Secondary Compression Behaviour of a Remolded Puget Sound Clay

by

James Cooper Madden IV

Submitted in Partial Fulfillment
of The Degree of Bachelor of Science
at the
Massachusetts Institute of Technology

May 1960

Signature of Author . Signature redacted
Department of Civil and Sanitary Engineering
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Certified by. . . . Signature redacted
Thesis Supervisor

Accepted by . . . . Signature redacted
Chairman, Departmental
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Accepted by . . . . John B. Wilbur . . . .
Chairman, Departmental
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Abstract

Pore pressures are measured in a remolded Puget Sound Clay which is loaded in small increments after secondary compression has been allowed to occur for a month under a previous load. These pressures are measured both with the Penman apparatus and with Dynisco pressure transducers.

It was found that the coefficient of consolidation, c_v, decreased with the pressure increments, approaching the value obtained during virgin compression, as the particle bond formed during secondary compression was broken.

Acknowledgements

The author is indebted to Prof. James E. Roberts of the Soil Engineering Laboratory who originally suggested this investigation.

Mr. Carl Stahle is to be thanked for making the adaptations for pore pressure measurement in the apparatus.

Mr. Kent Healy of the Soils Lab is to be thanked for his help in adapting the Dynisco transducers for the pore pressure measurement.

Miss Linda Lizza typed the manuscript.

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Introduction

Consolidation in a loaded soil consists of two phenomena. The first is a result of the rapid build-up and the slow dissipation of excess hydrostatic pressure. As the water drains from the material, the total load is gradually transferred to the soil skeleton from the liquid phase and the void volume decreases by the amount of water leaving the material. This primary compression is completed when the excess pore pressure is dissipated. On a typical strain versus log time plot for a loaded sample in the laboratory, this point is marked by a break in the curve. For a sample with pore-pressure measurements taken during consolidation, the end of primary compression can be more accurately located by a zero reading.

Further reduction of void volume under the same load increment with zero excess pore-pressure is termed secondary compression. This portion of the consolidation curve is a straight line with a flatter slope. Taylor has suggested that this secondary consolidation is a reorientation of the soil particles after the initial consolidation. Parrish, in a thesis (MIT, 1959)[1] showed that for the material used in this investigation, the slope of secondary compression is independent of the sample thickness and directly proportional to the load increment.

1. Ref, #3

Terzaghi (1941)[1] suggests that this secondary compression is a continued process of squeezing out water from between the particles as they reorientate themselves under the pressure increment. The soil particles are surrounded by a viscous layer of water which acts as a lubricant, enabling them to slide over one another, and reorientate themselves during this phase. At this point the particles develop what Terzaghi calls a film bond. If the load is kept on the soil the particles will continue to squeeze out this viscous layer of water and a solid bond will form between the individual particles. This is termed by Terzaghi as a process of solidification.

This secondary compression and subsequent solidification or grain bond is characterized by a strength regain for the soil. Consider a typical void ration versus log pressure for a soil. If the increments are increased after primary compression has occurred the plot will be a straight line. If the load is left on the sample, and secondary compression is allowed to occur there will be a vertical drop in the plot. If the particles are allowed to bond with one another under this load, the strength regain will become evident when there is no further significant decrease in void ratio for small increases in the load increment.

This load may be increased until the bond between the

1. Ref. #4

particles is broken and the void ratio continues to decrease. The load increase to this point will be horizontal on the e log p curve. Beyond this point where the grain bond has been broken, the soil will act as if their were no delay in loading and return to the virgin curve.

The object of this investigation was to investigate the pore-pressure changes of a loaded sample occuring during the strength regain portion of the e-log p curve.

According to the Terzaghi consolidation theory, the dissipation of pore-pressures whithin a loaded soil is given by the relation:[1]

$$T = \frac{c_v t}{H^2}, \quad \text{where} \quad c_v = \frac{k(1+e)}{a_v \gamma_w}$$

This indicates that the pore-pressure dissipation is inversely proportional to a_v, the coefficient of compressibility, which is the slope of the e versus p curve. If the increase in the applied load results in an e versus p curve that is almost horizontal as the grain bond is broken, the value for a_v will be very small. This would mean a very rapid pore-pressure dissipation or practically no excess pore-pressure build up at all, since the void ratio was not expected to change significantly. In either case it was anticipated that the measurement of the pore pressure in this range would be very difficult indeed.

1. All notation and terminology in this paper is consistant with soil mechanics practice as in Taylor, Fundamentals of Soil Mechanics. Wiley & Sons, 1948.

Measurement of Pore-Pressure

In any soil test in the laboratory the measurement of the pore-pressure within a loaded sample is a very tricky proposition. The pore pressure must be measured with a minimum of volume change of water. This means that all lines, valves, and connections must be carefully filled, de-aired and absolutely leak proof.

In this investigation, one of the drains at the base of the lucite consolidometer was sealed and the other was used as a connection for the pore pressure measurement.

Karal Warner Model 50-PP

The first attempt at pore-pressure measurement was with the Karal-Warner Model 50-PP pore-pressure device. The instrument equalizes the pressure from the sample with a 100 psi air source through a regulator. The device is supposed to prevent flow from the sample and read the pressure within it with no volume loss. Unfortunately when the device was tested, there was quite a leak in the regulator valve and it was unable to function properly. This device was not used at all and there was no data to determine whether or not the instrument is at all accurate.

Penman Apparatus

In this device a Bourden gage is operated by the pressure from the expansion of heated oil. The oil heater is turned on and off by a relay which is in turn activated by mercury rising and dropping in a capillary tube as it equalizes the pressure within the sample.

The mercury rises in the tube as the oil is heated until it makes contact with a platinum wire running down the tube. When contact is made, the relay turns off the heater; and the oil cools down, decreases in volume; and the mercury drops in the tube, pushed by the water from the sample. When contact with the platinum wire is broken, the heater is turned on and the mercury rises once more as the oil is heated and it makes contact with the wire again and the heater is turned off. In this manner the cycle continually repeats itself and the pressure within the sample is read with practically no volume change or water loss provided there is no air or leak within the line running from the sample.

When the apparatus is running properly this cycle is about 5 seconds long. The mercury column moves about 2 milimeters in this process which is a volume charge of about 1.5 cubic milimeters which is negligible.

The device is unable to register a rapid change in pressure increments and there was a noticeable time lag between the loading and the indicated pressure. A 100% pore-pressure response was never indicated in the consolidation tests. The best response was less than 90%. This may have been because part of the pore-pressure had dissipated before the instrument was able to build up to an equalizing pressure.

It was necessary to take the whole apparatus apart and refill it with clean water and oil so that it would operate effectively during the testing procedure. Since the fittings were copper and not lucite, it was not possible to make certain there was no air in the system until the test begain. The arcing of the mercury with the platinum wire leaves a burnt spot on the capillary tube making the cycle irregular and the volume changes larger.

Dynisco Transducers

The most successful measurement of the pore-pressure was with Dynisco pressure Transducers. A paper[1] written for presentation at the Paris, 1961 Conference of the International Society of Soil Mechanics and Foundation Engineering describes the use of these devices for pore-pressure measurement and compares their performance with conventional testing methods.

The transducer was set into the consolidometer unit as shown on figure 13a, and its out put was measured with an Autograph X-Y recorder. Since the transducers were needed by other students in the lab, they were not always available. The device shown in figure 13b was designed to facilitate rapid removal and replacement of the transducer to take pressure readings without disturbing the sample. Upon testing, it was found that the adapter leaked and could not be used in

1. Ref. #6

the experiments. It is felt that the device could be made leak proof with closer tolerances and would be suitable.

Due to my unfamiliarity with transducers it was difficult to make absolute readings of pore-pressure. They were very sensitive to the slightest change in applied load. This made it difficult to define a zero point. By calibrating them with a Bourdon Gage it was found that they are not linear through a pressure range. Because of this difficulty in calibration, the percent of pore pressure plotted in these figures is not absolutely accurate but rather an indication of relative magnitudes.

Preparation of Samples

The material tested was a Puget Sound Clay, the same material used by MacLain and Parrish in their theses of 1959. The material has a liquid limit of 100%, a plastic limit of 46%, and a specific gravity of 2.6.

The samples tested were consolidated in two lucite consolidometers. They consist of a base, cylinder, and a piston, 2.75 inches in diameter. They are constructed to allow double drainage and can consolidate a sample up to 4 inches in depth. In these experiments drainage was allowed to occur only at the top and the pore-pressure was measured at the bottom of the sample or at a point where Z/H equals 1.

The cylinder was first filled half way with distilled de-aired water and a vacuum was applied to remove any air bubbles in the drainage line to the valve where the pore-pressure was measured. It was quite difficult to remove all these bubbles and a lot of time was spent in opening the valve, draining water, closing it, and applying a vacuum once more until all the air from the line was removed.

After the bottom stone was in place, the sample was prepared by spooning small amounts into the cylinder half-full of water and applying a vacuum to remove any air from the sample. This process was repeated until the sample was at the desired thickness.

It was evident that there was some separation of particles as the material settled but it could be kept to minimum by decreasing the amount of water in the sample initially. It was felt that the small degree of separation that did occur did not significantly change the characteristics of the sample.

When the top stone was in place the water at the top and the fine particles that seeped around the edges of the stone and settled on the top were removed by rinsing the top with clean, freshly de-aired water. This process eliminated much of the flow of soil around the stone and up between the piston and cylinder, thus greatly reducing the frictional effects during consolidation.

After letting the stone settle on the sample overnight and after connecting the device for pore-pressure measurement the soil was ready to be tested on the consolidation machine.

Testing Procedure

Two regular compression tests were run, doubling the increments, from 1/4 to 16 T/ft^2. In each increment dial readings for change of height and pore-pressure measurements were taken. When the primary condition was complete, as indicated by a zero pore-pressure reading, the next increment was to be added.

After the initial tests, two more were run up to 4 T/ft^2. At this time secondary compression was allowed to occur for almost a month.

At the end of this period one sample was loaded again with 1/2 ton increments up to 8 T/ft^2. For each increment pore-pressure readings were made and the next increment was added when the primary compression was complete.

The other sample was loaded in the same manner to 5 T/ft^2 and secondary compression was allowed to occur at this point for 12 days.

Results

The e vs. log p curves for the compression tests are plotted on figure 1, 2, 3, and 4. The C_c values were computed by the slope of the best straight line through the points for the region of primary compression. They are tabulated in table one.

Two plots were made of the strain versus time and pore pressure dissipation versus time for the 2-4 T/ft^2 increment, one for pore-pressure readings with the Penman device and the other for the readings with the transducer. Note that the pore-pressure is plotted as a percent of the initial response, not the absolute pore-pressure. These plots appear in figures 1 and 2.

Graphs of strain versus time and pore-pressure dissipation for the 1/2 ton increments from 4 to 8 T/ft^2 are plotted on figures 7, 8, 9, and 10. Again the pore-pressure readings are relative.

There were two 1/2 ton increments in the loading sequence of Test 2B also. The data is not presented because the Penman device was not working properly at this time. There was a significant delay before the device builds up to the internal pressure and by this time a good deal of the pressure within the sample had already been dissipated.

Table 2 indicates the method of determining the response and tabulates the peak value for each increment. On figure 11 these peak values are plotted in relation to the pressure increment.

Table 3 tabulates the computed values of the coefficient of consolidation, c_v for the 1/2 ton increments. The calculations on page 35, indicate how these values were determined. These values of c_v are plotted in relation to the pressure increment on Figure 12.

Discussion of Results

Both figure 1 and two for pore-pressure measurement with the transducer and with the Penman device indicate that the end of primary compression is more accurately located by the point of tangency with the asymptote to the secondary compression line than by the intersection of the primary compression line and the secondary compression line.

The average value of .657 for the coefficient of compression, C_c, for primary compression is somewhat less than the average value of .685 MacLain found on working with the same material in 1959[1].

It can readily be seen that there is quite a large scatter of data and a straight line through these points is a gross approximation. The scatter is due to several reasons. One explanation is due to errors in measurement and the fact that the computed height due to measurements with the Ames dial did not agree with the computed value by use of the depth gadge. This is probably the reason for two different computed values of the void ration at a single pressure.

The other explanation concerns the time effects. It can be seen from the tabulated time interval between any two

1. Ref. #2

points that in some cases secondary compression was allowed to occur for a while before the next increment was added. It was not always possible to add another increment at the end of primary compression. In many instances the end of primary compression was not accurately located. Quite often the pore-pressure device would indicate a residual excess pressure for a long time after the break in the dial reading-vs-time curve had occured. Not all pressure increments came out as nicely as the 2-4 Ton/ft^2 increments for Tests 2B and 2C which are presented in figures 1 and 2.

It is possible that friction between the walls of the piston and the cylinder may explain some of these scattering effects. However, upon disassembling at the end of a loading sequence, the piston seemed to move quite freely and it is thought that friction was not a significant factor in these tests.

The C_c value for the compression after the secondary compression had been allowed to occur for a month was about .180, roughly a third of the value for the primary compression range.

In figure 6, test 2C, the hump that Terzaghi described that occurs as the film bond of the particles is broken can be observed. It was assumed that at the end of the eight ton load any bond would have been broken and that the void ratio at the end of the sixteen ton increment would have fallen on the original straight line of virgin compression. Unfortunately, it was not near the original line at all and it is difficult to explain why unless the particle bond was not completely broken at the time of loading this increment.

In test 2B, figure 5, it was expected that point m, the void ratio after a long period of secondary compression from 5 T/ft^2, would fall on a line with point j, the void ratio after prolonged secondary compression from 4 T/ft^2, parallel to the initial curve. This phenomena was not observed. If there had been enough time another test would have been conducted to see if this would occur.

For the half-ton increment loading it was expected that the pore-pressure response would not be a hundred percent because the soil skeleton would be able to take some of the stress due to the bonding of the particles. It was expected that as the bond was broken due to successive loading increments, that the initial pore-pressure response would increase. Table 2 and figure 11 indicate that this increase was not detected during this testing sequence.

In consolidation test 2C the transducer behaved erratically for the first two increments. It was removed and heated in the oven for 5 1/2 hours to drive off any moisture within it that made it behave in this fashion. After its replacement it worked very well for the succeeding increments. This explains the large time lag between the 4 1/2 to 5 ton increment and the 5 to 5 1/2 ton increment, (fig 6 and 7).

As was expected the coefficient of consolidation, C_v decreased , (fig. 12), approaching the value for the 4-8 T/ft^2 increment with no previous secondary compression. This phenomena occurs because as the bond between the particles is broken, the material becomes more compressible, as it takes longer for primary compression to occur.

Conclusions

A clay loaded in small load increments after secondary compression has occured for a long period of time will show a very rapid build up and dissipation of pore-pressures compared to the build up and dissipation for the primary compression region. The initial response is only about 2/3 of the pressure increment. With each successive increment as the particle bond breaks, the coefficient of consolidation decreases as the time for 100% dissipation increases. As this particle bond is broken, c_v approaches the value for pore-pressure dissipation for the loaded material with no previous secondary compression.

Pore Pressure Measurement

With a great deal of care accurate pore-pressure readings can be made with a Penman device. The apparatus cannot respond rapidly to changes in the loading increment. Pore-pressure measurement with Transducers is easier and with a little care in calibration, very accurate. These elements have a very rapid response to pressure changes and are well suited to measuring pore-pressure response where the load is changed rapidly.

Results

Table One

Coefficient of Compression

$$C_c = \frac{\Delta e}{\Delta \log P}$$

Cons. Test 1A	C_c = .609
Cons. Test 1B	C_c = .623
Cons. Test 2B	C_c = .609
Cons. Test 2C	C_c = .790
average	C_c = .657

For Secondary Compression Zone

Cons Test 2B	C_c = .178
Cons Test 2C	C_c = .222
average	C_c = .200

Table two

Pore Pressure Response For Loading Increments

Cons Test 2C 1/2 Ton Increments 4-8 Tons/ft^2

Increment	Percent Response
4 - 4 1/2	80.6
4 1/2 - 5	74.8
5 - 5 1/2	58.3
5 1/2 - 6	62.4
6 - 6 1/2	61.4
6 1/2 - 7	69.0
7 - 7 1/2	61.8
7 1/2 - 8	67.0

See Graph, page 32

Note that these values are not absolute, but rather an indication of relative values proportioned from the output psi relation for the transducer.

Table three

Coefficient of Consolidation, c_v

Cons Test 2C   1/2 Ton increments   4 - 8 Tons/ft^2

Increments		c_v $\times 10^{-2}$
4 - 4 1/2	.117 in^2/min	1.25 cm^2/sec
4 1/2 - 5	.083 in^2/min	.89 cm^2/sec
5 - 5 1/2	.084 in^2/min	.90 cm^2/sec
5 1/2 - 6	.058 in^2/min	.62 cm^2/sec
6 - 6 1/2	.045 in^2/min	.48 cm^2/sec
6 1/2 - 7	.042 in^2/min	.45 cm^2/sec
7 - 7 1/2	.041 in^2/min	.44 cm^2/sec
7 1/2 - 8	.024 in^2/min	.26 cm^2/sec

See Fig. 12, page 33

The method used in the calculations of these values is outlined on page 35.

(Penman Device)

Figure 1

Strain and Pore Pressure vs Time

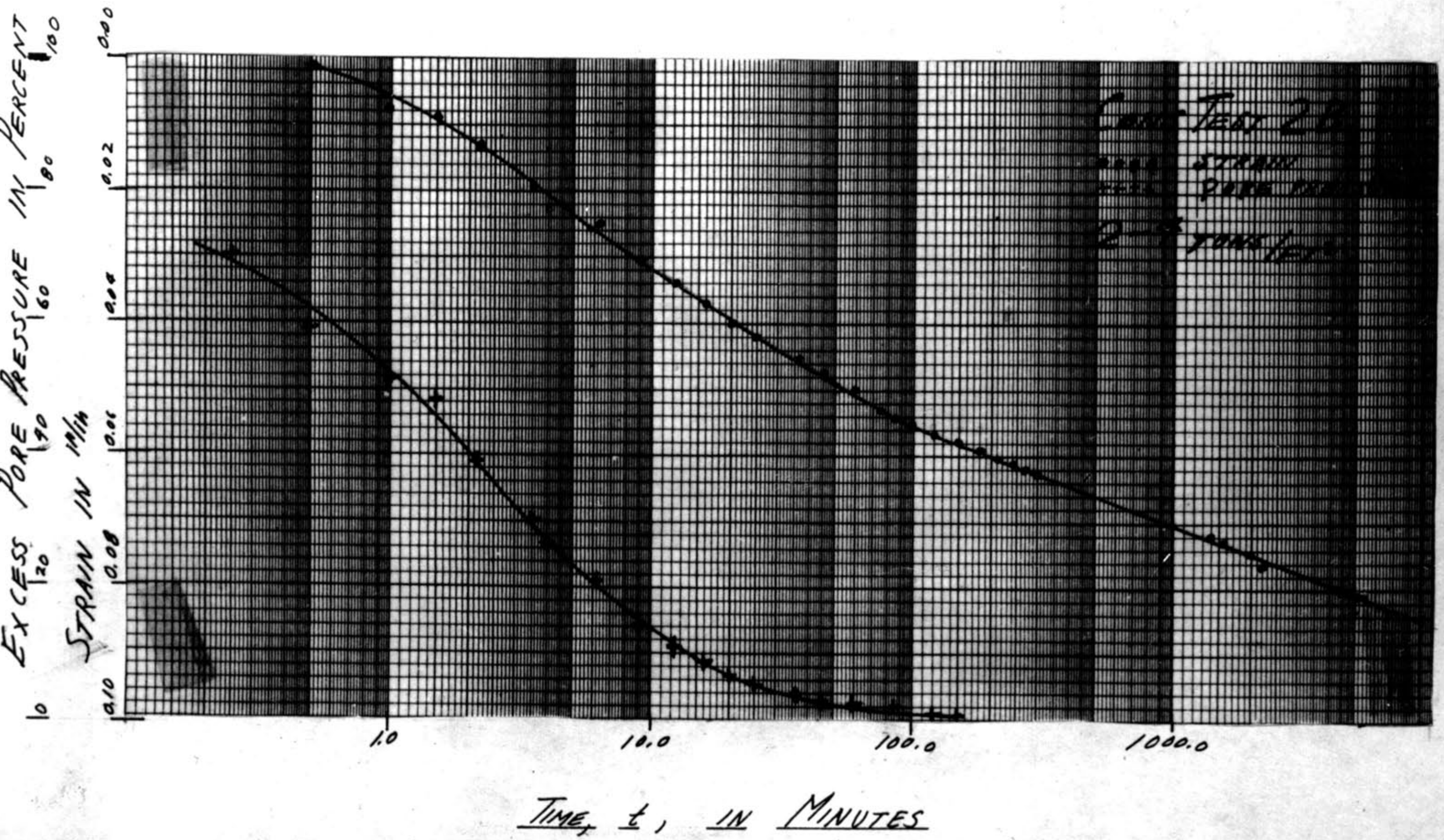
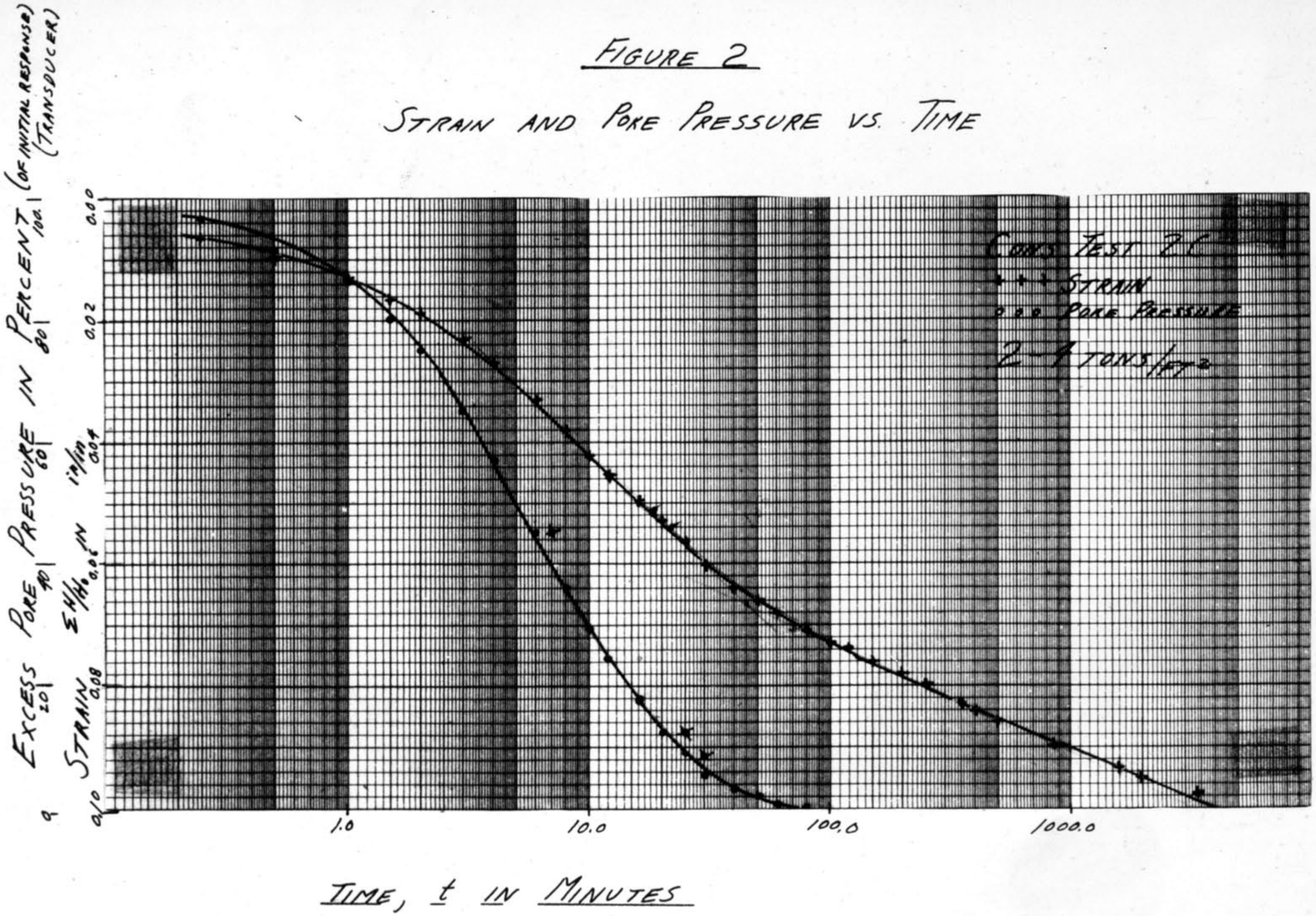
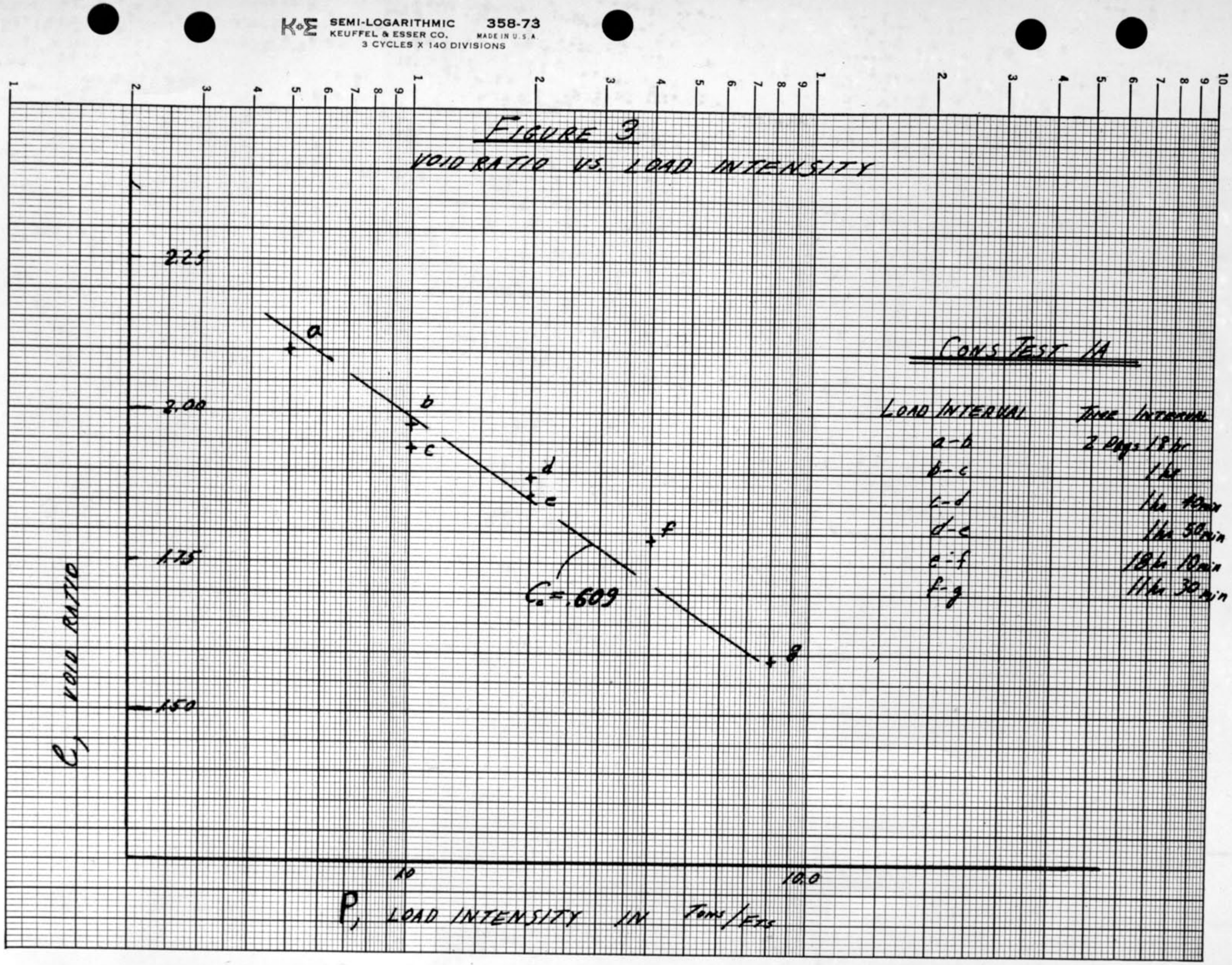


FIGURE 2

STRAIN AND PORE PRESSURE VS. TIME




K+E SEMI-LOGARITHMIC 358-73
KEUFFEL & ESSER CO. MADE IN U.S.A.
3 CYCLES X 140 DIVISIONS
FIGURE 3
VOID RATIO VS. LOAD INTENSITY
2.25
2.00
1.75
1.50
e, VOID RATIO
a
b
c
d
e
f
g
C_c = .609
CONS TEST 1A
LOAD INTERVAL TIME INTERVAL
a-b 2 Days 18 hr
b-c 1 hr
c-d 1 hr 40 min
d-e 1 hr 50 min
e-f 18 hr 10 min
f-g 11 hr 30 min
1.0
10.0
P, LOAD INTENSITY IN Tons/Ft²

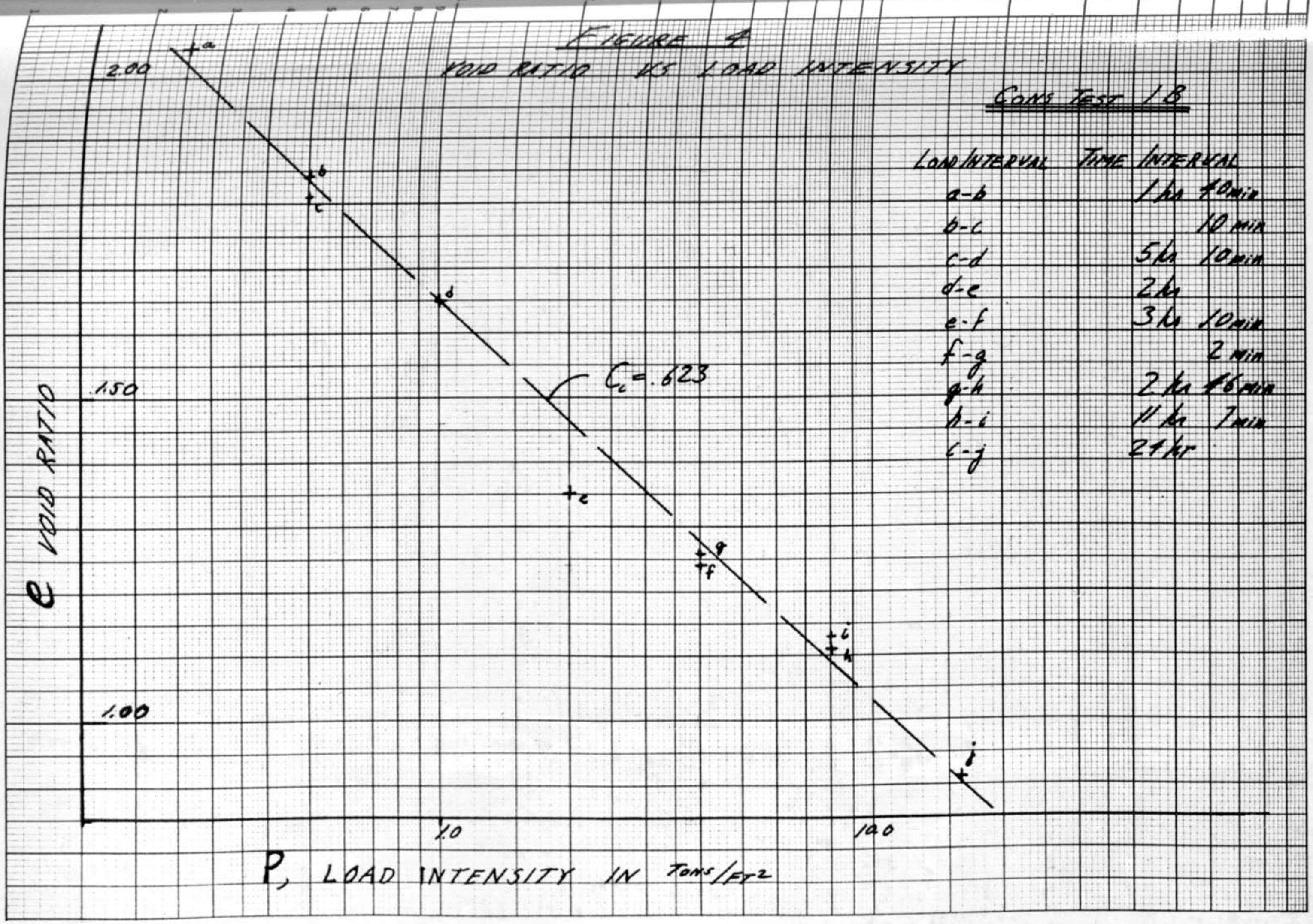


FIGURE 4
VOID RATIO VS LOAD INTENSITY
CONS TEST 1B
LOAD INTERVAL TIME INTERVAL
a-b 1 hr 40 min
b-c 10 min
c-d 5 hr 10 min
d-e 2 hr
e-f 3 hr 10 min
f-g 2 min
g-h 2 hr 46 min
h-i 11 hr 7 min
i-j 24 hr
C_c = .623
2.00
1.50
1.00
e VOID RATIO
1.0
10.0
P, LOAD INTENSITY IN TONS/FT^2
a
b
c
d
e
f
g
h
i
j

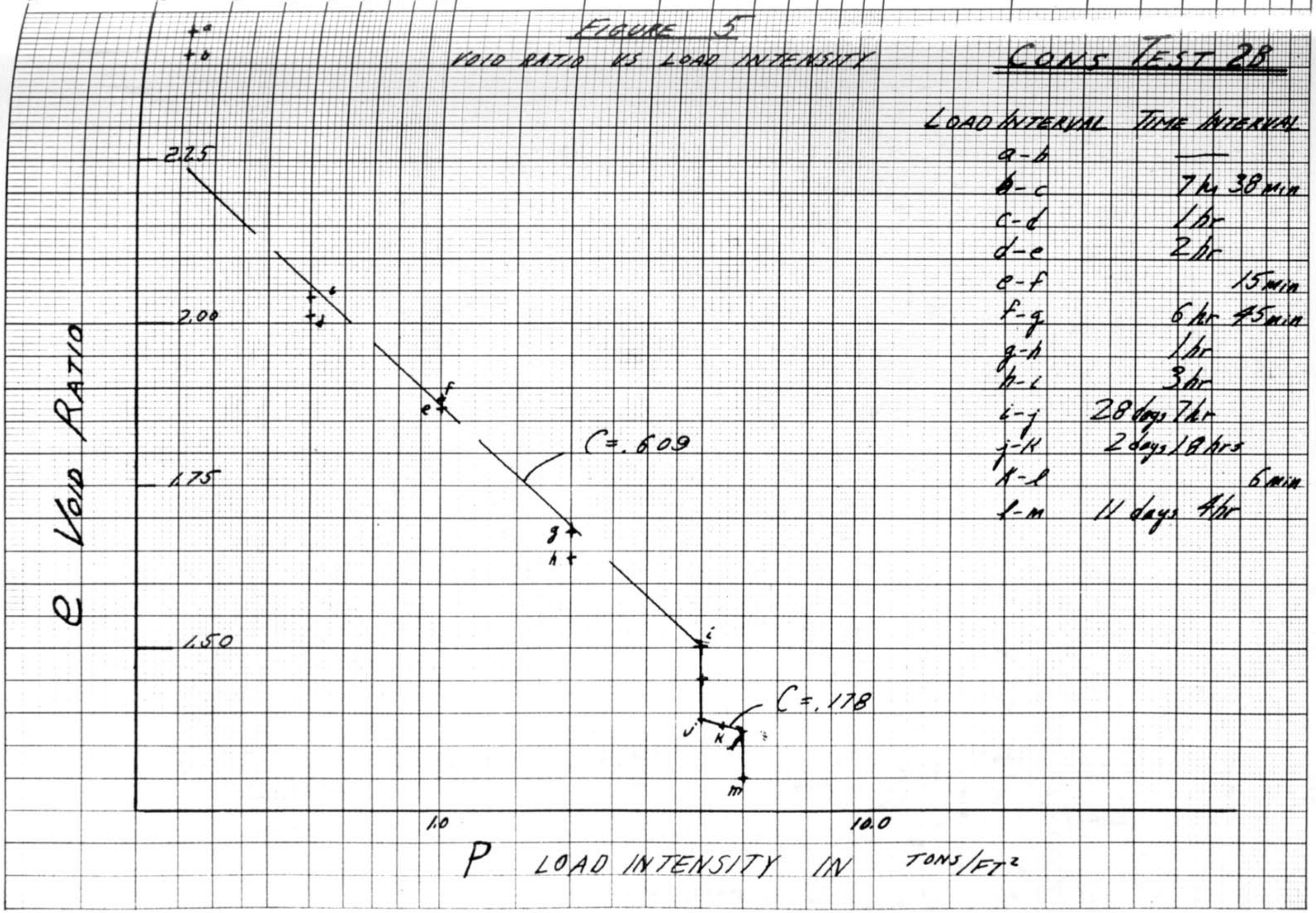
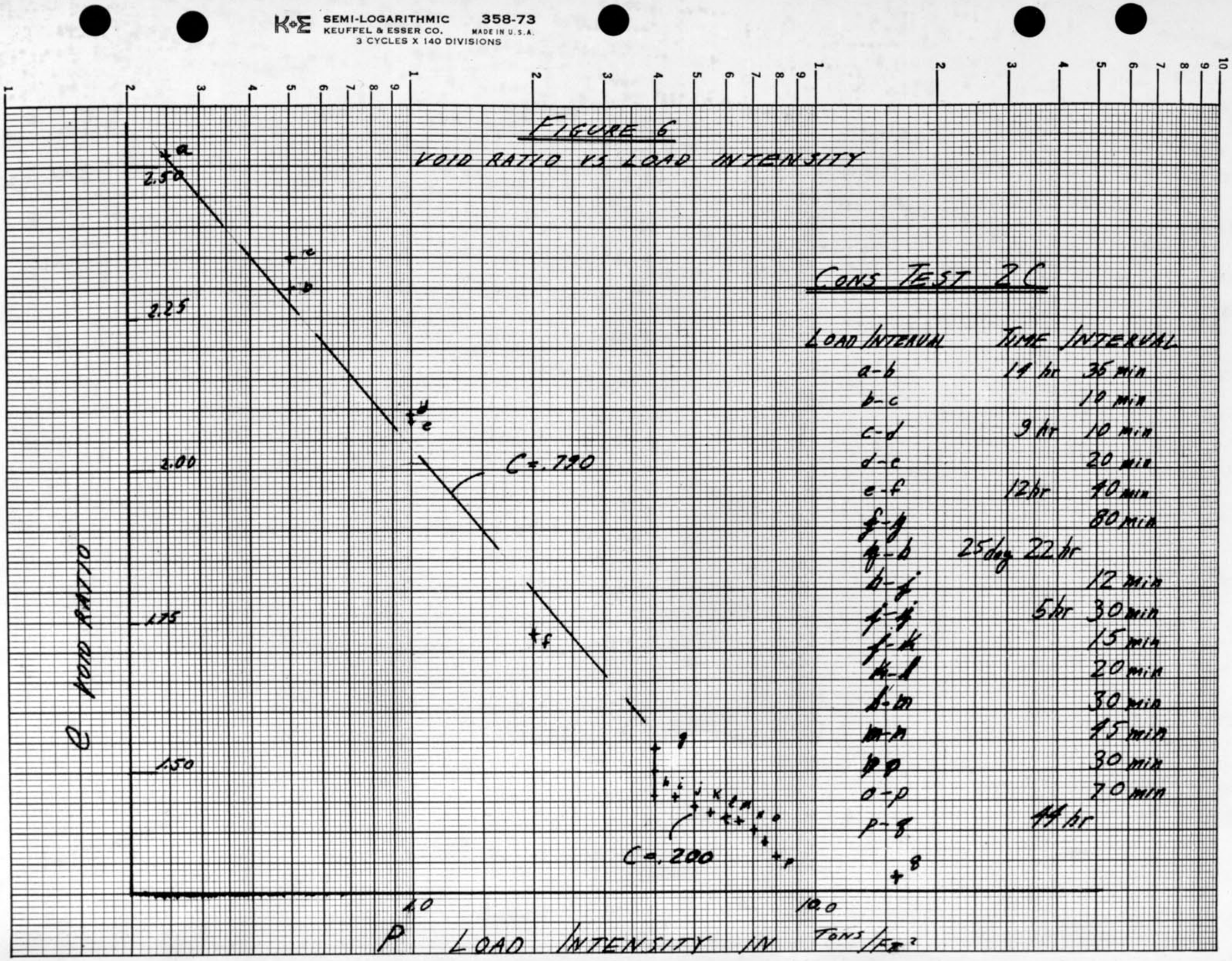
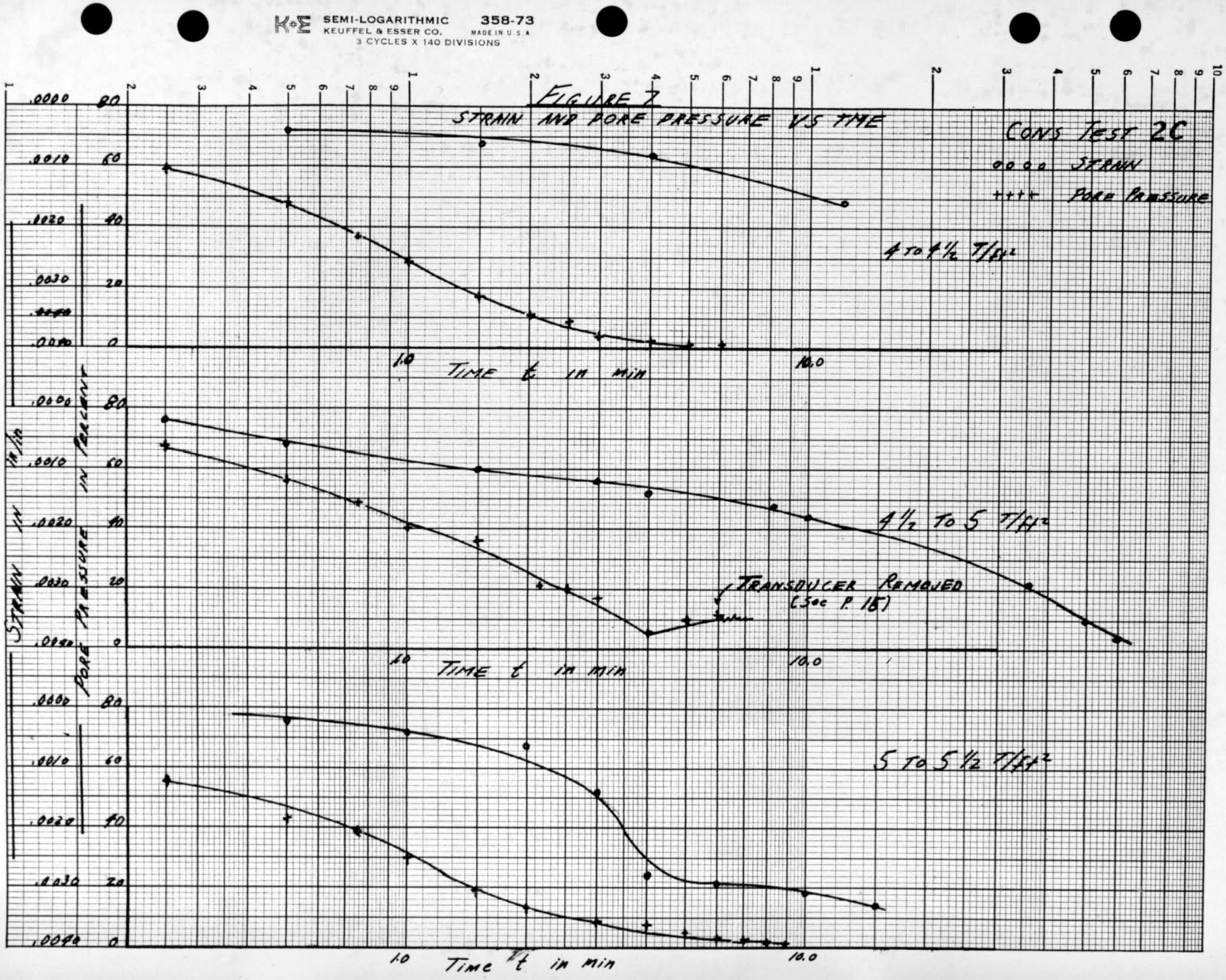


FIGURE 5
VOID RATIO VS LOAD INTENSITY
CONS TEST 28
LOAD INTERVAL TIME INTERVAL
a-b —
b-c 7 hr 38 min
c-d 1 hr
d-e 2 hr
e-f 15 min
f-g 6 hr 45 min
g-h 1 hr
h-i 3 hr
i-j 28 days 7 hr
j-k 2 days 18 hrs
k-l 6 min
l-m 11 days 4 hr
2.25
2.00
1.75
1.50
e VOID RATIO
C=.609
C=.178
1.0
10.0
P LOAD INTENSITY IN TONS/FT²


K•E SEMI-LOGARITHMIC 358-73
KEUFFEL & ESSER CO. MADE IN U.S.A.
3 CYCLES X 140 DIVISIONS
FIGURE 6
VOID RATIO VS LOAD INTENSITY
CONS TEST 2C
LOAD INTERVAL TIME INTERVAL
a-b 11 hr 35 min
b-c 10 min
c-d 9 hr 10 min
d-e 20 min
e-f 12 hr 40 min
f-g 80 min
g-h 25 day 22 hr
h-i 12 min
i-j 5 hr 30 min
j-k 15 min
k-l 20 min
l-m 30 min
m-n 45 min
n-o 30 min
o-p 70 min
p-q 11 hr
C=.790
C=.200
2.50
2.25
2.00
1.75
1.50
e VOID RATIO
1.0
10.0
P LOAD INTENSITY IN TONS/FT²



K+E SEMI-LOGARITHMIC 358-73
KEUFFEL & ESSER CO. MADE IN U.S.A.
3 CYCLES X 140 DIVISIONS
FIGURE 7
STRAIN AND PORE PRESSURE VS TIME
CONS TEST 2C
oooo STRAIN
++++ PORE PRESSURE
4 TO 4½ T/ft²
4½ TO 5 T/ft²
5 TO 5½ T/ft²
TRANSDUCER REMOVED (See P. 15)
TIME t IN MIN
STRAIN IN IN/IN
PORE PRESSURE IN PERCENT
1.0
10.0

-28-

-62-

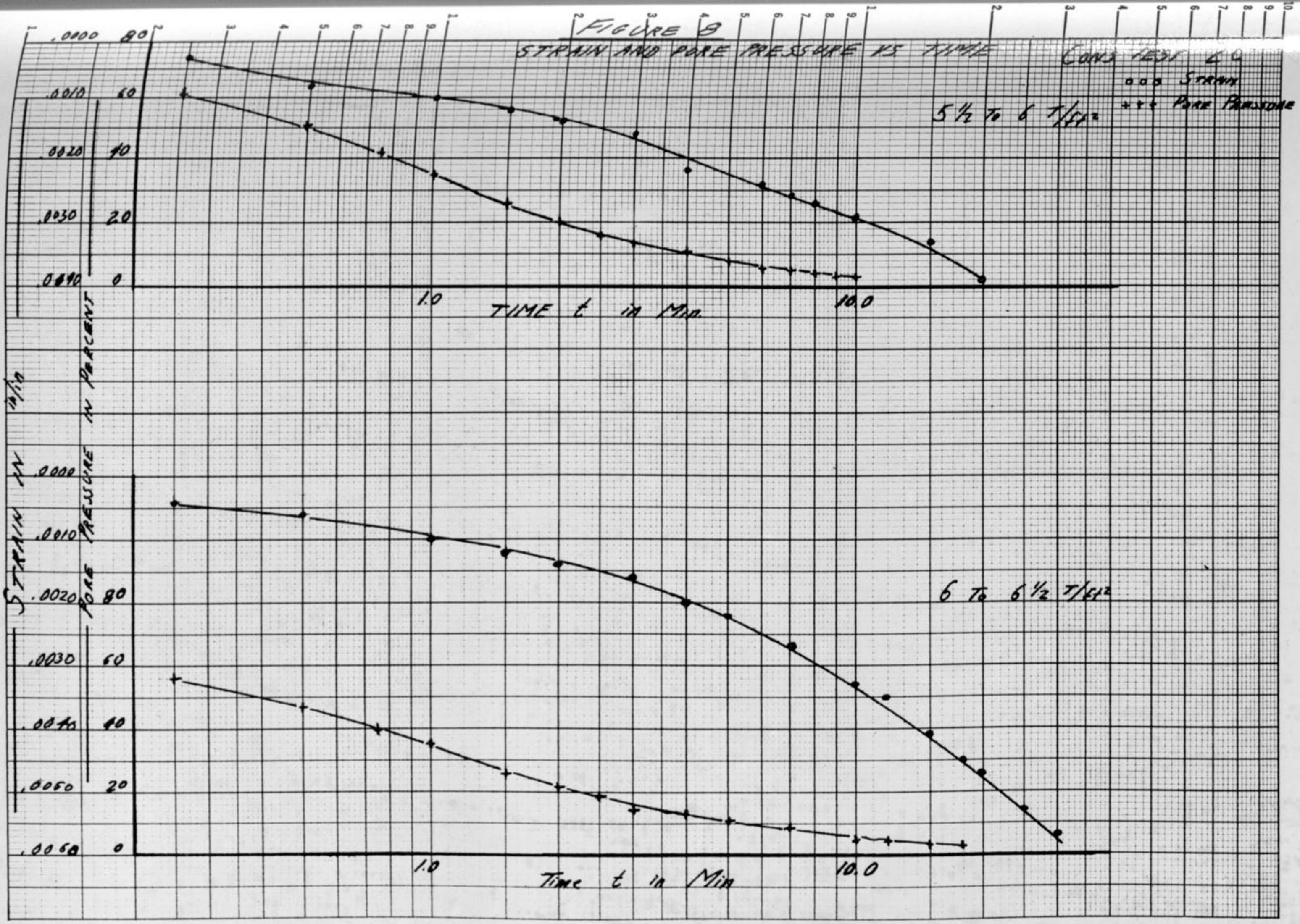


FIGURE 9
STRAIN AND PORE PRESSURE VS TIME
CONS TEST
ooo STRAIN
+++ PORE PRESSURE
5½ To 6 T/ft²
6 To 6½ T/ft²
STRAIN IN in/in
PORE PRESSURE IN PERCENT
.0000
.0010
.0020
.0030
.0040
.0050
.0060
80
60
40
20
0
1.0
10.0
TIME t in Min.
Time t in Min

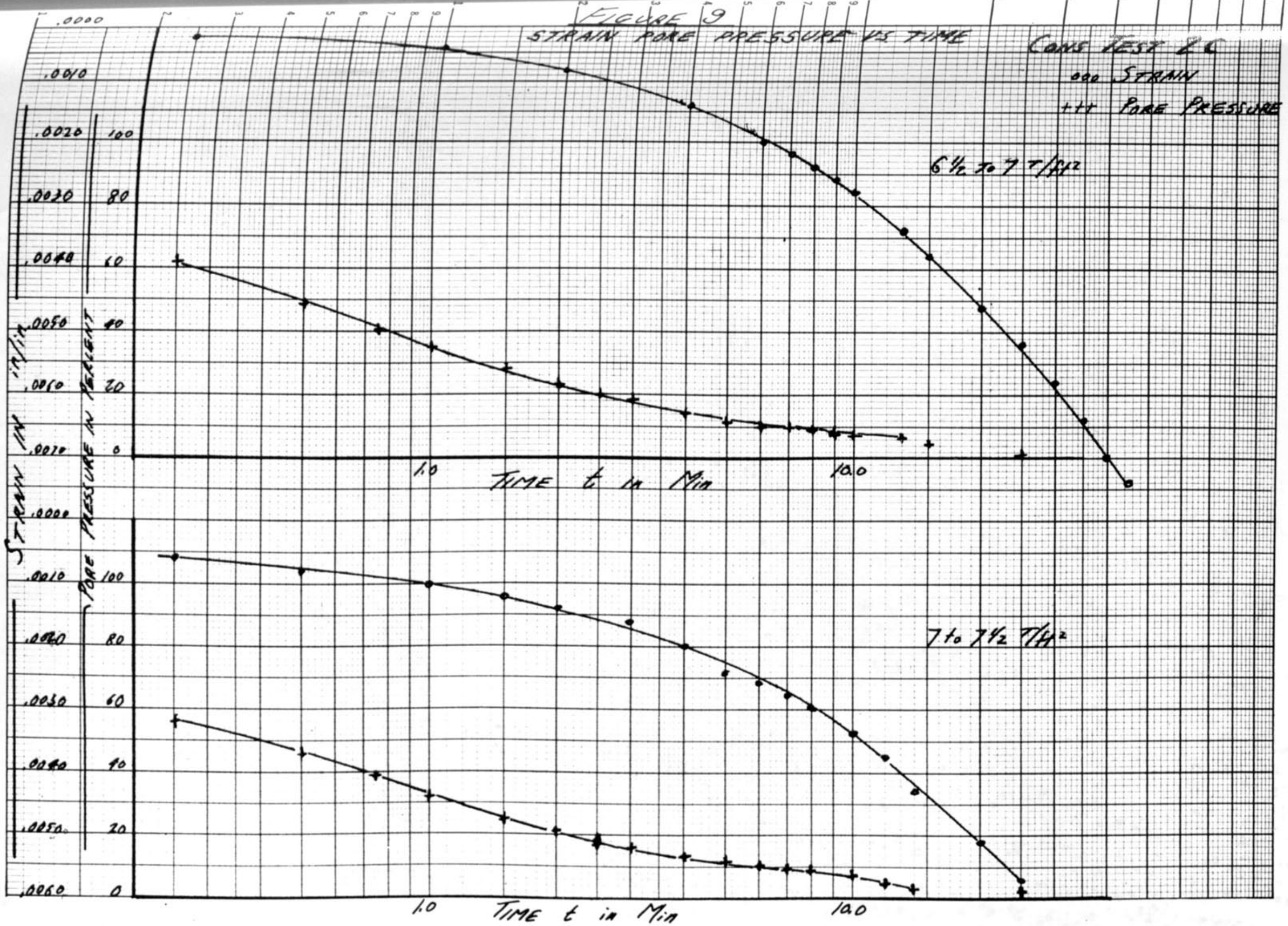


FIGURE 9
STRAIN PORE PRESSURE VS TIME
CONS TEST 2C
ooo STRAIN
+++ PORE PRESSURE
6 1/2 to 7 T/ft²
7 to 7 1/2 T/ft²
STRAIN IN in/in
PORE PRESSURE IN PERCENT
TIME t in Min
1.0
10.0
.0000
.0010
.0020
.0030
.0040
.0050
.0060
.0070
100
80
60
40
20
0

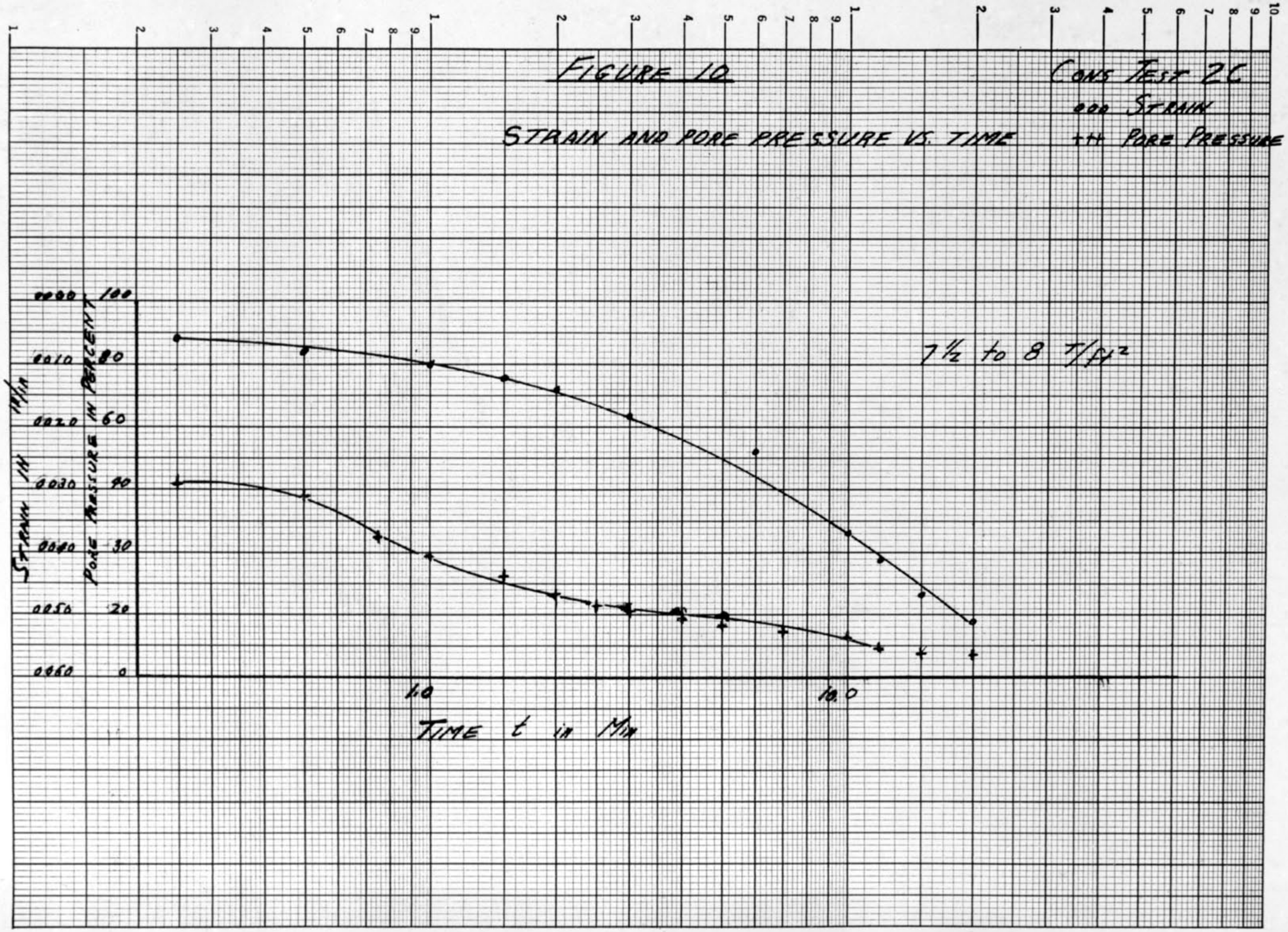
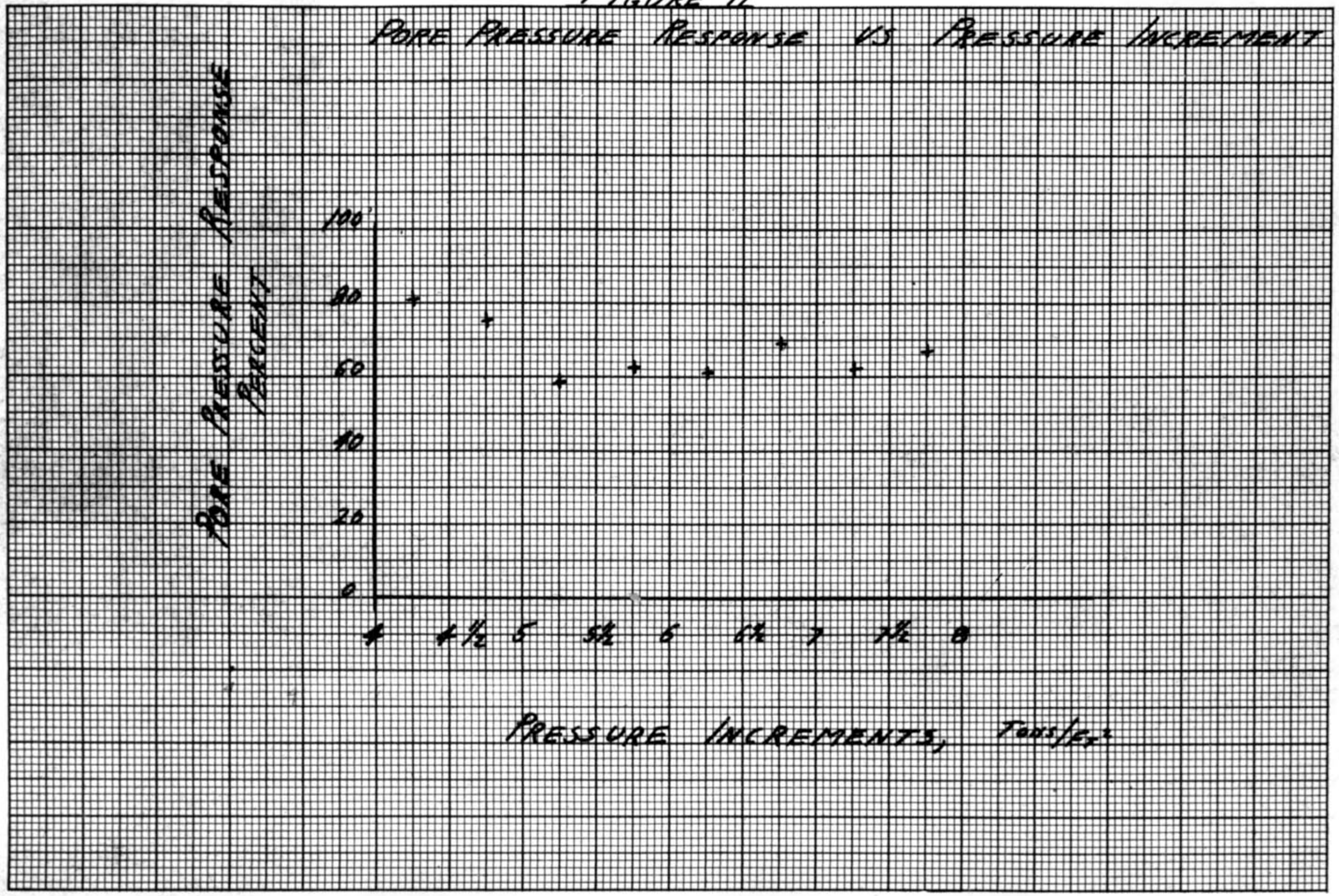


FIGURE 10
STRAIN AND PORE PRESSURE VS. TIME
CONS TEST 2C
••• STRAIN
+++ PORE PRESSURE
7½ to 8 T/ft²
STRAIN IN IN/IN
PORE PRESSURE IN PERCENT
.0000 100
.0010 80
.0020 60
.0030 40
.0040 30
.0050 20
.0060 0
1.0
10.0
TIME t in MIN

K+E 10 X 10 TO THE ½ INCH KEUFFEL & ESSER CO. 359-11½G MADE IN U.S.A.

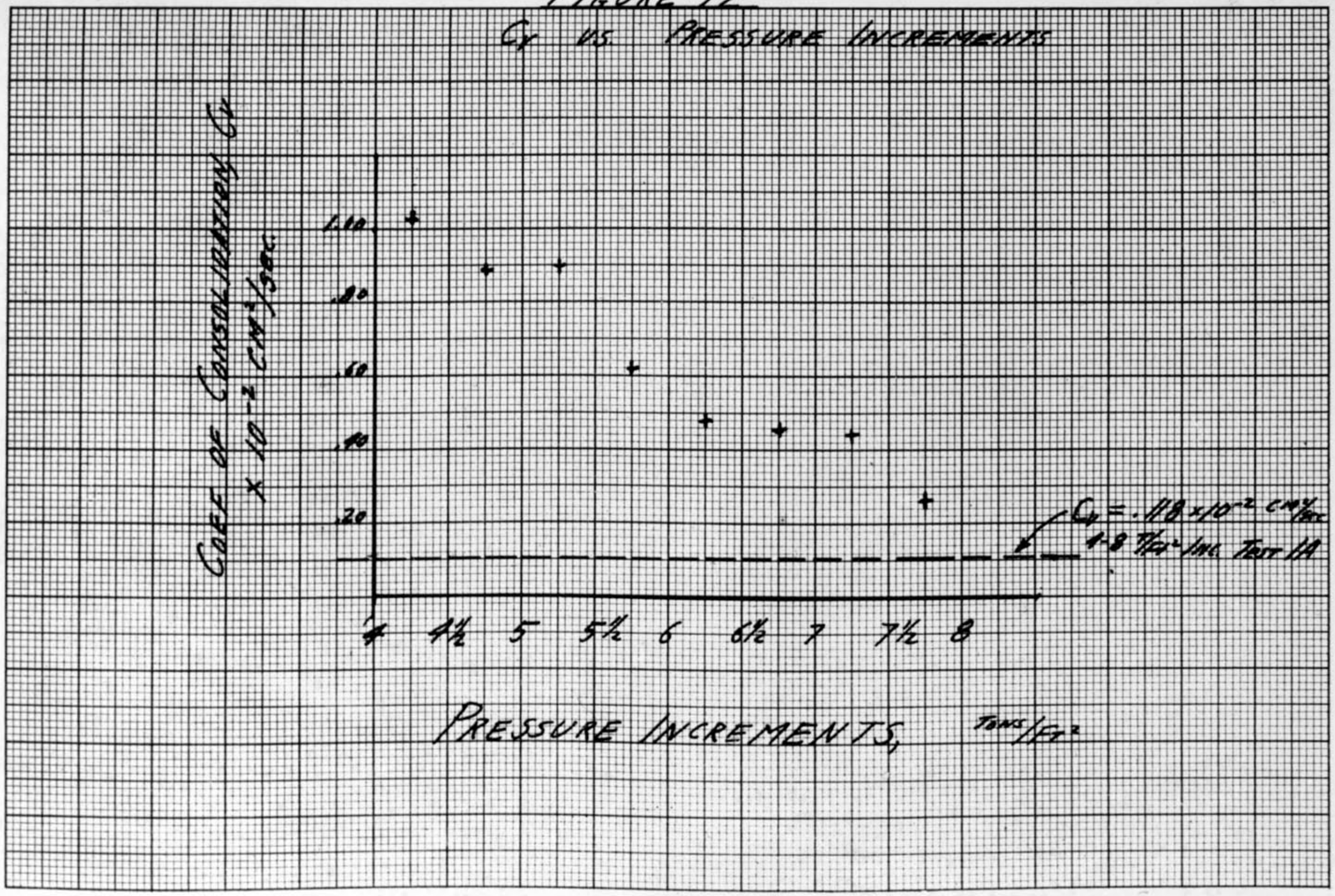
CONS TEST 2C

FIGURE 11



-32-

FIGURE 12


Cv VS. PRESSURE INCREMENTS
COEF OF CONSOLIDATION, Cv
× 10^-2 CM²/SEC.
1.00
.80
.60
.40
.20
Cv = .118 × 10^-2 CM²/SEC
4-8 T/FT² INC. TEST 1A
4
4½
5
5½
6
6½
7
7½
8
PRESSURE INCREMENTS, TONS/FT²

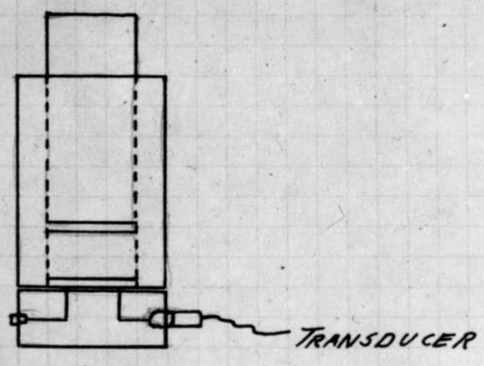


Fig 13 a
TRANSDUCER SET IN CONSOLIDOMETER

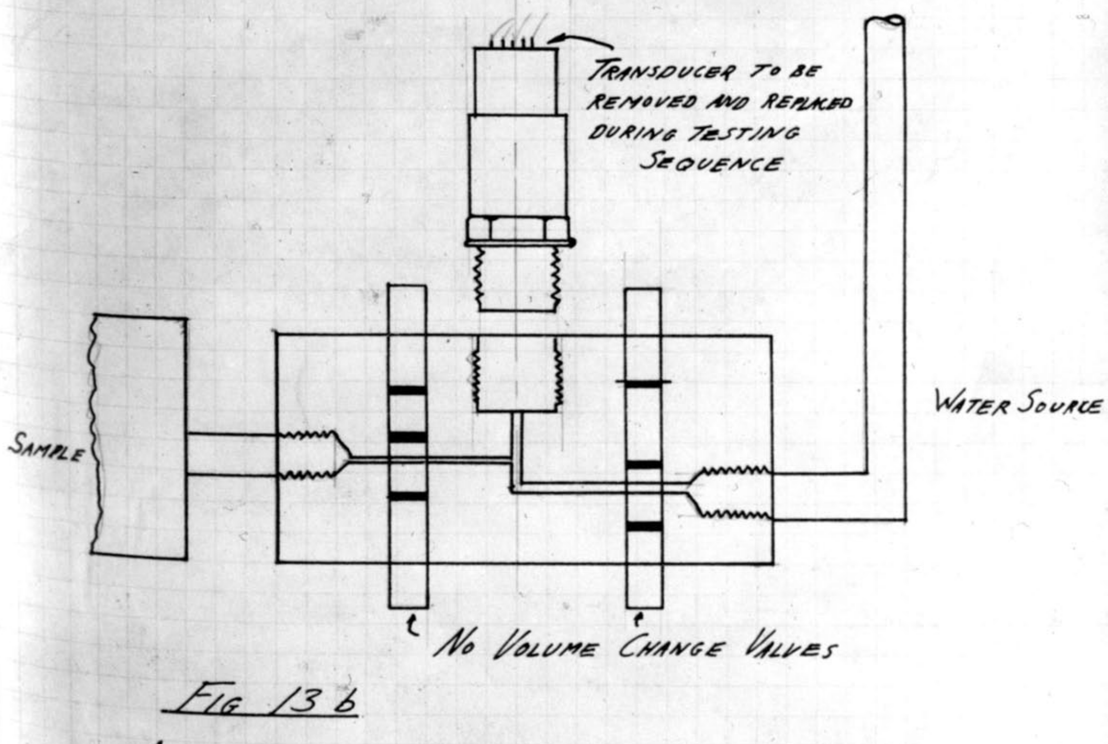


Fig 13 b

ADAPTER FOR REMOVAL AND REPLACEMENT OF
TRANSDUCER

Calculation of c_v

Coefficient of Consolidation

$$T = \frac{c_v t}{H^2} \qquad c_v = \frac{H^2 T}{t}$$

H; Average height of sample

T: Time Factor, From Fundamentals of Soil Mechanics, Taylor, Fig. 10.9: for 80% consolidation for Z/H equal to 1

$T = 0.75$

t: time in minutes

Sample Calculation for c_v:

Cons. Test 1A, 4-8 Ton/ft^2 increment

$$\left.\begin{array}{l} H_o = .472 \\ H_{80} = .437 \end{array}\right\} \quad 0.454 \text{ in}$$

t_{80} = 14 min

$$c_v = \frac{(.454)^2 \times .75}{14} = .0110 \text{ in}^2/\text{min} = 0.118 \times 10^{-2} \text{ cm}^2/\text{sec}$$

The c_v values for the 1/2 ton increments in test 2C were calculated in the same manner and they are tabulated in Table two and plotted in Fig. 12.

References

1. Christian, John T. Thixotropic Characteristics of a Laurentian Clay, Massachusetts Institute of Technology August, 1959

2. MacLain, Dennis E. The Effect of the Rate of Loading on the Secondary Compression Behaviour of a Puget Sound Clay Massachusetts Institute of Technology May, 1959

3. Parrish, Donald R. The Effect of Sample Thickness on Secondary Compression, Massachusetts Institute of Technology, June, 1959.

4. Taylor, Donald W. Fundamentals of Soil Mechanics, John Wiley and Sons, Inc., New York, 1948

5. Terzaghi, Karl, Undistnnbed Clay Samples and Undisturbed Clays. Journal of the Boston Society of Civil Engineers, July, 1941.

6. Whitman, R. V., Richardson, A. M., Healy, K. A. Time Lags in Pore Pressure Measurements. Division I, Fifth International Conference of Soil Mechanics and Foundation Engineering. Paris, 1961