

MEMORANDUM

To: Welland Project,  
East Main Design Committee.

FROM: Bridge Division,  
Downsview, Ontario.

DATE: April 10, 1968.

OUR FILE REF.

IN REPLY TO

SUBJECT:

Tunnel at East Main Street -  
Technical Committee

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The attached memorandum describes the East Main Design Committee and its functions. This continual review will keep all parties aware of problems, changes, etc., as they arise.

There will be a meeting of this Committee in Board Room West #1 at 2-P.M. on Thursday, 18 April, 1968. You are invited to attend.

  
B. R. Davis,  
Chairman.

BRD/vk  
Attach.  
c.c. Committee Members

## MEMORANDUM

To: Mr. J. Walter,  
Director of Design Branch,  
Executive Section,  
Admin. Bldg.

FROM: Bridge Division,  
Downsview, Ontario

DATE: March 28, 1968.

OUR FILE REF.

IN REPLY TO

## SUBJECT:

In order to reduce the time spent at meetings, the East Main Design Committee for the Welland Project will meet in its entirety only on the third Thursday of each month to review the current status of all aspects of the Project.

Day to day technical problems involving only part of the Committee will be held as required. Minutes of such meetings will be circulated at once and they will be discussed at the next monthly meeting.

The Committee will include:

4292920	B. R. Davis	Chairman
	K. Pullerits	Secretary
	H. Greenland )	
	B. S. Richardson)	
	J. H. Blevins )	D.H.O.
	R. G. Burnfield )	
	A. G. Scermac )	
	K. L. Coldwell )	
	W. A. O'Neil )	S.L.S.A.
	H. Landells	Engineer, City of Welland
	K. A. J. Williams	Engineer, County of Welland

After award of contract this committee will likely be re-constituted as the East Main Contract Committee under the chairmanship of H. Greenland.

BD/im

*B.D.*  
B. Davis,  
Bridge Engineer.

*Recommended*  
Approved

*J. Walter*  
.....  
J. Walter

APPROVED

*[Signature]*

Friday  
afternoon  
2 P.M.

H. Q. GOLDER & ASSOCIATES LTD.

CONSULTING CIVIL ENGINEERS

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

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REPORT

TO

DEPARTMENT OF HIGHWAYS, ONTARIO

ON

TRIAL SHAFT AT PROPOSED TUNNEL SITE  
EXECUTION, SAMPLING AND TEST RESULTS  
WELLAND ONTARIO

*Q.P. 130-61*

Distribution:

- 11 copies - Department of Highways, Ontario,  
Toronto, Ontario
- 6 copies - Gibb, Underwood and McLelland,  
Toronto, Ontario
- 2 copies - H. Q. Golder & Associates Ltd.,  
Toronto, Ontario

July, 1964

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## INTRODUCTION

H. Q. Golder & Associates Ltd. have been retained by the Department of Highways, Ontario, to carry out a detailed study of the soil conditions at the site of the proposed tunnel beneath the Welland Canal at Welland, Ontario. The consulting engineers for this tunnel are Gibb, Underwood & McLellan Ltd. of Toronto, Ontario. The proposed tunnel will largely be constructed in open cut and beneath the canal it will be formed by sinking floating pre-cast concrete tunnel units.

Preliminary soil investigations had been previously carried out in the general area of the proposed tunnel by H. Q. Golder & Associates Ltd. in 1960 and 1961; the results of these investigations are given in the following reports:

- (i) "Soil Conditions, Proposed Welland Canal Tunnel".  
Report 6022, dated December, 1960.
- (ii) "Soil Conditions, Proposed Welland Canal Tunnel".  
Report 6108, dated July, 1961.

These investigations established the general soil conditions in the area and provided information for feasibility studies for a tunnel and for the tunnel approaches.

In the summer of 1963, a detailed investigation was carried out jointly by the Department of Highways, Ontario and H. Q. Golder & Associates Ltd. The purpose of this investigation

was to obtain the additional data required for the detailed final design of the tunnel and approaches. Despite careful sampling and testing techniques, the properties and in particular the shear strength properties of layered clay below 45 ft. from ground surface could not be satisfactorily determined. It was therefore decided, in view of the critical importance of the properties of the clay in relation to the safety and economy of temporary and permanent cut slopes of excavations, that a test shaft be put down to permit direct examination of the clay and the taking of large 'undisturbed' block samples for laboratory testing.

The previous investigations had confirmed that the soil conditions were generally similar across the site and so the location of the test shaft (Fig. 1) was selected on the west bank of the canal where an open area was available.

This report describes the construction of the test shaft, supervised by H. Q. Golder & Associates Ltd., gives observations made during construction and details the sampling and testing techniques both in the field and the laboratory together with the results of the in situ and laboratory tests.

#### SITE AND GEOLOGY

The site is located in the physiographic region known

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as the Haldimand Clay Plain, which in this area is relatively flat. The area is underlain by clay deposits of glacial Lake Warren which are of the order of 100 feet thick. The glacial lake phase was possibly interrupted by two major retreats of the ice front which resulted in distinctively different deposits, i.e. non-stratified, relatively silty, homogeneous deposits laid down with the ice front fairly close, and heavily stratified very clayey deposits laid down when the ice front had retreated some distance. All of the deposits, except the upper 30 to 40 ft. which are desiccated, are lightly over-consolidated. The silt and clay are underlain by a thin layer of glacial drift followed by shales and dolomites of the Paleozoic era.

SECTION IFIELD OBSERVATIONS AND TESTSCONSTRUCTION OF SHAFT

The work on the shaft commenced on October 25, 1963, and concluded on December 11, 1963, and was carried out by McNamara Construction of Ontario Ltd.

The upper 15 feet of the shaft was excavated in open cut. The excavation was carried out using a 3/4 cu. yd. backhoe operating from ground level. The spoil was stockpiled to the south and to the west of the excavation. The open-cut excavation was rectangular in plan so that the spoil from the lower section of the shaft could be stockpiled at 15 feet below ground level. Details of the entrance to the shaft and the open-cut section are shown in Figs. 2, 11.

The cut slopes were mainly 1 horizontal to 1 vertical, but at the north end of the excavation were 1/2 horizontal to 1 vertical. The slopes remained stable throughout the periods of construction of the shaft except for minor sloughing following periods of heavy rain.

The test shaft commenced from a depth of 15 feet. The soil was dug out by a three-man crew using picks, shovels and

occasionally a pneumatic clay spade, the soil being manhandled to the surface. From 15 ft. deep to 30 ft. deep the shaft was  $7\frac{1}{2}$  ft. square. It was sheeted with vertical 2 in. by 6 in. spruce poling boards and braced with 6 in. by 6 in. struts every two or three feet of depth. Details of the sheeting and bracing for the square shaft are shown in Fig. 3.

The shaft was dug ahead of the sheeting by 5 feet each time. Initially, two poling boards forming each corner were nailed together and placed in position, then the walers or struts were placed to hold the corner poling boards in position and the remainder of the poling boards inserted. As each lift was completed another 5 feet was dug out and the sheeting placed. The bottom set of sheeting overlapped the sheeting above, by 6 in. The square shaft extended from 15 ft. to  $29\frac{1}{2}$  ft. depth.

The circular shaft was commenced at  $29\frac{1}{2}$  ft. depth near the north side of the square shaft as shown in Fig. 3. The shaft was dug ahead of the sheeting by one man, who loaded the soil into a bucket. The bucket was winched to 15 ft. depth by a second man while a third cleared it away.

When the bottom of the shaft was approximately 4 feet below the bottom of the sheeting the next set of sheeting was

installed. Each level of sheeting was braced with two steel rings, 60 in. and 64 in. diameter. These rings were prefabricated in halves each of 2 in. by 2 in. by  $\frac{1}{4}$  in. angle and bolted together in position. Four timber runners were first placed in position around the shaft. The upper (60 in. dia.) and lower (64 in. dia.) rings were then assembled on the bottom of the shaft and lifted into position and loosely secured with nails. The remainder of the shaft was trimmed and the runners inserted behind the rings. Wooden wedges were driven between the steel rings and the runners to hold them tight against the soil. One length of sheeting was purposely omitted from each level to display a continuous soil profile from top to bottom of the shaft. The sheeting was driven 6 in. into the undisturbed soil. The last 6 in. of soil was dug out carefully to prepare an undisturbed surface for block sampling.

This procedure was continued for the entire depth of the shaft. Details of the circular shaft are shown in Figs. 4 and 11.

At a depth of 77 ft. the bottom of the shaft began to feel spongy under foot. As the digging continued the bottom started to heave and no headway was made. When the runners were driven ahead of the excavation, it only caused increased loss of ground. The bottom of the shaft failed during the night and the next day the shaft was filled with water to 24 ft. below ground



level. The heaved soil was found to have filled the shaft to 72 ft. below ground level. Three cu. yds. of gravel were dumped to the bottom of the shaft to act as a filter and to prevent heave should it ever be necessary to pump out the shaft. The top of the gravel is at about 68 ft. below ground level. A detailed description of the shaft failure is given in Appendix I.

#### SOIL STRATIGRAPHY OBSERVED IN SHAFT

The soil stratigraphy observed in the shaft is presented in tabular form in Table 1.

TABLE 1  
SOIL STRATIGRAPHY

DEPTH	CONSISTENCY	COLOUR	SOIL DESCRIPTION	SPECIAL NOTES
0 to 30 ft.	Hard to very stiff	Brown to Reddish-Brown	Fissured silty clay with occasional gravel cobbles and boulders.	The soil is hard at ground surface becoming very stiff with depth. The majority of the fissures are vertical and the soil along the fissure faces is grey and softer.
30 to 45 ft.	Stiff	Reddish-Brown	Intact silty clay with occasional sand, gravel and cobbles.	The soil is apparently uniform with no visible stratification or layering. A few zones of more silty soil were encountered.
45 to 49.5 ft.	Stiff to firm	Reddish-Brown and Grey	Intact silty clay with layers of more plastic silty clay.	The first layering was encountered at 45 ft. It was 3 in. thick layer of more plastic grey silty clay and appeared to dip slightly to the south. Some of the grey deposits were found to be lenticular.
49.5 to 54 ft.	Stiff Firm Firm Soft "	Reddish-Brown Dark Brown Greyish-Brown Red Grey	Intact silty clay " " " " " " Plastic silty clay " " "	The layers were found to be $\frac{1}{2}$ to 1 in. thick. Occasional very thin layers grey clay and silt were encountered.
54 to 72.2 ft.	Stiff	Reddish-Brown	Intact silty clay with occasional sand gravel and cobbles	Occasional zones were encountered where the soil was slightly layered and more plastic
72.2 to 77 ft.	Firm	Brown Greyish-Brown Dark Brown Red Grey	Intact Plastic silty clay	All layers were very plastic. Layers generally $\frac{1}{4}$ to 1 in. thick.

## FIELD SAMPLING

### (a) Block Sampling

Two block samples were cut out from the bottom of the shaft every  $4\frac{1}{2}$  ft. to 5 ft. from a depth of 30 ft. to the bottom of the shaft. The technique used to obtain the samples is illustrated in Figs. 5, 12, and is as follows:

When each level of shoring had been placed the soil was prepared for sampling. Working from a semi-circular sheet of  $\frac{1}{2}$  in. plywood one man carefully removed 6 ins. of soil from the other half of the shaft exposing an undisturbed surface. The plywood was then transferred to the other half of the shaft and the remainder of the shaft trimmed of the upper 6 ins. of disturbed soil.

The bottom of the shaft was roughly divided into four quadrants. Block samples were removed from two of the quadrants, another quadrant was required to cut off the bottom of block sample and the fourth quadrant of soil was used to obtain various tube samples and to carry out vane tests.

The method of block sampling is illustrated in Fig. 6. The special metal sampling box detailed in Fig. 7 was used without the base, as a template to mark off the size of the soil block. Using clay knives, slots were cut 2 in. wide and 13 in. deep around the sides of the soil block. The metal sampling box was then pushed

down over the block so that it just trimmed the final 1/16 in. of soil away. One side of the block was then dug away so that a 12 in. blade shovel could be tapped gently underneath to break the block off from the soil. The block was then lifted by hand out of the sampling hole, the bottom trimmed, and the metal base attached to the sampling box. The block was then winched to the surface for transfer to the transport box. (Fig. 8).

The block was placed on the lid of the transport box and the sampling box dismantled. The top of the sample was clearly marked and the block wrapped in aluminum foil. The transport box was placed over the sample and turned over. Wax was poured into the space between the styrofoam and the block, pieces of wood being used as spacers. The same procedure was repeated for the second block sample using a common face with the first sample and tapping the shovel under the sample from the side of the hole left by the first block.

(b) Tube Sampling

Tube samples were taken at the same depth as the block samples. Three different types of tubes were used and samples were taken horizontally, vertically and at 45 degrees.

Type of Sample	Length of Sample	Area Ratio, per cent	
		With Lip	Without Lip
3 in.dia. Shelby tube	8 in.	12.1	9.7
2 in.dia. Shelby tube	8 in.	19.1	15.4
2 in.dia. Polished brass tube	6 in.	-	7.5

(c) Field Strength Test Sampling

Undisturbed samples were taken frequently in the field as the shaft was dug and were tested in the field laboratory. The results are plotted on Fig. 1A 'Summary of Field Results'.

From 30 ft. to 42 ft. the field tests were carried out on small block samples which were either cut out of the bottom or the side of the shaft and trimmed to size in the field laboratory. From 42 ft. to the bottom of the shaft the samples were obtained by pushing a 2 in. dia. 6 in. long polished brass tube into the soil and digging out. These samples were extruded in the field laboratory and tested.

The samples were generally 4 in. long and 2 in. dia. and were tested at a rate of axial strain of 2 per cent per minute.

(d) Additional Sampling

Disturbed samples were obtained from the open-cut and square section of the shaft and returned to the laboratory in Toronto to be available if needed for identification tests.

Atterberg limits were carried out approximately every foot from a depth of about 35 feet to the bottom of the shaft. In zones containing layered material, Atterberg limit tests were carried out on the individual layers.

Individual moisture content samples were taken from the side of the shaft every 6 in. depth and tested in the field. Pocket Penetrometer tests using a Soiltest penetrometer, were also carried out in a horizontal direction in the side of the shaft every 6 in. depth.

One side of the shaft was photographed from 30 ft. to 74 ft.

#### IN SITU TESTING

##### (a) Plate Load Tests

After each set of block samples had been taken from the bottom of the shaft, an additional 2 ft. of soil was dug out and the bottom prepared in the same manner as for the block samples.

The central section of the shaft was carefully prepared and a thin layer of sand placed and levelled. The 6 in. dia.  $\frac{1}{2}$  in. thick bearing plate was positioned and levelled. A reaction cross-beam made up of a 6 in. by 6 in. beam was hung from the deepest 64 in. diameter steel rib and a 4 ton 'Blackhawk' hydraulic jack was placed between the plate and the reaction beam, wooden blocks being used as packers between the jack and the reaction beam. Two reference beams with their legs 4 ft. apart were driven into the soil at either side of the bearing plate and a Mercer dial gauge

of 1 in. travel and reading to 0.001 in. was attached to each beam. Special wings had been welded onto the bearing plate to act as a reaction for the dial gauges. Details of the bearing plate and the test layout are shown in Fig. 9 and photographs shown in Fig. 13.

At the beginning of the test a load of 200 lb., equivalent to a plate pressure of 1,000 lb/sq.ft., was applied through the jack to hold the beam, blocks and jack in position. The bearing pressure was increased by 500 lb/sq.ft. every minute until failure took place. The settlement readings were taken at the end of each minute, just before the next load increment was added. The failure load was determined by inspection of the load settlement curves. (Fig. 14 to 23). A summary of all the results is presented in Table 2 below.

TABLE 2  
SUMMARY PLATE LOAD TESTS

Test Number	Depth (ft)	Failure Load (lb.)	Failure Bearing Capacity $Q$ , lb/sq.ft.	$C_1$	$\frac{Q}{C_1}$	$C_2$	$\frac{Q}{C_2}$	$C_3$	$\frac{Q}{C_3}$	Average $C$	$\frac{Q}{C}$	lb/sq.ft./in. settlement
1	35.5	1150	5710	1230	4.65	735	7.80	-	-	983	5.82	$4.7 \times 10^4$
2	40.0	1500	7440	1310	5.68	1360	5.45	-	-	1335	5.57	$3.39 \times 10^4$
3	44.0	1600	7940	1760	4.51	945	8.40	-	-	1353	5.86	$5.24 \times 10^4$
4	48.0	2000	9930	1270	7.83	870	11.4	-	-	1070	9.28	$6.12 \times 10^4$
5	52.2	1400	6950	949	7.32	980	7.1	694	10.0	874	7.95	$4.05 \times 10^4$
6	57.0	1500	7440	1050	7.08	1110	6.70	982	7.60	1047	7.10	$4.97 \times 10^4$
7	62.2	1350	6700	1280	5.23	910	7.38	1120	5.98	1103	5.94	$5.65 \times 10^4$
8	67.0	1700	8440	1405	6.00	1040	8.12	1022	8.25	1156	7.30	$9.93 \times 10^4$
9	71.3	1450	7200	1123	6.40	1000	7.2	943	7.63	1022	7.04	$4.46 \times 10^4$
10	75.6	1500	7440	1055	7.05	784	9.50	821	9.05	887	8.40	$5.8 \times 10^4$

Notes

$C_1$  = Measured undrained shear strength in vertical direction, lb/sq.ft.

$C_2$  = " " " " " horizontal " "

$C_3$  = " " " " at 45 deg. lb/sq.ft.



(b) Vane Tests

At the same level as the block samples, a set of vane tests was taken in the bottom of the shaft. The vane was pushed into the soil vertically, horizontally and at 45 degrees and the undisturbed and remoulded shear strength of the soil measured.

The results of these tests are shown on the Fig. 1A 'Summary of Field Testing' and are presented in Table 3 below.

TABLE 3  
VANE SHEAR STRENGTH

Depth Below Ground Level	VERTICAL			HORIZONTAL			45 DEGREES		
	V	V <sub>R</sub>	S <sub>t</sub>	V	V <sub>R</sub>	S <sub>t</sub>	V	V <sub>R</sub>	S <sub>T</sub>
35.9				1020	580	1.8			
39.3	1220	620	2.0						
43.5	1240	640	1.9	1160	680	1.7			
44.0				1180	460	2.6			
46.4							1140	560	2.1
47.7	1060	560	1.9						
52.0							840	180	4.6
53.0							760	120	6.3
53.2	1120	240	4.6						
55.2				1060	320	3.3			
56.4							1140	360	3.2
56.7	1140	360	3.2						
61.0							1060	320	3.3
61.3	1100	340	3.2						
61.3	1200	260	4.6						
64.5				1000	260	3.8			
66.0							960	300	3.2
66.2	1040	380	2.7						
69.0				1100	460	2.4			
70.3							1060	420	2.5
70.6	1080	420	2.6						
73.5				1260	360	3.5			
75.0							1280	280	4.6
75.2	1360	280	4.9						

V = Undisturbed shear strength, lb/sq.ft.

V<sub>R</sub> = Remoulded shear strength, lb/sq.ft.

S<sub>t</sub> = Sensitivity =  $V/V_R$

(c) Pocket Penetrometer Tests

The unconfined strength of the soil was tested with a 'Soiltest' Pocket Penetrometer every 6 in. from a depth of 24 ft. to 73 ft. and in the end of every sample tested in the field laboratory. The results of these tests are shown on Fig. 1A, 'Summary of Field Testing'.

(d) Caisson Ring Diameter Measurements

The upper steel ring of each level of runners in the circular shaft was marked with two sets of measurement points which were drilled on approximately the north-south and east-west centre-lines of the shaft. The point was drilled by making a hole approximately 1/16 in. deep with a 3/16 in. drill. The diameter was measured using a special strain gauge which consisted of a 1½ in. mild steel "Tee" Section with a Mercer dial gauge measuring to 0.001 in. mounted at one end and a fixed contact point mounted at the other end. Details of the strain gauge are shown in Fig. 10 and a photograph is given in Fig. 13.

While the readings were being taken the strain gauge was suspended at the centre from the top of the shaft. The consecutive readings of each ring were always taken with the dial gauge at the same side of the shaft. The readings are tabulated, Table 4, below and converted to shaft diameters. These figures do not represent absolute diameters of the shaft but only relative diameters from one set of readings to another. By this method a measure of the change in the diameter of the rings was obtained.

TABLE 4  
CAISSON RING DIAMETER CHANGES

Date of Initial Reading	Ring	Direction of Points	Depth	Initial Diam. Inches	Change Nov. 25	Change Nov. 28	Change Dec. 2	Change Dec. 4	Change Dec. 6	Shape ↑ N	Average Shear Strength p.s.f.
20 Nov.	I	N-S E-W	29.8	59.534 59.699	+0.001 +0.010	+0.012 +0.010	+0.010 +0.005	-0.009 +0.005	-0.010 +0.005	○	1600
20 Nov.	III	N-S E-W	34.0	59.424 59.506	-0.011 +0.015	-0.011 +0.021	-0.026 +0.048	+0.033 +0.022	-0.068 +0.022	○	1080
25 Nov.	V	N-S E-W	38.7	59.443 59.350	- -	+0.007 +0.009	+0.002 -0.044	-0.136 +0.032	-0.163 +0.027	○	1200
25 Nov.	VIII	N-S E-W	42.5	59.407 59.838	- -	-0.017 +0.022	-0.024 +0.049	-0.047 +0.025	-0.062 +0.028	○	1300
25 Nov.	X	N-S E-W	46.7	59.614 59.726	- -	-0.002 +0.028	-0.014 +0.031	-0.063 +0.040	-0.086 +0.048	○	1080
28 Nov.	XII	N-S E-W	51.5	60.051 59.296	- -	- -	+0.057 obstructed	+0.053 -	+0.076 -	○	310
1 Dec.	XIV	N-S E-W	56.2	58.820 59.294	- -	- -	- -	-0.029 -0.039	-0.029 -0.053	○	1300
1 Dec.	XVI	N-S E-W	60.3	59.413 59.546	- -	- -	- -	-0.028 -0.037	-0.025 -0.042	○	1090
4 Dec.	XVIII	N-S E-W	64.9	59.883 59.416	- -	- -	- -	- -	+0.029 -0.022	○	1150
6 Dec.	XX	N-S E-W	69.3	59.601 59.368	- -	- -	- -	- -	- -		

(a) Observations of Groundwater Level

As the shaft was dug water began to trickle into the excavation at 16 ft. to 24 ft. below ground level. It appeared to come in along the fissures in the clay and amounted on the average to between 50 gals. to 75 gals. overnight. However, during heavy rain considerably more water collected in the shaft overnight. This is shown in Fig. 24 where a chart of overnight water accumulation is plotted against the daily rainfall as measured in the City of Welland.

The upper fissured zone appeared to be the only source of water encountered during construction until the bottom of the shaft boiled. The water was encountered even in the early stages of construction while the weather was quite dry after a two month drought.

Four N.G.I. porous brass piezometers were driven into the ground between the shaft and the canal to measure possible drawdown of the groundwater level due to the shaft construction. Details of the piezometer locations are shown in Fig. 2. The piezometers were installed by initially washing with A-rods and then driving the piezometer, attached to E-rods, to the required depth.

Piezometer	Tip Elev.	Gr. Elev.	Method of Installation	
			Washed	Driven
P1	541	581.5	0 - 20'	20' - 40'
P2	532	581.6	0 - 40'	40' - 50'
P3	511	581.5	0 - 50'	50' - 70'
P4	542	581.6	0 - 34'	34' - 40'

The piezometers were read at regular intervals during the sinking of the shaft. The results of these readings are plotted in Fig. 25. Despite the variation in the depth of the piezometers and the distance from the shaft, they showed no change in the groundwater level as the excavation was advanced.

SECTION IILABORATORY TESTING AND RESULTSLABORATORY TECHNIQUES

The block samples and tube samples taken in the shaft were returned to the Materials and Research Laboratory of the Department of Highways, Ontario and to the laboratory of H. Q. Golder & Associates Ltd. for detailed inspection and testing.

The wooden transport boxes were dismantled and the block samples were stored in the laboratory. Prior to testing each block was cut into four or more vertical slices each of which was immediately waxed. Smaller blocks approximately  $2\frac{1}{2}$  in. x  $2\frac{1}{2}$  in. x 6 in. were cut from the slices at various angles as shown in Figs. 26 to 36 and D.H.O. Figs. 1 to 12.

In general, samples were cut at angles measured between the longitudinal axis of the samples and the original horizontal plane of the block at 0, 30, 45, 60 and 90 degrees. (In samples tested by the Department of Highways, Ontario the angles quoted are in degrees between the longitudinal axis of the sample and the original vertical axis of the block). In some instances samples were cut from the blocks at other angles as noted in the figures.

The small blocks of soil were trimmed to 2 in. dia. cylinders, 4 in. high. These were tested in unconfined compression

at a rate of strain of 1 percent per minute by Golder & Associates and at a rate of 2 percent per minute by Department of Highways, Ontario. Where the size of the samples differed from that above (Block J, Fig. 35) it is noted on the relevant figures.

The results of the unconfined compression tests are plotted on Figs. 26 to 36 and D.H.O. Figs. 1 to 12.

Two consolidation samples, one to be consolidated vertically and one horizontally, were cut and trimmed from each block, with the exception of Block B. The samples generally 2.0 in. dia. and 0.5 in. high were consolidated to a maximum pressure of 80 tons/sq.ft. The results of the consolidation tests are plotted as void ratio,  $e$ , versus logarithm of pressure,  $p$ , on Figs. 49 to 67, and the results are summarized in Fig. 68. Each increment of load in the tests was maintained for 24 hours before the next load increment was applied.

At least one thin slice from each block was carefully dried out and inspected for layering and stratification. Typical slices of blocks were photographed and are shown on Figs. 43 and 44. Detailed stratigraphic drawings of Blocks E, F, G, H and K are presented in Figs. 37 to 40 and 42 together with a summary of the moisture contents, Atterberg limits and the percentage of clay



size particles (finer than 0.002 mm.)

A series of consolidated undrained triaxial tests with pore pressure measurements was carried out on tube samples from the same depth as Block F. The results of these tests are presented in Fig. 69. A series of consolidated drained triaxial tests was carried out on Slice d of Block K and the results of these are presented in Fig. 70. Reference should also be made to D.H.O. Fig. 13.

#### DETAILED STRATIGRAPHY AND PROPERTIES OF BLOCK SAMPLES

Each block sample was examined in detail and the following summarizes the observations and test results. The term "vertical unconfined compressive strength" applies to the unconfined compressive strength of a sample which has been cut from a block sample so that its longitudinal axis corresponds with the vertical axis of the block. The term "horizontal unconfined compressive strength" applies to the unconfined compressive strength of a sample which has been cut from a block sample so that its longitudinal axis corresponds with horizontal plane of axis of the block.

#### Block A Elev. 552.7 to 551.7 (Figs. 26, 49, 50)

The block consists of a reddish-brown slightly desiccated silty clay with occasional sand, gravel and cobbles. There was one  $\frac{1}{2}$  in. layer where the soil was more silty.

The average vertical unconfined compressive strength was found to be 2.1 ton/sq.ft. at about 5 percent failure strain. This decreases uniformly with increase in the orientation of the sample from the vertical to a horizontal unconfined compressive strength of 1.5 ton/sq.ft. at a failure strain of 13 percent.

The average natural moisture content ( $w$ ) of the sample was 19 percent and the plastic ( $w_p$ ) and liquid limit ( $w_l$ ) were 17 and 28 respectively. The percentage of clay size particles (less than 2 microns or  $2\mu$ ) was about 30 percent, thus the activity ( $I_p / < 2\mu$ ) of the clay is low.

The consolidation tests (Figs. 49, 50) showed the vertical and horizontal compression index,  $C_c$ , to be 0.21 and 0.15 at initial void ratios of 0.61 and 0.55 respectively.

Block B Elev. 548.7 to 547.7 (Figs. 27, 43)

The block consists of a reddish-brown silty clay with occasional sand and gravel. There were vertical zones of more silty and more clayey soils running through the block as shown in Fig. 43.

The average vertical unconfined compressive strength was found to be 1.3 tons/sq.ft. at about 7 percent failure strain.

The average moisture content ( $w$ ) of the block was about 22 percent and the plastic ( $w_p$ ) and liquid limit ( $w_l$ ) were 16 and 25 respectively. The percentage clay size particles ( $<2\mu$ ) was about 29 percent.

Block C Elev. 554.9 to 543.9 (Figs. 28, 51 and 52)

The block consists of a reddish-brown silty clay with occasional sand and gravel.

The average vertical unconfined compressive strength was 1.7 ton/sq.ft. at a failure strain of 3 percent. This decreased for samples with increasing orientation from the vertical to an average horizontal unconfined compressive strength of 1.0 ton/sq.ft. at a failure strain of 12 percent.

The average natural moisture content ( $w$ ) of the sample was 21 and the plastic ( $w_p$ ) and liquid limit ( $w_l$ ) were 18 and 26 respectively.

The percentage of clay size particles ( $<2\mu$ ) was 30 percent.

The consolidation tests (Figs. 51, 52) showed the vertical and horizontal compression index,  $C_c$ , to be 0.18 and 0.18 at initial void ratios of 0.64 and 0.62 respectively.

Block D Elev. 540.4 to 539.4 (Figs. 29, 53 and 54)

The block consists of a reddish-brown silty clay with occasional sand, gravel and cobbles. There was a grey, more clayey layer, about  $\frac{1}{2}$  in. thick passing through the sample.

The average vertical unconfined compressive strength was 1.6 ton/sq.ft. at a failure strain of about 5 percent. This decreased for samples with increasing orientation from the vertical to a horizontal unconfined compressive strength of 1.2 ton/sq.ft. at a failure strain of 13 percent.

The average natural moisture content ( $w$ ) of the sample was 21 percent and of the grey more clayey layer 28 percent. The plastic ( $w_p$ ) and liquid limit ( $w_l$ ) of the principal soil present was 16 and 26 respectively. The percentage of clay size particles ( $<2\mu$ ) was 29 percent.

The consolidation tests (Figs. 53, 54) showed the vertical and horizontal compression indices ( $C_c$ ) to be 0.17 and 0.16 at initial void ratios of 0.55 and 0.56 respectively.

Block E Elev. 536.3 to 535.3 (Figs. 30, 37, 43, 55 and 56)

The block generally consists of a reddish-brown silty clay with occasional sand, gravel and boulders with a 3 in. layer of grey more plastic silty clay which occurred at about elevation 535.8 (Figs. 37, 43).

The average vertical unconfined compressive strength of the samples cut entirely from the reddish-brown silty clay is 1.5 ton/sq.ft. at a failure strain of 4 percent. This decreased fairly uniformly with increasing orientation from the vertical to horizontal unconfined compressive strength of 0.9 ton/sq.ft. at a failure strain of 15 percent. The average vertical unconfined compressive strength of the samples cut intersecting the grey more plastic layer is 1.3 ton/sq.ft. at a failure strain of 6 percent. This decreased to a minimum value of 0.7 ton/sq.ft. (failure strain less than 2 percent) at an orientation of about 45 degrees and increased to give a horizontal unconfined compressive strength of 1.1 ton/sq.ft. at a failure strain of 12 percent.

The average natural moisture content ( $w$ ) of reddish-brown and the grey silty clay was 22 percent and 29 percent respectively. The plastic ( $w_p$ ) and liquid limit ( $w_l$ ) of the reddish-brown silty clay are 16 and 26 respectively, of the grey silty clay 19 and 35. The percentage of clay size particles ( $<2\mu$ ) of the reddish-brown soil was 28 percent, and of the grey soil 48 percent.

The consolidation tests (Figs. 55, 56) the vertical from the reddish-brown soil and the horizontal from the grey soil, showed the vertical and horizontal compression index,  $C_c$ , to be 0.17 and 0.27

at initial void ratios of 0.57 and 0.79 respectively.

Block F Elev. 531.7 to 530.7 (Figs. 31, 38, 43, 46 and 57)

The block consists of a complex series of red, grey, dark brown, greyish-brown, reddish-brown and brown silty clays of varying clay content (Fig. 38). The layers vary from  $\frac{1}{2}$  in. to 3 in. thick. The majority of the layers, particularly the red and the grey layers are highly plastic; there appears to be a former slip plane across the bottom of the sample (Fig. 43) and which is oriented parallel to the banks of the Welland Canal.

The average vertical unconfined compressive strength is 1.0 ton/sq.ft. at a failure strain of 3 percent. This decreased to a minimum value of 0.5 ton/sq.ft. at failure strain of 1 percent at an orientation of about 50 degrees from the vertical and increased to give a horizontal unconfined compressive strength of 0.9 ton/sq.ft. at a failure strain of 3 percent.

The natural moisture content ( $w$ ) of the layers varies from 16 percent for the reddish-brown silty clay to 51 percent for the grey silty clay. The plastic ( $w_p$ ) and liquid limits ( $w_l$ ) vary from 16 and 25 to 24 and 65 respectively. The percentage of clay size particles ( $<2\mu$ ) of the different layers range from 23 percent for the reddish-brown silty clay to 83 percent for both the grey and

the red silty clay. (Complete details of the layers are presented as Fig. 38).

One consolidation test carried out vertically on a sample showed the compression index,  $C_c$ , to be 0.44 at an initial void ratio of 1.12.

Block G Elev. 527.2 to 526.2 (Figs. 32, 39, 58 and 59)

The block consisted mainly of a reddish-brown silty clay with several more clayey and some more silty thin layers inclined at about 30 degrees to the horizontal (Fig. 39).

The average unconfined compressive strength of the samples cut vertical to the layering was 1.6 ton/sq.ft. at a failure strain of about 1 percent. This decreased to a minimum value of about 1.3 ton/sq.ft. at an orientation of about 60 degrees between the longitudinal axis of the sample and the layering and increased slightly to give an average unconfined compressive strength parallel to the layers of about 1.4 ton/sq.ft. at a failure strain of 2 percent.

The average moisture content ( $w$ ) of the predominant soil was 35 percent with the layers varying from 24 percent to 38 percent. The plastic ( $w_p$ ) and liquid limits ( $w_l$ ) of the predominant soil were 21 and 45 and the average percentage of clay size particles ( $<2\mu$ ) was 65 percent.

The two consolidation tests (Figs. 58, 59) carried out show that the vertical and horizontal compression indices,  $C_c$ , are 0.38 and 0.41 at initial void ratios of 1.00 and 1.08 respectively.

Block H Elev. 522.7 to 521.7 (Figs. 33, 40, 44, 60 and 61)

The block consisted of a complex series of dark brown, brown, reddish-brown and grey silty clay zones of varying clay contents (Fig. 40). There is a zone of more plastic grey silty clay at about elevation 522 (Fig. 44).

The average vertical unconfined compressive strength of the samples intersecting the grey more plastic soil is 1.4 ton/sq.ft. at a failure strain of 2 percent. This decreased to a minimum value of 1.1 ton/sq.ft. at an orientation of about 60 degrees from the initial vertical axis of the block and increased to give a horizontal unconfined compressive strength of 1.2 ton/sq.ft. at a failure strain of 2 percent. The samples cut from outside the grey clay zone showed one unconfined compressive strength of 1.4 ton/sq.ft. at a failure strain of 2 percent although the average of several would possibly be greater and of the order of 1.7 ton/sq.ft. The compressive strength decreased uniformly to show a horizontal unconfined compressive strength of 1.2 ton/sq.ft. at a failure strain of 2 percent.

The moisture content (w) of the zones outside the grey



silty clay varies from 30 percent to 38 percent and the average moisture content of the grey silty clay was 42 percent. The average plastic ( $w_p$ ) and liquid limits ( $w_l$ ) of the soil zones vary from 19 and 38 to 24 and 50, the latter being the Atterberg limits of the grey zone. The percentage of clay size particles ( $<2\mu$ ) varied from 53 percent to 80 percent, again the latter figure corresponding to the grey clayey zone.

The two consolidation tests (Figs. 60, 61) carried out on samples from outside the grey clayey zone show the vertical and horizontal compression indices,  $C_c$ , to be 0.33 and 0.34 at initial void ratios of 0.87 and 0.95.

Block I Elev. 517.7 to 516.7 (Figs. 34, 46, 62 and 63)

The block consisted of a uniform reddish-brown silty clay with occasional sand, gravel and cobbles (Fig. 44).

The average vertical unconfined compressive strength was 1.5 ton/sq.ft. at a failure strain of 2 percent. This decreased uniformly with orientation of the samples from the vertical to give a horizontal unconfined compressive strength of 1.2 ton/sq.ft. at a failure strain of 3 percent.

The average moisture content ( $w$ ) of the block was 31 percent and the plastic ( $w_p$ ) and liquid limits ( $w_l$ ) 21 and 39

respectively. The percentage of clay size particles ( $<2\mu$ ) was about 55 percent.

Two consolidation tests (Figs. 62, 63) gave vertical and horizontal compression indices,  $C_c$ , to be 0.41 and 0.33 at initial void ratios of 0.99 and 0.87 respectively.

Block J Elev. 513.3 to 512.3 (Figs. 35, 41, 64 and 65)

The block consisted of a uniform reddish-brown silty clay with occasional sand and gravel.

The average vertical unconfined compressive strength was 1.6 ton/sq.ft. at a failure strain of 4.0 percent. This decreased with orientation of the samples from the vertical to give a horizontal unconfined compressive strength of 1.2 ton/sq.ft. at a failure strain of 13 percent.

The average moisture content ( $w$ ) of the block was 28 percent and the plastic ( $w_p$ ) and liquid limits ( $w_l$ ) were 18 and 32 respectively. The percentage clay size particles ( $<2\mu$ ) was 42 percent.

Two consolidation tests showed the vertical and horizontal compression indices,  $C_c$ , to be 0.27 and 0.30 at initial void ratios of 0.83 and 0.85 respectively.

Block K Elev. 508.7 to 507.7 (Figs. 36, 42, 44, 47, 66 and 67)

The block consists of a complex series of red, grey, brown, light grey, grey brown silty clays of varying clay content (Figs. 42, 44). The layers vary from  $\frac{1}{2}$  in. to 2 in. thick. All the layers are highly plastic.

The average vertical unconfined compressive strength is 1.3 ton/sq.ft. at a failure strain of 2 percent. This decreased to a minimum value of 0.9 ton/sq.ft. at a failure strain of 1 percent at an orientation of about  $45^{\circ}$  and increased to give a horizontal unconfined compressive strength of 1.4 ton/sq.ft. at a failure strain of 1 percent.

The natural moisture content ( $w$ ) of the layers varied from 27 percent for the thin light grey silty clay to 53 percent for the grey brown silty clay. The plastic ( $w_p$ ) and liquid limits ( $w_l$ ) for the major soil layers varied from 23 and 50 for the red silty layers to 26 and 67 for the grey brown silty clay. The percentage of clay size particles ( $<2\mu$ ) for the same layers varied from 55 percent to 87 percent respectively.

Two consolidation tests showed that the vertical and horizontal compression indices,  $C_c$ , were 0.53 and 0.43 and initial void ratios of 1.24 and 1.06 respectively.

### SECTION III

#### SOIL PROPERTIES IN RELATION TO DESIGN

##### UNDRAINED SHEAR STRENGTH

There is considerable evidence to show that the use of the undrained shear strength in the total stress, or  $\phi=0$  method of analyses, cannot be justified in analyzing the long term stability of slopes in over-consolidated or fissured clays. (Bishop, Bjerrum, 1960); the error in estimating the factor of safety,  $F$ , in this case can be extremely large. An error on the unsafe side by a factor of as high as 20 has been reported. (Cassel, 1948). The undrained shear strength can however be used in estimating the stability of temporary cuts or slopes.

For slopes in normally consolidated clays, the errors in estimating the long term stability using the undrained shear strength, and  $\phi=0$  analysis, are not large, and the error, particularly in sensitive clays, is generally on the safe side. (Bjerrum and Kjaernsli, 1957; Kjaernsli and Simons, 1962).

At the site, below a depth of 45 ft. to 50 ft. the clay deposit is either normally consolidated or lightly over-consolidated. Except for the upper 30 ft. which is fissured, the clay is intact. Since the majority of cuts in the construction of the proposed tunnel will be temporary it is justifiable to examine the stability of these

temporary cuts in terms of total stress. The undrained shear strength of the clay is therefore of critical importance. Those cuts which are to be permanent must also be examined in terms of effective stress. The effective shear strength parameters of the clay are discussed in a further section of this report.

Since the clay sampled in the test shaft is intact and only lightly over-consolidated, the undrained strength of the clay was determined generally by unconfined compression tests. Where possible, in situ vane tests were also carried out at the bottom and sides of the shaft (Fig. 1A).

(a) Effect of Sampling Disturbance (Mainly related to Block F)

It is normally assumed that the process of sampling causes considerable changes in the soil structure and hence the sample obtained is "disturbed". The laboratory strength measured for the samples is therefore below that which may exist in the ground. It has been suggested that for some soils the strength of 'normal' samples may only be 40 percent of that for 'perfect' samples (Ladd, Lambe, 1963). It has further been reported that in a highly sensitive clay the strengths of samples trimmed from block samples were markedly greater than those of samples obtained with a high quality piston sampler or strengths measured in situ using vane apparatus. (Coates, McRostie, 1963).

It is of interest to examine the strengths measured in the zone of softest clay in the shaft, depth 47 ft. to 52 ft., corresponding to Zone F. (Fig. 1A). The laboratory shear strengths given are taken as 1/2 the unconfined vertical compression strength.

TABLE 5

Lab. Shear Strength ( $=\frac{1}{2}q_u$ ) lb/sq.ft.				Vane Shear Strength, lb/sq.ft.
Block Trimmed	Failure Strain	Tube from Shaft	Failure Strain	In situ
930	3	1,000	16	1,060
1,030	3	970	8	
1,050	2½	1,050	9	1,120
1,040	3	980	12	
870	6			

There is no marked difference between the strengths measured by the different methods. The block samples were cut carefully from the base of the shaft and then trimmed and waxed. Generally over a month elapsed between initially cutting the block and then trimming individual samples from the block for testing. In this time there could be significant relief of stress; however, in the samples trimmed from the blocks, there was no marked difference in strength between those cut from the outside of a block and those from the centre of a block. On the other hand, the tube samples were taken with a 2 in. dia. polished brass tube (area ratio 7.5 percent) within the shaft as it was excavated. They were then tested immediately in a field laboratory. The tube samples were probably

more disturbed than block samples in the actual sampling process but suffered less disturbance in handling, transportation and trimming.

(b) Effect of orientation, strain at failure

In contrast to the fact that there was little difference between the strengths measured by different methods for vertical samples, invariably there is a marked difference in the compressive strength of inclined samples between samples taken from sampling in tubes in the shaft and samples trimmed from blocks. For example, the results of tests on samples inclined at 45 degrees, in Zone F, are summarized below:

<u>Sample</u>	<u>Unconfined Comp. Strength, lb/sq.ft.</u>		
Tube, Area ratio 7.5%	1,380 at 6% failure strain		
	1,360 at 4%	"	"
Block, trimmed specimens	960 at $\frac{1}{2}\%$	"	"
	940 at 1%	"	"
	1,140 at $1\frac{1}{2}\%$	"	"
	1,300 at $1\frac{1}{2}\%$	"	"

It is significant to note in this case that not only were the test results for block samples lower than those for tube samples, but that these lower strengths were at lower failure strain. This applies only to the heavily stratified material from Zone F. This is contrary to the usual assumption that disturbance in sampling tends to increase the strain to failure and leads to a decrease in

the strength. Block C, a relatively 'isotropic' material, exhibits this latter effect. (Fig. 71). In general, the 'isotropic' block samples (Blocks A, B, C, D) tended to show the highest unconfined compressive strength in the vertical samples and the lowest in the horizontal samples. The vertical samples failed at the lowest strain and the horizontal samples at the highest. The inclined samples failed at intermediate strength and strain values.

The low strain at failure in the block samples from Zone F is probably due to the heavily stratified character of the deposit and in particular to the apparently sensitive character of two clay layers, red and grey in colour, specified as 'M' and 'N' in Fig. 38. Generally, failure in samples inclined between  $30^{\circ}$  and  $60^{\circ}$  to the original horizontal plane of the soil in situ occurred along these two layers and at low failure strains. Where the sample were close to the horizontal or vertical, failure took place across the layers. The observed range in strength with strain at failure for the stratified samples from Zone F is illustrated in Fig. 48. It is apparent that the stress/strain characteristics of the layers 'M' and 'N' markedly affect the strength of the clay, and further, that the stratified deposits exhibit directional shear strength properties.



(c) Relation of Undrained Shear Strength to Slope Stability

From examination of the test results, the undrained shear strengths recommended for use in design are given below.

TABLE 6

## SUMMARY OF UNDRAINED SHEAR STRENGTHS TO BE USED IN DESIGN

Summary of Undrained Shear Strengths to be Used in Design					
Depth below ground level in ft.	Undrained Shear Strength *lb/sq.ft.				Remarks
	Lowest Values Measured		Recommended Design Values		
	Horizontal	Vertical	Horizontal	Vertical	
0 - 15	-	-	-	-	Tension Crack
15 - 32	*1,500	*1,500	*1,500	*1,500	Fissured Soil
32 - 47	900	1,050	1,050	1,200	Non-stratified Soil
47 - 52	500	950	500	1,000	Stratified Soil
52 - 72	900	1,000	1,050	1,200	Non-stratified Soil
72 - 77	700	900	800	1,000	Stratified Soil

\*The terms horizontal and vertical undrained shear strength refer to shearing along horizontal and vertical planes respectively in the soil.

\*Undrained shear strength in the fissured zone is generally greater than 1,500 lb/sq.ft. but is assumed to decrease with time to 1,000 lb/sq.ft.

These values are based on the field strength tests and on the strength tests carried out in the laboratory on the block samples by the Department of Highways, Ontario and H. Q. Golder & Associates Ltd.

**GOLDER & ASSOCIATES**

The lowest values listed above represent the minimum strengths which are believed to exist on any plane. However what do the lowest strength values represent and what is the significance of testing samples variously oriented? The unconfined compression test results may generally be divided into two types; those carried out on blocks of non-stratified or 'isotropic' soil and those carried out on blocks of layered or stratified soil.

The 'isotropic' soils did not show any great variations in strength with orientation of the samples, except that tests on samples taken vertically had a slightly higher strength and lower strain at failure than those taken horizontally. Ratio of the maximum to minimum shear strength is about 1.5.

The layered soils were remarkably different. The greatest shear strengths were obtained when the deviator stress was applied either perpendicular or parallel to layers, with the failure plane cutting across the layers. The lowest shear strengths were obtained when the samples were cut at angles, thus causing the failure plane to follow the layers. The ratio of maximum to minimum shear strength is about 2.4.

The apparent difference in strength of the layered samples may be explained by the geometry of the compression tests. Assume that

a soil mass of shear strength,  $S_0$ , is intersected by a single plane of weakness of shear strength,  $S_1$ . It is assumed that  $S_0 > S_1$ .

Considering a cylindrical sample, acted on by principal stresses  $\sigma_1$  and  $\sigma_3$  (Fig. 72) the shear stress,  $\tau$ , developed on a plane inclined at  $\beta$  to the horizontal, can be determined. For varying inclinations of the plane, the shear stress  $\tau_\beta$  will vary as shown in Fig. 73.

In Fig. 74, the soil strength,  $S_0$ , is represented by a hypothetical strength envelope DFE; the strength of the plane of weakness, assumed to be infinitely thin,  $S_1$ , is represented by envelope PQR. Now, let  $\sigma_3$  be held constant and  $\sigma_1$  increased, as in triaxial compression. For lower values of  $\sigma_1$ , corresponding to the dotted circle AW, failure along the plane of weakness will be possible for some orientations of this plane but failure in the soil mass is not possible. When  $\sigma_1$  is increased so that the Mohr circle for  $\sigma_1$  and  $\sigma_3$  approaches DE at F, failure may take place in the plane of weakness if its inclination is such that the point X lies in the arc QFR; if the point X lies in the arcs AQ or RC, failure along the plane of weakness is not possible, but failure will take place through the soil mass at the Coulomb-Navier angle of  $\frac{\pi}{4} + \frac{\phi}{2}$  and this plane will intersect or cross the plane of weakness.

In applying this hypothesis to the results of the uncon-

finned compression tests, where  $\phi=0$ , it may be seen that a feasible method of evaluating the undrained shear strength of the thin, weak soil layers is to test a sample cut at an angle to the horizontal. It will be noted that for planes of weakness inclined from between  $30^\circ$  and  $60^\circ$  to the horizontal, the shear strength is approximately one-half of the compressive strength, as usually assumed in practice. (At an inclination of  $45^\circ$  to the horizontal, the shear strength is equal to exactly one-half of the compressive strength; at  $30^\circ$  and  $60^\circ$  to the horizontal the shear strength is 0.43 times the compressive strength). Further, the shear strength of a horizontal thin layer cannot be measured in a compression test where the sample is either cut vertically or horizontally from the soil.

Therefore, it can be concluded that one-half of the compressive strength, measured in the inclined samples (Range of  $30^\circ$  to  $60^\circ$  from the horizontal) of the layered soils, represents a realistic shear strength of the weak layer.

It may be assumed that, in the stability of cuts or slopes, along a potential surface of sliding, there will be re-orientation of principal planes of stress. (Fig. 75). In a  $\phi=0$  method of analysis this may be of minor significance in 'isotropic' soils, but in a layered deposit with directional strength properties, the re-orientation of principal stresses can be critical, since the

shear strength of the soil along the layers can be markedly different from the shear strength across the layers. This point is examined below in relation to the form of slope failure.

(d) Forms of Slope Failure

The excavation of a cut increases the shear stresses in the subsoil forming the foundation of the slopes. The slope will remain stable providing the total shear forces along any surface do not exceed the mobilized shear strength of the soil along the same surface.

A potential surface of failure may have any shape but it has been observed that most failures generally take place along curved surfaces, generally assumed to be cylindrical. For isotropic soils, this may be a valid assumption; however, where the soil has layers of weak soil, or has anisotropic strength properties, a failure surface may follow along a plane of weakness, thus giving a non-cylindrical failure surface as illustrated in Fig. 76.

A comparison was made between a circular arc analysis and a composite circle analysis for the conditions that exist in the present banks of the Welland Canal. (Fig. 77). It was found that similar factors of safety were obtained from a circular arc analysis using the shear strength of the weak layer as an average overall shear strength and from a composite circle analysis using

the shear strength along the layers for the section of the surface through the weak layer and the shear strength across the layers for the section of the surface outside the weak layer. Thus the directional strength properties of the layered clay markedly control the stability of the slopes.

(e) Examination of Known Failures - Welland Canal

Since the construction of the Welland Canal, several slope failures have taken place adjacent to the site of the proposed tunnel in Welland. From local information it is known that these failures caused some heave of the bottom of the canal but were never sufficient to impede shipping. However, after the failures it was always necessary to dredge the canal to maintain a required width of 200 feet at the bottom. From the known geometry of the failures (Peckover and fanner) it can be reasonably assumed that the failures were not deep but were caused by the presence of a layer of weak soil at a depth of approximately 45 to 50 feet.

Six failures that have taken place within 2,000 ft. of the site of the proposed tunnel, have been analysed assuming the depth of all failure circles to be 45 feet. These analyses are summarized below.

TABLE 7

SUMMARY OF CANAL BANK FAILURES

Station	Depth of Cut. Ft.	Slope Degrees	Depth of Arc. Ft.	Computed $S_u$ lb/sq.ft. for $F=1$
848+50 W	46	28.5	46	510
932+50 E	45	27	45	490
951+07 W	40	23	45	470
989+20 W	45	28.5	45	510
1030+00 W	42	26.5	45	480
1047+00 W	45	21	45	430

Average undrained shear strength,  $S_u = 480$  lb/sq.ft.

Thus, the figure of 500 lb/sq. for the undrained shear strength of the weak clay layer in Zone F, as determined from the laboratory tests, agrees with the figure computed from analysis of known slope failures in the area.

SHEAR STRENGTH IN TERMS OF EFFECTIVE STRESS

Since the long term stability of the approach cuts at the site must be examined, the stability of these cuts should be analyzed in terms of effective stress.

It is known that the angle of shearing resistance in terms of effective stress  $\phi'$  or  $\phi_d$  for non-stratified silty clay at the site is approximately  $30^\circ$  (Previous reports 6022, 6108). The effective shear strength properties of the stratified clay, occurring mainly in Zones F and K, are difficult to evaluate. Triaxial compression tests were carried out on samples trimmed from blocks cut from Zones F and K. The samples were trimmed so that the longitudinal axis of the sample was at  $45^\circ$  to the original horizontal plane of the soil in situ and so that failure would generally take place along the weaker clay layers. These tests are detailed in Figs. 69, 70, and in D.H.O. Figs. 13, 14.

Consolidated undrained tests were carried out on samples from Zone F. (Fig. 69; D.H.O. Figs. 13, 14). Pore pressures were measured at the base of the sample and also by means of hypodermic probes inserted along or adjacent to the probable failure plane. The effective in situ vertical overburden stress,  $\sigma_v$ , for Zone F, is about 28 lb/sq.in. The samples consolidated under an all round stress  $\sigma_3$  less than  $\sigma_v$  generally behaved as over-consolidated material. Samples consolidated under  $\sigma_3$  greater than  $\sigma_v$  behaved generally as normally consolidated material. Consequently, it is difficult to evaluate the significance of the cohesion intercept,  $c'$ . It is considered that the shear strength in terms of effective stress



can conservatively be represented by assuming that  $c' = 0$  and  $\phi'$  is approximately  $20^\circ$ .

In examining the results of consolidated undrained tests carried out on trimmed block samples from Zone F (Fig. 70), the decision as to what represents failure becomes more critical than for undrained tests since, if failure is defined as at the strain where failure planes are first observed in the sample, then  $\phi_d$  is approximately  $17^\circ$ , but if failure is defined to take place at maximum deviator stress, then  $\phi_d$  is approximately  $20^\circ$ . Considering the complexly stratified character of the deposit, the obvious differences in plasticity and water content of the various layers and the probable different stress-strain characteristics of the different layers, it is difficult to establish a single precise value for the shear strength parameters. However, for design purposes it would seem to be valid to assume  $c' = 0$  and  $\phi'$  or  $\phi_d$  to be approximately  $20^\circ$ .

#### CONSOLIDATION PROPERTIES

A summary of the consolidation properties is presented in Fig. 68, for both 'vertical' samples and 'horizontal' samples.

For the vertical samples, the general pattern of pre-consolidation pressures estimated from the tests suggests that the

material is over-consolidated throughout, by a pressure in excess of the existing overburden pressure of at least 1 ton/sq.ft., and possibly of 1.5 ton/sq.ft.

The pre-consolidation pressure estimated from the tests on horizontal samples is significantly lower than that estimated from tests on vertical samples. From examination of the results it would seem that the ratio of horizontal to vertical pre-consolidation pressure is about 0.7. However, the horizontal samples exhibit a distinct loss in pre-consolidation effect below a depth of about 60 ft. and below this depth the horizontal samples are apparently normally consolidated. Significantly, however, the ratio between horizontal pre-consolidation pressure at this depth (namely, the existing overburden pressure) to the estimated pre-consolidation pressure for the vertical samples is again approximately 0.7.

In general, samples both vertical and horizontal from the upper silty clay (Zones A to D inclusive) have a low initial void ratio,  $e_o$ , and a low compression index,  $C_c$ , which is generally about 0.2. Samples below this depth (Zones G to K inclusive) have an initial void ratio,  $e_o$ , of 1.0 or slightly greater, and a compression index,  $C_c$ , of about 0.4.

The relationship of the initial void ratio,  $e_o$ , to the

compression ratio,  $\frac{C_c}{1 + e_o}$ , is shown on Fig. 78 and agrees remarkably well with the relationship given by Peck and Reed for glacial clays in the Chicago area.



for

V. Milligan, P. Eng.

VM:IMB  
6375

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APPENDIX IWELLAND CANAL TUNNELNOTE ON SHAFT FAILURESATURDAY, DECEMBER 7, 1963

Depth of Shaft	76.5 ft.
Depth of Runners	74.0 ft.
Rainfall	nil

There were approximately 50 gallons of water in bottom of shaft at start of morning shift. (8 hours elapsed since end of night shift)

Starting at 7 a.m. the shaft was dug to 77 ft. where the bottom began to feel spongy under foot. This was the first indication of a potential failure. A small hole dug at the centre of the shaft to 77.5 ft. appeared to encounter some more silty layers in a quick condition. It was possible at this stage to push a piece of 1 in. by 2 in. timber at least 6 ft. ahead of the excavation by hand. As the contractor dug deeper, he began to lose ground so that by noon the bottom of the shaft had risen to 76.5 ft. During the morning approximately 1 cu. yd. of soil had been dug from the shaft.

After noon wooden runners were positioned around the shaft and an attempt was made to drive them ahead of the excavation with a sledge hammer. This was found to be extremely difficult and they were only driven to 77.5 ft. depth or about 12 ins. below the depth of deepest excavation. By this stage the bottom of the shaft had risen to at least 76.0 ft. and it was necessary to put a sheet of plywood on the bottom of the shaft so that men could stand on the soil.

The evening shift drove the runners to 78.5 ft. using a pneumatic sheeting driver. The runners went down very easily with the pneumatic driver but the bottom of the shaft rose to nearly 74.5 ft. which is 2.5 ft. up from the depth of deepest stable excavation.

This was carried out by 9 p.m. but the bottom of the shaft was still too soft to support a man. The shaft was then left until Monday morning to see if the bottom had been stabilized by the runners.

SUNDAY, DECEMBER 8, 1963

Rainfall - 0.34 inches.

The shaft was inspected visually from the top at about 3 p.m. The water appeared to be about 60 ft. or lower below ground level.

MONDAY, DECEMBER 9, 1963

Rainfall - 0.27 inches.

The water level in the shaft had risen to 24 ft. below ground level - 9 ft. below top of 7.5 ft. square shaft. A sounding showed the bottom of the shaft to be at approximately 72 ft. The contractor started pumping at 8 a.m. using an air pump calibrated at 550 gallons per hour.

By noon the water level was down to 29.5 ft.

By 2.30 p.m. the water level was down to 32.0 ft.

At this stage it was obvious that water was entering the shaft from a source below 77 ft. and the pumping was stopped. A sounding at 3 p.m. showed the bottom of the shaft to be still at 72 ft.

#### Rate of Boiling

Water rose in shaft from at least 60 ft. to 24 ft. from 3 p.m. Sunday to 8 a.m. Monday - 17 hours.

Volume = 1010 cu. ft. = 6,300 gals.

Minimum Average Inflow Rate =  $\frac{6300}{17}$  = 370 gal/hour

#### Rate of Pumping Water

Time	8 a.m. to 12 noon - 4 hours
Volume	= 310 cu. ft. = 1,950 gal.
Rate of Lowering water	= 500 gal/hour
Rate of Pumping	= 550 " "
Rate of Inflow	= 50 " "

**GOLDER & ASSOCIATES**

Time	12 noon to 2.30 p.m.	- 2.5 hours
Volume	= 55 cu. ft.	= 350 gals.
Rate of Lowering water	=	150 gal/hour
Rate of Pumping	=	550 " "
Rate of Inflow	=	400 " "

TUESDAY, DECEMBER 10, 1963

Four feet of gravel was dumped in the bottom, the shaft built up to the surface, the open cut excavation backfilled and a protective cover put of the top of the shaft.

## LIST OF ABBREVIATIONS

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

### I. SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
DS	Denison type sample
FS	foil sample
RC	rock core
ST	slotted tube
TO	thin-walled, open
TP	thin-walled, piston
WS	wash sample

### II. PENETRATION RESISTANCES

**Dynamic Penetration Resistance:** The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

**Standard Penetration Resistance, *N*:** The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH	sampler advanced by static weight—weight, hammer
PH	sampler advanced by pressure—pressure, hydraulic
PM	sampler advanced by pressure—pressure, manual

### III. SOIL DESCRIPTION

#### (a) *Cohesionless Soils*

<i>Relative Density</i>	<i>N, blows/ft.</i>
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) *Cohesive Soils*

<i>Consistency</i>	<i>c<sub>u</sub>, lb./sq. ft.</i>
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

### IV. SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	combined analysis, sieve and hydrometer <sup>1</sup>
Q	undrained triaxial <sup>2</sup>
R	consolidated undrained triaxial <sup>2</sup>
S	drained triaxial
U	unconfined compression
V	field vane test

### NOTES:

<sup>1</sup>Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

<sup>2</sup>Undrained triaxial tests in which pore pressures are measured are shown as  $\bar{Q}$  or  $\bar{R}$ .



## LIST OF SYMBOLS

### I. GENERAL

$\pi$	= 3.1416
$e$	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of $a$
$\log_{10} a$ or $\log a$	logarithm of $a$ to base 10
$t$	time
$g$	acceleration due to gravity
$V$	volume
$W$	weight
$M$	moment
$F$	factor of safety

### II. STRESS AND STRAIN

$u$	pore pressure
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\bar{\sigma}$ is also used)
$\tau$	shear stress
$\epsilon$	linear strain
$\epsilon_{xy}$	shear strain
$\nu$	Poisson's ratio ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

#### (a) Unit weight

$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	unit weight of submerged soil
$G_s$	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
$e$	void ratio
$n$	porosity
$w$	water content
$S_r$	degree of saturation

#### (b) Consistency

$w_L$	liquid limit
$w_P$	plastic limit
$I_P$	plasticity index
$w_S$	shrinkage limit
$I_L$	liquidity index = $(w - w_P) / I_P$
$I_C$	consistency index = $(w_L - w) / I_P$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$D_r$	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

#### (c) Permeability

$h$	hydraulic head or potential
$q$	rate of discharge
$v$	velocity of flow
$i$	hydraulic gradient
$k$	coefficient of permeability
$j$	seepage force per unit volume

#### (d) Consolidation (one-dimensional)

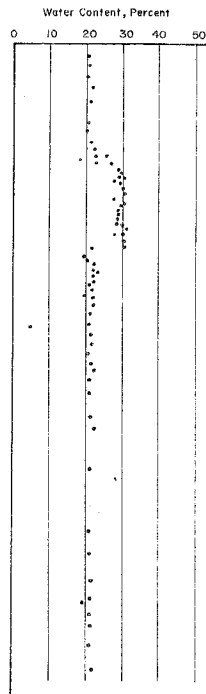
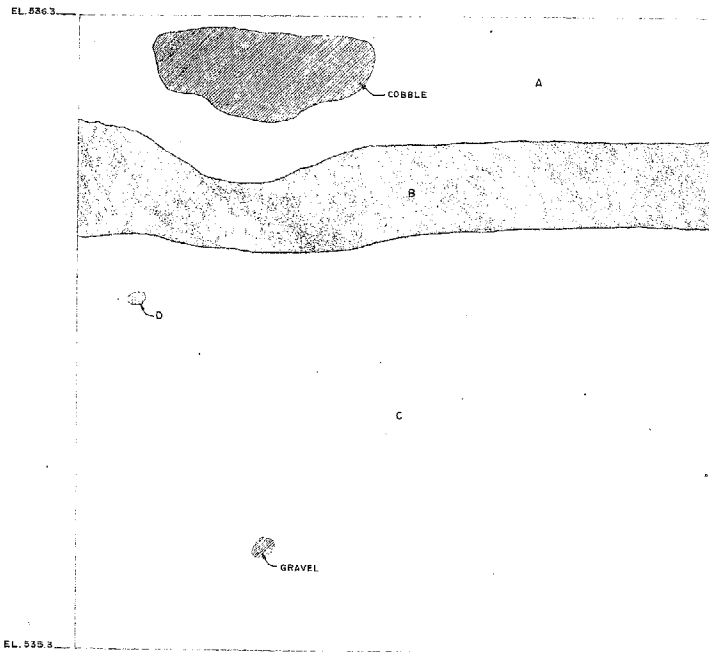
$m_v$	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
$C_c$	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
$c_s$	coefficient of consolidation
$T_v$	time factor = $c_v t / d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

#### (e) Shear strength

$\tau_f$	shear strength
$c'$	effective cohesion
$\phi'$	effective angle of shearing resistance, or friction
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\mu$	coefficient of friction
$S_c$	sensitivity

$\left. \begin{array}{l} \text{intercept} \\ \text{stress} \end{array} \right\} \text{in terms of effective}$   
 $\left. \begin{array}{l} \text{shearing resistance, or friction} \end{array} \right\} \tau_f = c' + \sigma' \tan \phi'$   
 $\left. \begin{array}{l} \text{apparent cohesion*} \\ \text{apparent angle of shearing resistance, or friction} \end{array} \right\} \text{in terms of total stress}$   
 $\left. \begin{array}{l} \text{shearing resistance, or friction} \end{array} \right\} \tau_f = c_u + \sigma \tan \phi_u$

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



LAYER	DESCRIPTION	W <sub>L</sub>	W <sub>p</sub>	W, Water content, percent	< 2 $\mu$ *
A	Reddish Brown Silty Clay	25.8	16.5	22.4, 19.5, 23.3, 20.2, 21.6, 20.4, 20.5, 20.4, 19.8, 22.1, 21.7, 20.5, 21.1, 25.5, 22.0.	27 %
B	Grey Silty Clay	34.9	19.4	31.0, 29.8, 28.7, 30.4, 28.8, 29.8, 29.4, 29.2, 27.7, 27.6, 28.8, 28.8, 30.1, 30.2, 30.2, 27.5, 29.9, 30.0, 30.3, 30.2, 29.2.	48 %
C	Reddish Brown Silty Clay	26.9 25.8	16.2 15.2	21.2, 21.4, 21.4, 21.6, 21.4, 22.5, 22.8, 19.8, 21.0, 20.6, 22.2, 22.4, 22.1, 21.6, 21.4, 21.1, 21.8, 19.2, 20.8, 21.2, 20.9, 20.4, 22.0, 22.8, 21.6, 21.0, 22.5, 21.2, 22.1, 22.2, 20.8, 21.2, 22.0, 27.9, 21.9.	30 % 26 %
D	Red Silt Pocket		4.0		

NOTE: FOR PHOTOGRAPH OF BLOCK SEE FIGURE 43.

\* < 2  $\mu$ , Percent less than 0.002 mm. in size.

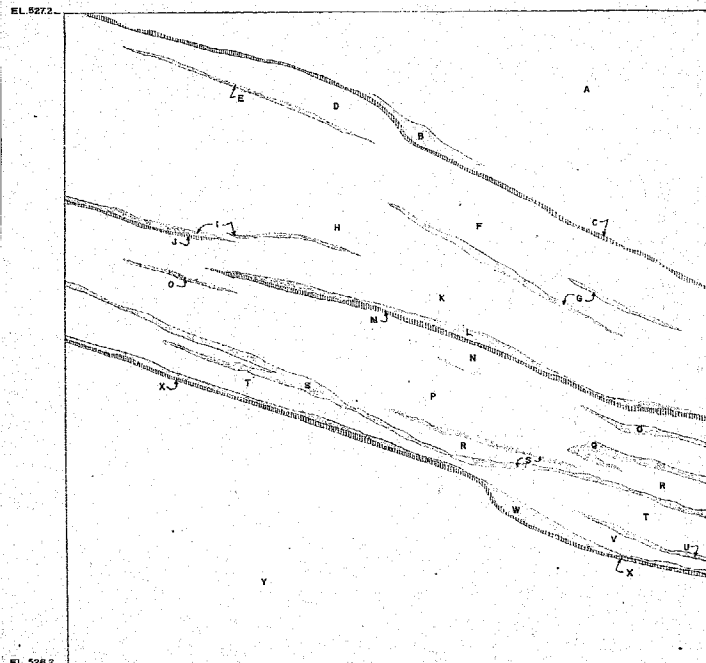
Water Content, Percent



LAYER	DESCRIPTION	WL	Wp	W. Water Content, percent	
A	Dark Brown Silty Clay			26.7	
B	Red Silty Clay	40.1	21.2	35.6, 29.1	
C	Grey Silty Clay	64.8	21.9	40.2, 45.1	
D	Dark Brown Silty Clay	34.2	17.9	30.6, 32.0	
E	Greyish Brown Silty Clay	52.6	20.6	41.8	
F	Dark Brown Silty Clay	35.2	17.9	30.8, 26.8, 25.6	
G	Reddish Brown Silty Clay	26.1	17.3	24.2, 22.8	
H	Dark Brown Silty Clay	32.3	16.3	27.6, 26.3	
I	Reddish Brown Silty Clay	25.4	16.6	25.4	
J	Dark Brown Silty Clay				
K	Reddish Brown Silty Clay	27.6	17.9	28.9, 23.5, 23.0, 27.6, 25.2, 22.4, 20.9, 24.4, 24.6, 26.5, 24.8, 24.9, 27.6, 26.1, 22.4, 19.1, 22.8, 23.0, 23.5, 22.6, 23.5, 21.0, 22.4, 24.8, 23.4, 19.1, 26.2, 23.0, 26.5, 26.3, 24.7, 24.4, 24.3, 24.6, 26.2, 26.0, 28.9, 24.2, 24.5, 24.4, 26.2, 26.6, 24.7, 24.6, 24.3, 22.9, 23.9, 23.5	47%
L	Dark Brown Silty Clay	35.4	19.1	30.2, 29.0, 31.2, 30.0, 30.8, 30.8, 26.6, 30.0, 31.4, 30.3	
M	Red Silty Clay	46.8	22.5	32.4, 28.2, 35.8, 36.8, 39.6, 37.5, 35.5, 30.6, 26.0, 36.4, 36.8, 35.6	75%
N	Grey Silty Clay	65.5	23.5	37.0, 36.8, 43.2, 44.0, 48.6, 41.0, 51.0, 35.2, 37.8, 39.4	76%
O	Grey Clay and Silt	31.1	16.8	22.3, 26.0, 26.2, 26.2, 30.2, 29.4	38%
P	Greyish Brown Silty Clay	61.0	22.7	37.4, 37.1, 38.4, 43.1, 37.2, 42.1, 40.4, 29.4, 32.5, 26.9, 45.5, 43.7	
Q	Reddish Brown Silty Clay	29.4	17.6	24.8, 20.2, 23.5, 24.8, 23.0, 22.8, 26.8, 25.3, 25.5, 26.5, 18.3, 25.7, 25.0, 25.3	34%
R	Brown Silty Clay			28.0, 25.0, 25.0, 27.0, 25.6, 28.0, 25.4, 25.6, 28.4, 28.5, 26.2, 25.2, 28.9, 24.5, 26.4, 21.8, 27.0, 25.8, 24.9, 26.7, 29.2, 26.6, 25.9	
S	Dark Brown Silty Clay	30.6	16.1	31.4, 31.1, 29.9, 28.9, 32.6, 33.0, 33.5, 35.9	32%
T	Brown Silty Clay	35.1	16.4	27.4, 26.3, 20.4, 26.0, 26.9, 27.1	42%
U	Dark Brown Silty Clay	39.1	16.2	27.0, 31.2, 26.7, 27.5, 27.4	45%
V	Reddish Brown Silty Clay			24.4, 24.8, 24.1, 24.0	
W	Dark Brown Silty Clay			32.4, 28.1, 33.2, 29.0, 32.5, 30.3, 31.4	
X	Red Silty Clay			36.7, 37.0, 34.2, 33.5, 40.0, 33.5, 32.4	
Y	Grey Silty Clay			38.6, 47.4, 36.2, 34.2, 35.5, 33.5	
Z	Greyish Brown Silty Clay			33.5, 33.4, 36.4, 37.6, 37.5, 33.6, 37.7, 33.1, 33.7, 34.9, 27.0, 37.2, 37.1, 38.6	
Az	Reddish Brown Silty Clay	32.0	19.0		47%
Bz	Brown Silty Clay			32.4	
Cz	Dark Brown Silty Clay	41.2	19.6	36.0, 36.6	47%
Dz	Red Silty Clay	52.8	23.8	41.0, 40.2	83%
Ez	Grey Silty Clay	64.5	23.7	41.6, 42.4	83%
Fz	Greyish Brown Silty Clay				
Gz	Dark Brown Silty Clay			27.6	

NOTE: FOR PHOTOGRAPH OF BLOCK SEE FIGURE 4-8.

\* < 2μ, Percent less than 0.002 mm. in size.



Water Content, Percent

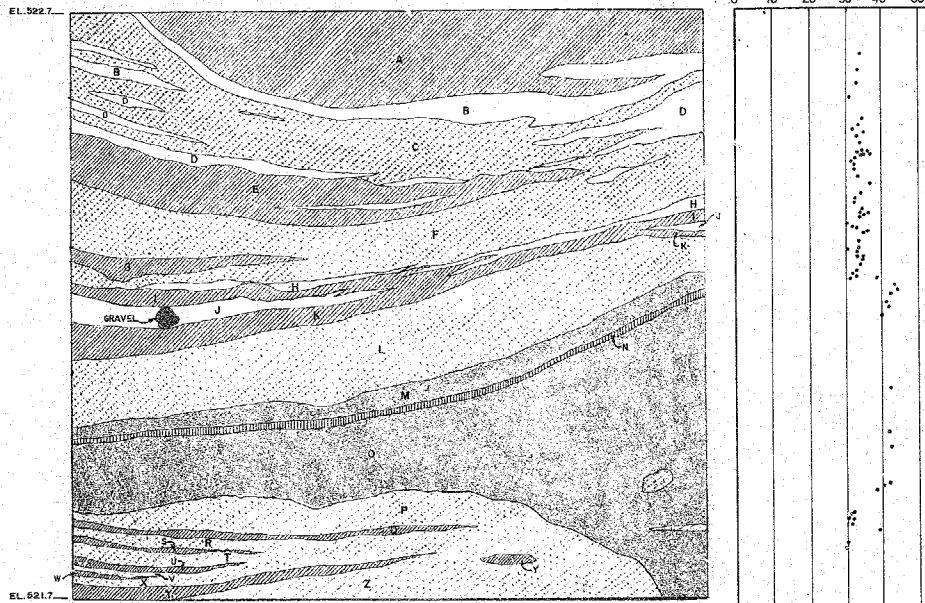
0 10 20 30 40 50

LAYER	DESCRIPTION	WL	Wp	W, Water content, percent	< 2 $\mu$ *
A	Reddish Brown Silty Clay	45.0	20.9	36.4, 36.4, 36.2, 35.0, 35.8, 35.5, 34.6, 34.7, 37.0, 36.6, 36.2, 36.4, 35.7, 35.6, 34.8, 36.0, 35.8, 35.2, 35.9, 35.3, 36.6, 36.5, 35.4, 34.9, 35.2, 31.4	67 %
B	Brownish Red Silt and Clay			35.8, 34.5, 33.0, 28.0	
C	Greyish Brown Silty Clay			36.1, 34.8, 34.6, 33.2, 32.2, 35.9, 37.3, 32.5, 34.0, 35.0	
D	Reddish Brown Silty Clay			36.2, 36.0, 36.0, 35.6, 34.8	
E	Brownish Red Silt and Clay			35.8, 31.2, 35.2, 36.6, 34.6, 33.8, 23.0, 35.8	
F	Reddish Brown Silty Clay	45.0	22.0	34.8, 34.7, 34.2, 35.0, 35.0, 35.4, 35.6, 34.4, 34.4	63 %
G	Brownish Red Silt and Clay			30.3, 29.4, 31.4	
H	Reddish Brown Silty Clay			34.9, 35.0	
I	Brownish Red Silt and Clay			27.9, 34.4, 33.3	
J	Greyish Brown Silty Clay			32.0	
K	Reddish Brown Silty Clay			33.4, 33.1, 33.5, 33.0, 34.5	
L	Brownish Red Silt and Clay			30.6, 26.9	
M	Greyish Brown Silty Clay			32.0, 34.4	
N	Reddish Brown Silty Clay			34.4, 34.0, 33.6	
O	Brownish Red Silt and Clay			34.8, 33.0	64 %
P	Reddish Brown Silty Clay			35.2, 36.2, 34.4, 33.8, 36.0, 33.4, 35.0	
Q	Brownish Red Silt and Clay			33.5, 30.8, 33.4	
R	Reddish Brown Silty Clay			34.7, 33.9, 33.5, 35.4	
S	Brownish Red Silt and Clay			28.0, 25.4, 35.8, 33.2, 33.1, 36.2	
T	Reddish Brown Silty Clay	43.7	21.8	32.0, 34.2, 34.4, 33.6, 33.8, 33.6, 35.1	
U	Brownish Red Silt and Clay				
V	Reddish Brown Silty Clay				
W	Brownish Red Silt and Clay			32.8, 32.9	
X	Greyish Brown Silty Clay			33.0, 32.2, 30.9, 35.0, 34.3, 36.4, 33.6	
Y	Reddish Brown Silty Clay	44.7	21.1	35.6, 33.0, 34.8, 35.5, 35.8, 35.4, 35.0, 35.4, 35.0	66 %

\* < 2  $\mu$ , Percent less than 0.002 mm. in size.

# DETAILED SOIL STRATIGRAPHY BLOCK SAMPLE H

FIGURE 40



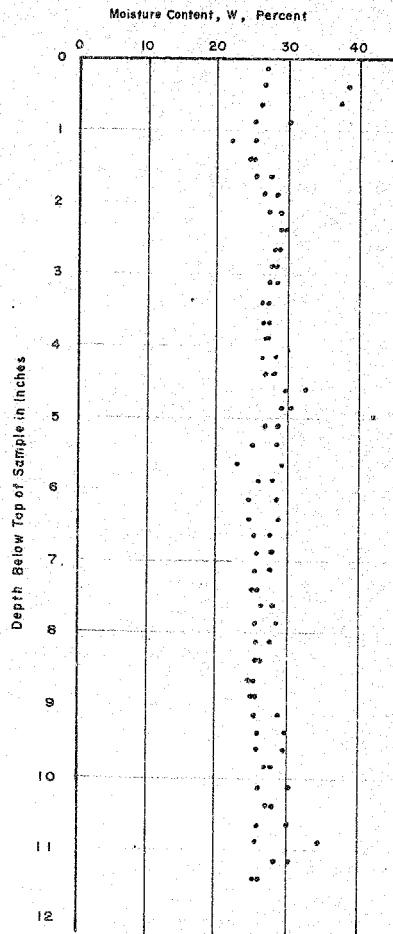
LAYER	DESCRIPTION	W <sub>L</sub>	W <sub>p</sub>	W, Water content, percent	< 2 $\mu$
A	Dark Brown Silty Clay			34.0, 33.2	
B	Brown Silty Clay			33.4, 34.8, 32.8, 31.7, 32.9	60 %
C	Reddish Brown Silty Clay	41.6	20.8		
D	Brown Silty Clay	41.3	19.5	33.4, 30.8, 34.2	63 %
E	Dark Brown Silty Clay	46.0	20.9	35.9, 32.8, 33.7, 34.2, 34.4, 36.2	65 %
F	Reddish Brown Silty Clay	39.8	17.1	31.1, 32.8, 32.1, 32.1, 36.2, 32.4	63 %
G	Dark Brown Silty Clay			33.4	
H	Brown Silty Clay	41.0	20.7	32.4, 32.2, 33.9	54 %
I	Dark Brown Silty Clay			33.7, 34.3, 35.9, 34.6	
J	Brown Silty Clay			32.6, 30.2, 33.2, 31.6	
K	Dark Brown Silty Clay			34.6, 35.7	
L	Reddish Brown Silty Clay	42.2	21.4	30.4, 34.6, 33.8, 32.3, 31.5, 33.4, 33.3, 32.9, 34.6, 32.8, 30.8	60 %
M	Grey Silty Clay			38.5, 32.5, 42.0	
N	Dark Grey Silty Clay	42.1	20.6	44.0, 40.4, 43.2	53 %
O	Grey Silty Clay	50.4	24.0	38.2, 41.4, 41.1, 41.4, 40.6, 41.5, 41.8, 42.2, 41.3	80 %
P	Reddish Brown Silty Clay	38.6	18.6	31.0, 31.6, 31.6, 30.3, 31.2	54 %
Q	Dark Brown Silty Clay			38.7	
R	Reddish Brown Silty Clay			30.0	
S	Dark Brown Silty Clay			29.5	
T	Reddish Brown Silty Clay				
U	Dark Brown Silty Clay				
V	Reddish Brown Silty Clay				
W	Dark Brown Silty Clay				
X	Reddish Brown Silty Clay				
Y	Dark Brown Silty Clay				
Z	Reddish Brown Silty Clay				

NOTE: FOR PHOTOGRAPH OF BLOCK SEE FIGURE 44.

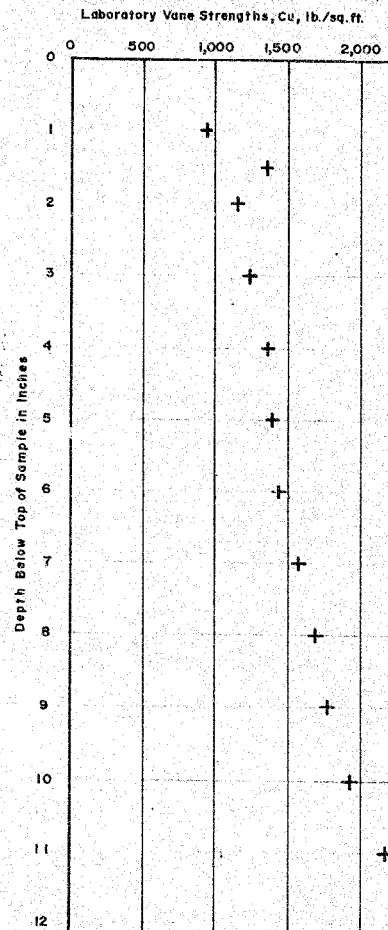
< 2  $\mu$ , Percent less than 0.002 mm. in size.

# SUMMARIZED SOIL PROPERTIES BLOCK J - ELEVATION 512.7

FIGURE 41



Moisture Contents	Block J Sample No.	Plastic Limit	Liquid Limit	* < 2 $\mu$
270				
267, 38.5				
260, 37.4				
251, 30.1				
250, 22.4				
242, 24.8				
251, 27.4	22	20.2	34.0	44%
264, 28.1	7	19.0	31.1	
270, 28.3				
291, 29.3	8	18.6	31.9	
280, 28.1				
285, 27.9				
276, 28.5	30	18.8	32.7	
270, 26.5	23	20.1	33.6	
265, 27.0				
269, 27.0				
259, 28.2				
264, 28.1	21	17.9	32.1	
296, 32.6				
307, 28.8				
263, 28.4				
247, 28.4				
228, 29.1				
259, 27.5				
247, 27.8				
247, 28.3				
250, 27.6				
237, 27.7				
275, 24.2	32	17.7	32.4	
257, 24.2				
263, 27.3	24	19.9	30.5	40%
255, 28.3				
256, 27.3				
256, 25.4				
253, 24.7				
249, 25.3				
250, 28.6				
258, 29.8	25	18.8	31.6	
255, 29.2	9	16.9	30.6	
268, 26.7				
254, 30.5	33	19.2	33.4	
277, 27.3	10	16.4	31.1	
257, 30.3				
252, 34.2				
283, 30.3	26	18.4	31.3	42%
252, 25.7				



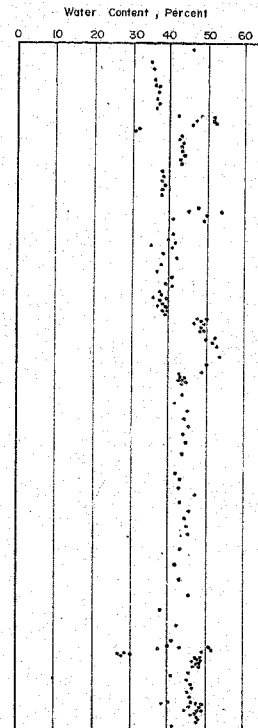
LABORATORY VANE - 0.5 IN. DIAMETER BY 0.5 IN. LONG  
VANE PENETRATION - 1.5 IN.

DEPTH FROM TOP, IN.	SHEAR STRENGTH LB./SQ. FT.	ROTATION AT FAILURE
1	980	30°
2	1160	17°
3	1240	13°
4	1350	10°
5	1380	8°
6	1430	9°
7	1570	10°
8	1650	10°
9	1740	12°
10	1910	10°
11	2140	17°

\* < 2  $\mu$ , Percent less than 0.002 mm. in size.

GOLDER & ASSOCIATES

Made *ML*  
Chkd: *ML*  
Appd: *ML*



LAYER	DESCRIPTION	W <sub>L</sub>	W <sub>p</sub>	W, Water content, percent	< 2 $\mu$ #
A	Brown Silty Clay	53.7	23.6	46.3, 35.4, 35.2	78 %
B	Red Silty Clay	54.9	22.9		
C	Grey Silty Clay	43.0	23.4	35.9, 37.3, 37.0, 36.9, 37.2, 36.5, 36.2	55 %
D	Light Grey Silty Clay	50.2	23.1	46.5, 51.5, 47.0, 46.0, 42.3, 32.3, 51.8	78 %
E	Brown Silty Clay			32.0, 31.2	
F	Red Silty Clay	52.5	25.0	43.2, 42.7, 43.7, 43.5, 43.8, 44.0, 42.9, 43.3	77 %
G	Grey Silty Clay	45.8	23.6	36.2, 36.6, 38.2, 38.8, 38.2, 38.1	60 %
H	Light Grey Silty Clay	59.0	25.2	47.8, 53.8, 45.0, 48.9, 40.7, 48.2	73 %
I	Brown Silty Clay				
J	Red Silty Clay	48.7	22.7	41.0, 39.8, 35.1, 42.6, 41.2, 38.4, 42.0, 37.8, 36.7, 40.7, 39.3, 40.9	72 %
K	Grey Silty Clay	48.7	23.5	37.4, 38.0, 38.6, 37.9, 38.3, 38.8, 39.2, 36.8, 38.4, 38.4, 39.1	67 %
L	Grey Brown Silty Clay			50.0, 48.3, 46.3, 49.3, 48.6, 48.2, 47.5, 48.5	78 %
M	Light Grey Brown Silty Clay	67.5	25.9	52.6, 49.6, 48.1, 52.5, 49.4, 53.3, 50.0	87 %
N	Dark Brown Silty Clay	54.5	25.3	44.2, 42.8, 42.6, 43.4, 44.3, 43.4, 42.9	81 %
O	Brown Silty Clay	52.6	22.7	43.4, 41.6, 44.8, 44.2, 45.1, 43.8, 44.3, 43.8	78 %
P	Red Silty Clay	54.8	25.9	42.0, 43.2, 42.4, 46.9, 42.9, 45.4, 44.2, 44.6, 45.0	73 %
Q	Grey Silty Clay	59.9	25.2	43.3, 41.8, 42.8, 45.4, 38.0, 42.5, 41.0	78 %
R	Light Grey Silty Clay			40.3, 42.9, 50.0, 37.2, 51.1, 49.0	60 %
S	Brown Silty Clay			26.9, 28.7, 30.0, 26.8	
T	Red Silty Clay	58.7	25.4	48.6, 46.2, 47.4, 46.5, 47.6, 48.1, 47.4, 48.2, 48.4	85 %
U	Grey Silty Clay	60.4	26.9	48.9, 40.9, 44.8, 46.0, 48.1, 45.0, 45.4	83 %
V	Brown Silty Clay			46.0, 40.2, 38.5, 45.7, 48.0, 48.6, 47.3	70 %
W	Red Silty Clay	55.9	24.8	48.8, 47.8, 47.7, 46.0, 44.2, 48.6, 47.1	80 %
				33.0, 41.2	

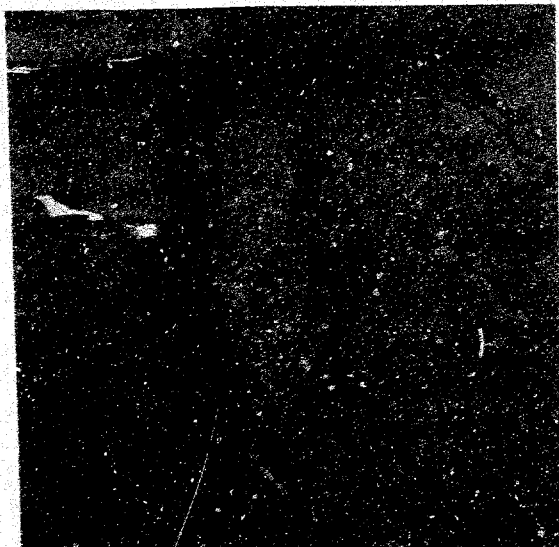
NOTE: FOR PHOTOGRAPH OF BLOCK SEE FIGURE 44.

< 2  $\mu$ , Percent less than 0.002 mm. in size.

PROJECT No. 6375

# PHOTOGRAPHS BLOCK SAMPLES

FIGURE 43



BLOCK B , ELEV. 548.7 TO 547.7 , REDDISH  
BROWN SILTY CLAY WITH POSSIBLE SLUMP  
ZONE.



BLOCK E , ELEV. 536.3 TO 535.3 , REDDISH  
BROWN SILTY CLAY WITH 2 INCH GREY MORE  
CLAYEY LAYER.

0 2 4 6 8 10 12  
SCALE , INCHES



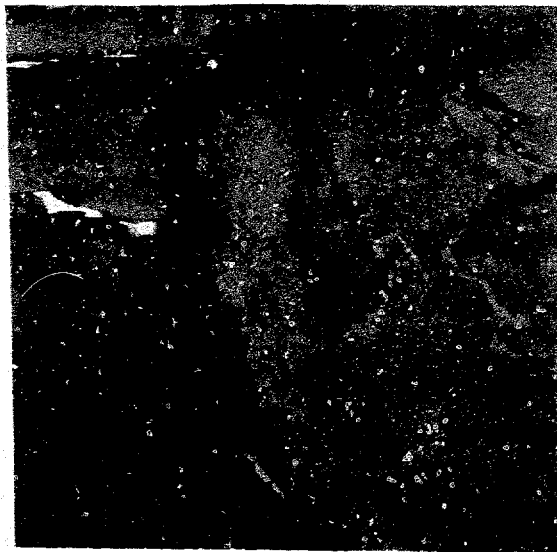
BLOCK F , ELEV. 531.7 TO 530.7 , LAYERED  
REDDISH - BROWN , GREYISH - BROWN , RED , GREY  
AND DARK BROWN SILTY CLAYS , WITH POSSIBLE  
FORMER SLIP PLANE.

GOLDER & ASSOCIATES



PHOTOGRAPHS  
BLOCK SAMPLES

FIGURE 43



BLOCK B , ELEV. 548.7 TO 547.7, REDDISH  
BROWN SILTY CLAY WITH POSSIBLE SLUMP  
ZONE.



BLOCK E , ELEV. 536.3 TO 535.3, REDDISH  
BROWN SILTY CLAY WITH 2 INCH GREY MORE  
CLAYEY LAYER.



BLOCK F , ELEV. 531.7 TO 530.7 , LAYERED  
REDDISH - BROWN, GREYISH - BROWN, RED, GREY  
AND DARK BROWN SILTY CLAYS, WITH POSSIBLE  
FORMER SLIP PLANE.

0 2 4 6 8 10 12  
SCALE , INCHES

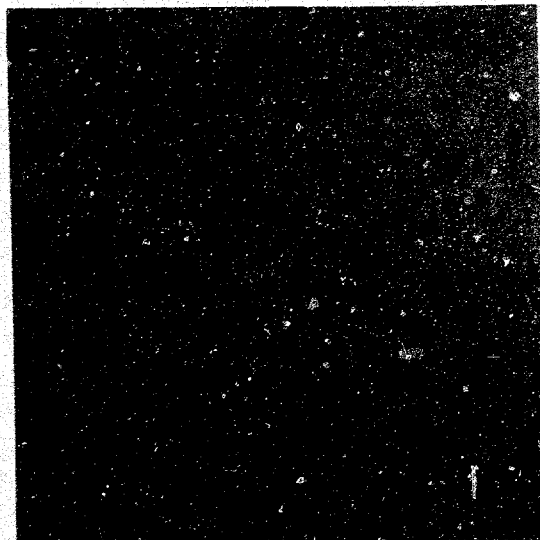
PROJECT No. 537.5

# PHOTOGRAPHS BLOCK SAMPLES

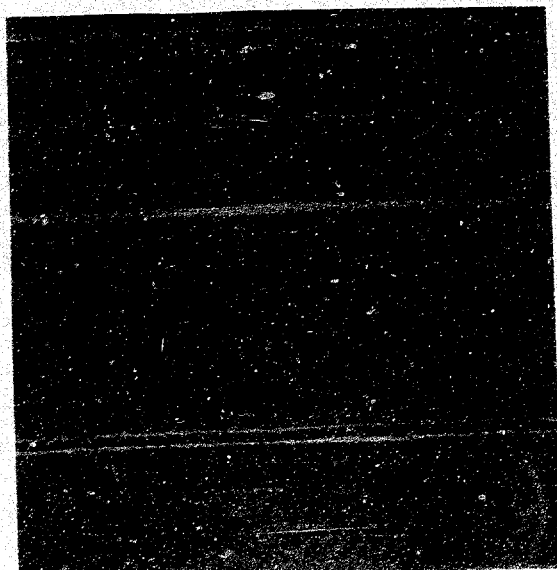
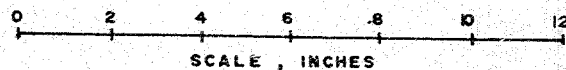
FIGURE 44



BLOCK H, ELEV. 522.7 TO 521.7, FAINTLY LAYERED  
REDDISH BROWN, BROWN AND DARK BROWN  
SILTY CLAY WITH DISTINCT ZONE OF GREY SILTY  
CLAY.



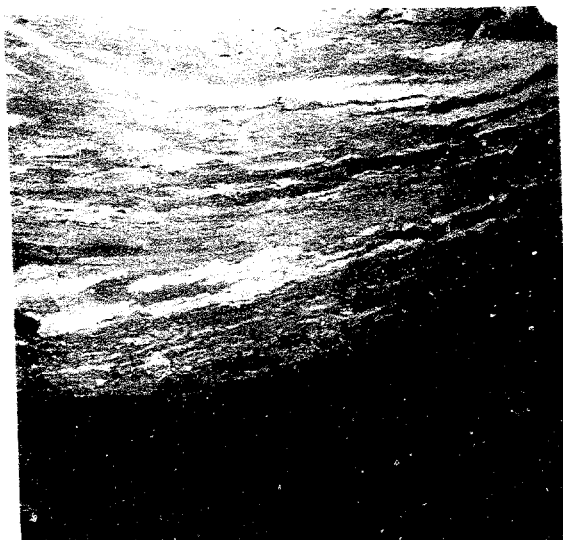
BLOCK I, ELEV. 517.7 TO 516.7, TYPICAL  
REDDISH BROWN HOMOGENEOUS SILTY CLAY.



BLOCK K, ELEV. 508.7 TO 507.7, LAYERED  
RED, BROWN, DARK BROWN AND GREY SILTY  
CLAY.

PHOTOGRAPHS  
BLOCK SAMPLES

FIGURE 44

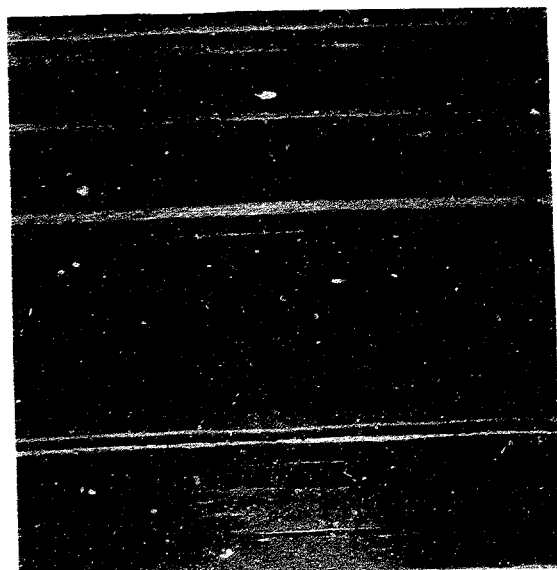


BLOCK H, ELEV. 522.7 TO 521.7, FAINTLY LAYERED  
REDDISH BROWN, BROWN AND DARK BROWN  
SILTY CLAY WITH DISTINCT ZONE OF GREY SILTY  
CLAY.



BLOCK I, ELEV. 517.7 TO 516.7, TYPICAL  
REDDISH BROWN HOMOGENEOUS SILTY CLAY.

0 2 4 6 8 10 12  
SCALE, INCHES

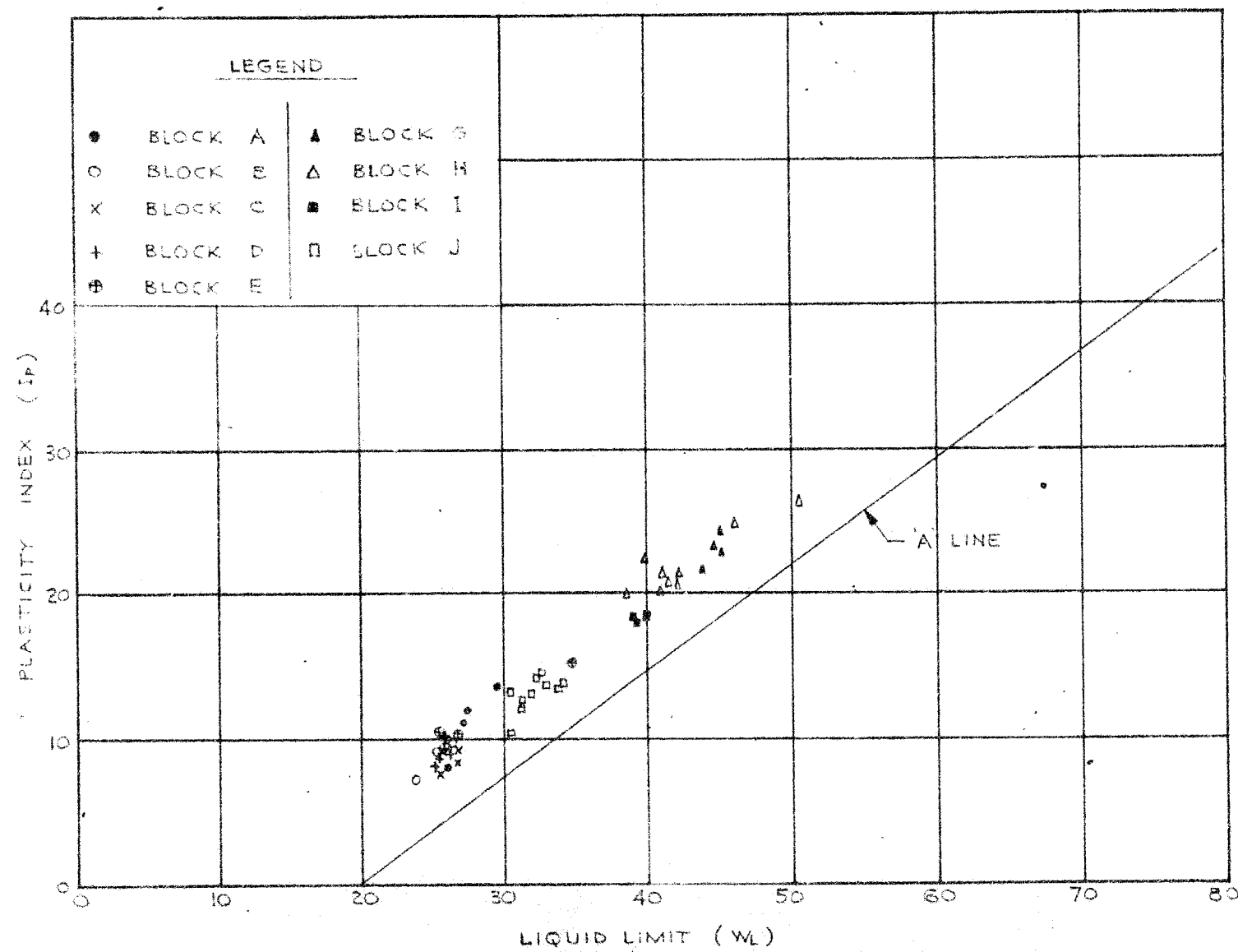


BLOCK K, ELEV. 508.7 TO 507.7, LAYERED  
RED, BROWN, DARK BROWN AND GREY SILTY  
CLAY.

PLASTICITY CHART  
BLOCKS A TO E AND G TO J INCLUSIVE

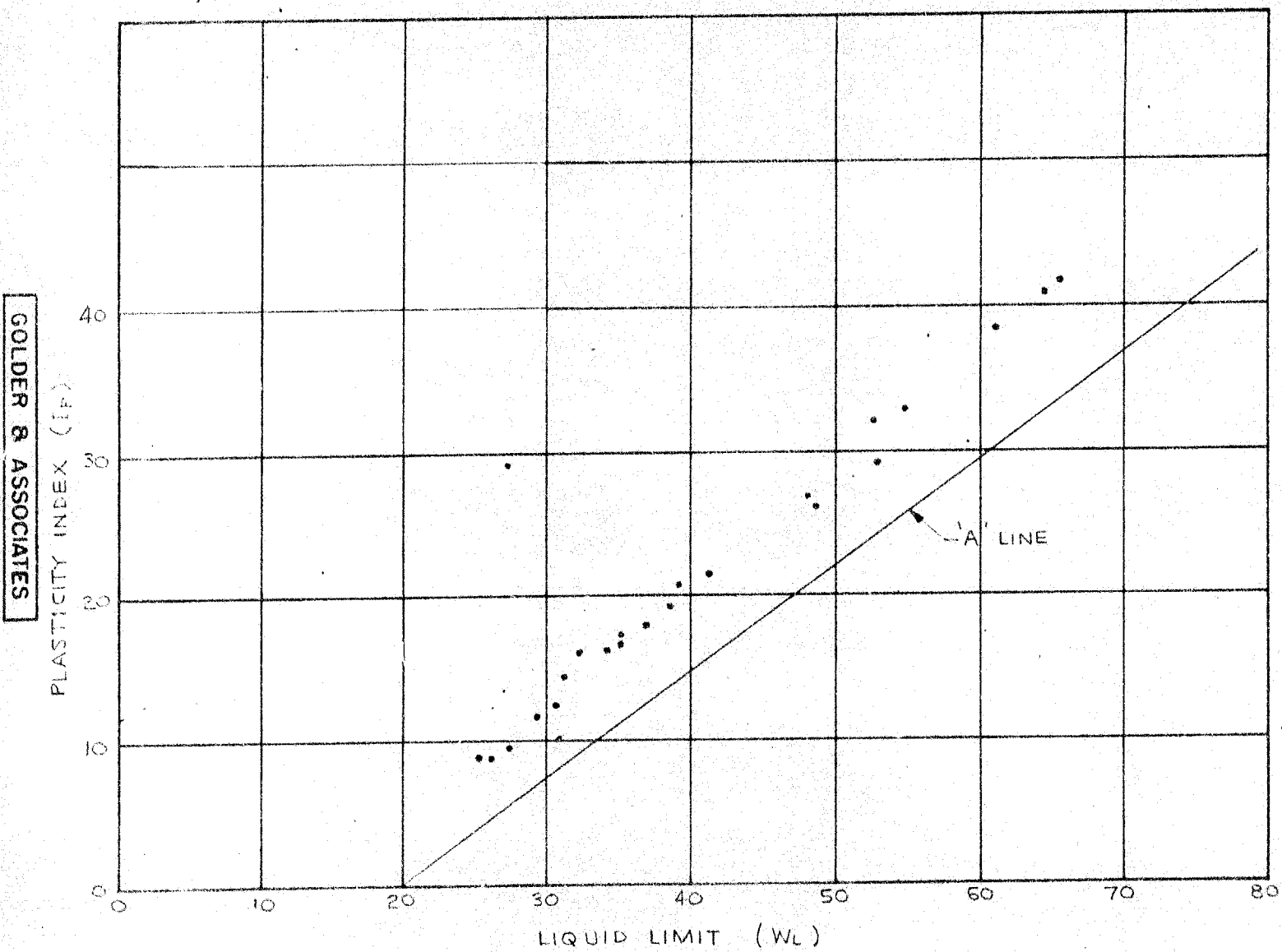
FIGURE 45

GOLDER & ASSOCIATES



PLASTICITY CHART  
BLOCK F

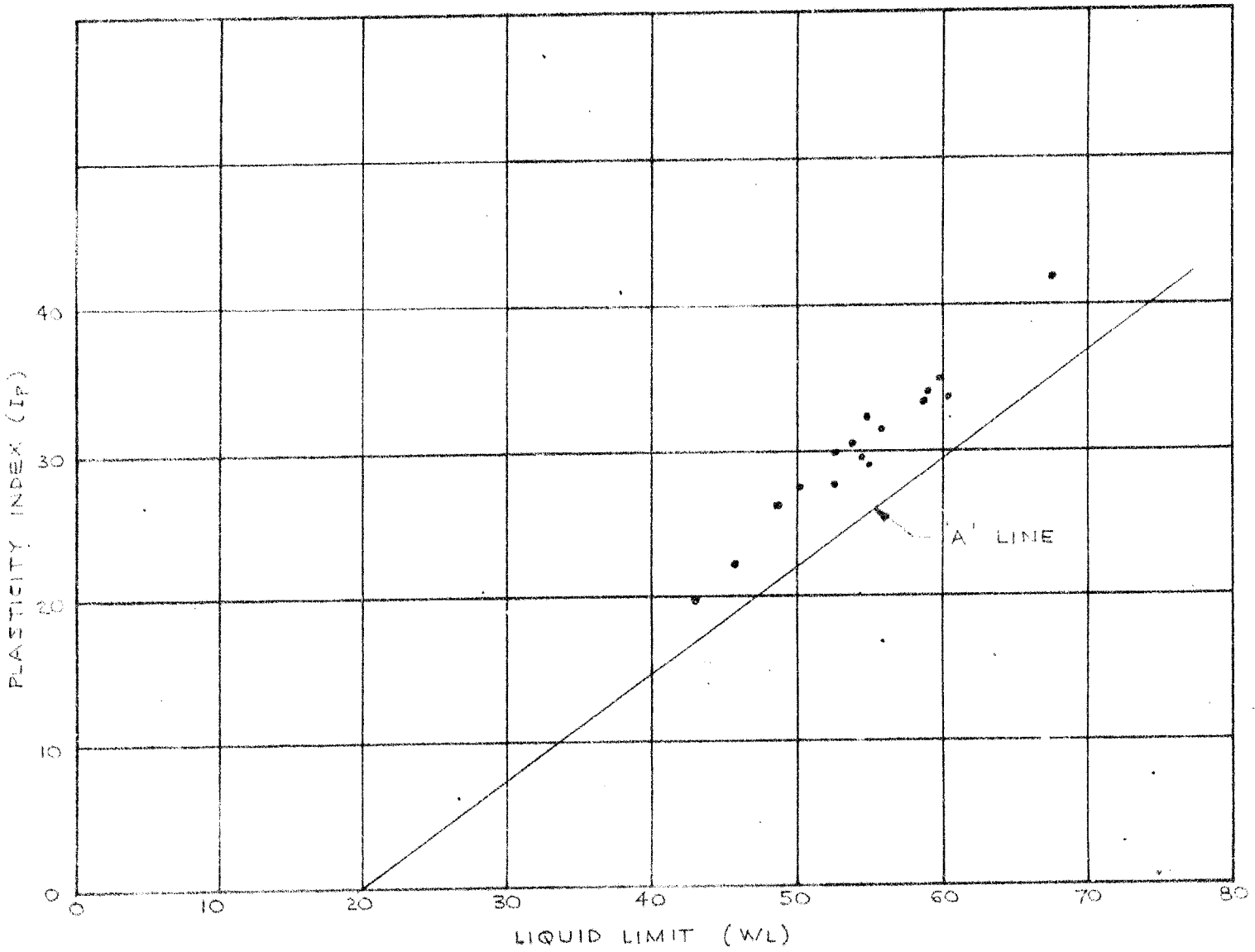
FIGURE 4c



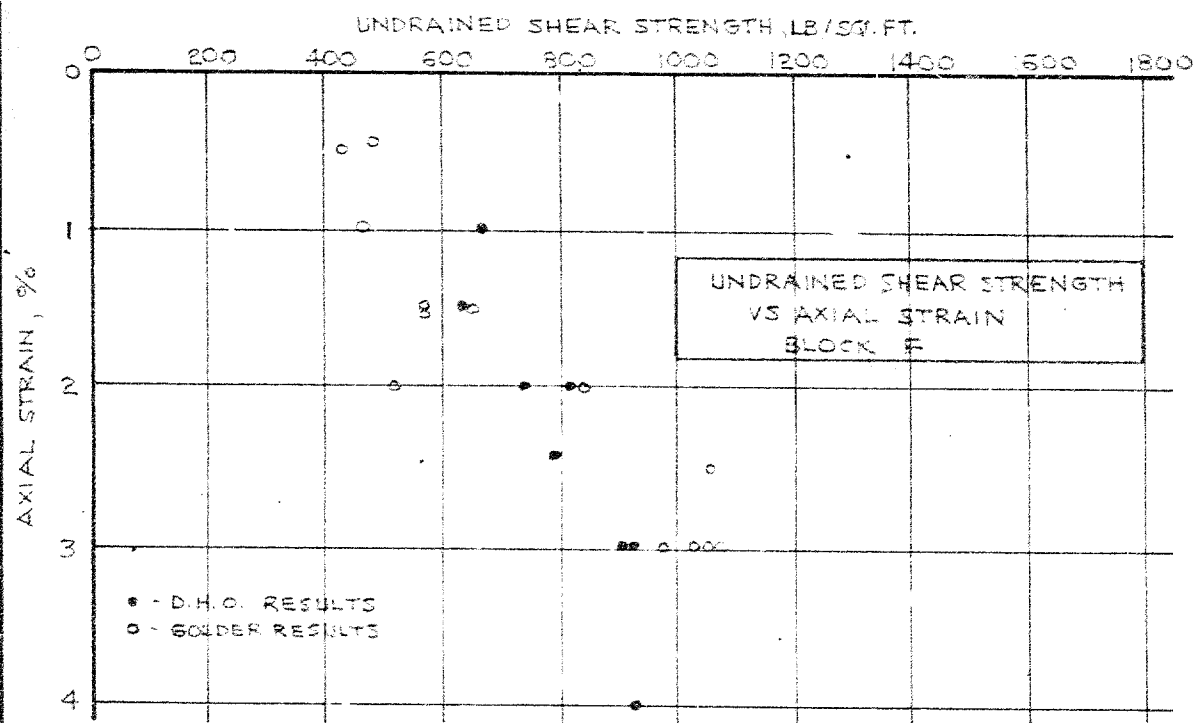
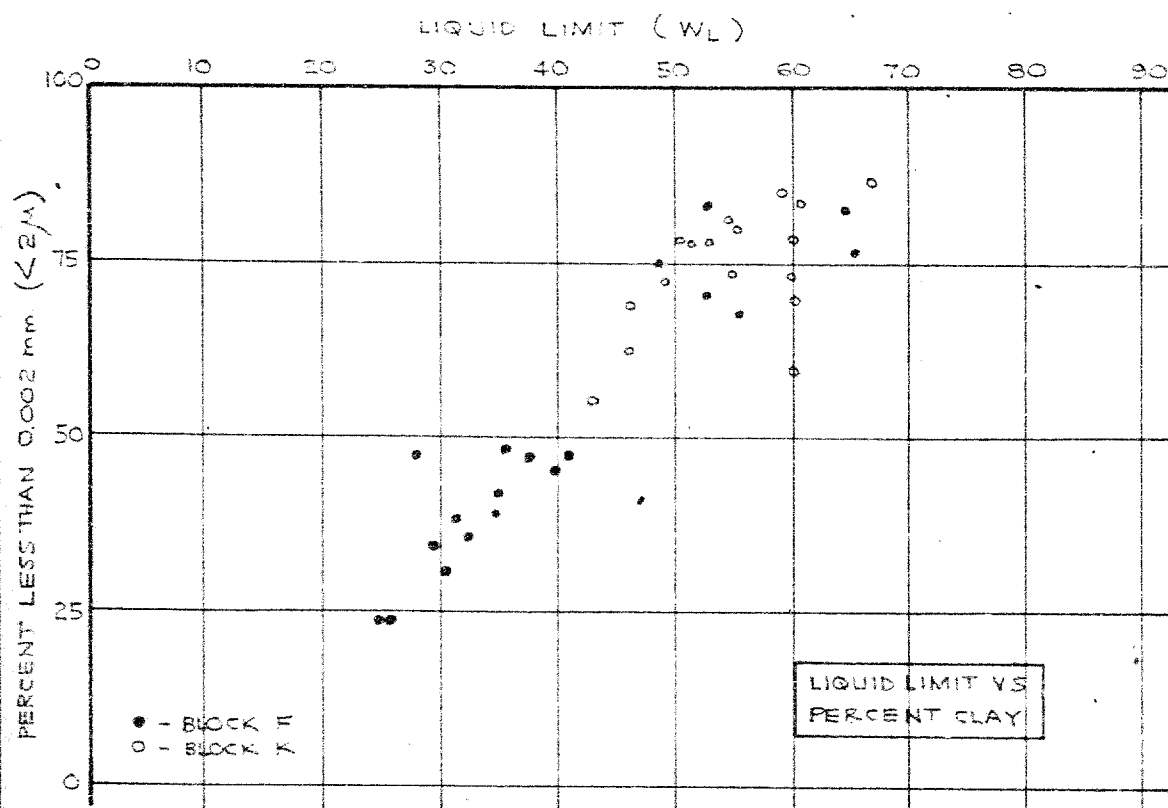
GOLDER & ASSOCIATES

PLASTICITY CHART  
BLOCK K

FIGURE 47



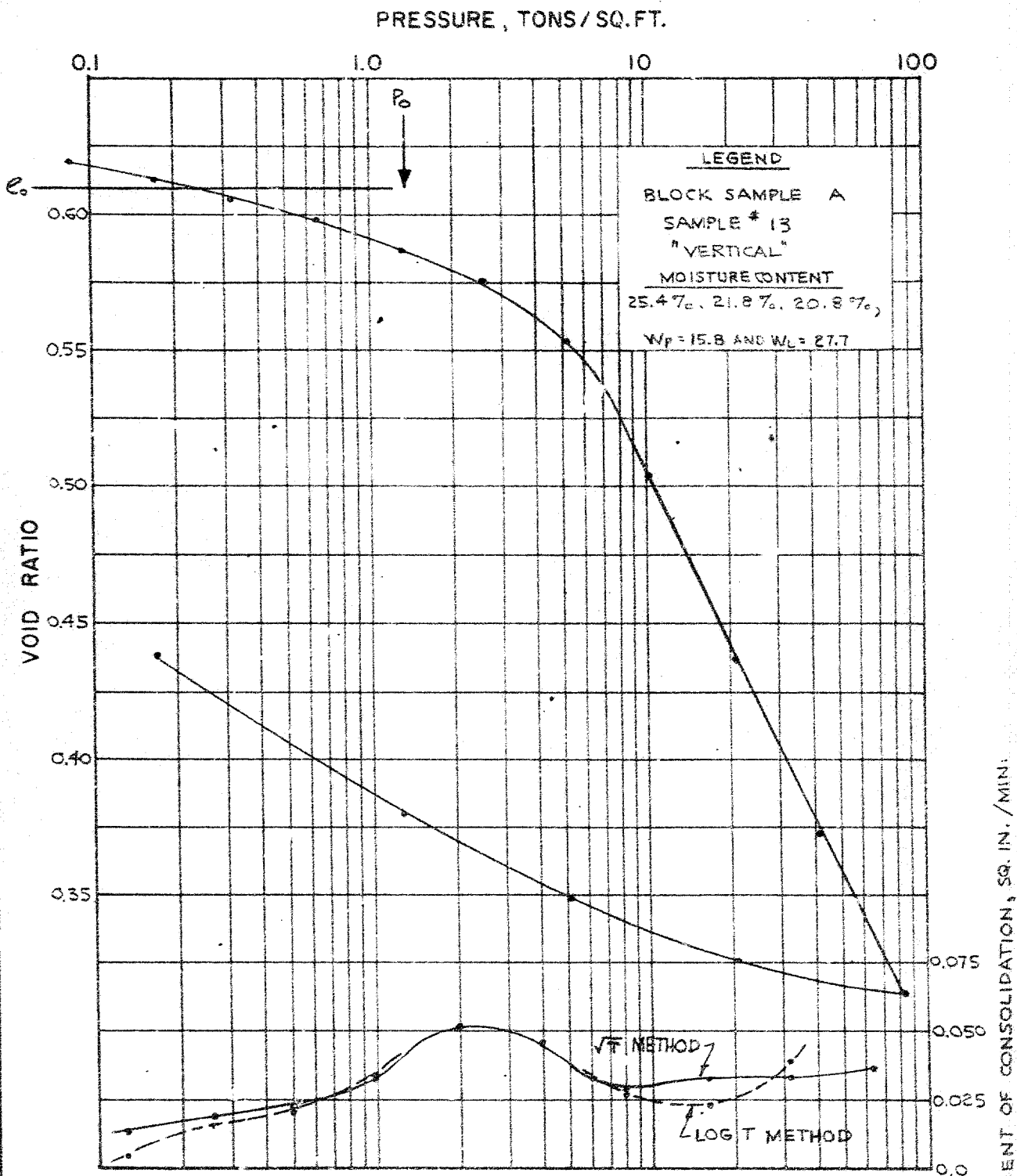
GOLDER & ASSOCIATES



PROJECT NO. ... 9312 ...

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST.

FIGURE 49



NOTE: SAMPLE DIAMETER 1.98"

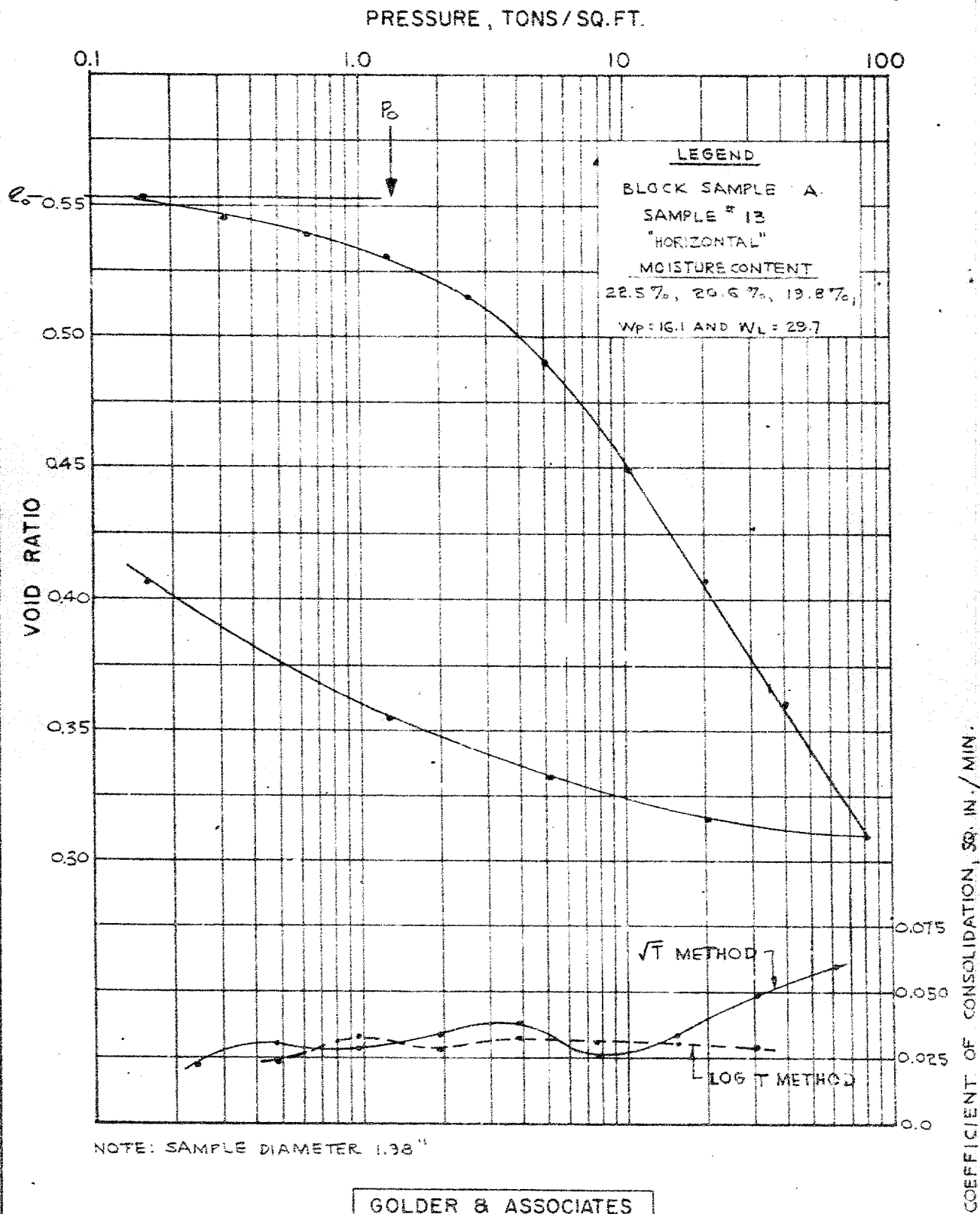
GOLDER & ASSOCIATES



PROJECT No. 57.75-1199

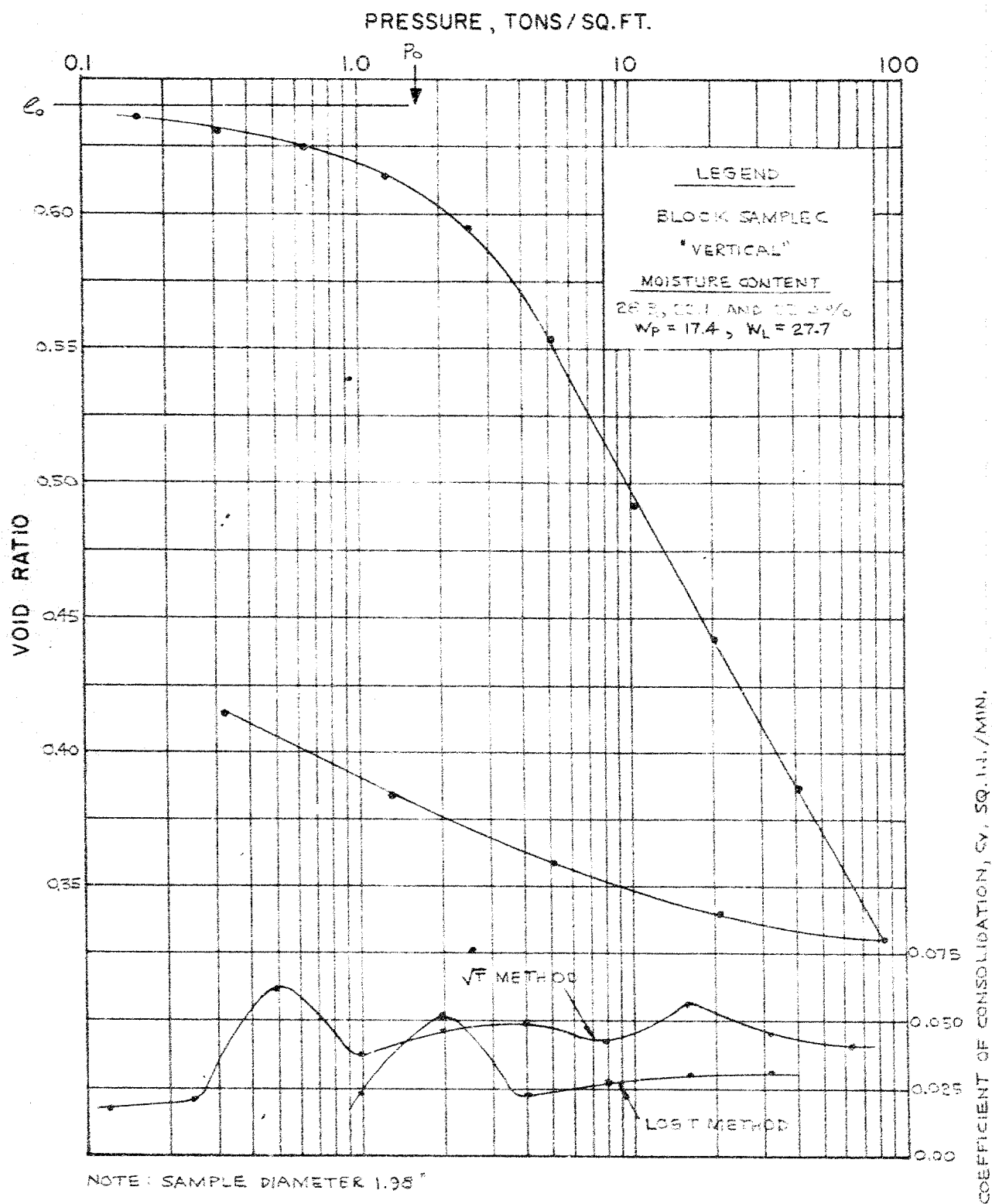
# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 50



# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

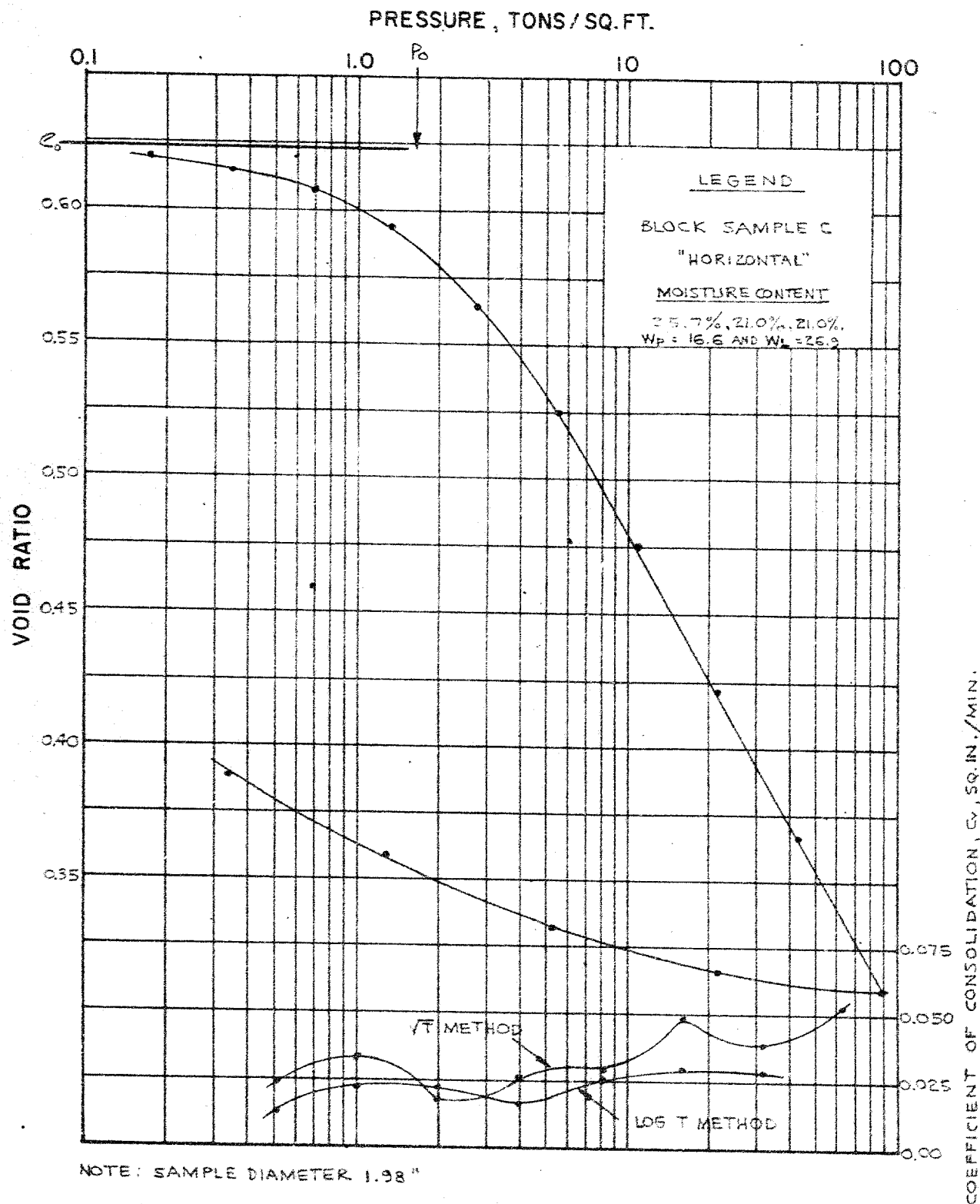
FIGURE 31



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

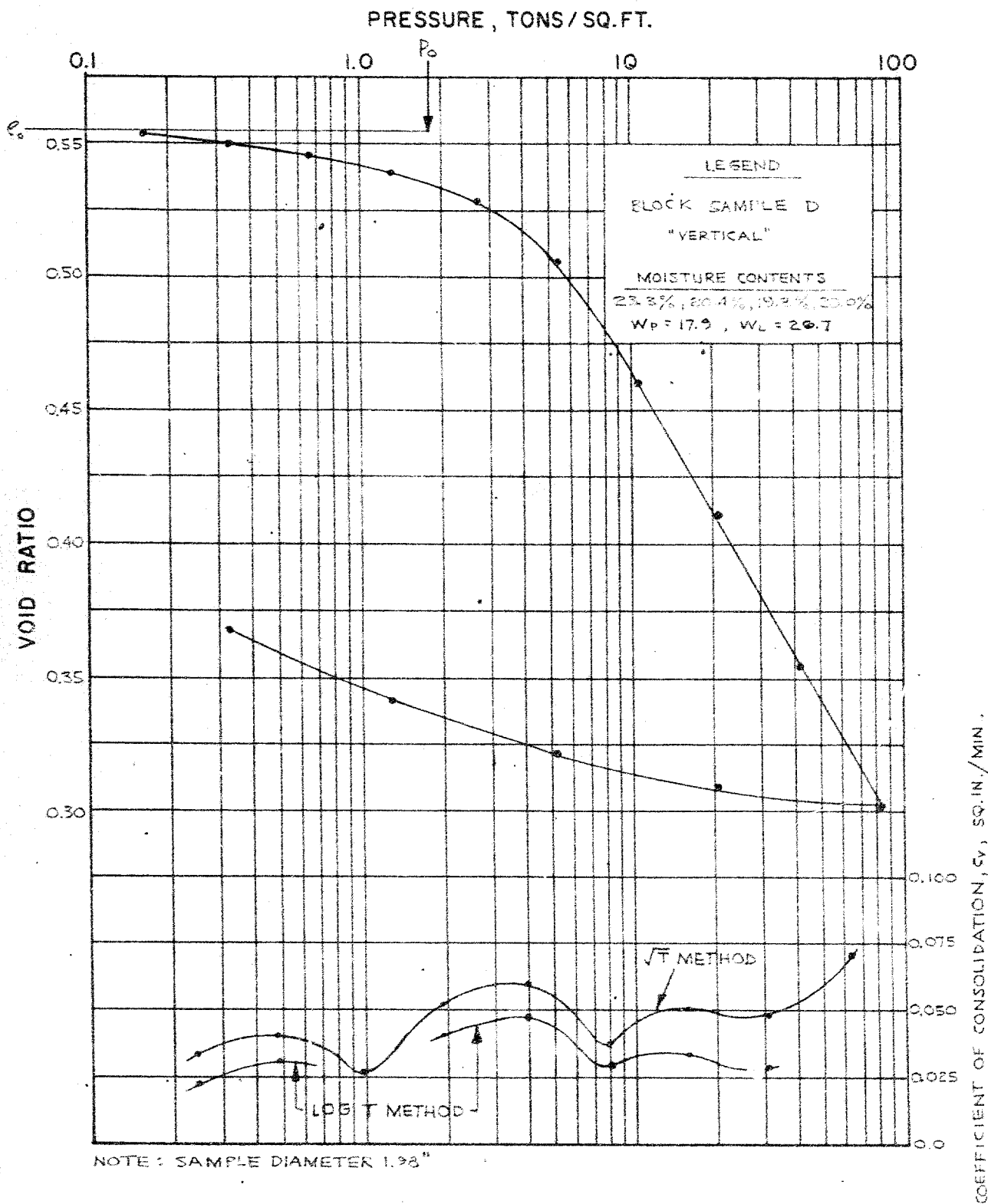
FIGURE 52



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

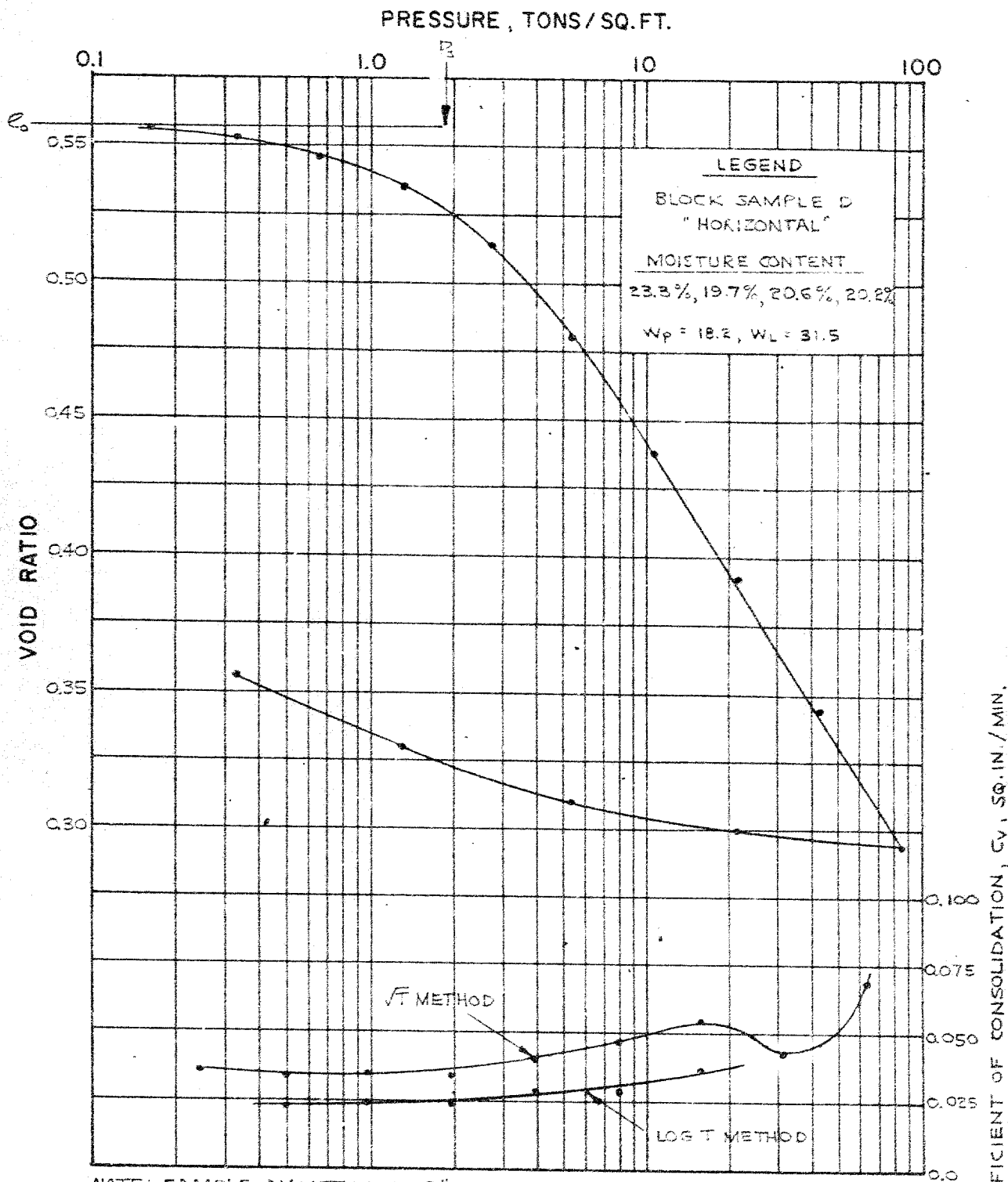
FIGURE 53



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

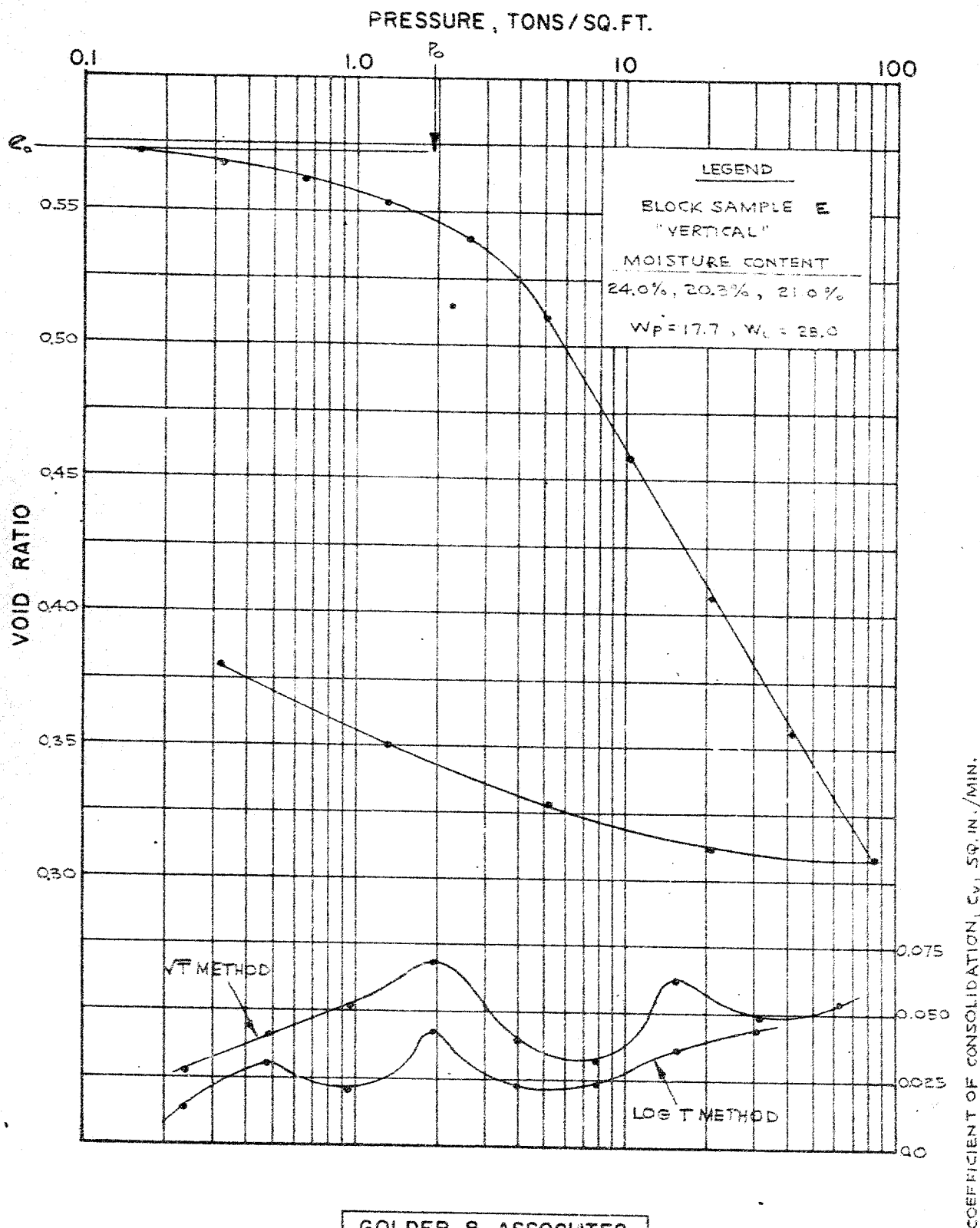
FIGURE 54



GOLDER & ASSOCIATES

VOID RATIO - PRESSURE CURVES  
CONSOLIDATION TEST

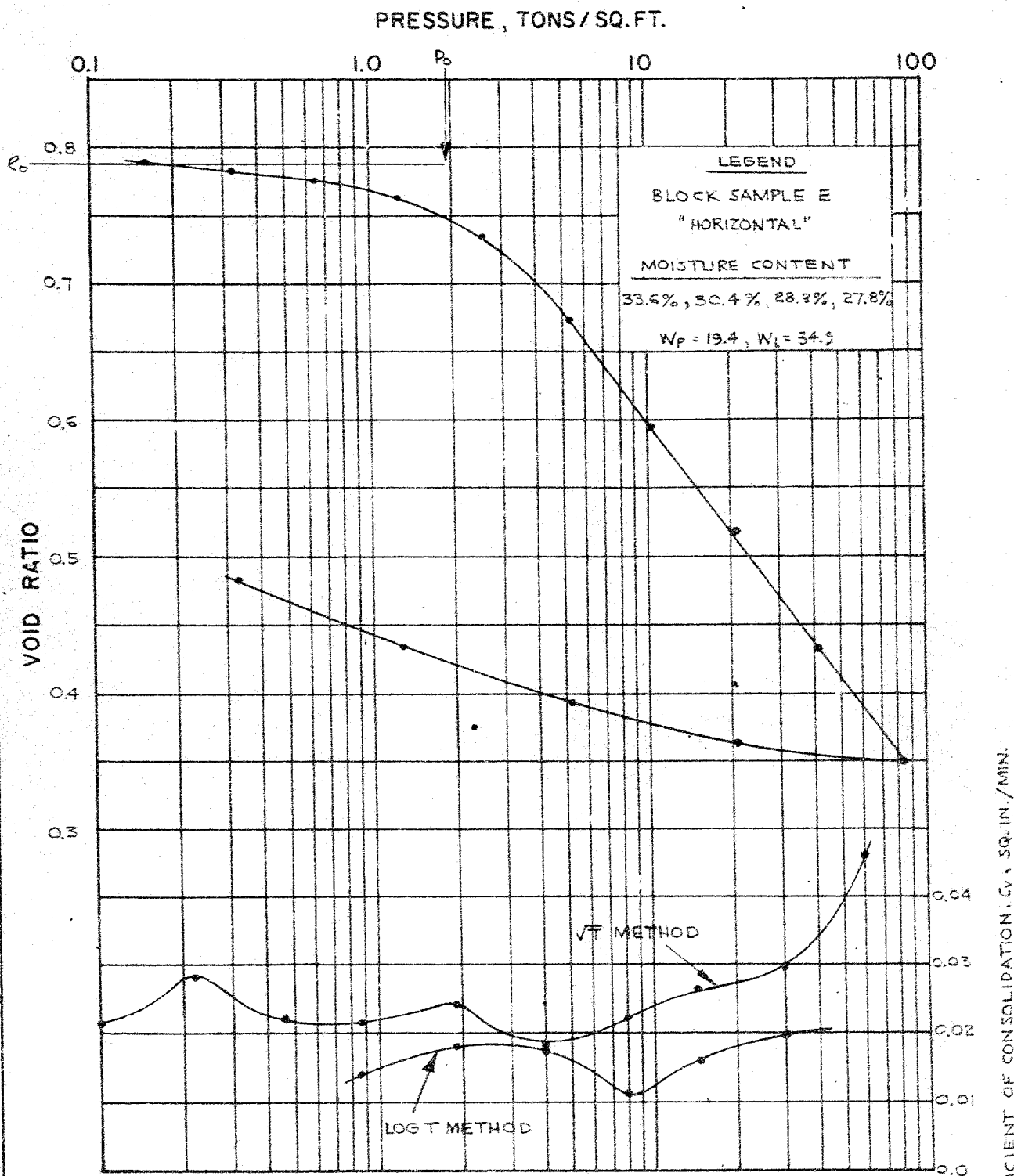
FIGURE 55



GOLDER &amp; ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 56



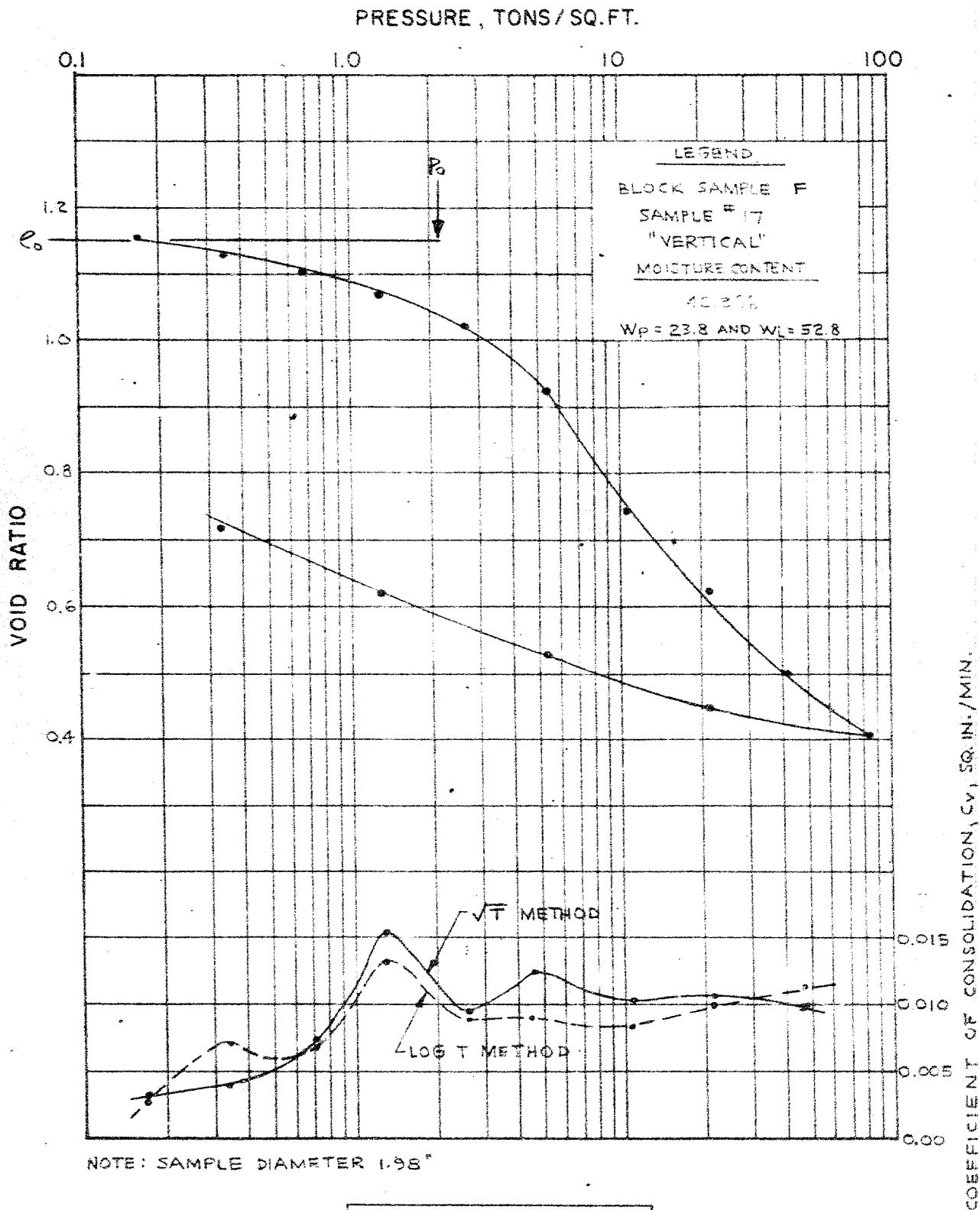
NOTE: SAMPLE DIAMETER 1.38"

GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 57

PROJECT No. 6374



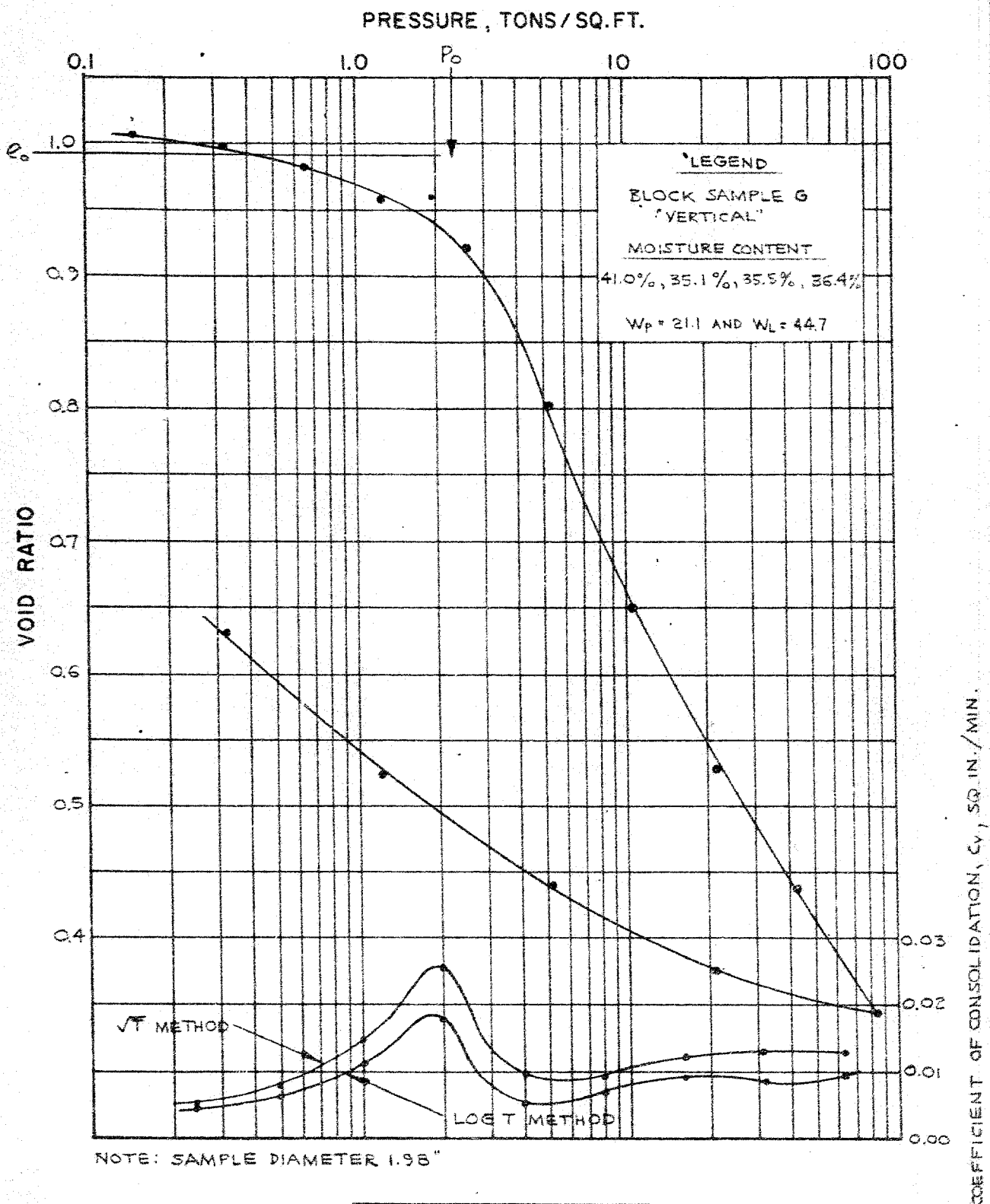
GOLDER & ASSOCIATES



PROJECT No. 6375

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

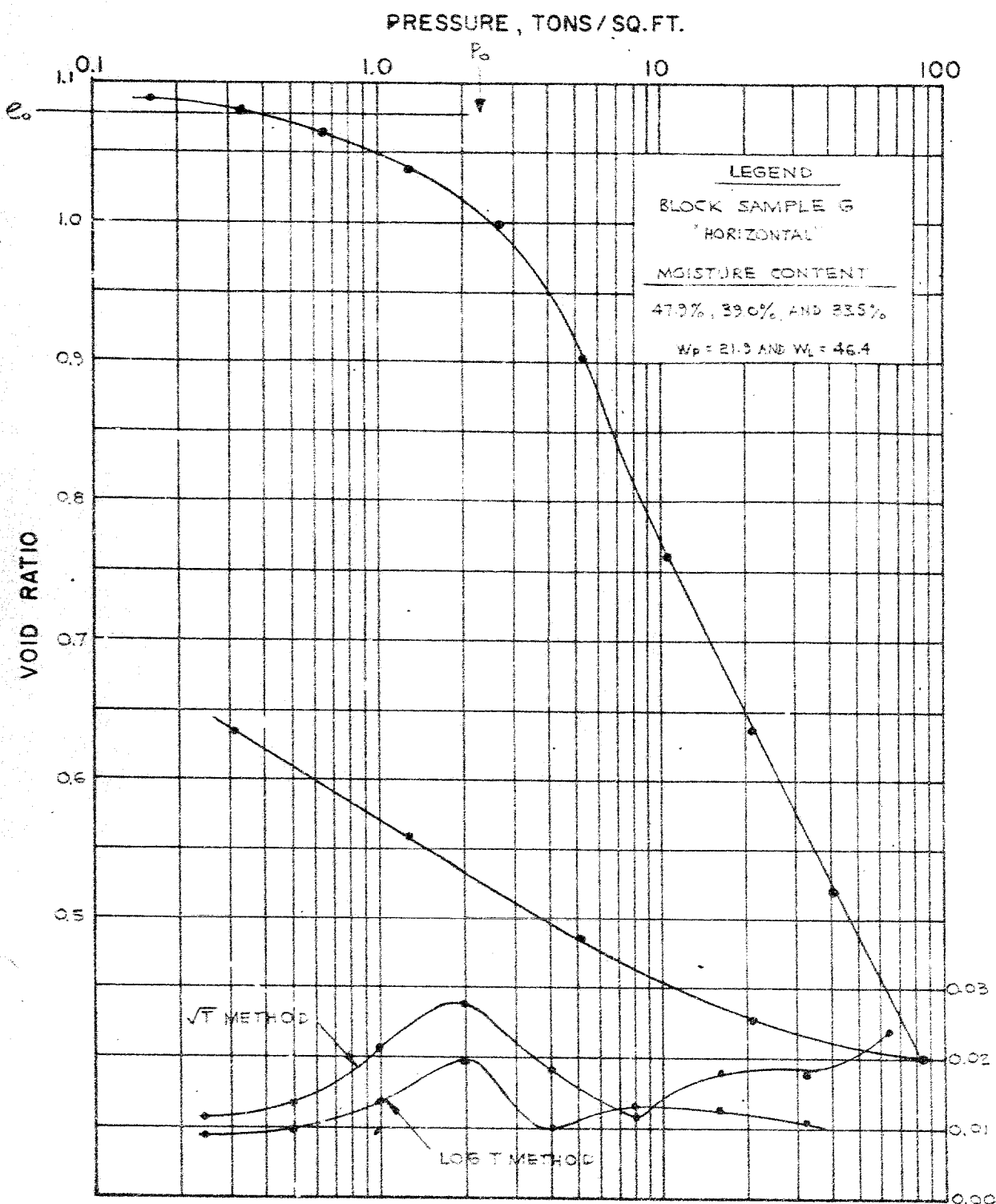
FIGURE 58



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 59

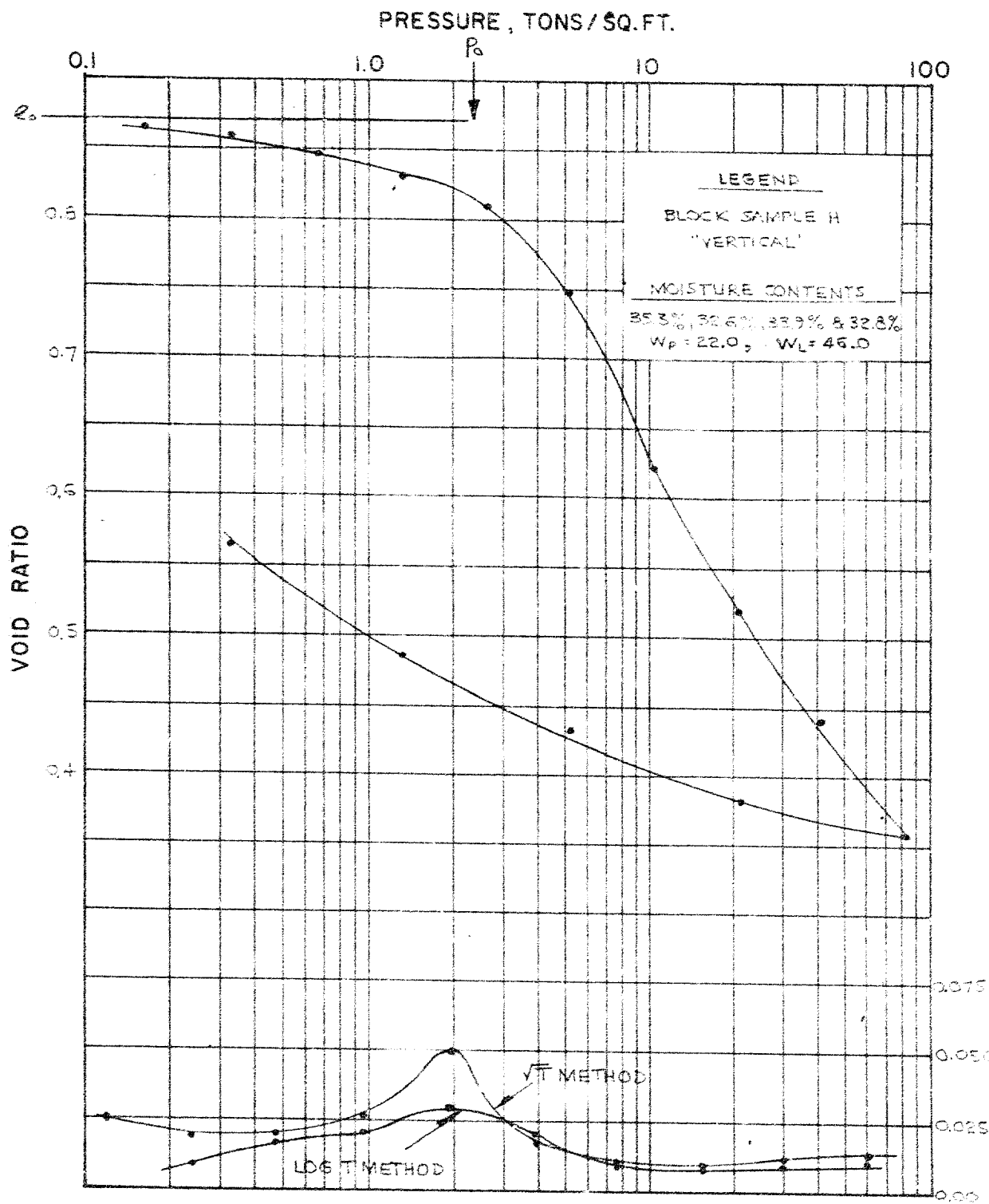


NOTE: SAMPLE DIAMETER 1.98"

GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 60

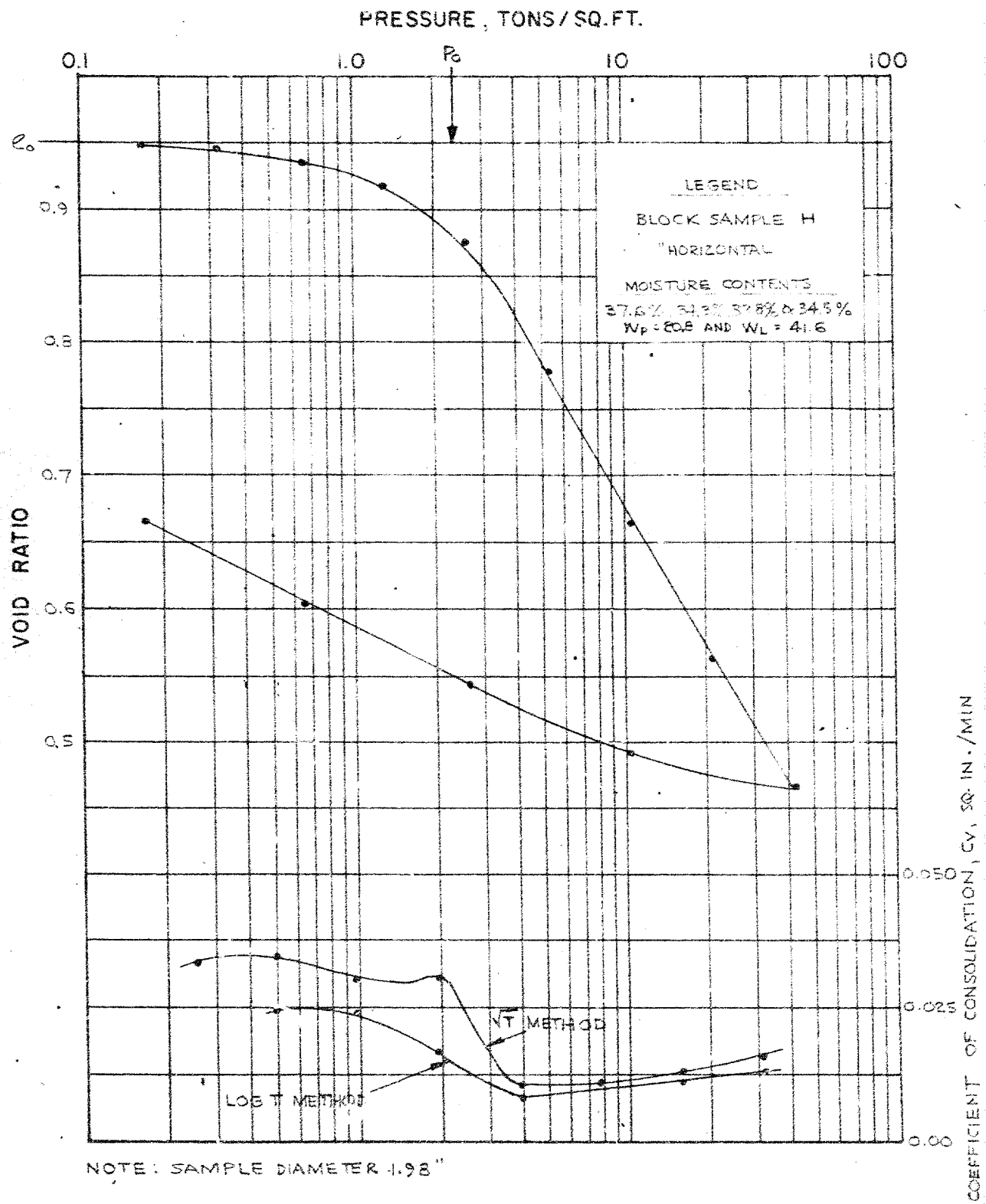


GOLDER & ASSOCIATES

PROJECT NO. 6-2-2-2

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

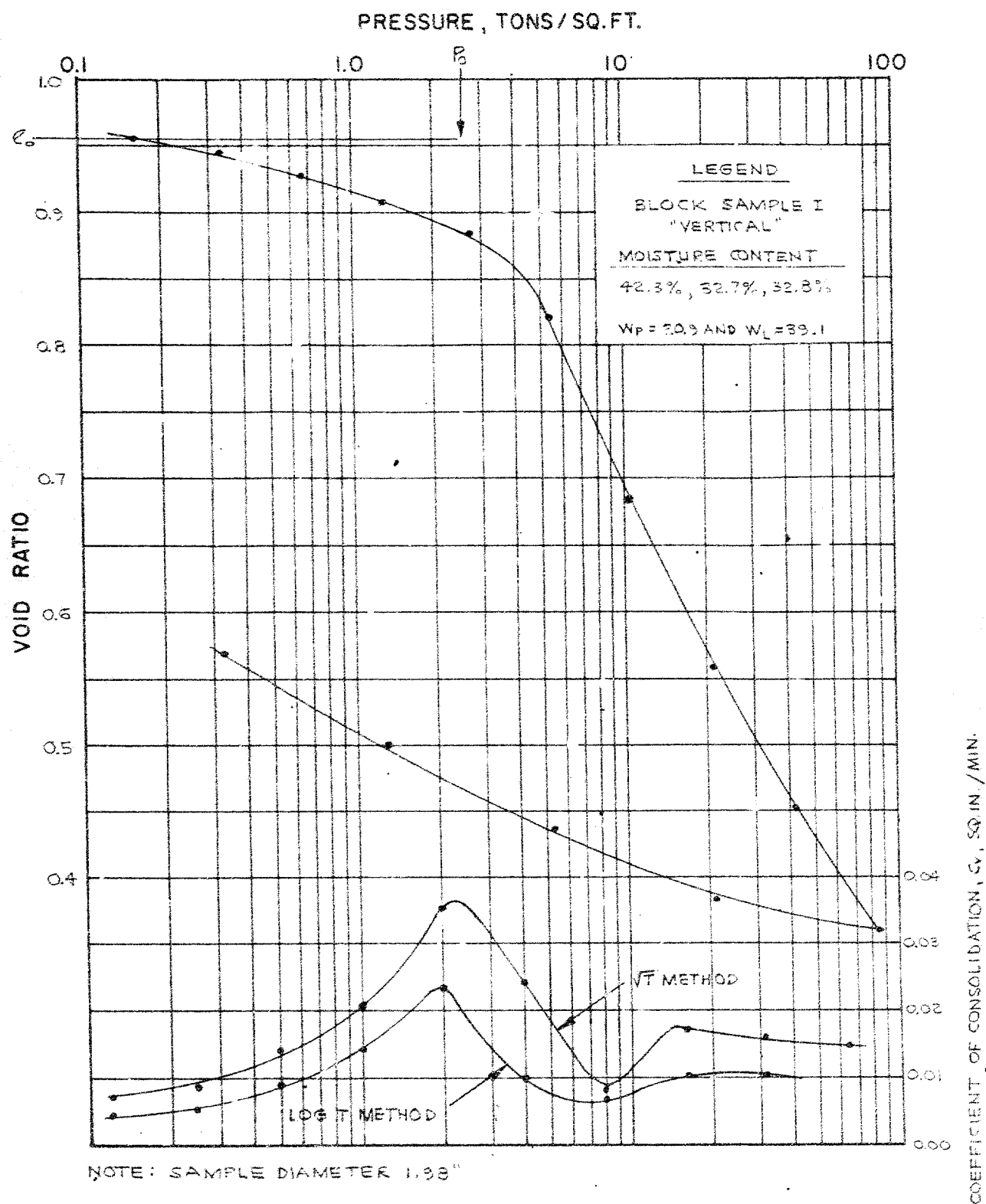
FIGURE 61



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

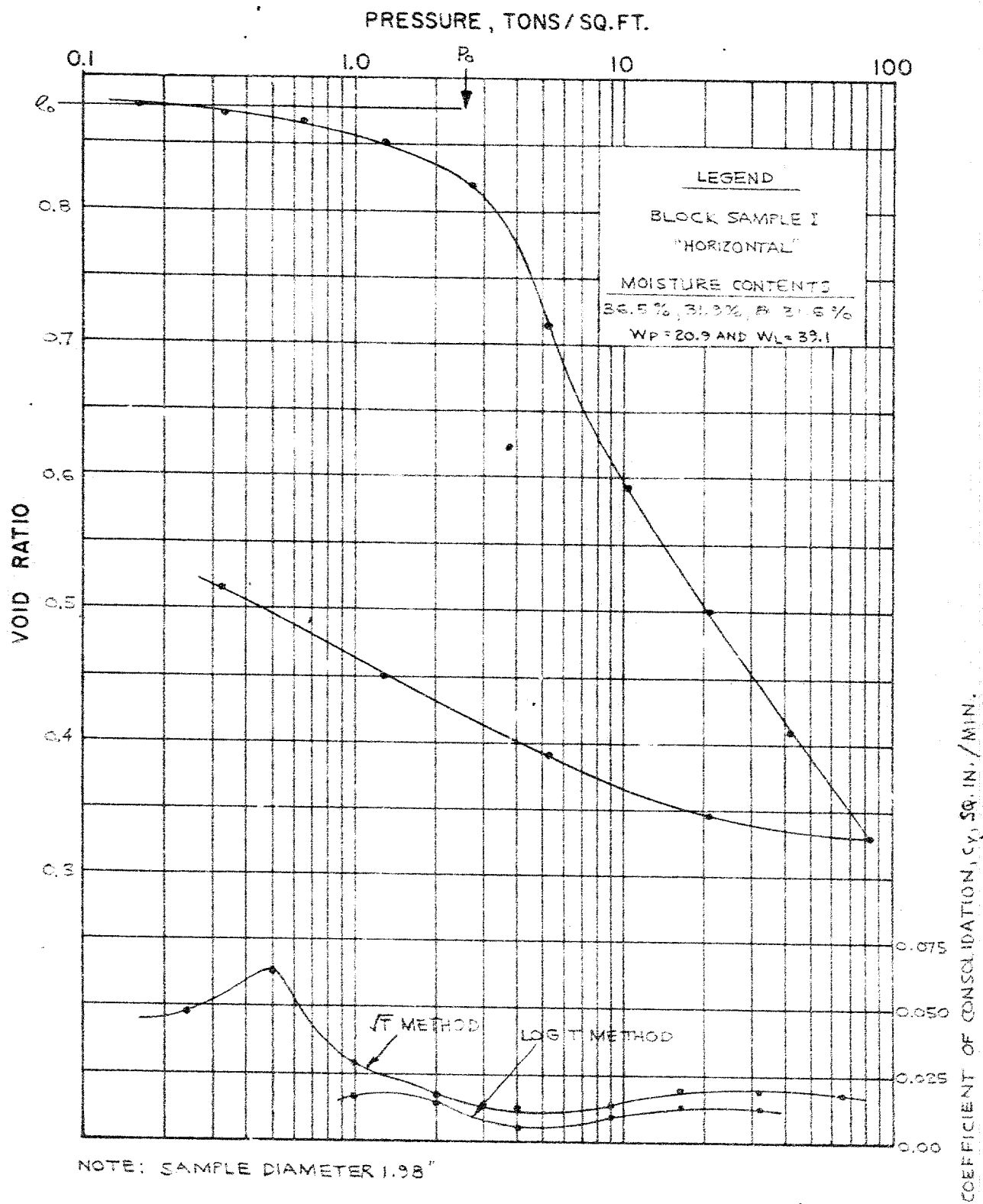
FIGURE 62



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 63

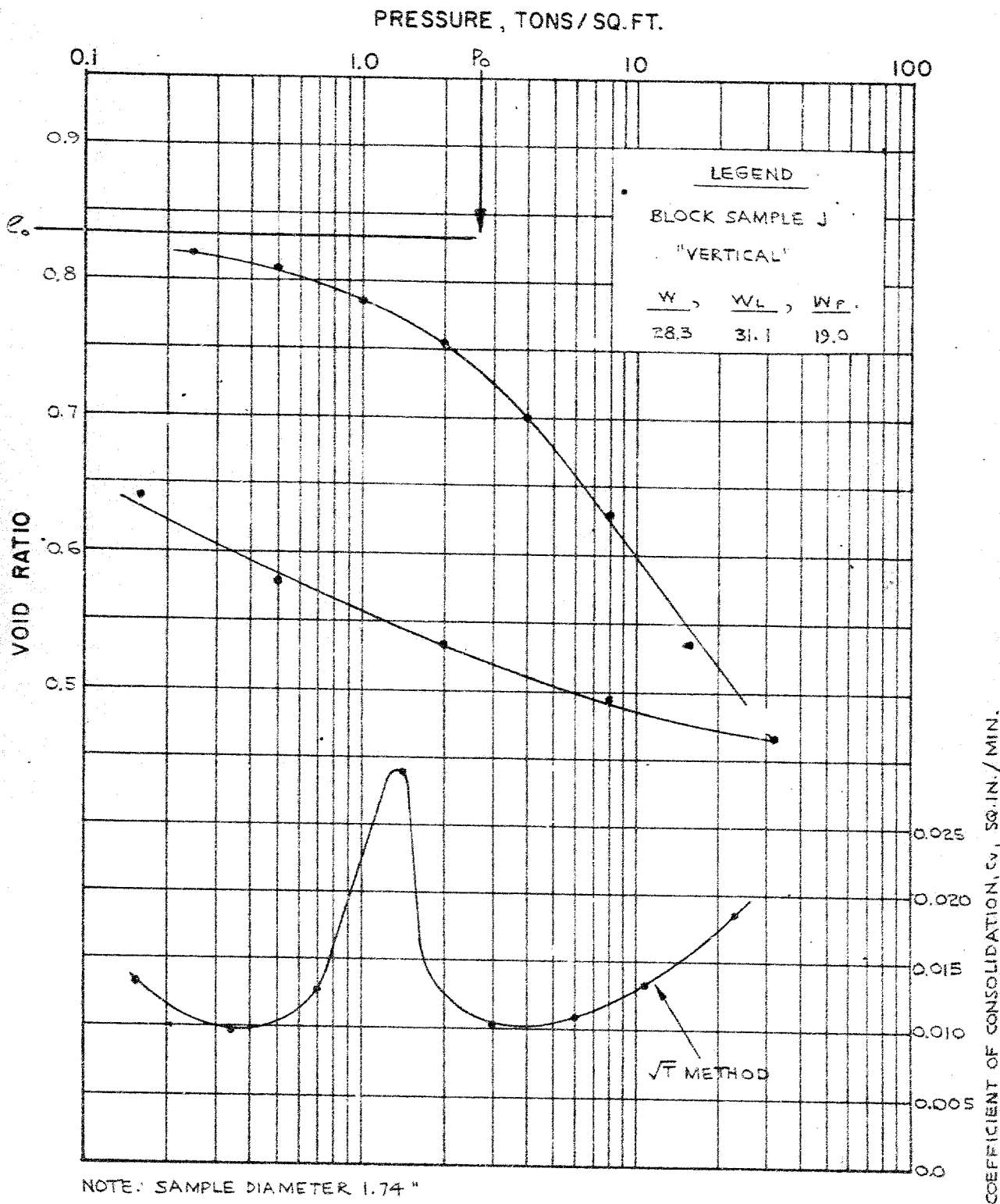


# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 64

124

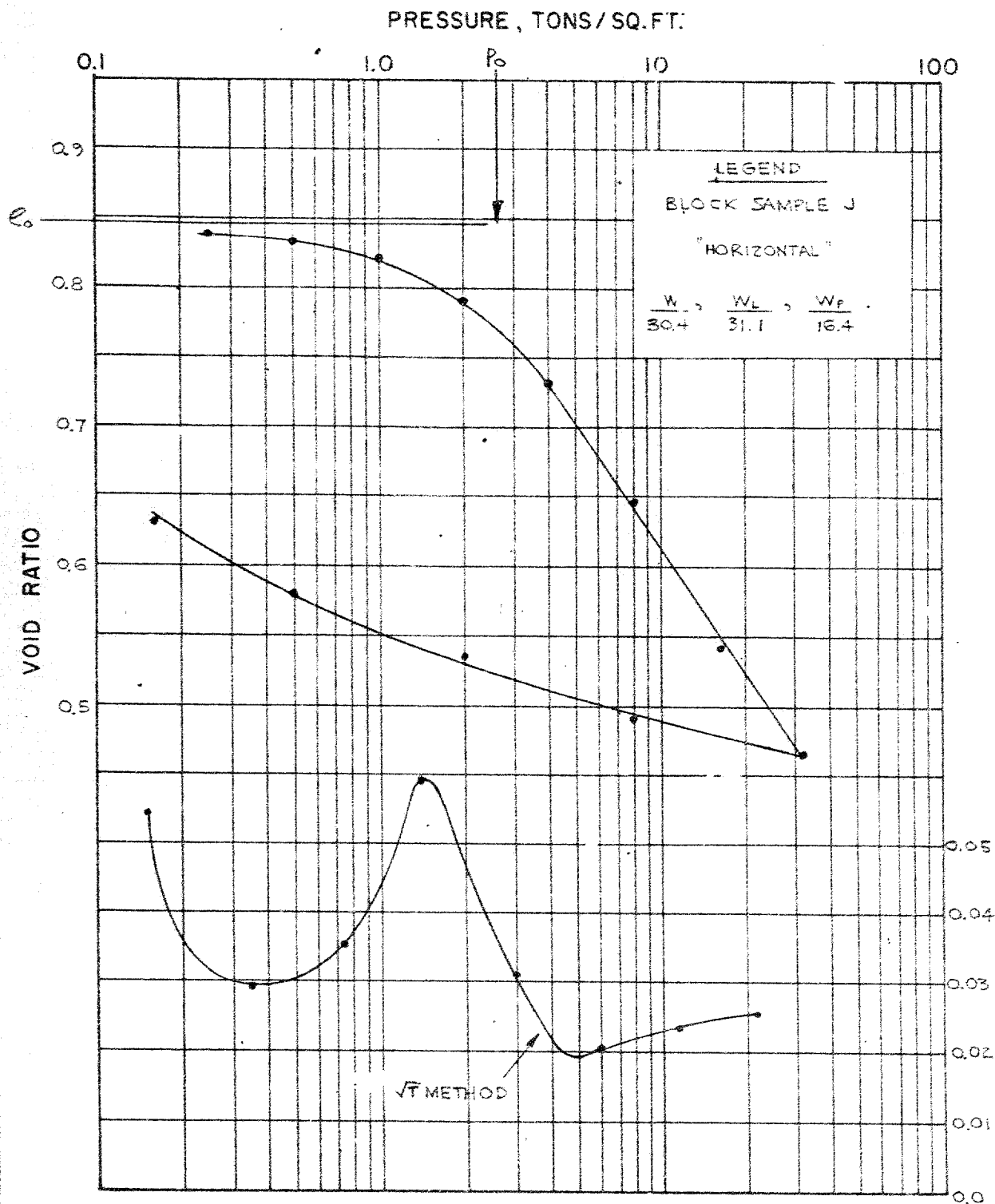
PROJECT No. 6375



GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 65



NOTE: SAMPLE DIAMETER 2.46"

GOLDER & ASSOCIATES



TRUDEL, IND.

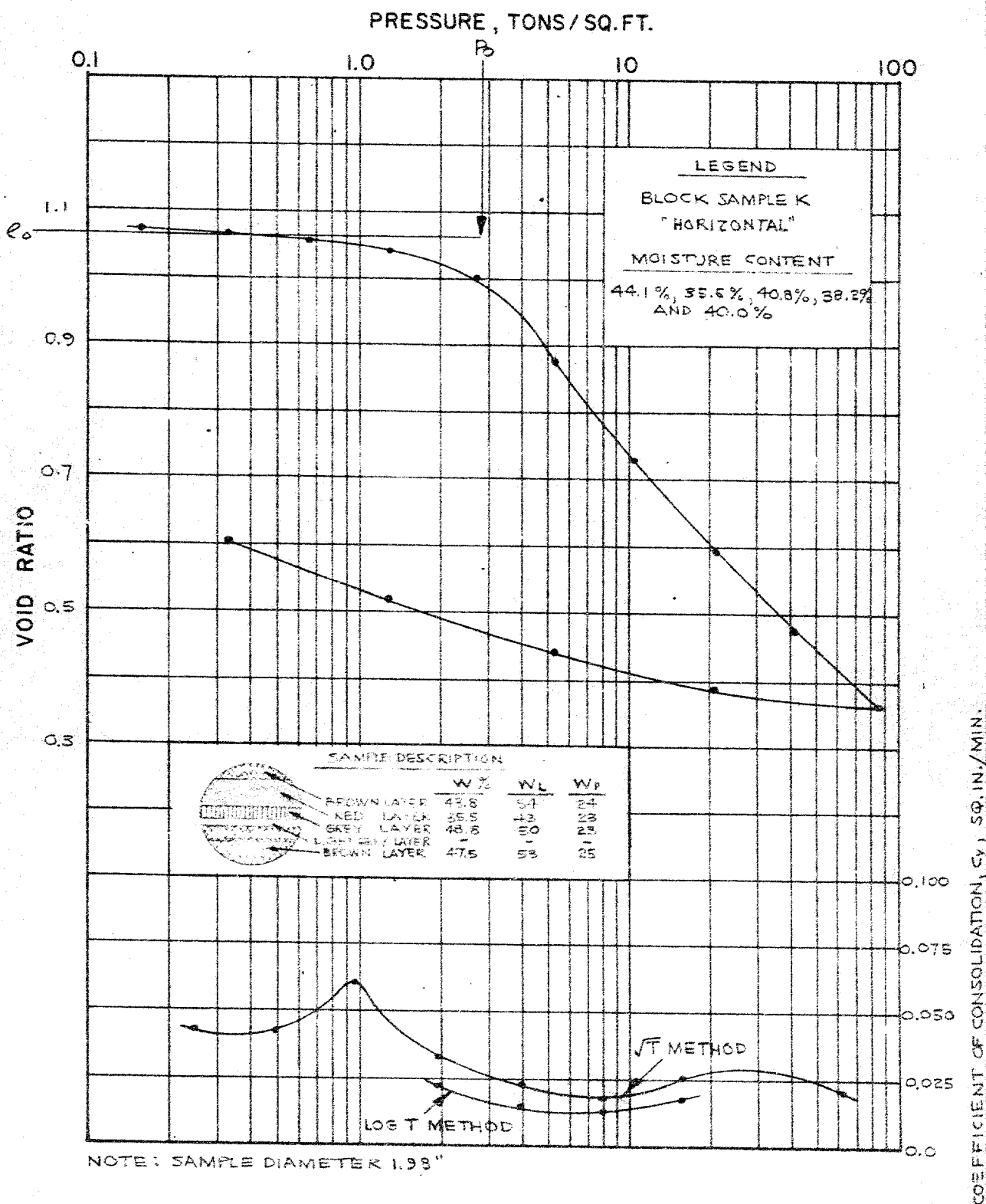
TRUDEL, IND.



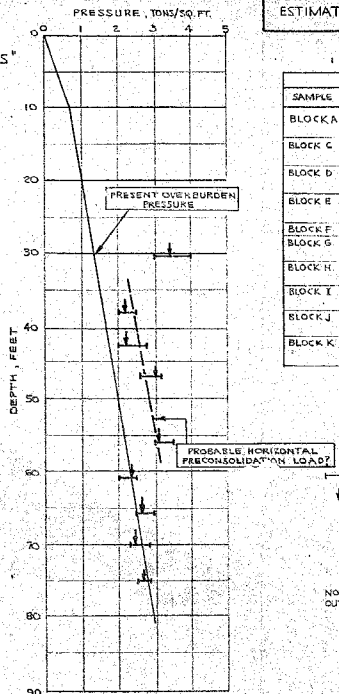
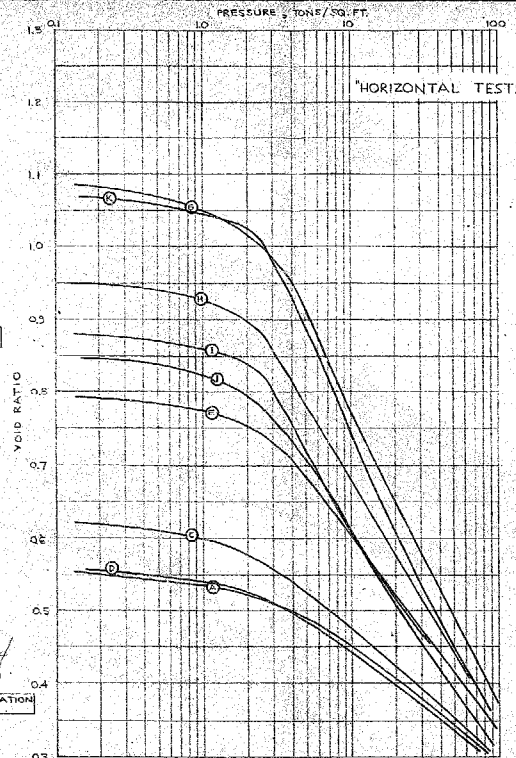
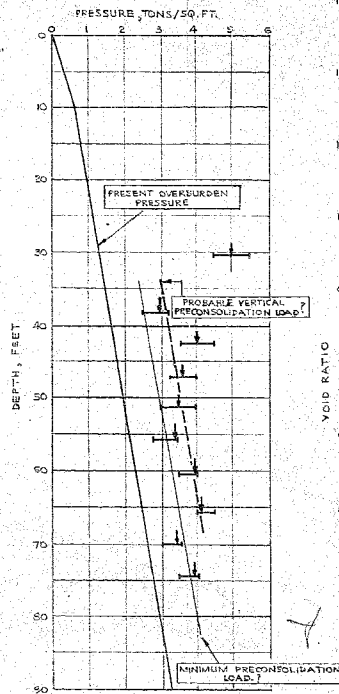
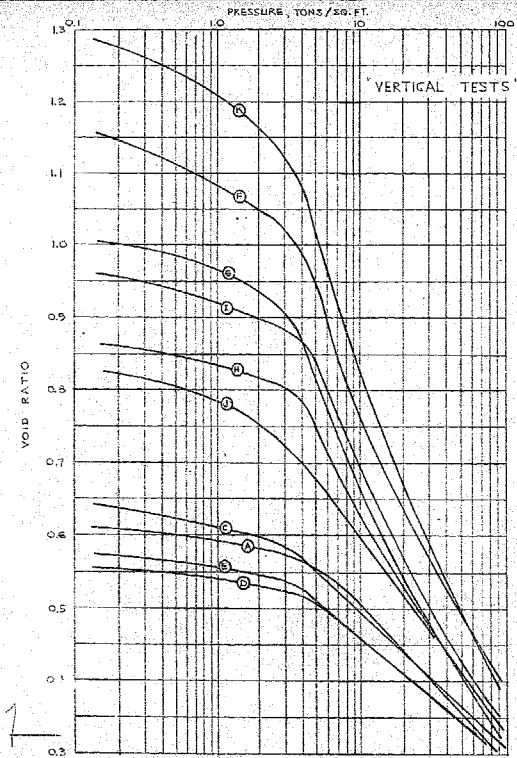
GOLDER & ASSOCIATES

# VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 67



GOLDER & ASSOCIATES



SUMMARY OF CONSOLIDATION TESTS AND ESTIMATED PRECONSOLIDATION LOAD

FIGURE 68

SAMPLE	TYPE	ELEV.	DEPTH	$e_0$	$P_0$	$W_L$	$W_P$	$W$
BLOCK A	VERT.	552.2	30.5	0.610	1.35	27.7	15.8	22.6
BLOCK C	HORIZ.	544.4	38.5	0.553	1.67	27.7	16.1	19.9
BLOCK D	VERT.	539.5	42.8	0.640	1.84	26.9	16.6	22.6
BLOCK E	HORIZ.	535.8	44.9	0.558	1.96	26.7	17.5	21.0
BLOCK F	VERT.	531.2	51.5	0.573	2.11	26.7	17.7	21.8
BLOCK G	HORIZ.	526.7	56.0	0.783	2.41	24.9	19.1	22.9
BLOCK H	VERT.	522.2	60.5	1.124	2.81	22.8	23.8	47.5
BLOCK I	HORIZ.	517.2	65.5	1.004	2.94	21.1	21.1	37.1
BLOCK J	VERT.	512.8	69.9	1.076	3.16	21.5	21.5	40.7
BLOCK K	HORIZ.	508.2	74.5	0.969	3.36	22.0	22.0	36.3
BLOCK L	VERT.	503.2	79.5	0.955	3.53	20.9	20.9	35.2
BLOCK M	HORIZ.	502.8	83.9	0.875	3.67	20.1	20.1	32.7
BLOCK N	VERT.	502.8	83.9	0.832	3.81	16.4	16.4	28.4
BLOCK O	HORIZ.	502.8	83.9	0.847	3.81	15.0	15.0	30.4
BLOCK P	VERT.	502.8	83.9	1.240	2.84	28.3	28.3	49.4
BLOCK Q	HORIZ.	502.8	83.9	1.062	3.00	24.0	24.0	42.0

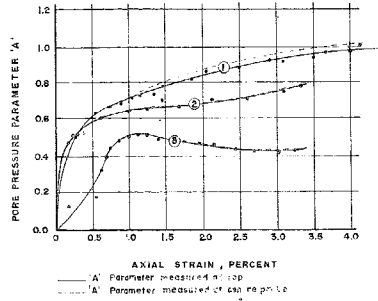
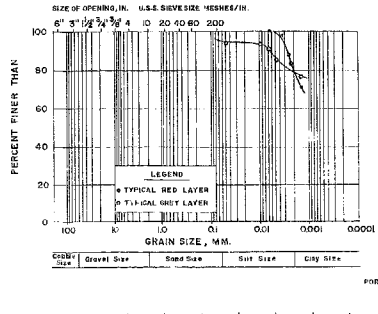
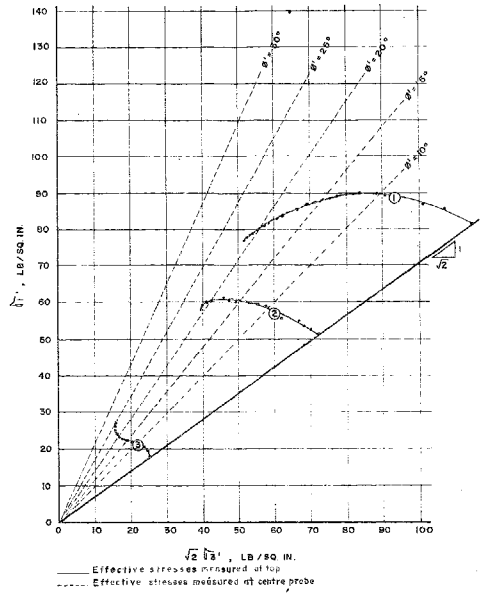
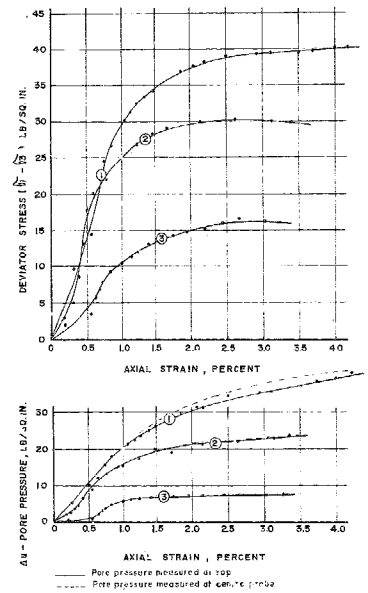
\* AVERAGE OF THE LAYERS DETAILED ON FIGURE 48.

LEGEND

ESTIMATED RANGE OF PRECONSOLIDATION LOAD

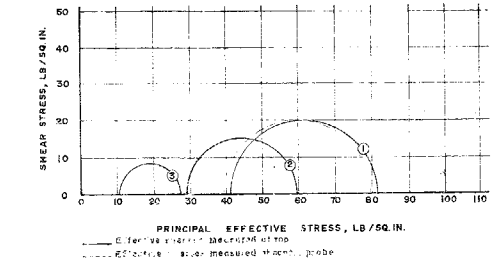
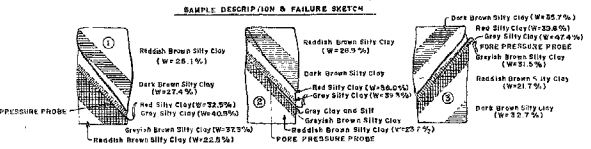
MOST PROBABLE PRECONSOLIDATION LOAD

NOTE: ALL CONSOLIDATION TESTS, EXCEPT J, WERE CARRIED OUT ON SAMPLES 2.0" IN DIAMETER AND 0.5" HIGH.  
BLOCK J "VERTICAL" 1.75" DIA. 0.5" HIGH.  
BLOCK J "HORIZONTAL" 2.5" DIA. 0.75" HIGH

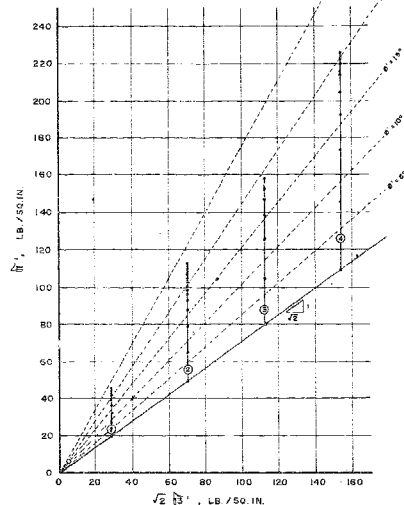
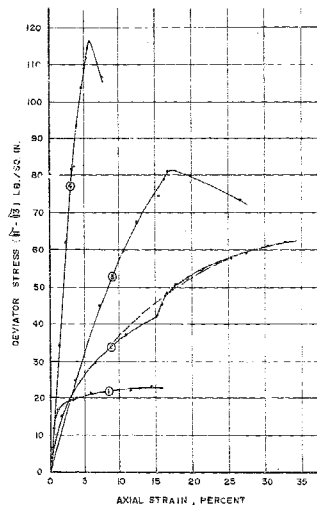
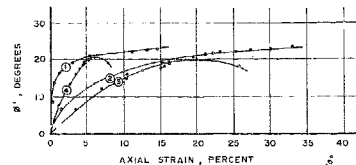
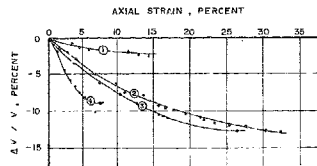


TEST No.	TUBE TYPE	CELL PRESSURE	BACK PRESSURE	NET CELL PRESSURE	RATE OF STRAIN	TIME TO FAILURE	UNIT WEIGHT	FLUSHING TIME	1st. CONSOLIDATION TIME	RECONSOLIDATION TIME
1	2" SHELBY	100 p.s.i.	20 p.s.i.	80 p.s.i.	1.2 %/hr.	4.7 hr.	122 p.c.f.	14 days	12 days	1 day *
2	2" SHELBY	100 p.s.i.	20 p.s.i.	80 p.s.i.	1.2 %/hr.	3.0 hr.	121 p.c.f.	10 days	12 days	12 hr. *
3	2" SHELBY	100 p.s.i.	20 p.s.i.	80 p.s.i.	1.2 %/hr.	2.5 hr.	125 p.c.f.	—	1 day	—

NOTE  
 SAMPLES FROM WELAND TEST SHAFT, ZONE F, ELEVATION 551, SAMPLED AT 42" TO HORIZONTAL LAYERING.  
 \* AFTER FIRST CONSOLIDATION,  $\sigma_3$  PORE PRESSURE COEFFICIENT IN SAMPLES 1 & 2 WAS LESS THAN 1. APPROXIMATELY 1 VOID VOLUME OF WATER WAS FLUSHED THROUGH SAMPLES. AFTER RECONSOLIDATION,  $\sigma_3$  PORE PRESSURE COEFFICIENT WAS APPROX. 1.



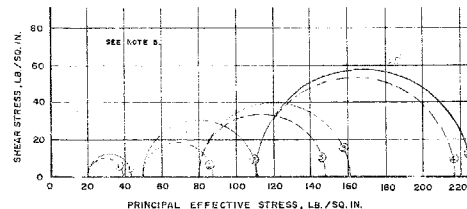
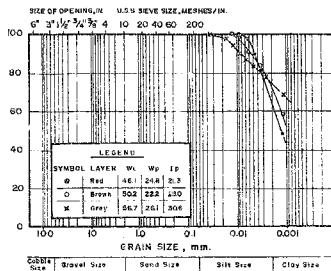
where 'A' =  $\frac{\Delta u}{\Delta \sigma_1 - \Delta \sigma_3}$  for saturated soil



TEST No.	BLOCK SAMPLE	SAMPLE No.	CELL PRESSURE	TIME TO FAILURE	UNIT WEIGHT
①	Zone K	23	20 p.s.i.	8 days	113 p.c.f.
②	"	21	50 p.s.i.	21 days	115 p.c.f.
③	"	20	80 p.s.i.	12 days	117 p.c.f.
④	"	22	110 p.s.i.	10 days	116 p.c.f.

1. THE SAMPLES ARE FROM WELLAND TEST SHAFT. BLOCK SAMPLE FROM ZONE K, ELEVATION 507.7 TO 508.7; INDIVIDUAL SAMPLES WERE CUT AT 45° TO THE HORIZONTAL PLANE OF THE BLOCK SAMPLE, FOR POSITIONS OF SAMPLES CUT FROM BLOCK SEE FIGURE 56.

2. THE SAMPLES HAVE GENERALLY BEEN LOADED AT 24 HOUR INTERVALS.



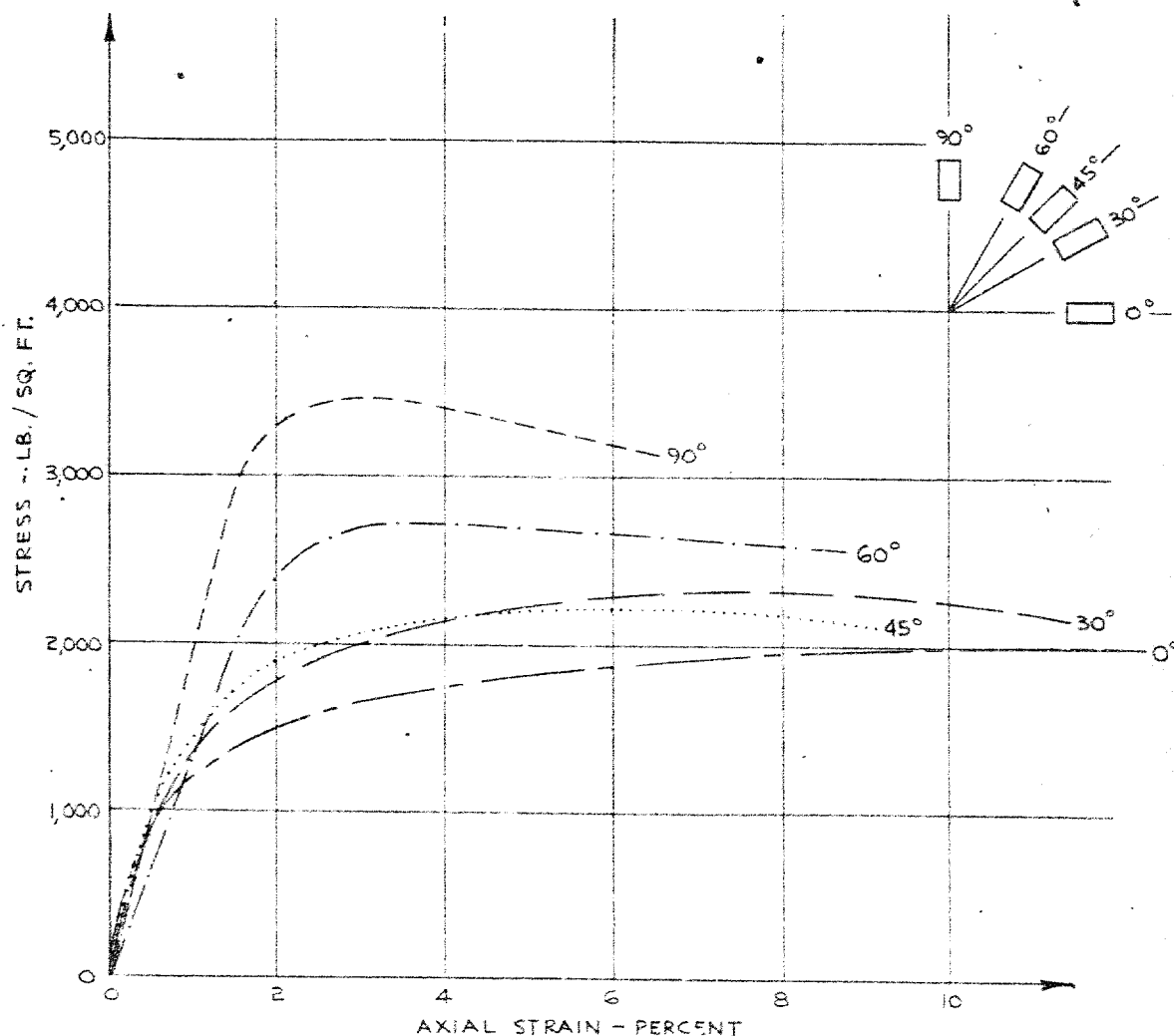
5. FILTER STRIPS 1/4" WIDE WERE PLACED 1/4" APART VERTICALLY AROUND SAMPLES TO AID DRAINAGE.
4. GENERALLY THE SAMPLES STRAINED IN FOUR STAGES:
  - i) NORMAL STRAIN PATTERN WITH INCREASING STRENGTH.
  - ii) FAILURE PLANE WOULD DEVELOPE AND SOME SHEAR MOVEMENT WOULD TAKE PLACE ALONG PLANE.
  - iii) NORMAL STRAIN PATTERN WITH INCREASING STRENGTH.
  - iv) ABRUPT FAILURE ALONG SHEAR PLANE.

5. MOHR'S CIRCLE DIAGRAM: THE DASHED LINE REPRESENTS STRESS CONDITIONS WHEN FIRST NOTICEABLE SHEAR PLANE DEVELOPED, SOLID LINE REPRESENTS STRESS CONDITIONS AT COMPLETE FAILURE.

SAMPLE DESCRIPTION AND FAILURE SKETCH	MOISTURE CONTENTS	
	BEFORE TESTING	AFTER TESTING
Gray Silty Clay	42.3	46.0
Brown Silty Clay	37.5	37.3
Red Silty Clay	41.7	40.5
Gray Silty Clay	42.3	46.0
Brown Silty Clay	37.5	37.3
Red Silty Clay	41.7	40.5
Gray Silty Clay	42.3	46.0
Brown Silty Clay	37.5	37.3
Red Silty Clay	41.7	40.5
Gray Silty Clay	42.3	46.0
Brown Silty Clay	37.5	37.3
Red Silty Clay	41.7	40.5

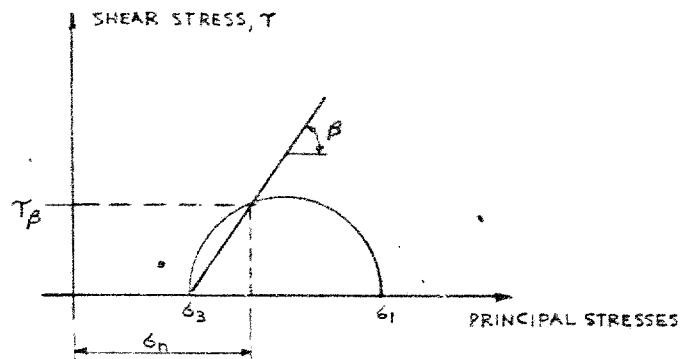
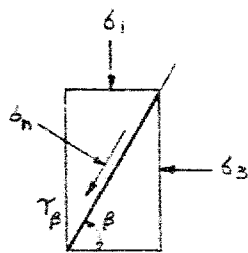
# TYPICAL STRESS-STRAIN CURVES, BLOCK C NON - STRATIFIED SOIL

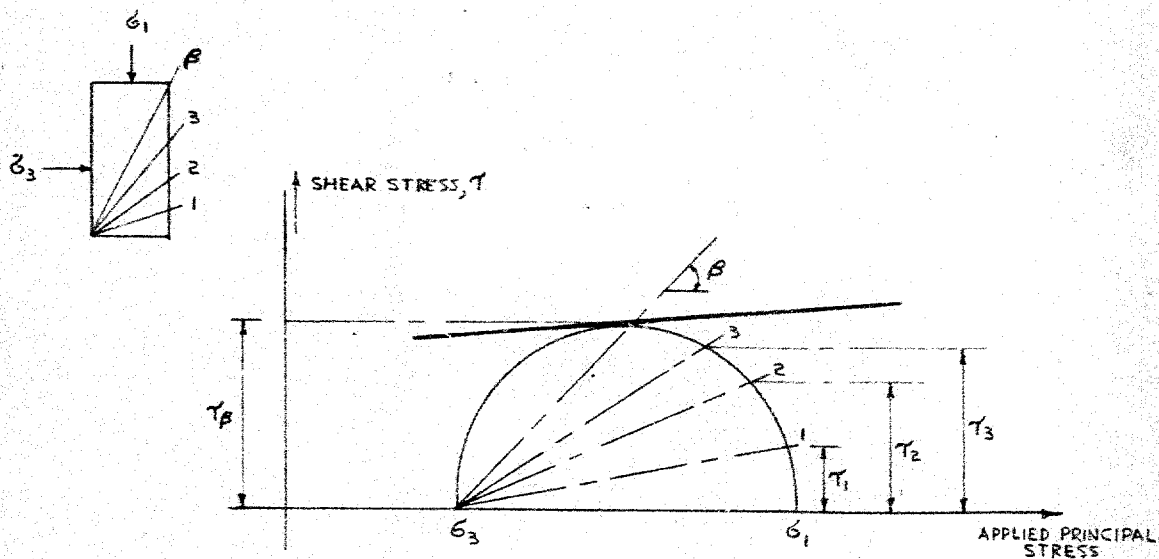
FIGURE 71



GOLDER & ASSOCIATES

Made J.A.  
Chko.  
Appd. 7

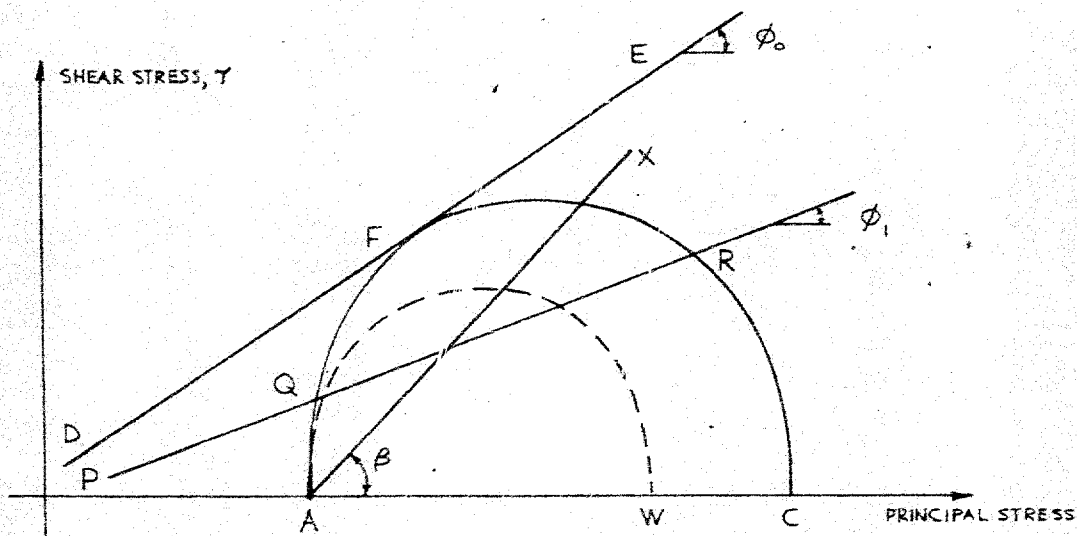






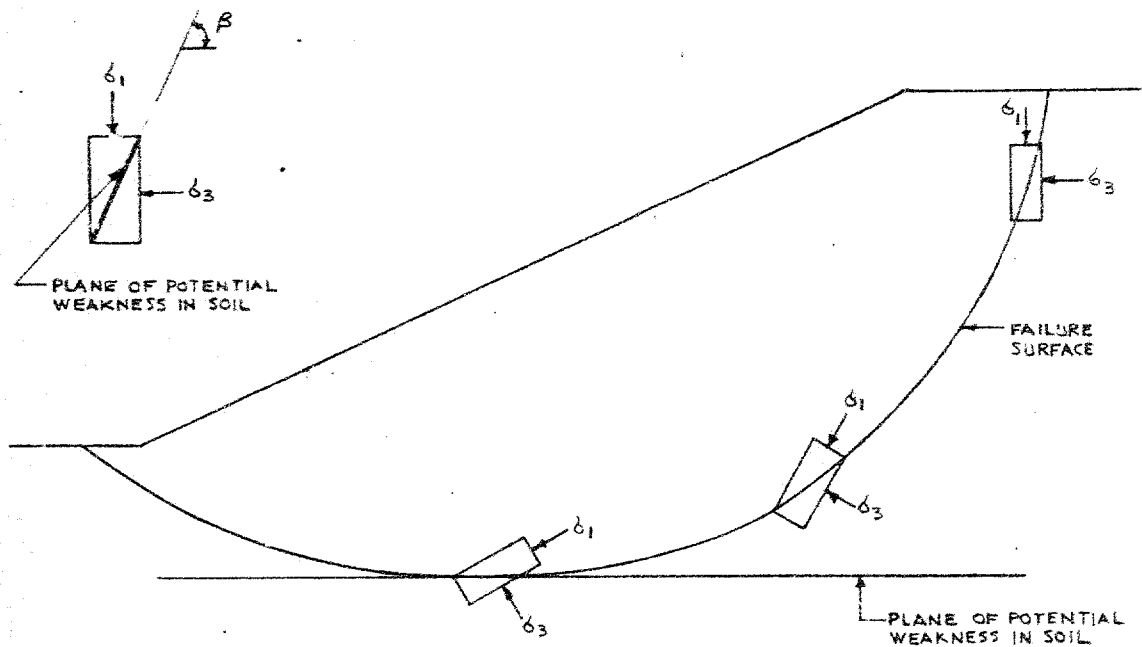
# FAILURE THROUGH SOIL MASS ACROSS A PLANE OF WEAKNESS (AFTER J.C. JAEGER, 1956)

FIGURE 74



# PRINCIPAL STRESSES AROUND FAILURE SURFACE IN SOIL MASS

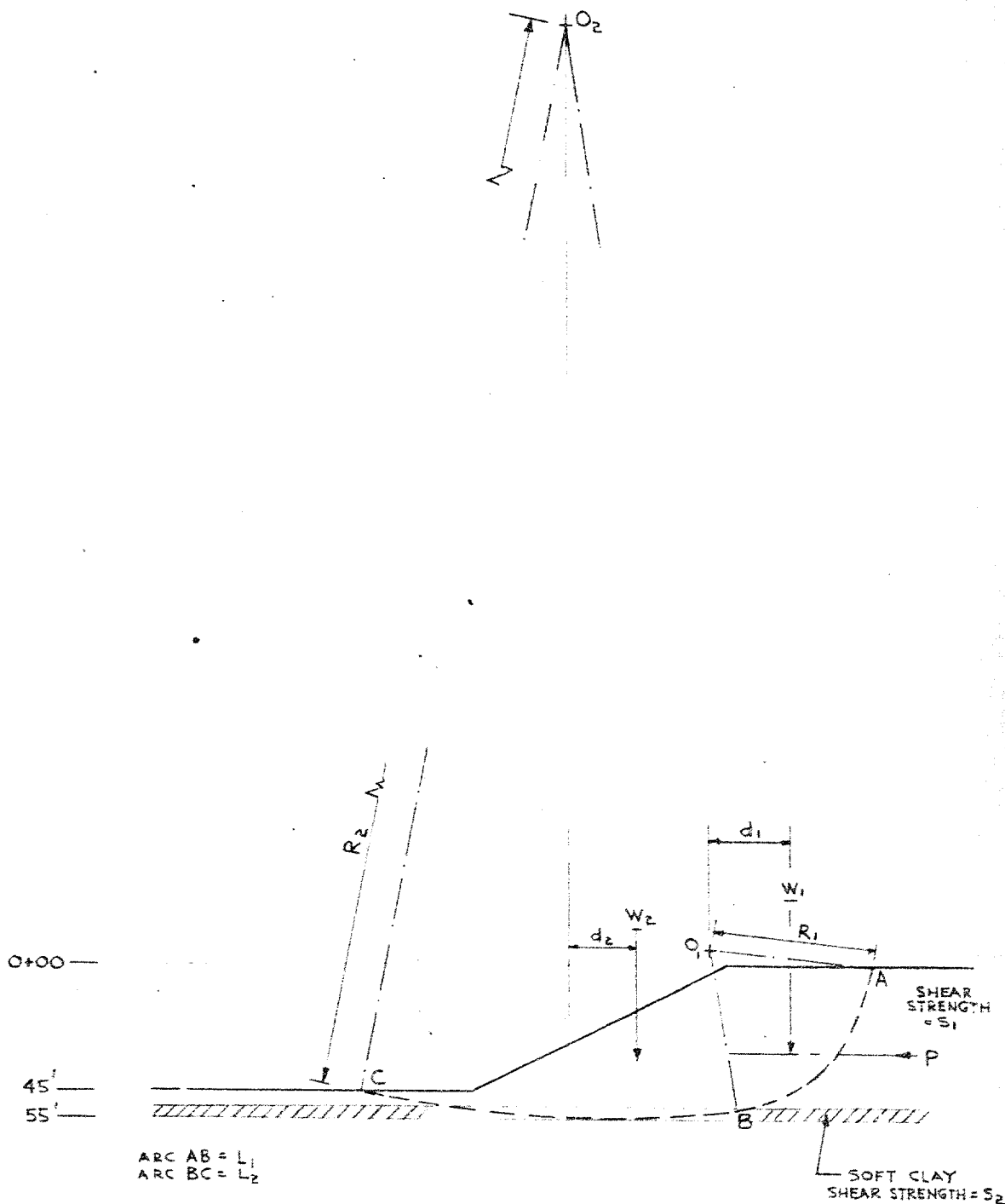
FIGURE 75



GOLDER & ASSOCIATES

Made J.A.  
Chkd. 47  
Appd. 47

PROJECT NO. 9311



GENERAL FORM OF EQUATION FOR FACTOR OF SAFETY,  $F$

$$L_2 \cdot R_2 \frac{S_2}{F} = W_2 \cdot d_2 + \frac{d_2}{d_1} \left( W_1 d_1 - \frac{S_1}{F} \cdot L_1 \cdot R_1 \right)$$

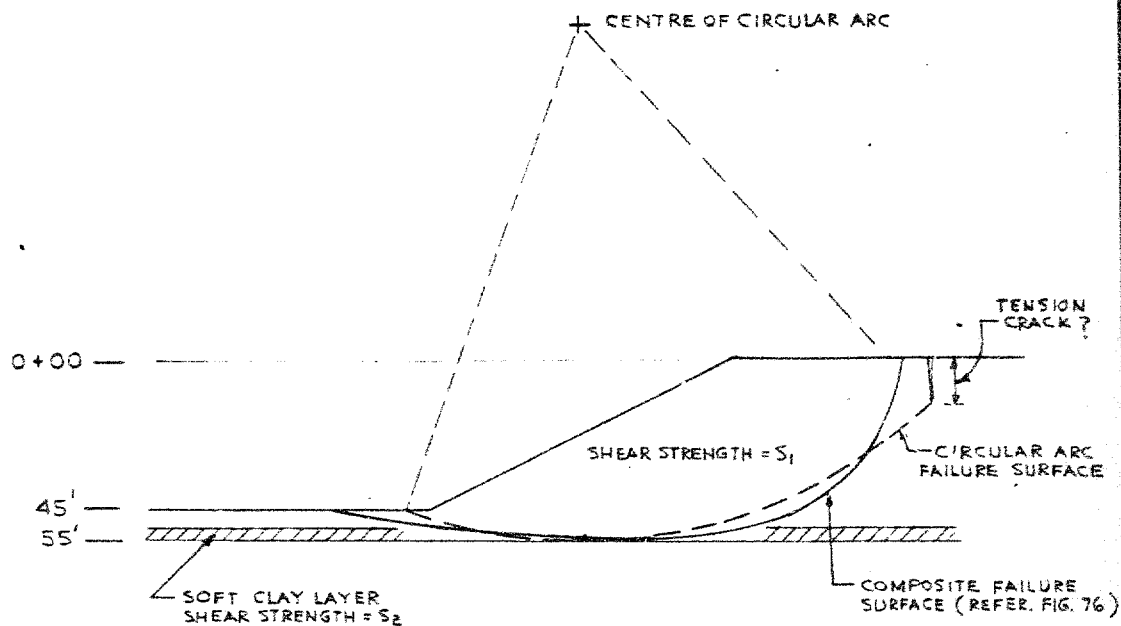
GOLDER & ASSOCIATES

Made J.A.  
Chkd. [Signature]  
Appd. [Signature]

PROJECT NO. 11111

# COMPARISON OF CIRCULAR ARC AND COMPOSITE FAILURE SURFACES

FIGURE 77



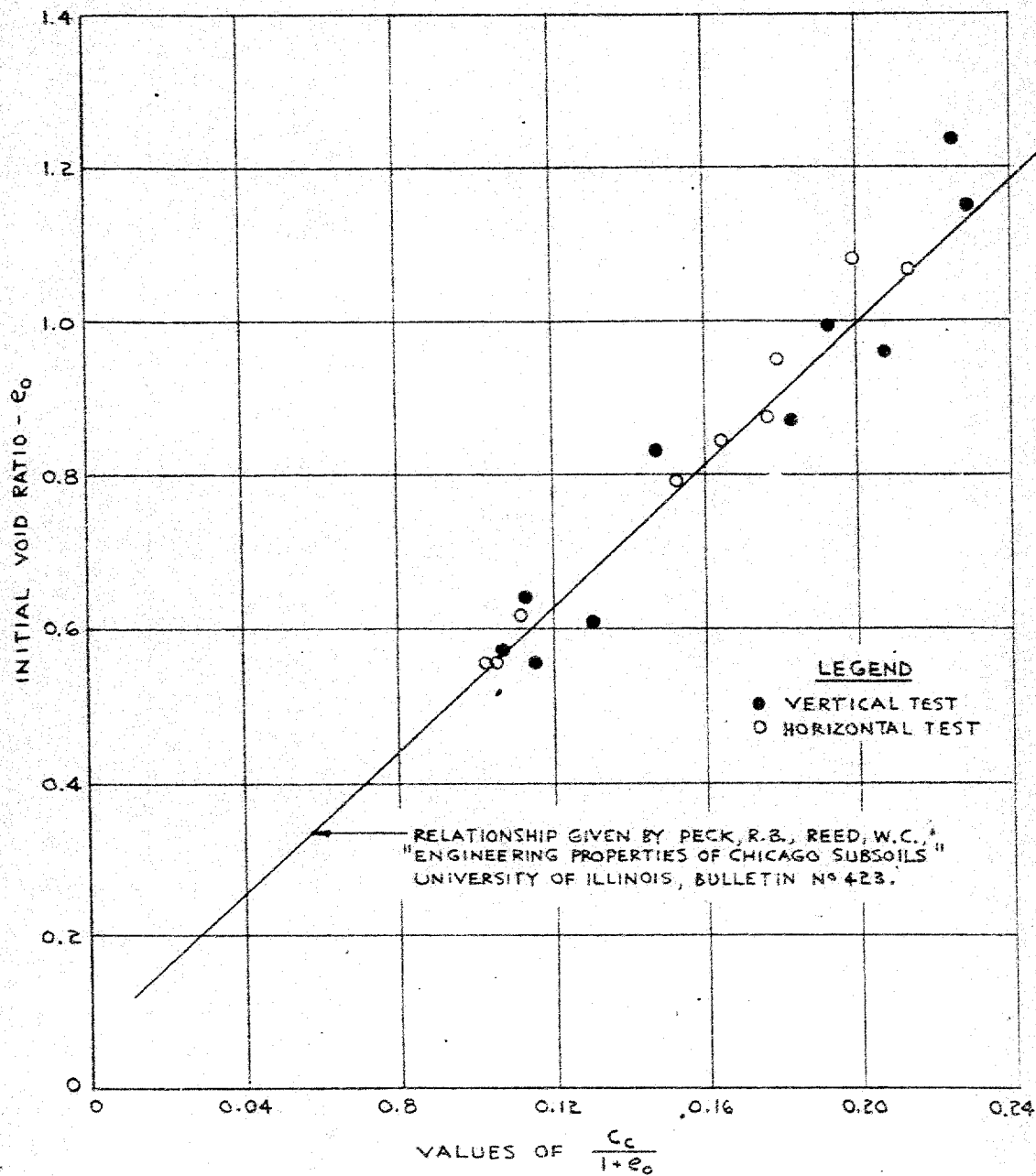
COLDER & ASSOCIATES

Made 1.5  
Chkd. 1.7  
Appd. 1.7

PROJECT No. 6325

# RELATIONSHIP OF INITIAL VOID RATIO, $e_0$ TO COMPRESSION RATIO $\frac{C_c}{1+e_0}$

FIGURE 78

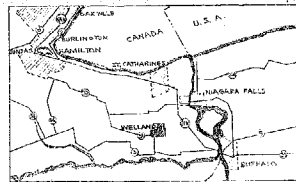


GOLDER & ASSOCIATES

Made J.A.  
Chkd. 4-7  
Appd. 4-7

DEPARTMENT OF HIGHWAYS, ONTARIO LABORATORY TEST RESULTS

**GOLDER & ASSOCIATES**

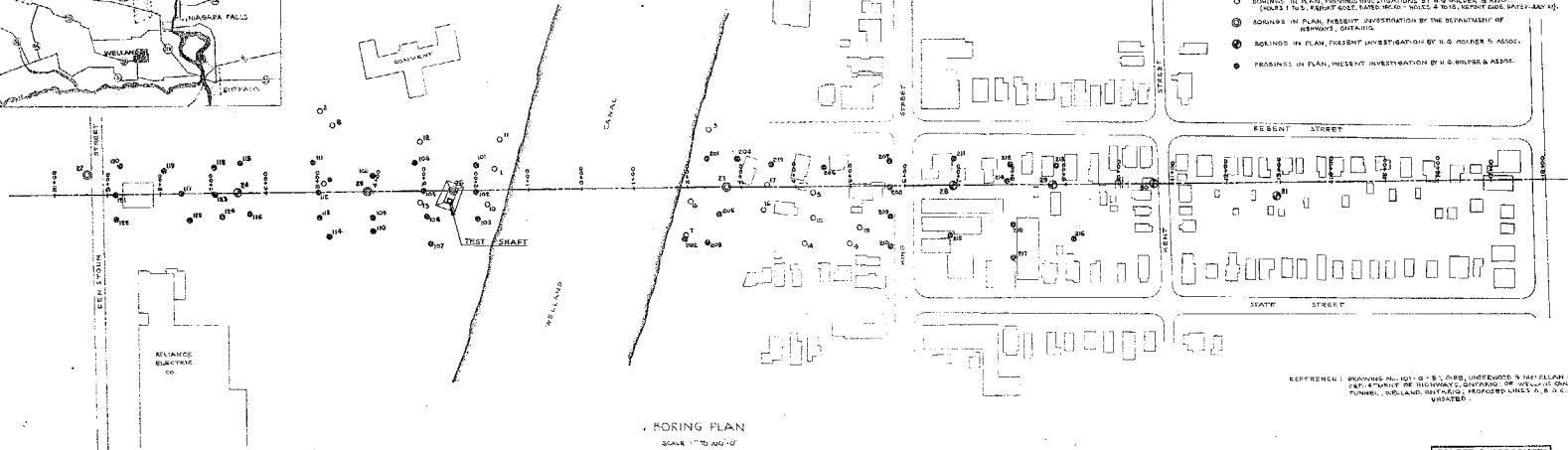


KEY PLAN  
SCALE 1" TO 14 MILES

## LEGEND

- BORINGS IN PLAN, PREVIOUS INVESTIGATIONS BY H.G. GOLDER & ASSOC.  
(HOLES 1 TO 5, REPORT G.O.L.D. DATED 1960; HOLES 6 TO 10, REPORT G.O.L.D. DATED 1961)
- ① BORINGS IN PLAN, PRESENT INVESTIGATION BY THE DEPARTMENT OF HIGHWAYS, ONTARIO
- ② BORINGS IN PLAN, PRESENT INVESTIGATION BY H.G. GOLDER & ASSOC.
- PROBINGS IN PLAN, PRESENT INVESTIGATION BY H.G. GOLDER & ASSOC.

N



REFERENCE: DRAWING NO. 101-G-B-1, G.B.D. UNDERWOOD & McLELLAN FOR  
DEPARTMENT OF HIGHWAYS, ONTARIO: OF WELLAND CANAL  
TUNNEL, WELAND, ONTARIO, PROPOSED LINES A, B & C.  
DATED

GOLDER &amp; ASSOCIATES

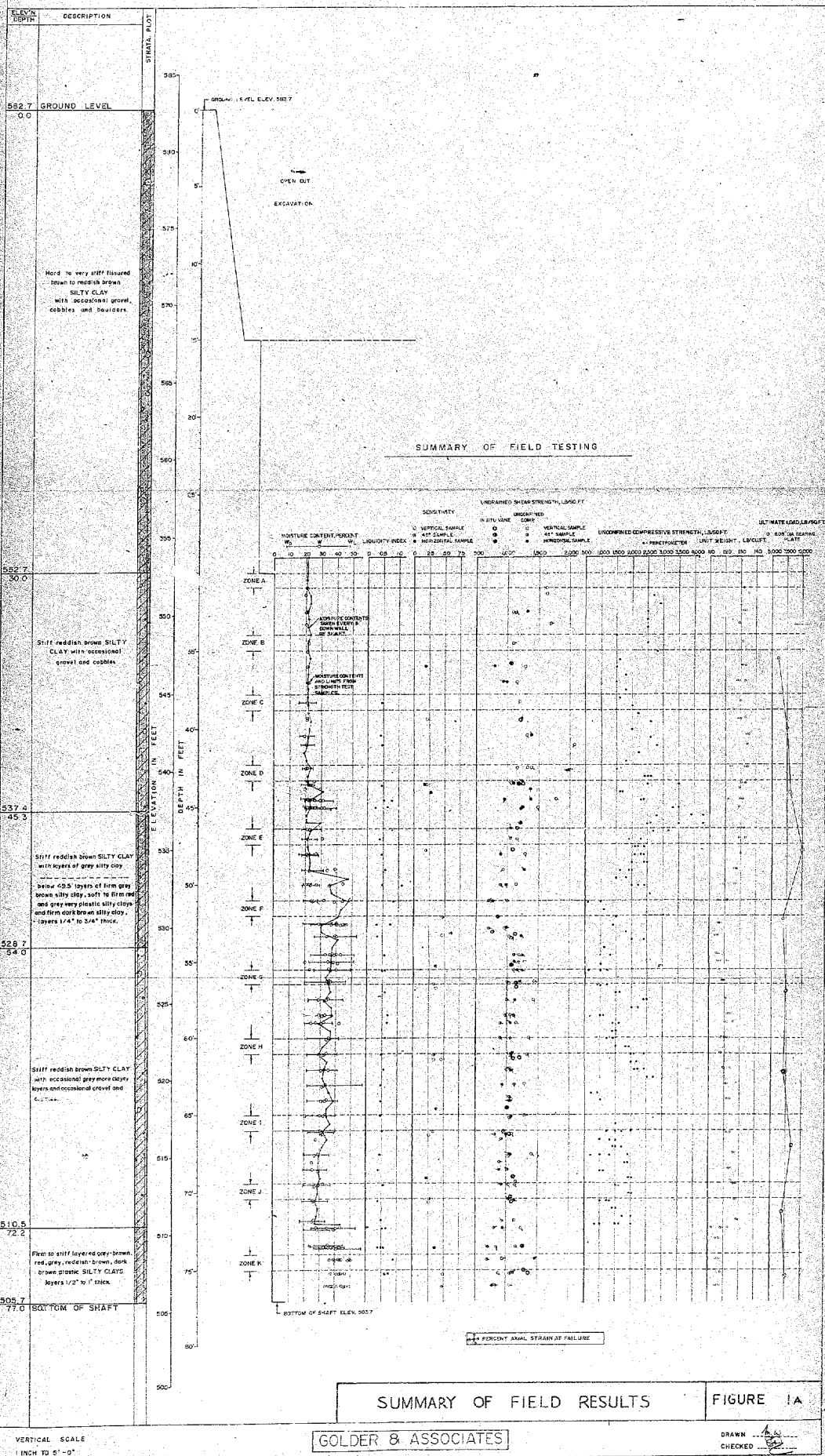
Mod. 05/60  
Chk. 05/60  
Appd. 05/60

## WELLAND TEST SHAFT

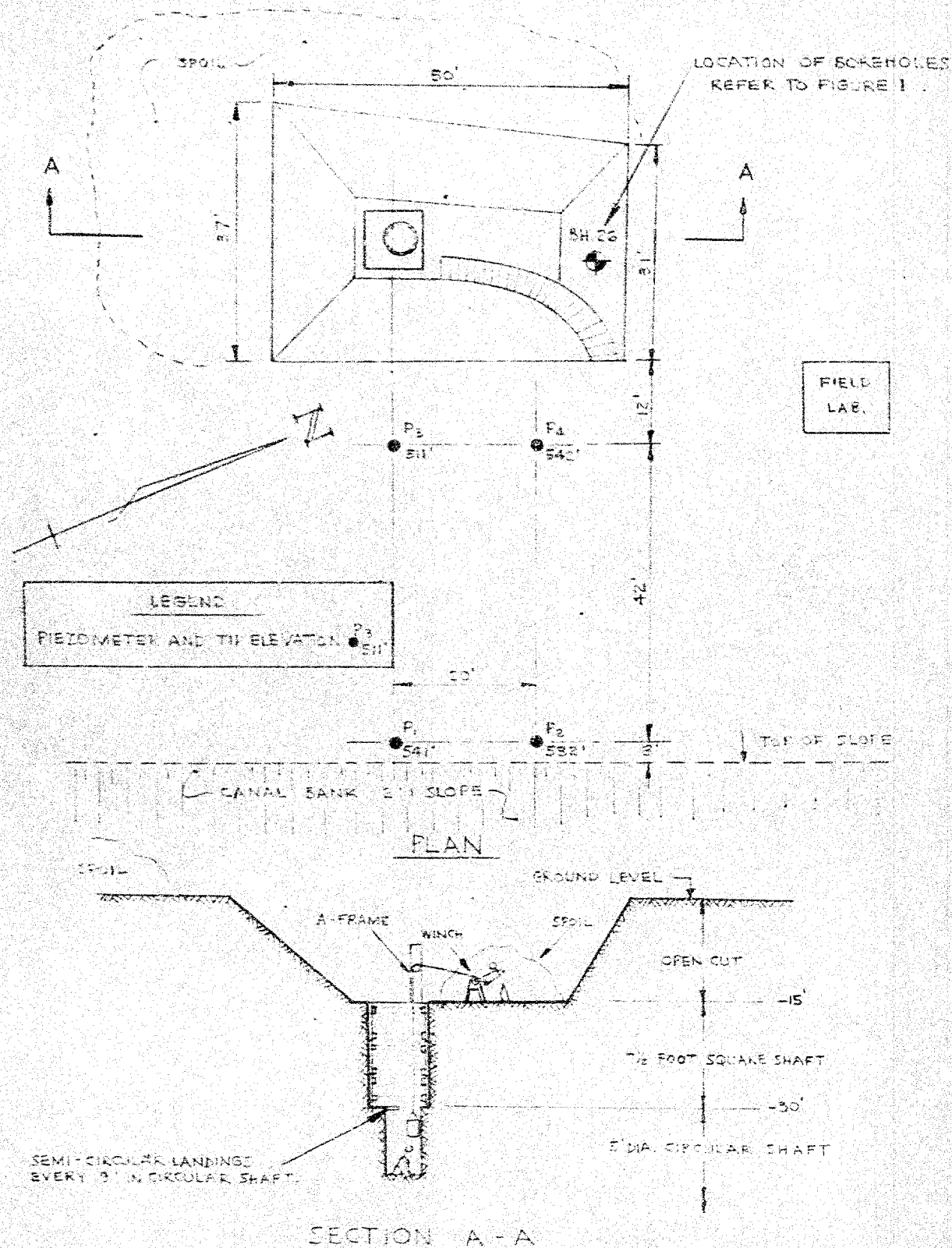
LOCATION . SEE FIGURE 1

DATE , OCTOBER 25 - DECEMBER 11, 1963

DATUM, GEODETIC







NOT TO SCALE

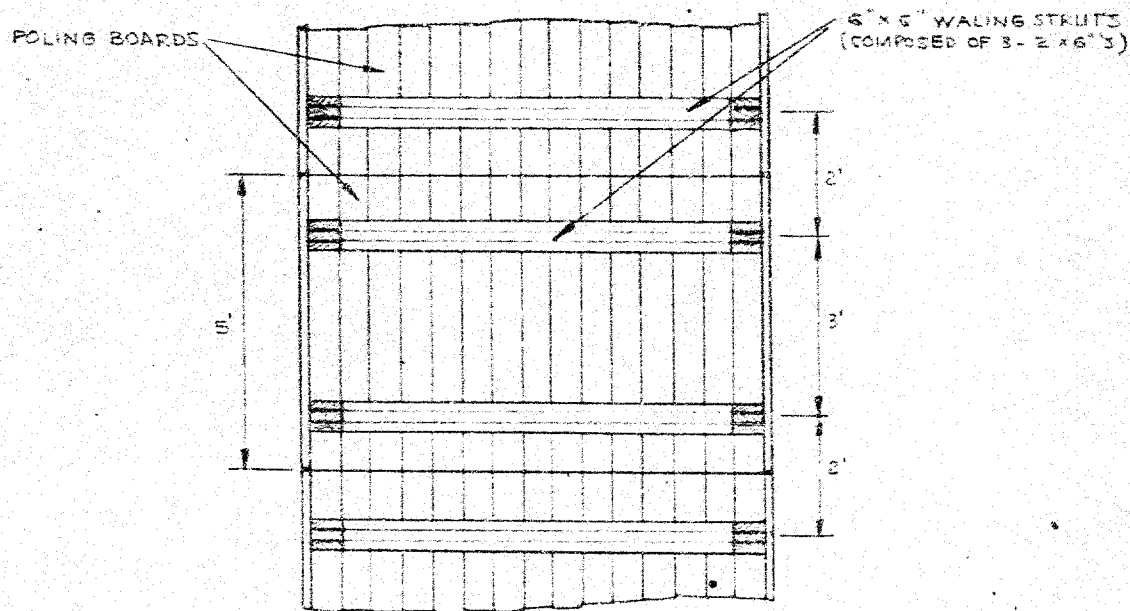
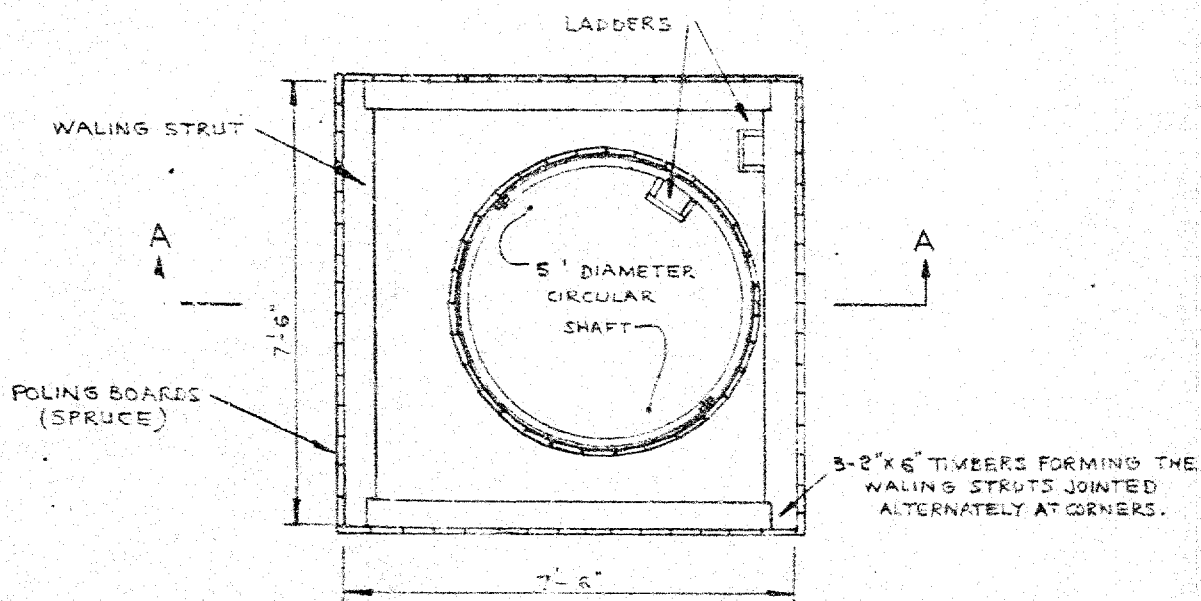
COLDER & ASSOCIATES

Made *M.D.*  
Chkd *+*  
Appd *+*

PROJECT NO. 55375

# TIMBERING OF SQUARE SHAFT

FIGURE 3



## SECTION A-A

NOT TO SCALE

NOTE: SQUARE SHAFT EXTENDED FROM ELEV. 567.2 TO ELEV. 553.2.

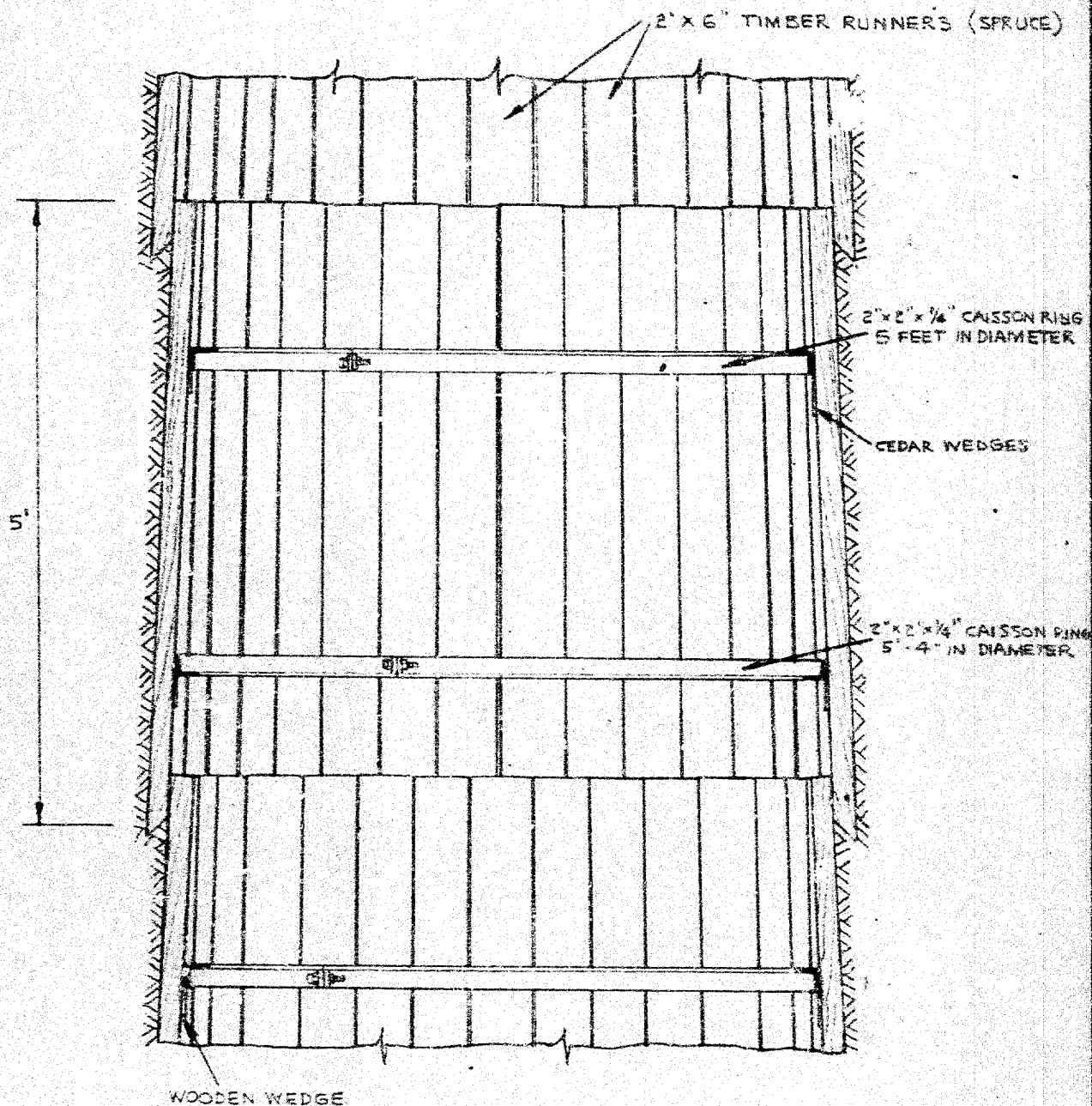
GOLDER & ASSOCIATES

Made *M.W.*  
Chkd *T-7*  
Appd *T-7*

PROJECT No. 5575

# TIMBERING OF CIRCULAR SHAFT

FIGURE 4



SECTION THROUGH CIRCULAR SHAFT

NOT TO SCALE

NOTE: CIRCULAR SHAFT EXTENDED FROM ELEV. 553.2 TO ELEV. 505.7

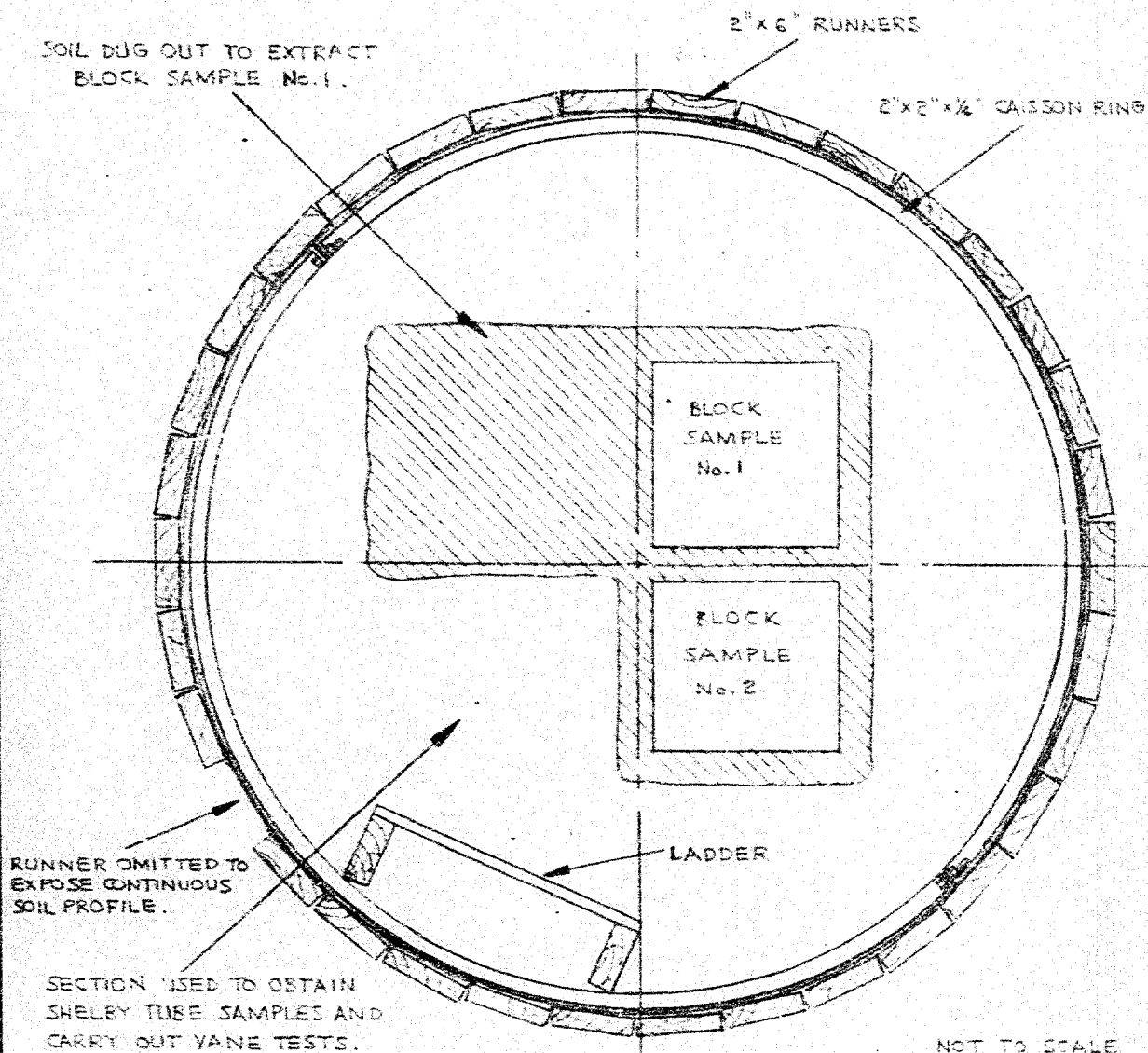
GOLDER & ASSOCIATES

Made *M.N.*  
Chkd *[initials]*  
Appd *[initials]*

PROJECT No. -- 5325 --

# DETAILS OF BLOCK SAMPLING

FIGURE 5



PLAN

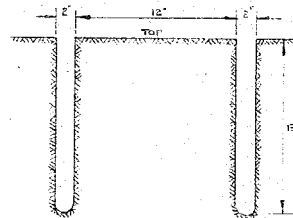
## ORDER OF SAMPLING

1. UPPER 6 INCHES OF SOIL REMOVED CAREFULLY.
2. BLOCK SAMPLE No. 1 CUT OUT.
3. BLOCK SAMPLE No. 2 CUT OUT.
4. SHELBY TUBES PUSHED IN AND CUT OUT.
5. VANE TESTS.

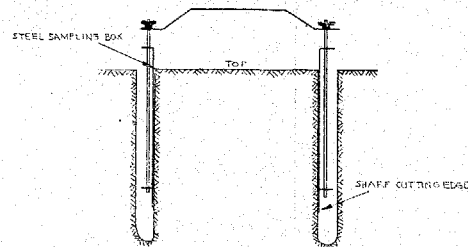
GOLDER & ASSOCIATES

Made 11/62  
Chkd.           
Appd.

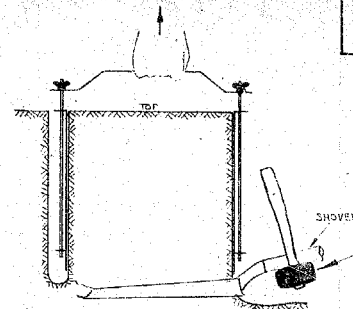
# SAMPLE REMOVAL FROM GROUND



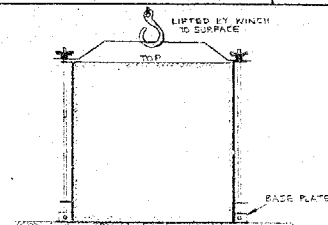
1. TRENCHES CUT ON FOUR SIDES OF SAMPLE USING CLAY KNIVES.



2. STEEL SAMPLING BOX, PUSHED DOWN OVER SAMPLE.

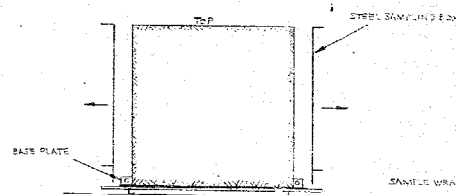


3. ONE SIDE OF BLOCK DUG AWAY AND 12" BLADE SHOVEL TAMPED UNDERNEATH. BLOCK THEN LIFTED OUT OF HOLE.

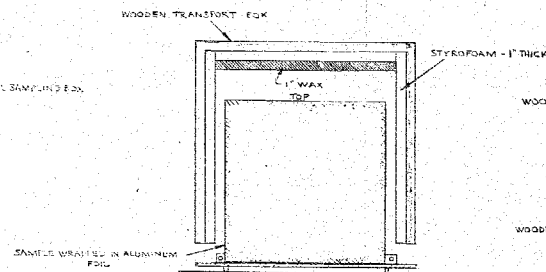


4. BOTTOM OF SAMPLE TRIMMED AND BASE PLATE ATTACHED. SAMPLE TIED TO SURFACE BY WINCH.

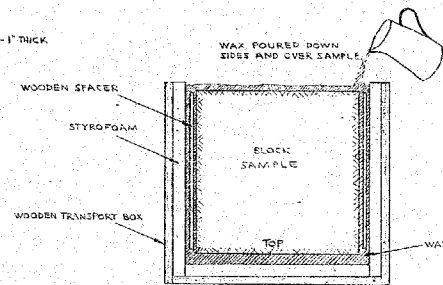
# SAMPLE HANDLING IN FIELD LABORATORY



5. BLOCK PLACED ON LID OF TRANSPORT BOX AND SAMPLING BOX DISMANTELED.



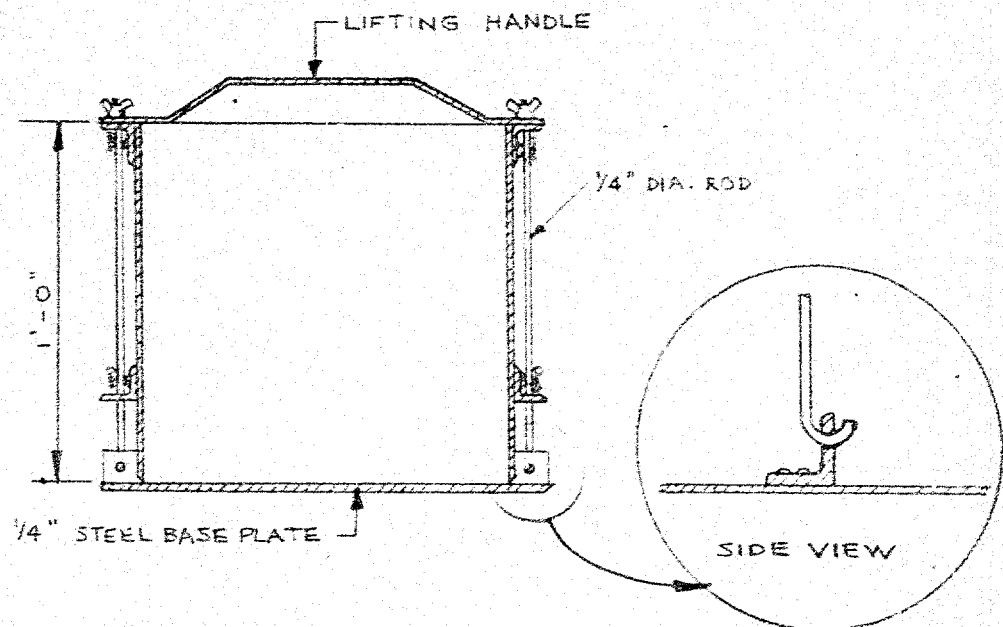
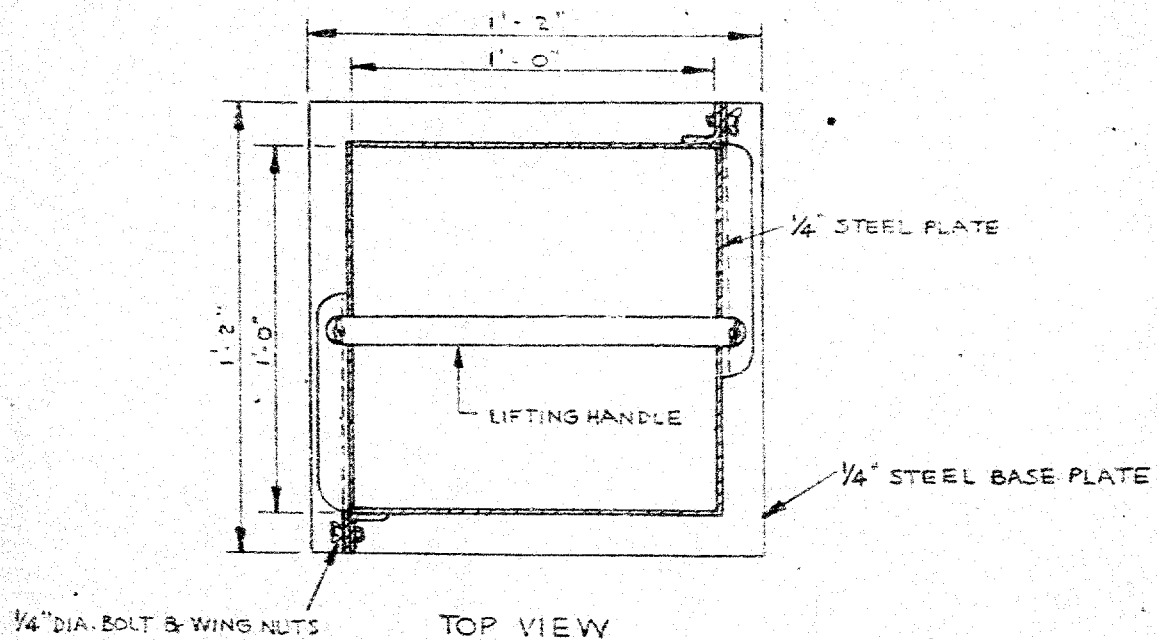
6. SAMPLE WRAPPED IN ALUMINUM FOIL. WOODEN TRANSPORT BOX, WITH 1" WAX ON BOTTOM, PLACED OVER SAMPLE.



7. TRANSPORT BOX TURNED OVER. WOODEN SPACERS INSERTED BETWEEN SAMPLE AND STYROFOAM AND SAMPLE SURROUNDED AND COVERED BY WAX.

DETAILS OF STEEL SAMPLING BOX AND WOODEN TRANSPORT BOXES SHOWN IN FIGURES 2 & 3.

NOT TO SCALE



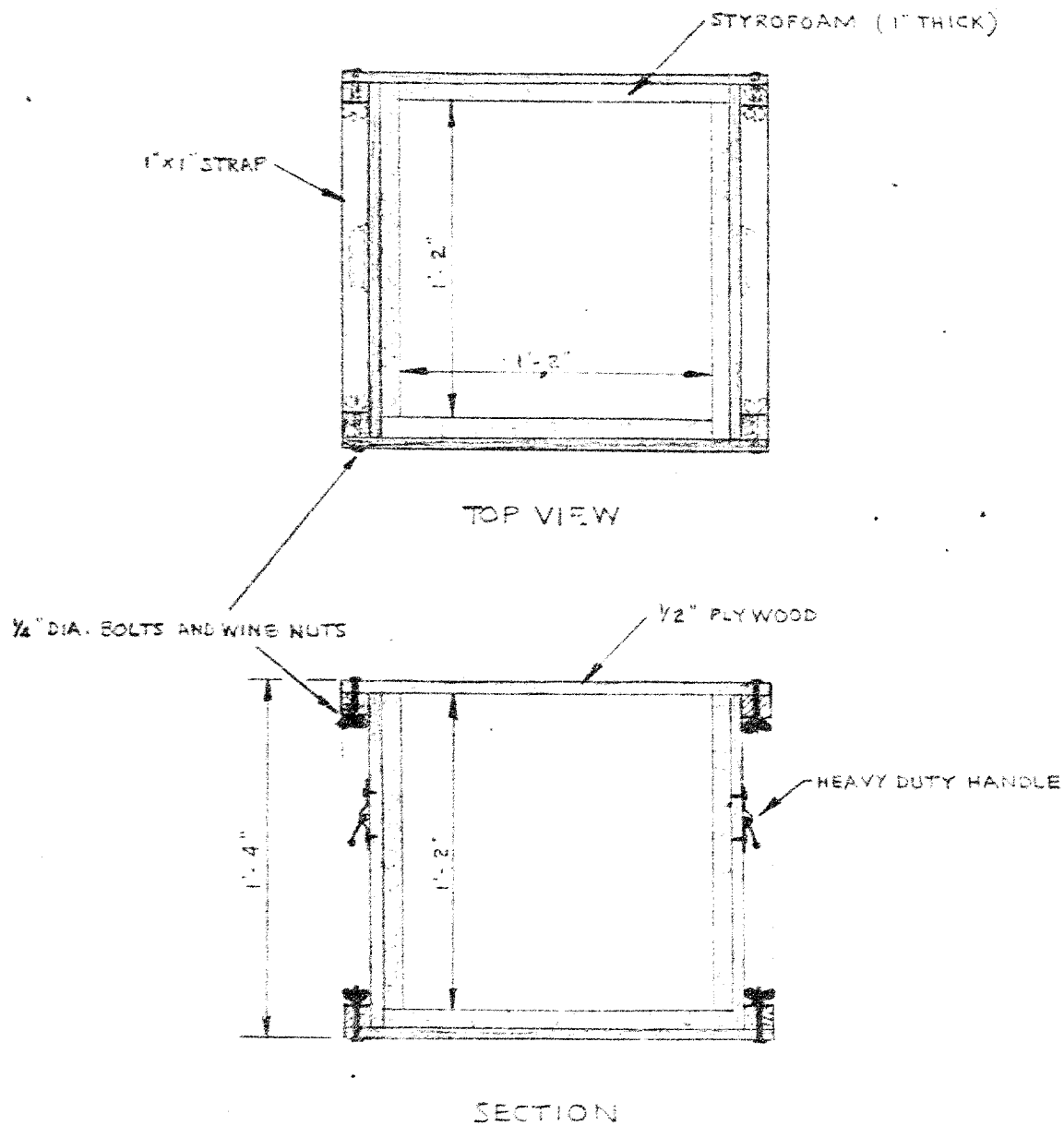
NOT TO SCALE

GOLDER &amp; ASSOCIATES

PROJECT NO. 100-1000000

# WOODEN TRANSPORT BOX

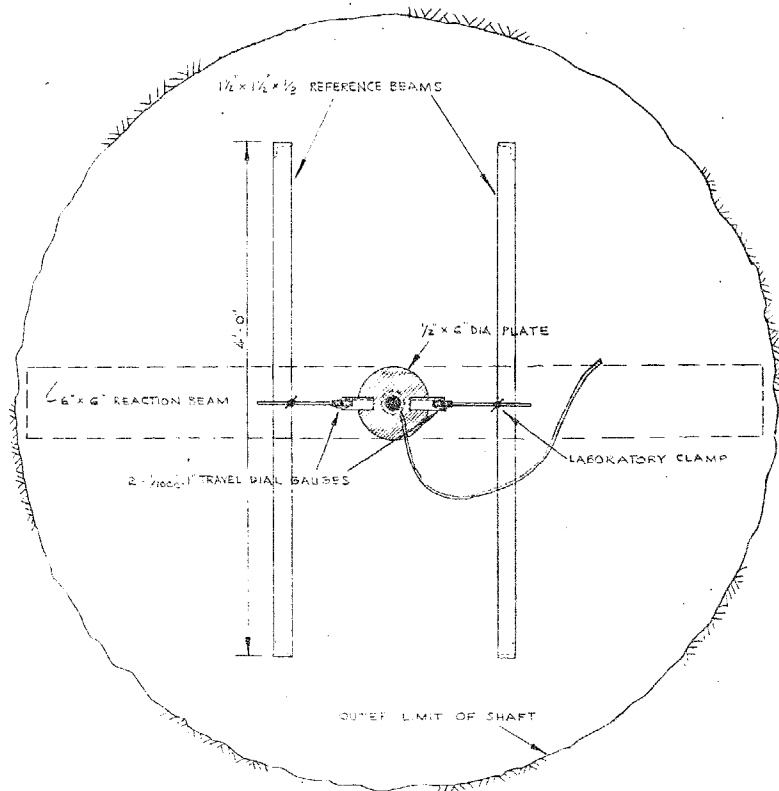
FIGURE 8



NOTE: ALL SIDES REMOVABLE

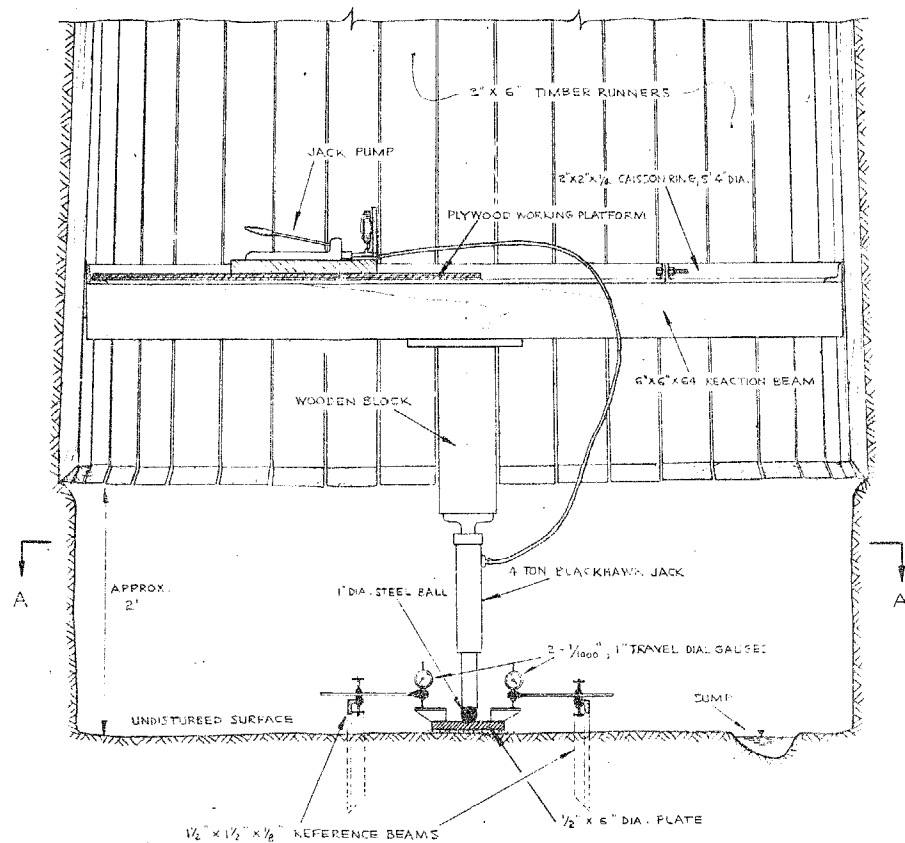
NOT TO SCALE

GOLDER & ASSOCIATES



VIEW A - A

NOT TO SCALE



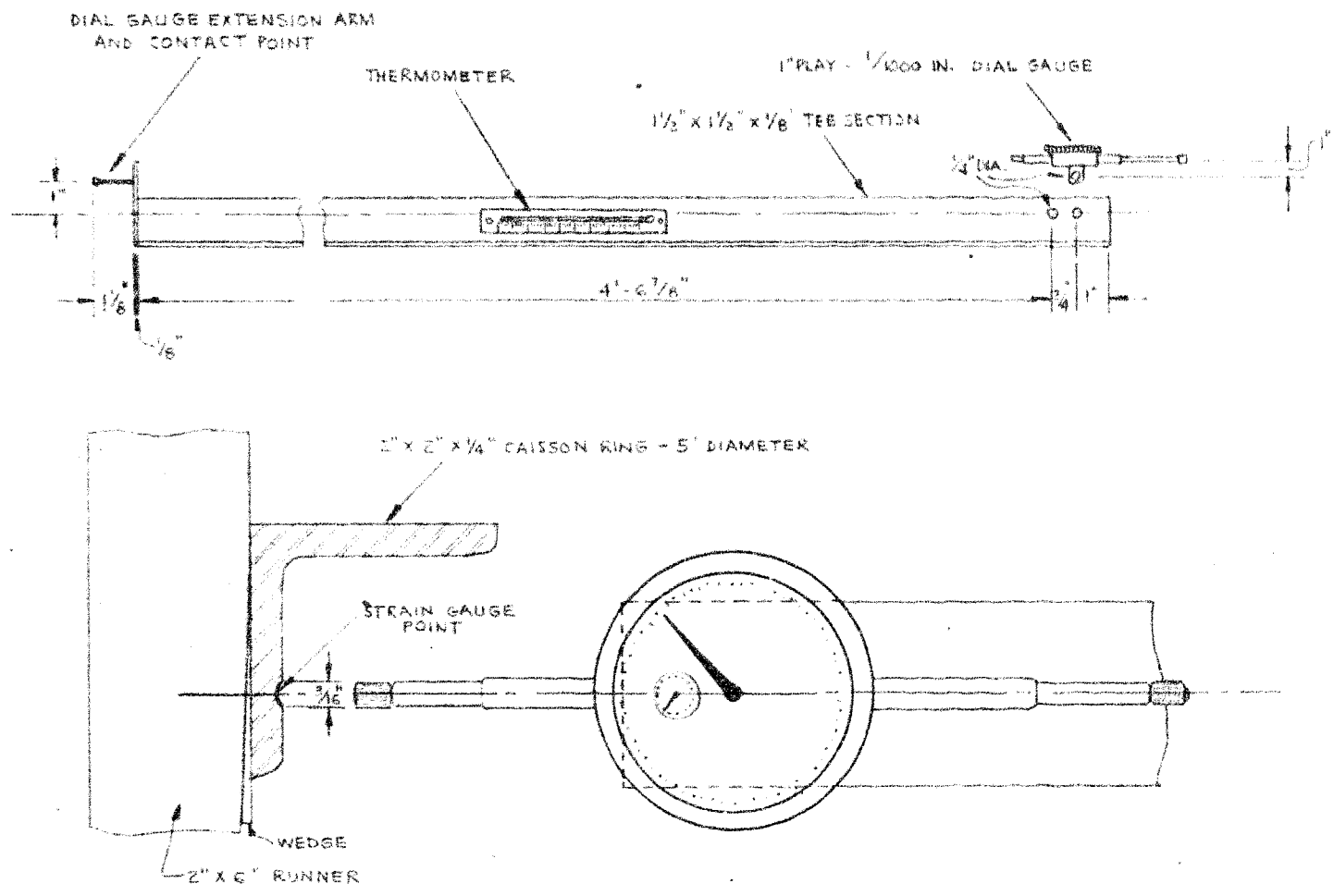
GOLDER & ASSOCIATES

Made  
Chkd.  
Appd.



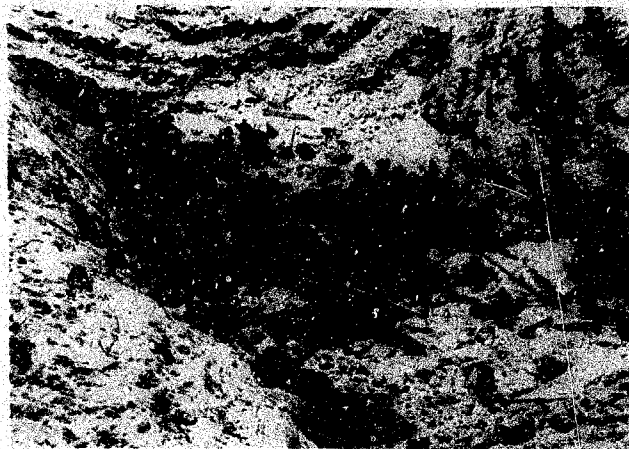
DETAILS OF STAIN GAUGE FOR MEASURING  
DIAMETER OF CHICAGO CAISSON RINGS

FIGURE 10

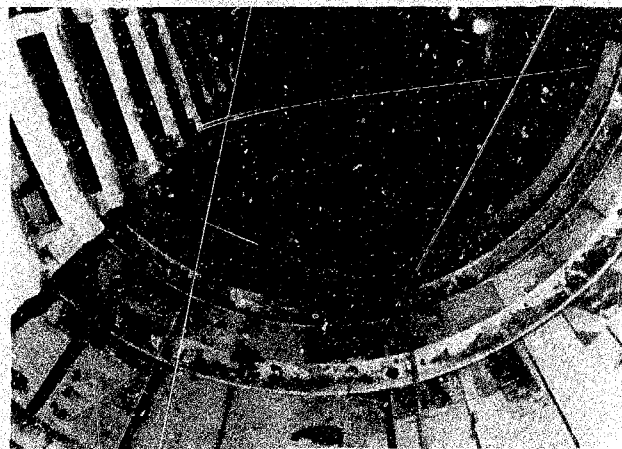


GOLDER & ASSOCIATES

Made (M.L.)  
Chkd. (P.T.)  
Appd. (P.T.)



1. OPEN - CUT EXCAVATION AND ENTRANCE TO 7.5 FT. SQUARE SHAFT.



2. LOOKING UP SHAFT FROM - 45 FT. LEVEL.

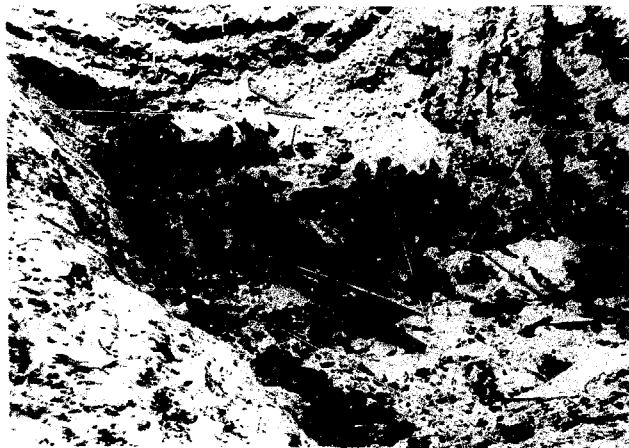


3. SIDE OF SHAFT - 51 TO - 52 FT. SHOWING LAYERED CLAYS FOUND IN BLOCK SAMPLE F.

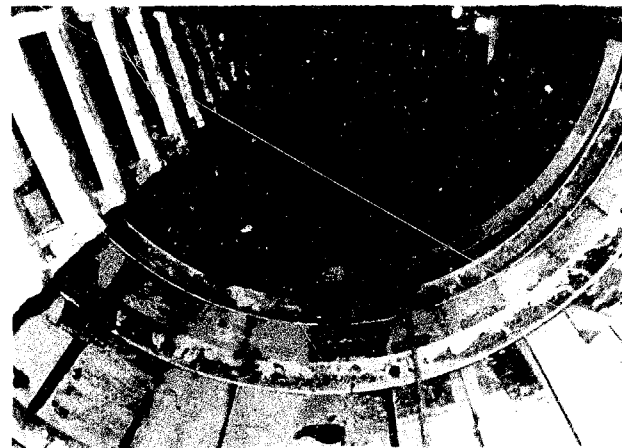


4. SIDE OF SHAFT - 60 TO - 61 FT. SHOWING OVERLAP OF SHORING AND 8.0 FT. DIAMETER RINGS.

GOLDER &amp; ASSOCIATES



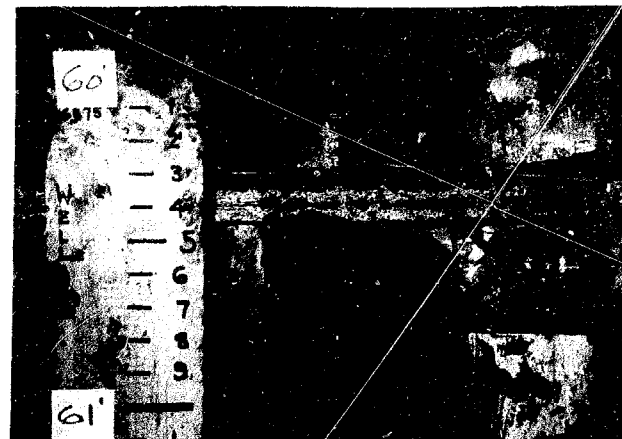
1. OPEN-CUT EXCAVATION AND ENTRANCE TO 7.5 FT. SQUARE SHAFT.



2. LOOKING UP SHAFT FROM - 45 FT. LEVEL.



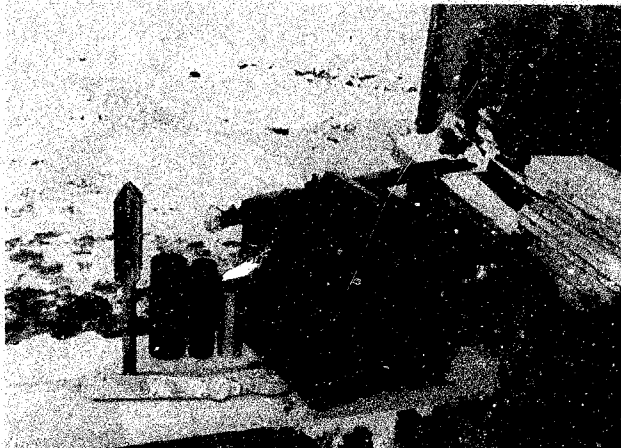
3. SIDE OF SHAFT - 51 TO - 52 FT. SHOWING LAYERED CLAYS FOUND IN BLOCK SAMPLE F.



4. SIDE OF SHAFT - 60 TO - 61 FT. SHOWING OVERLAP OF SHORING AND 5.0 FT. DIAMETER RINGS.

PHOTOGRAPHS  
SHAFT

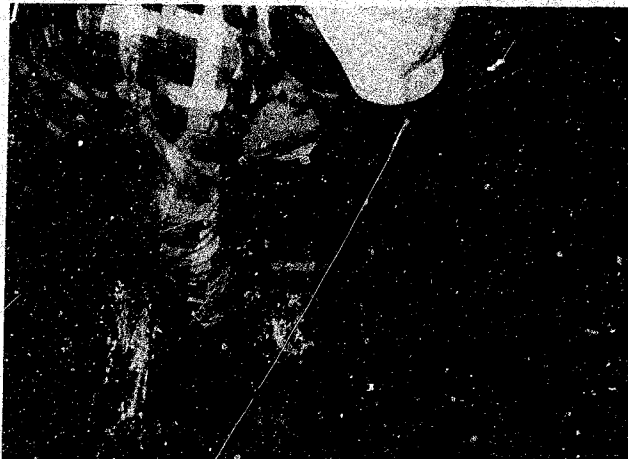
FIGURE 11



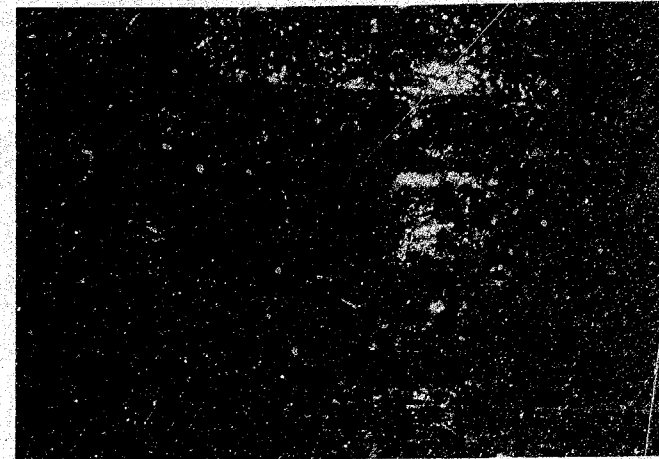
5. BLOCK SAMPLING BOX , CLAY KNIFE, 3 IN. AND 2 IN. SHELBY TUBES , 2 IN. BRASS SLEEVE, STANCARD D.H.O. VANE AND TORQUE WRENCH.



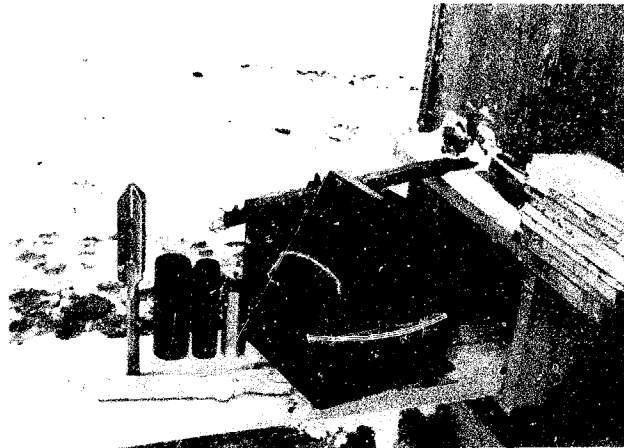
6. ISOLATING BLOCK SAMPLE BY DIGGING 2 IN. WIDE TRENCHES AROUND BLOCK WITH CLAY KNIVES.



7. SLIDING SAMPLING BOX OVER ISOLATED BLOCK SAMPLE.



8. CUTTING THE BASE OF BLOCK SAMPLE WITH 12 IN. BLADE SHOVEL.



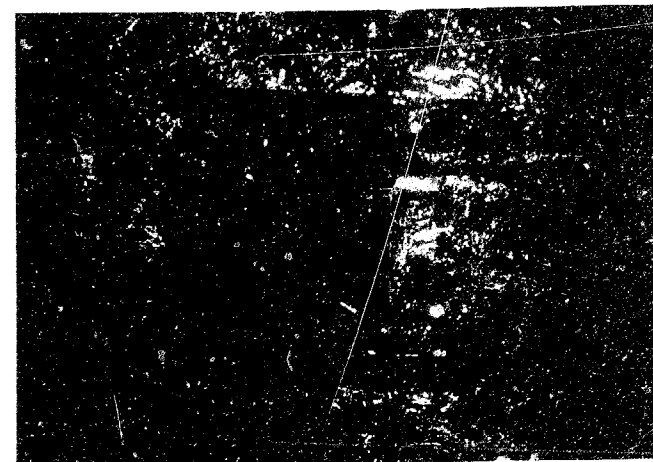
5 BLOCK SAMPLING BOX , CLAY KNIFE, 3 IN AND 2 IN SHELBY TUBES , 2 IN BRASS SLEEVE, STANDARD D.H.O. VANE AND TORQUE WRENCH.



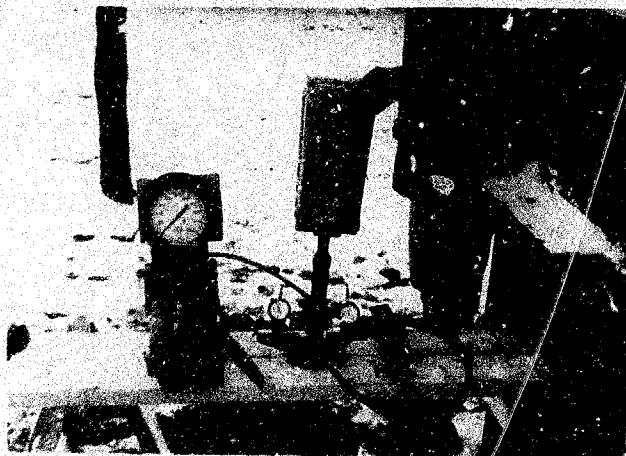
6 ISOLATING BLOCK SAMPLE BY DIGGING 2 IN. WIDE TRENCHES AROUND BLOCK WITH CLAY KNIVES.



7 SLIDING SAMPLING BOX OVER ISOLATED BLOCK SAMPLE.



8 CUTTING THE BASE OF BLOCK SAMPLE WITH 12 IN. BLADE SHOVEL.



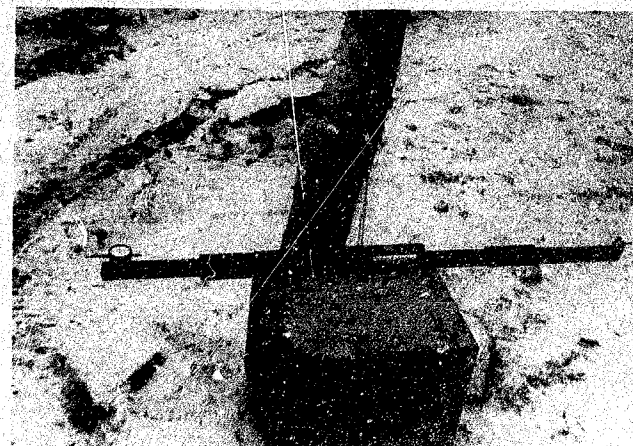
9. IDEALIZED PLATE LOAD TEST LAYOUT SHOWING HYDRAULIC PUMP, PRESSURE GAUGE, 4 TON JACK, BEARING PLATE AND DIAL GAUGES.



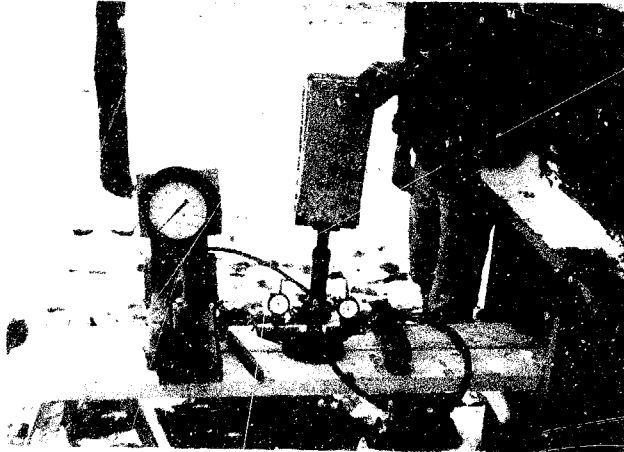
10. PLATE LOAD TEST AT -75.6 FT. SHOWING BEARING PLATE, JACK, DIAL GAUGES AND REFERENCE BEAMS.



11. PLATE LOAD TEST AT -75.6 FT. SHOWING REACTION BEAM AND HYDRAULIC PUMP.



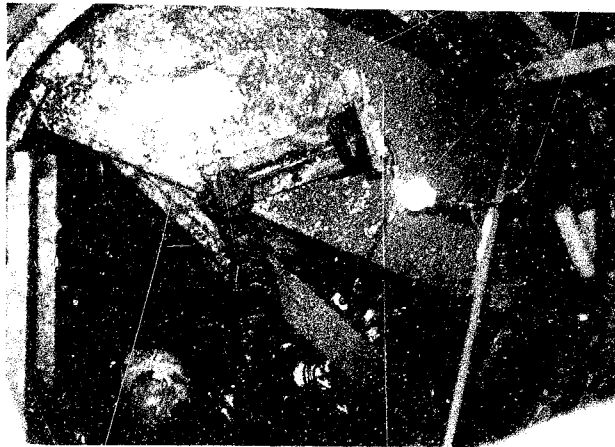
12. STRAIN GAUGE USED TO MEASURE 5.0 FT. DIAMETER CAISSON RINGS.



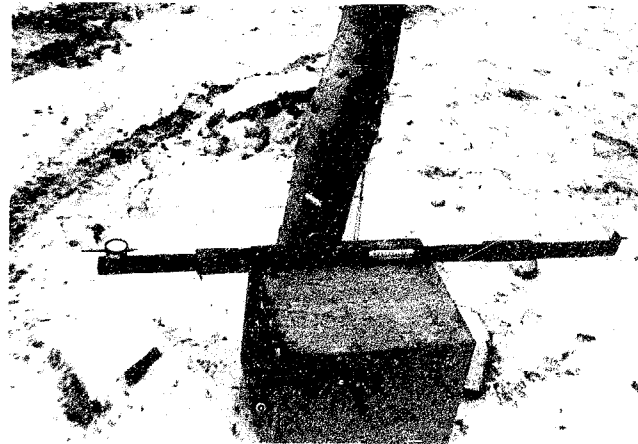
9. IDEALIZED PLATE LOAD TEST LAYOUT SHOWING HYDRAULIC PUMP, PRESSURE GAUGE, 4 TON JACK, BEARING PLATE AND DIAL GAUGES.



10. PLATE LOAD TEST AT -75.6 FT. SHOWING BEARING PLATE, JACK, DIAL GAUGES AND REFERENCE BEAMS.



11. PLATE LOAD TEST AT -75.6 FT. SHOWING REACTION BEAM AND HYDRAULIC PUMP.



12. STRAIN GAUGE USED TO MEASURE 5.0 FT. DIAMETER CAISSON RINGS.

GOLDER & ASSOCIATES

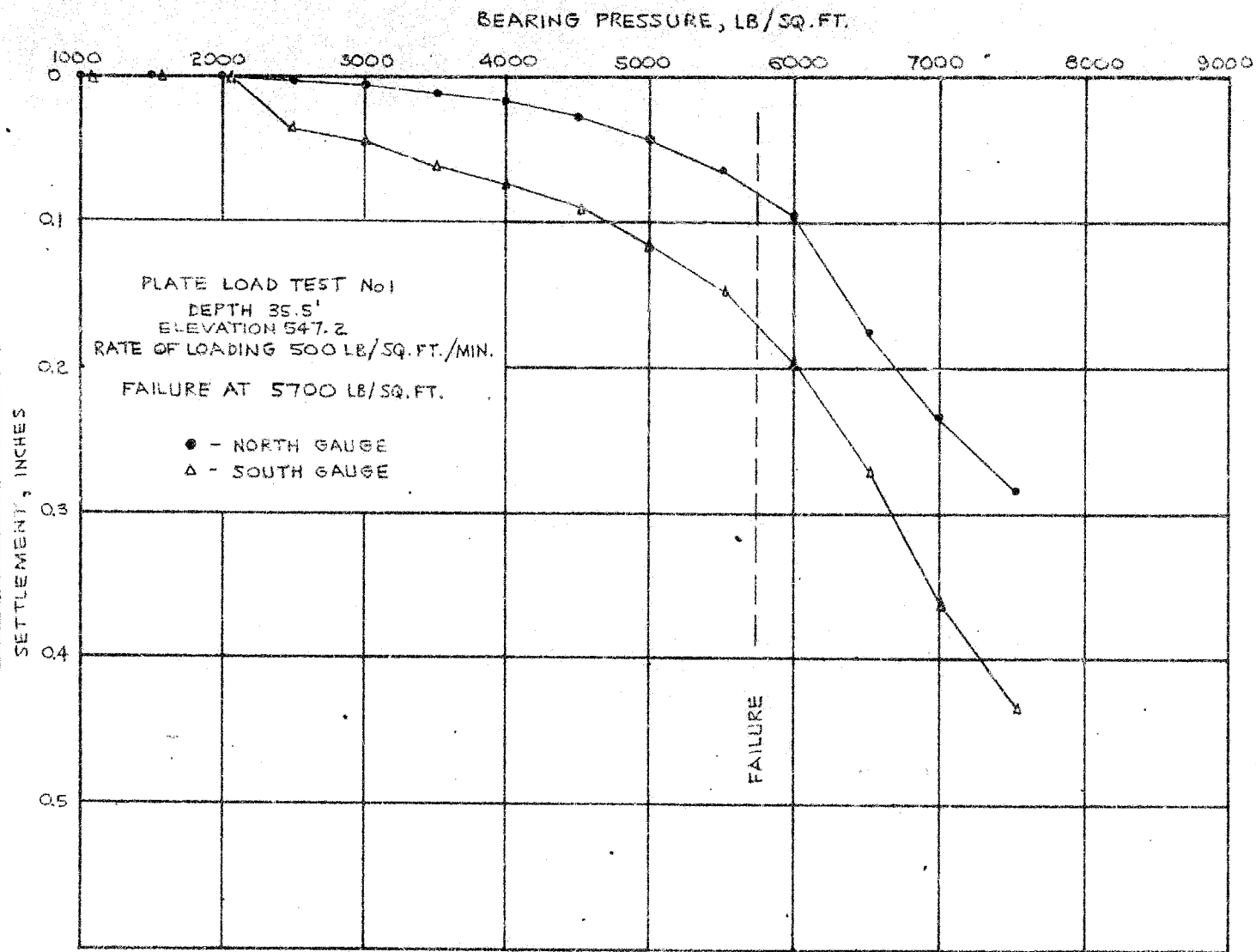
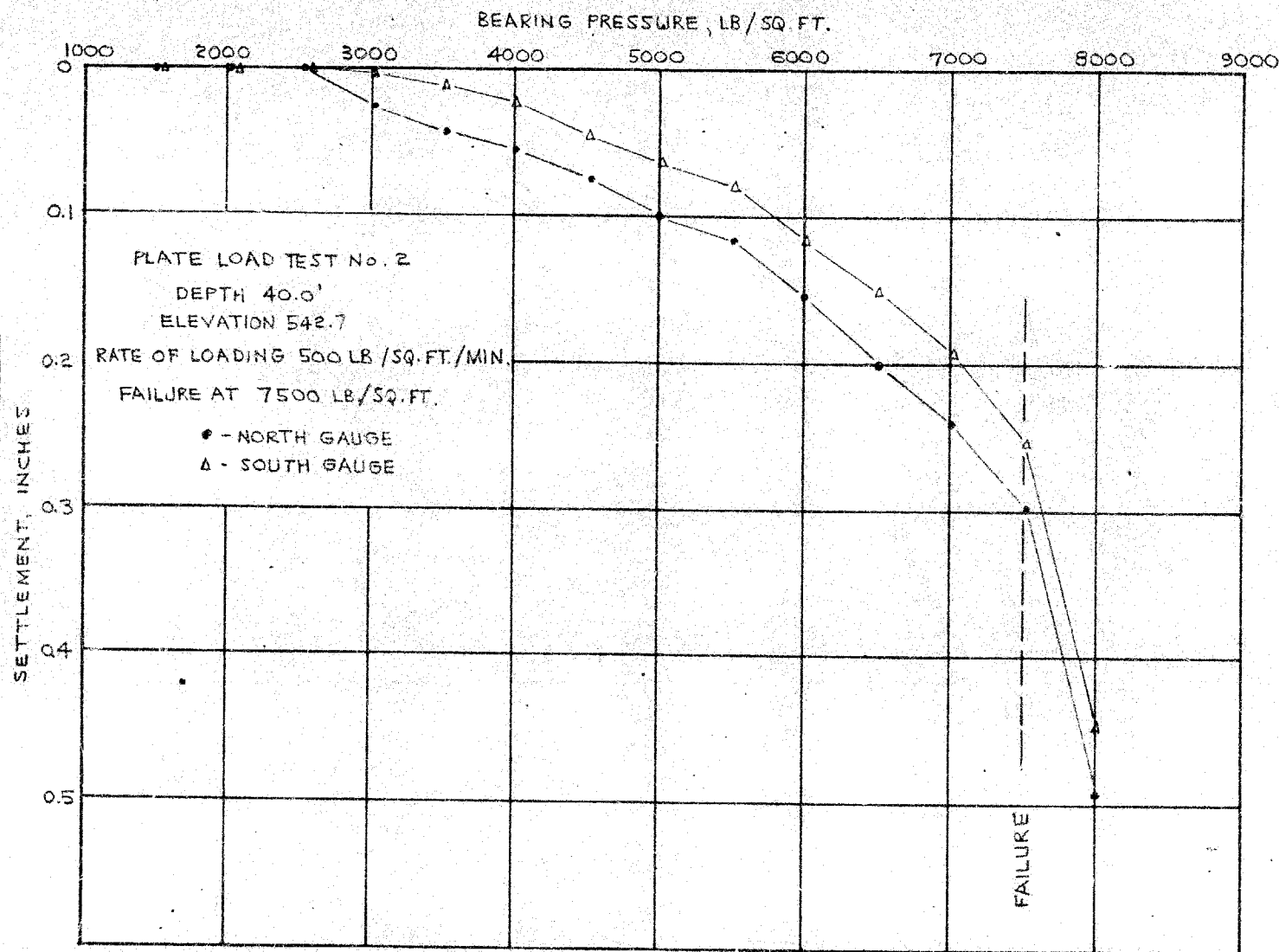




PLATE LOAD TEST No. 2

FIGURE 15

GOLDER &amp; ASSOCIATES



GOLDER &amp; ASSOCIATES

SETTLEMENT, INCHES

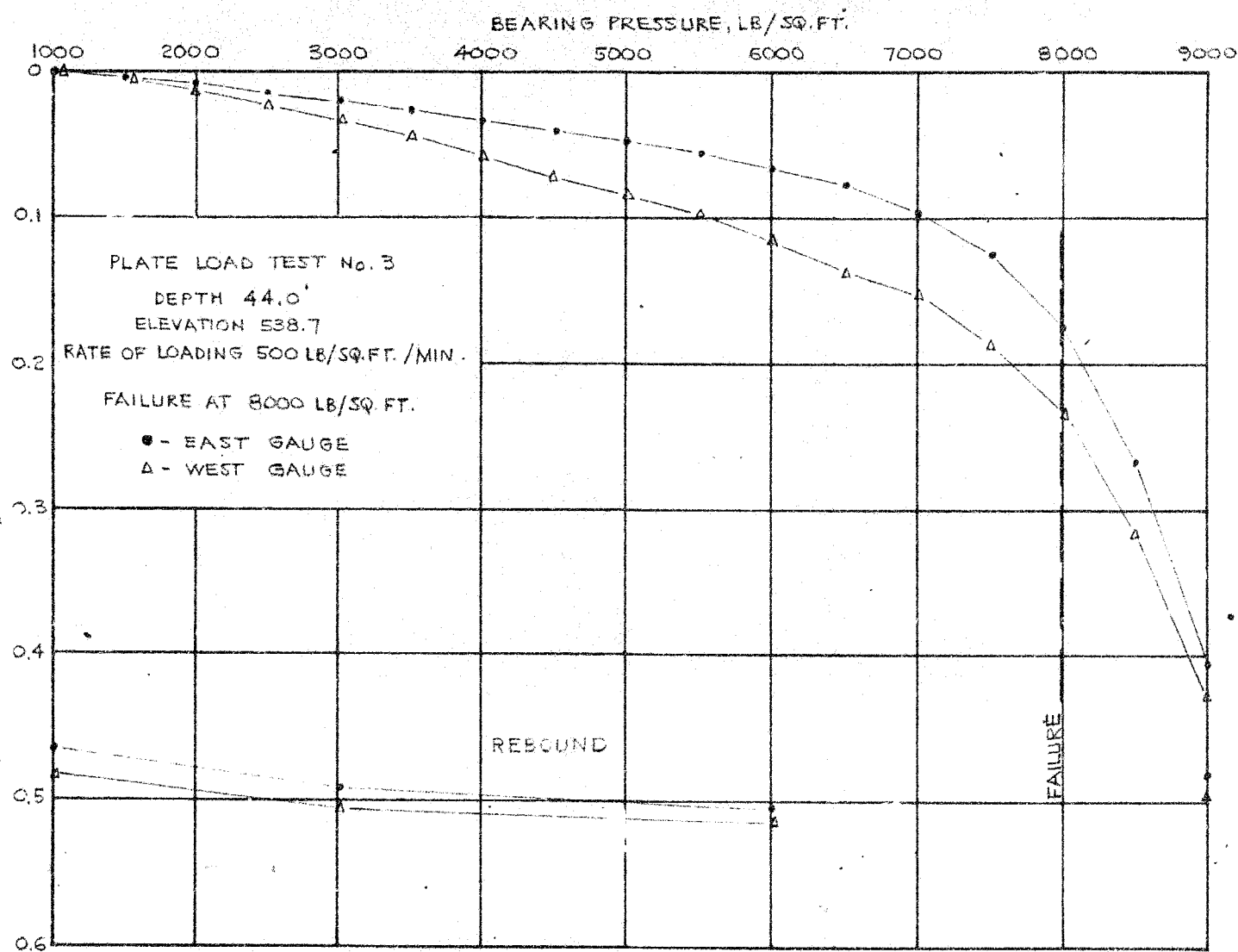


PLATE LOAD TEST No. 3

FIGURE 16

GOLDER &amp; ASSOCIATES

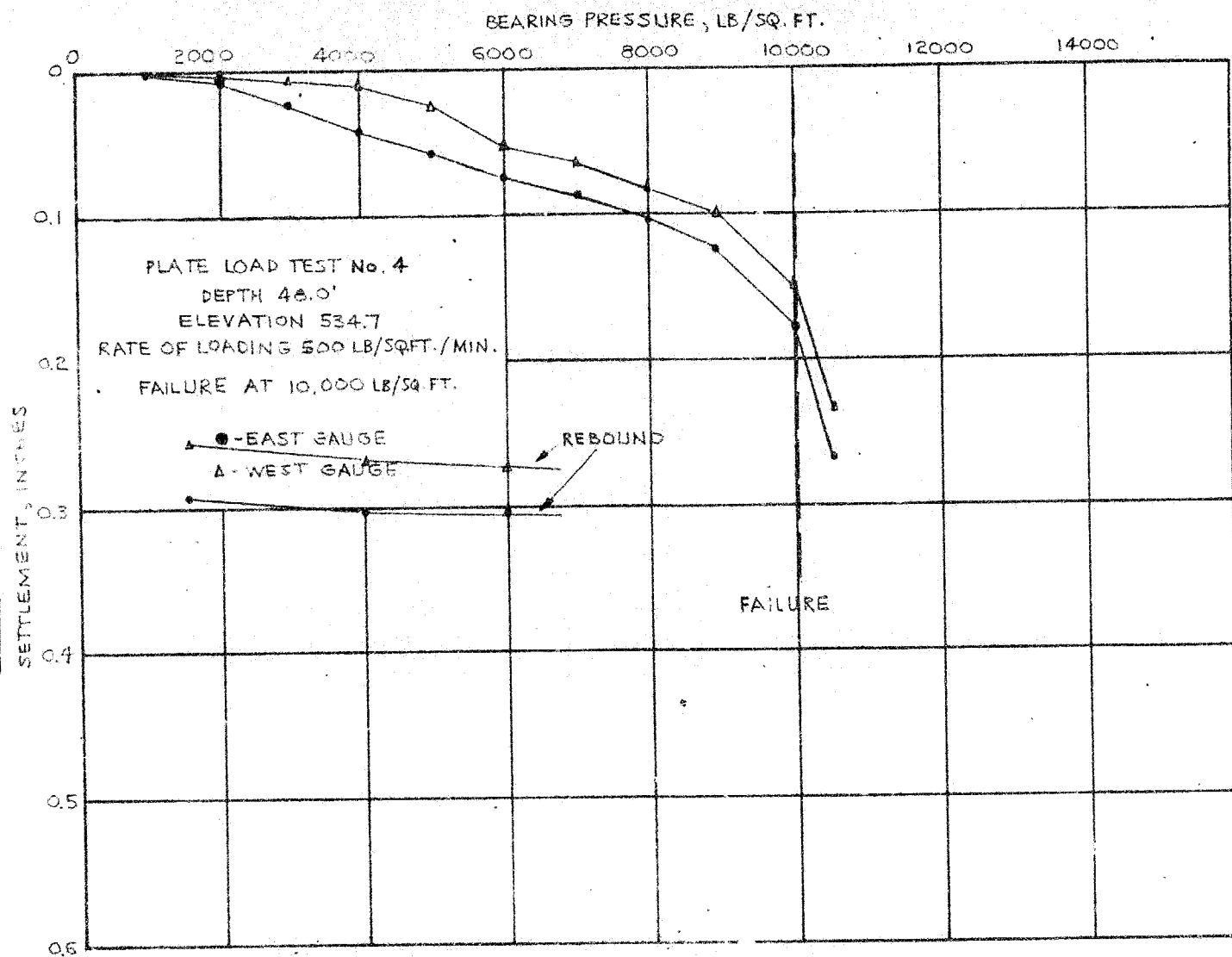


PLATE LOAD TEST No. 4

BEARING PRESSURE, LB/SQ. FT.

1000 2000 3000 4000 5000 6000 7000 8000 9000

PLATE LOAD TEST No. 5

DEPTH 52.2'

ELEVATION 530.5

RATE OF LOADING 500 LB/SQ. FT./MIN.

FAILURE AT 7000 LB/SQ. FT.

● - EAST GAUGE

△ - WEST GAUGE

FAILURE

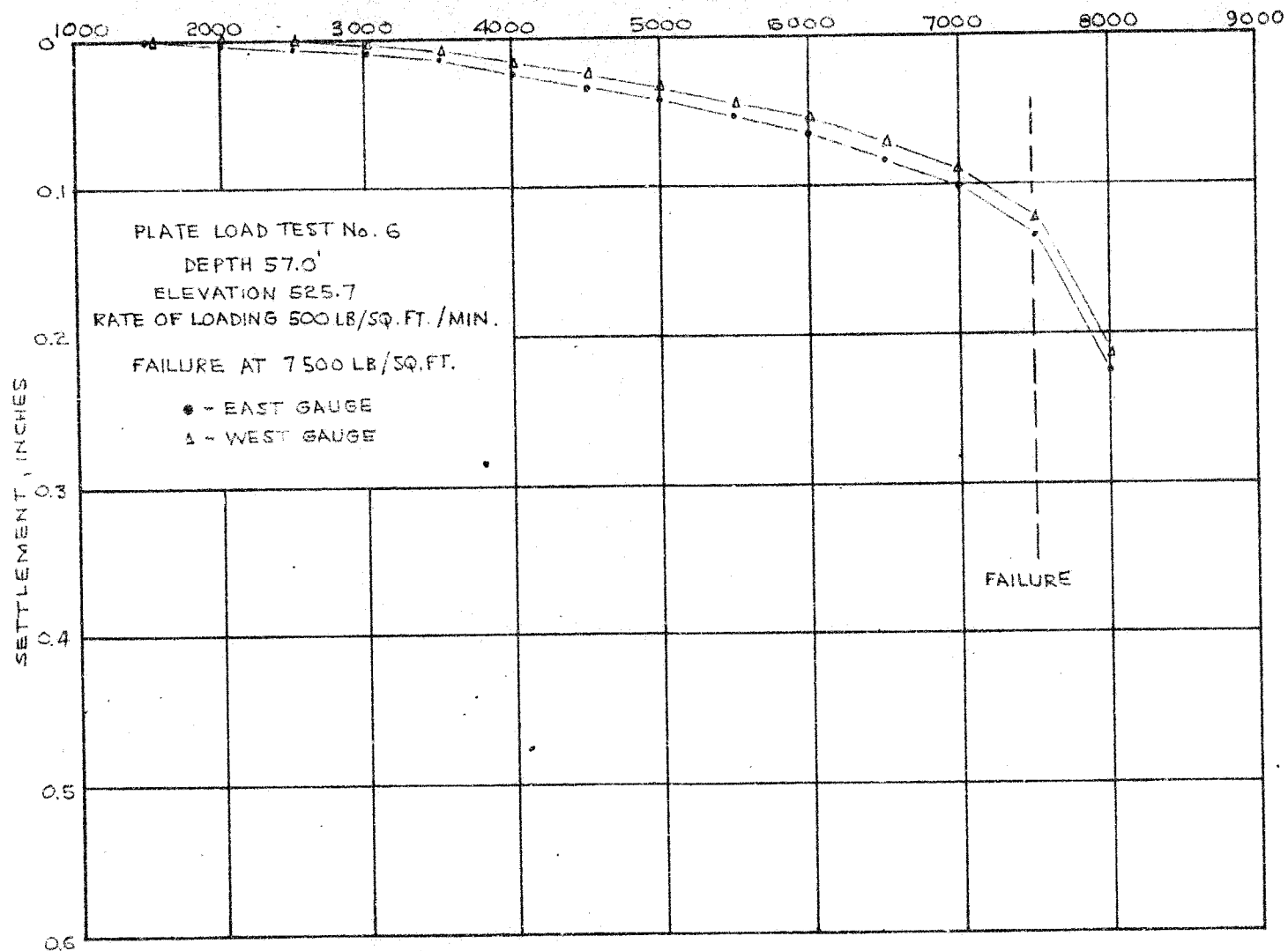
SETTLEMENT, INCHES

GOLDER &amp; ASSOCIATES

PLATE LOAD TEST No. 5

FIGURE 16

BEARING PRESSURE, LB/SQ. FT.



GOLDER &amp; ASSOCIATES

PLATE LOAD TEST No. 6

FIGURE 19

BEARING PRESSURE, LB/SQ. FT.

1000 2000 3000 4000 5000 6000 7000 8000 9000

PLATE LOAD TEST No. 7  
DEPTH 62.2'  
ELEVATION 520.5  
RATE OF LOADING 500 LB/SQ. FT./MIN.

FAILURE AT 7,000 LB/SQ. FT.

● - EAST GAUGE  
△ - WEST GAUGE

FAILURE

GOLDER &amp; ASSOCIATES

SETTLEMENT, INCHES

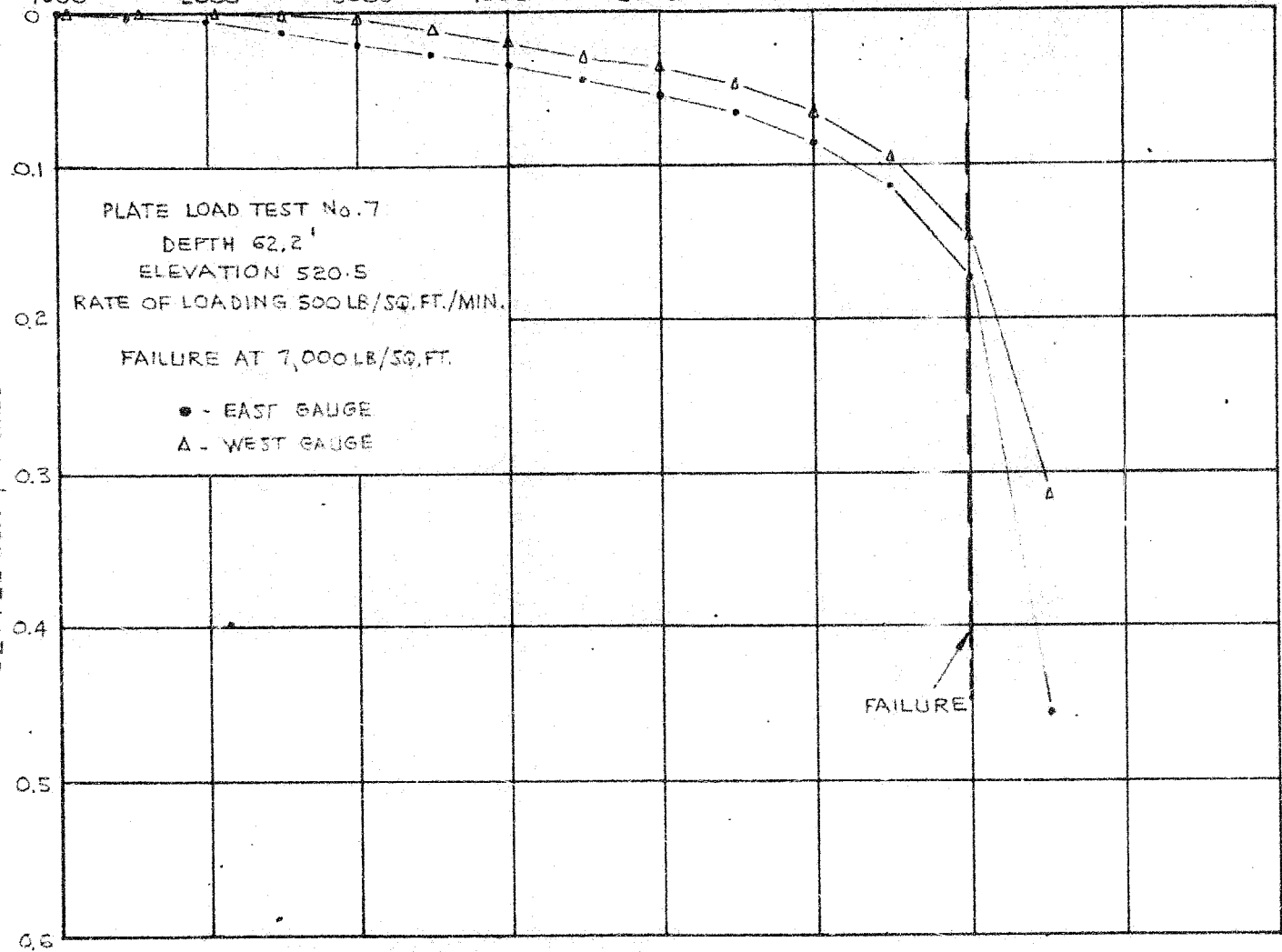
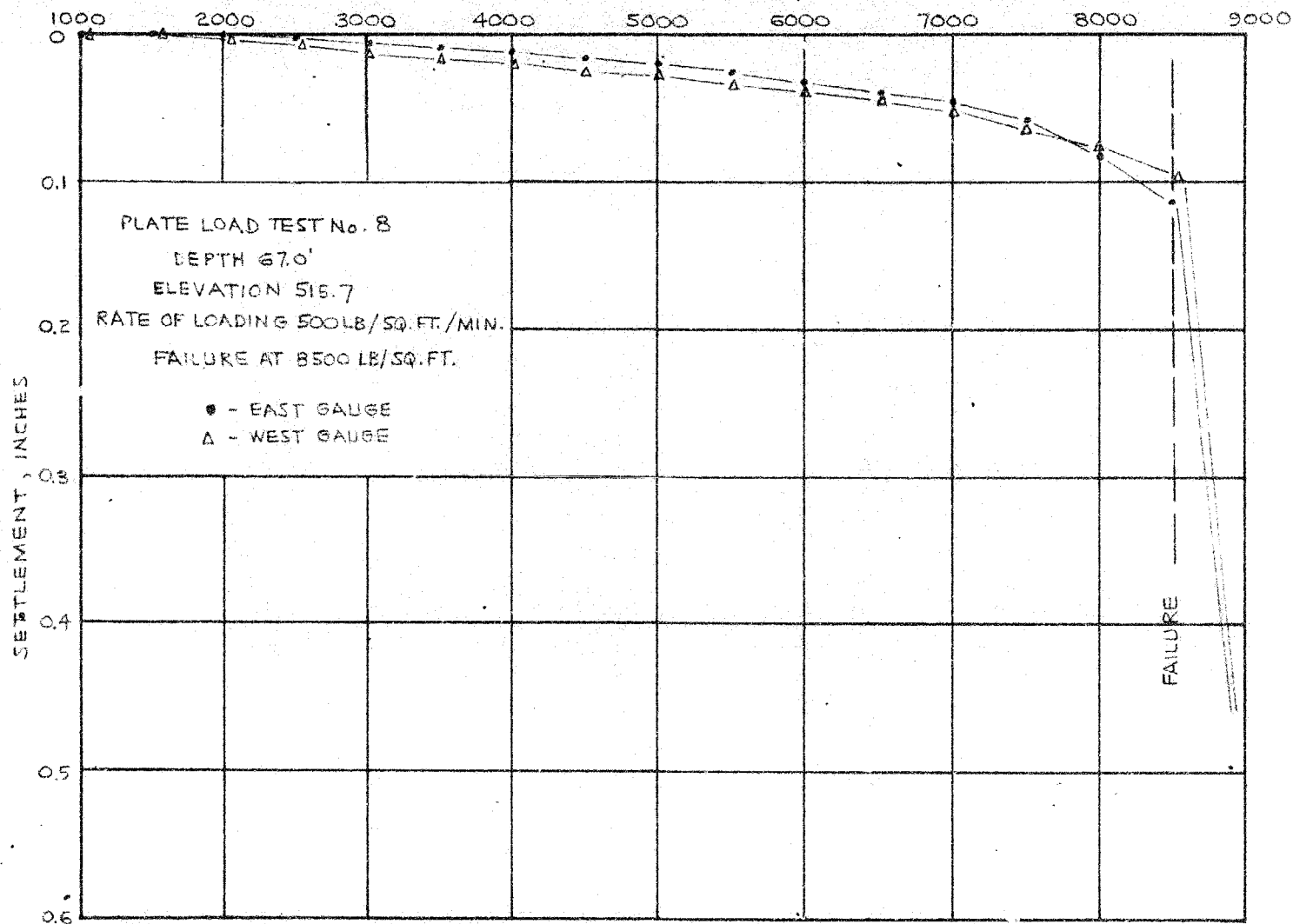


PLATE LOAD TEST No. 7

BEARING PRESSURE, LB/SQ. FT.

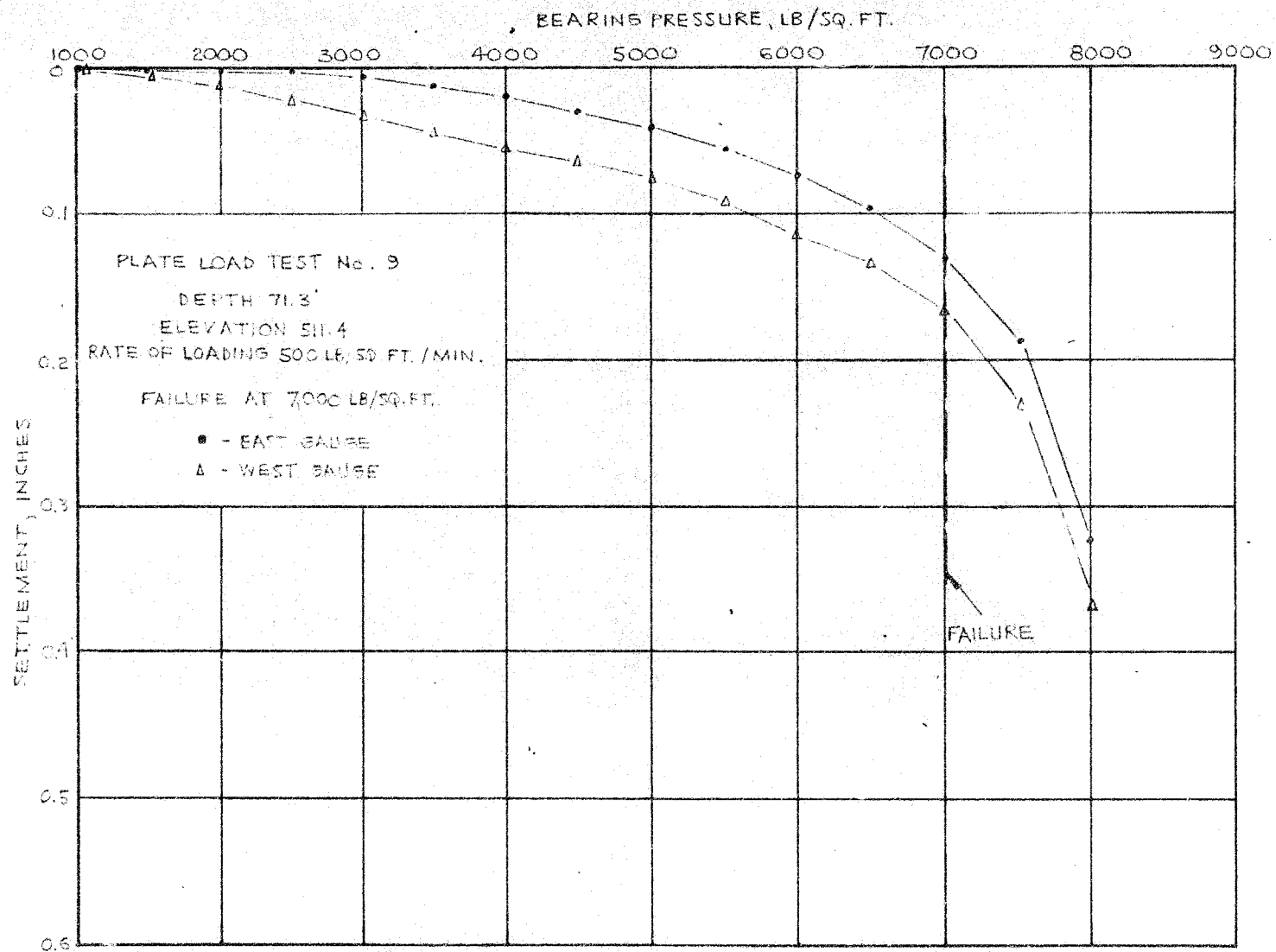


GOLDER &amp; ASSOCIATES

PLATE LOAD TEST No. 8

FIGURE 21

81



GOLDER &amp; ASSOCIATES

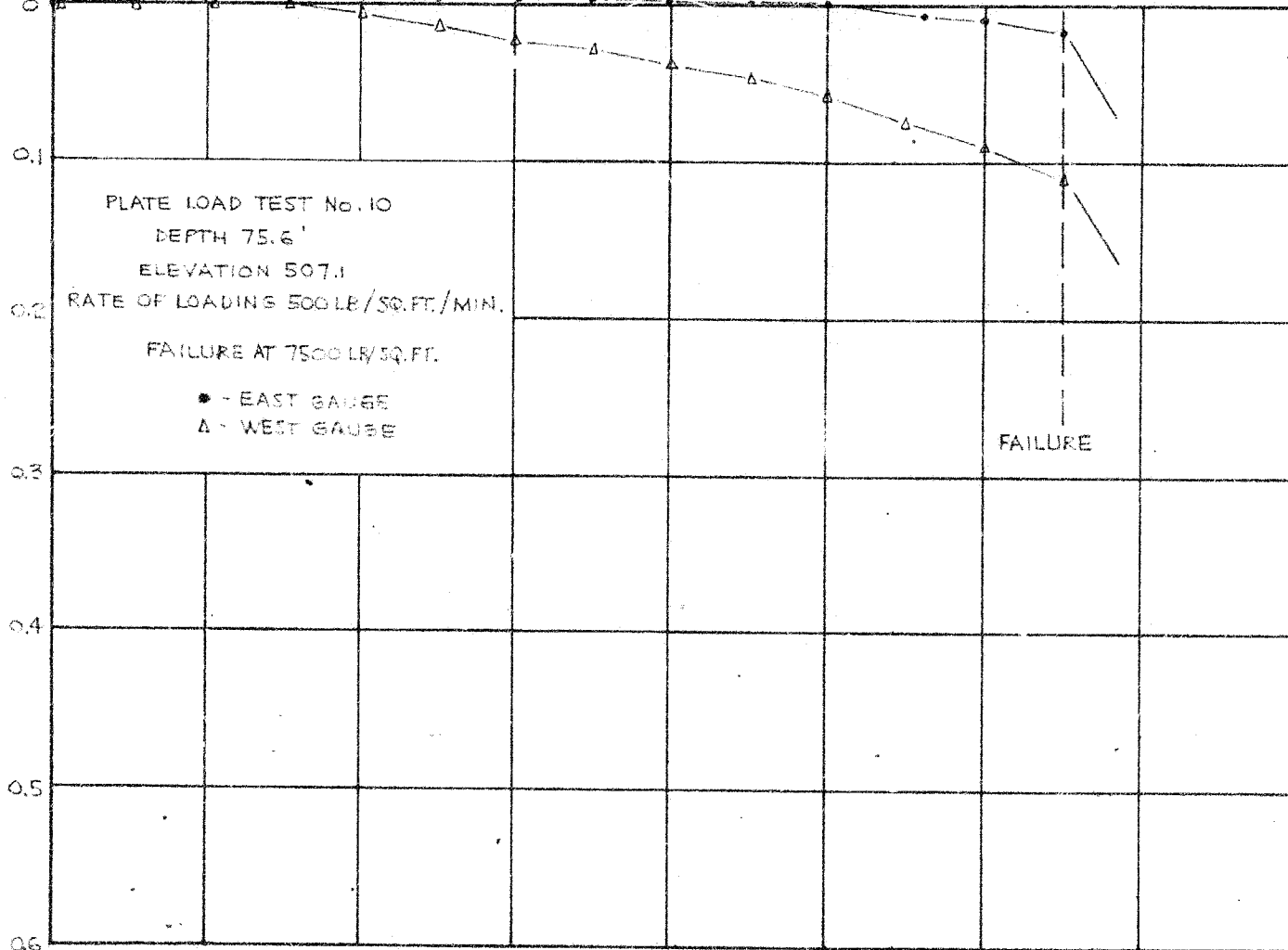
PLATE LOAD TEST No. 9

FIGURE 20



BEARING PRESSURE, LB/SQ. FT.

1000 2000 3000 4000 5000 6000 7000 8000 9000



GOLDER &amp; ASSOCIATES

SETTLEMENT, INCHES

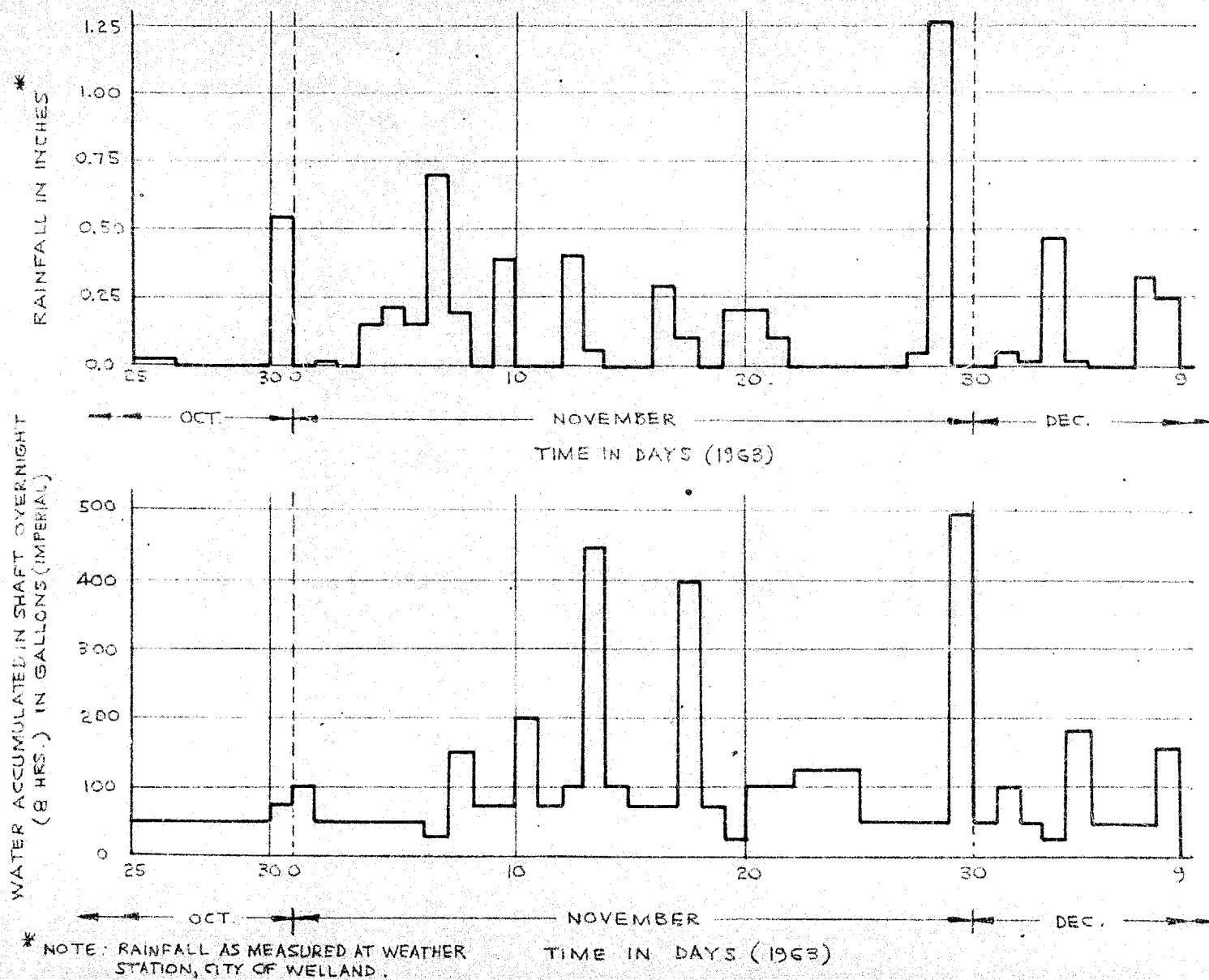
PLATE LOAD TEST No. 10

FIGURE 23

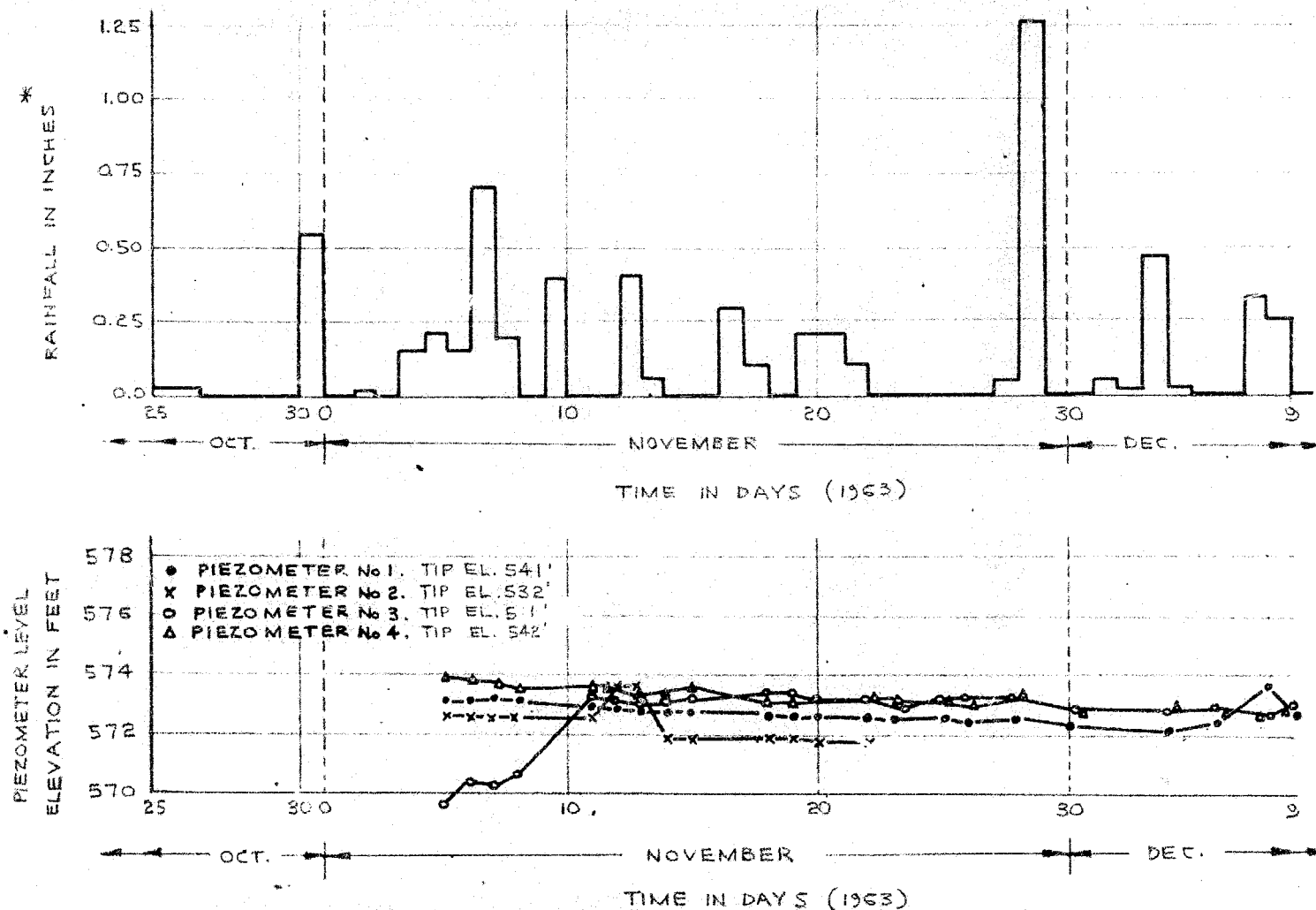
DAILY RAINFALL AND OVERNIGHT WATER  
ACCUMULATION CHART

FIGURE 24

GOLDER &amp; ASSOCIATES



GOLDER &amp; ASSOCIATES

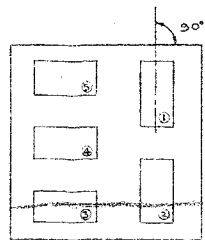
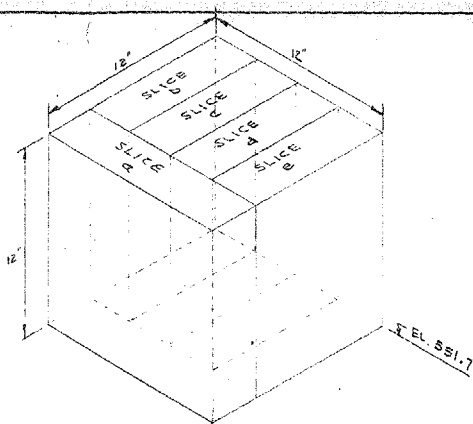


\* NOTE: RAINFALL AS MEASURED AT WEATHER STATION, CITY OF WELLAND.

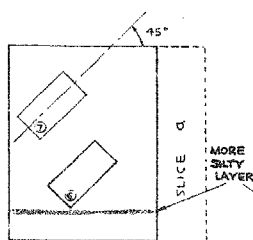
FOR LOCATION OF PIEZOMETERS SEE FIGURE 2.

DAILY RAINFALL AND PIEZOMETER LEVELS

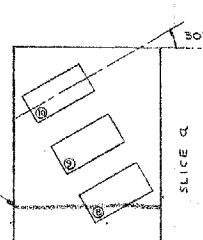
FIGURE 25



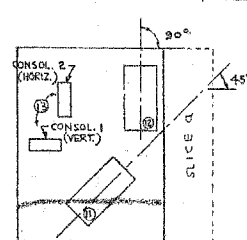
SLICE a



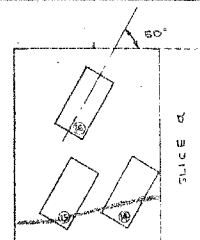
SLICE b



SLICE c



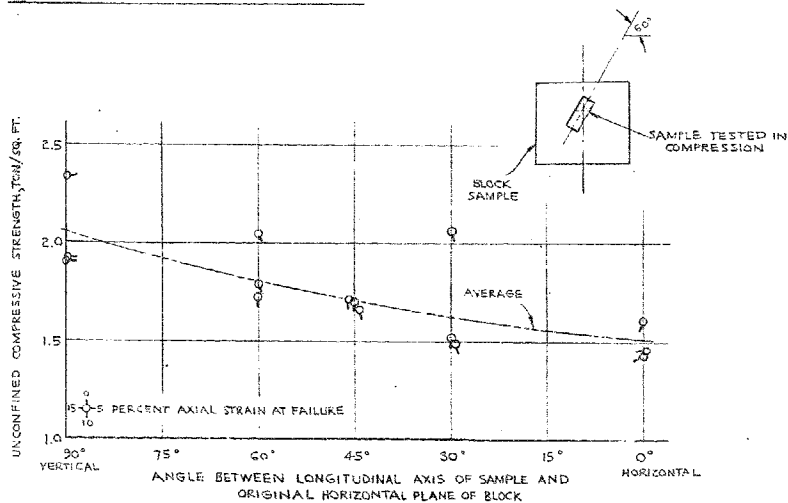
SLICE d



SLICE e

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLES

SUMMARY OF UNDRAINED SHEAR STRENGTH RESULTS  
BLOCK A - ELEVATION 551.7 TO 551.7

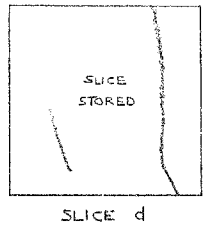
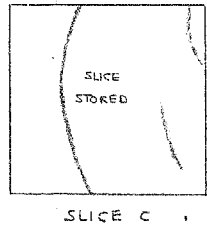
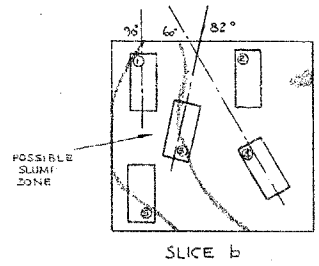
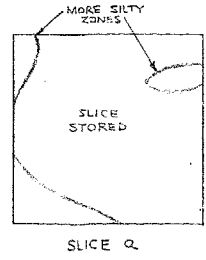
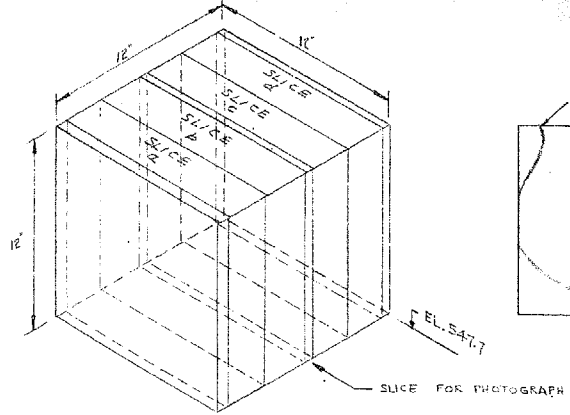
FIGURE 26

BLOCK A, SAMPLE #	ANGLE OF SAMPLE TO HORIZONTAL PLANE	POSITION IN BLOCK	MOISTURE CONTENT (%) BEFORE AFTER	PLASTIC LIMIT	LIQUID LIMIT	UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ. FT.)	FAILURE STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
1	90°	TOP	20.6 22.2			1.90	5	—
2	90°	BOTTOM	19.3 19.2			2.33	4½	132.8
3	0°	BOTTOM	19.6 20.3			1.47	15	133.0
4	0°	MIDDLE	19.9 20.0			1.61	11	132.8
5	0°	TOP	20.8 21.2			1.44	14	135.8
6	45°	BOTTOM	19.6 20.0			1.72	10	134.2
7	45°	TOP	21.0 21.6	16.1	29.7	1.66	9	131.1
8	30°	BOTTOM	18.4 19.2			1.49	8	131.8
9	30°	MIDDLE	19.2 19.3	15.8	27.7	2.06	9	131.6
10	30°	TOP	16.2 18.1			1.82	10	132.7
11	45°	BOTTOM	19.3 19.9	18.1	26.8	1.71	10	133.9
12	90°	MIDDLE	20.3 21.9			1.91	5	131.1
14	60°	BOTTOM	19.5 19.4			1.76	10	133.2
15	60°	BOTTOM	20.1 19.4	16.2	27.5	2.05	8	132.9
16	60°	TOP	17.9 19.9			1.80	8	133.1

NOTE: SAMPLE DIMENSIONS, DIAMETER 2.0 INCHES

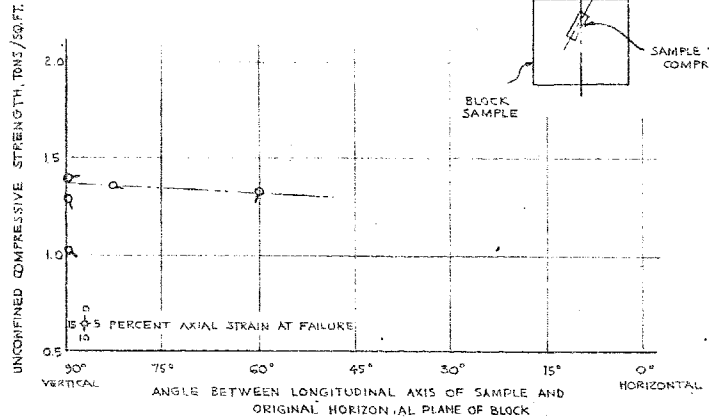
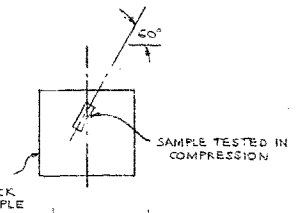
FOOTER & ASSOCIATES

Made by  
CHB



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

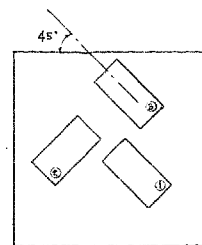
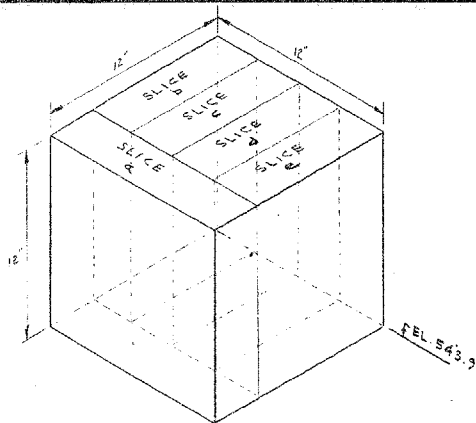
BLOCK SAMPLE SHOWING SLICING



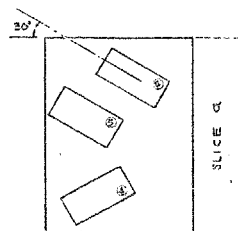
VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLES

TABLE OF RESULTS									
BLOCK & SAMPLE #	ANGLE OF SAMPLE TO HORIZONTAL PLANE	POSITION IN BLOCK	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ. FT.)	FAILURE STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)	
1	90°	TOP	25.0, 20.5, 20.2, 20.5, 42.0, 51.5	16.4	26.4	1.02	7	130.3	
2	90°	TOP	21.1, 19.6, 19.5, 19.8, 28.5	16.6 16.4	23.9 25.4	1.26	8½	134.6	
3	82°	MIDDLE	15.4, 40.6, 22.7, 20.8, 21.6, 22.3, 20.8	—	—	1.32	6	132.2	
4	60°	BOTTOM	18.8, 17.1, 18.9, 19.4, 20.0, 19.5	—	—	1.31	11	135.8	
5	90°	BOTTOM	14.0, 22.6, 22.1, 21.8	—	—	1.36	4½	130.2	

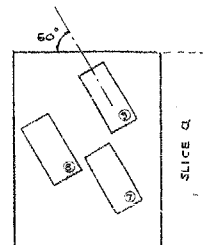
NOTE: SAMPLE DIMENSIONS, DIAMETER 2.0 INCHES  
HEIGHT 4.0 "



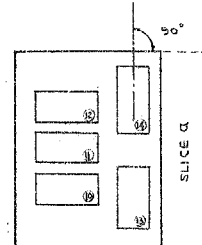
SLICE a



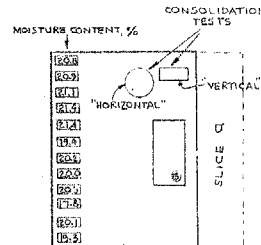
SLICE b



SLICE c



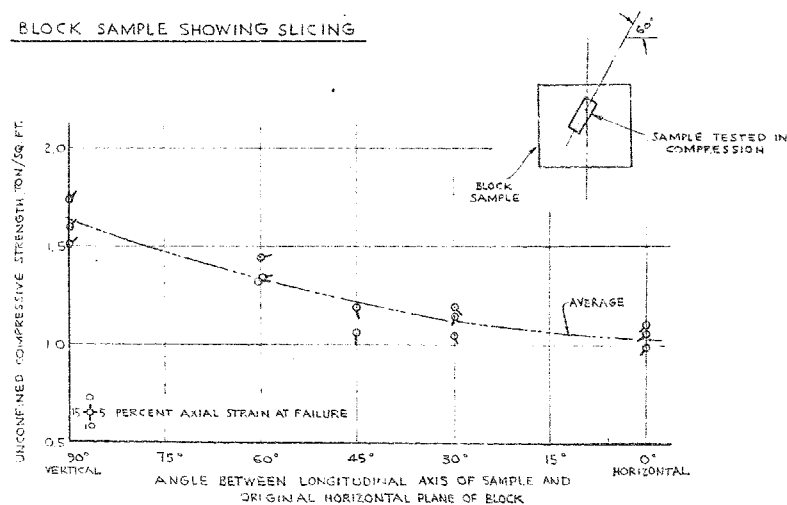
SLICE d



SLICE e

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING

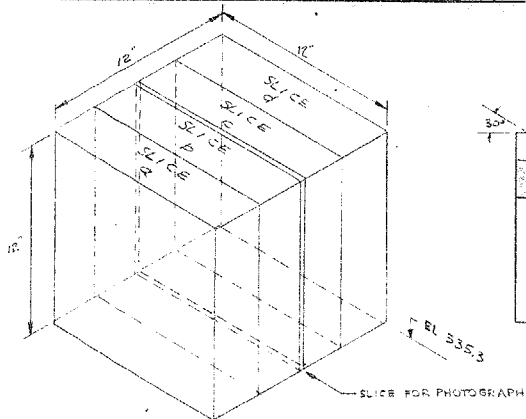


VARIAION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLES

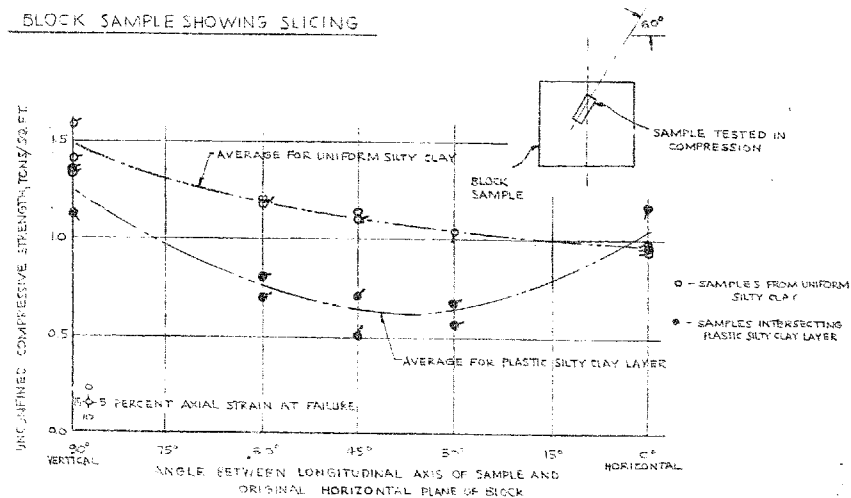
BLOCK C, SAMPLE #	ANGLE OF SAMPLE TO HORIZONTAL PLANE	POSITION IN BLOCK	MOISTURE CONTENT (%)			PLASTIC LIMIT	LIQUID LIMIT	UNCONFINED COMPRESSIVE STRENGTH (LBS/SQ. FT.)	FAILURE STRAIN (%)	UNIT WEIGHT (LB/CU. FT.)
			UPPER	MIDDLE	LOWER					
1	45°	BOTTOM	22.2	21.8	21.8	18.4	26.3	1.07	10	129.7
2	45°	MIDDLE	21.6	22.2	22.2	17.4	26.8	—	—	—
3	45°	TOP	21.3	21.2	21.2	17.5	25.4	1.19	9	132.8
4	30°	BOTTOM	21.5	21.6	22.4	—	—	1.05	9	130.4
5	30°	MIDDLE	21.6	21.7	21.6	—	—	1.14	11	130.5
6	30°	TOP	20.7	21.7	20.1	—	—	1.18	8	132.0
7	60°	BOTTOM	22.6	22.0	22.9	—	—	1.44	4	130.9
8	60°	MIDDLE	21.2	21.7	20.7	—	—	1.33	4	131.2
9	60°	TOP	21.3	22.2	22.0	—	—	1.32	4½	130.2
10	0°	BOTTOM	21.7	22.0	22.1	—	—	1.12	12	131.1
11	0°	MIDDLE	20.6	22.2	21.3	—	—	0.99	12	130.0
12	0°	TOP	16.8	21.5	21.0	—	—	1.06	13	131.3
13	90°	BOTTOM	20.6	21.1	21.6	—	—	1.66	3	130.0
14	90°	TOP	19.0	21.4	20.3	—	—	1.52	3½	132.6
15	90°	MIDDLE	16.3	21.9	17.2	—	—	1.75	3	131.2

NOTE: SAMPLE DIMENSIONS, DIAMETER 2.0 INCHES  
HEIGHT 4.0 "





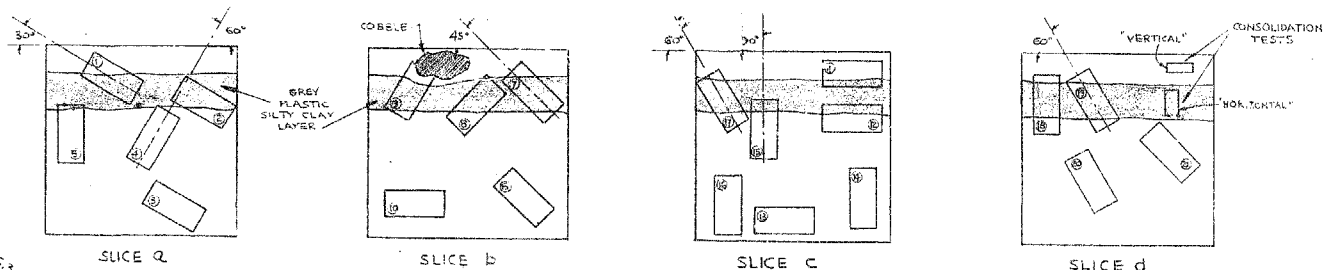
BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLES

SUMMARY OF UNDRAINED SHEAR STRENGTH RESULTS  
BLOCK E - ELEVATION 536.3 TO 535.3

FIGURE 30



SLICES OF BLOCK SAMPLE, SHOWING POSITIONS OF SAMPLES TESTED

BLOCK E SAMPLE #	ANGLE OF SAMPLE TO HORIZONTAL PLANE	POSITION IN BLOCK	UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ. FT.)	FAILURE STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)	TYPE OF TEST	REMARKS
1	30°	TOP	0.68	4.0	126.2	U	GREY SILTY CLAY LAYER
2	30°	TOP	0.57	4.0	123.0	U	" " " "
3	30°	BOTTOM	1.05	11.0	130.9	U	UNIFORM REDDISH BROWN SILTY CLAY
4	60°	MIDDLE	1.18	2.5	128.0	U	" " " "
5	90°	MIDDLE	1.33	3.5	128.4	U	" " " "
6	45°	BOTTOM	1.16	8.0	129.8	U	" " " "
7	45°	TOP	0.51	1.0	127.7	U	GREY SILTY CLAY LAYER
8	15°	TOP	0.71	2.0	125.9	U	" " " "
9	60°	TOP	—	—	—	—	LOST, COBBLE AT BASE
10	0°	BOTTOM	0.96	15.0	131.5	U	UNIFORM REDDISH BROWN SILTY CLAY
11	0°	TOP	0.98	14.0	132.1	U	GREY SILTY CLAY LAYER
12	0°	MIDDLE	1.18	11.0	123.8	U	" " " "
13	0°	BOTTOM	0.97	15.0	131.2	U	UNIFORM REDDISH BROWN SILTY CLAY
14	90°	BOTTOM	1.58	3.5	130.9	U	" " " "
15	90°	MIDDLE	1.34	3.5	126.1	U	GREY SILTY CLAY LAYER
16	90°	BOTTOM	1.40	4.5	131.3	U	UNIFORM REDDISH BROWN SILTY CLAY
17	60°	TOP	0.80	4.5	127.5	U	GREY SILTY CLAY LAYER
18	90°	TOP	1.13	8.0	127.5	U	" " " "
19	60°	TOP	0.71	4.0	125.9	U	" " " "
20	60°	BOTTOM	1.21	6.0	132.5	U	UNIFORM REDDISH BROWN SILTY CLAY
21	45°	MIDDLE	1.12	7.0	132.0	U	" " " "

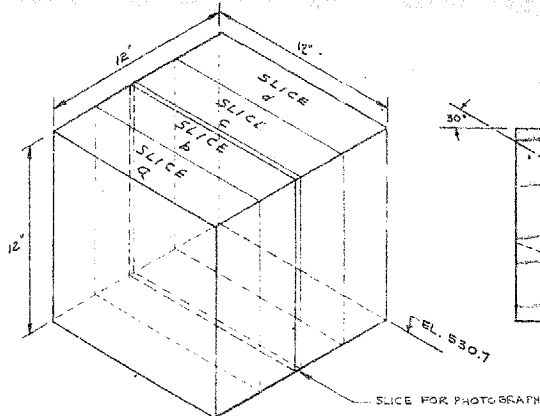
NOTE: SAMPLE DIMENSIONS, DIAMETER 2.0 INCHES  
HEIGHT 4.0 "

NOTE: FOR FURTHER DETAILED TEST RESULTS  
SEE FIGURE 37.

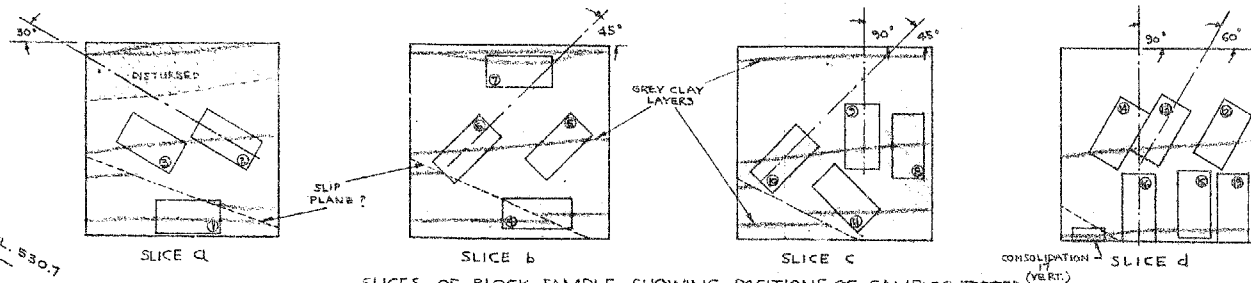
GOLDER & ASSOCIATES

Made 10/24/20  
Chkd 2/20/20  
Appd 1/1/20

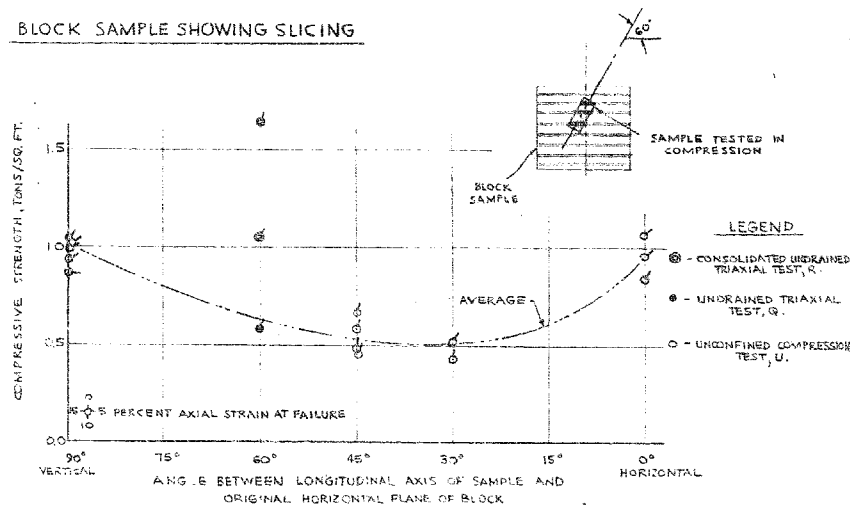




BLOCK SAMPLE SHOWING SLICING



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED



VARIATION IN COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLES

SUMMARY OF UNDRAINED SHEAR STRENGTH RESULTS  
BLOCK F - ELEVATION 531.7 TO 530.7

FIGURE 31

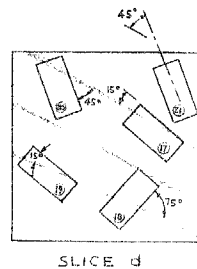
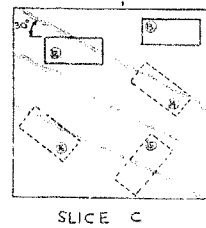
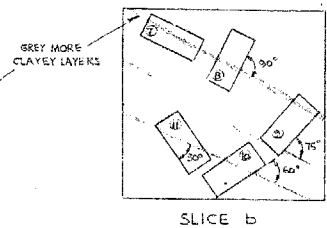
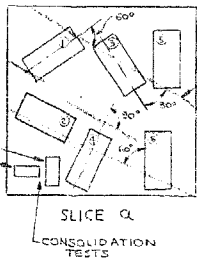
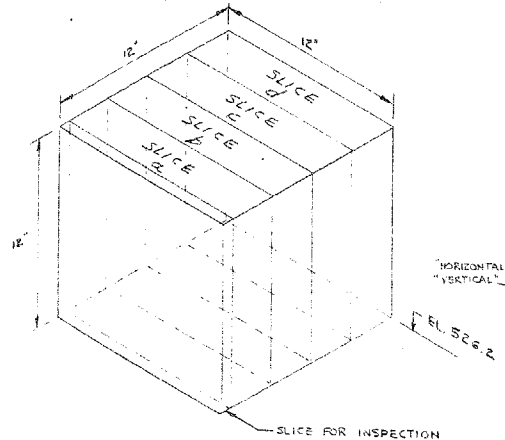
BLOCK F SAMPLE #	ANGLE OF SAMPLE TO HORIZONTAL PLANE	POSITION IN BLOCK	COMPRESSIVE STRENGTH (TONS/SQ. FT.)	FAILURE STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)	TYPE OF TEST	REMARKS
1	0°	BOTTOM	0.97	3	118.3	U.	GREY CLAY LAYER - NO SLIP PLANE
2	30°	MIDDLE	0.42	1/2	121.4	U.	" " " " " "
3	80°	MIDDLE	0.51	2	122.3	U.	" " " " " "
4	0°	BOTTOM	0.83	2	118.5	U.	GREY CLAY LAYER & SLIP PLANE
5	45°	MIDDLE	0.48	1/2	120.8	U.	GREY CLAY LAYER - NO SLIP PLANE
6	45°	MIDDLE	0.47	1	122.2	U.	" " " " " "
7	0°	TOP	1.06	3	122.1	U.	ALMOST BETWEEN GREY CLAY LAYERS
8	90°	MIDDLE	0.93	3	122.1	U.	GREY CLAY LAYER - NO SLIP PLANE
9	90°	MIDDLE	1.03	3	123.5	U.	" " " " " "
10	45°	MIDDLE	0.65	1 1/2	122.8	U.	" " " " " "
11	45°	BOTTOM	0.57	1 1/2	120.8	U.	" " " " " "
12	60°	MIDDLE	1.66	1 1/2	123.2	R.	LATERAL PRESSURE 2.8 LB./SQ. IN.
13	60°	MIDDLE	0.57	1 1/2	124.1	Q.	" " " " 2.8 LB./SQ. IN.
14	60°	MIDDLE	1.05	8	124.1	R.	" " " " 5 LB./SQ. IN.
15	30°	BOTTOM	1.05	2 1/2	121.3	U.	GREY CLAY LAYER - NO SLIP PLANE
16	30°	BOTTOM	1.04	3	120.8	U.	" " " " " "
19	30°	BOTTOM	0.87	6	120.0	U.	" " " " " "

NOTE: SAMPLE DIMENSIONS, DIAMETER 2.0 INCHES  
HEIGHT 4.0 "

NOTE: FOR FURTHER DETAILED TEST RESULTS SEE FIGURE 38.

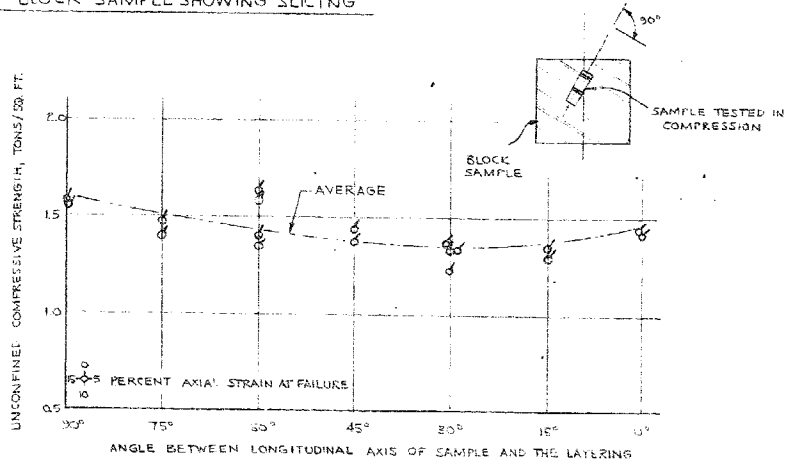
GOLDER & ASSOCIATES

MADE IN CANADA



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING



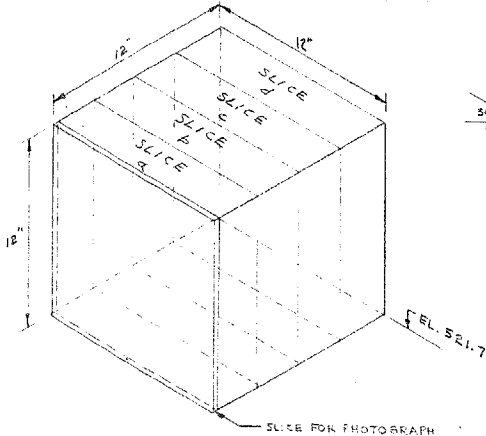
VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLES

TABLE OF RESULTS						
BLOCK G SAMPLE #	POSITION IN BLOCK	ANGLE OF SAMPLE TO LAYERING	UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ. FT.)	FAILURE STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)	TYPE OF TEST
1	TOP	60°	1.55	2.5	118.8	U.
2	MIDDLE	0°	1.42	2.4	118.9	U.
3	TOP	30°	1.54	0.8	119.0	U.
4	BOTTOM	30°	1.57	1.3	118.4	U.
5	TOP	60°	1.58	1.3	118.0	U.
6	BOTTOM	60°	1.63	1.4	119.3	U.
7	TOP	0°	1.43	2.4	118.2	U.
8	TOP	30°	1.55	1.4	118.2	U.
9	MIDDLE	75°	1.48	1.2	119.0	U.
10	BOTTOM	60°	1.39	1.4	118.5	U.
11	MIDDLE	30°	1.23	1.8	117.9	U.
12	TOP	30°	1.35	2.3	118.6	U.
13	TOP	30°	1.35	2.2	118.0	U.
14	MIDDLE	-	-	-	-	-
15	BOTTOM	-	-	-	-	-
16	MIDDLE	-	-	-	-	-
17	MIDDLE	15°	1.35	1.4	118.3	U.
18	BOTTOM	75°	1.40	1.4	119.2	U.
19	BOTTOM	15°	1.40	1.5	119.0	U.
20	TOP	45°	1.37	2.0	117.4	U.
21	TOP	45°	1.42	1.4	117.3	U.

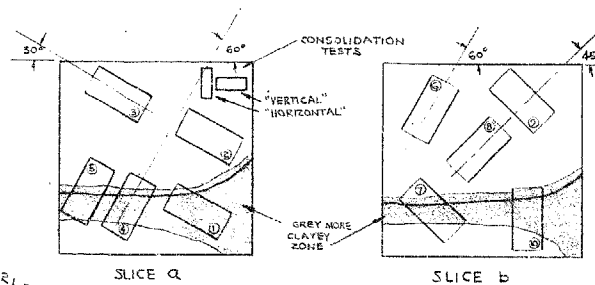
SAMPLE CRIED OUT PRIOR TO TEST.

NOTE: SAMPLE DIMENSIONS: DIAMETER 2.0 INCHES  
HEIGHT 4.0

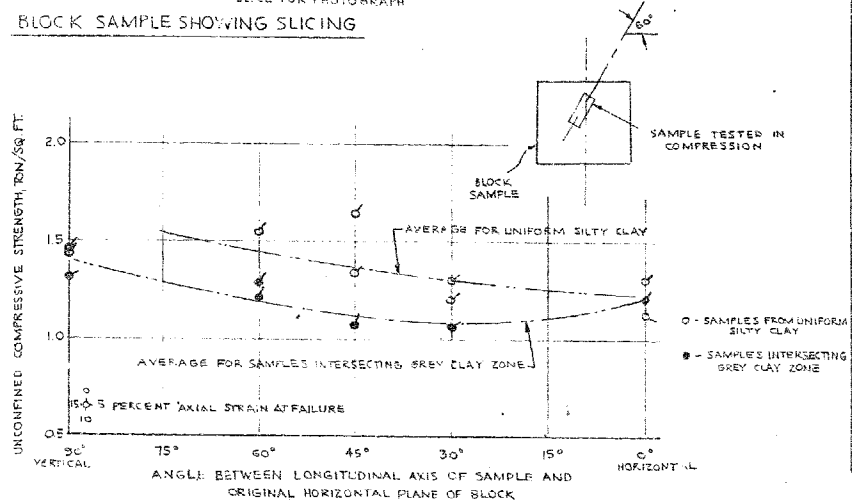
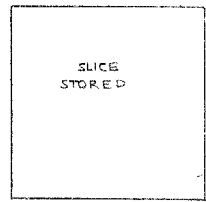
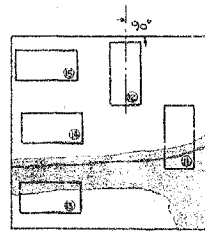
NOTE: FOR FURTHER DETAILED TEST RESULTS SEE FIGURE 33.



BLOCK SAMPLE SHOWING SLICING



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

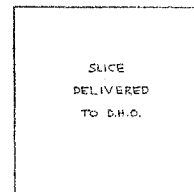
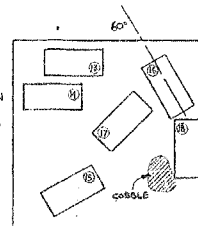
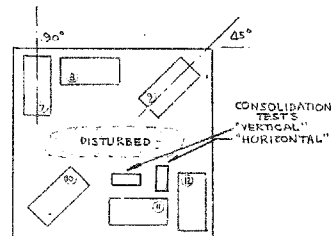
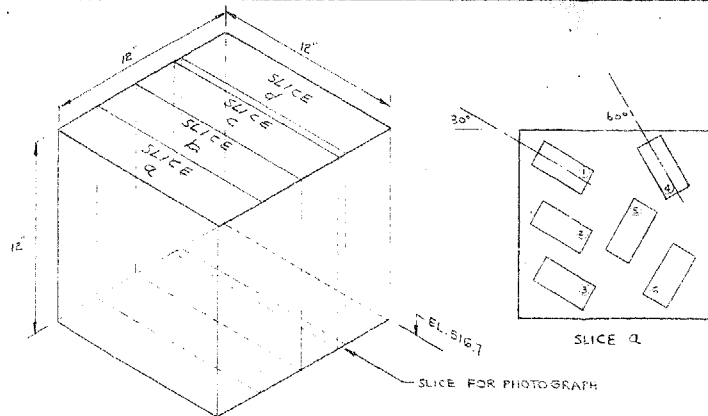


VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLES

BLOCK H SAMPLE #	ANGLE OF SAMPLE TO HORIZONTAL PLANE	POSITION IN BLOCK	UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ. FT.)	FAILURE STRAIN (%)	UNIT WEIGHT (LB/CU. FT.)	TYPE OF TEST	REMARKS
1	30°	BOTTOM	1.07	3.5	117.1	U.	INTERSECTING GREY CLAY ZONE
2	30°	MIDDLE	1.30	3.5	121.0	U.	UNIFORM SILTY CLAY
3	30°	TOP	1.20	3.0	120.2	U.	"
4	80°	BOTTOM	1.21	0.9	118.2	U.	INTERSECTING GREY CLAY ZONE
5	60°	MIDDLE	1.29	1.3	120.2	U.	"
6	60°	TOP	1.55	1.3	120.3	U.	UNIFORM SILTY CLAY
7	45°	BOTTOM	1.08	1.1	118.6	U.	INTERSECTING GREY CLAY ZONE
8	45°	MIDDLE	1.64	1.8	120.2	U.	UNIFORM SILTY CLAY
9	45°	TOP	1.34	2.5	121.5	U.	"
10	90°	BOTTOM	1.31	3.3	117.0	U.	INTERSECTING GREY CLAY ZONE
11	90°	MIDDLE	1.46	2.3	119.7	U.	"
12	90°	TOP	1.45	1.3	121.0	U.	UNIFORM SILTY CLAY
13	0°	BOTTOM	1.31	2.5	119.0	U.	INTERSECTING GREY CLAY ZONE
14	0°	MIDDLE	1.22	2.0	121.0	U.	UNIFORM SILTY CLAY
15	0°	TOP	1.13	7.0	120.3	U.	"

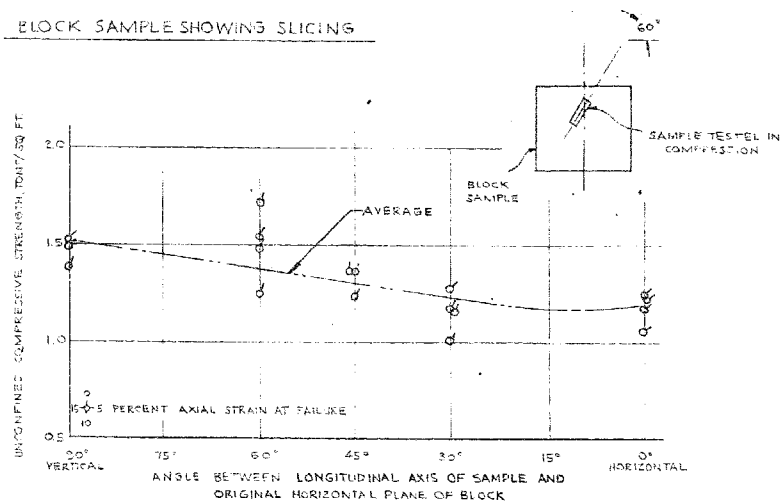
NOTE: SAMPLE DIMENSIONS, DIAMETER 2.0 INCHES  
HEIGHT 4.0 "

NOTE: FOR FURTHER DETAILED TEST RESULTS  
SEE FIGURE 40.



SLICES OF FLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

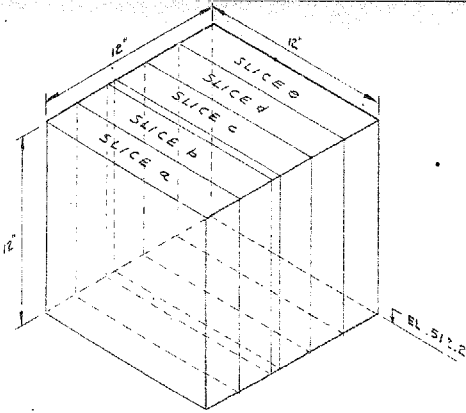
BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLES

BLOCK I SAMPLE #	ANGLE OF SAMPLING TO HORIZONTAL PLANE	POSITION IN BLOCK	UNCONFINED COMPRESSIVE STRENGTH (TONS/30 FT)	FAILURE STRAIN (%)	WATER CONTENT (%)	UNIT WEIGHT (LB/CU. FT.)	TYPE OF TEST	REMARKS
1	50°	TOP	1.18	1.6	31.2, 30.1, 31.5	121.6	U.	LL = 40.0, PL = 21.3, PI = 18.7
2	30°	MIDDLE	1.17	1.0	30.4, 31.2, 31.5	121.3	U.	LL = 39.1, PL = 20.0, PI = 18.2
3	30°	BOTTOM	1.02	1.3	30.7, 31.5, 31.4, 32.7	121.8	U.	LL = 39.0, PL = 20.8, PI = 18.2
4	60°	TOP	1.70	1.4	30.6, 33.4, 31.4	121.2	U.	
5	60°	MIDDLE	1.48	1.1	30.7, 32.2, 31.2	121.5	U.	
6	60°	BOTTOM	1.26	1.1	31.1, 30.6, 31.5	121.8	U.	
7	30°	TOP	1.51	2.4	33.3, 32.5, 31.5	121.3	U.	
8	0°	TOP	1.07	3.5	31.7, 29.8, 31.7	121.5	U.	
9	45°	TOP	1.37	1.4	32.6, 31.5, 32.5	121.3	U.	
10	45°	BOTTOM	1.27	1.6	31.2, 31.3, 32.4	121.2	U.	
11	0°	BOTTOM	1.23	2.6	31.5, 31.1, 30.5	122.0	U.	
12	30°	BOTTOM	1.50	2.1	31.0, 30.6, 31.5	121.4	U.	
13	0°	TOP	1.15	2.3	30.6, 30.6, 31.6	121.5	U.	
14	0°	MIDDLE	1.23	2.3	32.4, 32.0, 32.1	121.0	U.	
15	30°	BOTTOM	1.23	2.3	32.5, 32.5, 31.0	121.5	U.	
16	60°	TOP	1.54	1.5	32.6, 34.6, 31.4	122.3	U.	
17	45°	MIDDLE	1.37	1.0	30.8, 31.4, 30.5	122.1	U.	
18	30°	MIDDLE	1.37	1.4	32.6, 31.6, 32.4	120.8	U.	SLIGHTLY DISTURBED

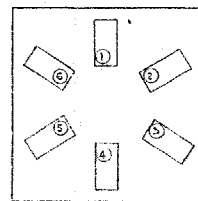
NOTE: SAMPLE DIMENSIONS, DIAMETER 2.0 INCHES  
HEIGHT 4.0 "



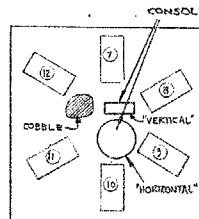
NOTE: FOR FURTHER DETAILED TEST RESULTS  
SEE FIGURE 41.

SUMMARY OF UNDRAINED SHEAR STRENGTH  
RESULTS  
BLOCK J - ELEVATION 513.2 TO 512.2

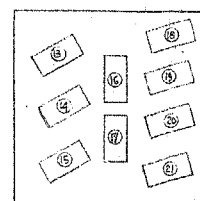
FIGURE 35



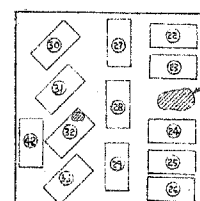
SLICE a



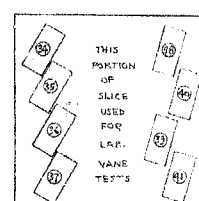
SLICE b



SLICE c



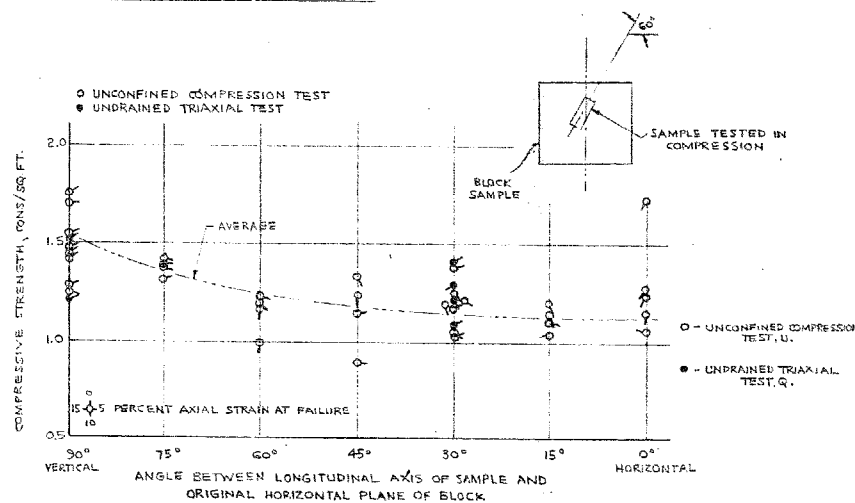
SLICE d



SLICE e

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING



VARIATION IN COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLES

BLOCK J SAMPLE #	ANGLE OF SA TO HORIZ. PLANE	MOISTURE CONTENTS	COMPRESSIVE STRENGTH (PONS/SQ. FT.)	FAILURE STRAIN (%)	TYPE OF TEST	BLOCK J SAMPLE #	ANGLE OF SA TO HORIZ. LAYERING	MOISTURE CONTENTS	COMPRESSIVE STRENGTH (PONS/SQ. FT.)	FAILURE STRAIN (%)	TYPE OF TEST
1	90°	25.5, 28.4	1.21	4.0	Q.	22	0°	28.8, 27.6, 27.3, 26.0	1.08	17.3	U.
2	30°	35.2, 28.5	1.22	8.0	Q.	23	0°	27.5, 27.1	1.25	12.0	U.
3	30°	31.3, 23.0	1.30	13.4	Q.	24	0°	26.6, 26.5	1.24	12.0	U.
4	90°	28.2, 30.6	1.53	3.4	Q.	25	0°	26.0, 25.5	1.21	12.0	U.
5	30°	26.1, 30.3	1.42	3.3	Q.	26	0°	27.2, 27.3	1.16	10.7	U.
6	30°	30.2, 28.0	1.10	3.3	Q.	27	90°	27.7, 28.0	1.25	6.7	U.
7	90°	27.9, 27.0	1.30	4.6	U.	28	90°	29.3, 28.1	1.43	3.3	U.
8	30°	30.3, 30.5	1.19	4.6	U.	29	90°	23.0, 25.7	1.44	2.7	U.
9	30°	26.6, 26.2	1.38	4.6	U.	30	45°	27.5, 26.5, 30.1	0.89	5.3	U.
10	30°	35.4, 26.4	1.72	5.0	U.	31	45°	26.6, 26.6, 23.6	1.34	8.0	U.
11	30°	26.9, 27.7	1.20	7.9	U.	32	45°	26.5, 27.1	1.24	10.0	U.
12	30°	30.0, 28.0	1.12	6.7	U.	33	45°	29.4, 44.0, 15.3	1.16	4.7	U.
13	30°	27.3, 27.5	1.04	3.7	U.	34	60°	27.5, 26.1, 32.8	1.01	10.7	U.
14	30°	28.7, 31.8	1.24	8.2	U.	35	60°	28.2, 28.7, 25.5	1.17	10.3	U.
15	30°	27.7, 28.2	1.22	6.7	U.	36	60°	27.8, 26.1, 25.8	1.13	7.3	U.
16	90°	27.9, 27.8	1.55	4.7	U.	37	60°	27.5, 27.8, 26.4	1.21	6.0	U.
17	90°	26.5, 28.0	1.78	3.3	U.	38	75°	28.5, 28.3, 28.4	1.40	5.0	U.
18	15°	28.0, 28.8	1.04	14.7	U.	39	75°	30.1, 28.0, 27.7	1.39	5.3	U.
19	15°	30.2, 27.8	1.12	6.7	U.	40	75°	28.3, 28.9, 28.7	1.33	3.7	U.
20	15°	26.5, 26.8	1.15	9.3	U.	41	75°	25.6, 26.7, 29.8	1.42	6.7	U.
21	15°	26.9, 28.1	1.20	9.2	U.	42	90°	27.8, 28.5, 26.4	1.46	4.0	U.

NOTE: SAMPLE DIMENSIONS, DIAMETER 1.5 INCHES  
HEIGHT 3.0 INCHES

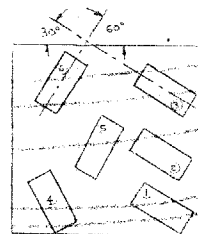
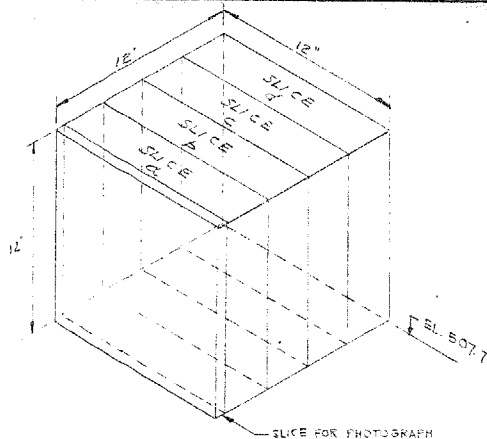
\* EXCEPT No. 18 - HEIGHT 2.2 INCHES  
No. 24 - HEIGHT 2.0 INCHES

GOLDER & ASSOCIATES

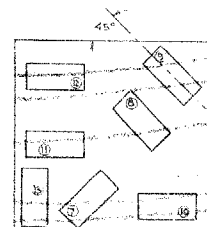
Made  
Chd.  
Appd.

SUMMARY OF UNDRAINED SHEAR STRENGTH  
RESULTS  
BLOCK K - ELEVATION 508.7 TO 507.7

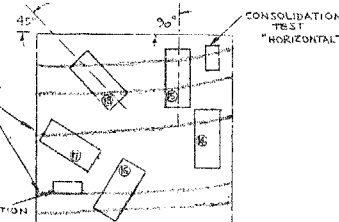
FIGURE 36



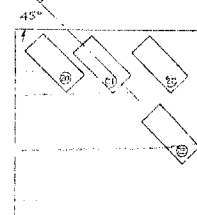
SLICE A



SLICE B



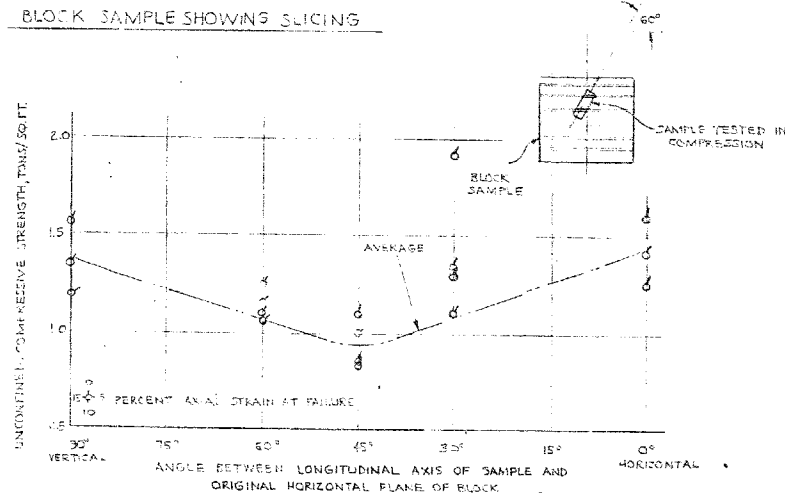
SLICE C



SLICE D

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLES

TABLE OF RESULTS

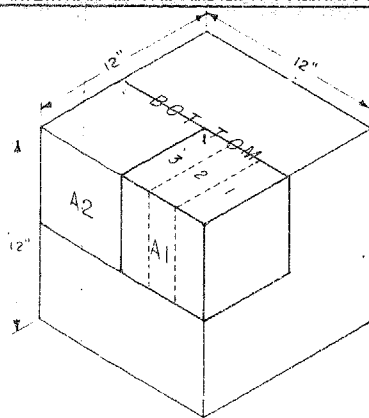
BLOCK K SAMPLE #	ANGLE OF SAMPLE TO HORIZONTAL PLANE	POSITION IN BLOCK	UNCONFINED COMPRESSIVE STRENGTH (TNS/SQ. FT.)	FAILURE STRAIN (%)	UNIT WEIGHT (LB/CU. FT.)	TYPE OF TEST	REMARKS
1	30°	BOTTOM	1.33	1.1	111.8	U.	
2	30°	MIDDLE	1.91	0.8	136.2	U.	
3	30°	TOP	1.10	1.5	113.0	U.	
4	60°	BOTTOM	1.06	2.8	111.2	U.	
5	60°	MIDDLE	1.30	1.1	113.3	U.	
6	60°	TOP	1.20	2.0	113.3	U.	
7	45°	BOTTOM	1.02	2.3	111.5	U.	
8	45°	MIDDLE	1.10	0.6	111.8	U.	
9	45°	TOP	0.66	0.9	113.7	U.	
10	0°	BOTTOM	1.26	1.3	111.2	U.	
11	0°	MIDDLE	1.61	0.6	113.8	U.	
12	0°	TOP	1.42	1.3	114.2	U.	
13	30°	BOTTOM	1.19	4.3	111.7	U.	
14	30°	MIDDLE	1.56	1.3	113.0	U.	
15	30°	TOP	1.33	1.8	114.6	U.	
16	60°	BOTTOM	1.11	1.8	112.3	U.	
17	30°	MIDDLE	1.30	0.5	111.7	U.	
18	45°	TOP	0.86	0.6	114.0	U.	
19	REMOULDED	—	0.15	20.0	113.8	U.	WATER CONTENT OF REMOULDED SAMPLE 41.4 %
20	45°	TOP	—	—	116.5	S.	DRAINED TRIAXIAL TEST.
21	45°	TOP	—	—	116.2	S.	
22	45°	TOP	—	—	116.2	S.	
23	45°	MIDDLE	—	—	112.0	S.	

NOTE: SAMPLE DIMENSIONS, DIAMETER 2.0 INCHES  
HEIGHT 4.0 "

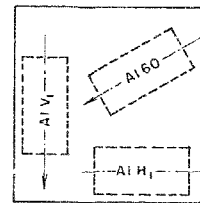
NOTE: FOR FURTHER DETAILED TEST RESULTS  
SEE FIGURE 42.

GOLDER & ASSOCIATES

Mod. Chkd. Appd.

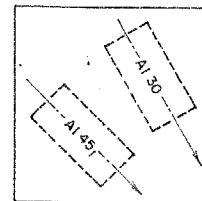


BOTTOM



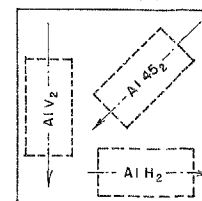
1

BOTTOM



2

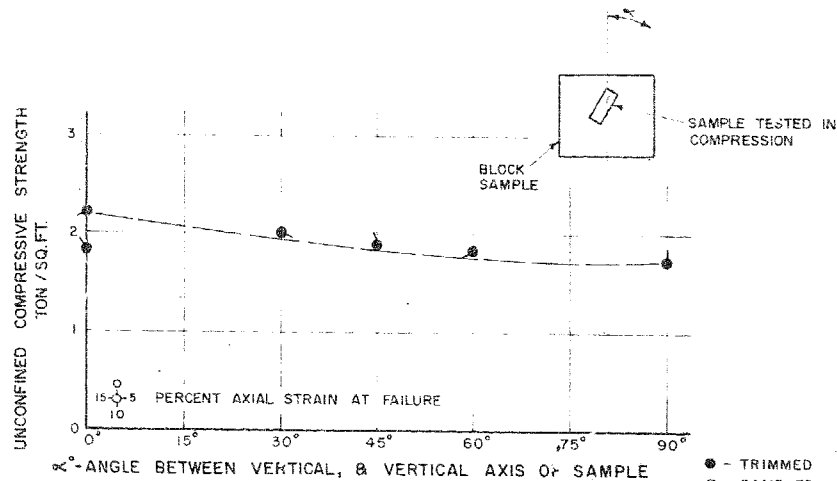
BOTTOM



3

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

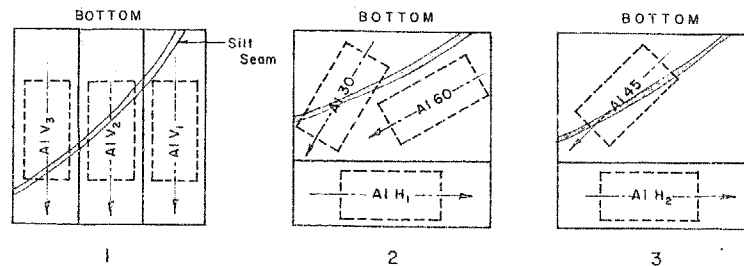
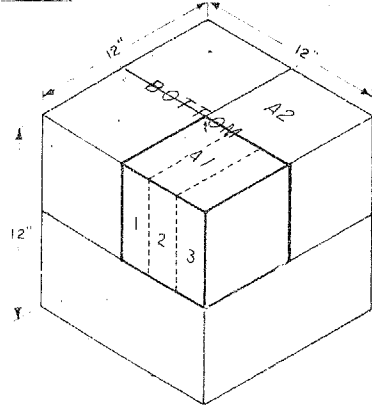
BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

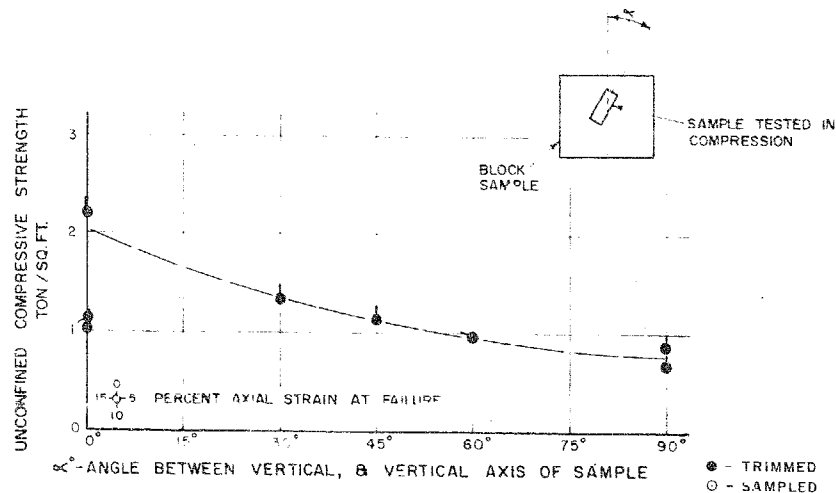
TABLE OF RESULTS

BLOCK SAMPLE #A	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT.)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
	A1 V1	19.5	17.4	28.5	1.83	19	131
	A1 H1	18.7	16.6	29.9	1.71	20	132
	A1 30°	19.6	18.7	29.8	2.00	7	133
	A1 45°	16.8	17.2	28.7	1.89	19	132
	A1 45°-2	19.7	18.3	32.6	4.44	6	133
	A1 60°	21.0	16.4	29.7	1.82	13	132
	A1 H2	19.0	17.8	32.5	3.74	5	133
	A1 V2	19.4	16.6	29.4	2.18	13	133



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING

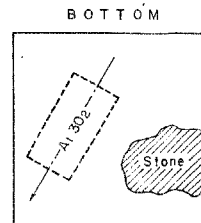
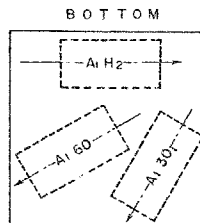
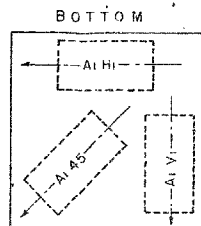
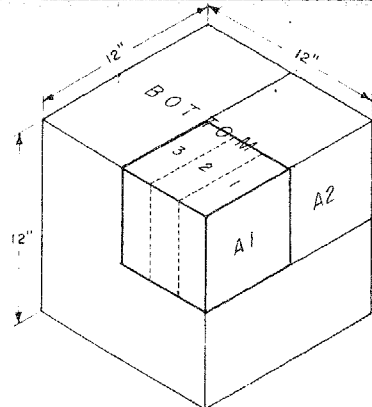


VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

TABLE OF RESULTS

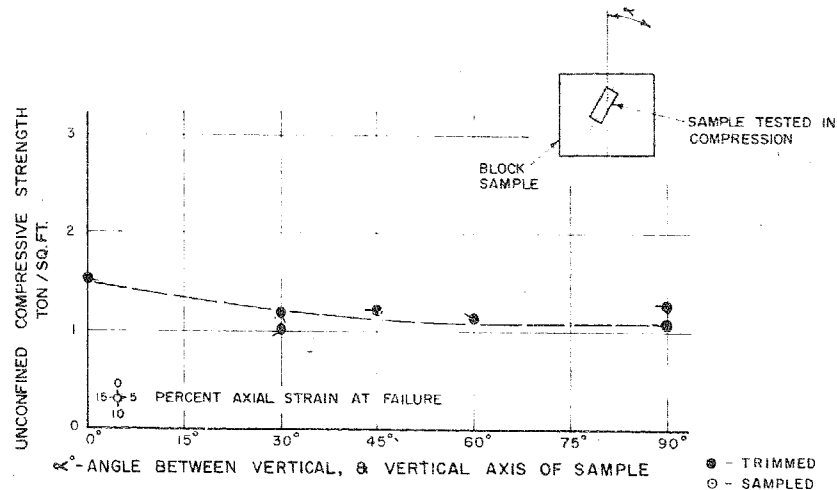
BLOCK SAMPLE # B	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT.)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
	A1 V1	21.5	17.2	28.5	2.20	20	132
	A1 V2	20.8	16.5	27.4	1.04	20	129
	A1 V3	21.0	16.3	27.8	1.10	12.5	132
	A1-30°	19.9			1.33	20	130
	A1-45°	21.2			1.12	20	128
	A1-60°	21.2	16.7	27.6	.98	17	129
	A1 H1	21.3			.69	20	128
	A1 H2	19.8	17.7	28.5	.90	20	128





SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING



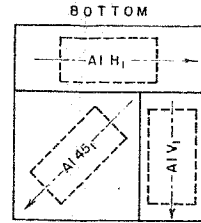
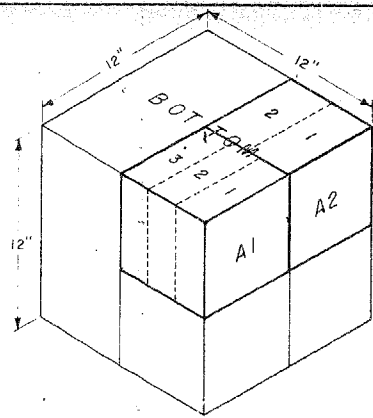
VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

SUMMARY OF LABORATORY RESULTS  
BLOCK SAMPLE 'C'  
FROM ELEVATION 544.9 TO 543.9

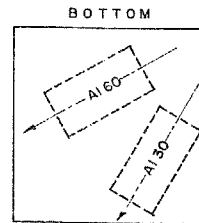
FIGURE 3

TABLE OF RESULTS

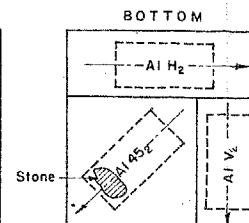
BLOCK SAMPLE # C	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
	A1 V1	21.3	17.5	28.5	1.51	6	130.0
	A1-30°	22.0	16.0	27.2			
	A1-30°-2	20.1	15.2	28.0	1.05	13	130.0
	A1-45°	19.6	17.0	28.6	1.20	8	130.6
	A1-60°	21.8	15.3	28.3	1.22	15	131.2
	A1 H1	19.5	17.4	28.0	1.13	17	129.2
	A1 H2	20.5	16.8	27.2	1.29	15	130.0
					1.09	15.5	127.9



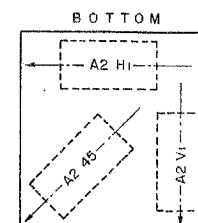
1



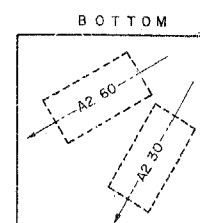
2



3



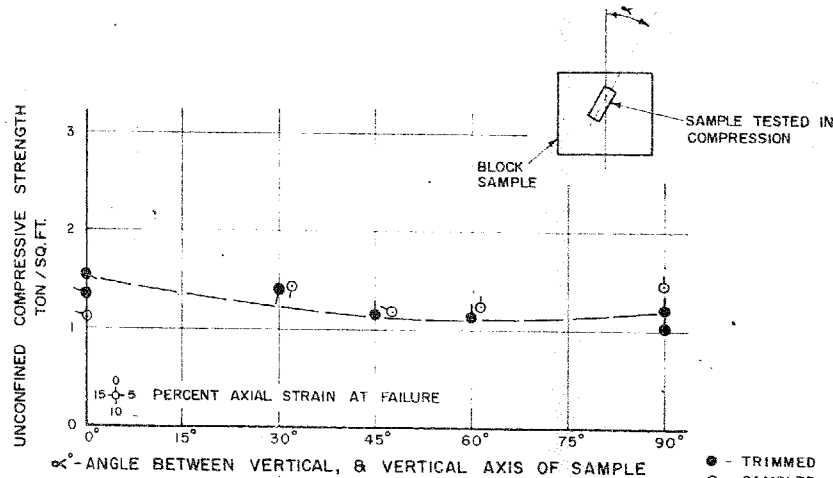
1



2

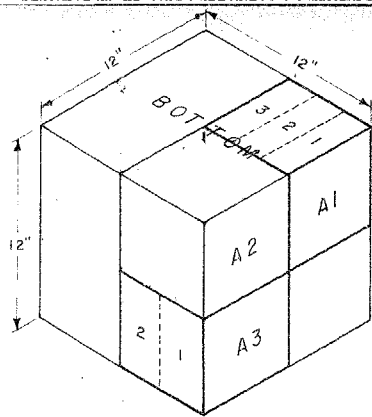
SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING

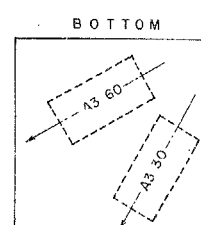
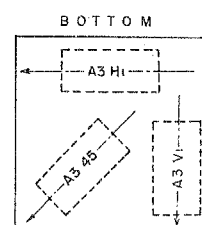
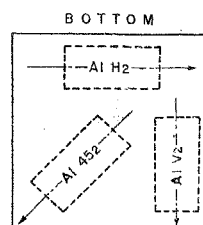
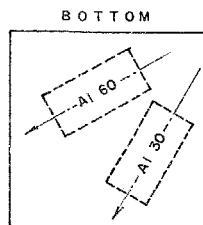
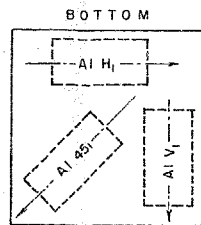


VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

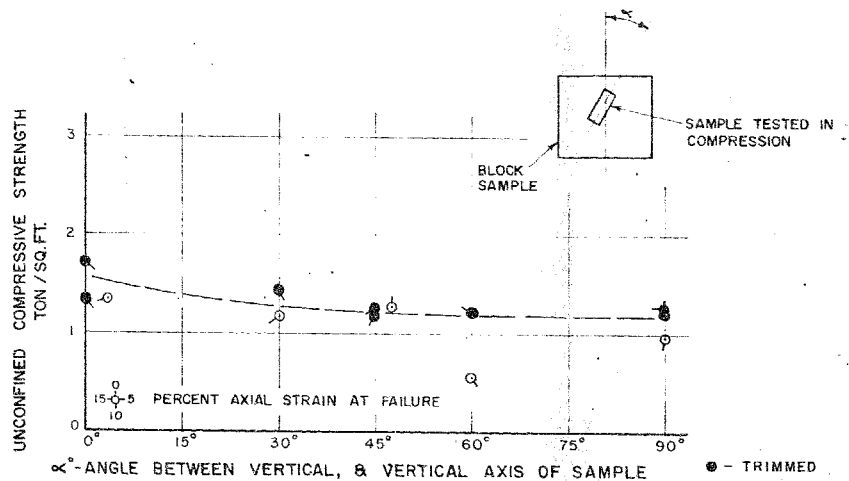
TABLE OF RESULTS							
BLOCK SAMPLE # D	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT.)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
	A1 V1	20.8			1.39	16	128
	A1 V2	21.0			1.54	7	128
	A1 30°	21.2			1.42	11	130
	A1-45°-1	20.5	16.7	27.4	1.16	20	130
	A1-45°-2						
	A1-60°	19.4			1.15	20	130
	A1 H1	19.9			1.01	20	132
	A1 H2	19.5			1.22	20	130
*	A2 V1	21.0			1.14	16	132
*	A2 30	20.0			1.44	11	131
*	A2 45	21.6			1.19	17	131
*	A2 60	18.6			1.25	20	132
*	A2 H1	18.8			1.46	20	132



BLOCK SAMPLE SHOWING SLICING



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED



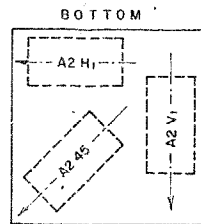
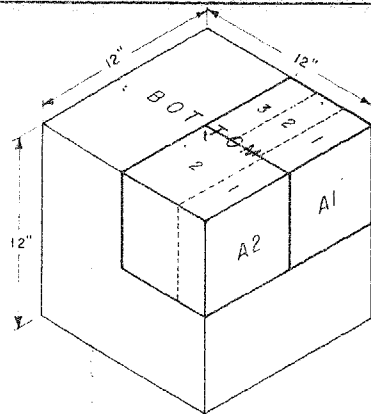
VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

SUMMARY OF LABORATORY RESULTS  
BLOCK SAMPLE 'E'  
FROM ELEVATION 536.3 TO 535.3

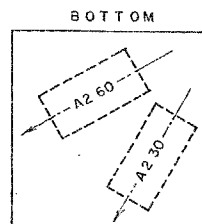
FIGURE 5

TABLE OF RESULTS

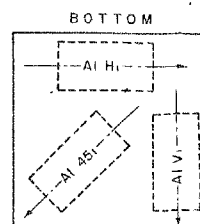
BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
E	A1 V1	21.3	15.8	28.0	1.37	8	133
	A1 V2	21.0			1.74	7	132
	A1 30°	21.1			1.46	8	131
	A1 45°	22.7	17.0	28.5	1.20	12	132
	A1 45°-2	22.2			1.28	13	131
	A1 60°	19.8	17.8		1.24	17	132
	A1 H1	20.2	16.3	27.2	1.22	20	133
	A1 H2	20.7			1.26	15	133
	A3 V1	11.1	15.0	33.1	1.36	14	127
	A3 30°	14.6	15.9		1.16	13	125
	A3 45	9.9	19.2	32.9	1.29	20	131
	A3 60	17.2	18.3	44.0	.57	8	121
	A3 H1	15.0	18.9	40.5	.98	11	119



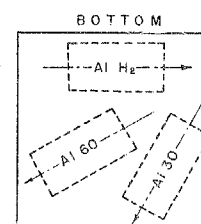
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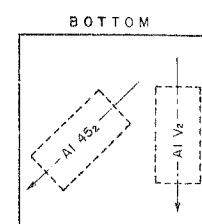
2



1



2



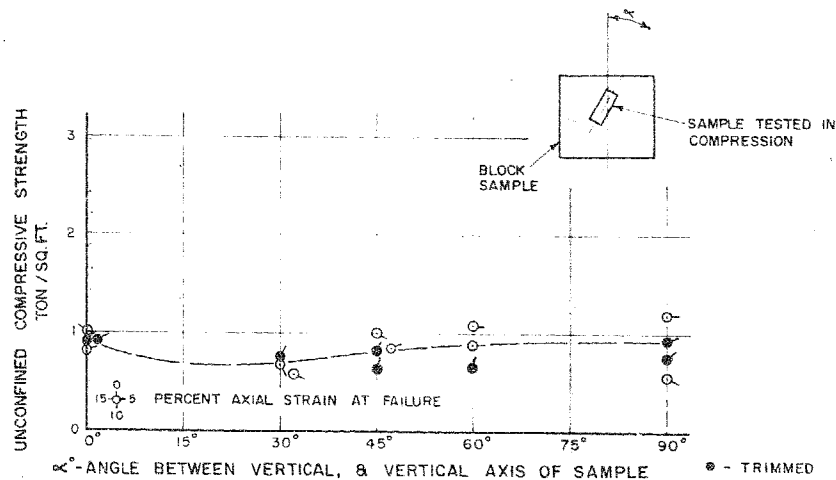
3

SUMMARY OF LABORATORY RESULTS  
BLOCK SAMPLE 'F'  
FROM ELEVATION 531.7 TO 535.7

FIGURE 6

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING

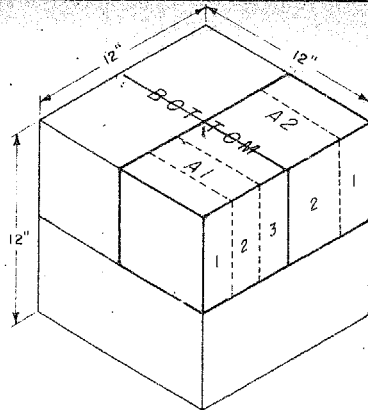


VARIAION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

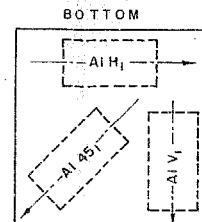
TABLE OF RESULTS

BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT)	STRAIN (%)	UNIT WEIGHT (LB./CU FT)
	A1 V1				.91	3	122
	A1 V2				.92	3	121
	A1 30°				.73	2	122
	A1 45°				.81	2	124
	A1-45°-2		20.2	45.5	.65	1.5	121
	A1 60°				.67	1	122
	A1 H1				.78	2.5	122
	A1 H2				.93	4	118

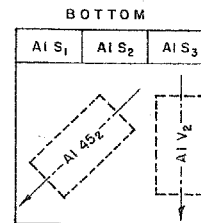




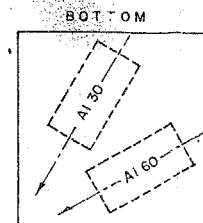
BLOCK SAMPLE SHOWING SLICING



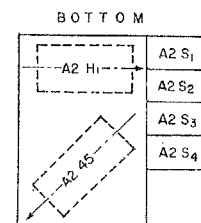
1



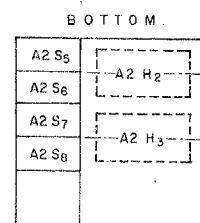
2



3

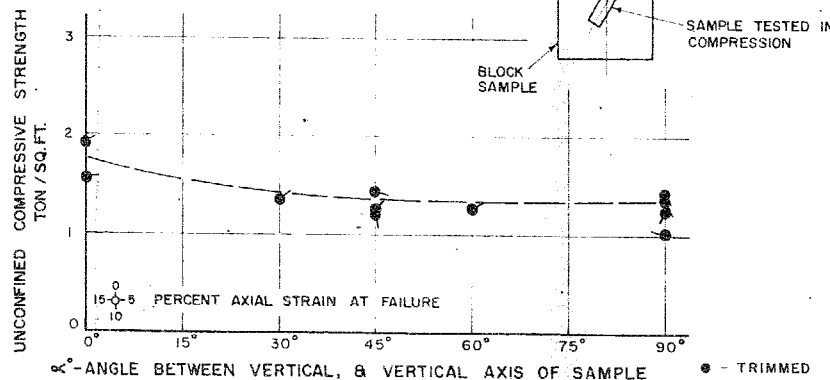


1



2

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED



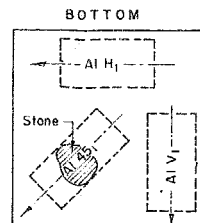
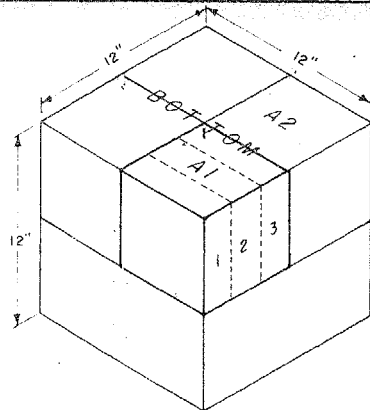
VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

SUMMARY OF LABORATORY RESULTS  
BLOCK SAMPLE 'G'  
FROM ELEVATION 527.2 TO 526.2

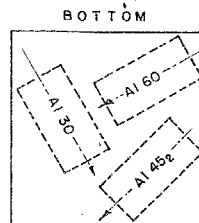
FIGURE 8

TABLE OF RESULTS

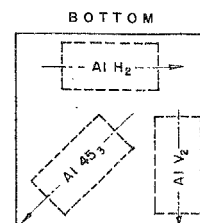
TABLE OF RESULTS							
BLOCK SAMPLE # G	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
	A1 V1	33.0	24.9	49.8	1.54	4	119
	A1 V2	33.4	23.5	48.2	1.92	3.5	120
	A1 30°		22.5	49.3	1.37	2.5	120
	A1 45°	34.1	23.1	48.4	1.27	9.0	120
	A1 45°-2		21.7	50.3	1.20	2	118
	A1 60°	32.4	23.9	47.9	1.27	3	119
	A1 H1	33.2	22.0	47.3	1.04	17	119
	A2 H1	31.9	18.9	44.9	1.26	12	121
	A2 H2	34.2	22.4	46.8	1.42	9	122
	A2 H3	34.2			1.39	9	121
	A2 45	33.5	21.7	47.4	1.43	5.5	121



1



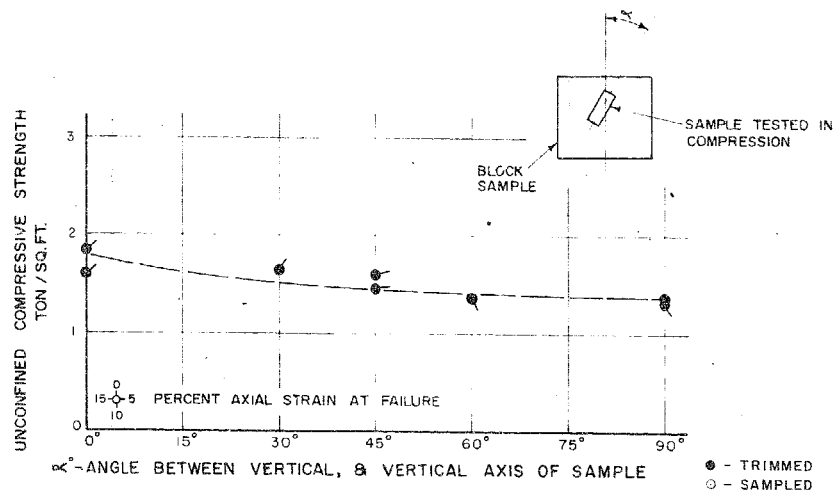
2



3

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

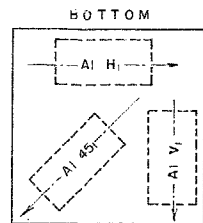
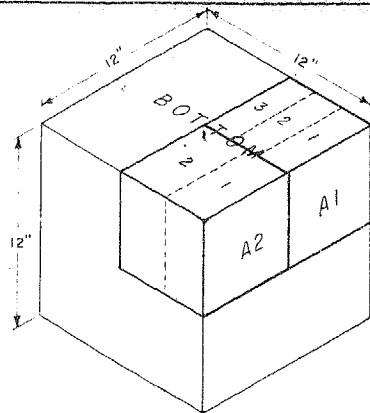
BLOCK SAMPLE SHOWING SLICING



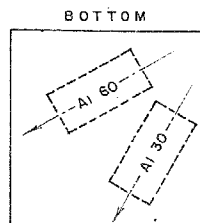
VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

TABLE OF RESULTS

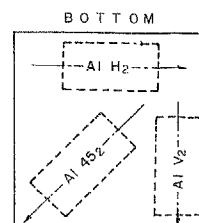
BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT.)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
H	A1 V1	33.9	20.8	44.3	1.83	2.5	120
	A1 V2	32.6	22.6	46.0	1.59	2.5	120
	A1 30	31.5	23.6	43.6	1.66	2	119
	A1 45		21.9	49.2	-	-	-
	A1 45-2	31.5	21.8	44.3	1.45	4	120
	A1 45-3	32.3			1.61	4	121
	A1 60	32.8	21.3	48.6	1.37	8	120
	A1 H1	31.1	19.0	43.3	1.38	15	121
	A1 H2	31.4			1.35	8	120



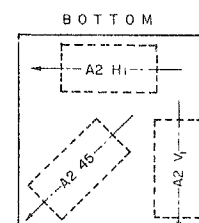
1



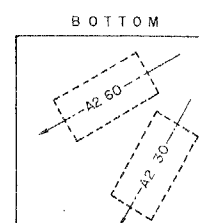
2



3



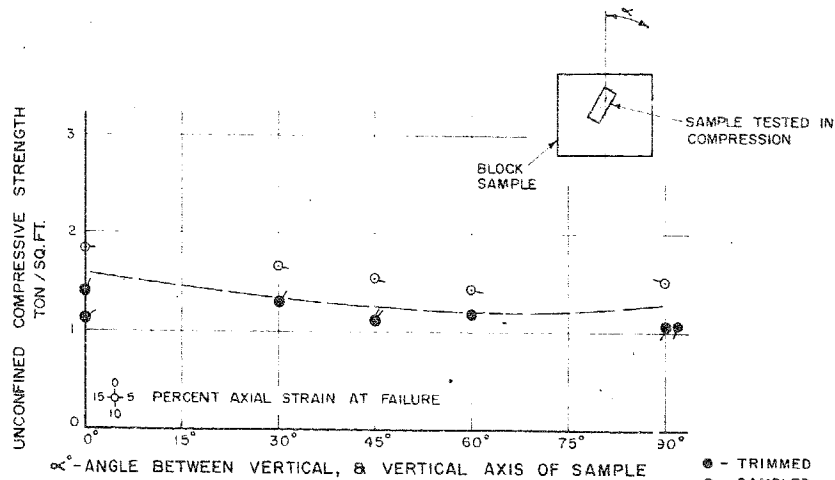
1



2

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

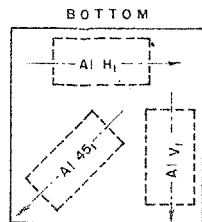
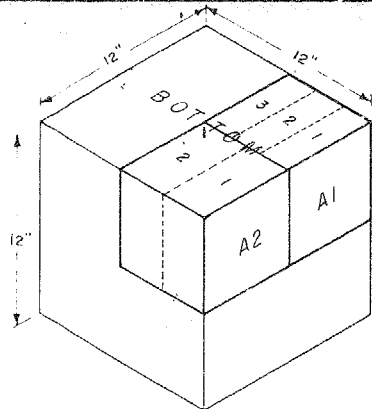
SUMMARY OF LABORATORY RESULTS  
BLOCK SAMPLE 'I'  
FROM ELEVATION 517.7 TO 516.7

FIGURE 10

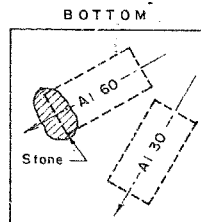
TABLE OF RESULTS

BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT.)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
I	A1 V1	31.7	21.6	44.3	1.13	2.5	123
	A1 V2	31.7			1.40	1.5	120
	A1 30°	31.9	19.9	43.0	1.30	2	121
	A1 45°	30.9			1.11	1.5	121
	A1-45-2	31.9			1.10	2	121
	A1 60	31.2			1.19	4	121
	A1 H1	30.7			1.07	11	122
	A1 H2	31.1			1.06	11	121
	A2 V1	30.9			1.84	5	123
	A2 30	30.7			1.67	6	123
	A2 45	30.1			1.56	7	123
	A2 60	31.4			1.44	7	124
	A2 H1	27.2			1.51	16	126
	*						

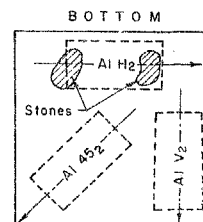




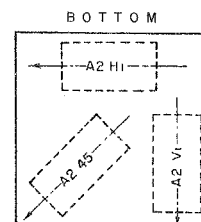
1



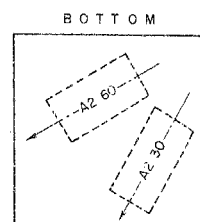
2



3



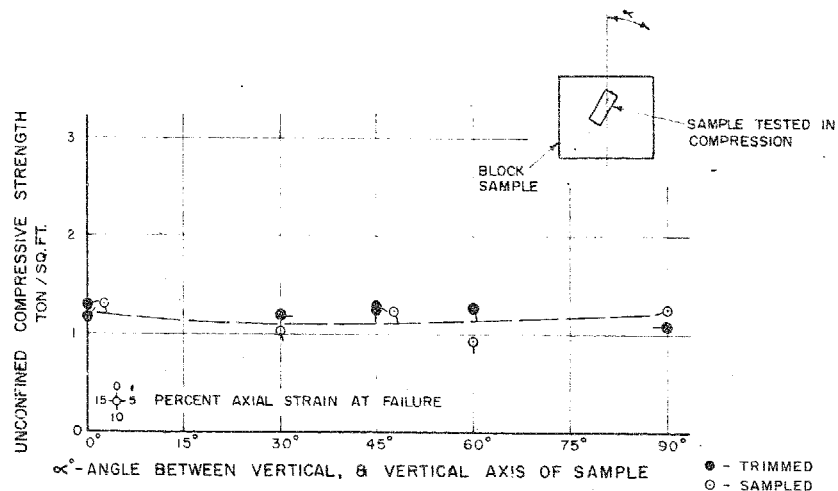
1



2

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

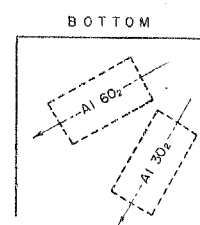
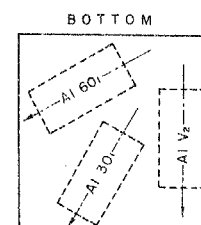
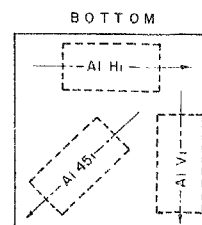
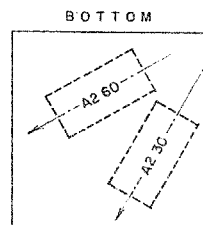
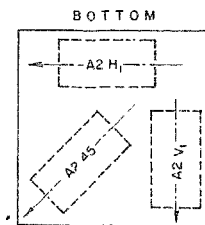
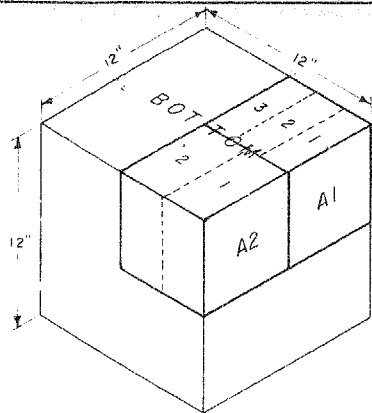
BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

TABLE OF RESULTS

BLOCK SAMPLE # J	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT)	STRAIN (%)	UNIT WEIGHT (LB./CU FT)
	A1 V1	29.8			1.29	4	125
	A1 V2	30.7			1.17	2.5	120
	A1 30°	29.2	18.4	39.0	1.19	5	125
	A1 45°	26.6			1.30	6	126
	A1 45°-2	26.4			1.27	10	126
	A1 60°	25.2			1.30	9	111
	A1 H1	26.6	20.9	36.8	1.10	15.5	126
*	A2 V1	25.6			1.31	9	100
*	A2 30°	30.7			1.03	9	95
*	A2 45°	30.0			1.24	9	97
*	A2 60°	30.2			.93	10	93
*	A2 H1	26.8			1.26	14	100

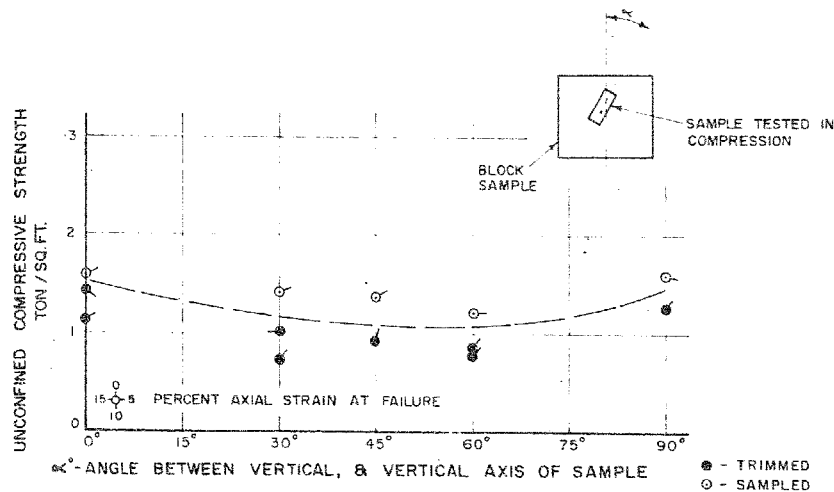


SUMMARY OF LABORATORY RESULTS  
BLOCK SAMPLE 'K'  
FROM ELEVATION 508-7 TO 507-7

FIGURE 12

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

TABLE OF RESULTS						
BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT.)	STRAIN (%)
						UNIT WEIGHT (LB./CU. FT.)
*	A2 V1				1.59	3.5
*	A2 30				1.42	3.5
*	A2 45				1.38	3.5
*	A2 60				1.22	5
*	A2 H1				1.59	6
	A1 V1				1.42	7
	A1 V2				1.12	3
	A1 30°				1.02	15
	A1 30°-2				.73	2
	A1 45				.96	1.5
	A1 60				.87	2.5
	A1 60-2				.82	3
	A1 H1				1.28	2.5
						112
						112
						112
						114
						114
						114
						114
						114
						114
						112

## BLOCK 'F'

TEST NO.	CONSOLIDATION PRESSURE lbs/sq.in.	INITIAL w %	FINAL w %	$(\sigma_1 - \sigma_3)$ lb./sq.in.
A5 45 <sub>1</sub>	1.0	30.5	30.5	17.05
A6 45 <sub>1</sub>	2.0	32.6	31.0	23.10
A6 45 <sub>2</sub>	3.0	29.3	27.8	30.80
A6 45 <sub>3</sub>	5.0	31.3	28.2	37.30

TEST METHOD: CIU,  $\sigma_1$  INCREASING

NOTE: Nominal correction 2 lbs/sq.in. for membranes and drains.

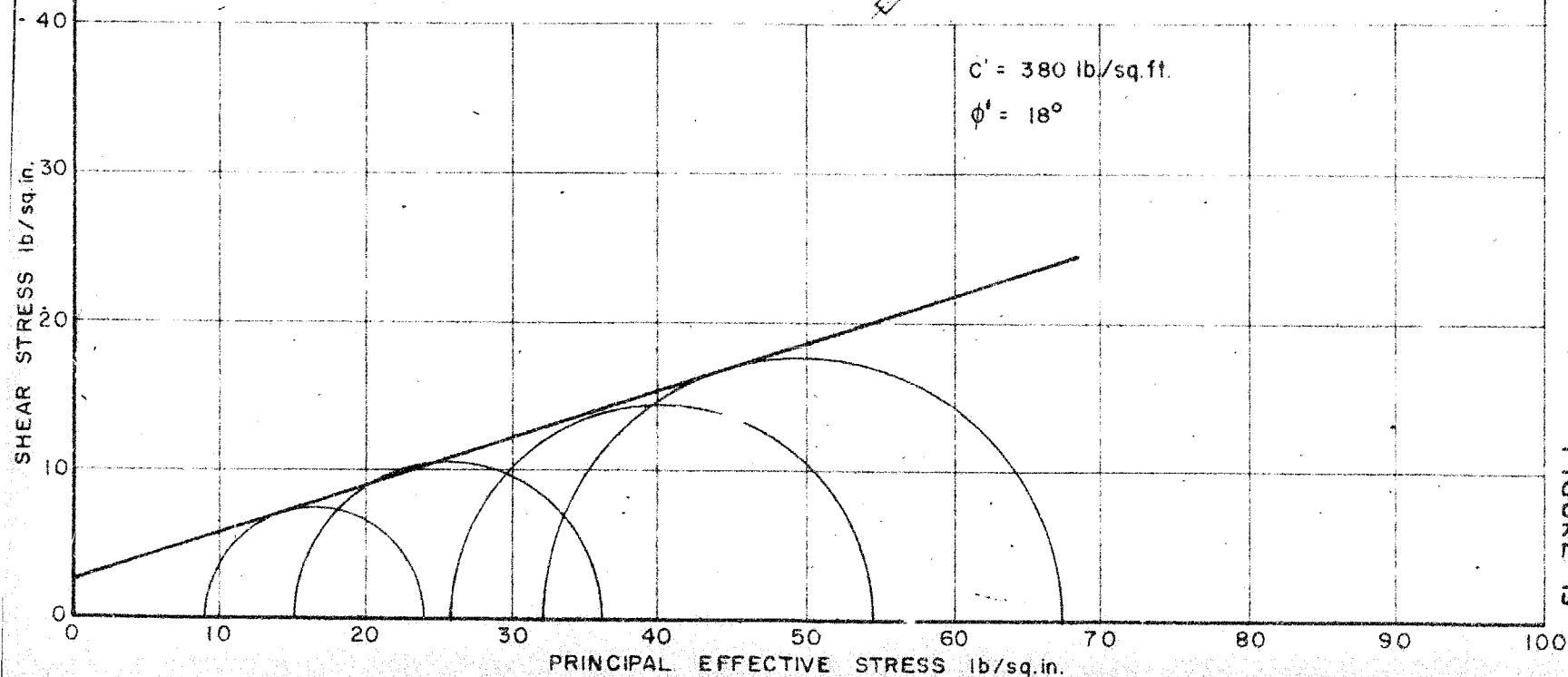
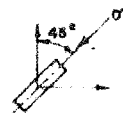
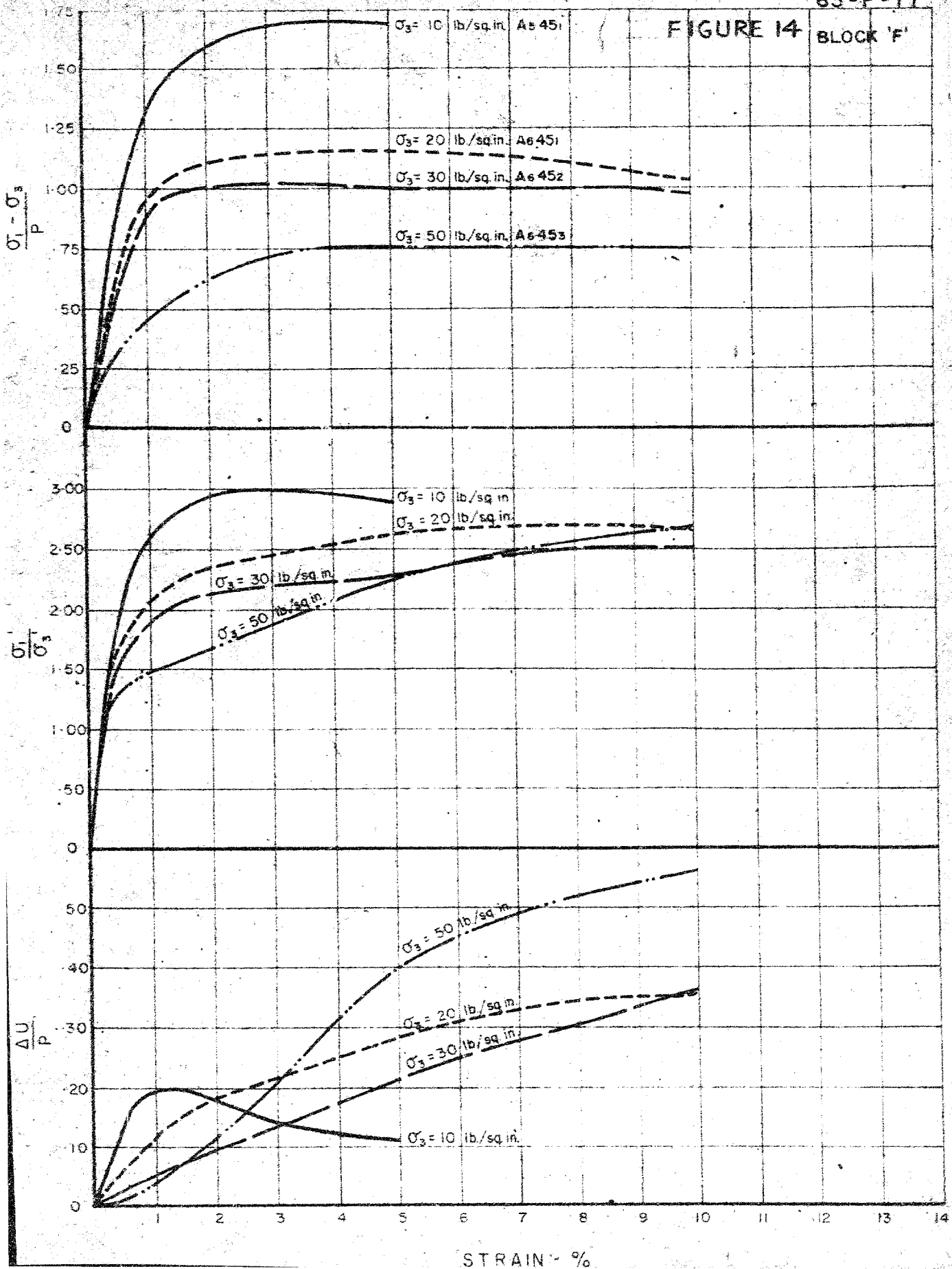


FIGURE 13

FIGURE 14  
BLOCK 'F'

**H. Q. GOLDER & ASSOCIATES LTD.**

**CONSULTING CIVIL ENGINEERS**

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

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

June 7, 1966.

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

Attention: Mr. S. M. Devata, P.Eng.,  
Senior Foundation Engineer.

Dear Sirs:

Further to yesterday's telephone conversation we enclose, as requested, a copy of the following figures from our report 6375, dated July, 1964 entitled "Trial Shaft at Proposed Tunnel Site - Execution, Sampling and Test Results - Welland, Ontario".

Figures 1A, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35,  
36, 38, 48, 68, 69, 70, 73 and 74.

Yours truly,

H. Q. GOLDER & ASSOCIATES LTD.,



JLS:hdg  
6375

J. L. Seychuk, P.Eng.

Enclosures:

WELLAND TEST SHAFT

TEST RESULTS OF BLOCK SAMPLES

March 1964 W.J.63-F-77

April 1, 1964.

Gibb Underwood & McLellan,  
Consulting Engineers,  
42 Charles Street East,  
Toronto, Ontario.

Attention: Mr. Roy Backhorse.

Enclosed please find our results for the  
block samples taken from the Welland Test Shaft.

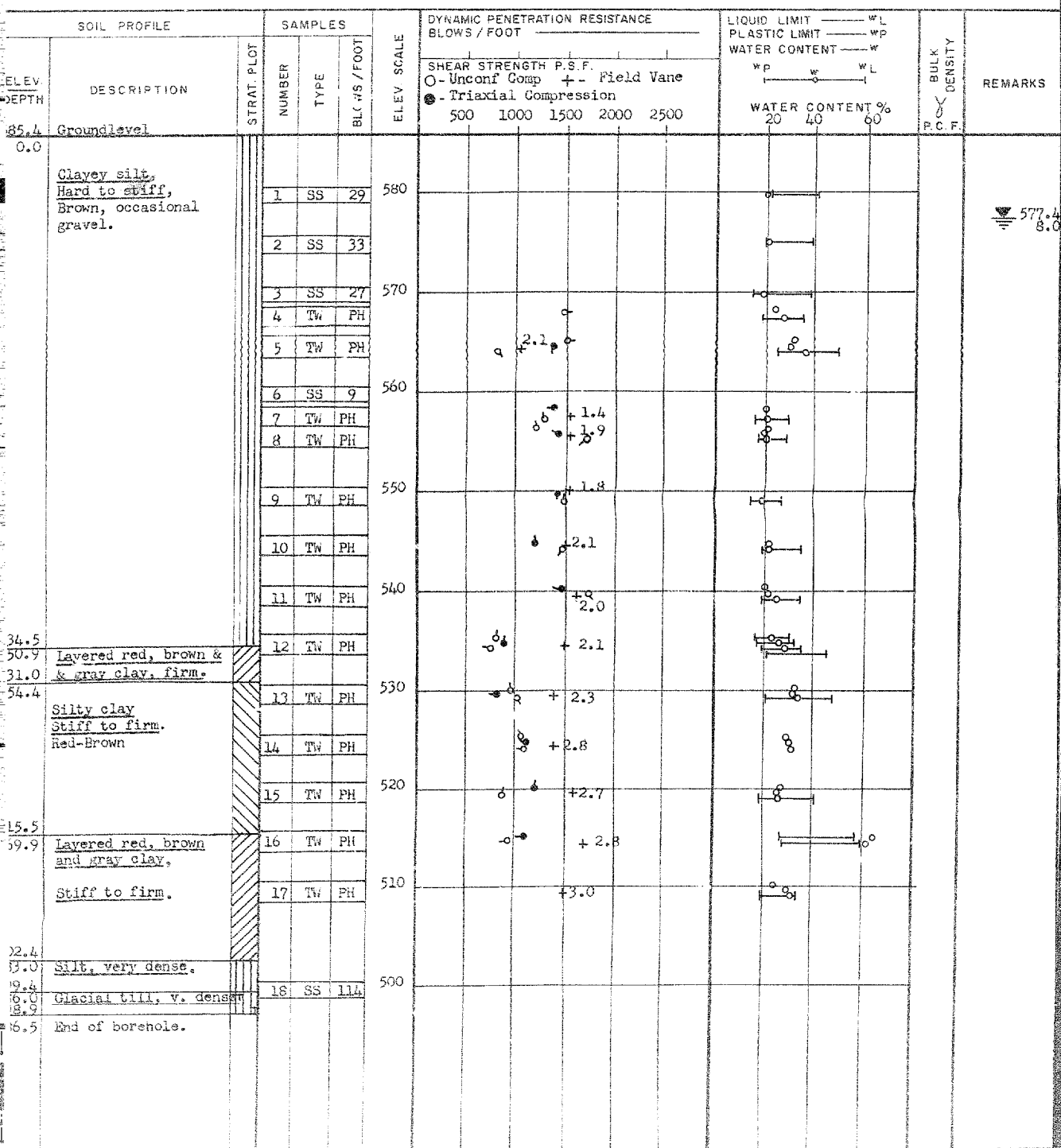
Yours truly,



A. G. Stermac  
Principal Foundation Eng'r.

Encl.  
AGS/tt

JOB 63-F-77 LOCATION Welland Tunnel; Sta. 9+41 W 39' Rt. of E ORIGINATED BY G.C.  
W.P. 130-61 BORING DATE Aug. 12, 1963. COMPILED BY G.C.  
DATUM Geodetic 585.4 BOREHOLE TYPE Auger Boring CHECKED BY K.S.





DIVISION

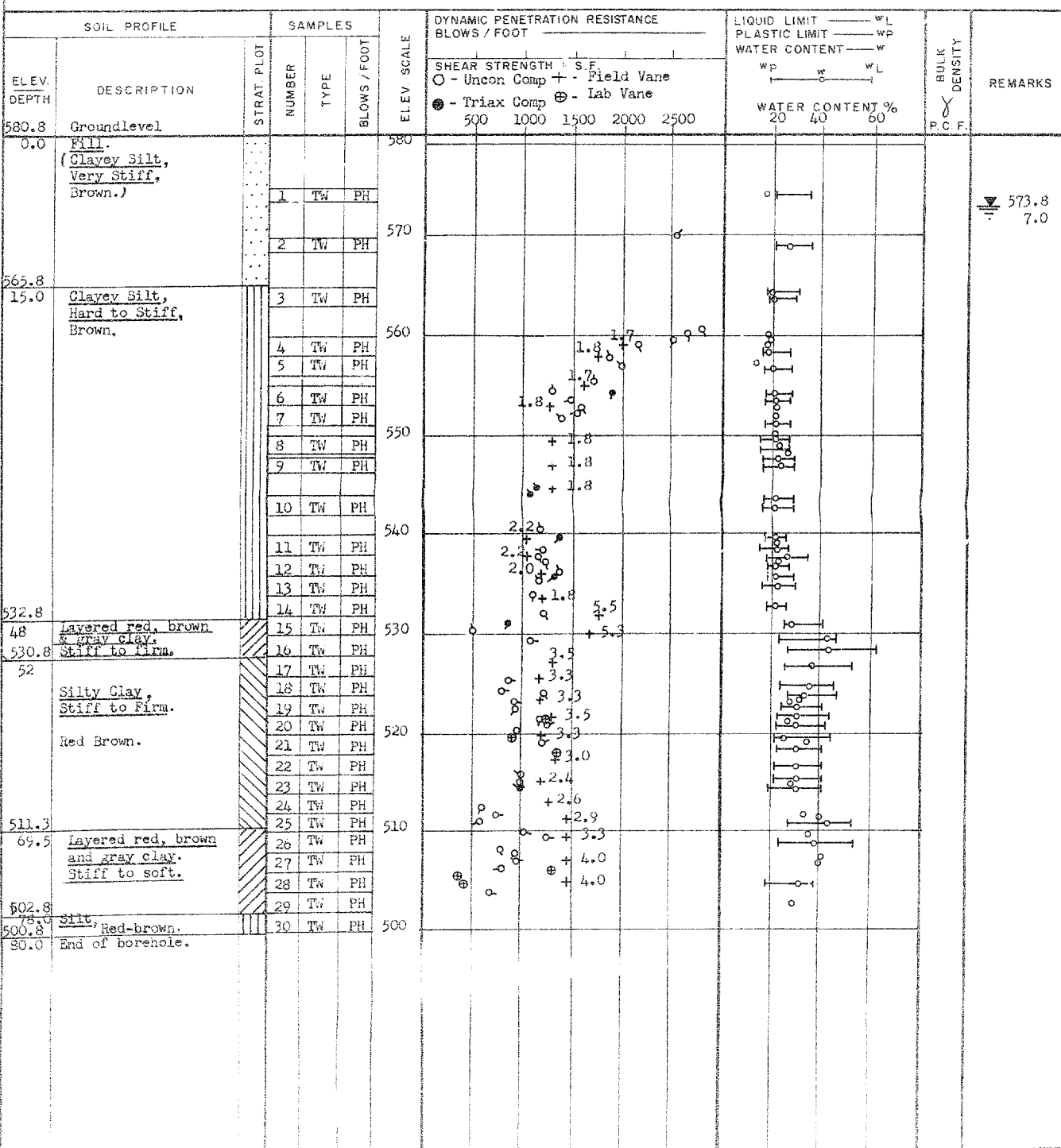
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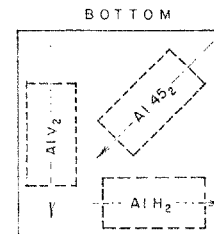
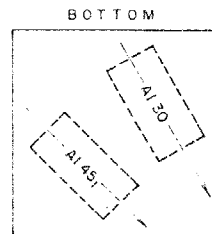
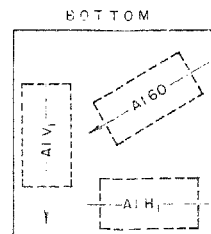
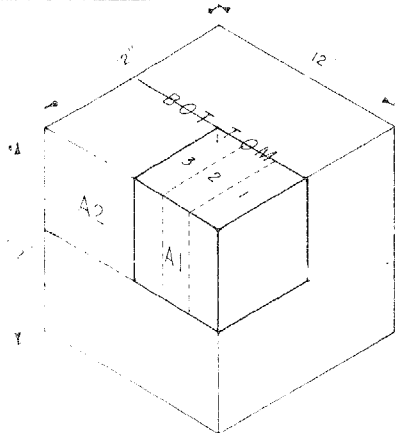
DIVISION

DIVISION

DIVISION

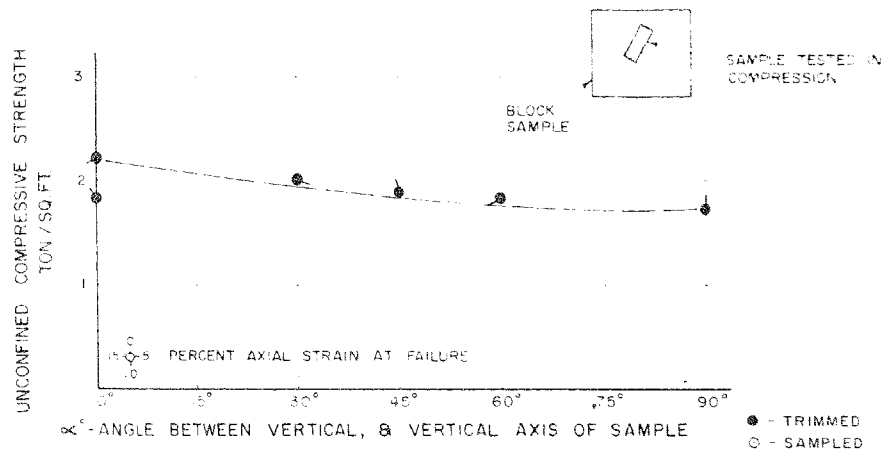
JOB 63-F-77 LOCATION Welland Tunnel Sta. 2439 3' Lt. of E OR INITIATED BY G.C.  
W.P. 130-61 BORING DATE July 9, 1963. COMPILED BY G.C.  
DATUM Geodetic 580.8 BOREHOLE TYPE Penn. Drill CHECKED BY K.S.





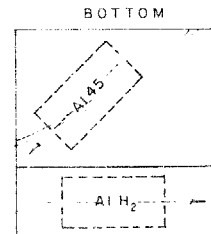
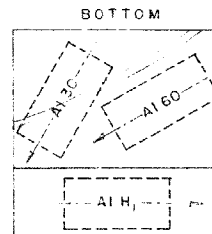
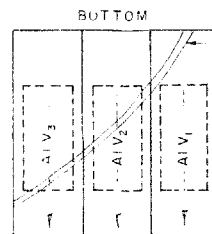
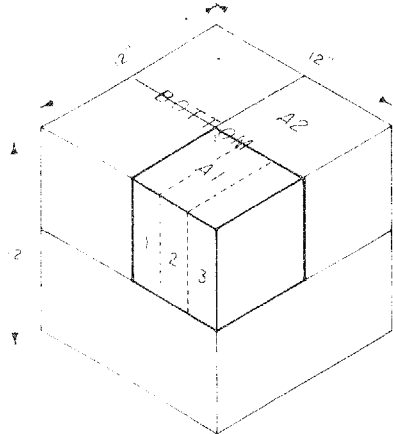
SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING



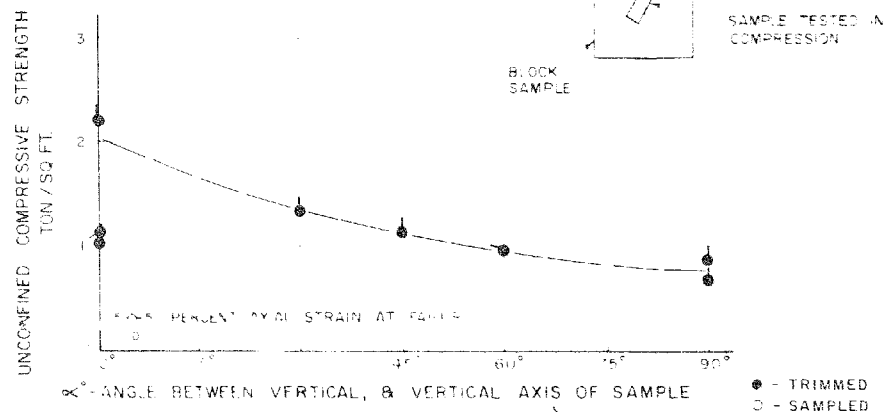
VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

TABLE OF RESULTS							
BLOCK SAMPLE #A	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQFT)	STRAIN (%)	UNIT WEIGHT (LB/CU. FT.)
	A1 V1	10.5	17.4	28.5	1.83	19	131
	A1 H1	18.7	16.6	29.9	1.71	20	132
	A1 30°	19.6	18.7	29.8	2.00	7	133
	A1 45°	16.8	17.2	28.7	1.89	19	132
	A1 45°-2	19.7	18.3	32.6	4.44	6	133
	A1 60°	21.0	16.4	29.7	1.82	13	132
	A1 H2	19.0	17.8	32.5	3.74	5	133
	A1 V2	19.4	16.6	29.4	2.18	13	133



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

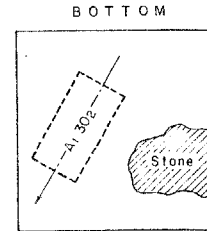
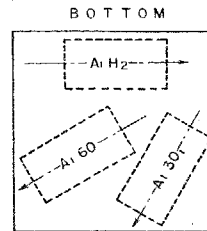
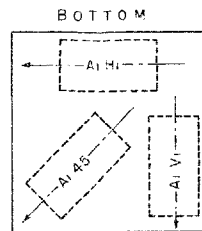
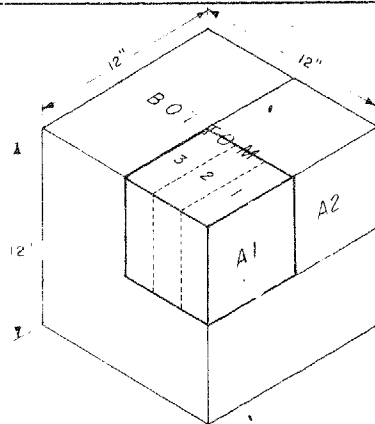
BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

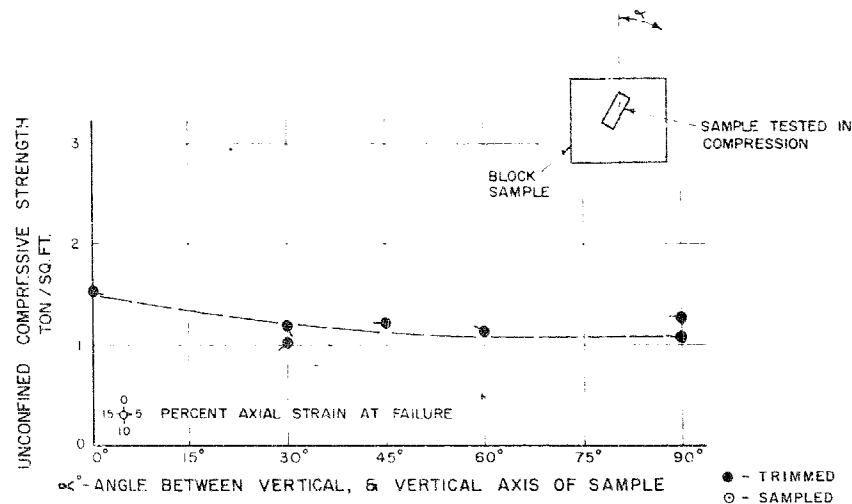
TABLE OF RESULTS

BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ. FT.)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
B	A1 V1	21.5	17.7	28.5	2.20	20	132
	A1 V2	20.8	16.7	27.4	1.04	20	129
	A1 V3	21.0	16.7	27.8	1.10	12.5	132
	A1-30°	19.9			1.33	20	130
	A1-45°	21.2			1.12	20	128
	A1-60°	21.2	16.7	27.6	.98	17	129
	A1 H1	21.3			.69	20	128
	A1 H2	19.8	17.7	28.5	.90	20	128



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

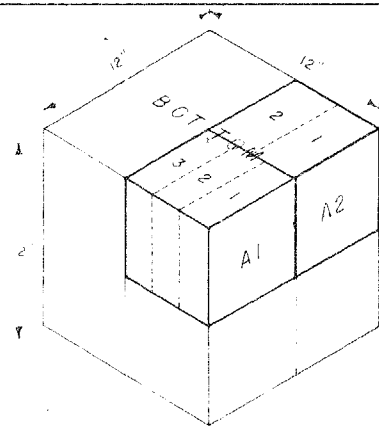
BLOCK SAMPLE SHOWING SLICING



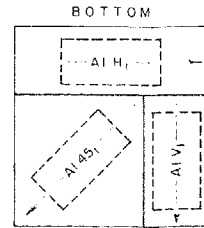
VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

TABLE OF RESULTS

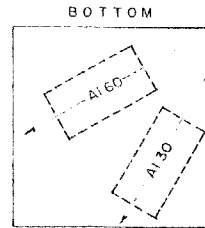
BLOCK SAMPLE # C	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT.)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
	A1 V1	21.3	17.5	28.5	1.51	6	130.0
	A1-30°	22.0	16.0	27.2			
	A1-30°-2	20.1	15.2	28.0	1.05	13	130.0
	A1-45°	19.6	17.0	28.6	1.20	8	130.6
	A1-60°	21.8	15.3	28.3	1.22	15	131.2
	A1 H1	19.5	17.4	28.0	1.13	17	129.2
	A1 H2	20.5	16.8	27.2	1.29	15	130.0
					1.09	15.5	127.9



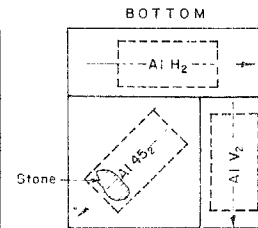
BLOCK SAMPLE SHOWING SLICING



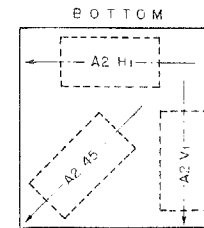
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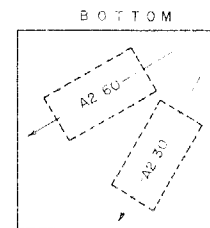
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3

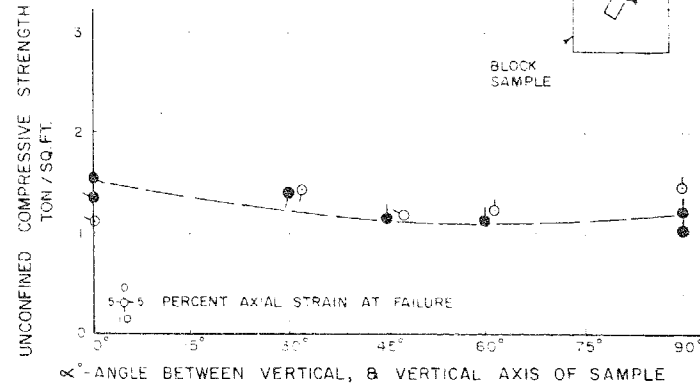


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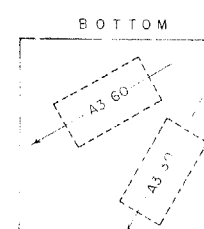
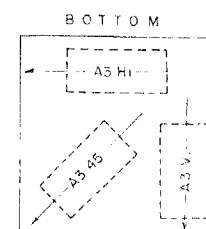
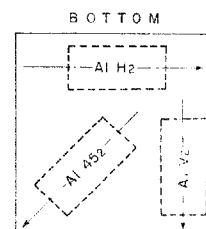
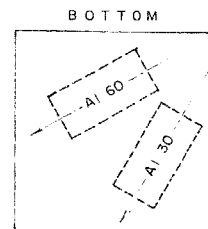
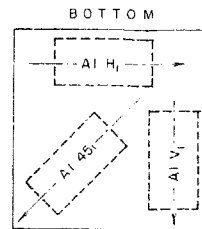
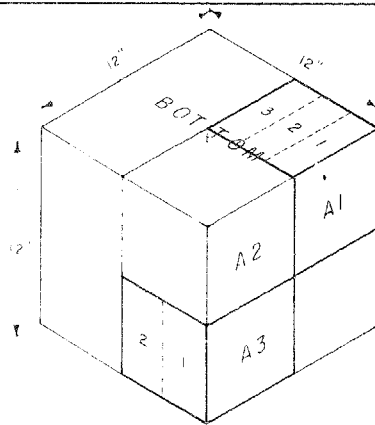
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SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED



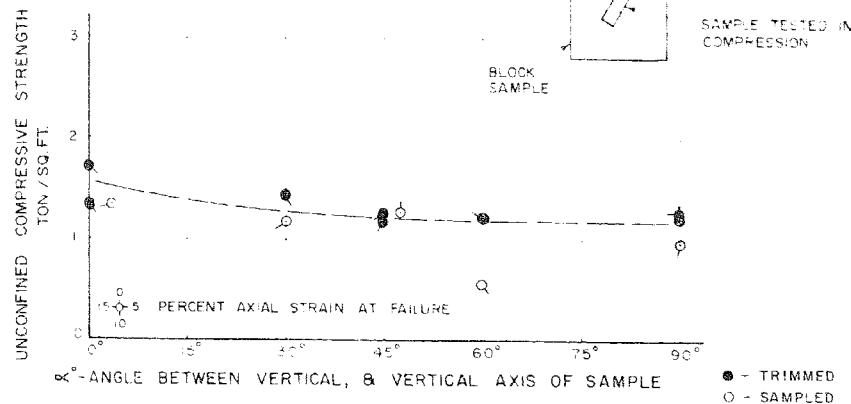
VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

TABLE OF RESULTS							
BLOCK SAMPLE # D	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ. FT.)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
	A1 V1	20.8			1.39	16	128
	A1 V2	21.0			1.54	7	128
	A1 30°	21.2			1.42	11	130
	A1-45°-1	20.5			1.16	20	130
	A1-45°-2						
	A1-60°	19.4			1.15	20	130
	A1 H1	19.9			1.01	20	132
	A1 H2	19.5			1.22	20	130
*	A2 V1	21.0			1.14	16	132
*	A2 30	20.0			1.44	11	131
*	A2 45	21.6			1.19	17	131
*	A2 60	18.6			1.25	20	132
*	A2 H1	18.8			1.46	20	132



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

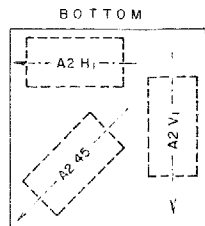
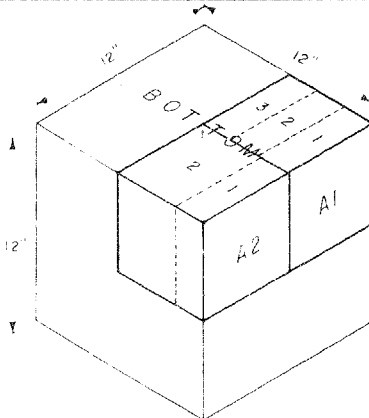
BLOCK SAMPLE SHOWING SLICING



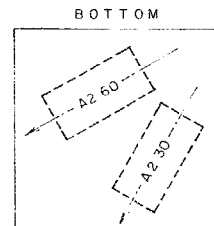
VARIAION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

TABLE OF RESULTS

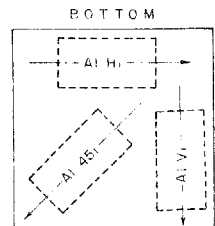
BLOCK SAMPLE # E	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT)	STRAIN (%)	UNIT WEIGHT (LB/CU FT.)
	A1 V1	21.3	15.8	28.0	1.37	4	133
	A1 V2	21.0			1.74	7	132
	A1 30°	21.1			1.46	8	131
	A1 45°	22.7	17.0	28.5	1.20	12	132
	A1 45°-2	22.2			1.28	13	131
	A1 60°	19.8	17.8		1.24	17	132
	A1 H1	20.2	16.3		1.22	20	133
	A1 H2	20.7			1.26	15	133
*	A3 V1	11.1	15.0	33.1	1.36	14	127
*	A3 30°	14.6	15.9	34.5	1.16	13	125
*	A3 45	9.0	19.2	32.9	1.29	20	131
*	A3 60	17.2	18.3	44.0	.57	8	121
*	A3 H1	15.0	18.9	40.5	.98	11	119



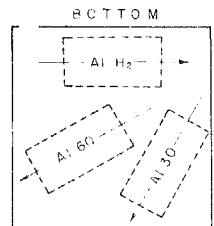
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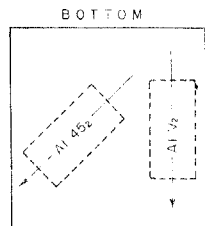
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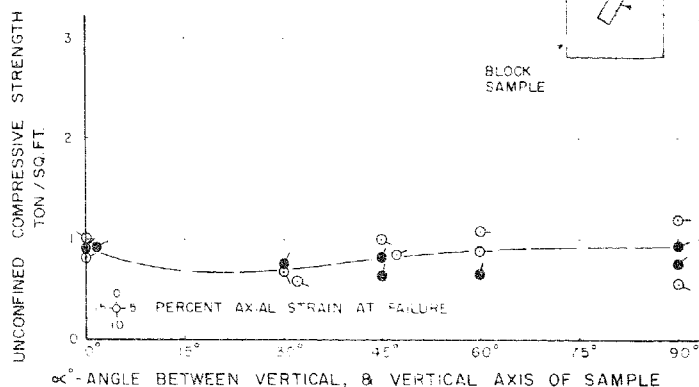
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3

SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING

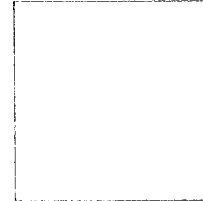
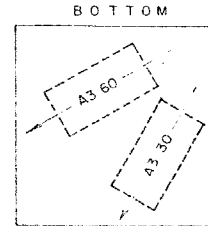
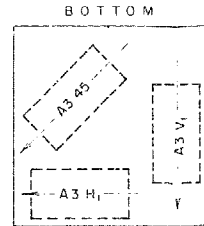
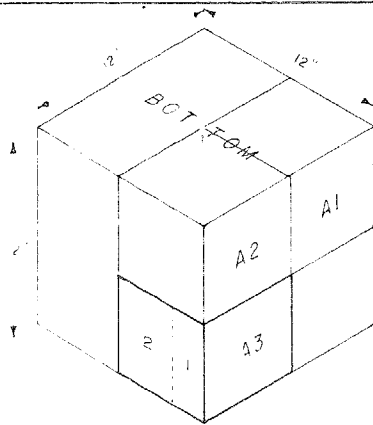


VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

TABLE OF RESULTS

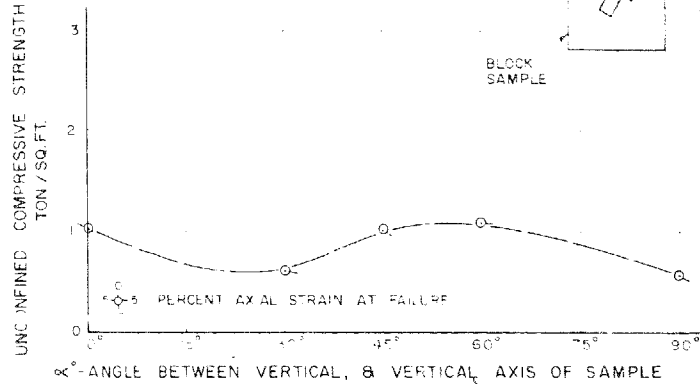
TABLE OF RESULTS							
BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT.)	STRAIN (%)	UNIT WEIGHT (LB./CU.FT.)
P	A1 V1	20.2	45.5	.91	3	122	
	A1 V2			.92	3	121	
	A1 30°			.73	2	122	
	A1 45°			.81	2	124	
	A1-45°-2			.65	1.5	121	
	A1 60°			.67	1	122	
	A1 H1			.78	2.5	122	
	A1 H2			.93	4	118	





SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

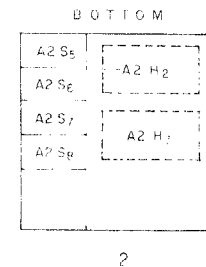
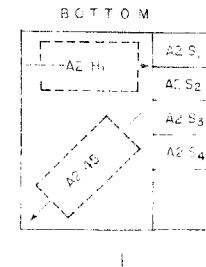
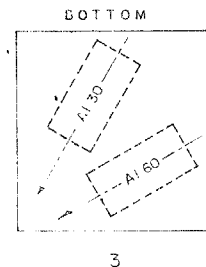
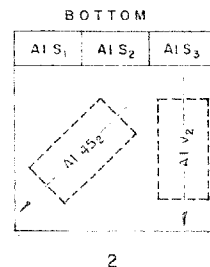
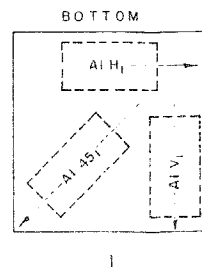
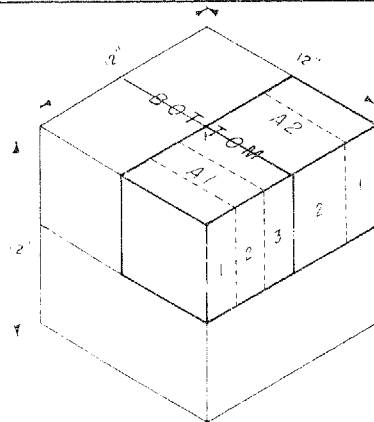
BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

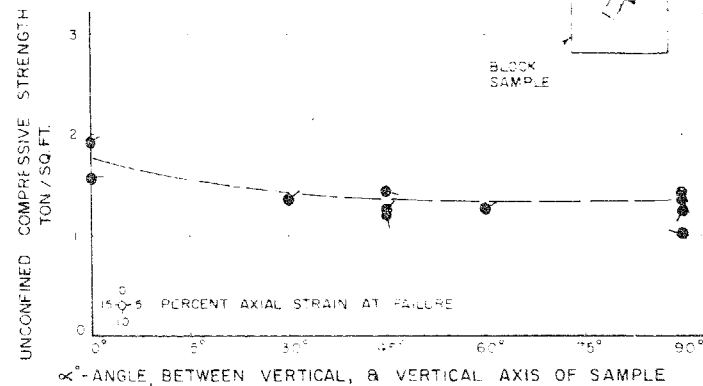
TABLE OF RESULTS

BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT.)	STRAIN (%)	UNIT WEIGHT (LB./CU.FT.)



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

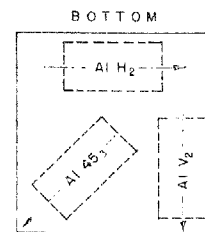
BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

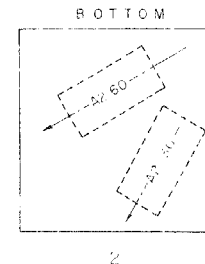
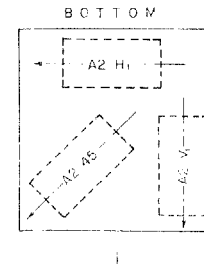
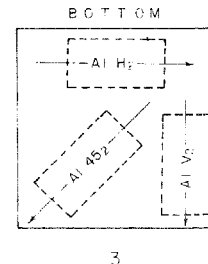
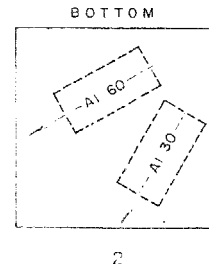
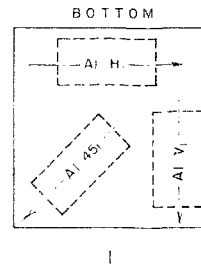
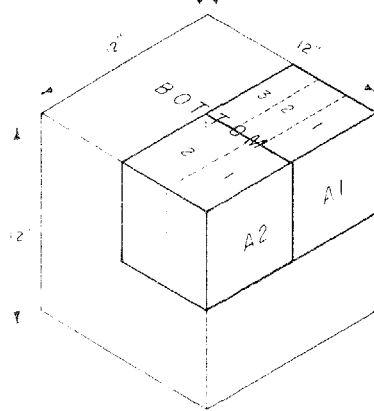
TABLE OF RESULTS

BLOCK SAMPLE *G	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQFT)	STRAIN (%)	UNIT WEIGHT (LB/CU. FT.)
	A1 V1	33.0	24.9	40.8	1.54	4	119
	A1 V2	33.4	23.5	48.2	1.92	3.5	120
	A1 30°		22.5	49.3	1.37	2.5	120
	A1 45°	34.1	23.1	48.4	1.27	9.0	120
	A1 45°-2		21.7	50.3	1.20	2	118
	A1 60°	32.4	23.9	47.9	1.27	3	119
	A1 H1	33.2	22.0	47.3	1.04	17	119
	A2 H1	31.0	18.9	44.9	1.26	12	121
	A2 H2	34.2	22.4	46.8	1.42	9	122
	A2 H3	34.2			1.39	9	121
	A2 45	33.5	21.7	47.4	1.43	5.5	121



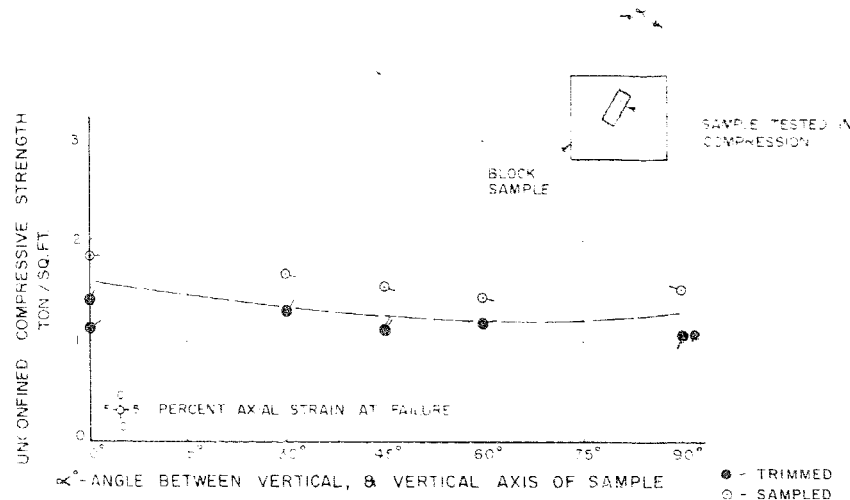
VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT)	STRAIN (%)	UNIT WEIGHT (LB./CU. FT.)
H	A1 V1	33.9	20.8	44.3	1.83	2.5	120
	A1 V2	32.6	22.6	46.0	1.59	2.5	120
	A1 30	31.5	23.6	43.6	1.66	2	119
	A1 45	-	21.9	49.2	-	-	-
	A1 45-2	31.5	21.8	44.3	1.45	4	120
	A1 45-3	32.3	-	-	1.61	4	121
	A1 60	32.8	21.3	48.6	1.37	8	120
	A1 H1	31.1	19.0	43.3	1.38	15	121
	A1 H2	31.4	-	-	1.35	8	120



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

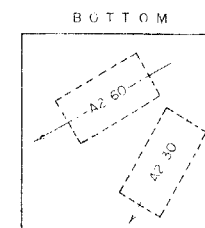
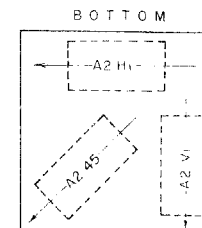
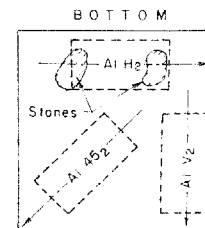
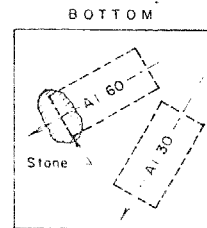
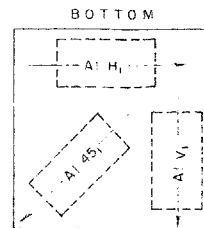
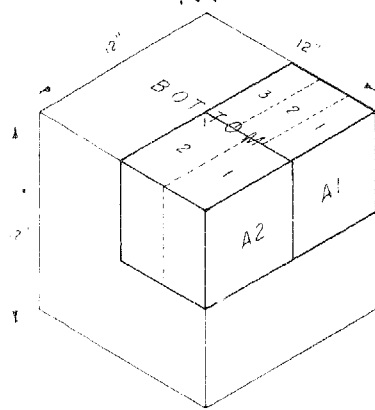
BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

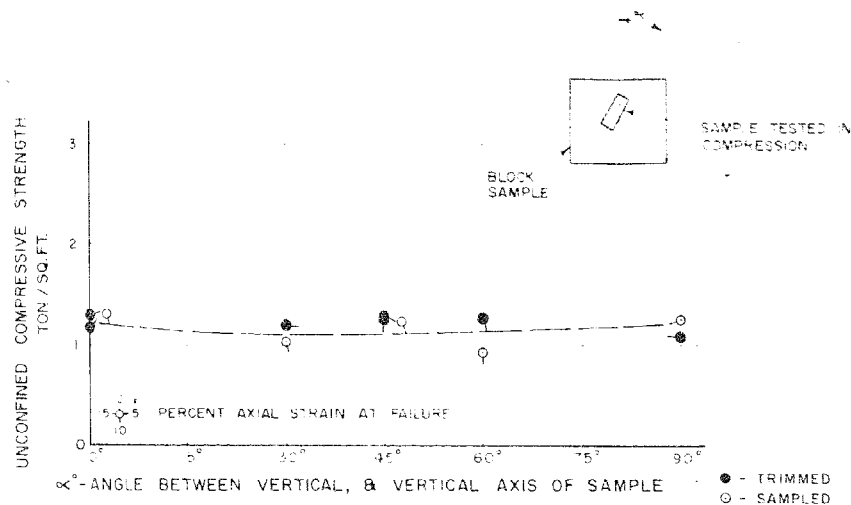
TABLE OF RESULTS

BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT)	STRAIN (%)	UNIT WEIGHT (LB./CU.FT.)
I	A1 V1	31.7	21.6	44.3	1.13	2.5	123
	A1 V2	31.7			1.40	1.5	120
	A1 30°	31.9	19.9	43.0	1.30	2	121
	A1 45°	30.9			1.11	1.5	121
	A1-45-2	31.0			1.10	2	121
	A1 60	31.2			1.19	4	121
	A1 H1	30.7			1.07	11	122
	A1 H2	31.1			1.06	11	121
	A2 V1	30.0			1.84	5	123
	A2 30	30.7			1.47	6	123
	A2 45	30.1			1.56	7	123
	A2 60	31.4			1.44	7	124
	A2 H1	27.2			1.51	16	126



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

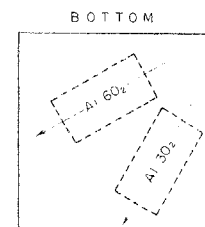
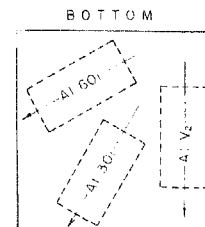
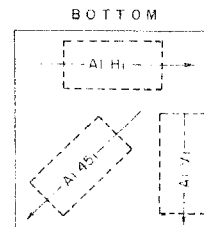
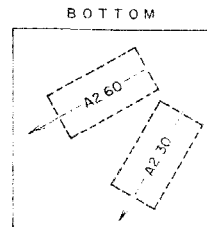
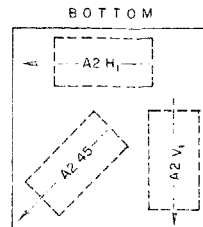
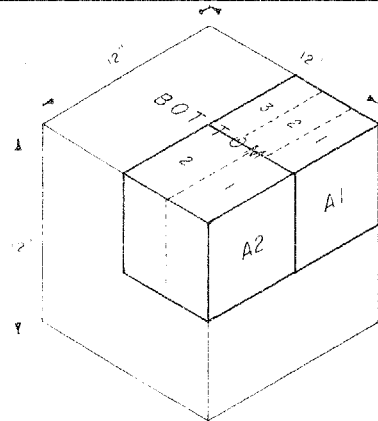
BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ORIENTATION OF SAMPLE

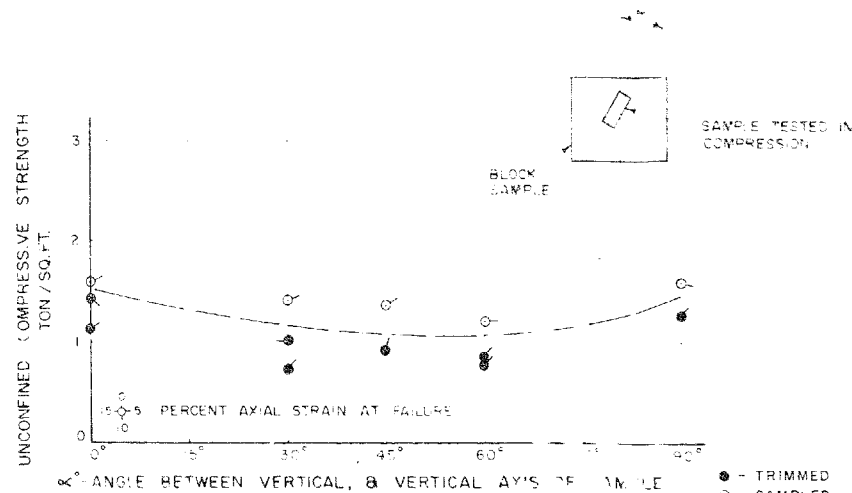
## TABLE OF RESULTS

TABLE OF RESULTS							
BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT)	STRAIN %	UNIT WEIGHT (LB / CU. FT.)
J	A1 V1	29.8	18.4	39.0	1.29	4	125
	A1 V2	30.7			1.17	2.5	120
	A1 30°	29.2			1.19	5	125
	A1 45°	26.6			1.30	6	126
	A1 45°-2	26.4	20.9	36.8	1.27	10	126
	A1 60°	25.2			1.30	9	111
	A1 H1	26.6			1.10	15.5	126
	* A2 V1	25.6			1.31	9	100
	* A2 30°	30.7			1.03	9	95
	* A2 45	30.0			1.24	9	97
	* A2 60	30.2			.93	10	93
	* A2 H1	26.8			1.26	14	100



SLICES OF BLOCK SAMPLE SHOWING POSITIONS OF SAMPLES TESTED

BLOCK SAMPLE SHOWING SLICING



VARIATION IN UNCONFINED COMPRESSIVE STRENGTH WITH ROTATION OF SAMPLE

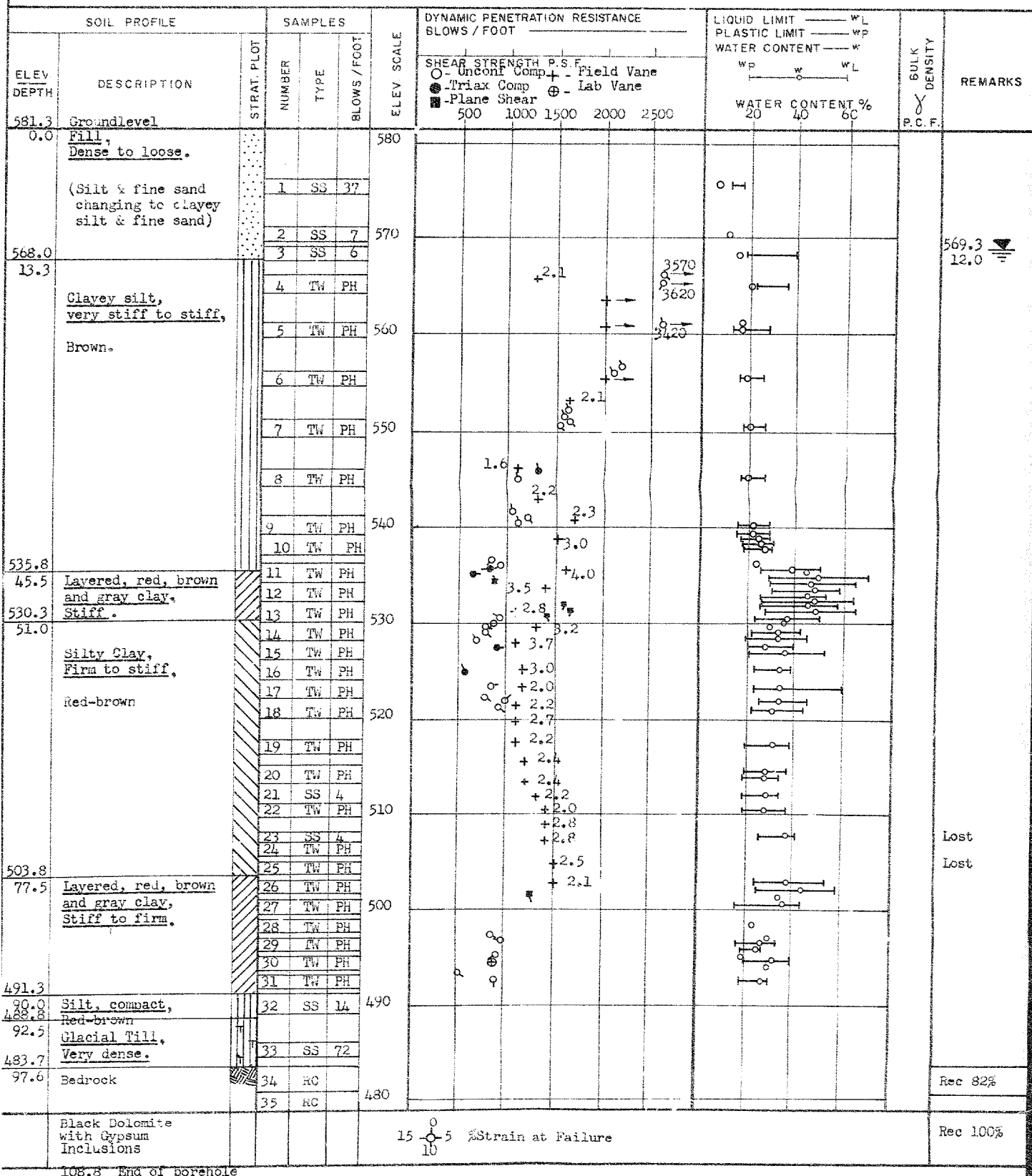
TABLE OF RESULTS

BLOCK SAMPLE #	ANGLE OF SAMPLE TO VERTICAL	MOISTURE CONTENT (%)	PLASTIC LIMIT	LIQUID LIMIT	STRESS (TONS/SQ.FT.)	STRAIN %	UNIT WEIGHT (LB./CU. FT.)
*	A2 V1				1.59	3.5	112
*	A2 30				1.42	3.5	112
*	A2 45				1.38	3.5	112
*	A2 60				1.22	5	-
*	A2 H1				1.40	6	112
	A1 V1				1.42	7	114
	A1 V2				1.12	3	114
	A1 30°				1.02	15	114
	A1 30°-2				.73	2	113
	A1 45				.96	1.5	114
	A1 60				.87	2.5	114
	A1 60-2				.82	3	114
	A1 H1				1.28	2.5	112

JOB 63-F-77 LOCATION Welland Sta. 4+00 W. on E ORIGINATED BY G.C.  
W.P. 130-61 BORING DATE Aug. 1, 1963. COMPILED BY G.C.  
DATUM Geodetic 581.2 BOREHOLE TYPE Auger Boring CHECKED BY K.S.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F. ○ — Unconfined Compression ● — Triaxial Compression					WP	W	WL		
581.2	Groundlevel						500	1000	1500	2000	2500	20	40	60		
0.0	<u>Clayey silt,</u> <u>very stiff to stiff,</u>  Brown.					580										577.2 4.0
			1	TW	PH											
			2	TW	PH	570										
			3	TW	PH											
			4	TW	PH	560										
			5	TW	PH											
			6	TW	PH	550										
			7	SS	11											
			8	SS	15	540										
536.2																
45.0	<u>Layered, red, brown</u> <u>and gray clay,</u> <u>Stiff.</u>		9	SS	12											
29.7			10	SS	10	530										
51.5	<u>Silty clay, stiff,</u> <u>red-brown.</u>		11	SS	8											
			12	SS	9	520										
16.0			13	SS	11											
65.2	<u>Layered red, brown</u> <u>and gray clay,</u> <u>Stiff.</u>		14	SS	14	510										
10.5			15	SS	115											
70.7	<u>Silt,</u> <u>Very dense,</u> <u>Red brown.</u>															
99.7						500										
81.5	End of borehole.					490										

JOB 63-F-77 LOCATION Welland Tunnel - Sta. 2458 E 6' Lt. of E ORIGINATED BY G.C.  
W.P. 130-61 BORING DATE Aug. 7, 1963. COMPILED BY G.C.  
DATUM Geodetic 581.3 BOREHOLE TYPE Auger Boring & Wash Boring CHECKED BY K.S.





**GIBB, UNDERWOOD & McLELLAN**

**CONSULTING PROFESSIONAL ENGINEERS**

**42 CHARLES STREET EAST  
TORONTO 5, CANADA**

**PARTNERS:**

SIR ALEXANDER GIBB & PARTNERS  
UNDERWOOD, McLELLAN & ASSOCIATES LIMITED

**CHIEF ENGINEER:**

TELEPHONE WALNUT 4-1452

A. C. R. ALBERY, M.C., B.A., M.I.N.S.T.C.E., M.E.I.C., M.A.S.C.E., M.I.S.T.R.U.C.T.E., P. ENG.

**DEPUTY CHIEF ENGINEER**

G. J. SLADEK, M.I.E.C., P. ENG.

November 15th, 1965.

Department of Highways,  
Materials & Research Division,  
Foundation Section,  
Parliament Buildings,  
Toronto 2, Ontario.

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

RE: Welland Canal Tunnel, Additional Site  
Investigation

Dear Sir:

Further to the discussion in your office on November 10th, 1965, we enclose two copies each of our drawings 153-G-1 and 154-G-1.

On drawing 153-G-1 we have indicated further requirements of soil borings together with piezometer locations for a pumping test to determine permeabilities of the upper layers of bedrock and the silty layer immediately above till.

On the west side of the Canal 5 additional boreholes numbered 301 to 305 inclusive are proposed to determine the soil properties at these locations and to enable interpolation between these holes and the already existing boreholes further south.

We are particularly looking for the shear parameters  $c^1$  and  $\phi^1$  to be used for the design of the permanent cut in the clay and also for the apparent cohesion  $c_u$ .

For ground water investigation, we propose to pump the existing shaft. Two sets of piezometers would be installed. P5 to P8 inclusive in the silty layer immediately above till and P9 and P10 approximately 4 feet into rock.

Cont'd.....

November 15th, 1965.

Piezometers and the water level in the shaft must be observed at regular intervals during steady pumping from the shaft for a sufficiently long time.

On the east side of the Canal, eight additional boreholes numbered 306 to 312 inclusive are proposed.

Holes 311 and 312 should provide sufficient information for the foundation design of the proposed overpass structures at these locations.

At this very preliminary stage, we visualize these overpass structures founded on spread footings approximately 15 to 20 feet below existing grade or alternatively, on piles. However, we intend to discuss this matter further with the Department of Planning before the boreholes at these locations are put down.

The remainder of the holes on this side of the Canal are intended to enable interpolation between these holes and the already existing boreholes further south.

Here, as on the west side, we are looking for the shear parameters  $c_1$  and  $\phi^1$ , but also for the apparent cohesion  $c_u$ .

The accuracy of scaling the drawing is sufficient for the layout of the boreholes in the field, but upon completion the location of holes should be measured in relation to the CO-ORDINATE CONTROL SURVEY (NO. W.O.H.O.(63-2)63-558 PLAN NO. C-S-6).

On the drawing 154-G-1 we have shown two alternate off-site drydock locations.

We are looking for soils information for the design of the necessary cut as well as permeability of the top layers of the rock.

Yours very truly,

for GIBB, UNDERWOOD & MCLELLAN,

*G. J. Sladek*  
G. J. Sladek.

GJS/aw  
encls.

c.c. F.I. Hewson  
c.c. V. Milligan, H.Q. Golder & Assoc.

H. Q. Golder

Attn: Mr V. Lulligan

Re: Welland Canal Tunnel, Additional investigation

W. P. 130-61

Consider this please your authorization to carry out the additional site investigation for the proposed Welland Canal Tunnel.

The extent of the investigation was discussed with the designer Gibb, Underwood and Mc Lelland. The consultant has subsequently summarized this discussion in his letter of November 15th. 1965 of which you received a copy.

We feel that the letter contains basically all the points of discussion and that it outlines the scope of the necessary field work to be carried out. Regarding the work concerning the permeability determinations of either soil or rock we feel that some more detailing of both intentions and operations is necessary. We are therefore requesting you to consider this problem carefully and prepare a proposal which should be then discussed with the designer and ourselves.

With reference to the shear strength parameters mentioned in the consultant's letter we would like to point out that the present investigation should firstly and mainly establish whether there is any significant

change or variation in soil stratigraphy or soil type. Should the ~~investigation~~ <sup>investigation</sup> disclose that

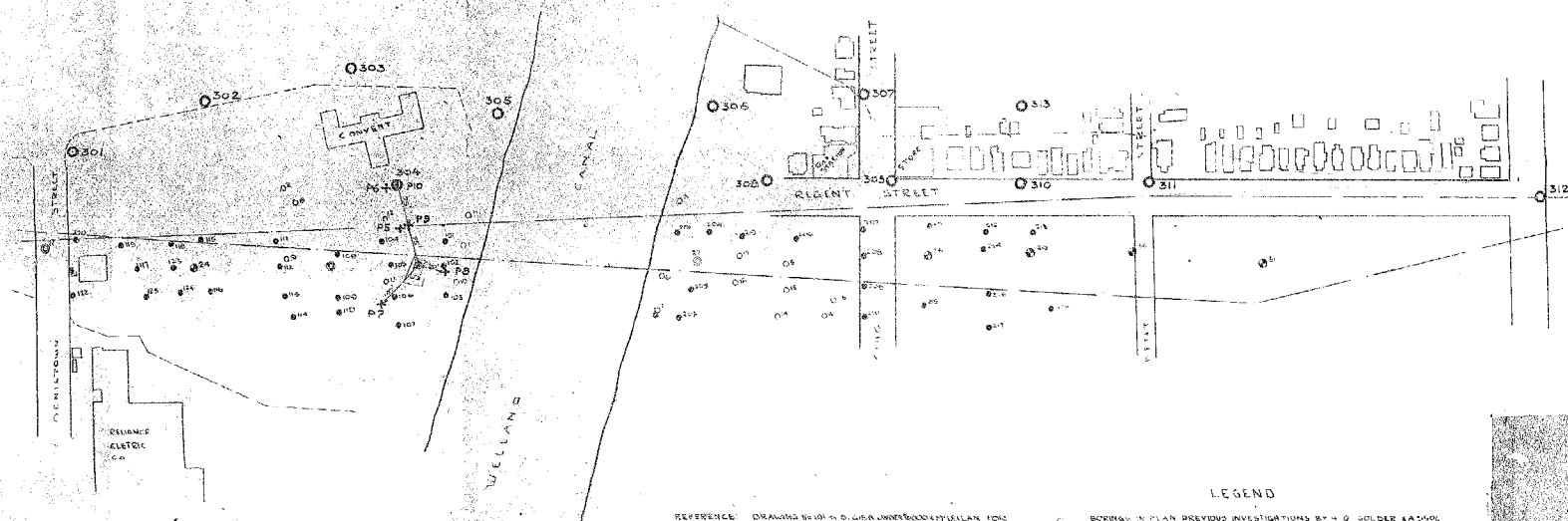
Should the identification and classification of the materials disclose the expected similarity with the materials already extensively tested we feel that it would be warranted to use the same ~~etc~~ shear strength parameters and disperse with additional lab testing at this stage.

We would also request you to discuss with us, prior to <sup>the</sup> commencement, the intended investigation of the ~~possible~~ <sup>proposed</sup> borrow areas.

Since the initial investigation was carried out as a combined venture of your organisation and ourselves we would request you to maintain a close liaison during the course of this additional work. The designer should also be kept well informed of the work progress as well as of any changes that we may agree upon in the course of the investigation.

The ~~necessary~~ drawings showing the locations of the proposed additional field work have been handed to your Mr V. Lilligan on Dec 3. 1965.

#63-F-77  
W.P. #130-61  
ADDITIONAL  
SITE INVEST.  
WELLAND  
CANAL TUNNEL

[illegible]

BORING PLAN  
SCALE 1" TO 100'-0"

REFERENCE: DRAWING NO. 101-0-D-128 UNDERBROOKTUNNELL FOR  
DEPARTMENT OF HIGHWAYS, ONTARIO OF WELLAND CANAL  
TUNNEL WALL AND ONTARIO PROPOSED LINE A & C.  
UNDATED

LEGEND

- ① BORINGS IN PLAN PREVIOUS INVESTIGATION BY W.D. COLLIER & ASSOC. (PILOT LOGS DATED FROM 1952-53 TO 1962) (ASSOC. 76)
  - ② BORINGS IN PLAN PRESENT INVESTIGATION BY THE DEPARTMENT OF HIGHWAYS' DISTRICT
  - ③ BORINGS IN PLAN PRESENT INVESTIGATION BY W.D. COLLIER & ASSOC.
  - ④ PROBLEMS IN PLAN PRESENT INVESTIGATION BY W.D. COLLIER & ASSOC.
- ⑤ 301: PROPOSED BORINGS
- ⑥ P.S. PROPOSED PIEZOMETER LOCATIONS

DEPARTMENT OF HIGHWAYS ON A BID

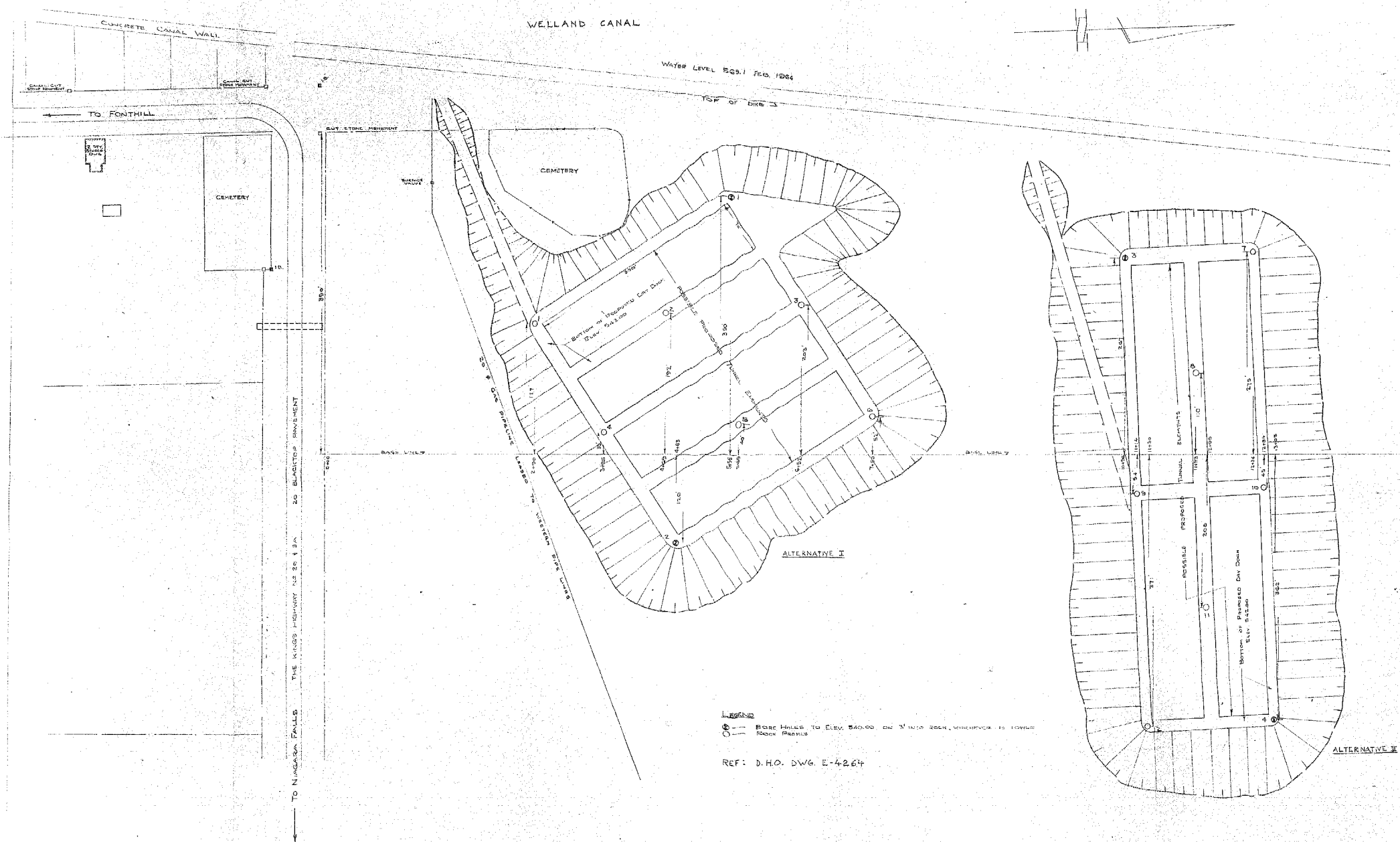
GIBB, UNDERWOOD & MCKELLAN  
CONSULTING ENGINEERS

WELLAND CANAL TUNNEL

## ADDITIONAL SITE INVESTIGATION

SCALE 1"=30' DATE NOV 19/68  
DWG. NO. 53-G-1



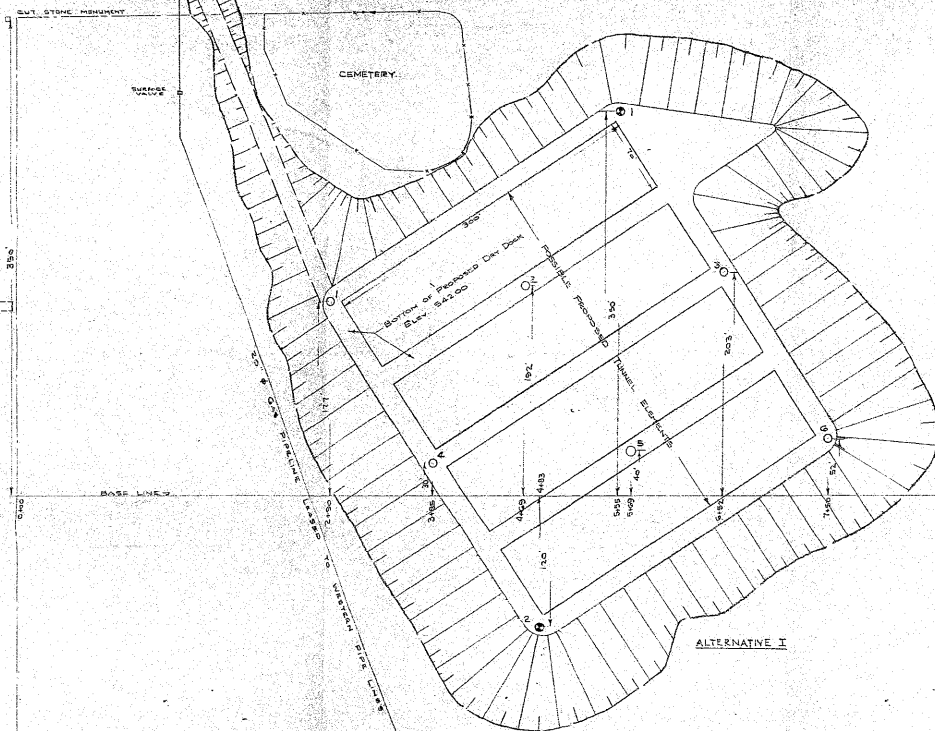




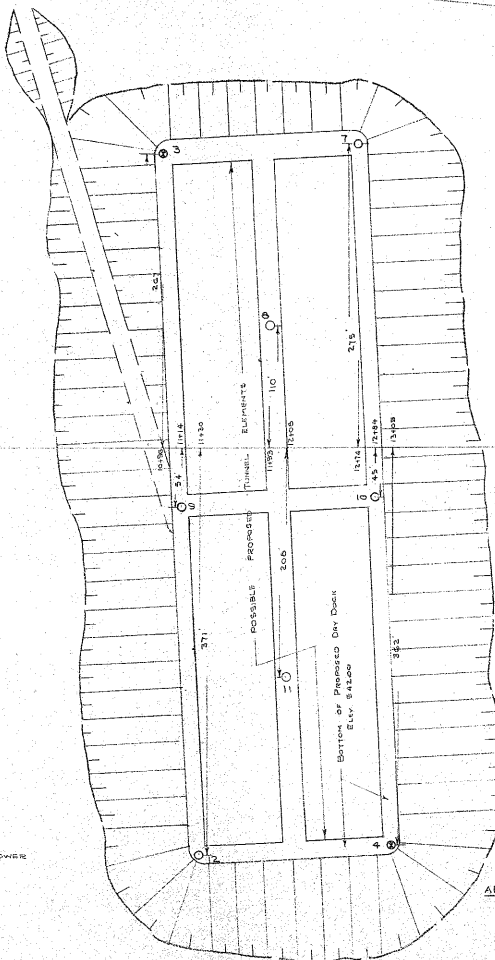
WELLAND CANAL

WATER LEVEL 559.1 Feb. 1964

TOP OF DIKE



ALTERNATIVE I



ALTERNATIVE II

LEGEND

- BORE HOLES TO ELEV. 540.00 OR 3' INTO ROCK, WHICHEVER IS LOWER
- ROCK PROBES

REF: D.H.O. DWG. E-4264

DEPARTMENT OF HIGHWAYS ONTARIO	
GIBB, UNDERWOOD & McLELLAN CONSULTING ENGINEERS	
WELLAND CANAL TUNNEL OFF-SITE DRY DOCKS PROPOSED SOIL BORINGS	
SCALE: 1"=50'	DATE: NOV 19, 65
DWG. No. 154-G-1	