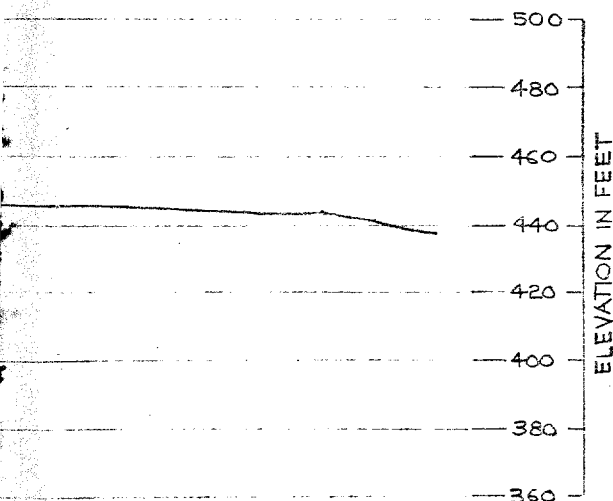
(101)

BOREHOLE IN ELEVATION (PRESENT)

(25)

BOREHOLE IN ELEVATION (PREVIOUS
BY F. C. C. LTD., 1953
BY GEOCON LTD., 1959)

STRATIGRAPHY



COMPACT TO DENSE BROWN SILTY CLAYEY SILT



COMPACT TO DENSE BROWN STRATIFIED FINE SAND, SOME SILT AND CLAY LAYERS.



SOFT TO FIRM GREY SILTY CLAY, TRACE OF SAND GRAVEL.



FIRM GREY SILTY CLAY WITH SAND, GRAVEL AND PEBBLES.



DENSE TO VERY DENSE SILTY COARSE TO FINE SAND AND LAYERS OF SILTY CLAY.



STIFF TO VERY STIFF GREY CLAYEY SILT

NOTE: STRATIGRAPHY DESCRIPTIONS BASED PRIMARILY OF PRESENT INVESTIGATION, 1962.

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

REFERENCE

DWG. NO.

DESCRIPTION

17-B-72

PLAN OF HIGHWAY 401, STA. 112+66.7 TO STA. 227+31.31 COMPILED FROM AERIAL PHOTOGRAPH, DATED SEPT. 1961 SCALE 1" TO 50'

DEPARTMENT OF HIGHWAYS, ONTARIO

TORONTO

ONTARIO

PROPOSED ALTERATIONS
LESLIE STREET AND HIGHWAY 401 INTERCHANGE

TORONTO

ONTARIO

SOIL STRATIGRAPHY

GOLDER &
CONSULTING

DATE: AUG. 13/62

MADE M.W. CHKD. RD APPD. 17

LEGEND

101

BOREHOLE IN ELEVATION (PRESENT INVESTIGATION)

25

BOREHOLE IN ELEVATION (PREVIOUS INVESTIGATION)

BY F. C. C. LTD., 1953

BY GEOCON LTD., 1959

STRATIGRAPHY



COMPACT TO DENSE BROWN SILTY CLAYEY SAND FILL.



COMPACT TO DENSE BROWN STRATIFIED FINE TO MEDIUM SAND, SOME SILT AND CLAY LAYERS.



SOFT TO FIRM GREY SILTY CLAY, TRACE OF SAND AND FINE GRAVEL.



FIRM GREY SILTY CLAY WITH SAND, GRAVEL AND PEBBLES.



DENSE TO VERY DENSE SILTY COARSE TO FINE SAND, AND LAYERS OF SILTY CLAY. (TILL)



STIFF TO VERY STIFF GREY CLAYEY SILT.

NOTE: STRATIGRAPHY DESCRIPTIONS BASED PRIMARILY ON RESULTS OF PRESENT INVESTIGATION, 1962.

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

30M14-142

FIGURE NO.

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO

GOLDER & ASSOCIATES
CONSULTING CIVIL ENGINEERS

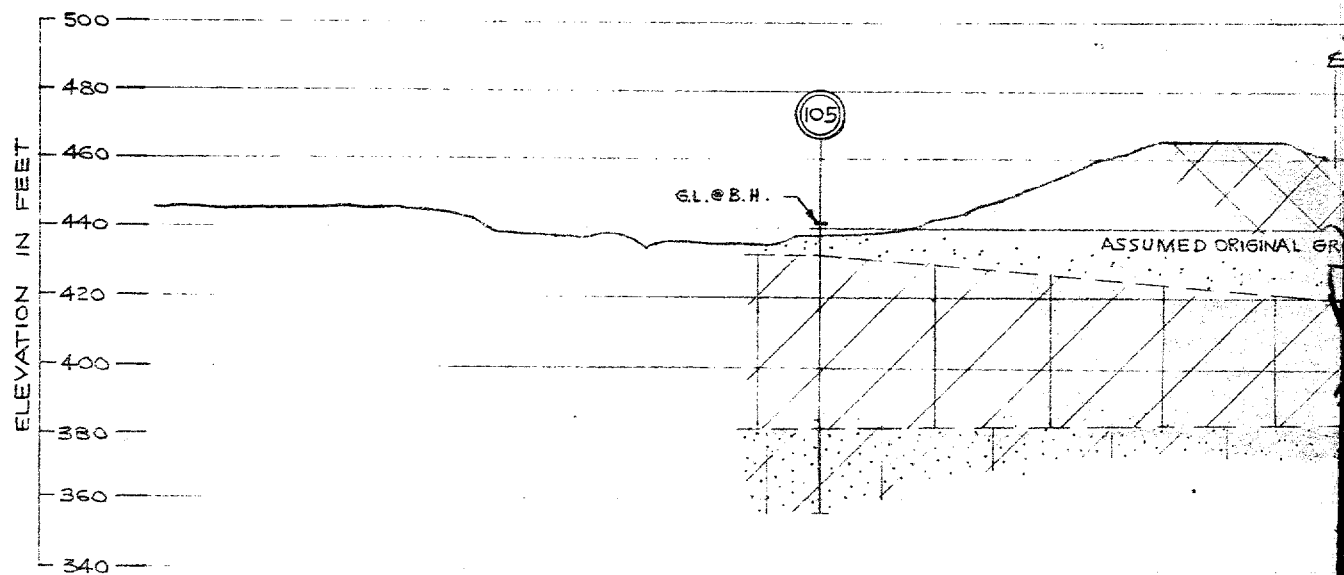
401, STA. 112+66.7
PILED FROM
DATED SEPT. 1961
to 50'

PROPOSED ALTERATIONS
LESLIE STREET AND HIGHWAY 401 INTERCHANGE
TORONTO ONTARIO

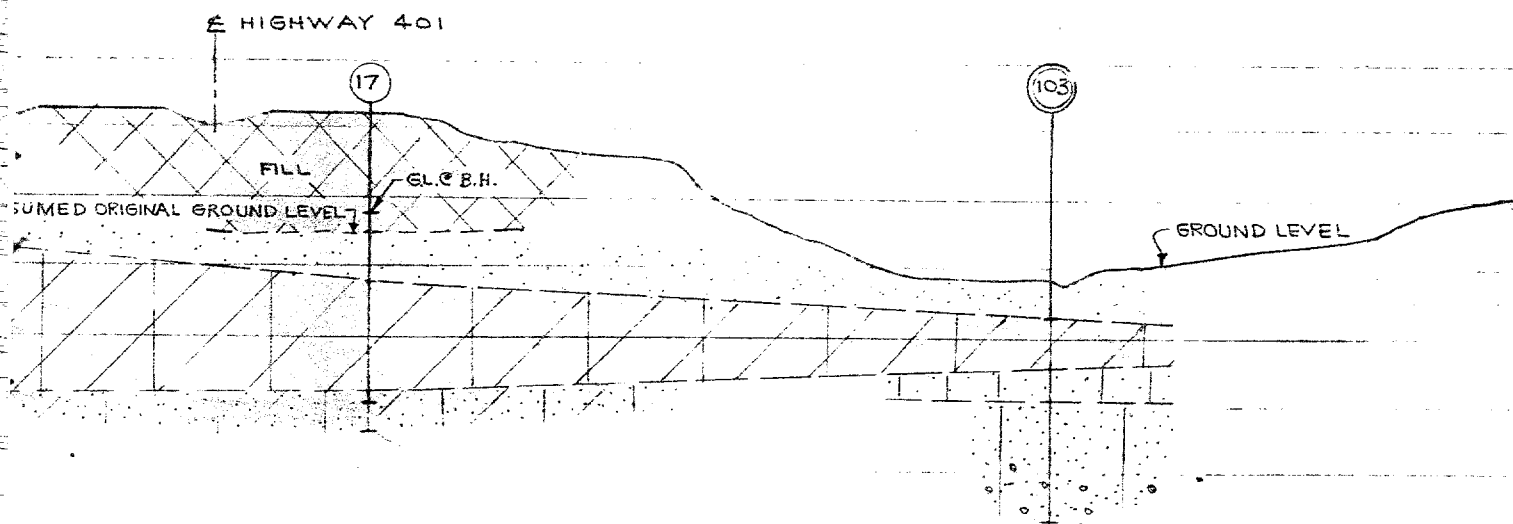
DATE: AUG. 13/62 SCALE: 1" TO 40'-0"

SOIL STRATIGRAPHY

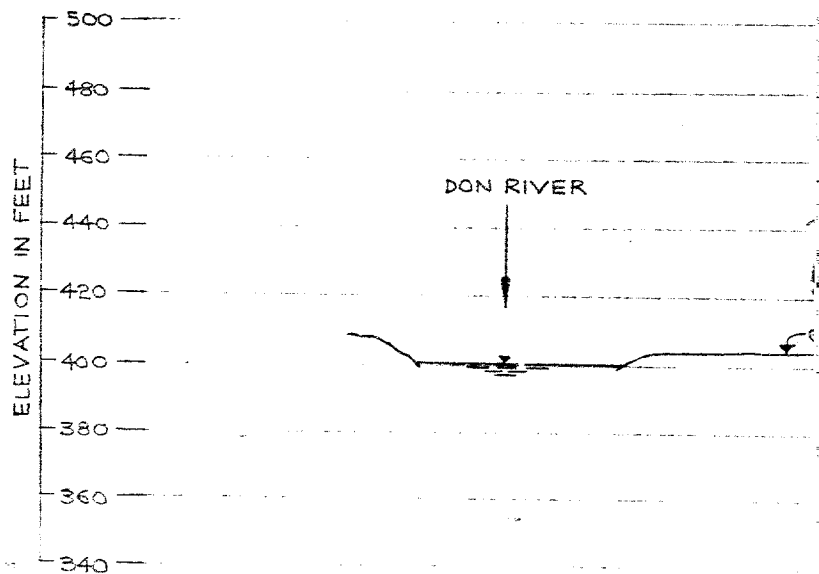
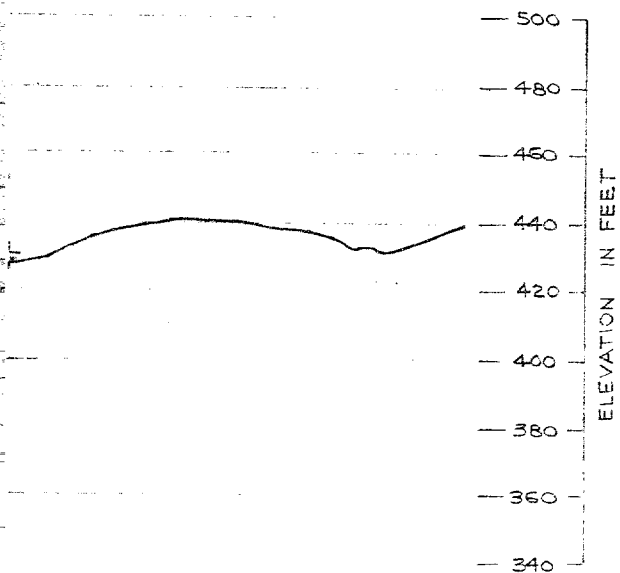
MADE	CHKD.	APPD.	FIGURE 6
M.W.	60	17	



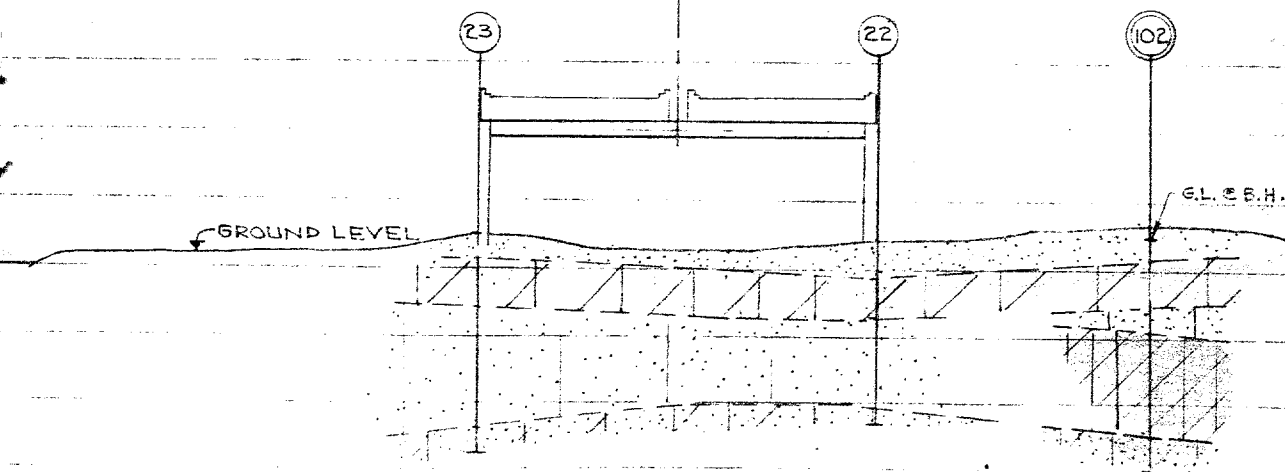
SECTION



SECTION AT STATION 143+50 (HWY 401)



E HIGHWAY 401



SECTION AT STATION 148+50 (HWY. 401)

DWG. NO.	
17-B-72	PLAN TO STA AERIA

LEGEND



BOREHOLE IN ELEVATION (PRESENT)



BOREHOLE IN ELEVATION (PREVIOUS)
BY F.C.C. LTD, 1953
BY GEOCON LTD, 1959

STRATIGRAPHY



LOOSE TO COMPACT BROWN SILTY FINE SAND OF CLAY & GRAVEL.



FIRM GREY SILTY CLAY, TRACE OF SAND.



LOOSE TO DENSE GREY SANDY SILT, CLAY.



COMPACT TO DENSE GREY SILTY FINE SAND WITH INCREASING DEPTH.



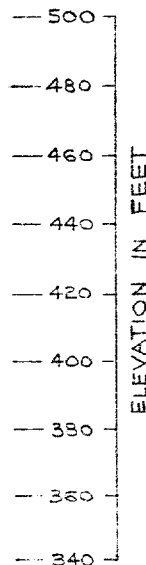
COMPACT TO VERY DENSE GREY SILT AND SOME SAND LAYERS.



DENSE TO VERY DENSE SILTY COARSE TO FINE SAND LENSES OR LAYERS OF SILT.

NOTE: STRATIGRAPHY DESCRIPTIONS BASED PRIMARILY ON DATA OF PRESENT INVESTIGATION, 1962.

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



REFERENCE	
DWG. NO.	DESCRIPTION
17-B-72	PLAN OF HIGHWAY 401 STA. 112+66.7 TO STA. 267+91.31 COMPILED FROM AERIAL PHOTOGRAPH, DATED SEPT. 1961 SCALE 1" TO 50'

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO ONTARIO
PROPOSED ALTERATIONS
LESLIE STREET AND HIGHWAY 401 INTERCHANGE
TORONTO ONTARIO
SOIL STRATIGRAPHY

GOLDER
CONSULT.

DATE: AUG. 13/62

MADE M.W. CHKD. AP

LEGEND

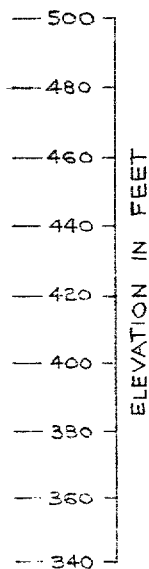


BOREHOLE IN ELEVATION (PRESENT INVESTIGATION)



BOREHOLE IN ELEVATION (PREVIOUS INVESTIGATION)
BY F.C.C.LTD, 1953
BY GEOCON LTD, 1959

STRATIGRAPHY



LOOSE TO COMPACT BROWN SILTY FINE SAND, TRACE OF CLAY & GRAVEL.



FIRM GREY SILTY CLAY, TRACE OF SAND AND GRAVEL.



LOOSE TO DENSE GREY SANDY SILT, OCCASIONAL PEBBLES.



COMPACT TO DENSE GREY SILTY FINE SAND, SOME GRAVEL WITH INCREASING DEPTH.



COMPACT TO VERY DENSE GREY SILT AND CLAY TO SILT, SOME SAND LAYERS.



DENSE TO VERY DENSE SILTY COARSE TO FINE SAND AND LENSES OR LAYERS OF SILTY CLAY. (TILL)

NOTE: STRATIGRAPHY DESCRIPTIONS BASED PRIMARILY ON RESULTS OF PRESENT INVESTIGATION, 1962.

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

30M 14-142

DEPARTMENT OF HIGHWAYS, ONTARIO

TORONTO

ONTARIO

PROPOSED ALTERATIONS

LESLIE STREET AND HIGHWAY 401 INTERCHANGE

TORONTO

ONTARIO

SOIL STRATIGRAPHY

GOLDER & ASSOCIATES

CONSULTING CIVIL ENGINEERS

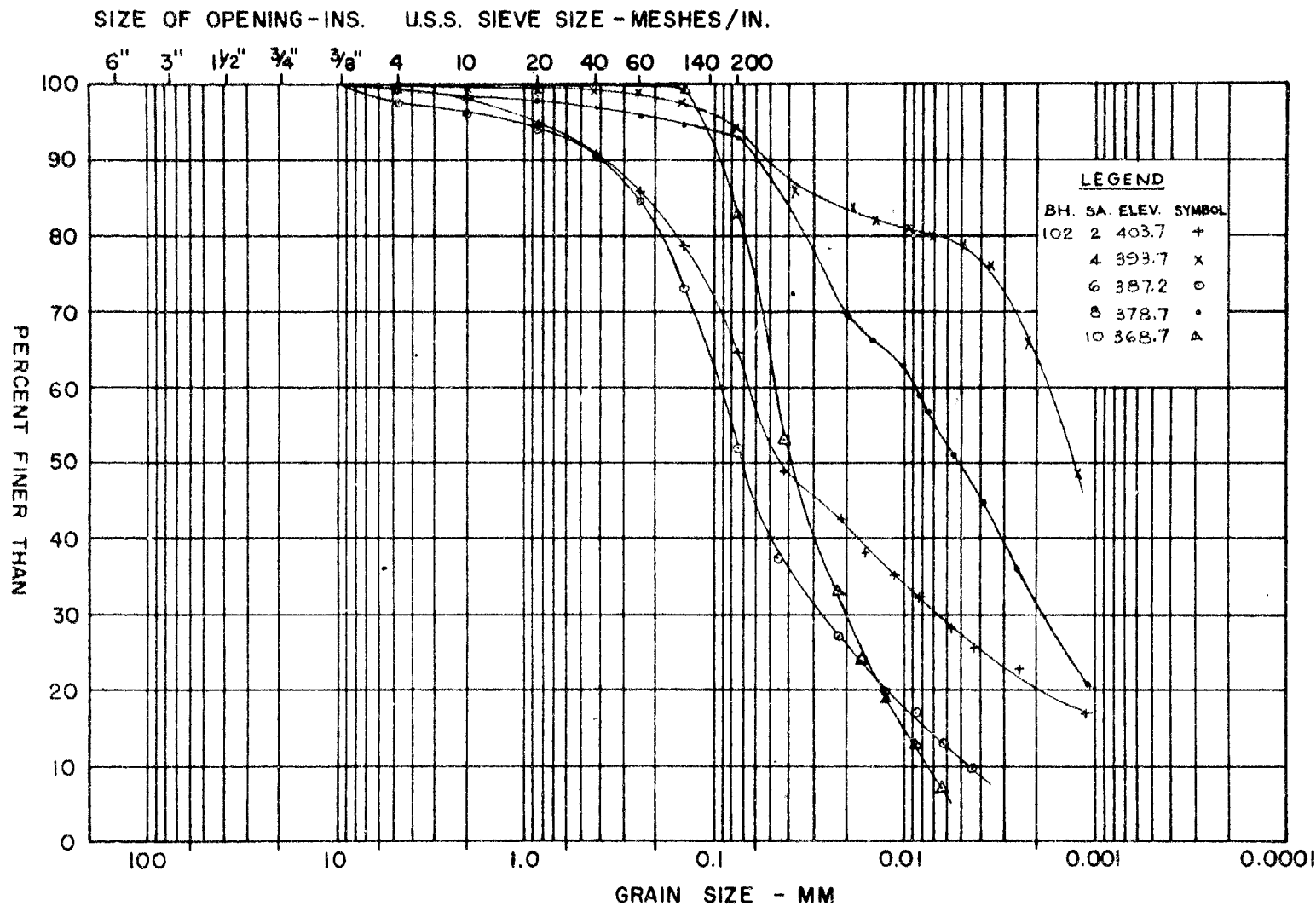
DATE: AUG. 13/62 SCALE: 1" TO 40'-0"

MADE M.W. CHKD. APPD.

FIGURE 7

401 STA. 112+66.7
MAILED FROM
H, DATED SEPT 1961
TO 50'

M.I.T. GRAIN SIZE SCALE



GOLDER & ASSOCIATES

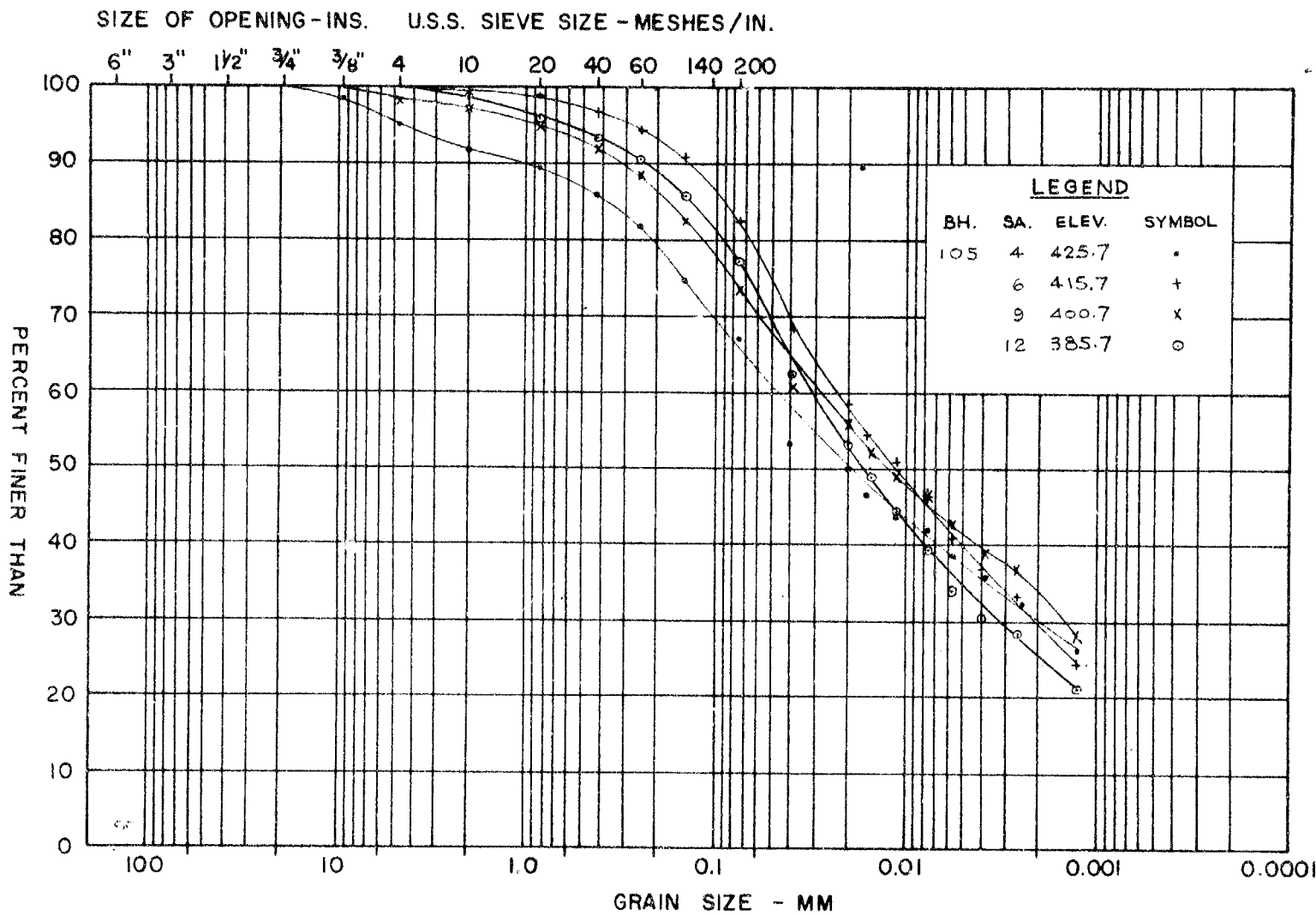
GRAIN SIZE DISTRIBUTION

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



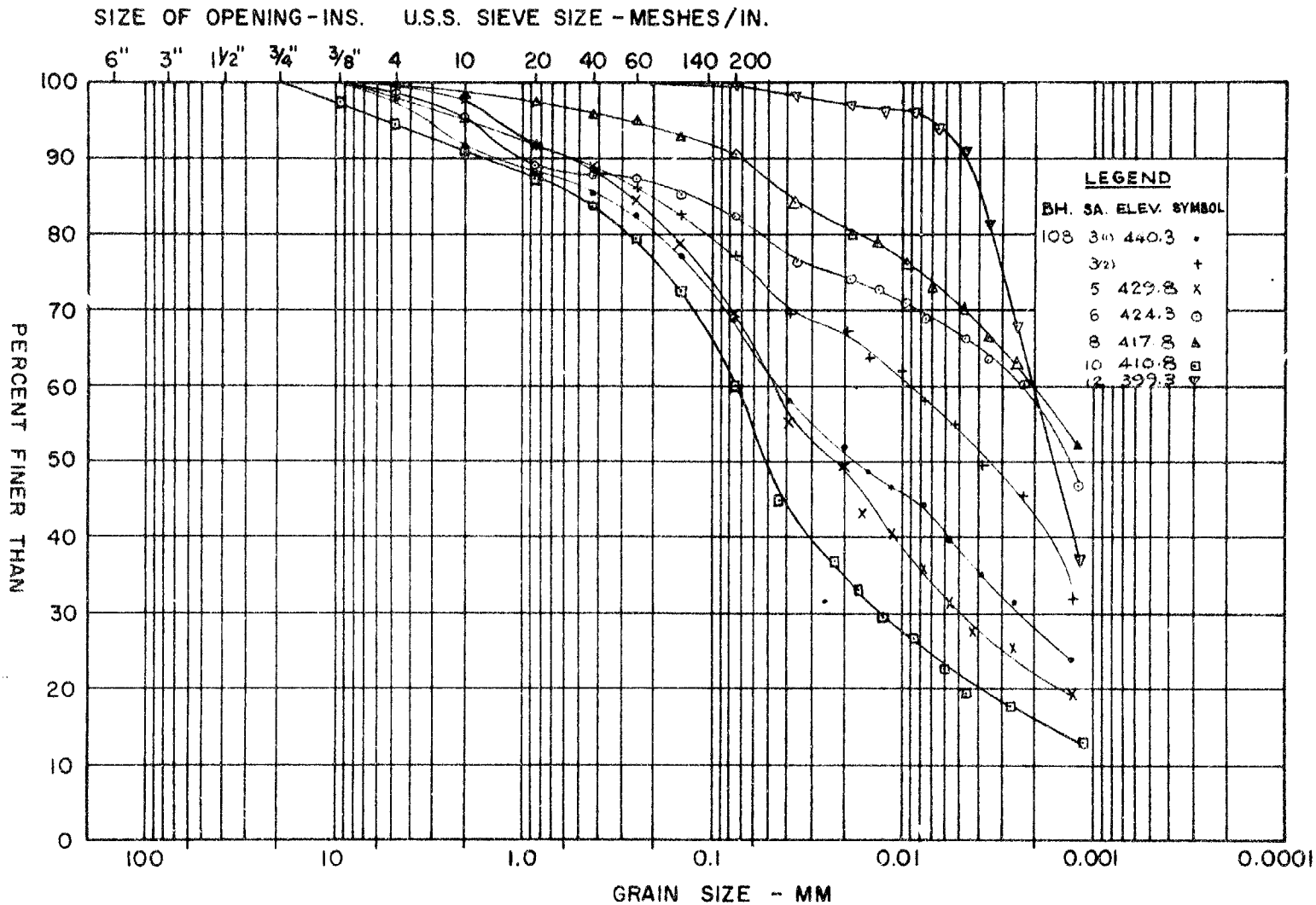
GOLDER & ASSOCIATES

GRAIN SIZE DISTRIBUTION

FIGURE 9

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

FIGURE 10

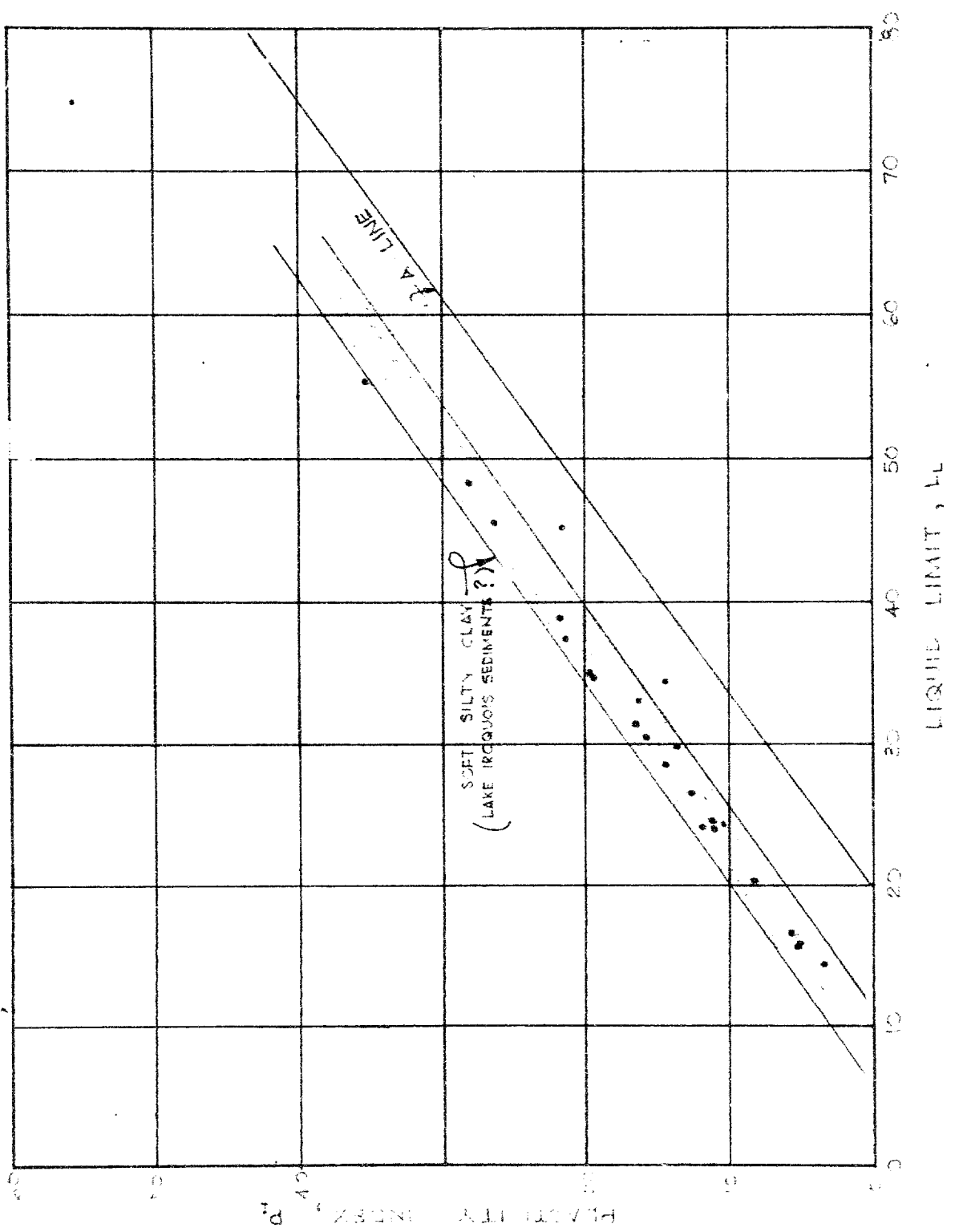
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

PLASTICITY CHART

SOFT SILTY CLAY

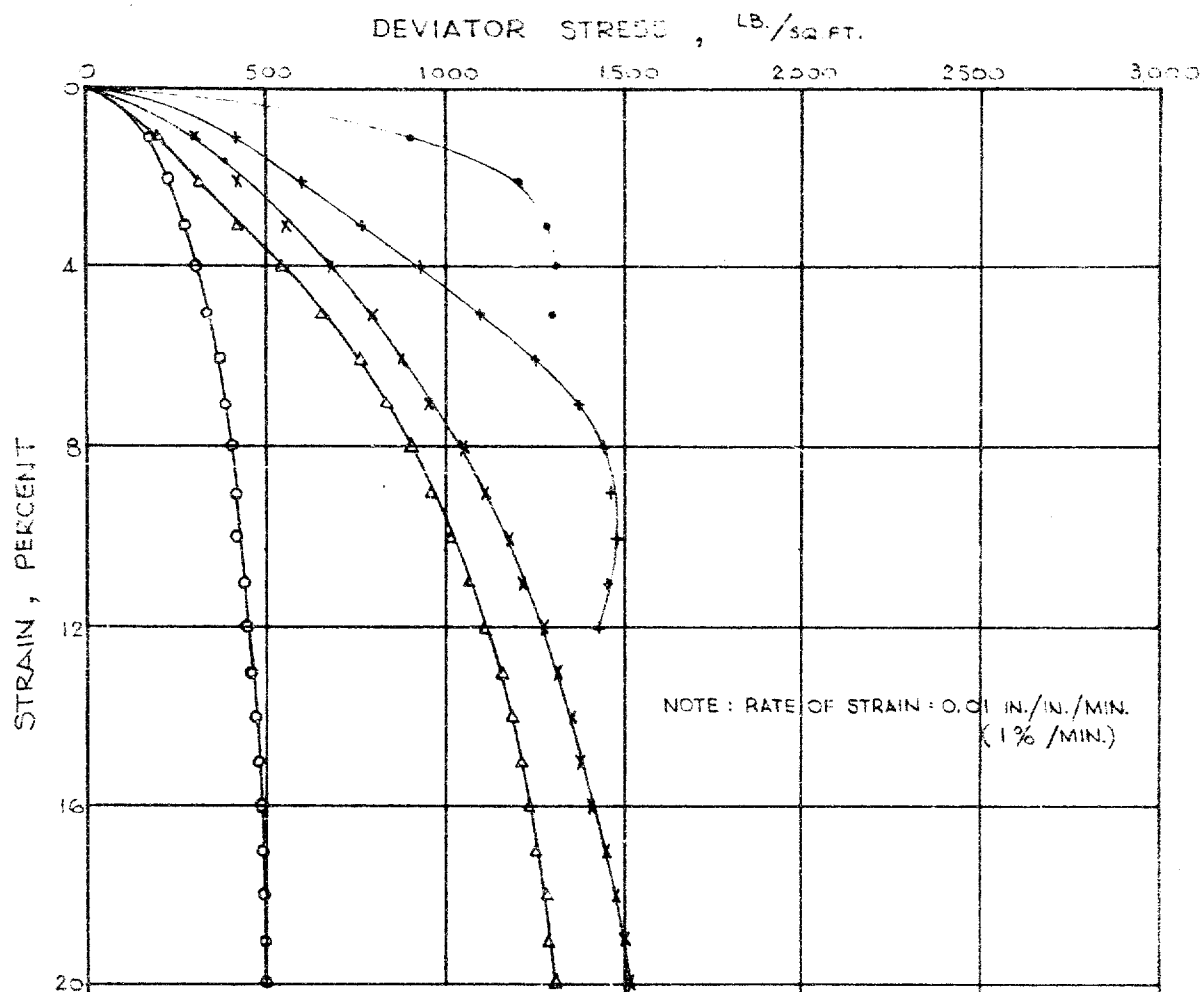
FIGURE 11

(PRESENT INVESTIGATION)



UNDRAINED TRIAXIAL COMPRESSION TESTS TYPICAL STRESS-STRAIN CURVES — SILTY CLAY STRATUM

FIGURE 12

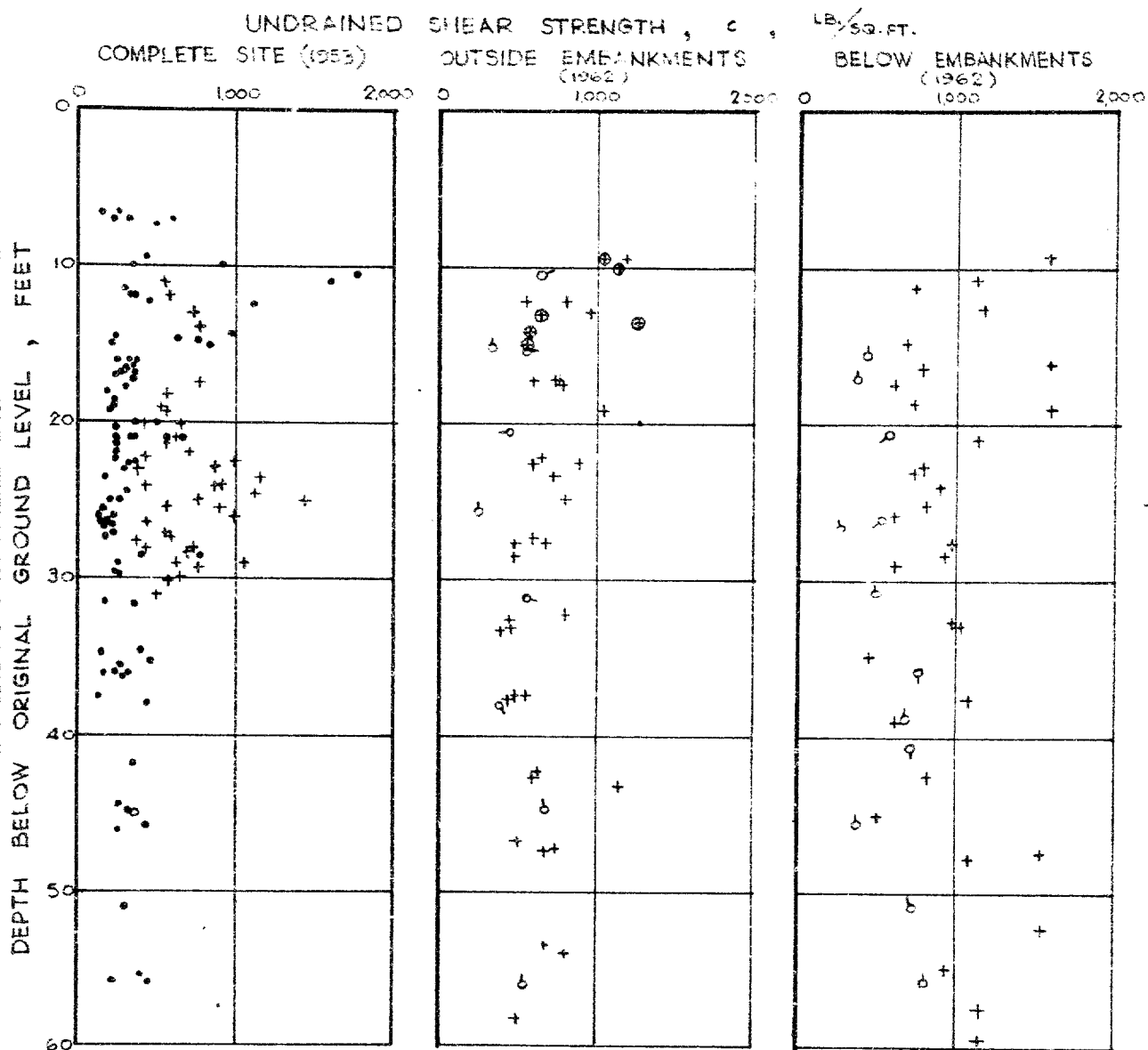


LEGEND

SYMBOL	HOLE	SAMPLE	ELEVATION
•	102	3	397.5
+	105	8	404.5
x	105	12	384.5
o	108	5	428.5
Δ	112	18	401.3

VARIATION OF UNDRAINED SHEAR STRENGTH
WITH DEPTH BELOW GROUND LEVEL 1953 - 1962
SILTY CLAY STRATUM

FIGURE 13



LEGEND

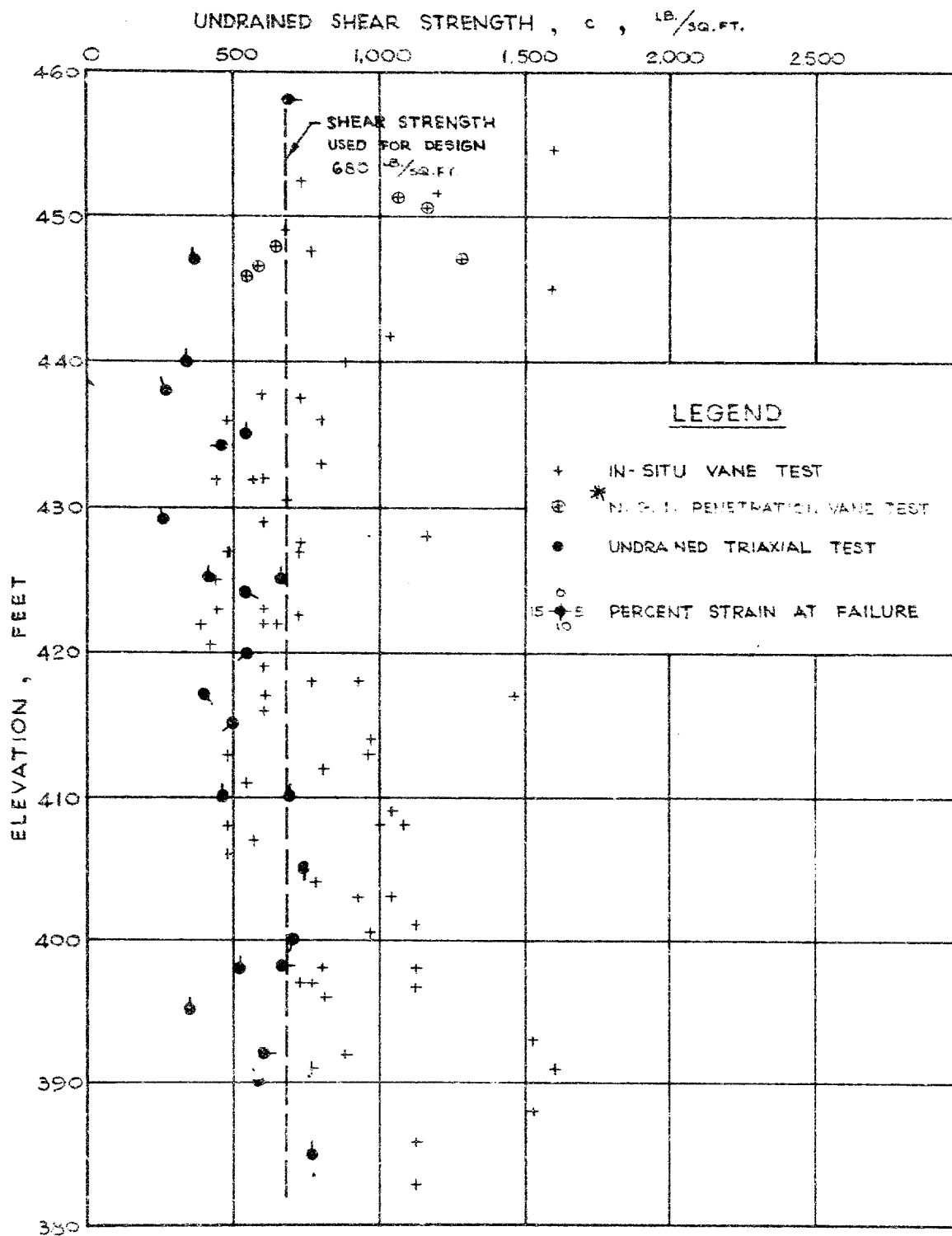
- + IN SITU VANE TEST
- ⊕ N.G.I. * PENETRATION VANE TEST
- UNCONFINED COMPRESSION TEST
- o UNDRAINED TRIAXIAL TEST
- 15 0 5 10 PERCENT STRAIN AT FAILURE

* NORWEGIAN GEOTECHNICAL
INSTITUTE

GOLDER & ASSOCIATES

VARIATION OF UNDRAINED SHEAR STRENGTH WITH ELEVATION SILTY CLAY STRATUM

FIGURE 14



* NORWEGIAN GEOTECHNICAL INSTITUTE.

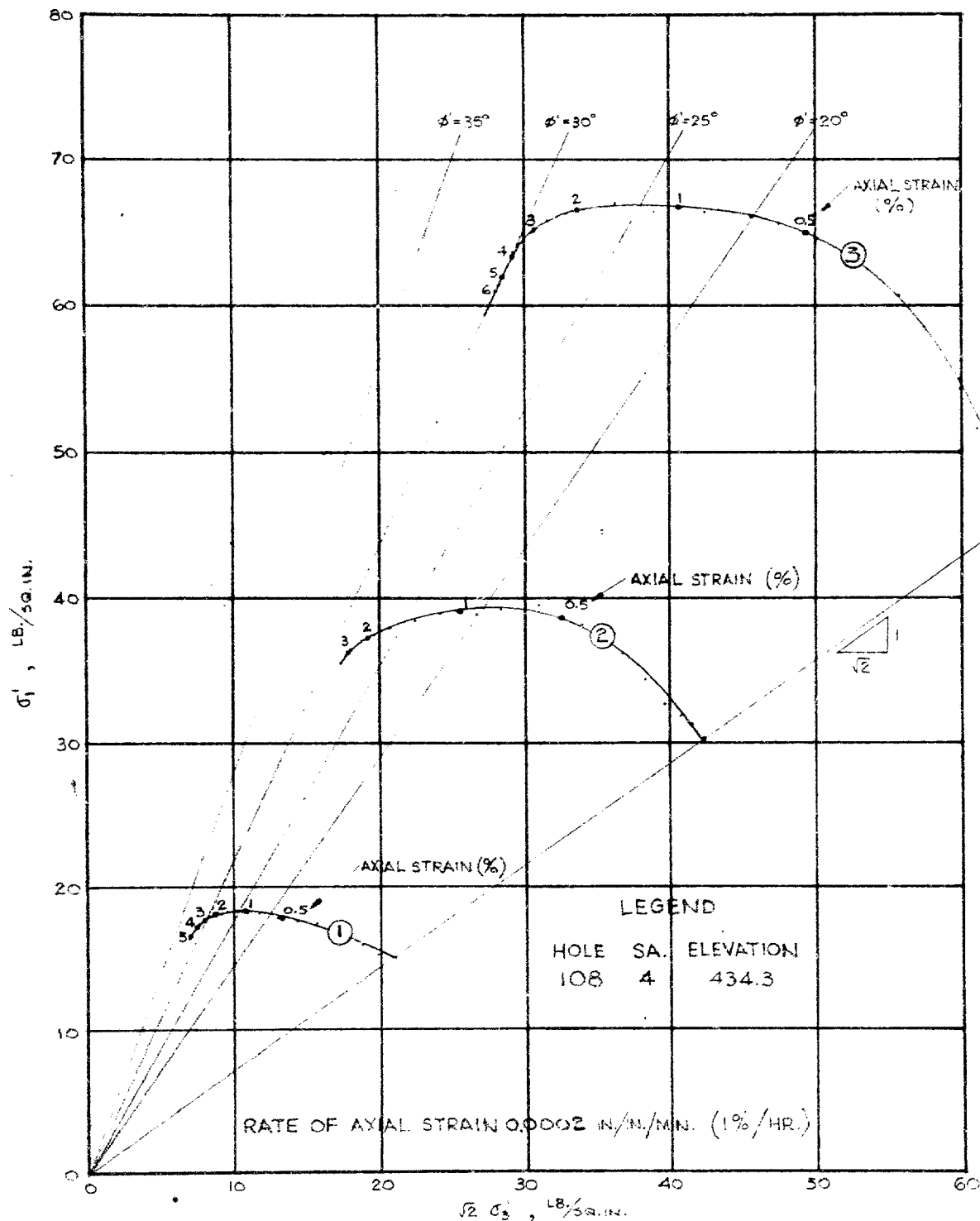
GOLDER & ASSOCIATES

PLOT OF STRESS PATHS IN CONSOLIDATED UNDRAINED TRIAXIAL TESTS SILTY CLAY STRATUM

FIGURE 16

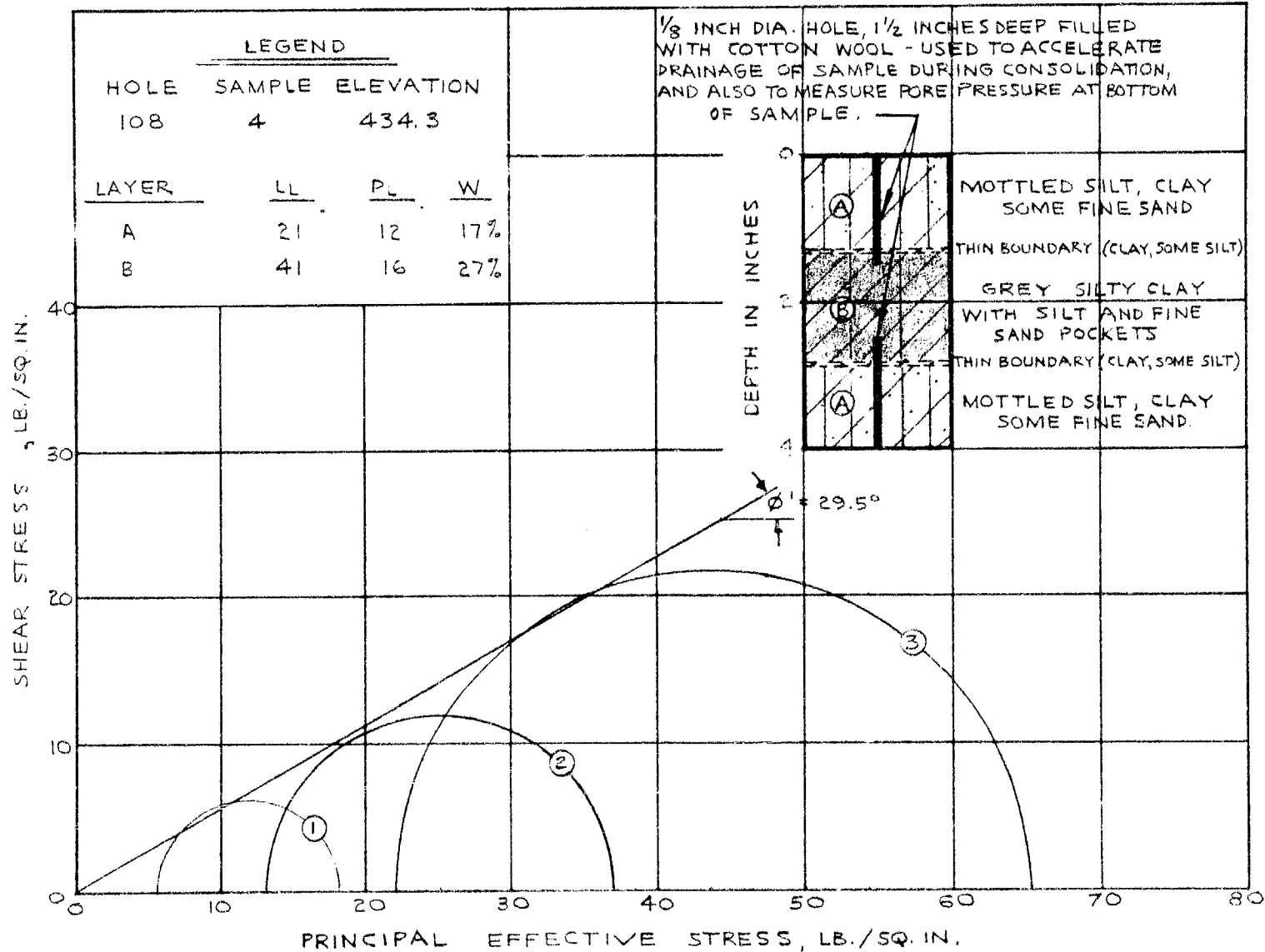
REFERENCE : "THE SHEAR STRENGTH OF SATURATED REMOULDED CLAYS"
BY D.J. HENKEL, A.S.C.E. RESEARCH CONFERENCE ON SHEAR
STRENGTH OF COHESIVE SOILS, BOULDER, COLORADO, 1960.

NOTE : FOR DETAILS OF SAMPLE
SEE FIGURE 15.



GOLDER & ASSOCIATES

NOTE: MOHR'S CIRCLES PLOTTED AT
MAXIMUM DEVIATOR STRESS.



GOLDER & ASSOCIATES

CONSOLIDATED UNDRAINED TRIAXIAL TESTS
WITH PORE PRESSURE MEASUREMENTS
MOHR'S CIRCLES
SILTY CLAY STRATUM

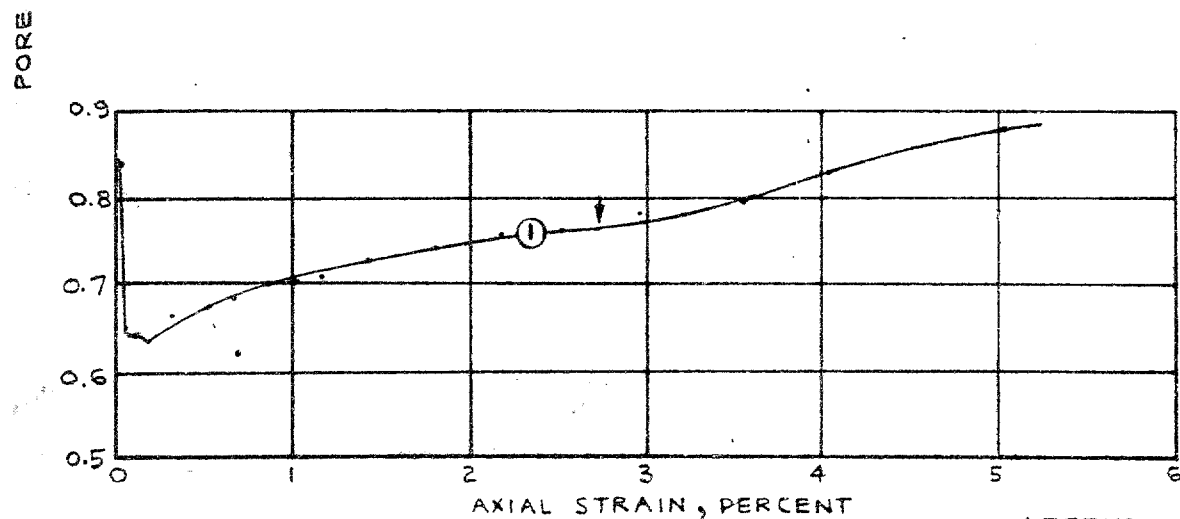
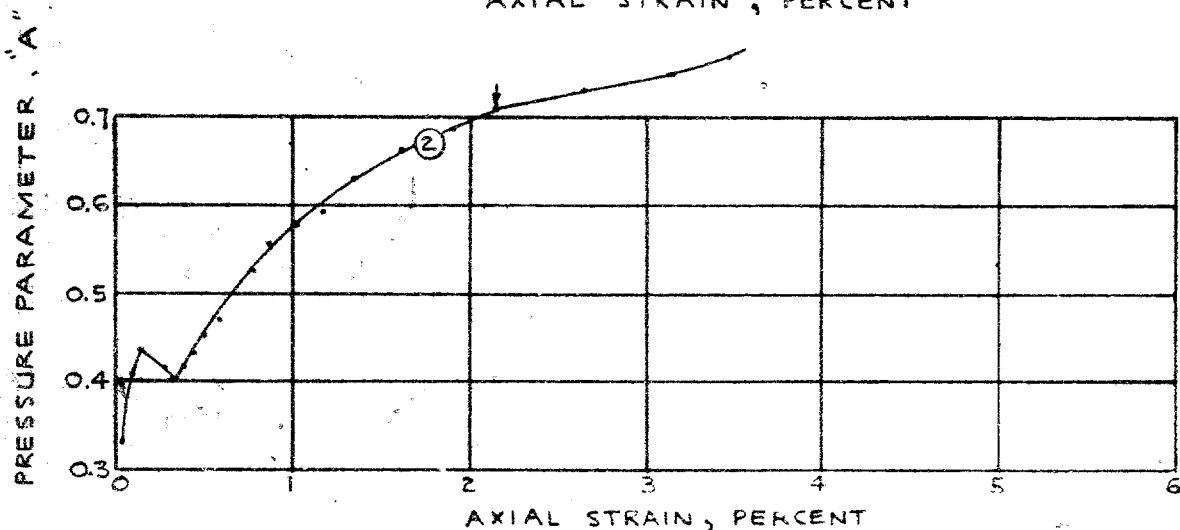
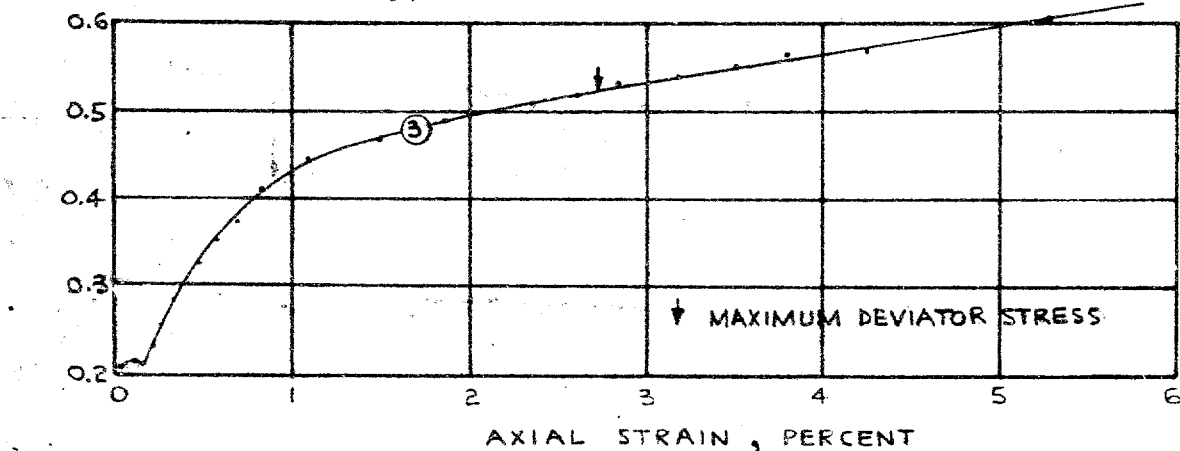
FIGURE 15

CONSOLIDATED UNDRAINED TRIAXIAL TESTS
WITH PORE PRESSURE MEASUREMENTS
PLOTS OF PORE PRESSURE PARAMETER, "A" VS STRAIN
SILTY CLAY STRATUM

FIGURE 17

NOTE: FOR DETAILS OF SAMPLE
SEE FIGURE 15.

RATE OF AXIAL STRAIN 0.0002 IN./IN./MIN.
OR 1% / HR.

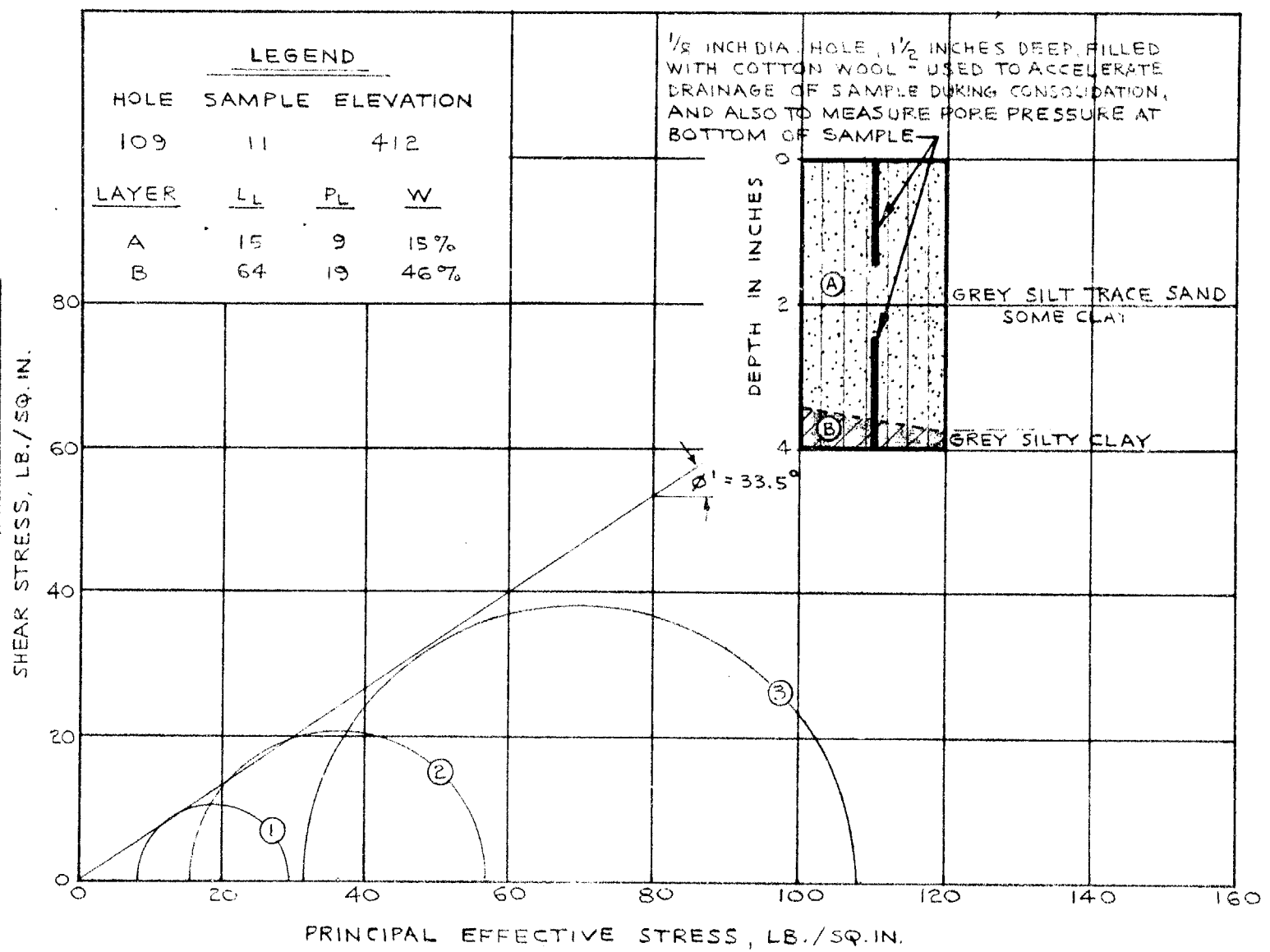


GOLDER & ASSOCIATES

LEGEND
HOLE SAMPLE ELEVATION
108 4 434.3

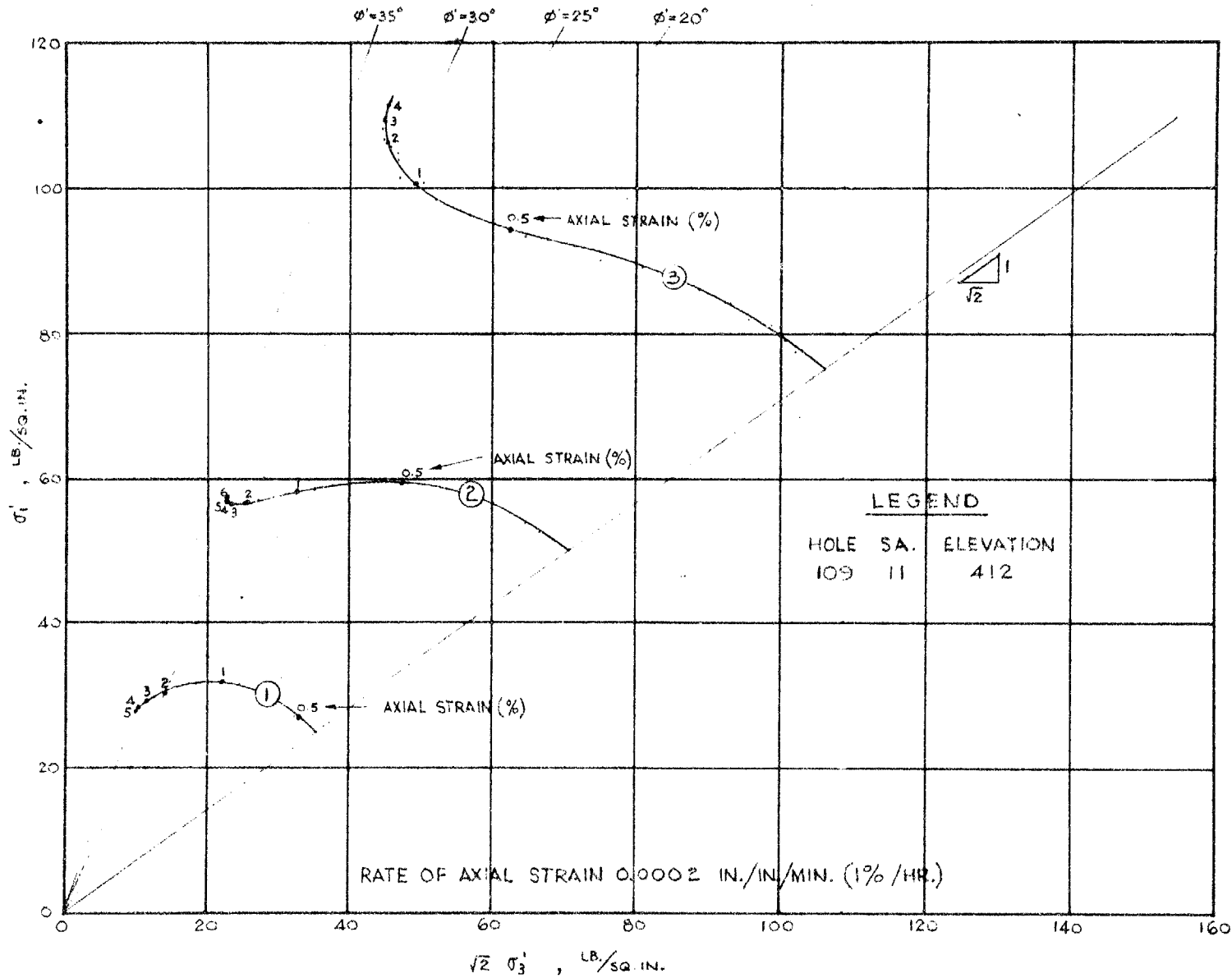
NOTE: MOHR'S CIRCLES PLOTTED AT
MAXIMUM DEVIATOR STRESS.

GOLDER & ASSOCIATES



CONSOLIDATED UNPAINED TRIAXIAL TESTS
WITH PORE PRESSURE MEASUREMENTS
MOHR'S CIRCLES
SILTY CLAY STRATUM

REFERENCE : "THE SHEAR STRENGTH OF SATURATED REMOULDED CLAYS"
BY D.J.HENKEL , A.S.C.E. RESEARCH CONFERENCE ON SHEAR
STRENGTH OF COHESIVE SOILS, BOULDER, COLORADO, 1960.



NOTE: FOR DETAILS OF SAMPLE
SEE FIGURE 18.

PLOT OF STRESS PATHS IN
 CONSOLIDATED UNDRAINED TRIAXIAL TESTS
 SILTY CLAY STRATUM

FIGURE 19

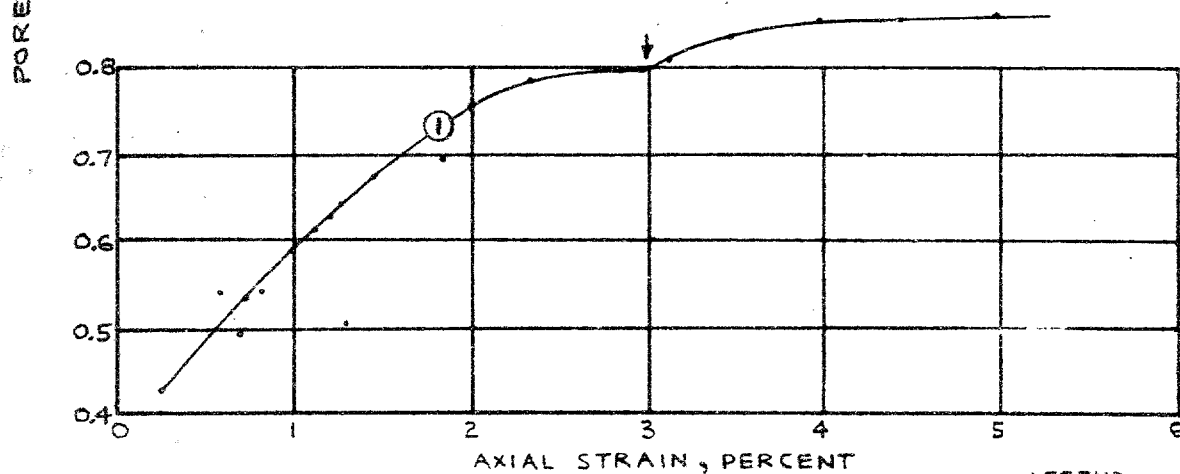
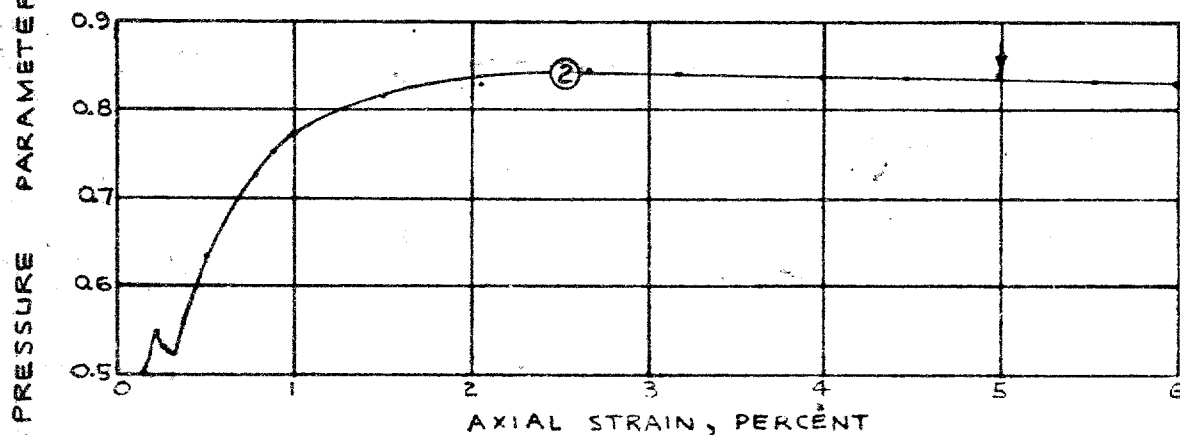
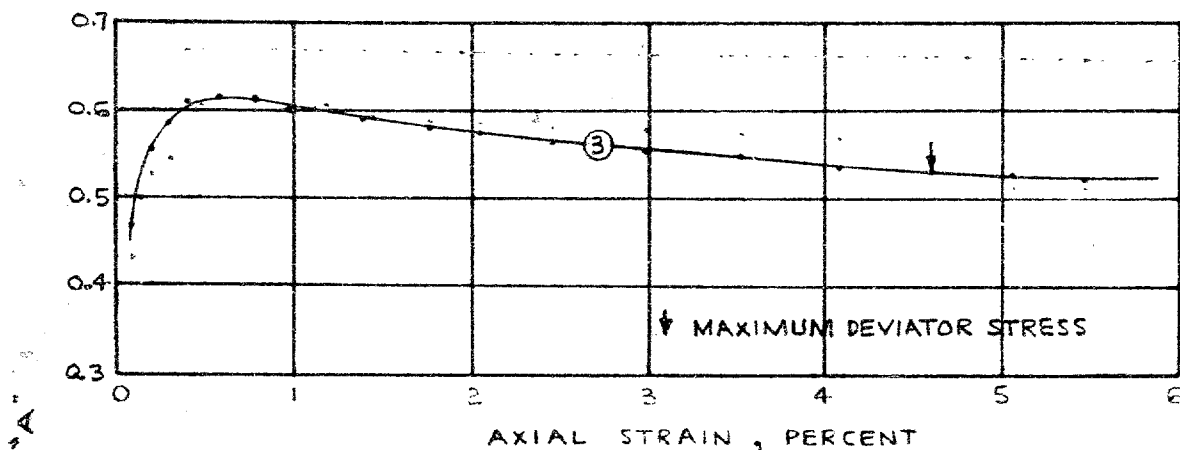
GOLDER & ASSOCIATES

CONSOLIDATED UNDRAINED TRIAXIAL TESTS WITH PORE PRESSURE MEASUREMENTS PLOTS OF PORE PRESSURE PARAMETER, "A" VS STRAIN SILTY CLAY STRATUM

FIGURE 20

NOTE: FOR DETAILS OF SAMPLE
SEE FIGURE 18.

RATE OF AXIAL STRAIN 0.0002 IN./IN./MIN.
OR 1% /HR.

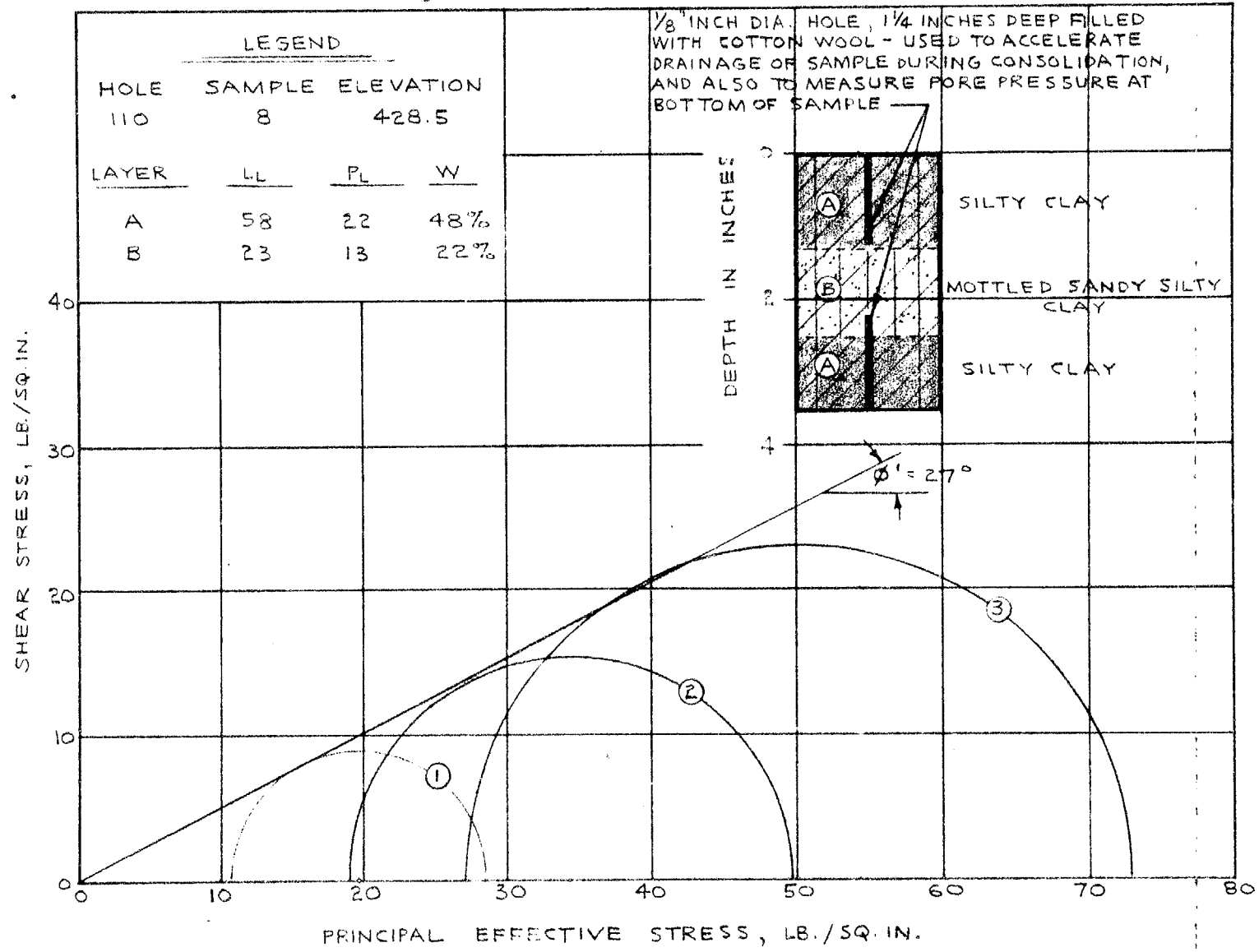


GOLDER & ASSOCIATES

LEGEND
HOLE SAMPLE ELEVATION
109 11 412.0

NOTE: MOHR'S CIRCLES PLOTTED AT
MAXIMUM DEVIATOR STRESS.

GOLDER & ASSOCIATES

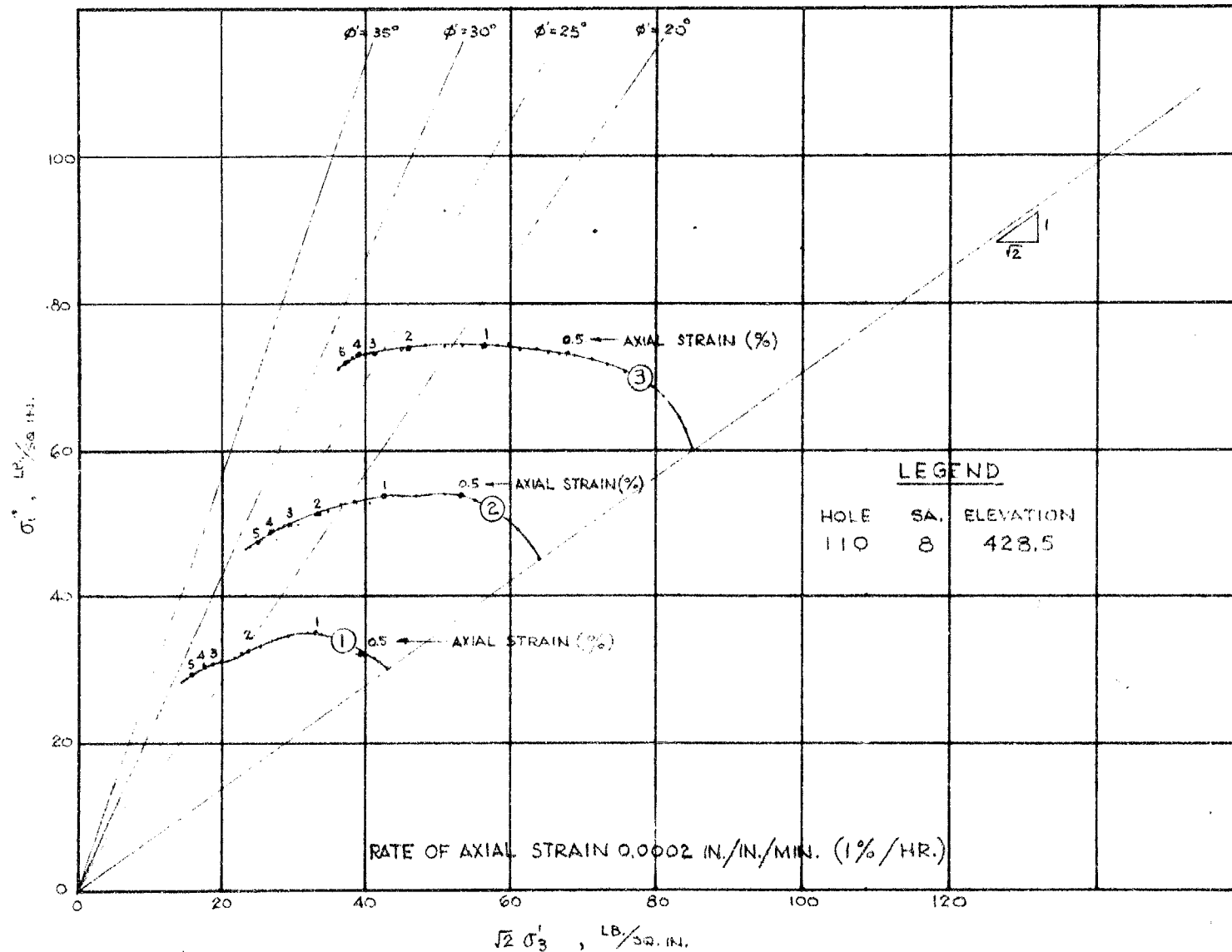


CONSOLIDATED UNDRAINED TRIAXIAL TESTS
WITH PORE PRESSURE MEASUREMENTS
MOHR'S CIRCLES
SILTY CLAY STRATUM

FIGURE 21.

REFERENCE : "THE SHEAR STRENGTH OF SATURATED REMOULDED CLAYS"
BY D.J. HENKEL, A.S.C.E. RESEARCH CONFERENCE ON SHEAR
STRENGTH OF COHESIVE SOILS, BOULDER, COLORADO, 1960.

GOLDER & ASSOCIATES



PLOT OF STRESS PATHS IN
CONSOLIDATED UNDRAINED TRIAXIAL TESTS
SILTY CLAY STRATUM

FIGURE 22

NOTE: FOR DETAILS OF SAMPLE
SEE FIGURE 21.

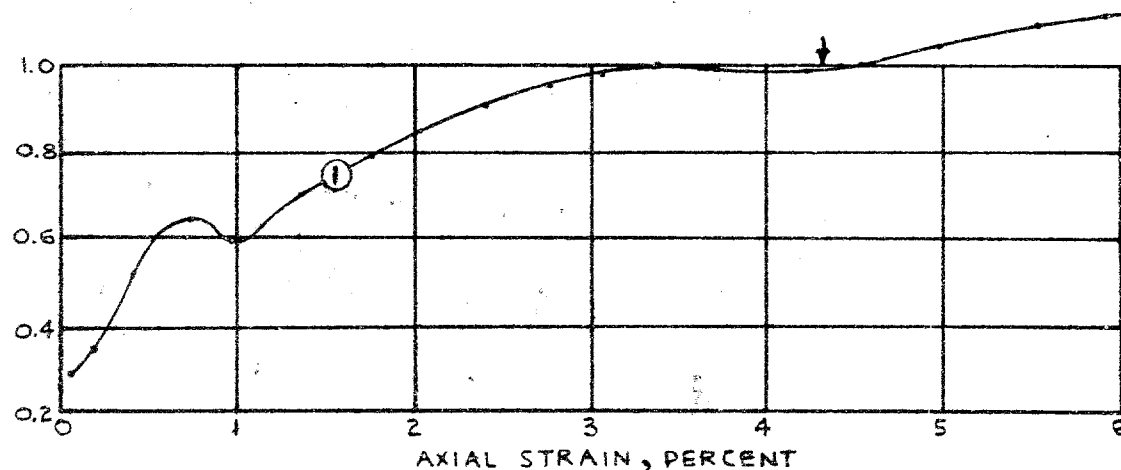
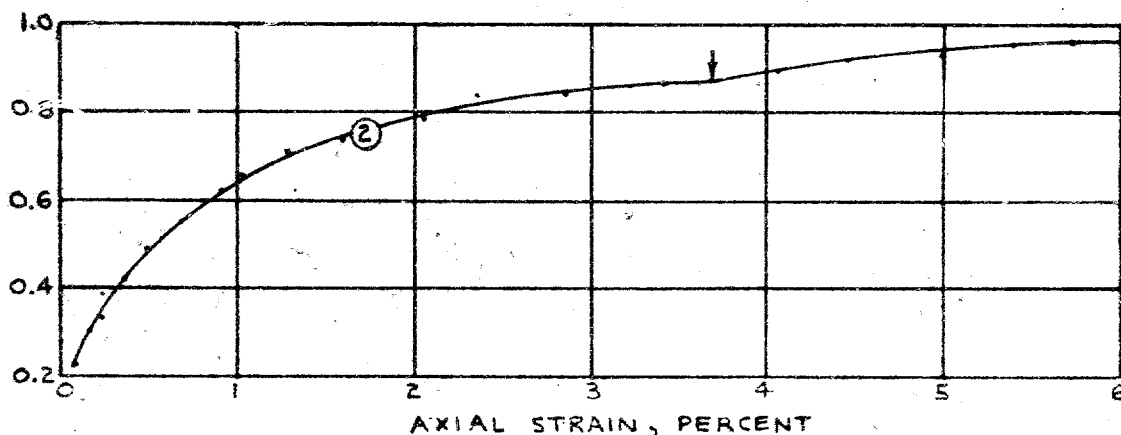
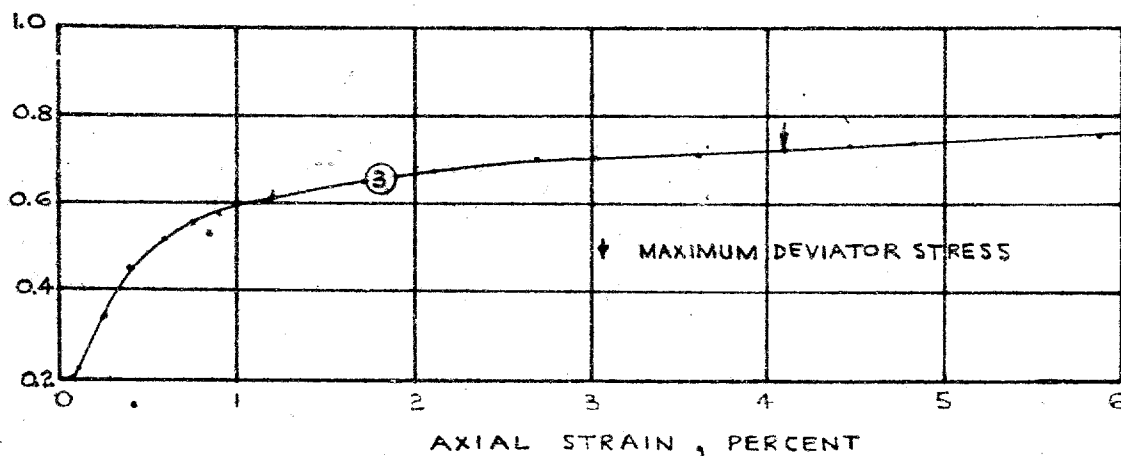
CONSOLIDATED UNDRAINED TRIAXIAL TESTS
WITH PORE PRESSURE MEASUREMENTS
PLOTS OF PORE PRESSURE PARAMETER, "A" VS STRAIN
SILTY CLAY STRATUM

FIGURE 23

RATE OF AXIAL STRAIN 0.0002 IN./IN./MIN.
OR 1% / HR.

NOTE: FOR DETAILS OF SAMPLE
SEE FIGURE 21.

PORE PRESSURE PARAMETER, "A"

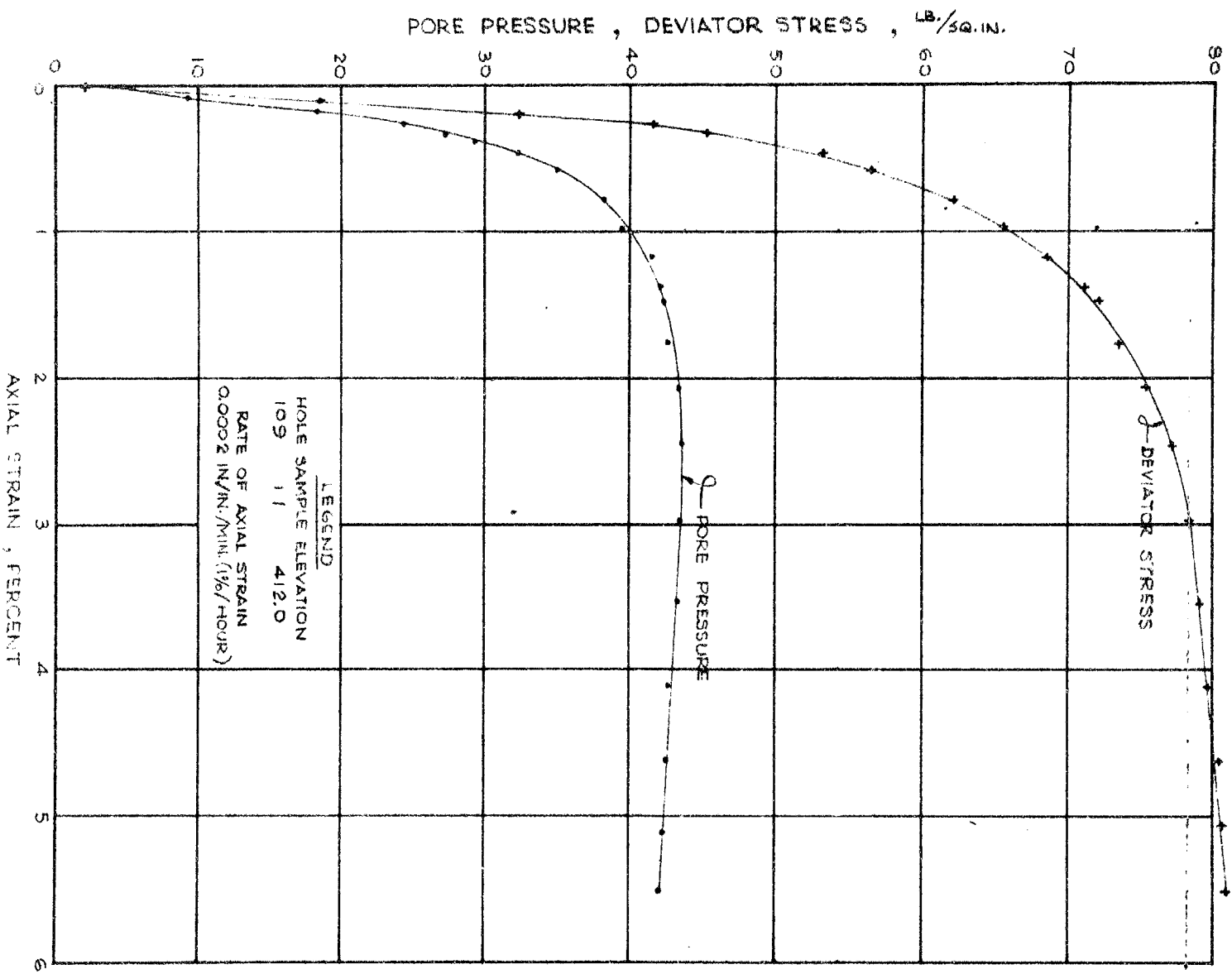


GOLDER & ASSOCIATES

LEGEND
HOLE SAMPLE ELEVATION
110 8 428.5

CONSOLIDATED UNDRAINED TRIAXIAL TESTS
TYPICAL STRESS-STRAIN & PORE PRESSURE-STRAIN CURVES
SILTY CLAY STRATUM

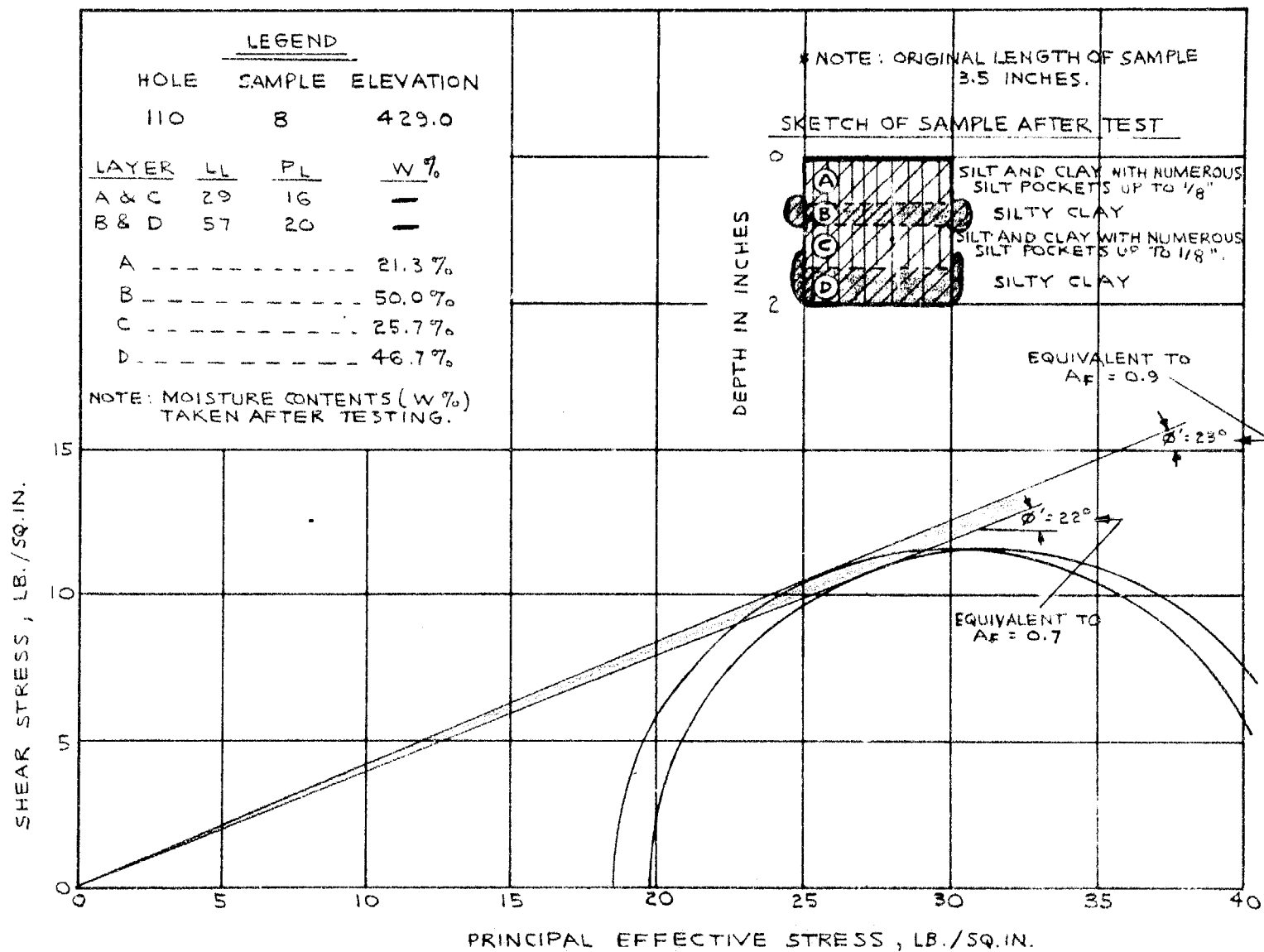
FIGURE 24



INCREMENTAL LOADING DRAINED TEST MOHR'S CIRCLES SILTY CLAY STRATUM

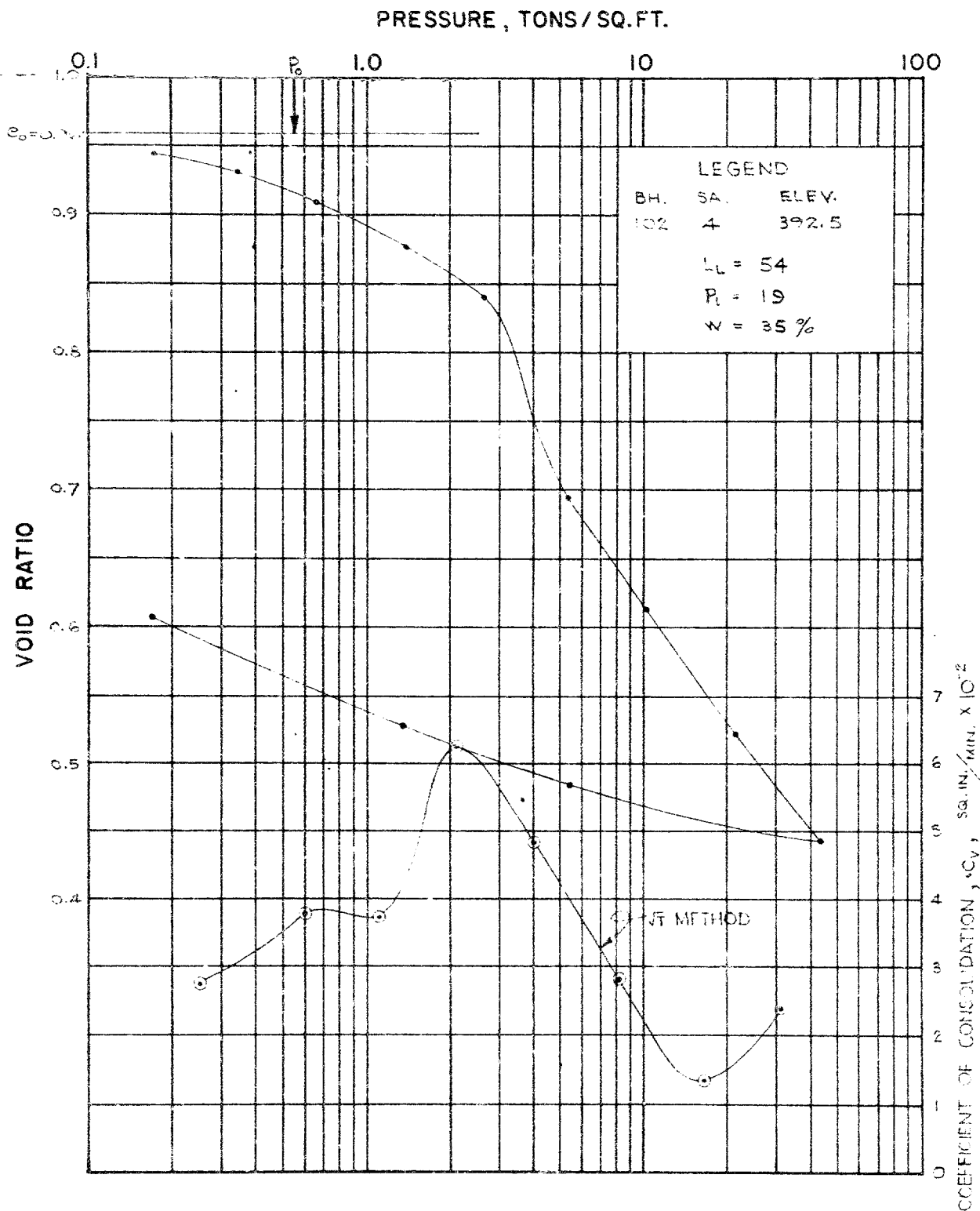
FIGURE 25

GOLDER & ASSOCIATES



VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 26

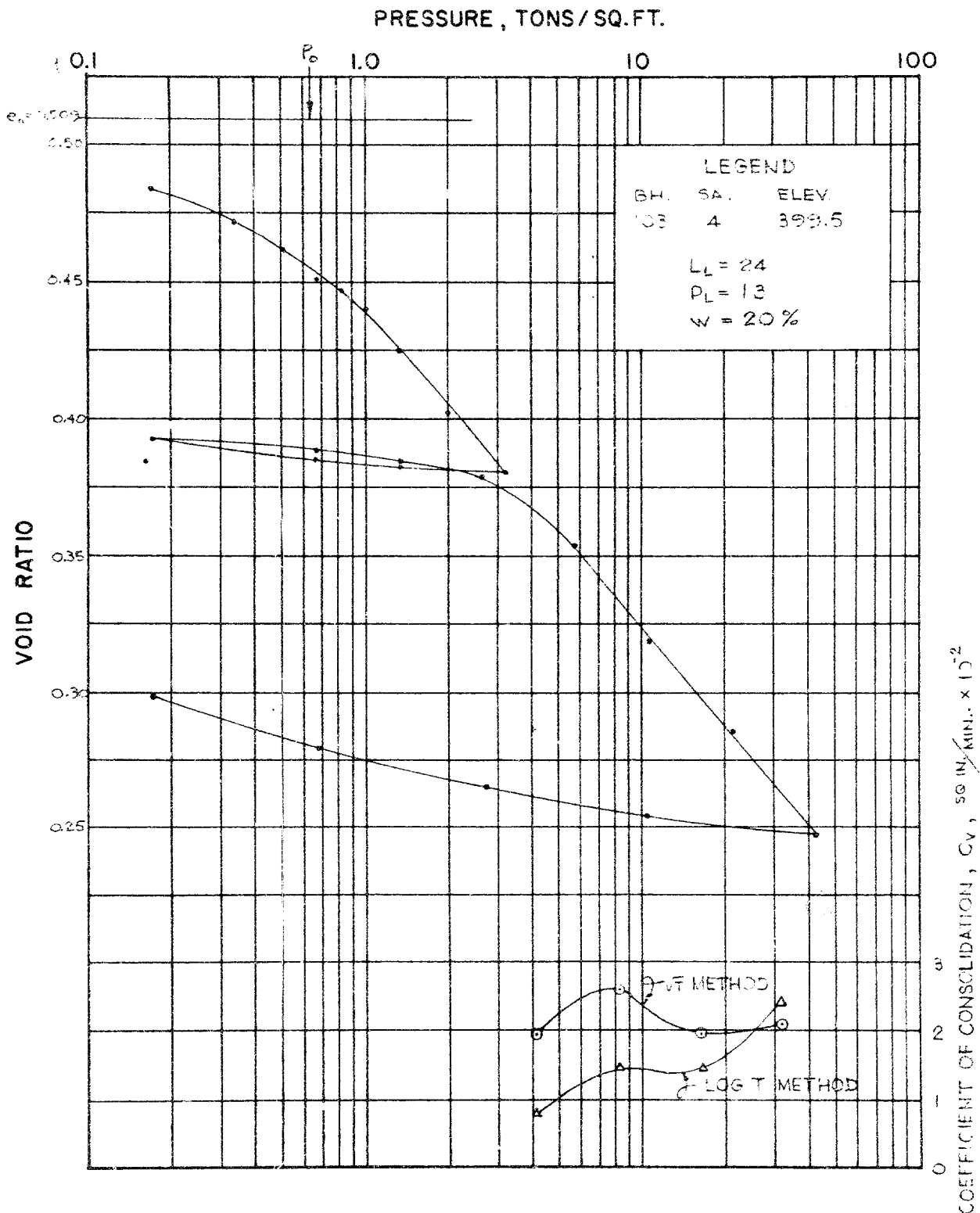


GOLDER & ASSOCIATES

PROJECT No. 62005

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

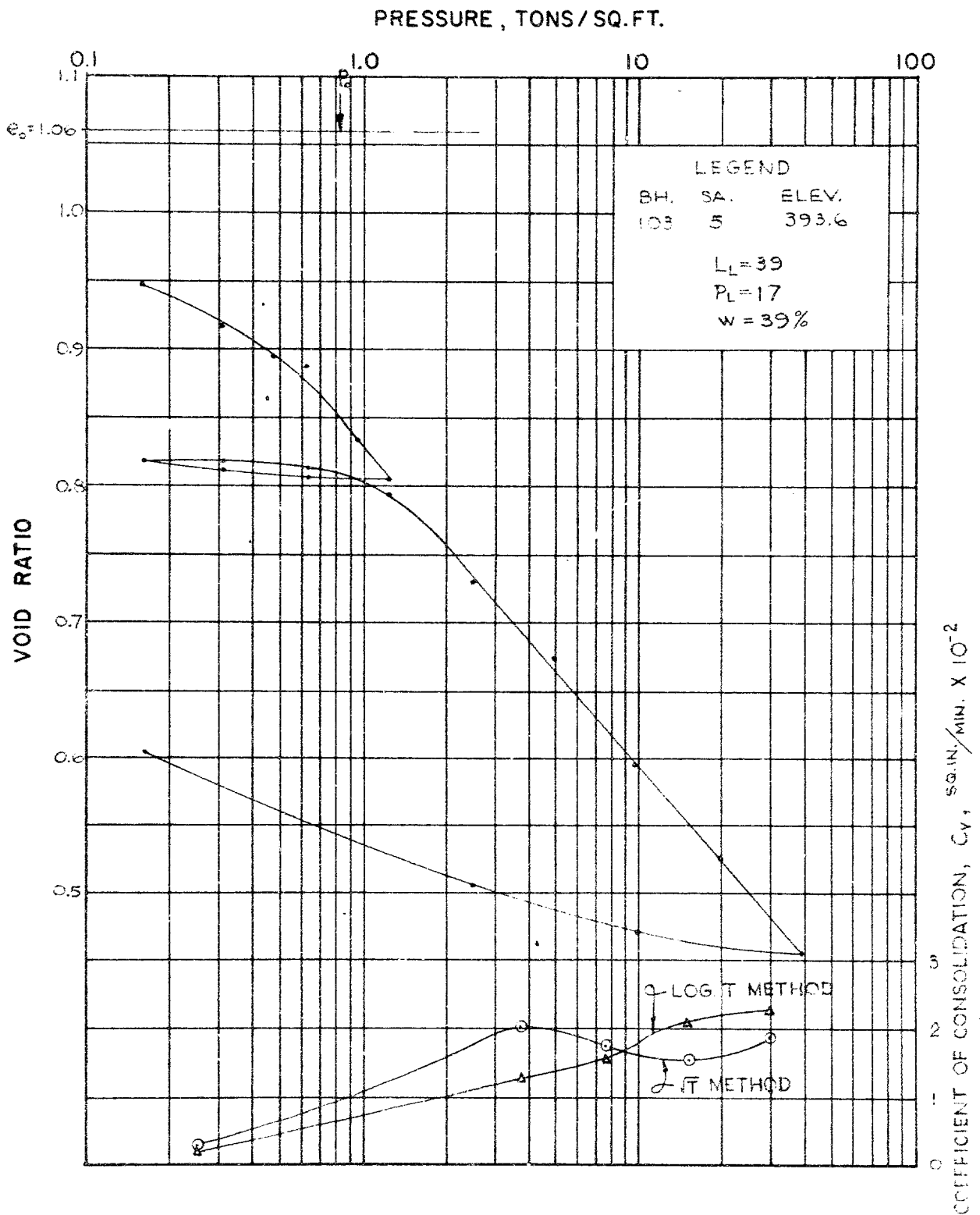
FIGURE 27



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

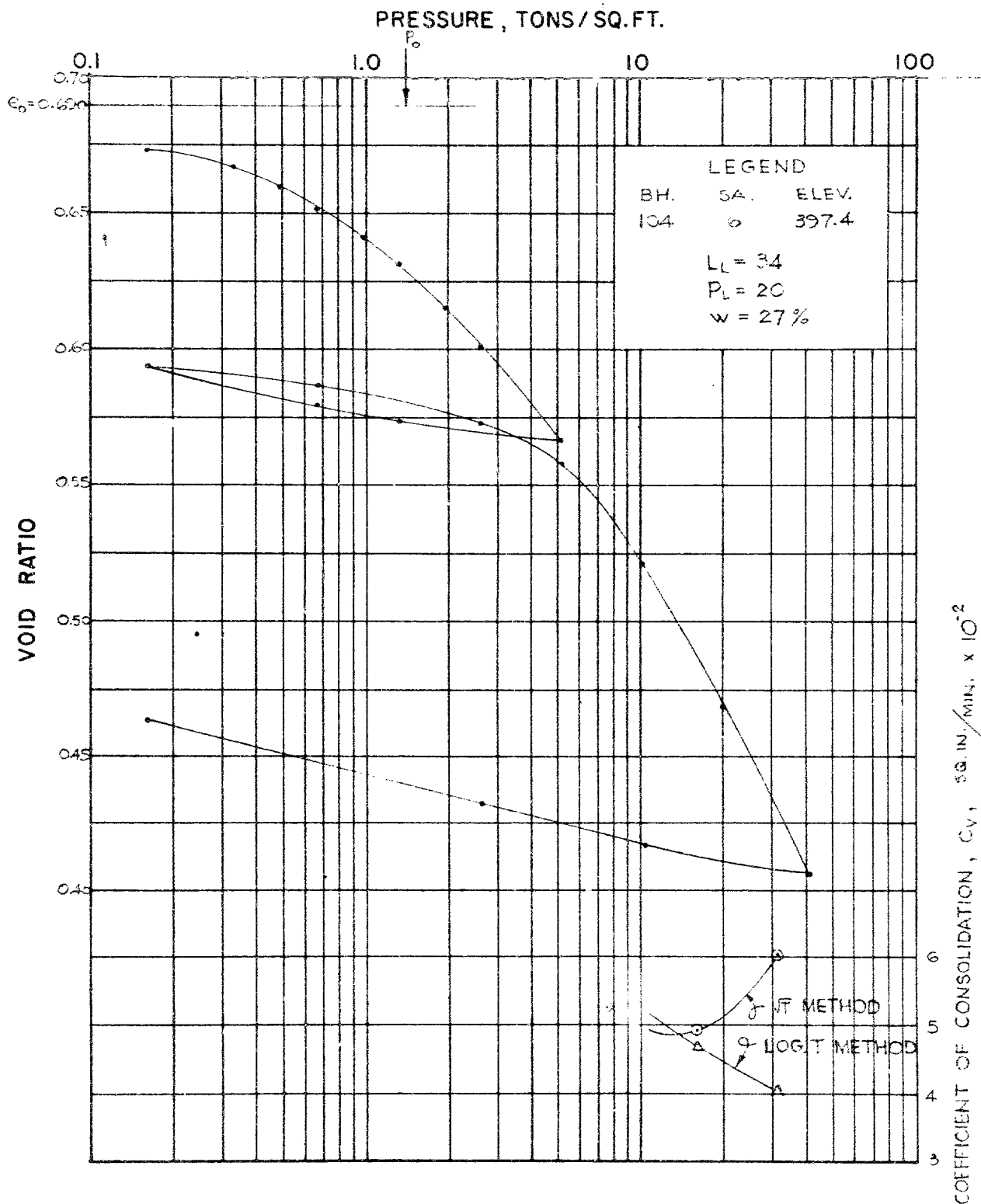
FIGURE 28



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

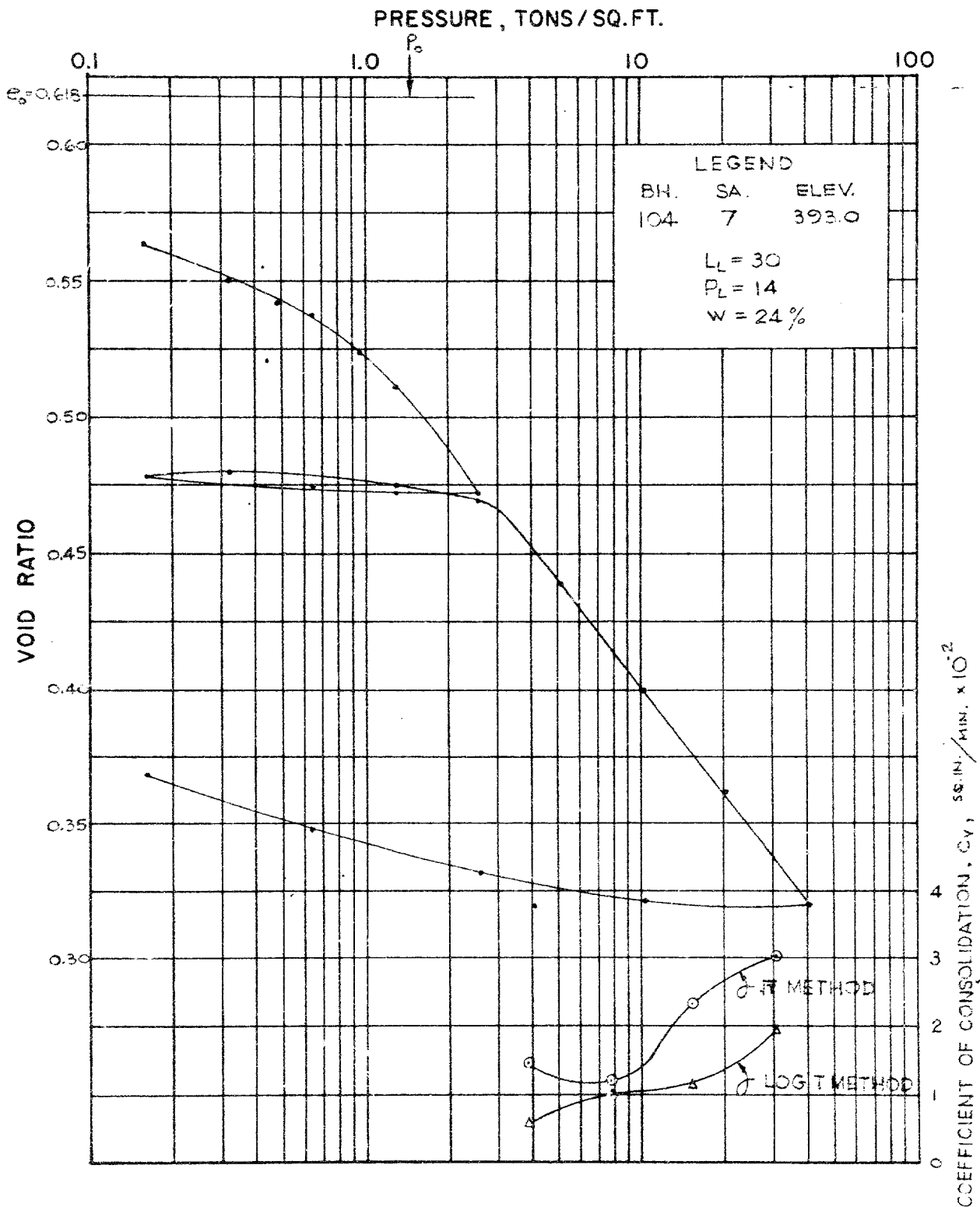
FIGURE 29



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

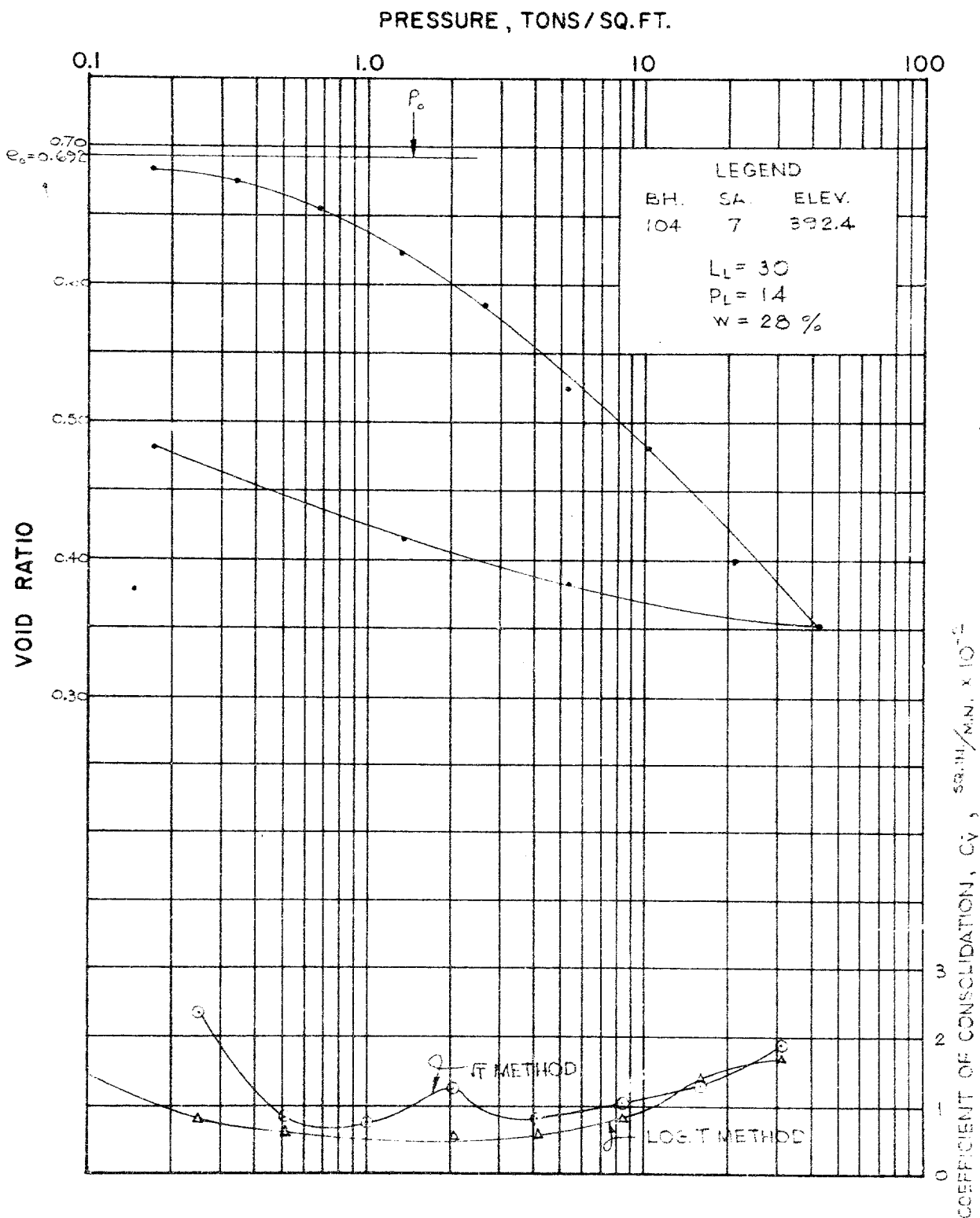
FIGURE 30



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

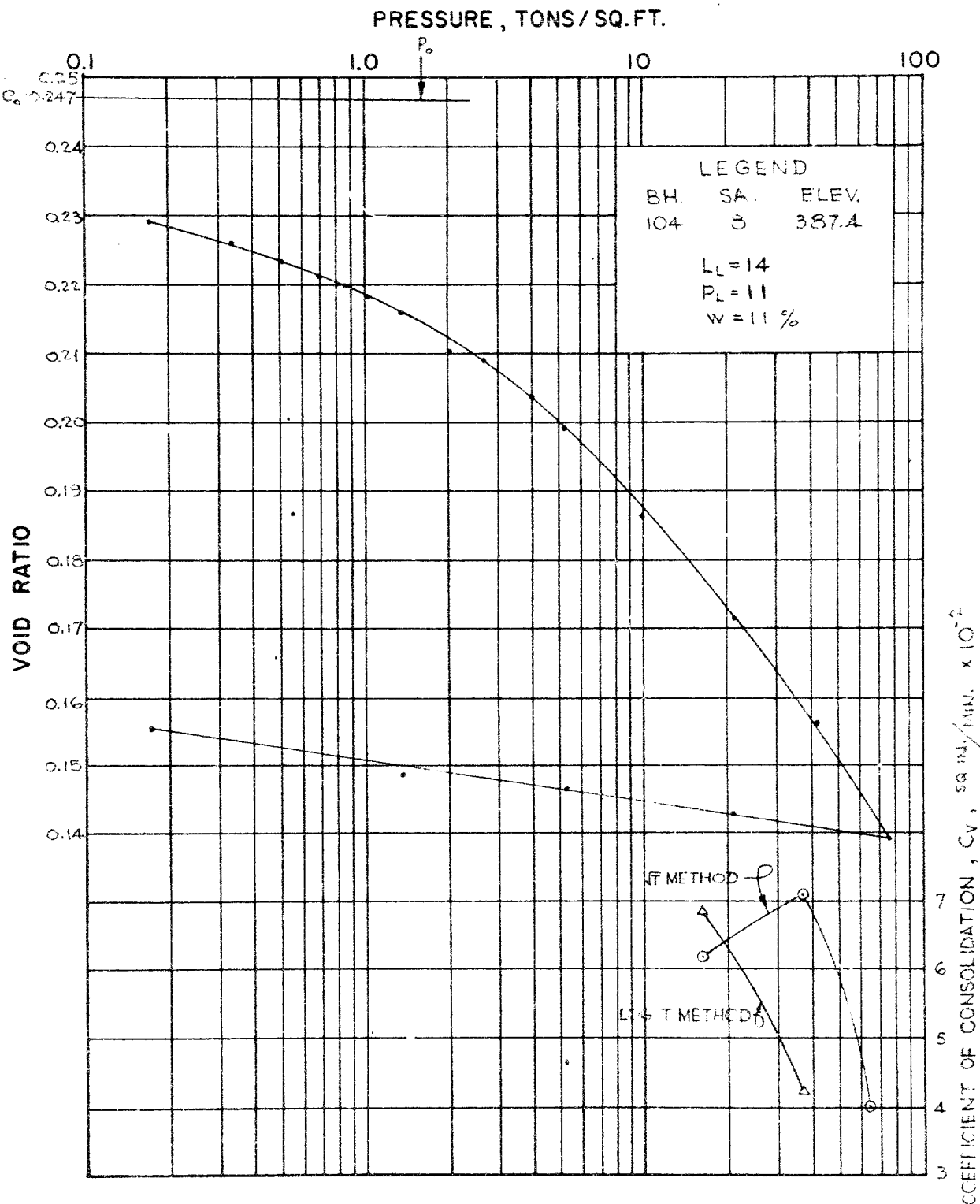
FIGURE 31



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

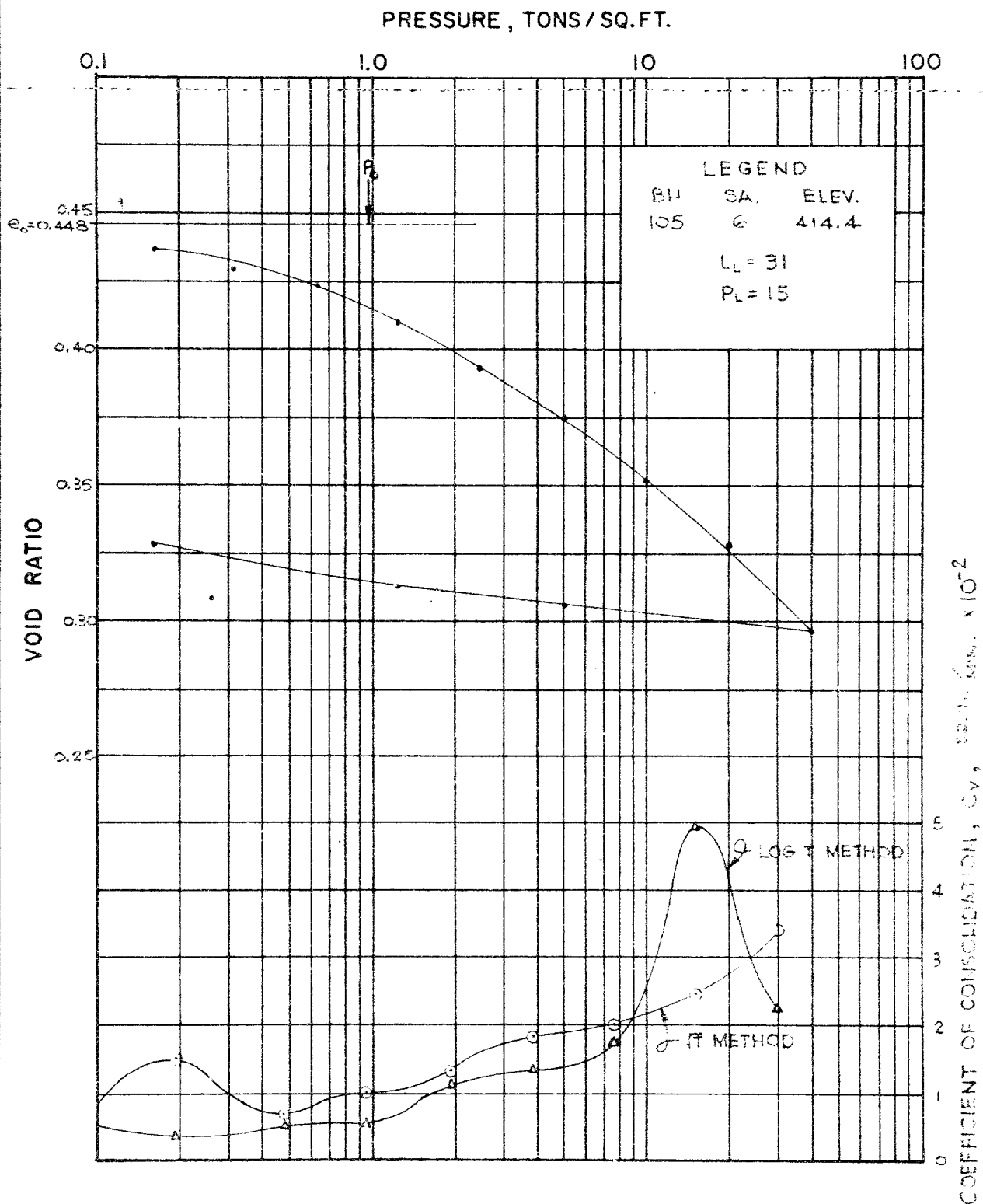
FIGURE 32



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

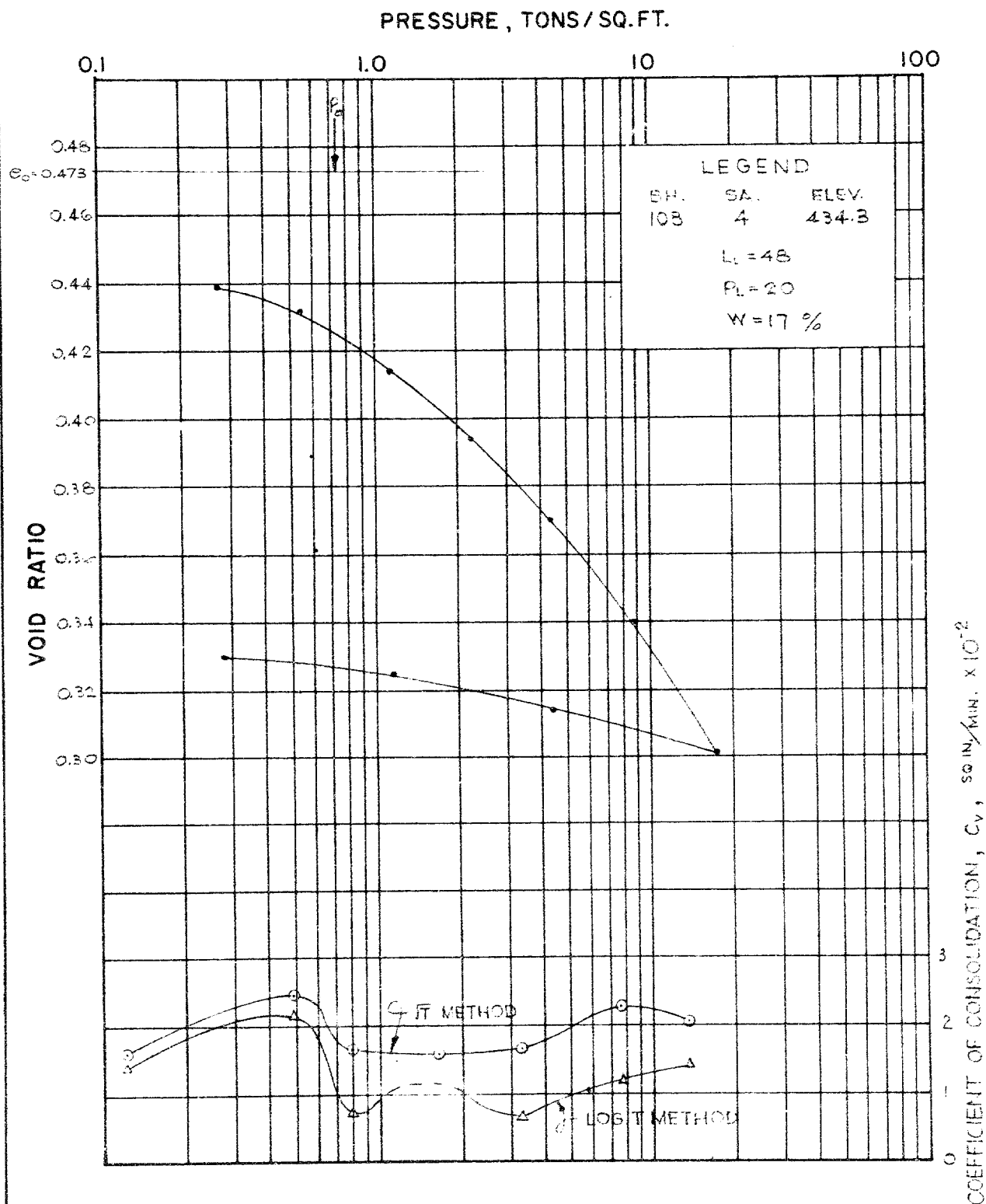
FIGURE 33



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

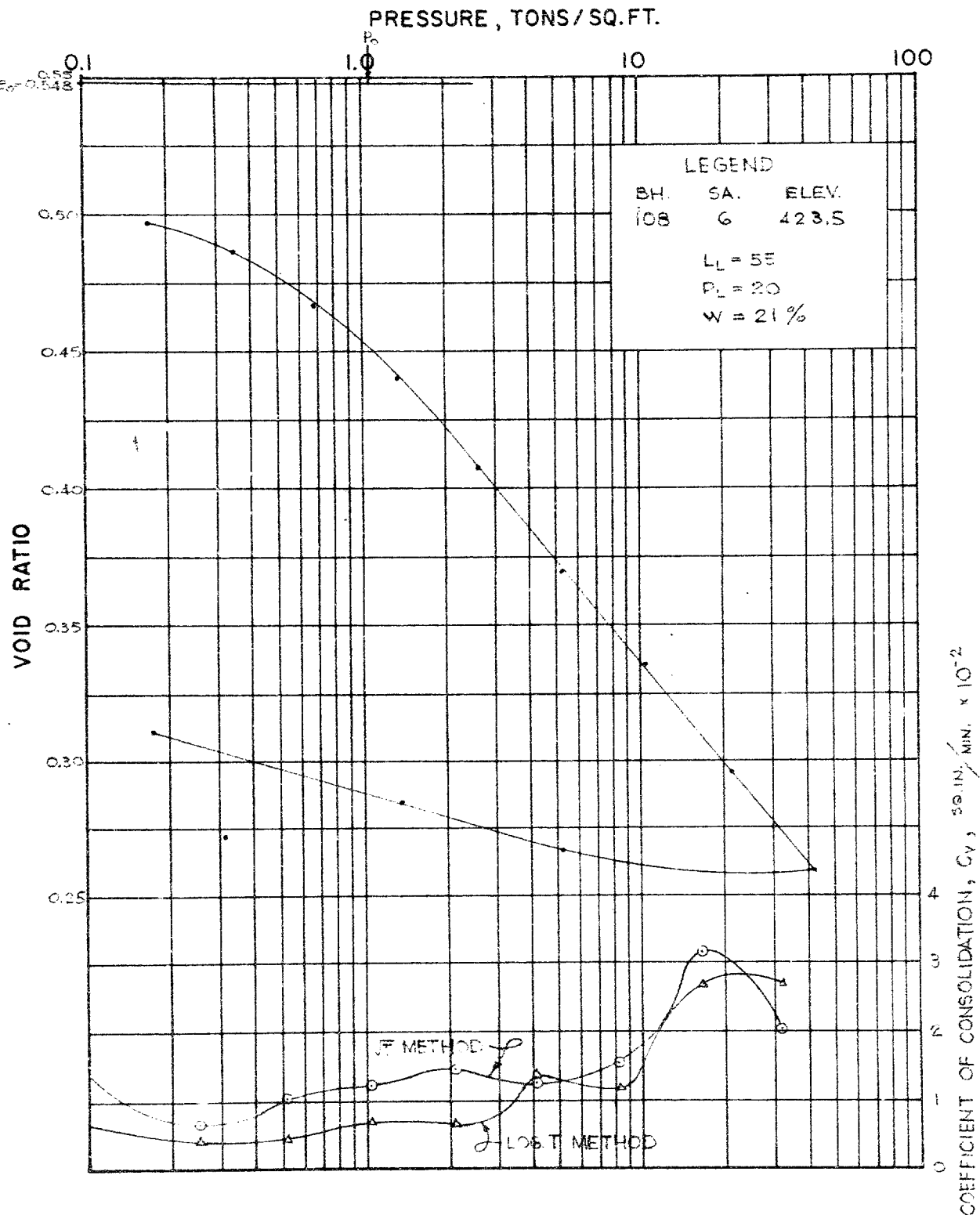
FIGURE 34



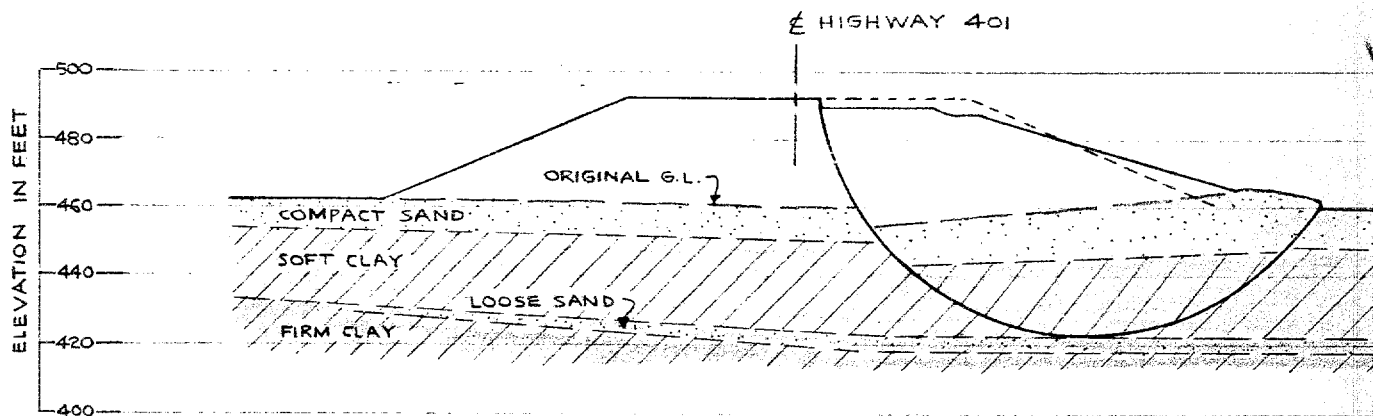
GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

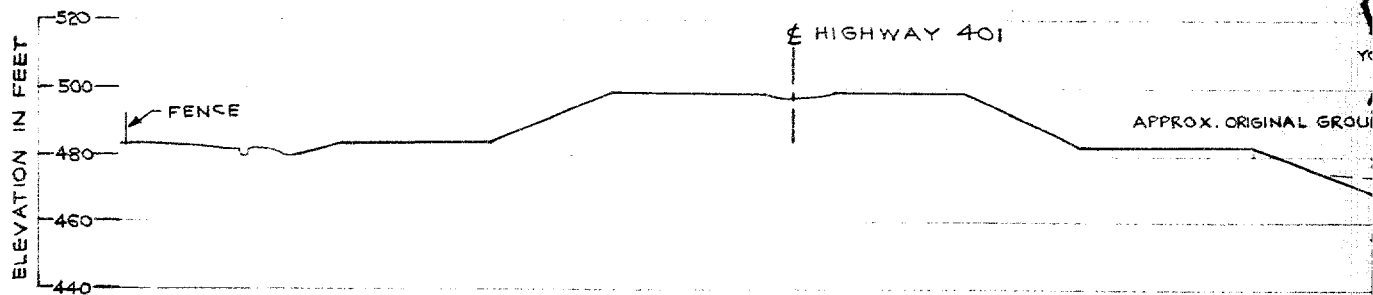
FIGURE 35



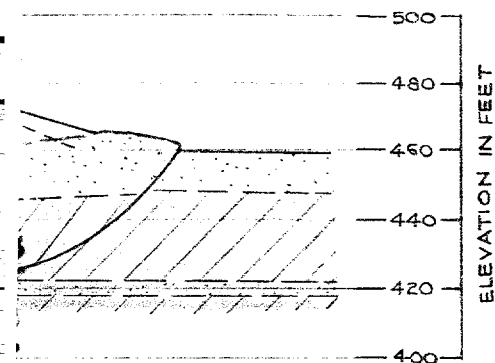
GOLDER & ASSOCIATES



SECTION AT STATION 131+00 AS FIRST CONSTRUCTED
(SHOWING 1953 FAILURE)



EXISTING SECTION AT STATION 124+65
(WEST END OF YORK FARMS BUILDING)



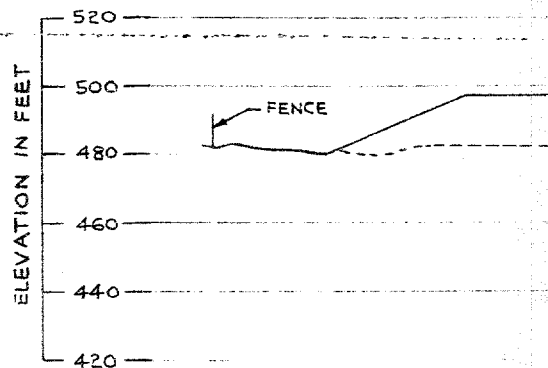
1953 FAILURE
FOR FAILURE AS AT LEFT,

HEIGHT OF FILL = 32'
DEPTH OF CIRCLE = 37'
DEPTH FACTOR $n_d = 2.2$
SLOPE $2\frac{1}{4}:1 = 24^\circ$
STABILITY FACTOR $N_s = 5.9$
(TERZAGHI & PECK p. 186)

$$\therefore C_u \text{ AT FAILURE} = \frac{\gamma H}{N_s}$$

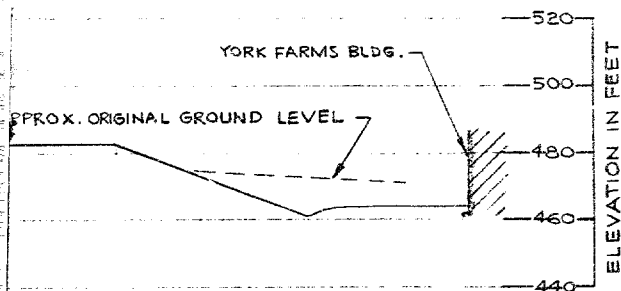
$$= \frac{125 \times 32}{5.9}$$

$$= 680 \text{ LB./SQ. FT.}$$



PR

CONSTRUCTED



EXISTING SLOPE

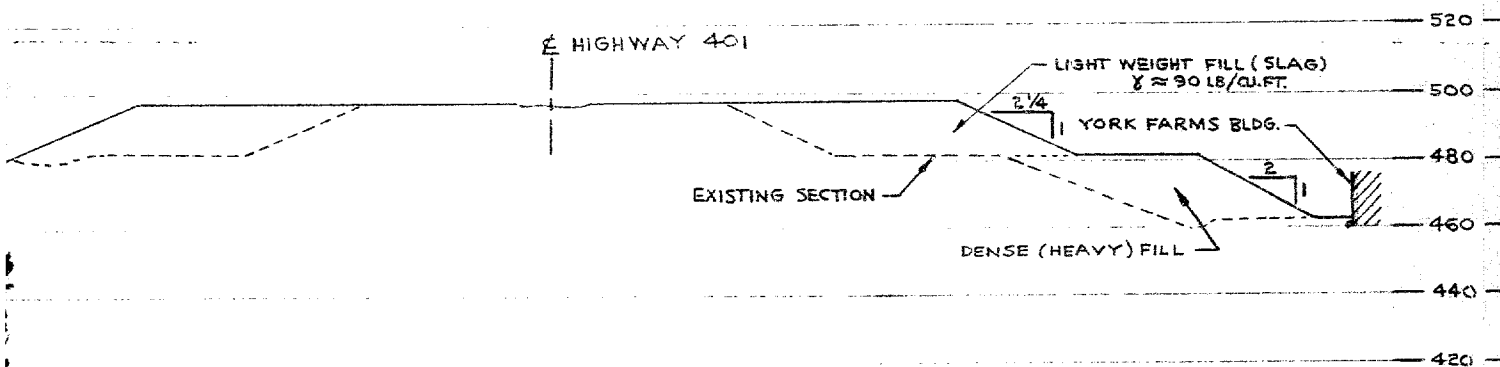
HEIGHT OF FILL = 36'
DEPTH FACTOR $n_d \approx 2.0$
AVERAGE SLOPE $4:1 = 14^\circ$
STABILITY FACTOR $N_s = 6.8$
 C_u REQUIRED FOR STABILITY
 $= \frac{125 \times 36}{6.8} = 663 \text{ LB./SQ. FT.}$
 $\therefore F = \frac{680}{663} = 1.03$

+65
NG)

NOTE: THIS SLOPE IS NOT AS ORIGINALLY CONSTRUCTED.
EXCAVATION TO PROVIDE DRIVEWAY AND PARKING
LOT FOR YORK FARMS HAS INCREASED ITS HEIGHT.

STABILITY OF EMBANKMENT ADJACENT TO YORK FARMS BLDG.

FIGURE 36



PROPOSED SECTION AT STATION 124 + 65 (WEST END OF YORK FARMS BUILDING)

PROPOSED SLOPE

HEIGHT OF FILL 36'

AVERAGE SIDE SLOPE 3 : 1 = 18° 25'

ASSUME FAILURE CIRCLE SAME DEPTH AS BEFORE

DEPTH FACTOR $n_d = 2.0$

STABILITY FACTOR $N_s = 6.4$

C_u REQUIRED FOR STABILITY = $\frac{\gamma H}{N_s} = \frac{125 \times 36}{6.4} = 700 \text{ LB./SQ.FT.}$

USING LIGHT WEIGHT FILL $\gamma = 90$, $\gamma_{\text{AVE.}} \approx 110 \text{ LB./CU.FT.}$

C_u REQUIRED = $\frac{110 \times 36}{6.4} = 620 \text{ LB./SQ.FT.}$

$F = \frac{680}{620} = 1.1$

BERMED AS SHOWN RATHER THAN UNIFORM SLOPE

$F \approx 1.15 \pm$

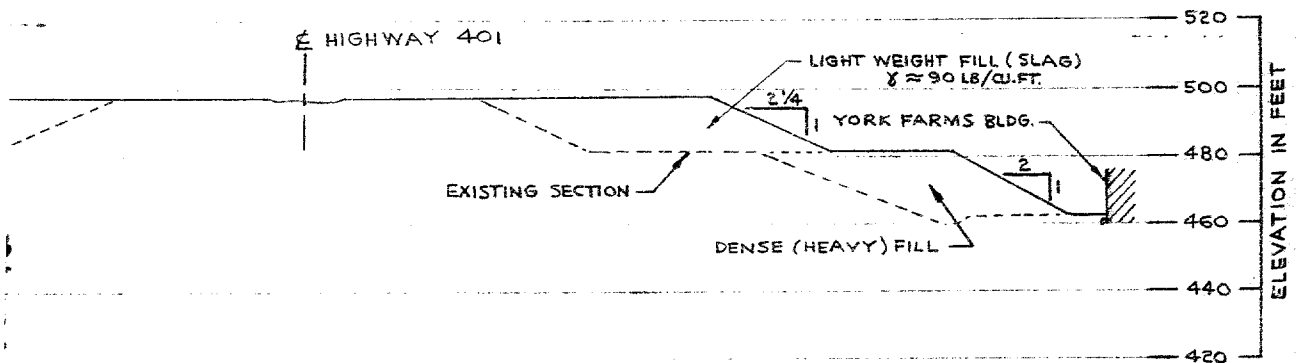
30M14-142
GEOCREG No.

GOLDER & ASSOCIATES

Made
Chkd.
Appd.

STABILITY OF EMBANKMENT ADJACENT TO YORK FARMS BLDG.

FIGURE 36



PROPOSED SLOPE

HEIGHT OF FILL 36'
 AVERAGE SIDE SLOPE 3:1 = $18^\circ 25'$
 ASSUME FAILURE ORCLE SAME DEPTH AS BEFORE
 DEPTH FACTOR $n_d = 2.0$
 STABILITY FACTOR $N_s = 6.4$
 U REQUIRED FOR STABILITY = $\frac{\gamma H}{N_s} = \frac{125 \times 36}{6.4} = 700 \text{ LB./SQ.FT.}$
 USING LIGHT WEIGHT FILL $\gamma = 90$, $\gamma_{\text{AVE.}} \approx 110 \text{ LB./CU.FT.}$
 U REQUIRED = $\frac{110 \times 36}{6.4} = 620 \text{ LB./SQ.FT.}$
 $= \frac{680}{620} = 1.1$
 PERMED AS SHOWN RATHER THAN UNIFORM SLOPE
 $\approx 1.15 \pm$

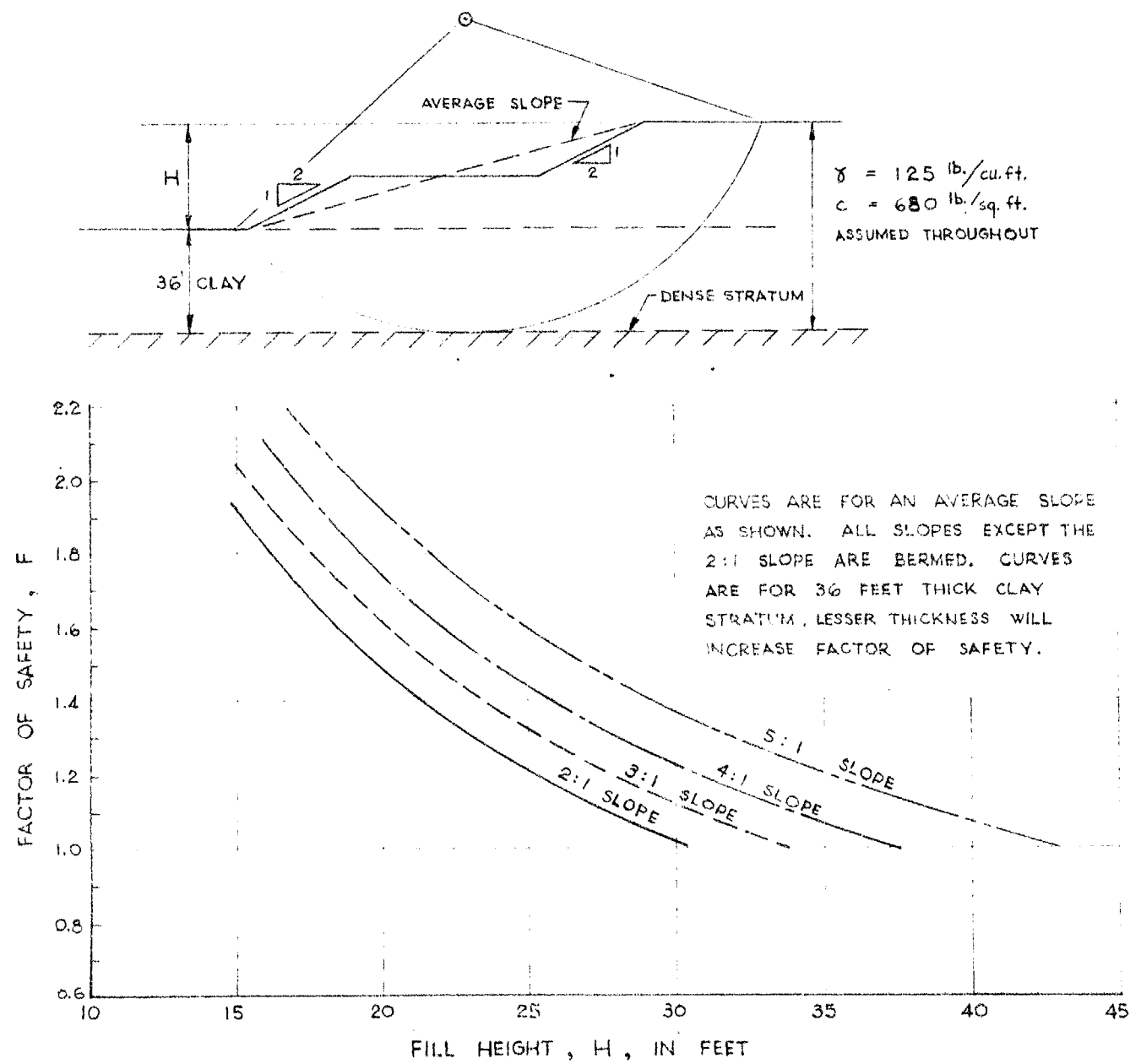
30M14-142
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GOLDER & ASSOCIATES

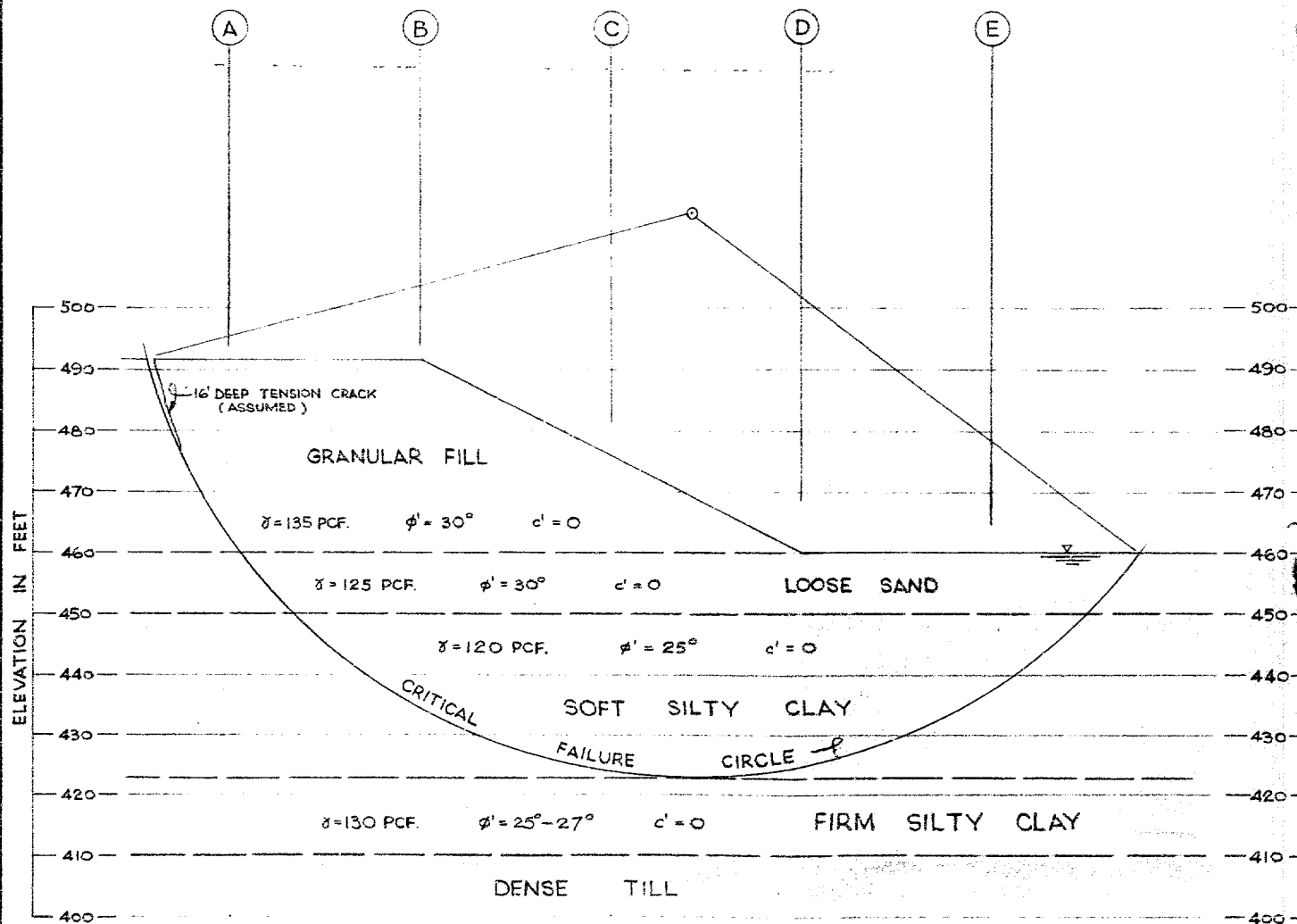
Made *ML*
 Chkd *E.D.L.*
 Appd. *King*

SUMMARY OF STABILITY ANALYSES ,
 $\phi = 0$ CONDITION

FIGURE 37



GOLDER & ASSOCIATES

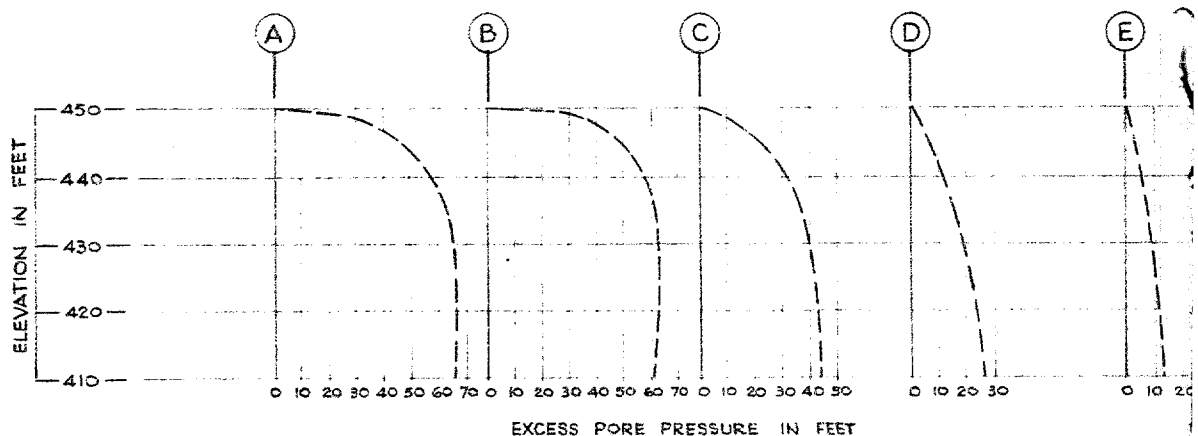
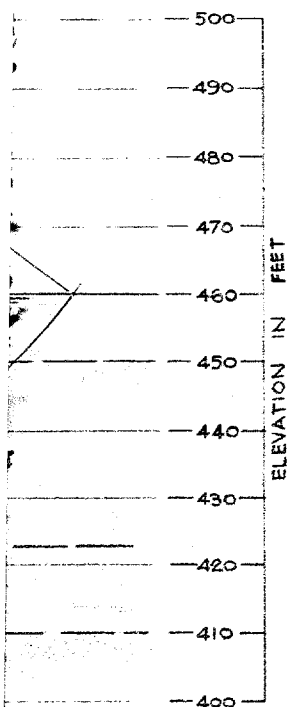
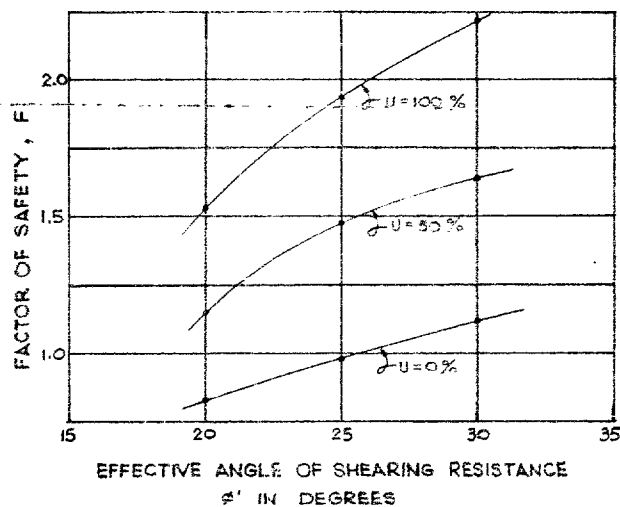


INFERRED SECTION OF 1953 FAILURE

SCALE: 1" TO 20'-0"

Au
B =
A =

EFFECTIVE STRESS ANALYSIS 1953 FAILURE



PORE PRESSURE DISTRIBUTION
IN SOFT TO FIRM CLAY

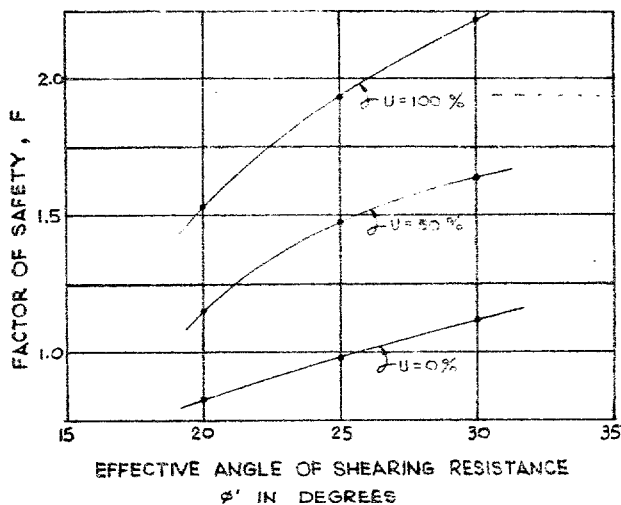
$$\Delta u = B [\Delta \sigma_3 + A (\Delta \sigma_1 - \Delta \sigma_3)]$$

$B = 1$ (SOIL SATURATED)
 $A = 0.8$ (FROM TRIAXIAL TEST)

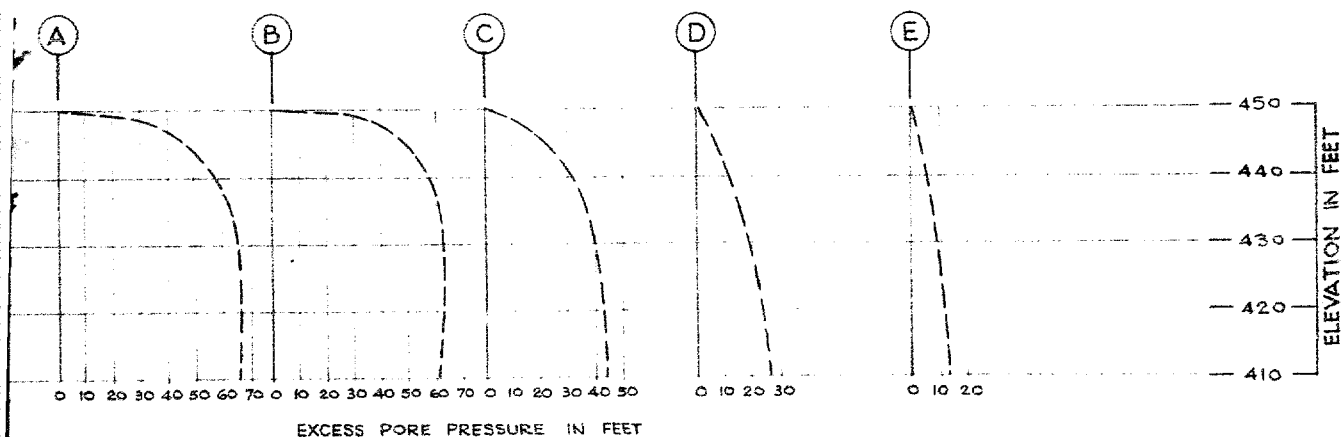
GOLDER & A

EFFECTIVE STRESS ANALYSIS 1953 FAILURE

FIGURE 38



NOTE : U = PERCENT CONSOLIDATION,
CLAY STRATA.



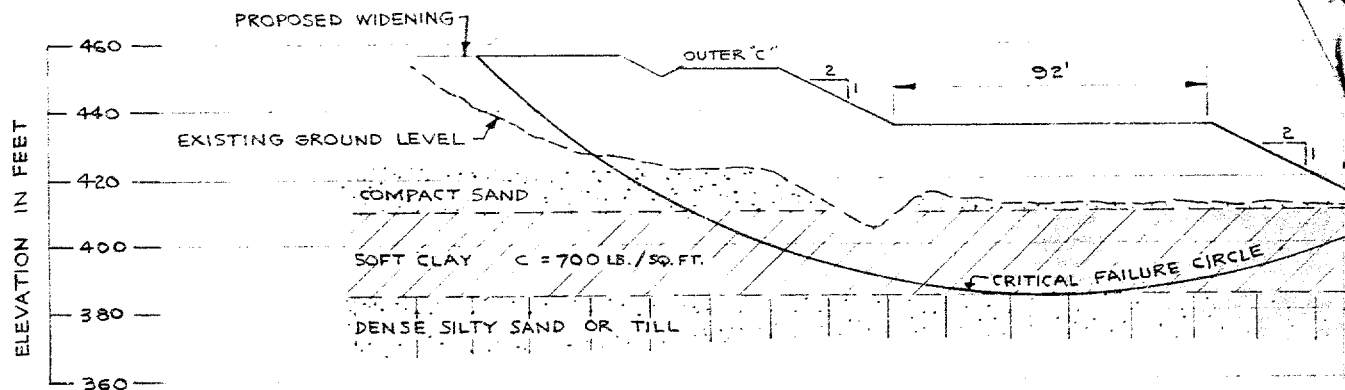
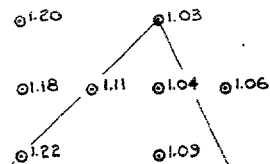
PORE PRESSURE DISTRIBUTION
IN SOFT TO FIRM CLAY

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Chkd. M.M.B.
Appd. K.F.

NOTE: FIGURES ARE FOR FACTOR OF SAFETY OF
CIRCLE WITH CENTRE AS SHOWN.
ALL CIRCLES TANGENT TO LINE AT
ELEVATION 385.0.



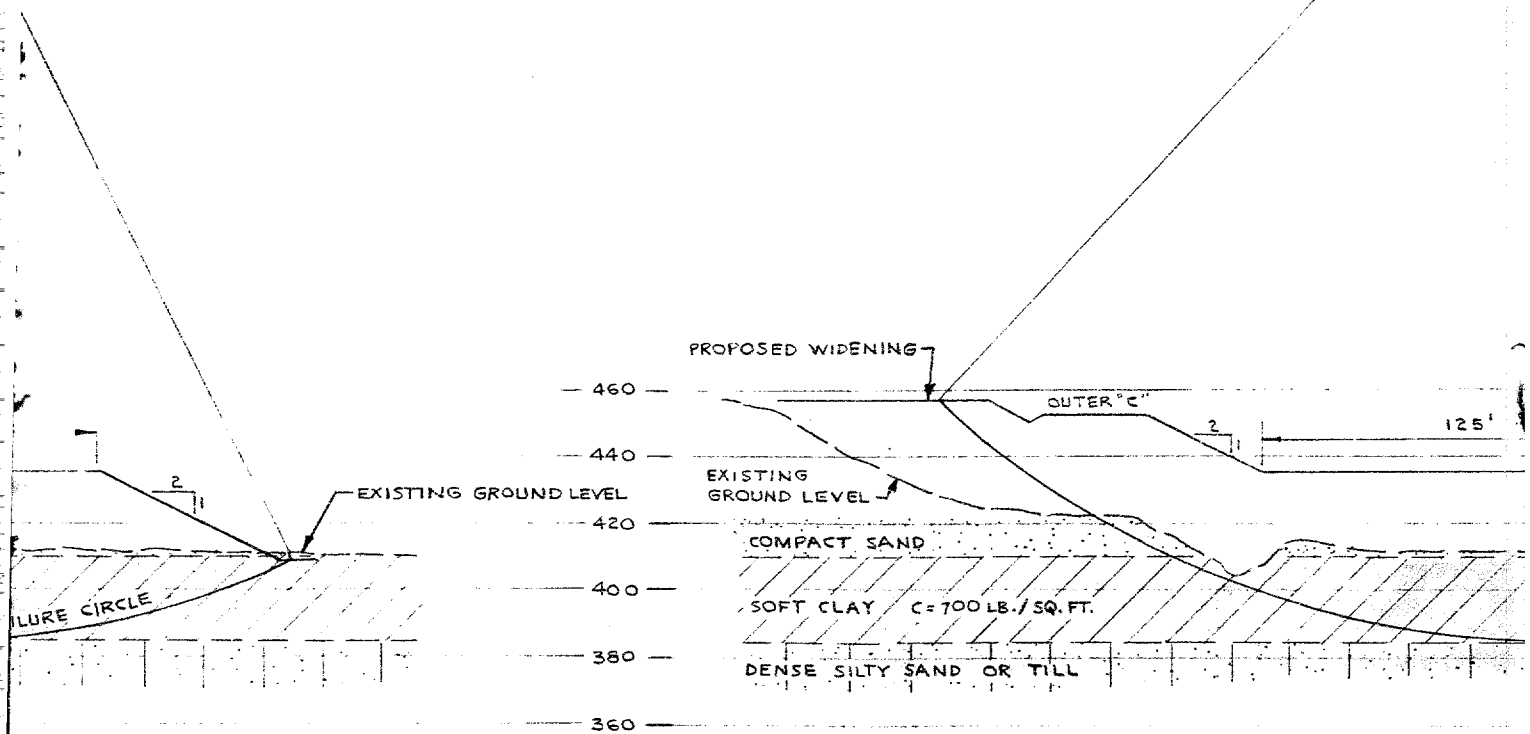
SECTION

ANALYSES ARE FOR BERM WIDTH OF 92 FEET, APPROXIMATELY 4 TIMES

06

NOTE: FIGURES ARE FOR FACTOR OF SAFETY OF
CIRCLE WITH CENTRE AS SHOWN.
ALL CIRCLES TANGENT TO LINE AT
ELEVATION 385.0

@1.27
@1.30
@1.22
@1.37



SECTION AT STATION 145+20 , OUTER LEG OF "C"

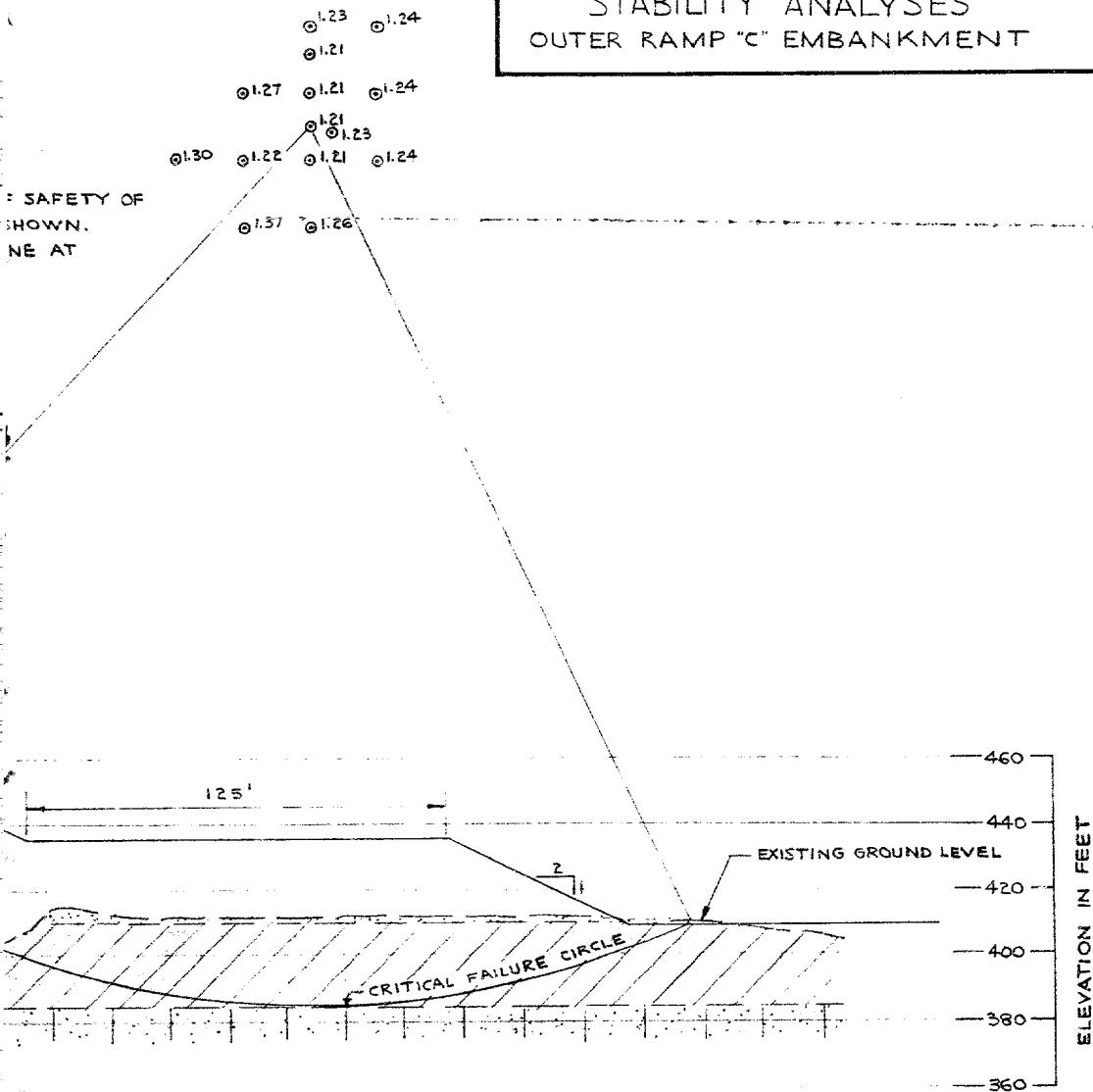
ATELY 4 TIMES BERM THICKNESS

ANALYSES ARE FOR BERM WIDTH OF 125 FEET, APPR

SCALE 1" TO 40'-0"

STABILITY ANALYSES OUTER RAMP "C" EMBANKMENT

FIGURE 39

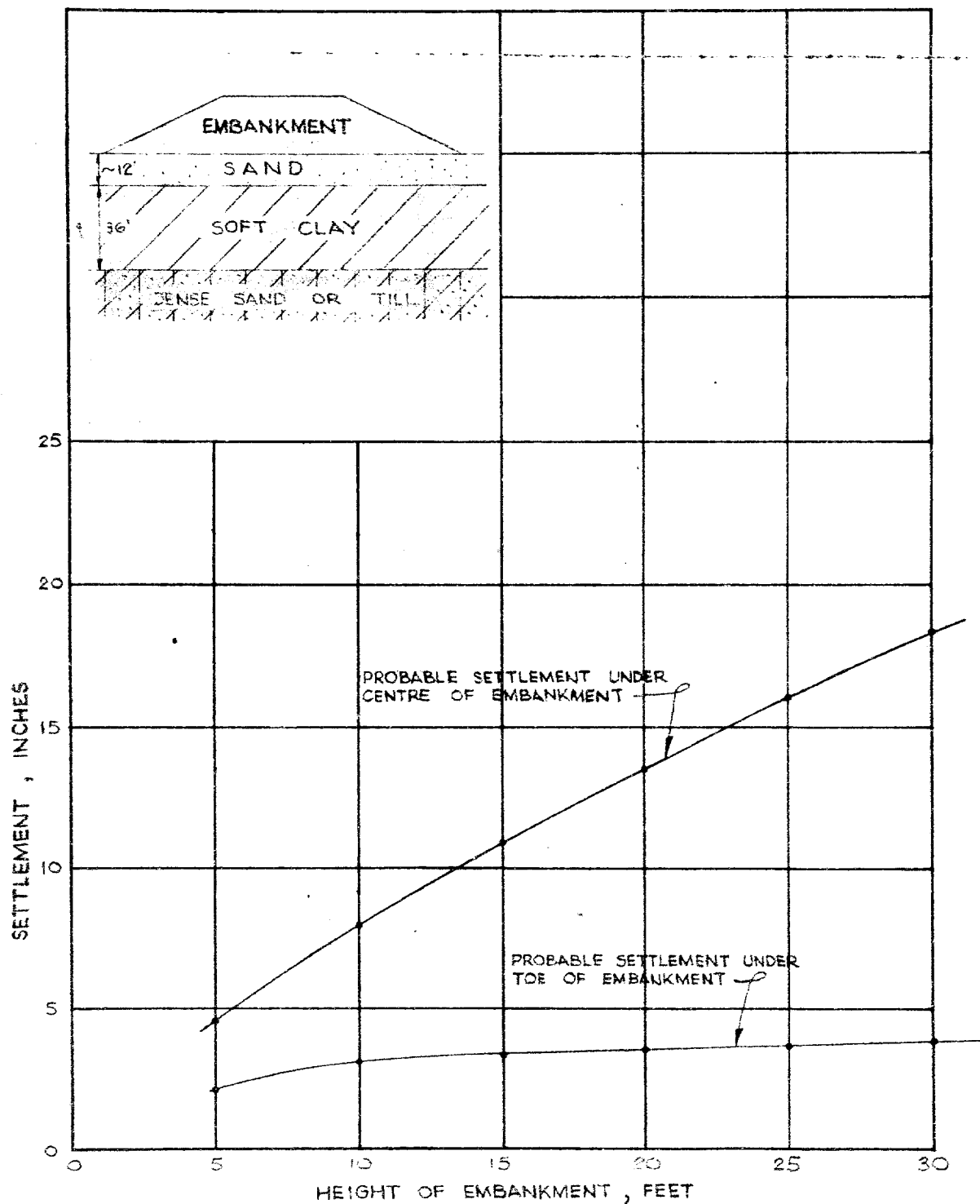


OF 125 FEET, APPROXIMATELY 5 TIMES BERM THICKNESS

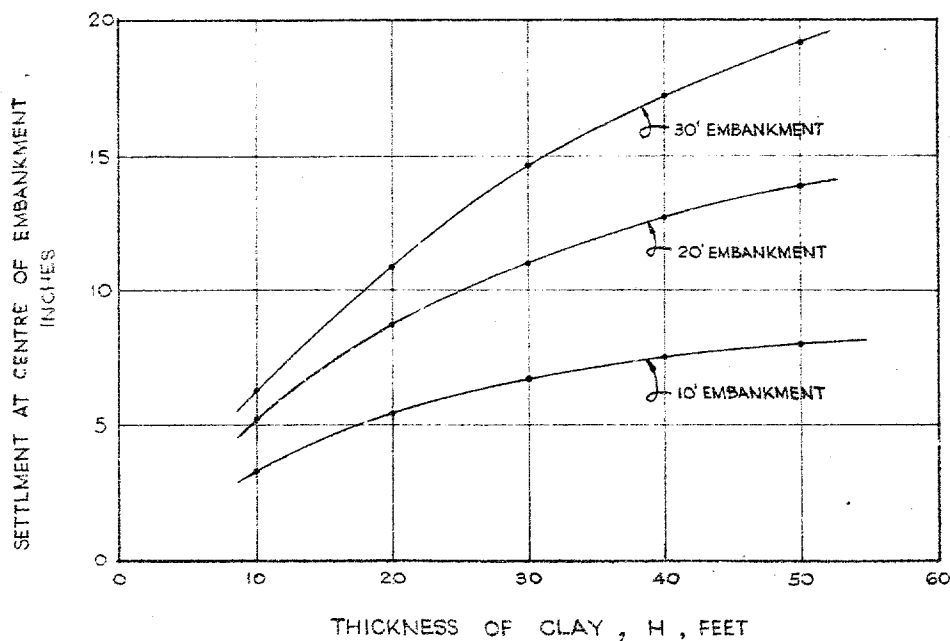
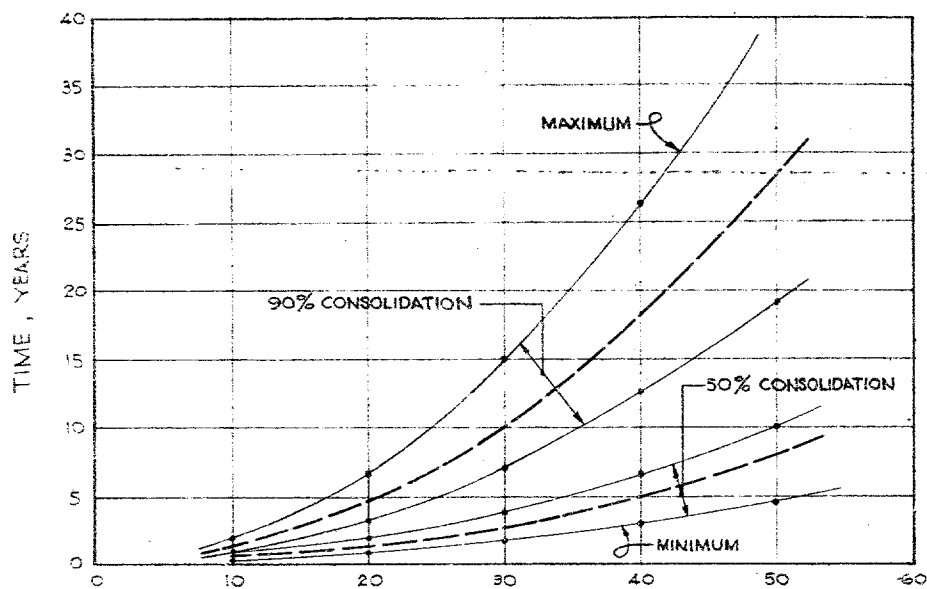
30M14-142
GEOCREP No.

GOLDER & ASSOCIATES

Made *M.W.*
Chkd. *EDEL*
Appd. *E.L.*



COMI
FOR VARI



* REFERENCE :

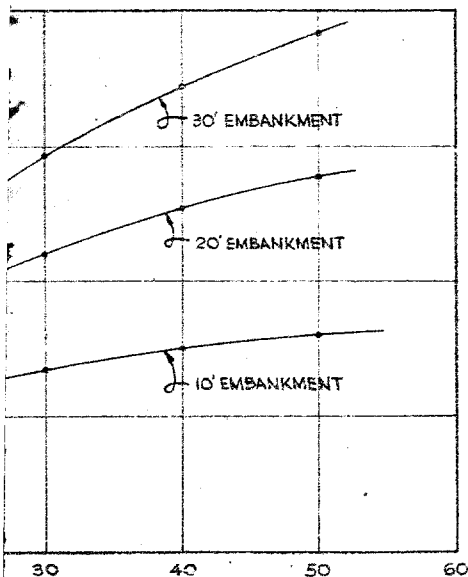
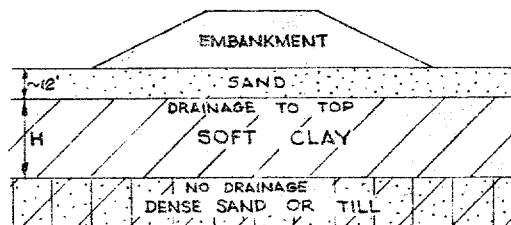
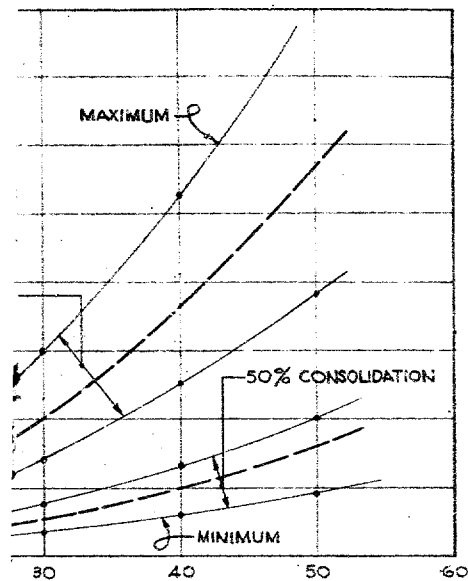
CLAY

COEFFICIENT

SETTLEMENT

COMPUTED SETTLEMENT FOR VARIOUS CLAY THICKNESSES

FIGURE 41



OF CLAY, H, FEET

CLAY PROPERTIES ASSUMED FOR SETTLEMENT COMPUTATIONS

COMPRESSION INDEX, $C_c = 0.18$

INITIAL VOID RATIO, $e_0 = 0.7$

COEFFICIENT OF CONSOLIDATION, $C_v = 1.4 \text{ TO } 3.0 \times 10^{-2} \text{ SO. IN. / MIN.}$

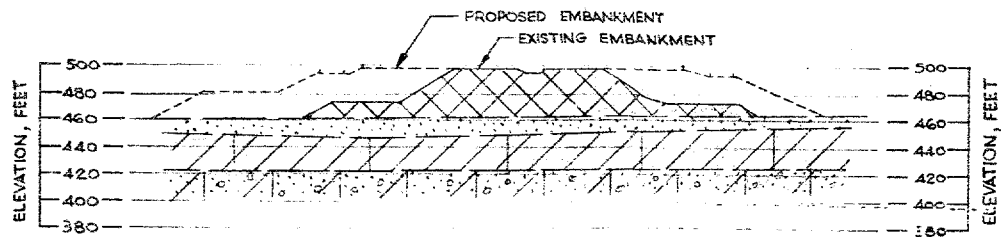
SETTLEMENT CORRECTION FACTOR, $\mu^* = 0.7$

* REFERENCE: "A CONTRIBUTION TO THE SETTLEMENT ANALYSIS OF FOUNDATIONS ON CLAY", A.W. SKEMPTON AND L. BJERRUM, GÉOTECHNIQUE, VOL. VII, NO. 4, DEC. 1957.

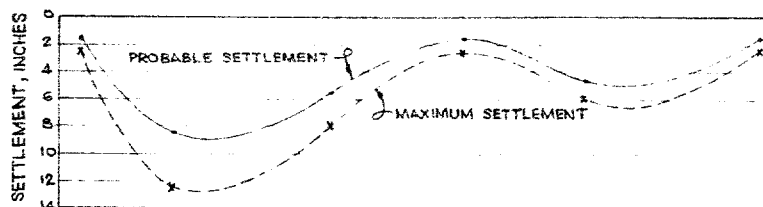
30M14-142
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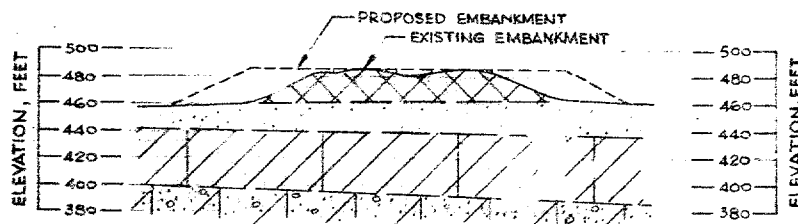
Made P.C.
Chkd. J.H.B.
Appd. H.J.



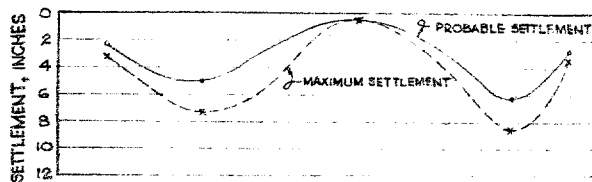
SECTION AT STATION 130+50



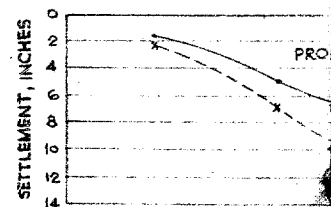
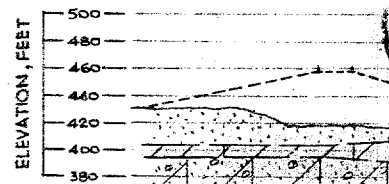
SETTLEMENT AT STATION 130+50



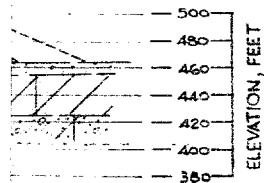
SECTION AT STATION 136+50



SETTLEMENT AT STATION 136+50



SCALE: 1" TO 100'-0"



STRATIGRAPHY



DENSE COMPACTED GRANULAR FILL.



COMPACT SAND.

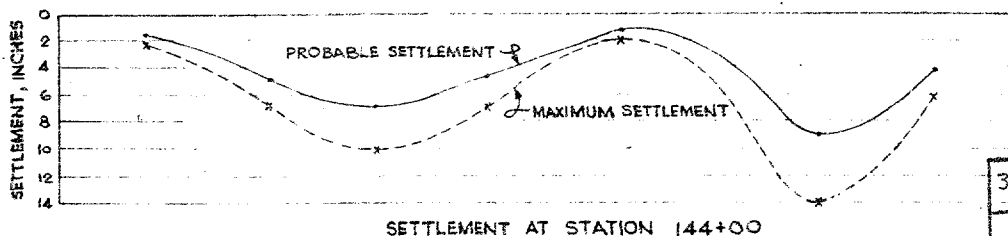
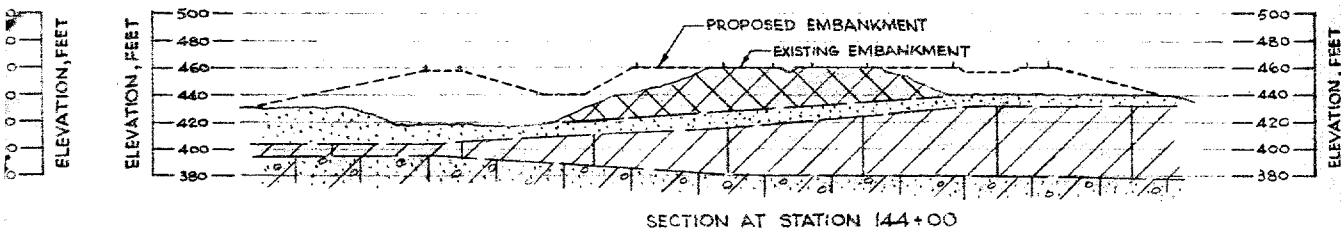


SOFT TO FIRM SILTY CLAY.



DENSE TILL.

NOTE: ALL SECTIONS LOOKING EAST.



30M14-142

GEOCRE No.

GOLDER & ASSOCIATES

Made P.C.
Chkd. *[signature]*
Appd. *[signature]*

SCALE: 1" TO 100'-0"

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 30M14-142

W.P. No. 150-61

CONT. No. -

W. O. No. -

STR. SITE No. -

HWY. No. 401

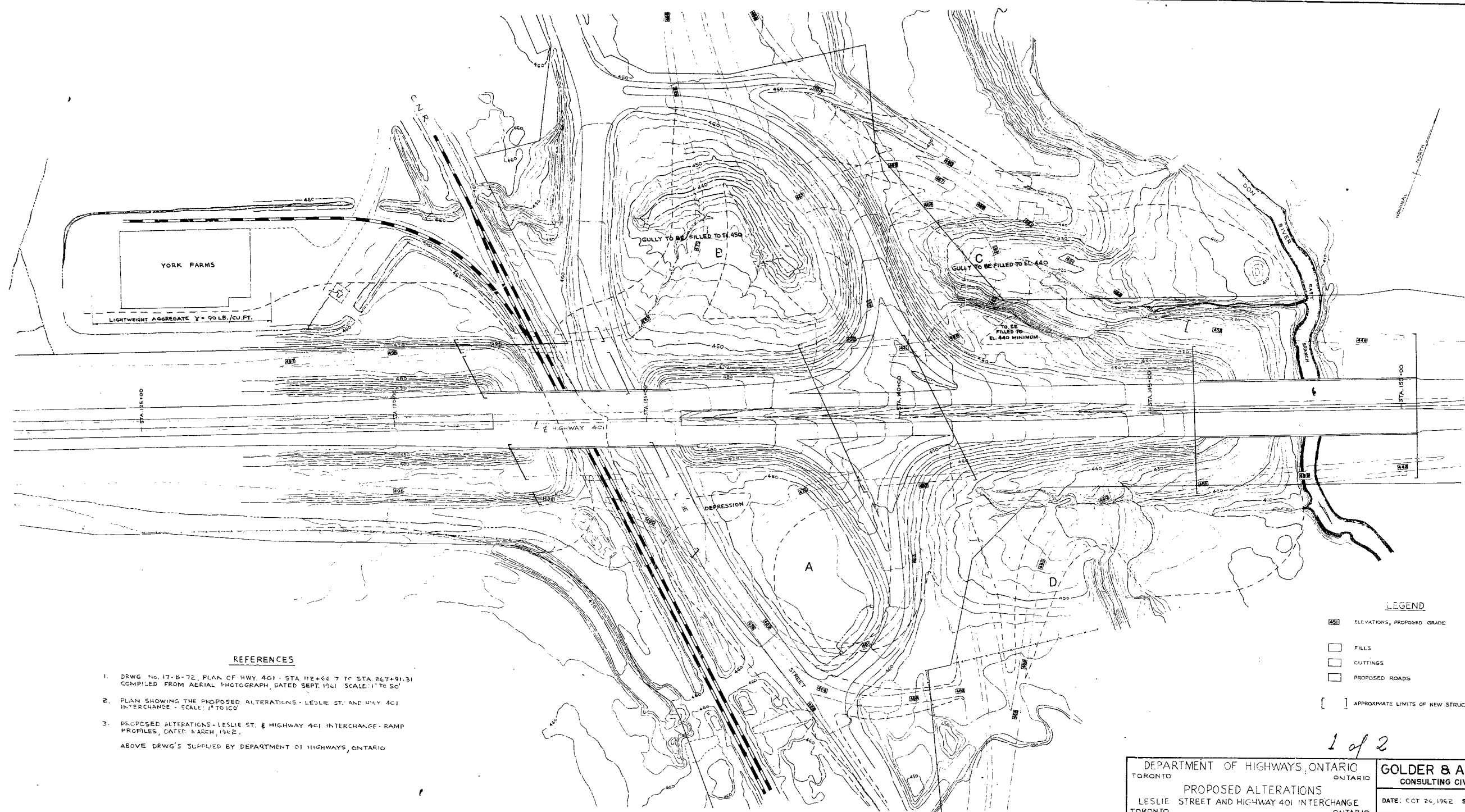
LOCATION LESLIE STREET

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

2 (Fig. 142)

REMARKS: _____

G.I.-30 SEPT. 1976



REFERENCES

1. DRWG NO. 17-B-72, PLAN OF HWY 401 - STA 112+66.7 TO STA 267+91.31 COMPILED FROM AERIAL PHOTOGRAPH, DATED SEPT. 1961 SCALE: 1" TO 50'
 2. PLAN SHOWING THE PROPOSED ALTERATIONS - LESLIE ST. AND HWY 401 INTERCHANGE - SCALE: 1" TO 100'
 3. PROPOSED ALTERATIONS - LESLIE ST. & HIGHWAY 401 INTERCHANGE - RAMP PROFILES, DATED MARCH, 1962.
- ABOVE DRWG'S SUPPLIED BY DEPARTMENT OF HIGHWAYS, ONTARIO

LEGEND

- [ELEV.] ELEVATIONS, PROPOSED GRADE
- [] FILLS
- [] CUTTINGS
- [] PROPOSED ROADS
- [] APPROXIMATE LIMITS OF NEW STRUCTURES

1 of 2

30M14-142
GEOCRE No.

DEPARTMENT OF HIGHWAYS, ONTARIO TORONTO		GOLDER & ASSOCIATES CONSULTING CIVIL ENGINEERS	
PROPOSED ALTERATIONS LESLIE STREET AND HIGHWAY 401 INTERCHANGE TORONTO		DATE: OCT 26, 1962 SCALE: 1" TO 100'	
MADE N.W.	CHKD. R.D.	APPD. V.H.	FIGURE 2

