HIGH HONORS PROJ.

"VERIFICATION OF TERZAGHI'S THEORY OF CONSOLIDATION BY CENTRIFUGAL TESTING"

Devo Seereeram

Spring Semester 1982

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University of Florida

Special thanks to Dr. Frank Townsend for his technical advisement, and to Mr. David Bloomquist for his supervision and assistance during the centrifugal tests.

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"There is no way to study the consolidation of kaolinite without getting your fingers messy and ruining a few T-shirts."

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ABSTRACT

This is a comparative study between consolidation testing under increased acceleration fields produced in a centrifuge, and the standard consolidation tests performed in the laboratory. The standard laboratory test samples are only subjected to the earth's gravitational field (i.e. 1 g), but the model that is placed in the centrifuge is subjected to an acceleration field much greater than the acceleration due to gravity on earth (maximum capability of University of Florida's soil testing centrifuge = 85 g's). Terzaghi's theory of consolidation is used as the basis for comparing the results of the centrifugal tests and the standard tests. The dimensional similitude between the centrifugal model and the standard sample depends largely on the validity of Terzaghi's theory. The results of this report would hopefully provide a good indication of the validity of the classical theory of consolidation (as proposed by Terzhaghi), and also examine the accuracy of the "modelling" factors used in relating the models under different acceleration fields.

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INTRODUCTION

Every process involving a decrease of the water content of a saturated soil without replacement of the water by air is called a "process of consolidation". The opposite process is called a "process of swelling", which involves an increase of the water content due to an increase of the volume of voids.[1] There are three main reasons for this time dependent change in volume (settlement or heave): 1) free water leaves the pores over a long term, 2) particles slip into denser configuration and 3) elastic bending of clay plates. This type of behavior does not occur in coarse grained (larger than #200 sieve) soils such as sands since the permeabilities are too high (generally larger than 10-5 cm/sec), and therefore pore water present would be quickly dissipated without creating any significant pore water pressures. Consolidation is directly related to the rate at which this pore water is dissipated, and since clay permeabilities are very low, the time taken for this water to seep out of the clay structure is considerable when compared to the actual duration of a construction project. It should be noted that settlement in sands is immediate, and can be estimated from elastic theory, plate load tests, and correlations with cone penetronometers or the standard penetration test.

Several parameters have been developed for use in consolidation analysis and the prediction of field settlements. The coefficient of compressibility A_v, is the slope of a void ratio (e) vs overburden pressure (p) curve. The compression index, C_c, is similar to the parameter, A_v, except the overburden pressure is plotted on a log scale

[1]Theoretical Soil Mechanics, Terzaghi, p. 265

and results in a linear relationship. The value of the compression index is therefore constant, and more useful than the coefficient of compressibility, A_v, in settlement computations. The other important parameter is the coefficient of consolidation, C_v; this value aids in the estimation of the rate of settlement.

The foundation of a structure transmits the dead load and live load of the structure to the underlying soil within the pressure bulb of that particular foundation. When the clay layers within the affected areas are subjected to this increased stress, the process of consolidation begins and the stability of the structure as a unit now depends on the subsequent settlement of the clay layers. Settlement can be important even though no rupture is imminent for three reasons: appearance of the structure, utility of the structure and damage to the structure.[2] Settlement can detract from the appearance of the building by causing cracks in exterior masonry walls and/or the interior plaster walls. It can also cause a structure to tilt enough for the tilt to be detected by the human eye (e.g. Leaning Tower of Pisa.) Settlement can interfere with the function of a structure in a number of ways, e.g. cranes and other such equipment may not operate properly; pumps, compressors, etc. may get out of line, and tracking units such as radar become inaccurate. Settlement can cause a structure to fail structurally and collapse even though the factor of safety against a shear failure in the foundation is high. A rigid mat normally undergoes a uniform settlement, but there is also the case of uniform tilt in which the entire structure rotates. There is also a type of non uniform settlement known as "dishing", where

[2]Soil Mechanics, Lambe & Whitman, p. 199

the center of the structure settles more than the edges thus producing a "dish" shaped settlement. The amount of settlement a structure can tolerate, i.e. the allowable or permissible settlement, depends on many factors including the type, size, location and the intended use of the structure, and the pattern, rate, cause and source of settlement.[3] In many instances, the total settlement is not important (e.g. storage tank with flexible connections), but the differential settlement may be crucial. For example, two closely spaced footings may settle 12 in. and 18 in. respectively, this differential settlement of 6 in. may render stresses within the superstructure beyond the allowable limits. However, if both footings settled uniformly, no additional stresses would be induced in the structure even though it may lie some distance below the level originally intended.

[3]Soil Mechanics, Lambe and Whitman, p. 199

CENTRIFUGE MODEL TESTING OF SOILS

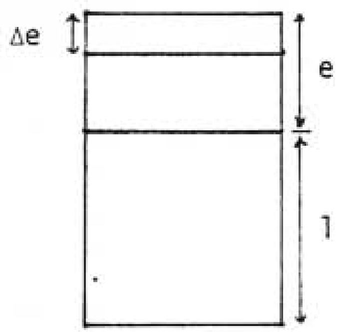
In a material as heterogeneous and variable as a clay, it is not possible to develop exact theories on the behavior of soil. It is common practice in many civil engineering fields such as hydraulics, aerodynamics and structures, to build models to examine and compare actual behavior with theoretical predictions. If the model testing is satisfactory, the results can be applied to the prototype with a successful correlation to the predicted results. In most instances, it is not possible to exactly duplicate an actual situation with a model, so it may be necessary to build an approximate model and then apply correction factors to the prototype behavior.

There are two general categories of soil model testing. The first consists of situations in which the boundary forces are the main contributors to the soil deformations, example: footings, retaining walls and piles. The second type involves a condition where the self weight of the soil is significant and the boundary forces are negligible. An example of such a case would be in the analysis of slope stability.

The tests performed in this report are due to boundary forces and examines consolidation in clay. The situation modelled is the settlement of a footing resting on a clay layer. An acceleration field much larger than the acceleration due to gravity is created within the centrifuge, and the strains created in the model due to various known surcharges are recorded.. This raw data collected is similar to that collected in a regular consolidation test; however, the normal consolidation test represents a test on a sample from which the various parameters are obtained and used to predict settlement and the rate of settlement in the field. The centrifugal test, in actuality, is a test on a real situation in the field since the self weight of the soil is increased to duplicate field stresses. Centrifugal testing provides a linear relationship with the strains in the model, but time, which is a very important parameter, is not so simply correlated. The theoretical relationship between the actual recorded time and the time effects experienced by the sample in the centrifuge is a square of the value of prototype/model.

PRINCIPLES OF CLASSICAL THEORY OF CONSOLIDATION

Consider the following phase diagram for a saturated soil,



e = void ratio = volume of voids/volume of soil

In this saturated sample, all of the voids are filled with water and any compression of this soil must be accompanied by flow of water out of the sample. If an added stress is imposed upon this soil sample in the field, the particles get closer together as the void ratio decreases by an amount Δe.

Therefore, the settlement of a layer of soil (clay) H feet high produces a settlement of magnitude ΔH; this can be represented by the equation below:

$$\frac{\Delta H}{H} = \frac{\Delta e}{1 + e}$$

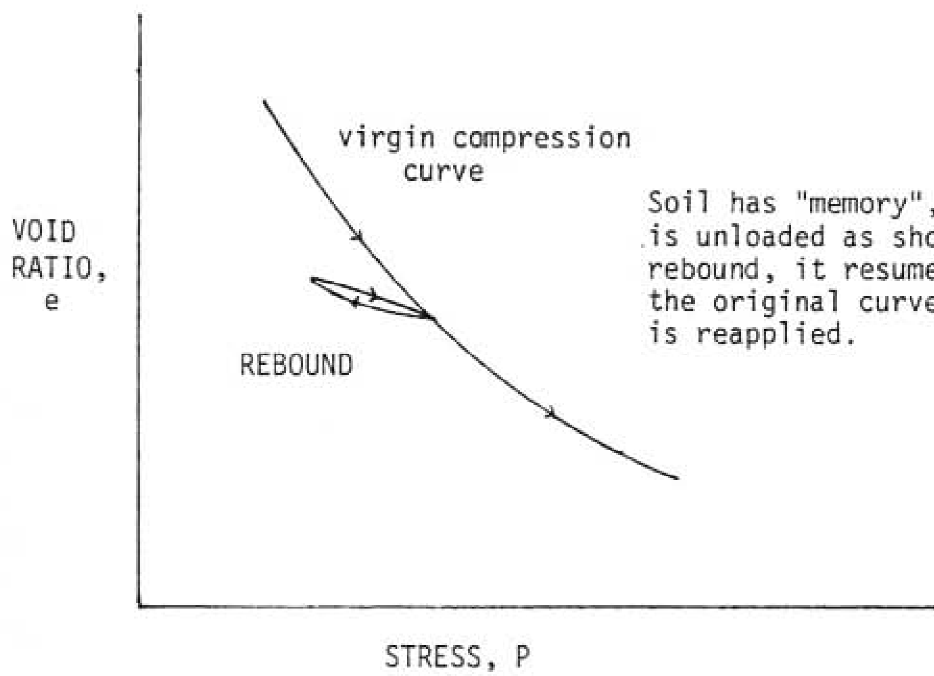
This shows that the settlement can be related to the change in void ratio. Since clay particles are very small, the voids within the particles are also very small. The flow of water through these minute voids is not rapid enough to create an immediate effect on the settlement of the clay. Consolidation is then a study of this time dependent nature of settlement.

The pressure acting on the soil sample also determines the extent to which the sample is going to settle. For any given clay sample, the pressure and corresponding void ratio can be correlated.

Several methods have been devised to compute pressures at various points in a soil layer due to an external load such as a footing. ex. Boussinesq equations for various footing shapes.

Since the pressures can be estimated with reasonable accuracy, it would be advantageous to use parameters of the soil that relate void ratio to the applied pressures.

TYPICAL VOID RATIO vs STRESS CURVE

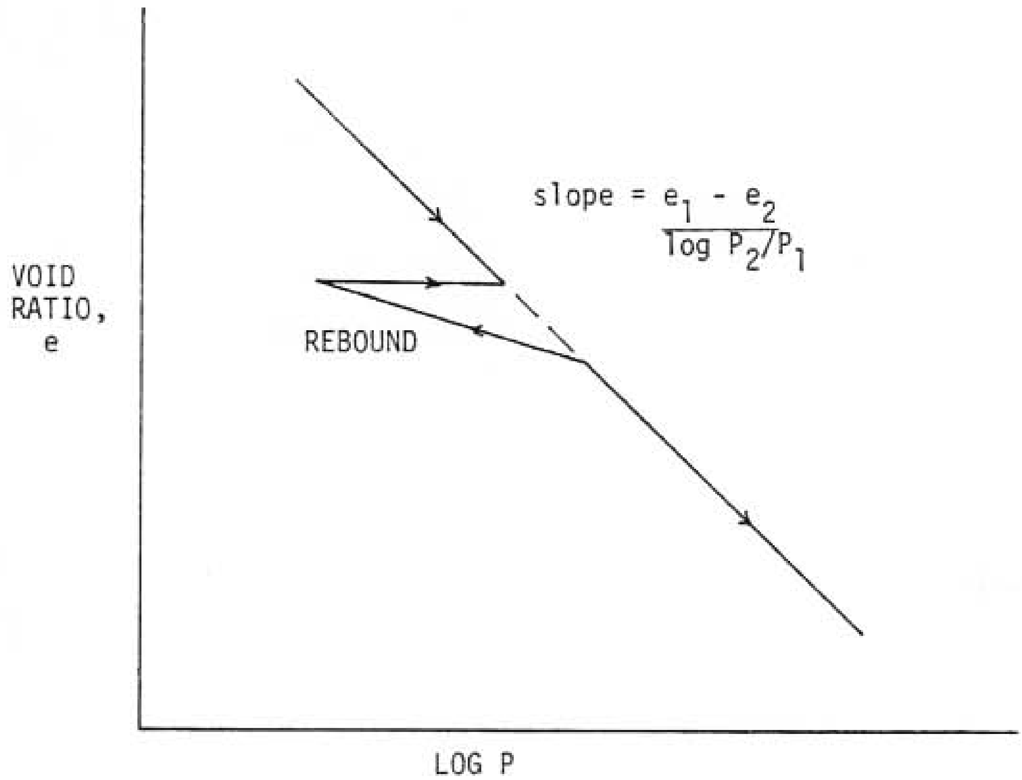


Soil has "memory", if the soil is unloaded as shown by the rebound, it resumes its path on the original curve when the stress is reapplied.

$\frac{\Delta e}{\Delta P} = a_v$ - this parameter is called the coefficient of compressibility.

This value varies at different points on the curve so it would be difficult to use in engineering calculations. It is better to obtain a linear relationship between the void ratio and pressure because the resulting parameter would be constant for a particular soil. This problem is alleviated by plotting e vs. Log P on semi-log scale.

TYPICAL e vs. LOG P PLOT



$$\text{The compression index, } C_c = \frac{e_1 - e_2}{\log P_2/P_1}$$

As consolidation takes place, more and more of the stress is supported by the mineral skeleton because most of the pore water is squeezed out. When the excess pore water pressures are positive, the soil tends to decrease in volume and the process is called consolidation. However, when the pore pressures are negative, the soil tends to increase in volume and the process is called swell or heave. It should be noted that consolidation initially takes place at the drained surfaces, and proceeds to the middle of the layer (for double drainage).

The percent consolidation can be thought of as the change in void ratio, Δe, that is produced by an effective stress change $\Delta\sigma'$, which has occurred after an elapsed time, t.

DARCY'S LAW: $Q = kiAt$ where Q - volume of water

k - Darcy's coefficient of permeability

i - gradient of piezometric head line

t - time

i = change in head/length required to achieve this head change

$i = \Delta h/\Delta l$; but $\Delta h = \Delta u$/sp. wt. of water = change in stress/sp. wt. of water

$$= \frac{\Delta\sigma}{\gamma_\omega}$$

$$\Delta Q = K\,\frac{\Delta\sigma}{\gamma_\omega}\cdot\frac{t}{(H/2)} \times \underbrace{1 \times 1}_{\text{unit area}}$$

Use (H/2) for double drainage and H for single drainage.

$$\Delta Q = \frac{K\,\Delta\sigma\,t}{\gamma_\omega\,(H/2)}$$

The change in volume is the same as the change in void ratio:

$$\Delta V = \frac{\Delta e}{1+e_o}\,(H/2)$$

e_o = initial void ratio

$\therefore$ % of water squeezed out

$$= \frac{\Delta Q}{\Delta V} = \frac{\dfrac{K\,\Delta\sigma\,t}{\gamma_\omega\,(H/2)}}{\dfrac{\Delta e}{1+e_o}\,(H/2)} = \frac{K\,\Delta\sigma\,t\,(1+e_o)}{\gamma_\omega\,\Delta e\,(H/2)^2}$$

REMEMBER:

$$a_v = \frac{\Delta e}{\Delta \sigma}$$

$\therefore$ % of water squeezed out = $\dfrac{K\,(1+e_o)\,t}{\gamma_\omega\, a_v\, (H/2)^2}$

$$\text{Time Factor, } T = \frac{K\,(1+e_o)\,t}{\gamma_\omega\, a_v\, (H/2)^2}$$

Let $C_v = \dfrac{K\,(1+e_o)}{\gamma_\omega\, a_v}$

C_v is the coefficient of consolidation

$$\text{Then, } T = \frac{C_v t}{(H/2)^2}$$

In this experiment; the time for 50% consolidation is used;

From Lambe & Whitman's "Theoretical Soil Mechanics", p. 411

Time Factor, T = 0.197

$$0.197 = \frac{C_v t_{50}}{(H/2)^2}$$

$$C_v = \frac{0.197\,(H/2)^2}{t_{50}}$$

The coefficient of compressibility is calculated from the straight line portion of the e - log P plot.

$$a_v = \frac{0.435\, C_c}{P}$$

The permeability of the soil under different pressures may then be calculated from the equation;

$$K = \frac{C_v \; a_v \; \gamma_\omega}{1 + e}$$

Assumptions in consolidation testing:

1). Soil is saturated; S = 100%

2). Water and soil grains are incompressible

3). Linear relationship between pressure and volume; $a_v = \frac{\Delta e}{\Delta P}$ - this is a poor assumption

4). Value of K remains constant; in actuality, as the void ratio decreases, the coefficient of permeability decreases. This may be critical in soft deposits exhibiting large consolidation strains.

5). Darcy's Law is valid

6). Temperature is constant, therefore the viscosity, n, is constant

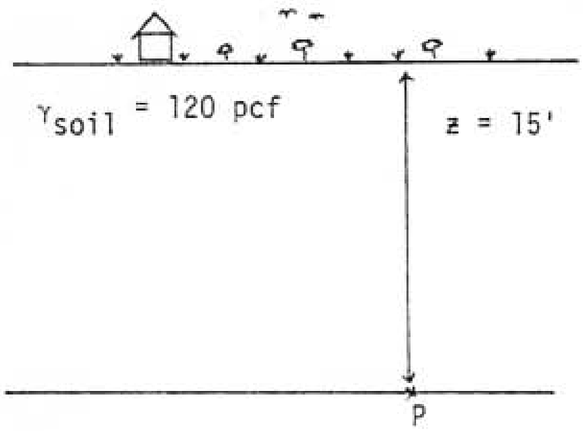
7). Consolidation is one dimensional - no radial drainage

8). Samples are undisturbed.

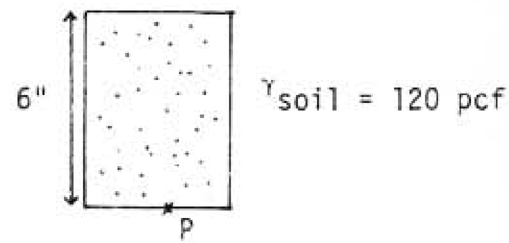
THEORETICAL DESCRIPTION OF CENTRIFUGAL TESTING OF SOIL

Consider the following model representation:

Field Situation



Laboratory Model



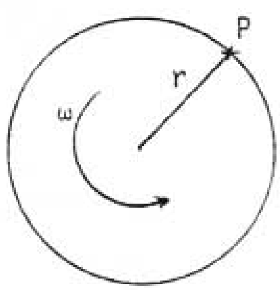
Vertical overburden pressure at P,
$\sigma_v = 15 \times 120 = 1800$ psf

Vertical overburden pressure at P,
$\sigma_v = 0.5 \times 120 = 60$ psf

Since it is required to duplicate the field conditions in the laboratory, the lab model would require an additional surcharge of (1800-60) = 1740 psf.

This additional stress can be achieved by applying the load through the lever arm principle thus avoiding the inconvenient situation of applying 342 lb. to the top of a 6-in. diameter sample to achieve this additional surcharge.

The self weight of the soil can be increased by placing it in a larger gravitational field.



The particle of soil, P, is now fixed to the rotating arm of radius, r, and constant angular velocity, ω.

The centripetal acceleration is equal to the square of the angular velocity multiplied by the radius of rotation.

$$a = \omega^2 r$$

The weight of the particle in the field is equal to its mass by acceleration due to gravity.

$$w_{field} = \text{mass of particle P} \times g$$

$$= m_p g$$

If the rotating arm achieves high velocities, the acceleration due to rotation (i.e. centripetal acceleration) would greatly exceed the acceleration due to gravity and hence the weight of the particle would be increased (note: the mass of the particle remains constant).

$\therefore$ weight of particle in model = m_px accn due to rotation

$$w_{model} = m_p \times \text{centripetal acceleration}$$

Assume that the acceleration due to rotation = 30 x accn due to gravity.

$$a_{rotation} = 30 \text{ g.}$$

$\therefore$ weight of particle in model = m_p x 30 g.

= 30 $m_p g$

From example described: γ_{soil} = 120 psf in field

σ_v in field = 1800 psf

Now in the model under centripetal acceleration, the self weight of the soil, γ = 30 x $m_p g$

= 30 x 120

= 3600 psf.

Therefore the overburden pressure at the base of the 6-in. sample = 3600 x 0.5 = 1800 psf.

This value is now exactly equal to the overburden pressure at point P in the field. The required stress is achieved since this model represents a 15 ft. layer of soil duplicated in the laboratory.

As previously described in the principles of consolidation; the following equation has been derived:

$$T = \frac{C_v t}{(H/2)^2}$$

where C_v - coefficient of consolidation

H - height of sample

T - Time Factor (depends on % consolidation).

t - time required to achieve this % consolidation

$$t = \frac{T\ (H/2)^2}{C_v}$$

$$= \frac{T\ H^2}{4\ C_v}$$

but $\frac{T}{4\ C_v}$ is a constant, therefore $t \propto H^2$.

The time effects in consolidation are very important and their relationship with modelling needs to be carefully examined.

Consider the 6-in. model again:

using 30 g's centripetal acceleration

Ratio of height of prototype to height of model = N. For this case N = 30.

$t \propto H^2$

but $H \propto N$

Therefore $t \propto N^2$

Simply, this means that 1 second in the centrifuge at 30 g acceleration would be equal to a prototype time of $1 \times 30^2 = 900$ seconds or 15 minutes.

Since the consolidation process takes a long time, centrifugal testing allows observation of long term effects in a much shorter time. From Reference 1:

Summary

These two laws of similarity may be stated as follows:

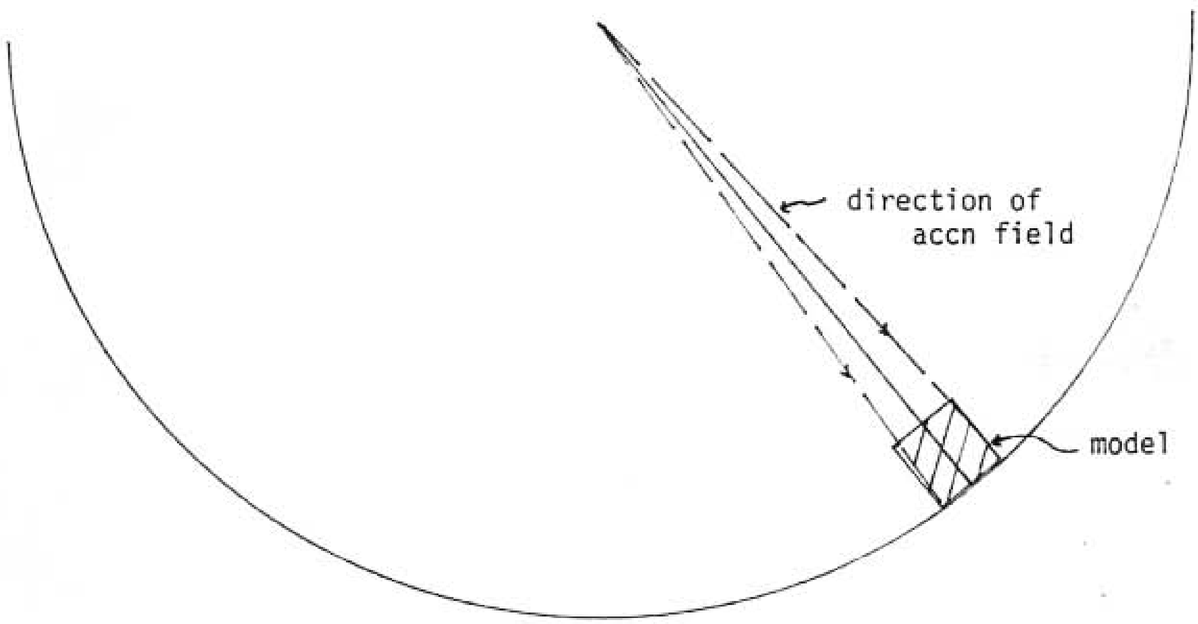
a. "If soils with identical friction, cohesion, and density are formed into two geometrically similar bodies, one a prototype of full scale and one a model of 1/N scale, and if the 1/N scale model is accelerated so that the self weight increases by N times, the stresses at corresponding points are then similar if they are similar on the boundaries."

b. "Once the excess pore water pressure distribution has been made to correspond in model and prototype, all subsequent primary flow processes of pore water are correctly modelled after time t_m in the model that is less than time t_p in the prototype in the ratio of the square of the scale factor N, i.e. $t_m/t_p = 1/N^2$."

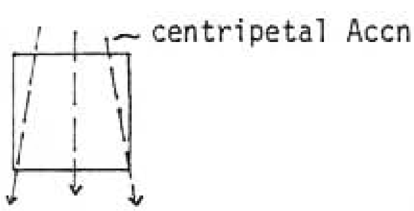
LIMITATIONS OF CENTRIFUGAL MODEL TESTING

1). The gravitational field created by the centrifuge is radially directed from the center of rotation. On the other hand, the acceleration due to gravity acts perpendicular to the earth's surface.

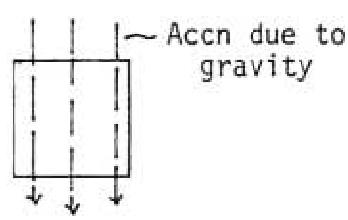
ACCELERATION FIELD IN MODEL:



Accn. field in model:

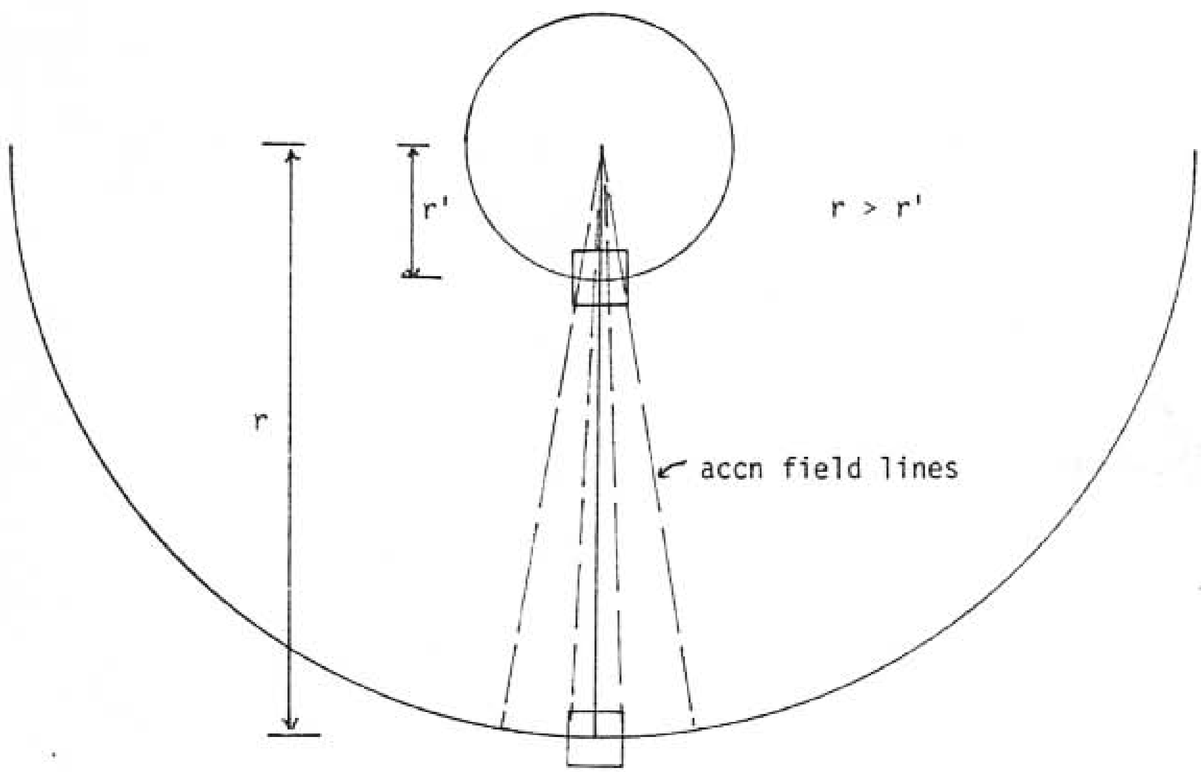


Accn. field in natural conditions.



This directional difference in accelerations makes the particles move in the direction dictated by the acceleration field. However, this problem may be minimized by increasing the radius of rotation so that the acceleration field lines that pass through the model are almost parallel.

This effect is shown in the diagram below:



The effect of increasing the arm of rotation to attain a "more parallel" acceleration field can only be realized if the sample dimensions are kept constant.

SUMMARY OF EFFECT OF MODEL SIZE AND RADIUS OF ROTATION

Radius of Rotation	Model Size	Effect on accn field in model
increase	constant	"more parallel"
increase	increase	if the increase is proportional "no change"
increase	decrease	"much more parallel"
decrease	increase	"worst possible" radial accn field
decrease	constant	"less parallel"
decrease	decrease	if decrease is proportional; "no change"

From Reference 2:

Avgherinos and Schofield have suggested that if the model height H_m is kept less than one-tenth of the radius r (i.e. $H_m < 0.1\ r$), the error in acceleration will be less than ±5 percent.

Proposed Model Height = 4-5 in.

Approximate Radius of Rotation = 43-44 in.

Therefore, the model used in this test will be within the limits for ±5% acceleration field correction.

2). Most soils in nature are pre-consolidated, i.e. they have been subjected to a higher pressure than their present overburden pressure. If the model has to be pre-consolidated in order to represent the field conditions, a surcharge load has to be applied to the model before testing in order to simulate the pre-consolidated nature of the soil. The application of this surcharge load in the model may not be the nature by which the prototype obtained its preconsolidation pressure. However, this problem would be eliminated in this experiment since the test is a comparison of testing procedures rather than the modelling of a prototype.

3). During the period when the centrifuge is accelerated to and decelerated from its required speed, the model is subjected to unsteady acceleration fields. The effective time in the model is proportional to the square of the scaling factor, N, (i.e. H_p/H_m).

$$t \propto N^2$$

During the period when the speed of the rotating arm is increasing or decreasing, the value of N is changing and since the effective time is equal to the product of the actual time by the square of N, it is difficult to assess observation during speed increase or decrease.

4). The subsurface conditions in the field may be characterized by many fissures and rock configurations. These features are very difficult to incorporate into the model because of the "surgical" nature of such a model preparation. These differences may have contrasting effects in the model and the prototype but it would be almost impossible to carve every subsurface feature into the model at such a very small scale.

5). If a small layer of sand is used at the top of a clay sample in a consolidation test, some of the applied load would be required to overcome frictional forces between the particles of sand and the interior container surface. These frictional losses can be minimized by reducting the height of the sand layer or eliminating the sand layer with the use of an air-tight piston with special permeable fabrics.

PHYSICAL DESCRIPTION OF CENTRIFUGE AND RELATED EQUIPMENT AT UNIVERSITY OF FLORIDA

1). Built by the Rucker Company of Oakland, California.

2). Powered by 1.5 h.p. electric motor (U.S. Electrical Motors Inc.)

3). Maximum acceleration capability = 85 g's

4). Maximum radius of rotation = 43 in.

5). Equipped with a tachometer reading up to 2000 rpm.

6). Varidrive with a gear ratio of 5.17

7). Slipring connected to rotating shaft; 12 outlets may be used for electrical measuring devices used within the centrifuge.

8). Lever used for varying speed of rotation; sensitivity is good enough to allow accurate setting of the acceleration.

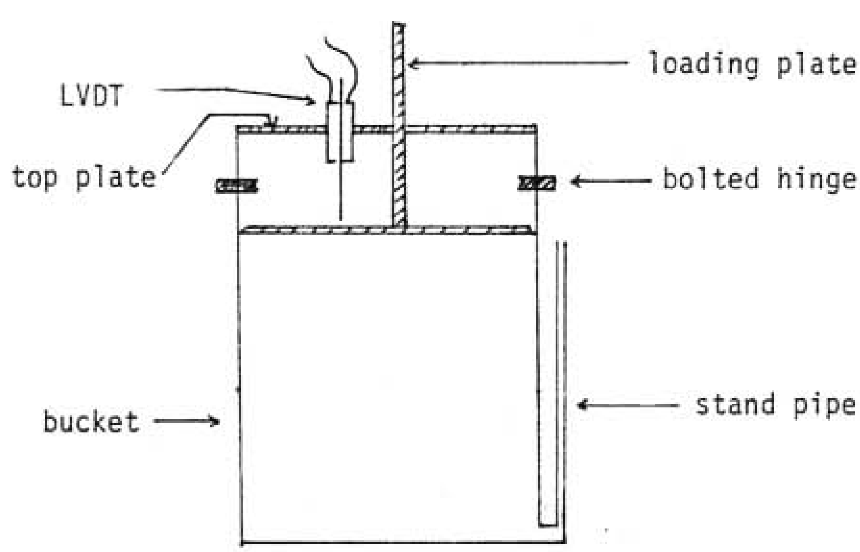
9). A gauge for measuring air pressure is attached to the dashboard or control panel.

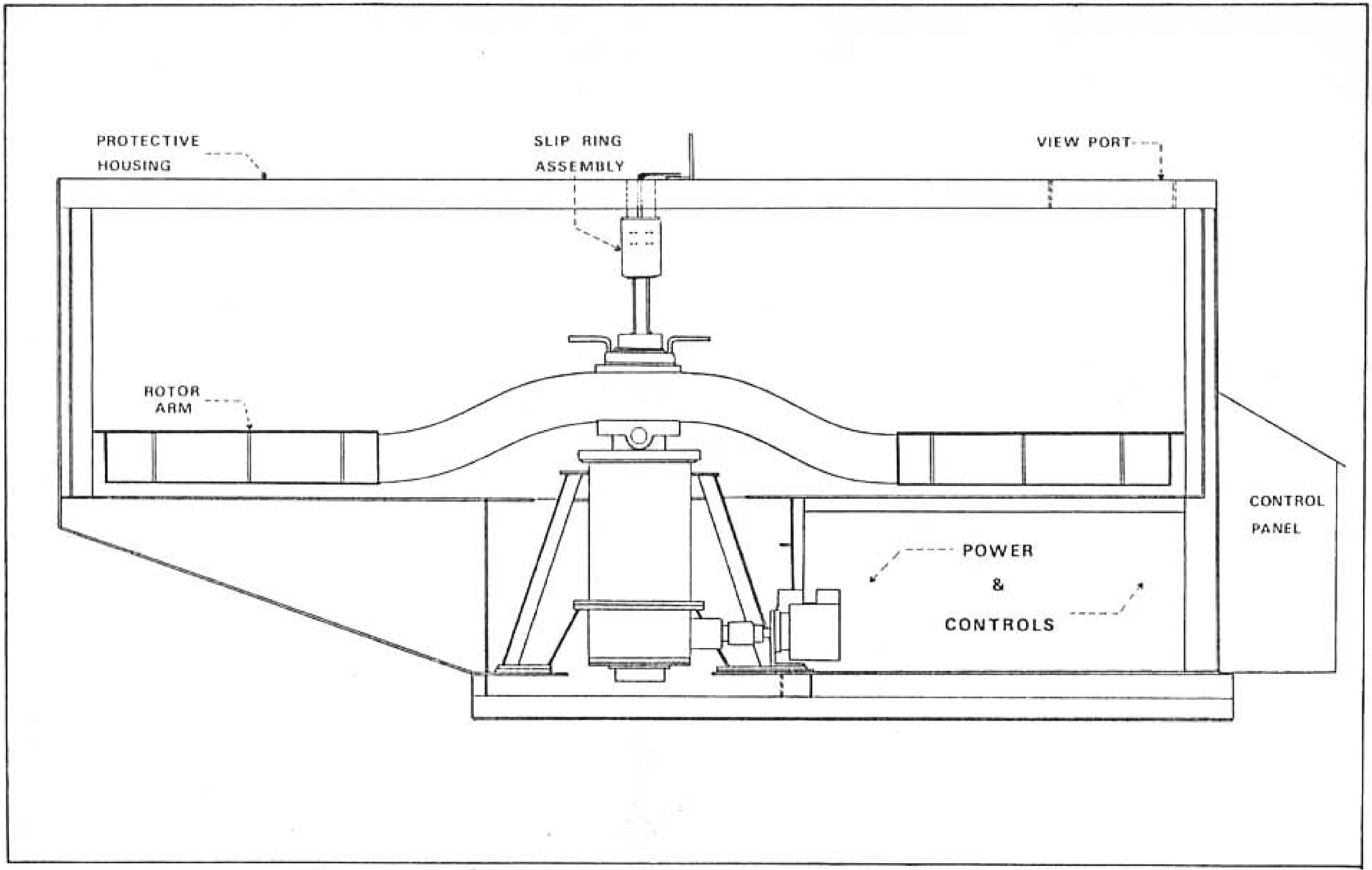
10). Sketch is provided on the following page.

11). The bucket used for testing is made of aluminum and hinged so that it swings up horizontally when the arm starts rotating.

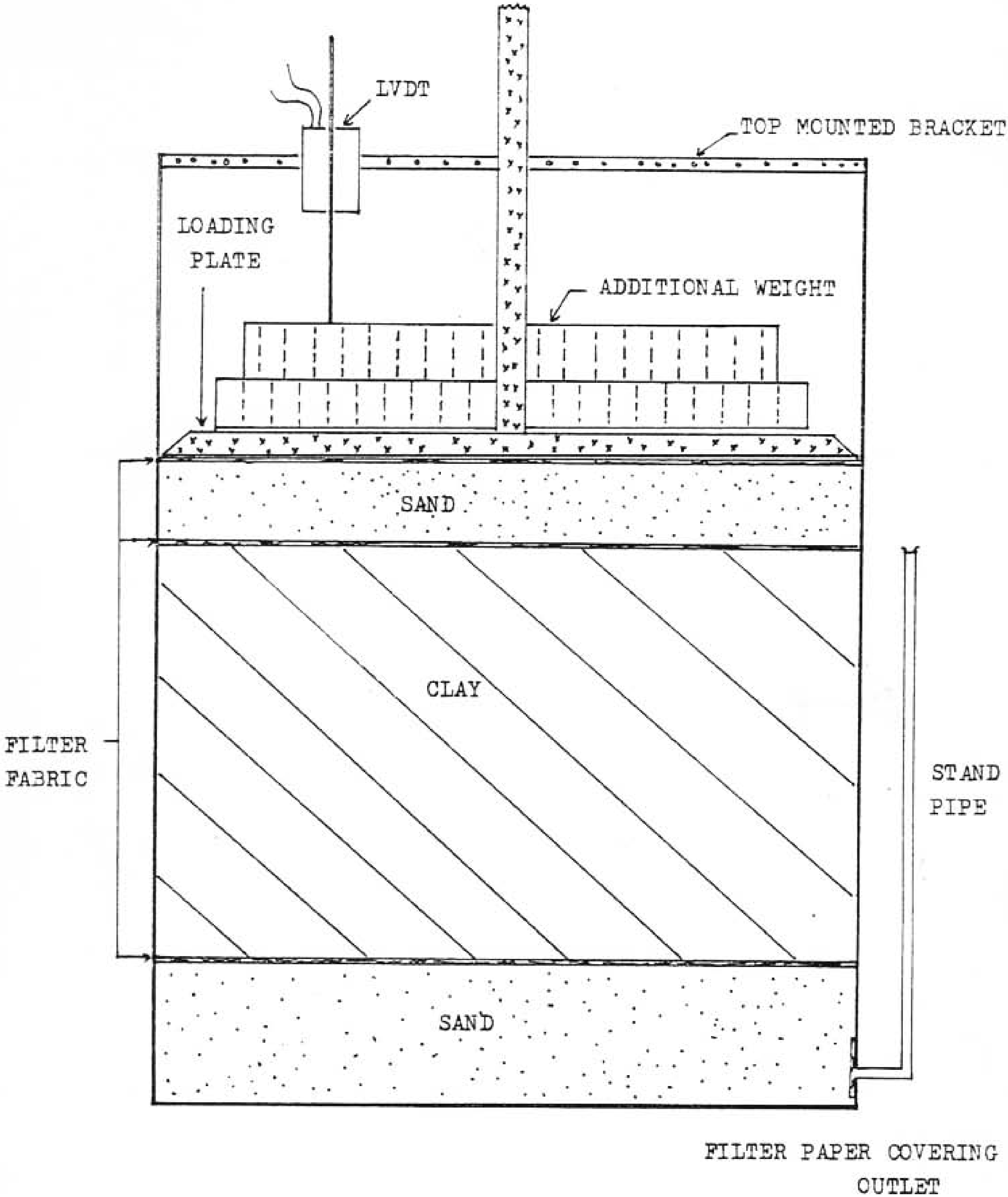
SKETCH OF BUCKET

Cross-Sectional View

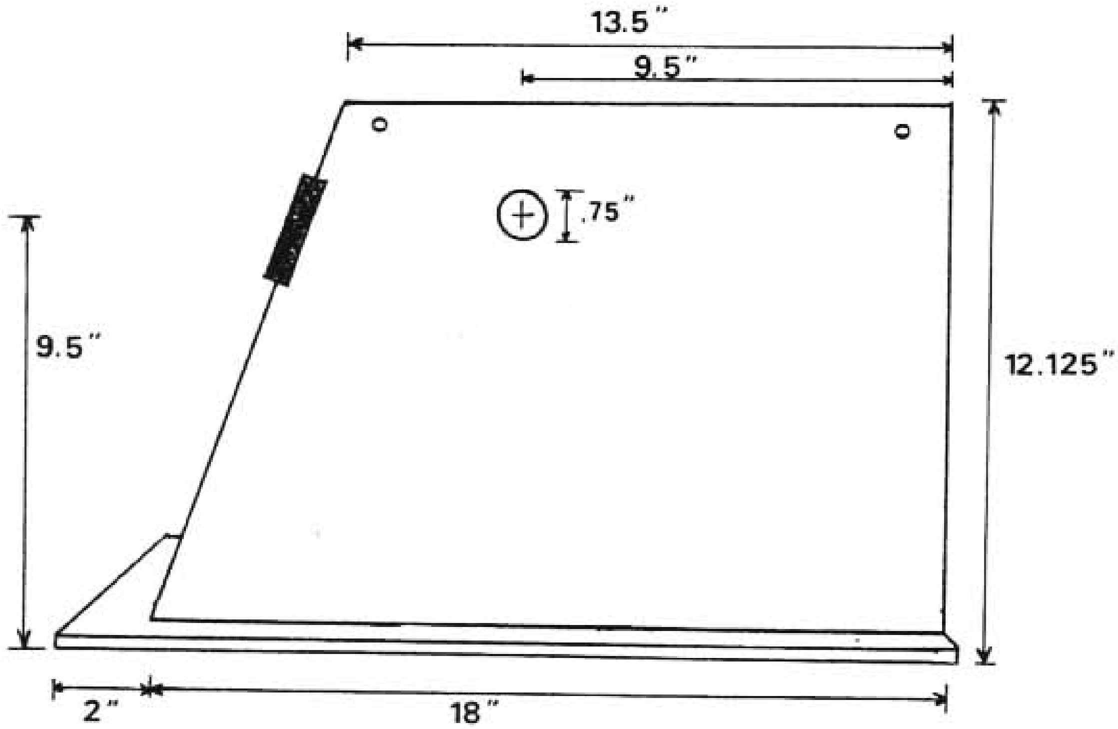




(Courtesy Mr. David Bloomquist)[22]

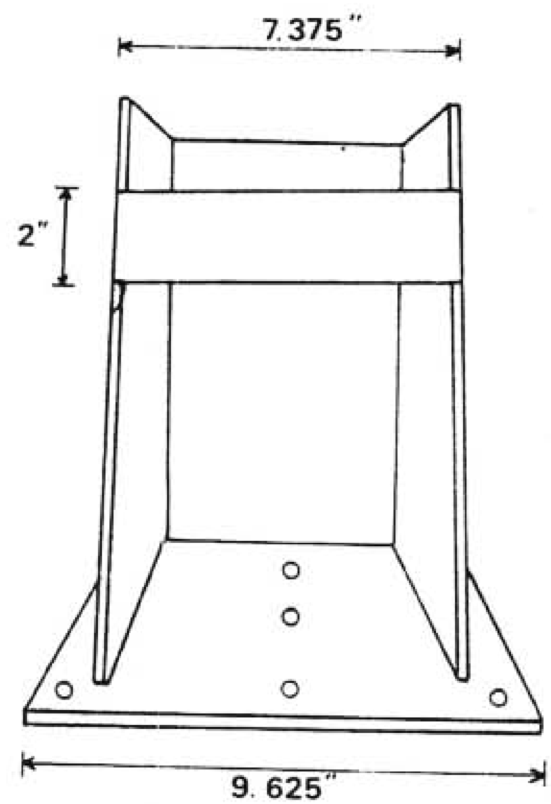


SKETCH OF MODEL IN BUCKET. (not to scale)



ELEVATION VIEWS

NOT TO SCALE



Sketch of Aluminum Bucket Housing

(Courtesy Mr. David Bloomquist)

EQUIPMENT USED FOR MEASURING SETTLEMENT AND ACCELERATION

A). LVDT - Linear voltage differential transformer; this is connected to the top of the loaded bucket and a "frictionless" metal rod from the LVDT rests on the loading plate. As the sample settles, the rod moves down effecting a voltage change. The LVDT operates on a 10 volt power supply and the potential difference is read from a digital display unit. Calibration of LVDT: 1 Volt/0.21835 in. Best results are obtained within the range of ±2 volts.

B). Accelerometer - this device is connected to the housing that contains the bucket. It provides an accurate reading of the magnitude of the acceleration at the point to which it is connected within the centrifuge. A correction is applied to the accelerometer reading so that the acceleration at the midpoint of the sample can be computed. The accelerometer operates on a 15 volt power supply and the potential difference is read on a multimeter similar to that used for the LVDT. Accelerometer calibration; lmV/lg.

C). Power Supply - two Hewlett Packard 6215 A power supplies are used.

D). Digital Multimeter -

For LVDT: Systron Donner Model 7003

For Accelerometer: Data Technology Model 30

Both the accelerometer and the LVDT are powered through the slip ring, and their output is also channeled through this slipring to the meters on the control panel.

A pore pressure transducer is also available but it was not used in the course of this experiment. This device is useful in monitoring the dissipation of pore water pressures.

SOIL DESCRIPTION

From Reference 3:

The clay used in the tests was a kaolinite from Edgar, Florida. The clay has a LL = 54.2% and a PL = 30.2%. The optimum moisture content, determined from the standard compaction test, was approximately 26%.

The specific gravity of the clay is assumed to be 2.65.

Color - white.

From Reference 4:

From Cornell, 1951

Kaolinite's compression index varies from 0.19 to 0.26.

From Jimenez Salas and Serratosa, 1953

Kaolinite's compression index = 0.6

The unit weight of the saturated sand is between 124 and 130 pcf. This sand was used as the surcharge in modelling.

MODEL PREPARATION

1). Thoroughly wash and dry bucket, making sure that the standpipe is unclogged.

2). Weigh the empty bucket to the nearest gram, and measure its internal height, h_1 (nearest 1/8 in.)

3). Moisten a small piece of filter paper so that it sticks against the drain hole located at the inside surface of the bucket near the base. This prevents sand from blocking this drainage exit during the test.

4). Place a layer of sand (1/2 to 1 in) at the base of the bucket, and compact it by shaking the bucket vigorously. Saturate the sand, and paper towels can be used to absorb any excess. Level the surface of the base layer and place a custom-cut filter fabric above it.

5). Weigh the bucket and saturated sand, and measure the internal height h_2. This weight is now equal to the tare for the clay layer.

6). Saturate the pulverized kaolinite in a mixer and blend for several hours. This minimizes air pockets within the clay, and provides a consistent, homogeneous mixture. Take a sample for moisture content.

7). Carefully place this soft clay onto the sand layer until the desired height is achieved. Place another custom-fitted piece of filter fabric above the clay layer and lightly tamp with the loading plate until it is level. Measure the internal height h_3 and weigh to the nearest gram.

8). Place another thin layer of sand above the clay layer and saturate. Blot off any excess water with paper towels, and level with filter fabric. Weigh again and measure the internal height h_4.

9). Make sure standpipe is full of water up to the level of the top of the clay layer.

PROCEDURE

1). The model in the bucket is now bolted onto the housing with the loading plate and any additional weights in place.

2). The top bracket that holds the LVDT is fastened to the bucket, and the LVDT is adjusted to a convenient position on the loading plate or the weights.

3). The centrifuge arm is balanced to reduce vibration and the rotating arm is accelerated until it reaches a pre-determined speed. This speed determines the acceleration which can be read off a digital millivoltmeter connected to the accelerometer. The acceleration can be accurately monitored by observing this millivoltmeter for any deviations from the pre-determined voltage.

4). One or two minutes after "start up" should be enough time to allow for proper "seating"; the voltage from the millivoltmeter connected to the LVDT is noted and this is now the reference voltage for this particular test.

5). "Voltage" readings are taken at appropriate time intervals to monitor the settlement of the model. A plot of displacement vs. square root of time is done as the test progresses to indicate the time required for settlement, and this reduces unnecessary testing.

6). When 100% primary consolidation has been achieved, the centrifuge is stopped and another load is incremented the following day; data retrieval is continued as described above.

7). After the final test has been completed, the bucket is removed from the centrifuge, and the loading plate and weights are carefully lifted off the model.

8). The internal height, h_4 is measured again to determine the total change in height of the model.

9) The top layer of sand is scraped off without disturbing the clay layer, and a micrometer is pierced at various points in the clay to determine an average height.

10) The clay is weighed and an undisturbed sample is extracted with a cutting ring for a regular consolidation test. Obtain a specimen for moisture content determination.

11). Regular consolidation test is performed using the specifications of EM 1110-2-1906 Appendix VIII.

SOURCES OF ERROR

1). The load to which the clay layer was subjected is computed by considering the saturated sand surcharge, the weight of the loading plate and any additional weights placed on the loading plate. The sand layer obviously had some frictional losses against the sides of the bucket which reduced the pressure on the clay layer as computed. The loading plate's supporting rod passed through a bracket at the top of the bucket, and this may have resulted in considerable frictional losses especially when the plate tilted from uneven settlement. It was observed that the bucket did not swing up in an exact horizontal position so the acceleration field varied slightly throughout the configuration of the loading weights; the effect would be an inaccurate (minimal) estimate of the actual stresses on the clay.

2). The LVDT calibration was only linear within the range ±2 volts so the few readings outside of these limits would have introduced some minor deviations.

3). The adjustment to the LVDT readings to compensate for sand settlement will be discussed in the "Evaluation of Data for Centrifugal Tests."

4). The clay layer settled unevenly because of the uneven pressure on the layer resulting from the bucket not swinging up horizontally. The height of the layer after testing had to be determined by testing various points, and then averaging the series of readings.

5). In order to maintain the bottom layer of sand in a saturated state, it was necessary to keep the standpipe filled with water up to the level of the top of the clay layer. It was impossible to monitor this situation during testing. The drainage hole that leads into the standpipe must permit free passage of escaping water during the test; there is a possibility that this exit did not function properly during the entire test, and may have resulted in a single drainage situation (i.e. only through the top layer) for a portion of the test.

6). As consolidation took place, pore water flowed into the top layer of sand (and the bottom layer of sand), but this top layer was already saturated so the excess water collected on top of the loading plate. (note: the loading plate is perforated.) This resulted in an additional surcharge that was not accounted for in the computation. The effect of this water surcharge would create more error in the centrifugal tests at higher acceleration since its specific weight would be increased markedly, and the result would be a larger contribution to the pressure.

7). There was a very small quantity of soft clay that seeped along the side of the bucket into the sand layer; this error could have led to erroneous results if larger quantities (say greater than 10 grams) had been lost in this fashion.

8). The chemical balances available did not have the capacity to measure the weight of the bucket with the model in place; regular "shop" scales were used which reduced the accuracy of the weight measurements.

ADJUSTMENT TO COMPUTE ACCURATE ACCELERATION VALUES

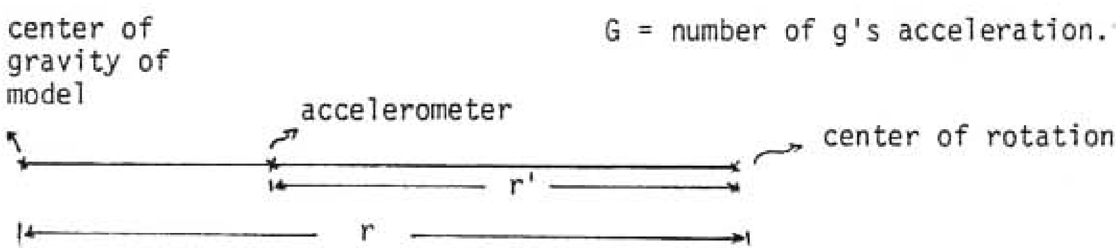
The acceleration field to which the sample is subjected is measured using a device known as an accelerometer. This instrument measures a potential difference for various values of acceleration; the acceleration field can be computed since the accelerometer is calibrated and the "voltage difference" can be read off the millivoltmeter. However, the reading provided by the accelerometer is only a guide or reference to the acceleration field at the center of the model because the accelerometer is positioned at a fixed point on the housing and does not represent the actual value at the center of the model.

$$G = \frac{r\,(N)^2}{9.4 \times 10^5}$$

where r = radius from axis of rotation to center of gravity of soil mass, in.

N = speed in rpm

G = number of g's acceleration.



r - radius from center of rotation to center of gravity of model

r' - radius from center of rotation to accelerometer

Say X g's are required;

$$X = \frac{r\ (N)^2}{9.4 \times 10^5} \qquad -1$$

Acceleration at position of accelerometer; say Y g's

$$Y = \frac{r'\ (N)^2}{9.4 \times 10^5} \qquad -2$$

From Eq. 1

$$N = \left\{ \frac{9.4 \times 10 \quad X}{r} \right\}^{\frac{1}{2}}$$

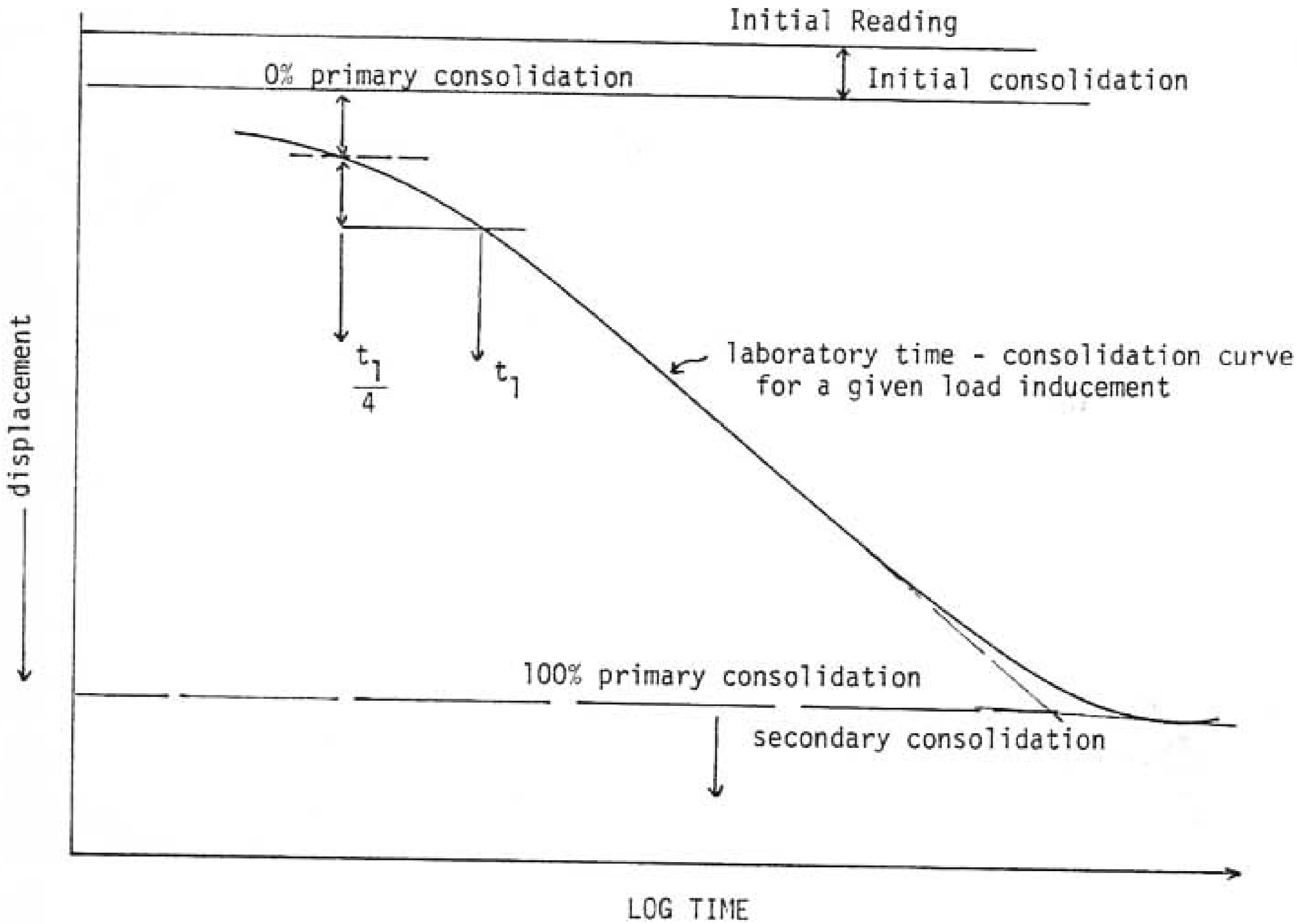
Subst. for N in Eq. 2

$$Y = \frac{r' \times 9.4 \times 10^5 \times X}{r \times 9.4 \times 10^5}$$

$$Y = \frac{r'}{r}\ X$$

To obtain an acceleration of X at the center of the sample, the acceleration at the accelerometer $Y = \frac{r'}{r} X$

PROCEDURE FOR DETERMINING STRAINS AT WHICH 0% AND 100% CONSOLIDATION TAKES PLACE USING STRAIN VS. LOG TIME CURVE

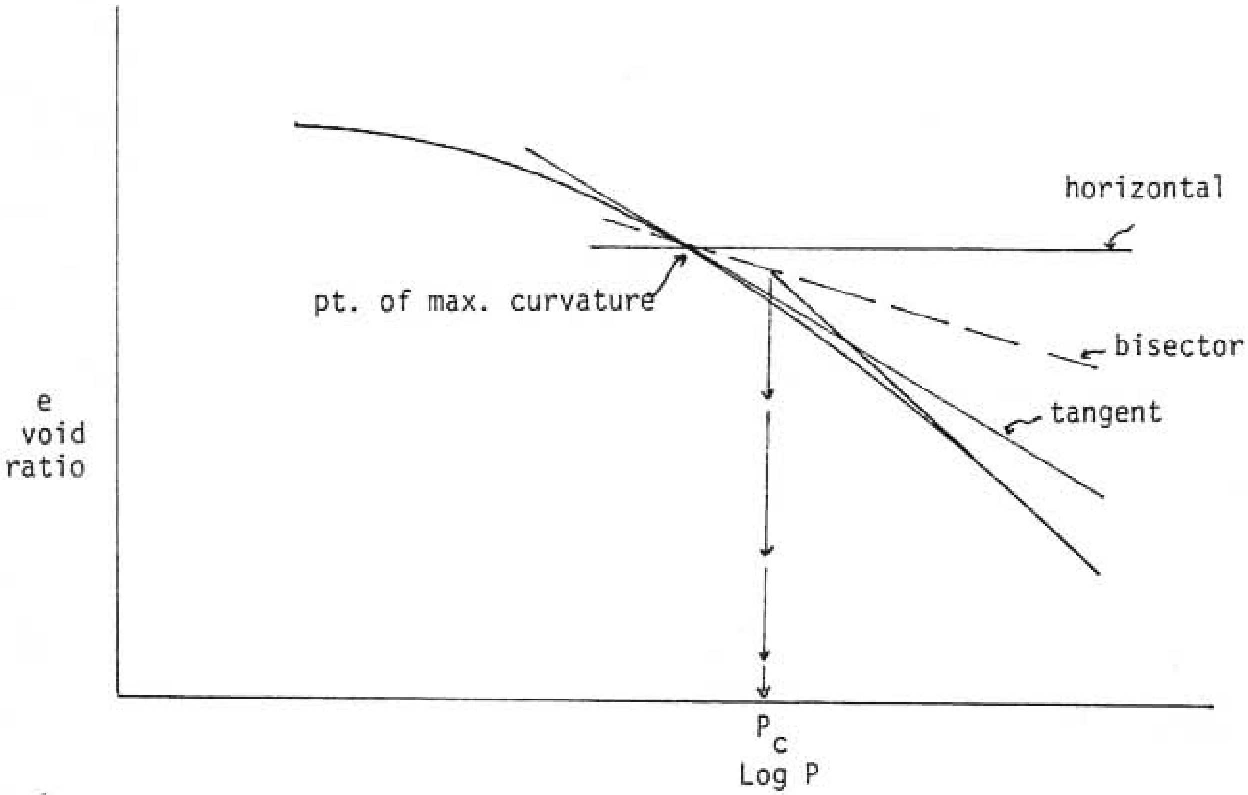


A. For 100% primary consolidation:

Intersection of the tangent to the curve at the point of inflection, with the tangent to the straight-line portion representing the secondary time effect.

B. Select a time t_1 in upper portion of curve and the time equal to $t_1/4$. The difference in strains between t_1 and $t_1/4$ is equal to a. Decrease the displacement at time $t_1/4$ by an amount equal to a and this would be the point of 0% primary consolidation.

CASAGRANDE'S PROCEDURE FOR DETERMINING THE PRECONSOLIDATION PRESSURE



1). "eyeball" the point of maximum curvature,

2). draw a horizontal and tangent to this point,

3). bisect this angle

4). project virgin curve back until it meets the bisector

RESULTS

All weights in grams.		BEFORE TEST				AFTER TEST	
		SPECIMEN		TRIMMINGS		SPECIMEN	
Tare + wet soil		654.08		41.00		134.96	
Tare + dry soil		-		29.70		100.07	
Water	W_w	W_{wo}	43.40	11.30		W_{wf}	34.89
Tare		514.24		4.60		4.51	
Dry Soil	W_s	96.44		25.1		95.56	
Water cont.	w	w_o	45.0 %	45.0 %		w_f	36.5 %

Area of specimen, A, sq. cm = 31.67

Height of specimen, H, in = 1.00

Sp. gr. of solids, G_s = 2.65

Height of solids, $H_s = \dfrac{W_s}{A * G_s * \gamma_w} = \dfrac{96.44}{31.67 * 2.65 * 1 * 2.54} = 4.52$ in

Original height of water, $H_{wo} = \dfrac{W_{wo}}{A * \gamma_w} = \dfrac{43.40}{31.67 * 1 * 2.54} = .540$ in

Final height of water, $H_{wf} = \dfrac{W_{wf}}{A * \gamma_w} = \dfrac{34.89}{31.67 * 1 * 2.54} = .434$ in

Net change in height of specimen at end of test, ΔH = 0.1473 in

Height of specimen at end of test, $H_f = H - \Delta H$ = 0.8527 in

Void Ratio before test, $e_o = \dfrac{H - H_s}{H_s} = \dfrac{1.00-0.452}{0.452} = 1.21$

Void Ratio after test, $e_f = \dfrac{H_f - H_s}{H_s} = \dfrac{0.8527-0.452}{0.452} = 0.89$

Degree of saturation before test, $S_o = \dfrac{H_{wo}}{H - H_s} = \dfrac{0.540}{1-0.452} = 98.5\%$

Degree of saturation after test, $S_f = \dfrac{H_{wf}}{H_f - H_s} = \dfrac{0.434}{0.853-0.452} = 108.3\%$

Dry density before test, $\gamma_d = \dfrac{W_s}{H * A} = \dfrac{96.44 * 62.4}{* \quad *2.54} = 74.8$ pcf

REMARKS: Specific Gravity is assumed; S = 100% O.K.

Regular Consolidation Test
Reference: EM 1110-2-1906
Appendix VIII

CONSOLIDATION TEST

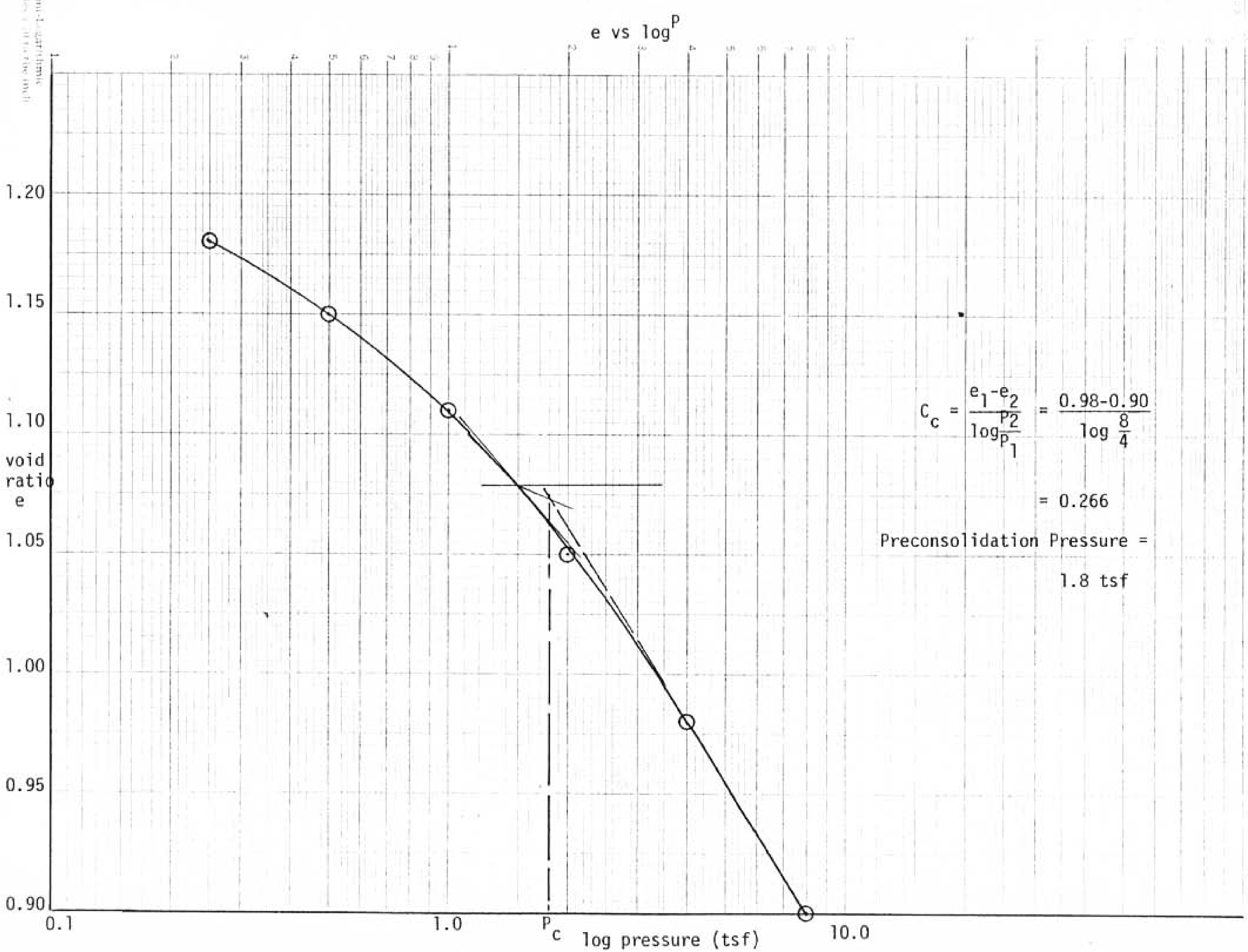
Computation of void ratio

Pressure, tsf	Date increment app.	Time increment effective	Displacement by 10^{-4} in	Change in height, ΔH	Height of voids	Void ratio, e
0.25	2/21/82	24 hr	Ref. 1188 1352	164	0.532	1.18
0.50	2/22/82	24 hr	1475	287	0.519	1.15
1.00	2/23/82	24 hr	1663	475	0.501	1.11
2.00	2/24/82	24 hr	1903	715	0.477	1.05
4.00	2/25/82	24 hr	2233	1045	0.444	0.98
8.00	2/26/82	10 hr	2609	1421	0.406	0.90

$H_s = 0.452$ in

Height of voids, $H_v = (H-H_s) - \Delta H =$

Void ratio, $e = \dfrac{H_v}{H_s}$

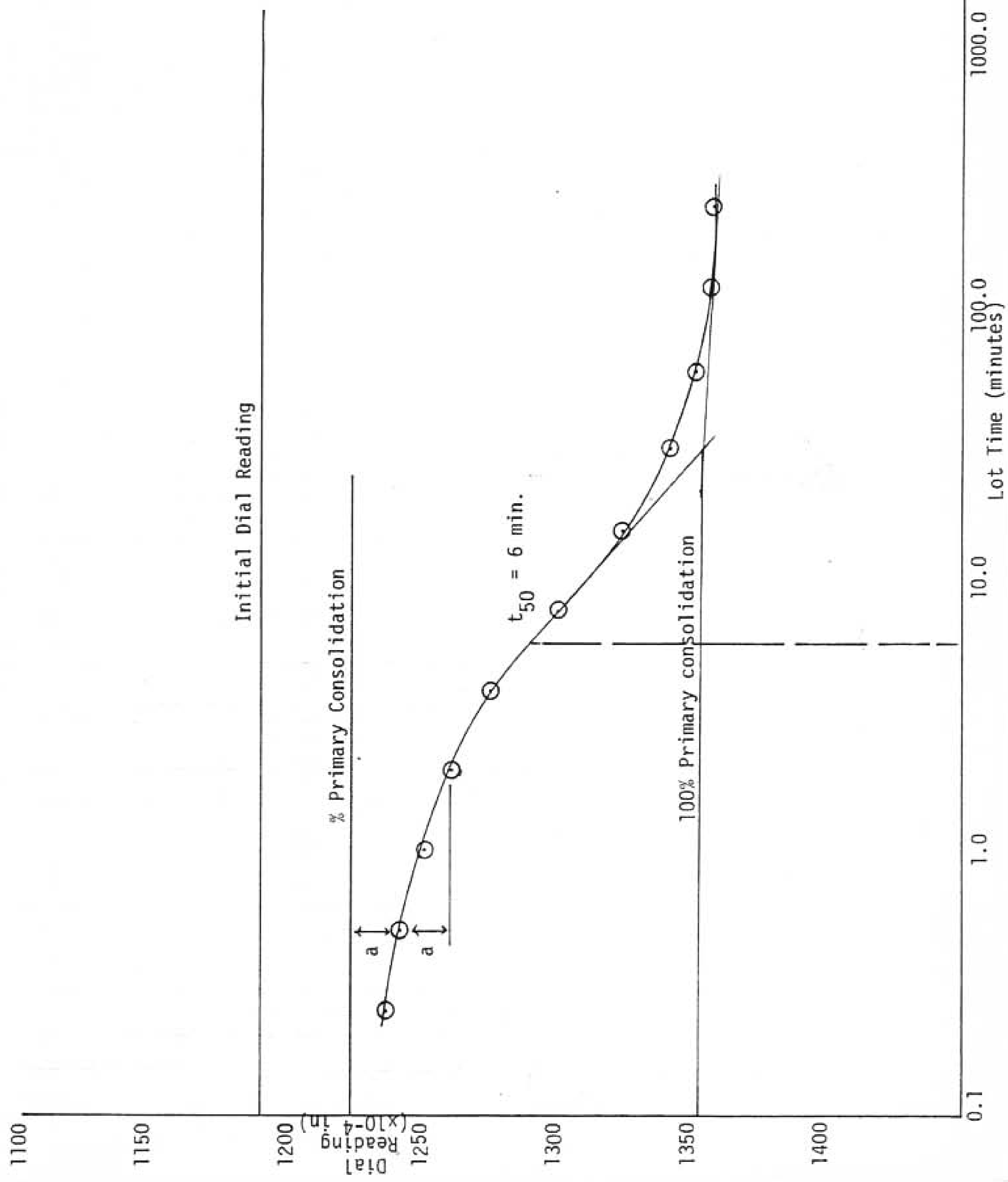

e vs log P
void ratio e
1.20
1.15
1.10
1.05
1.00
0.95
0.90
0.1
1.0
10.0
P_c
log pressure (tsf)
$C_c = \frac{e_1 - e_2}{\log \frac{P_2}{P_1}} = \frac{0.98-0.90}{\log \frac{8}{4}}$
$= 0.266$
Preconsolidation Pressure =
1.8 tsf

DATA RETRIEVED FROM REGULAR CONSOLIDATION TEST

Date & Pressure	Time of Day	Elapsed Time (min)	Dial Reading by 10^{-4} in	Person Reporting
Sunday Feb. 21st, 1982 0.25 tsf	9:00:00	START	1188	𝒥
	9:00:06	0.1	1220	"
	9:00:15	0.25	1235	"
	9:00:30	0.50	1241	"
	9:01:00	1.0	1248	"
	9:02:00	2.0	1257	"
	9:04:00	4.0	1273	"
	9:08:00	8.0	1297	"
	9:16:00	16.0	1321	"
	9:32:00	32.0	1339	"
	10:04:00	64.0	1349	"
	11:08:00	128.0	1354	"
	1:16:00	256.0	1358	"
	4:36:00	456.0	1360	"
				"

Sunday, Feb. 21st, 1982

Pressure = 0.25 tsf

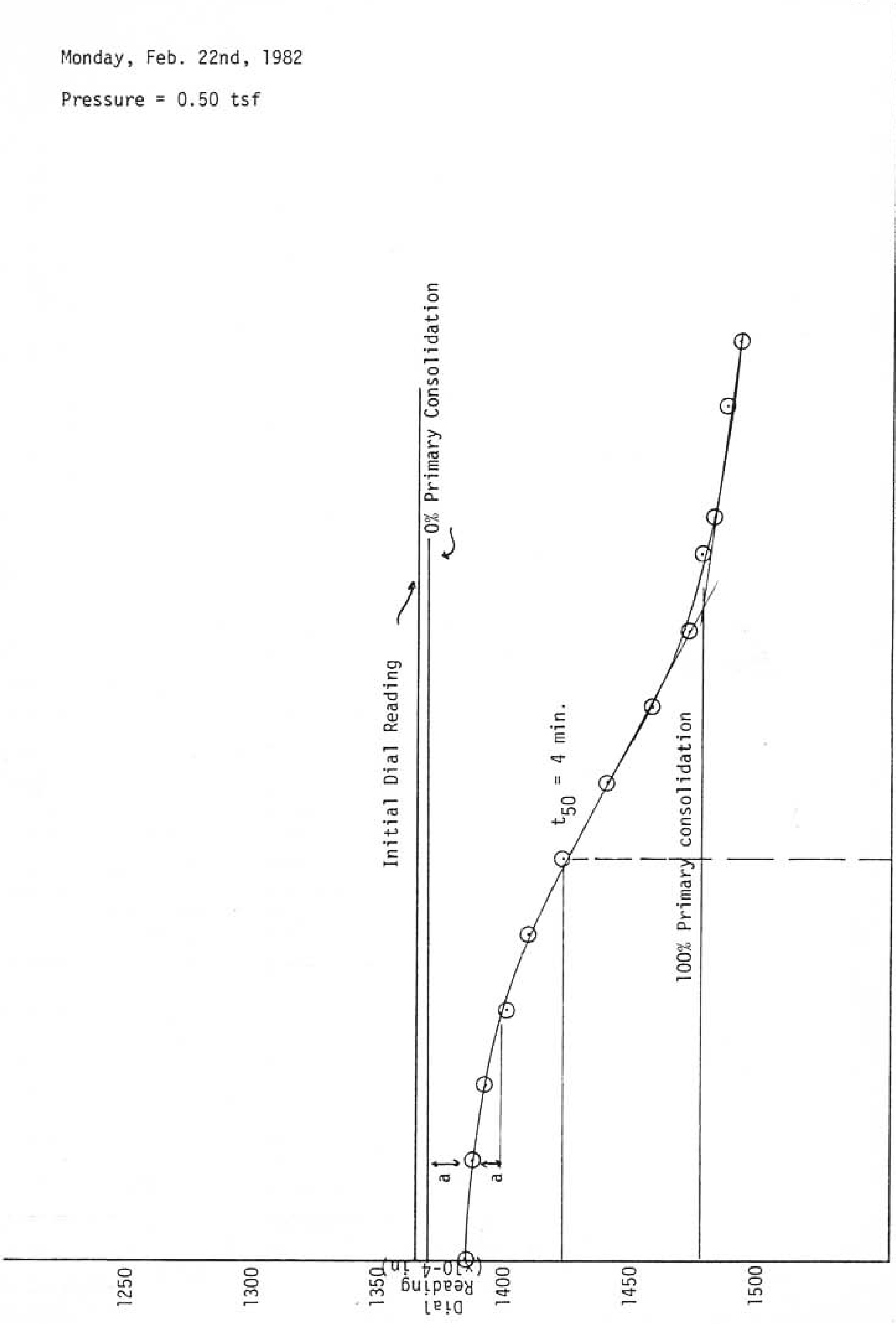


DATA RETRIEVED FROM REGULAR CONSOLIDATION TEST

Date & Pressure	Time of Day	Elapsed Time (min)	Dial Reading by 10^{-4} in	Person Reporting
Monday Feb. 22nd, 1982 0.50 tsf	8:00:00	START	1364	[illegible]
	8:00:06	0.1	1381	"
	8:00:15	0.25	1385	"
	8:00:30	0.50	1391	"
	8:01:00	1.0	1398	"
	8:02:00	2.0	1407	"
	8:04:00	4.0	1421	"
	8:08:00	8.0	1438	"
	8:16:00	16.0	1455	"
	8:32:00	32.0	1469	"
	9:04:00	64.0	1476	"
	9:33:00	93.0	1480	"
	12:16:00	256.0	1486	"
	3:30:00	450.0	1489	"
				"

Monday, Feb. 22nd, 1982

Pressure = 0.50 tsf

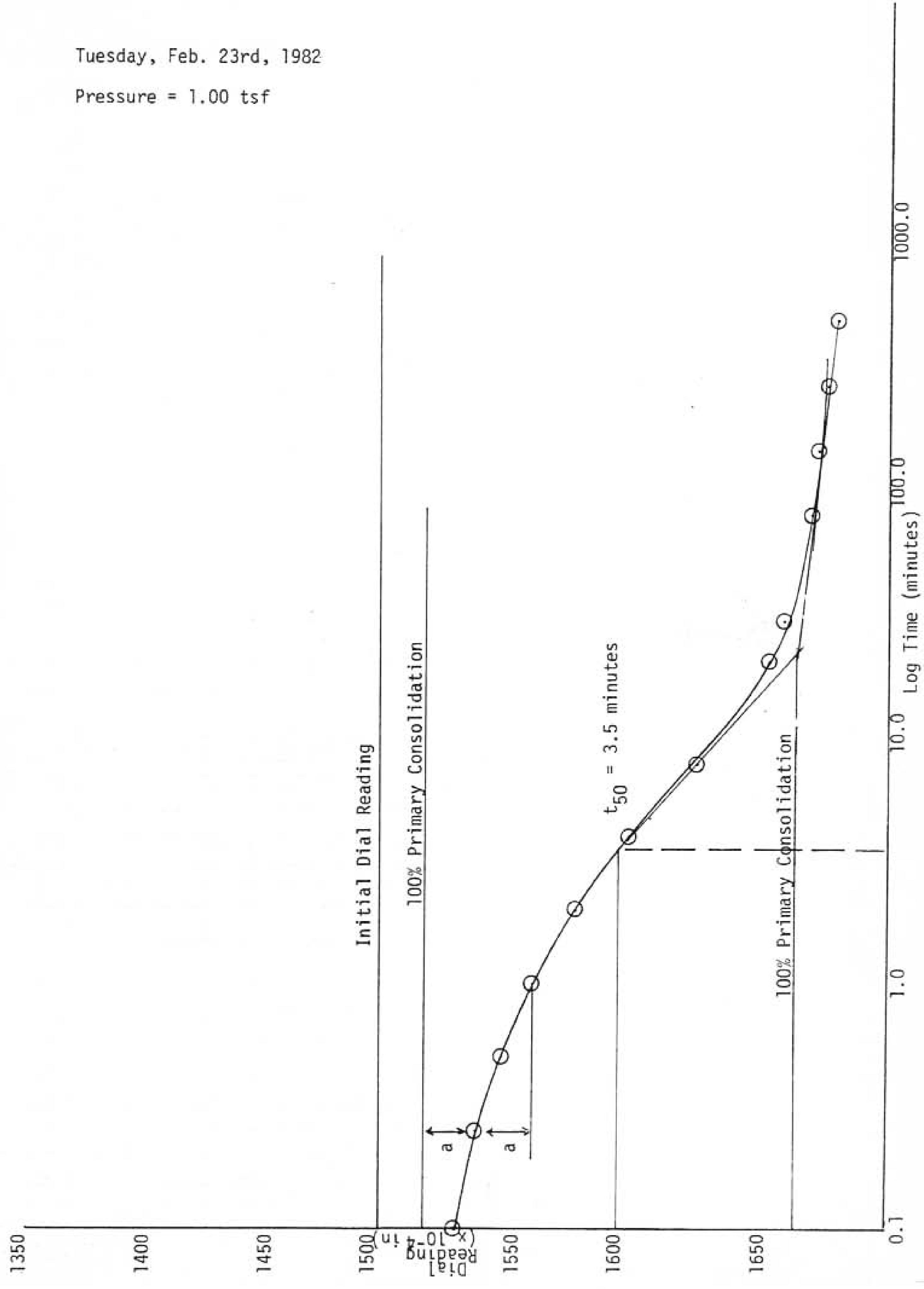


DATA RETRIEVED FROM REGULAR CONSOLIDATION TEST

Date & Pressure	Time of Day	Elapsed Time (min)	Dial Reading by 10^{-4} in	Person Reporting
Tuesday Feb. 23rd, 1982 1.00 tsf	9:00:00	START	1494	[illegible]
	9:00:06	0.1	1525	"
	9:00:15	0.25	1533	"
	9:00:30	0.50	1543	"
	9:01:00	1.0	1556	"
	9:02:00	2.0	1573	"
	9:04:00	4.0	1595	"
	9:08:00	8.0	1622	"
	9:21:00	21.0	1652	"
	9:30:00	30.0	1657	"
	10:24:00	84.0	1668	"
	11:28:00	148.0	1672	"
	1:27:00	267.0	1675	"
	5:00:00	480.0	1679	"
				"

Tuesday, Feb. 23rd, 1982

Pressure = 1.00 tsf

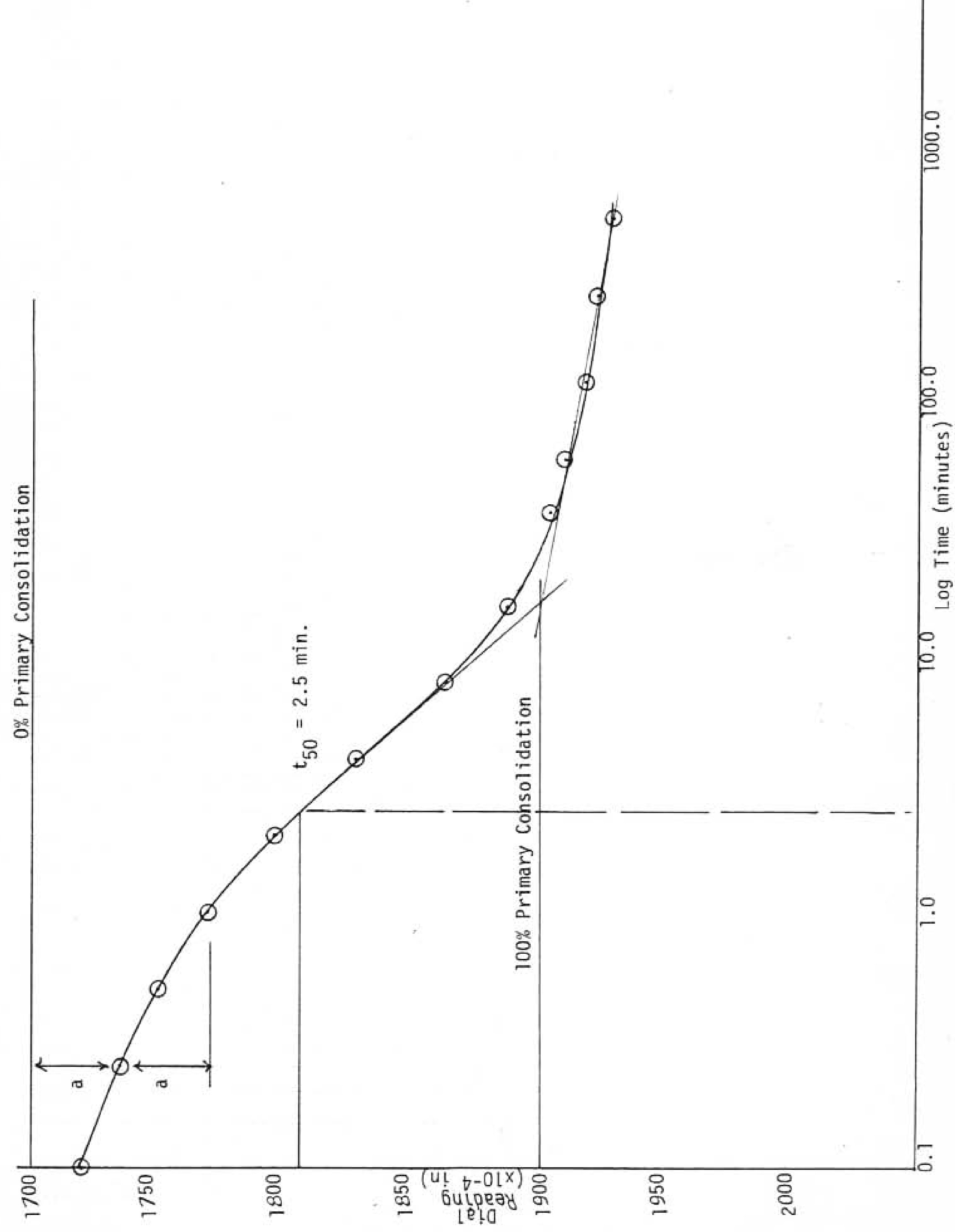


DATA RETRIEVED FROM REGULAR CONSOLIDATION TEST

Date & Pressure	Time of Day	Elapsed Time (min)	Dial Reading by 10^{-4} in	Person Reporting
Wednesday Feb. 24th, 1982 2.00 tsf	8:00:00	START	1685	
	8:00:06	0.1	1725	"
	8:00:15	0.25	1740	"
	8:00:30	0.50	1754	"
	8:01:00	1.0	1773	"
	8:02:00	2.0	1799	"
	8:04:00	4.0	1832	"
	8:08:00	8.0	1867	"
	8:16:00	16.0	1891	"
	8:36:00	36.0	1907	"
	9:00:00	60.0	1913	"
	10:00:00	120.0	1919	"
	12:12:00	252.0	1925	"
	4:32:00	512.0	1933	"
				"

Wednesday, Feb. 24th, 1982

Pressure = 2.00 tsf

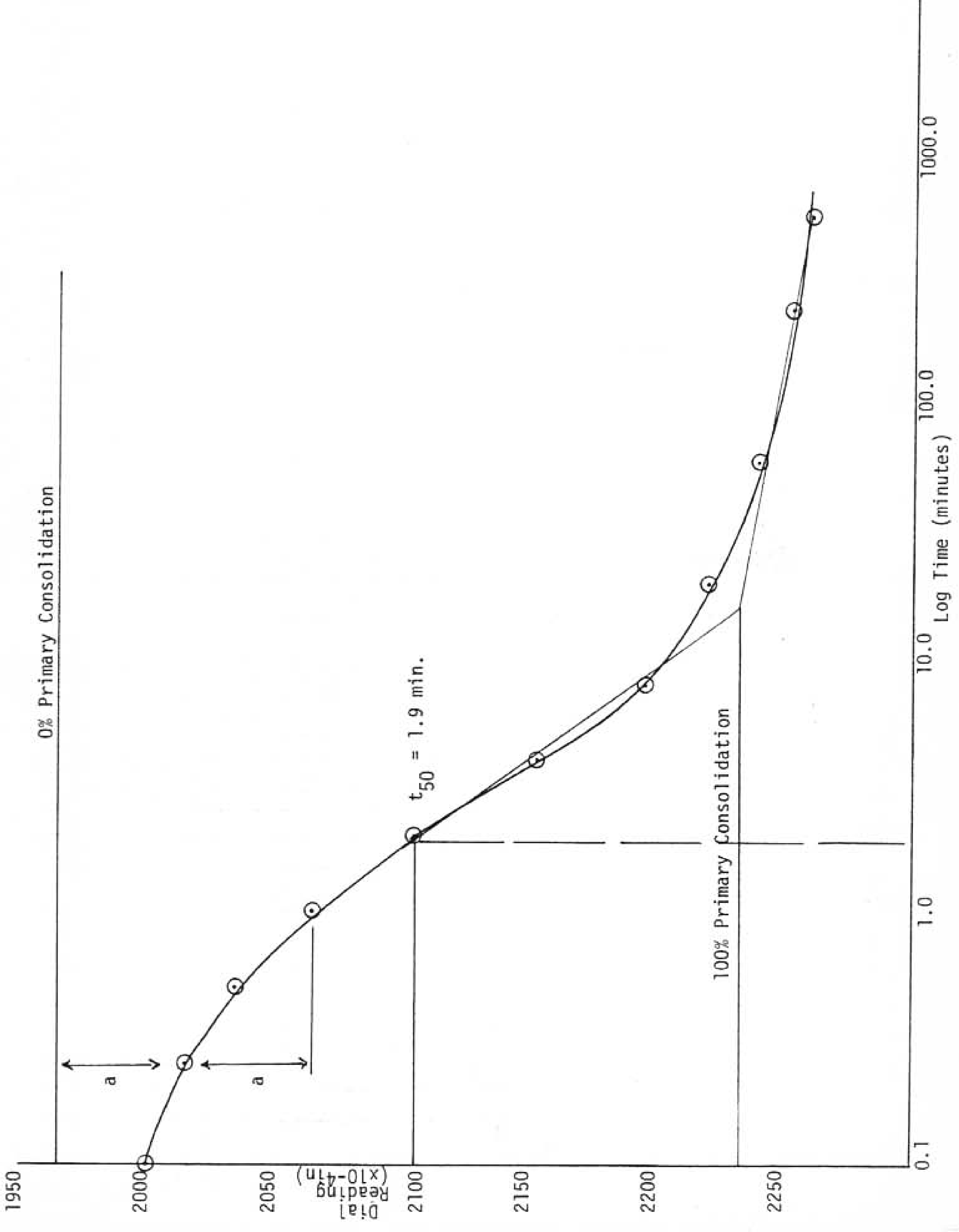


DATA RETRIEVED FROM REGULAR CONSOLIDATION TEST

Date & Pressure	Time of Day	Elapsed Time (min)	Dial Reading by 10^{-4} in	Person Reporting
Thursday, Feb. 25th, 1982 4.00 tsf	6:43:00 a.m.	START	1938	[illegible]
	6:43:06	0.1	2000	"
	6:43:15	0.25	2015	"
	6:43:30	0.50	2036	"
	6:44:00	1.0	2065	"
	6:45:00	2.0	2106	"
	6:47:00	4.0	2154	"
	6:51:00	8.0	2195	"
	7:03:00	20.0	2221	"
	7:13:00	30.0	2230	"
	7:43:00	60.0	2238	"
	8:54:00	131.0	2247	"
	10:43:00	240.0	2254	"
	3:33:00	530.0	2260	"
				"

Thursday, Feb. 25th, 1982

Pressure = 4.00 tsf

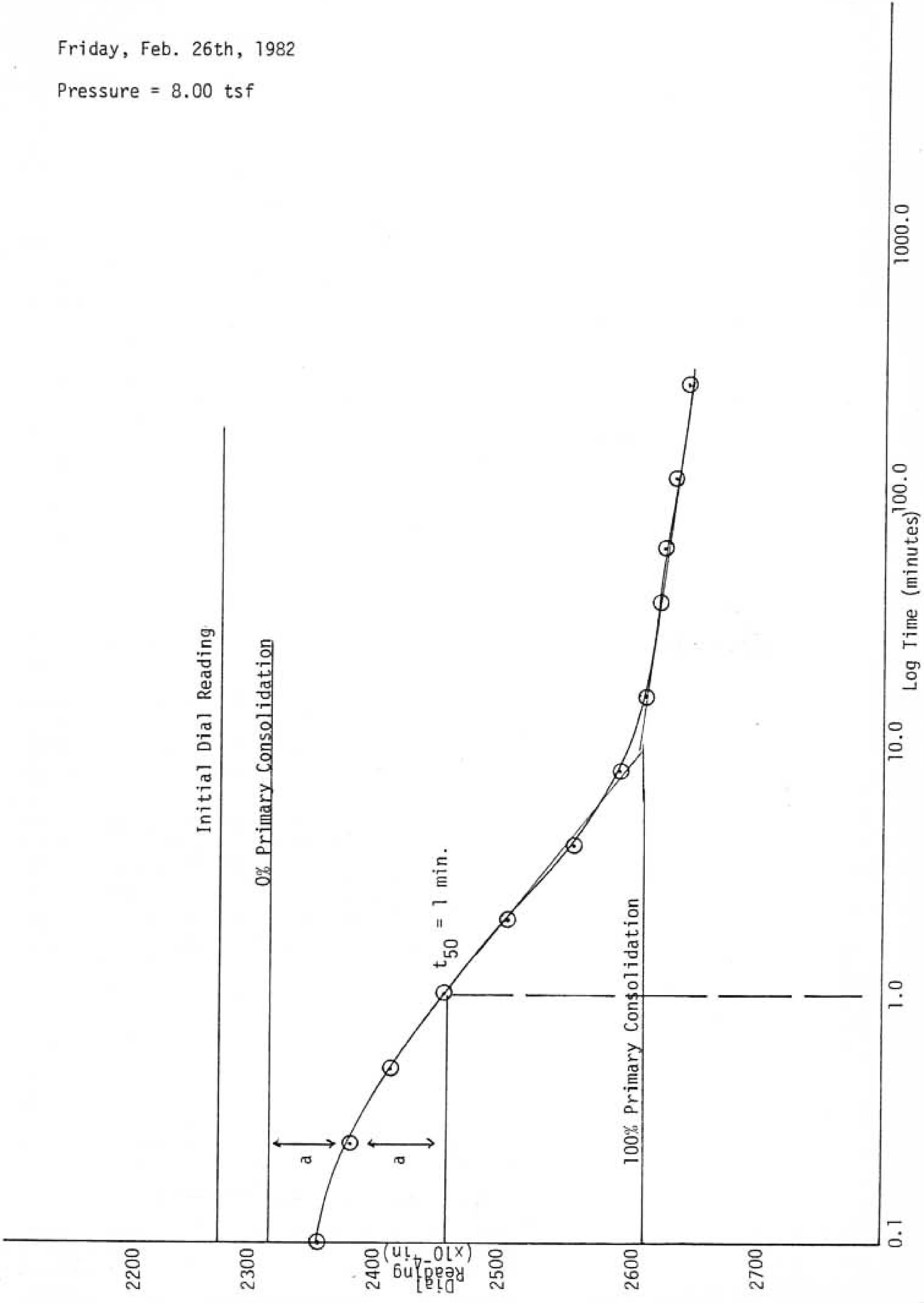


DATA RETRIEVED FROM REGULAR CONSOLIDATION TEST

Date & Pressure	Time of Day	Elapsed Time (min)	Dial Reading by 10^{-4} in	Person Reporting
Friday, Feb. 26th, 1982 8.00 tsf	9:00:00	START	2269	[illegible]
	9:00:06	0.1	2350	"
	9:00:15	0.25	2375	"
	9:00:30	0.50	2408	"
	9:01:00	1.0	2449	"
	9:02:00	2.0	2500	"
	9:04:00	4.0	2554	"
	9:08:00	8.0	2590	"
	9:16:00	16.0	2607	"
	9:37:00	37.0	2621	"
	10:03:00	63.0	2628	"
	11:01:00	121.0	2636	"
	1:28:00	268.0	2644	"
				"
				"

Friday, Feb. 26th, 1982

Pressure = 8.00 tsf



CALCULATION OF COEFFICIENT OF CONSOLIDATION, C_v,
FROM DATA OBTAINED IN REGULAR CONSOLIDATION TEST

Pressure	Void Ratio e_{50}	t_{50} (min)	t_{50} (sec)	$H'=1-\Delta H$ in	H' cm	$C_v = \frac{0.197\ (H'/2)^2}{t_{50}} \times 10^{-3}$ cm^2/sec
0.25	1.19	6.0	360	0.990	2.51	0.862
0.50	1.16	4.0	240	0.977	2.48	1.262
1.00	1.12	3.5	210	0.960	2.44	1.396
2.00	1.08	2.5	150	0.938	2.38	1.860
4.00	1.01	1.9	114	0.909	2.31	2.305
8.00	0.93	1.0	60	0.873	2.22	4.045

TEST No. 1 Centrifugal Test 52

All weights in grams.		BEFORE TEST			AFTER TEST	
		SPECIMEN		TRIMMINGS	SPECIMEN	
Tare + wet soil		5977.0		36.71	1530.0	
Tare + dry soil		-		22.28		
Water	W_w	W_{wo}	923.79	14.43	W_{wf}	475.0
Tare		3909.9		4.42		
Dry Soil	W_s	1143.31		17.86	1055.0	
Water cont.	w	w_o	80.8 %	80.8 %	w_f	45.0 %

Area of specimen, A, sq. cm = 182.4

Height of specimen, H, in = 2.75

Sp. gr. of solids, G_s = 2.65

Height of solids, $H_s = \dfrac{W_s}{A * G_s * \gamma_w} = \dfrac{1143.31}{182.4 * 2.65 * 1 * 2.54} = .93$ in

Original height of water, $H_{wo} = \dfrac{W_{wo}}{A * \gamma_w} = \dfrac{923.79}{182.4 * 1 * 2.54} = 1.99$ in

Final height of water, $H_{wf} = \dfrac{W_{wf}}{A * \gamma_w} = \dfrac{475.0}{182.4 * 1 * 2.54} = 1.025$ in

Net change in height of specimen at end of test, ΔH = 0.92 in

Height of specimen at end of test, $H_f = H - \Delta H =$ 1.83 in

Void Ratio before test, $e_o = \dfrac{H - H_s}{H_s} = \dfrac{2.75 - .93}{.93} = 1.96$

Void Ratio after test, $e_f = \dfrac{H_f - H_s}{H_s} = \dfrac{1.83 - .93}{.93} = 0.97$

Degree of saturation before test, $S_o = \dfrac{H_{wo}}{H - H_s} = \dfrac{1.99}{2.75 - .93} = 109.3\%$

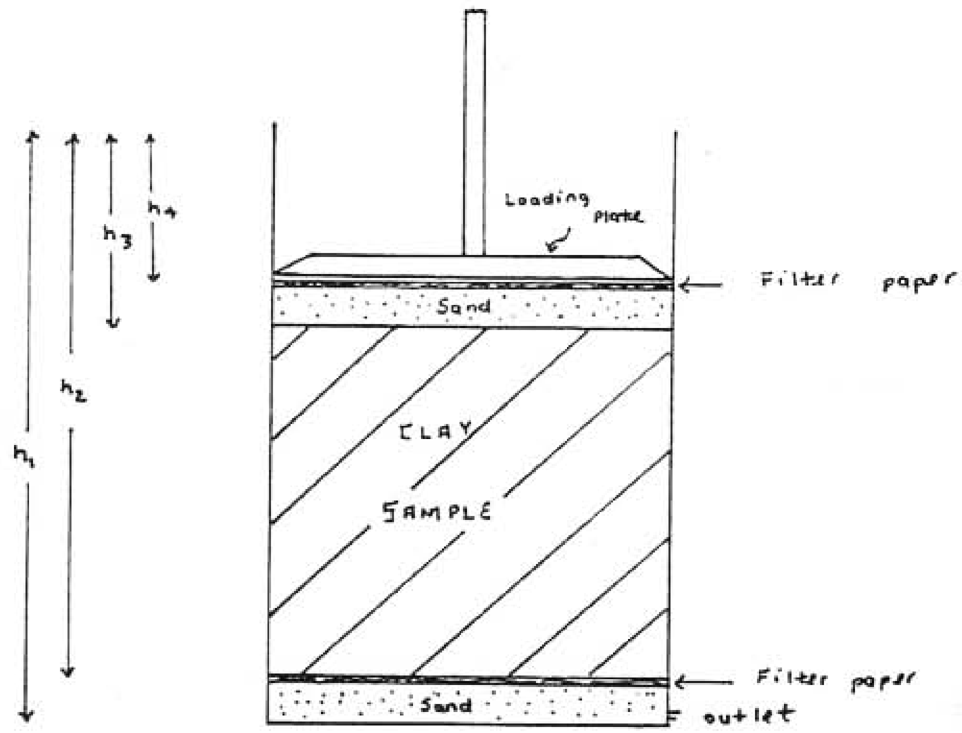
Degree of saturation after test, $S_f = \dfrac{H_{wf}}{H_f - H_s} = \dfrac{1.025}{1.83 - .93} = 113.9\%$

Dry density before test, $\gamma_d = \dfrac{W_s}{H * A} = \dfrac{1143.1 * 62.4}{2.75 * 1824 * 2.54} = 56$ pcf

REMARKS: S is greater than 100% because the sp. gr. is assumed.

Difference in W_s due to errors in estimating weight after consolidation.

PROCEDURE FOR MEASURING HEIGHT OF SAMPLE.



VARIABLE	VALUE
h_1	9.25"
h_2	8.00"
h_3	5.25"
h_4	3.75"

HEIGHT OF SAND AT BASE = $h_1 - h_2$ = 1.25"

HEIGHT OF CLAY = $h_2 - h_3$ = 2.75"

HEIGHT OF SAND SURCHARGE = $h_3 - h_4$ = 1.5"

ACCELERATION FIELD = 80 g's

PRESSURE = 0.81 tsf and 3.1 tsf

Saturated unit weight of sand = 124.3 pcf

Saturated unit weight of clay = 101.5 pcf

Centrifuge Test No. 1

REFERENCE VOLTAGE OF ACCELEROMETER = -21.8 mV
ACCELEROMETER CALIBRATION = 1 mV/g

r - dist. from center of rotation to mid-point of sample.
r′- dist. from center of rotation to accelerometer

Req'd accn. at midpoint of sample	r	r′	Accn at accelerometer	Voltage diff req'd	Voltage Reading
80 g's	43.75"	36.75"	67.2 g's	0.0672 Volts	-0.087 Volts

EQUATION:

$$Y = \frac{r'}{r} X$$

Y - acceleration at accelerometer
X - acceleration at mid-point of sample

Centrifugal Test No. 1

COMPUTATION OF PRESSURE ON MODEL.

Weight of loading plate, W = 0.945 lb   Area of loading plate, A = 0.196 ft^2

TEST NO.	Height of sand above sample.	Specific Weight of sand	Pressure due to sand surcharge	Additional Weight	W + additional weights	Pressure due to total wt.	Total pressure on sample	Accn. Field	Effective Pressure
1[a]	1.5"	.24.3 pcf	15.54 psf	-	0.945 lb	4.82 psf	20.36 psf	80 g's	0.81 tsf
1[b]	1.5"	.24.3 pcf	15.54 psf	11.1 lb	12.045 lb	61.45 psf	77 psf	80 g's	3.1 tsf

TEST NO: 1

ACCELERATION FIELD = 80 g's

17 th Feb. '82

LVDT REFERENCE VOLTAGE = -2.36Volts

LVDT CALIBRATION = 1 volt / 0.21835 inches

$N^2 = 6400$

Pressure = 0.81tsf

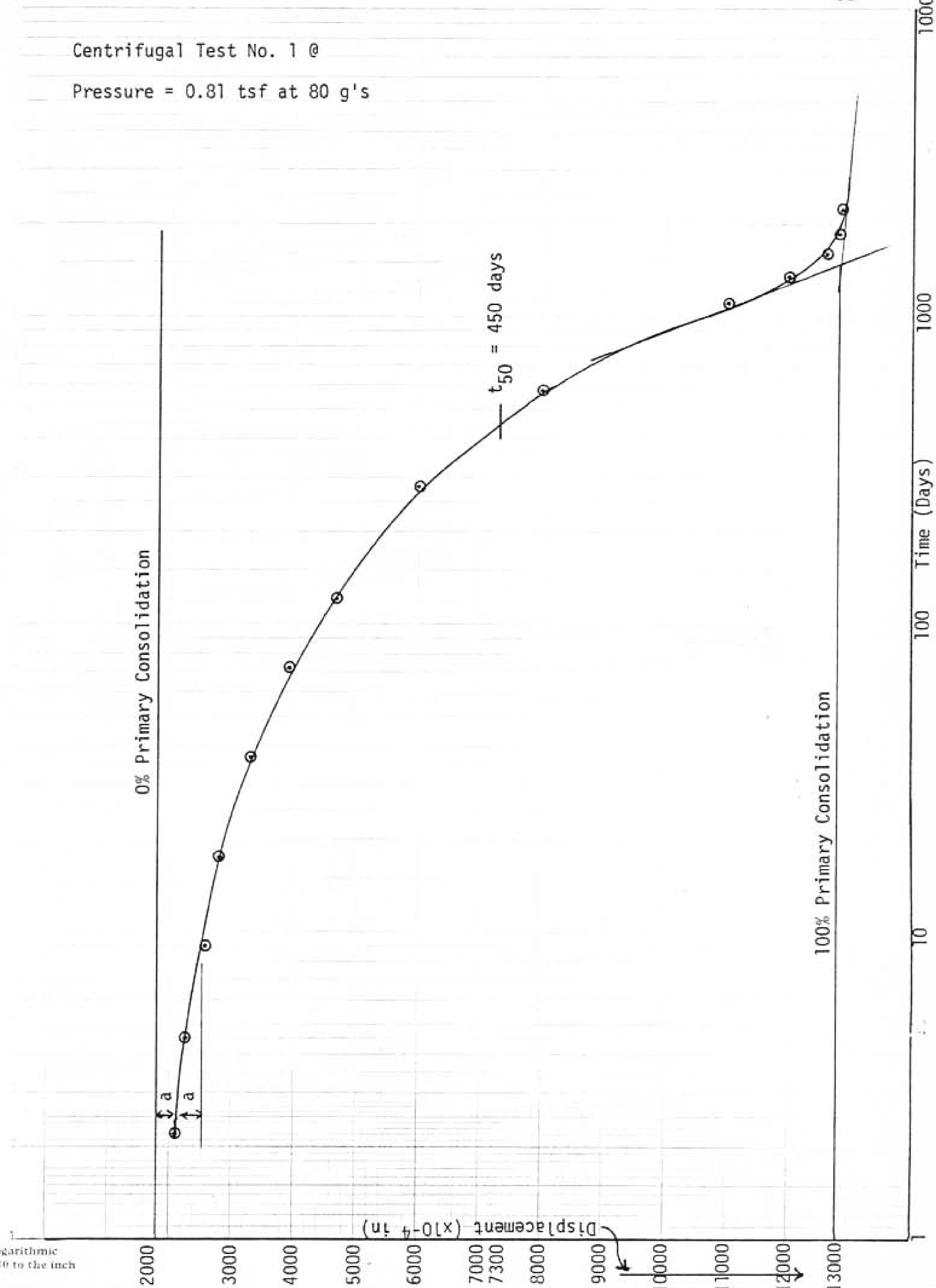
DATE / Pressure on model	TIME OF DAY	ELAPSED TIME	EFFECTIVE TIME days	LVDT Reading Volts	Voltage Difference Volts	DISPLACEMENT by 10^{-4} in	PERSON REPORTING
17 Feb 82	8:00:00	-	-	START			
0.81 tsf	8:00:30	30 sec	2.22 day	-1.40	0.96 V	2096	
	8:01:00	1 min	4.44	-1.31	1.05	2293	
	8:02:00	2 min	8.88	-1.20	1.16	2533	
	8:04:00	4 min	17.78	-1.04	1.32	2882	
	8:08:00	8 min	35.6	-0.85	1.51	3297	
	8:15:00	15 min	71.1	-0.60	1.76	3843	
	8:30:00	30 min	142.2	-0.21	2.15	4695	
	9:00:00	1 hr	284.4	+0.38	2.74	5983	
	10:00:00	2 hr	568.8	+1.30	3.66	7992	
	12:00:00	4 hr	1137.8	+2.69	5.05	11027	
	1:00:00	5 hr	1333.3	+3.14	5.50	12008	

TEST NO: 1 ACCELERATION FIELD = 80 g's 17 th Feb. '82

LVDT REFERENCE VOLTAGE = -2.36 V

LVDT CALIBRATION = 1 volt/ 0.21835 in N^2 =6400 Pressure = 0.81 tsf

DATE / Pressure on model	TIME OF DAY	ELAPSED TIME	EFFECTIVE TIME days	LVDT Reading volts	Voltage Difference volts	DISPLACEMENT by 10^{-4} in.	PERSON REPORTING
CONTINUED							
	2:00:00	6 hr	1600	3.41	5.77	12600	
	3:00:00	7 hr	1866.7	3.50	5.86	12800	
	4:00:00	8 hr	2275.6	3.53	5.89	12860	


Centrifugal Test No. 1 @
Pressure = 0.81 tsf at 80 g's
0% Primary Consolidation
100% Primary Consolidation
t_{50} = 450 days
a
a
Time (Days)
1
10
100
1000
10000
Displacement (x10-4 in)
2000
3000
4000
5000
6000
7000
7300
8000
9000
10000
11000
12000
13000

TEST NO: 1 ACCELERATION FIELD = 80 g's Feb. 18th '82

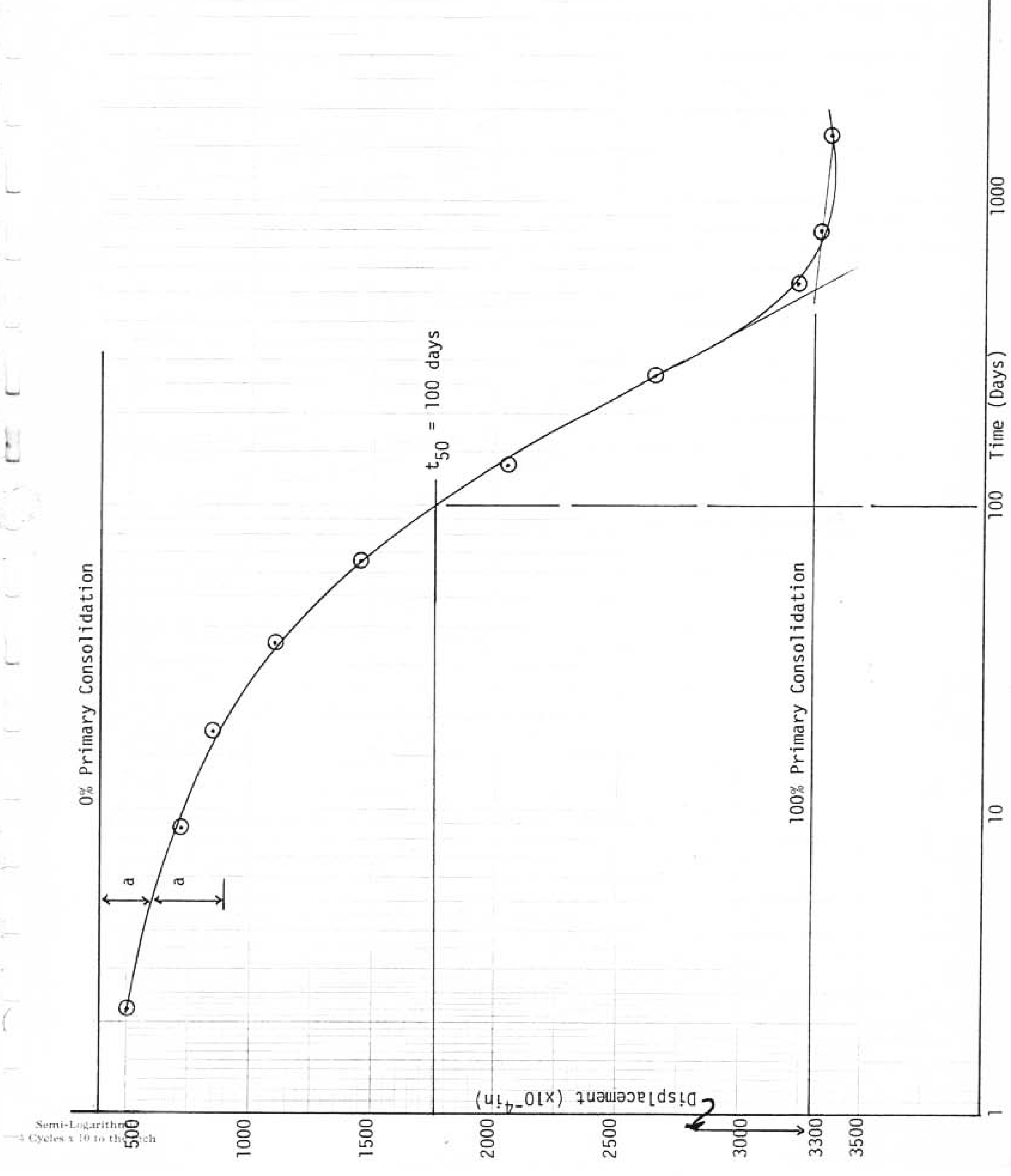
LVDT REFERENCE VOLTAGE = -2.40 V PRESSURE = 3.1 tsf

LVDT CALIBRATION = 0.21835 in/volt $N^2 = 6400$

DATE / Pressure on model	TIME OF DAY	ELAPSED TIME	EFFECTIVE TIME days	LVDT Reading Volts	Voltage Difference Volts	DISPLACEMENT by 10^{-4} in	PERSON REPORTING
3.1 tsf	8:00:00	START					
	8:00:30	30 sec	2.22	-2.17	0.23	502	
	8:01:00	1 min	4.44	-2.12	0.28	611	
	8:02:00	2 min	8.88	-2.06	0.34	742	
	8:04:00	4 min	17.8	-2.01	0.39	852	
	8:08:00	8 min	35.6	-1.90	0.50	1092	
	8:15:00	15 min	66.7	-1.73	0.67	1463	
	8:30:00	30 min	133.3	-1.46	0.94	2052	
	9:00:00	1 hr	266.7	-1.18	1.22	2664	
	10:00:00	2 hr	533.3	-0.92	1.48	3232	
	11:00:00	3 hr	800.0	-0.87	1.53	3341	
	12:00:00	4 hr	1066.7	-0.86	1.54	3363	

Centrifugal Test No 1 b

Pressure = 3.1 tsf at 80 g's



Centrifugal Test No. 1

CONSOLIDATION TEST

Computation of void ratio

Pressure, tsf	Date increment app.	Time increment effective	Displacement by 10^{-4} in	Change in height, ΔH	Height of voids	Void ratio, e
0.81	Feb. 17, '82	2275.6 days	7800	0.78	1.04	1.12
3.1	Feb. 18, '82	1066.7 days	9200	0.92	0.90	0.97
				$C_c = \frac{e_1 - e_2}{\log \frac{P_2}{P_1}} =$	$\frac{1.12 - 0.97}{\log \frac{3.1}{0.81}}$	
				=	0.26	

H_s = 0.93 in H = 2.75"

Height of voids, $H_v = (H - H_s) - \Delta H =$

Void ratio, $e = \frac{H_v}{H_s}$

All weights in grams.		BEFORE TEST			AFTER TEST	
		SPECIMEN		TRIMMINGS	SPECIMEN	
Tare + wet soil		6961 gm.		82.02 gm.	-	
Tare + dry soil		-		45.70 gm.	-	
Water	W_w	W_{wo}	1500 gm.	36.32 gm.	W_{wf}	781 gm.
Tare		3761 gm.		4.54 gm.	-	
Dry Soil	W_s	1700 gm.		41.16 gm.	1705 gm.	
Water cont.	w	w_o	88.2 %	88.2 %	w_f	45.8 %

Area of specimen, A, sq. cm = 182.4

Height of specimen, H, in = 4.75

Sp. gr. of solids, G_s = 2.65

Height of solids, $H_s = \frac{W_s}{A * G_s * \gamma_w} = \frac{1700}{182.4 * 2.65 * 1 * 2.54} = 1.38$ in

Original height of water, $H_{wo} = \frac{W_{wo}}{A * \gamma_w} = \frac{1500}{182.4 * 1 * 2.54} = 3.24$ in

Final height of water, $H_{wf} = \frac{W_{wf}}{A * \gamma_w} = \frac{781}{182.4 * 1 * 2.54} = 1.7$ in

Net change in height of specimen at end of test, ΔH = 1.8 in

Height of specimen at end of test, $H_f = H - \Delta H$ = 2.95 in

Void Ratio before test, $e_o = \frac{H - H_s}{H_s} = \frac{4.75 - 1.38}{1.38} = 2.44$

Void Ratio after test, $e_f = \frac{H_f - H_s}{H_s} = \frac{2.95 - 1.38}{1.38} = 1.14$

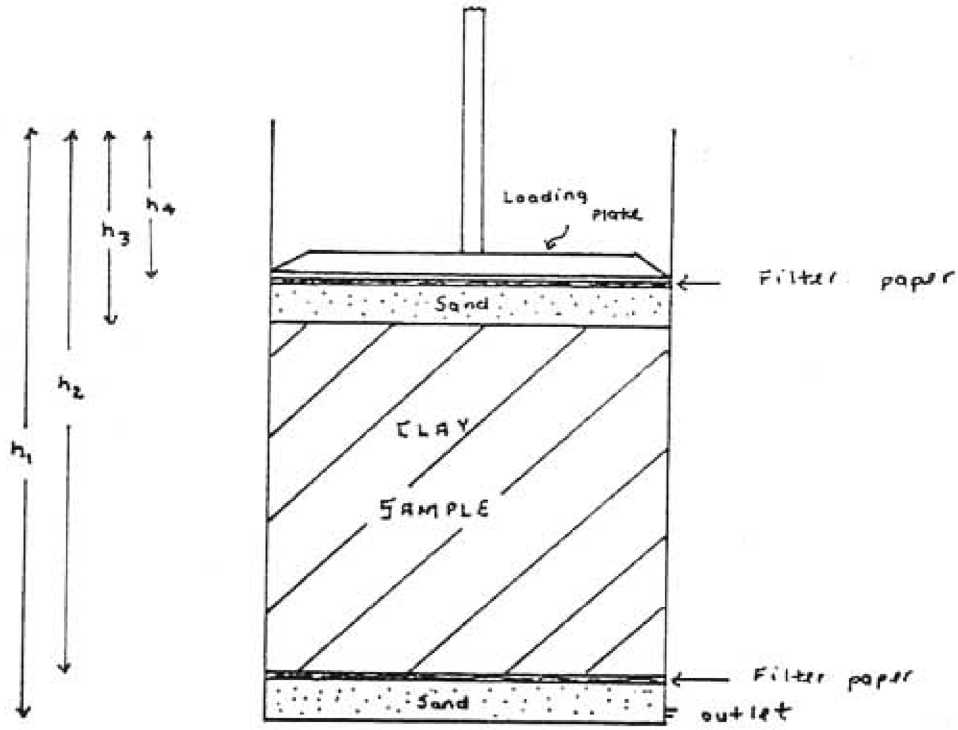
Degree of saturation before test, $S_o = \frac{H_{wo}}{H - H_s} = \frac{3.24}{4.75 - 1.38} = 96\%$

Degree of saturation after test, $S_f = \frac{H_{wf}}{H_f - H_s} = \frac{1.7}{2.95 - 1.38} = 108\%$

Dry density before test, $\gamma_d = \frac{W_s}{H * A} = \frac{1700 * 62.4}{4.75 * 1824 * 2.54} = 48.2$ pcf

REMARKS: Slightly different weights for dry soil.

PROCEDURE FOR MEASURING HEIGHT OF SAMPLE.



VARIABLE	VALUE
h_1	9.25"
h_2	8.25"
h_3	3.5"
h_4	2.75"

HEIGHT OF SAND AT BASE = $h_1 - h_2$ = 1.0"

HEIGHT OF CLAY = $h_2 - h_3$ = 4.75"

HEIGHT OF SAND SURCHARGE = $h_3 - h_4$ = 0.75"

Centrifugal Test No. 2

REFERENCE VOLTAGE OF ACCELEROMETER = -22.6 mV
ACCELEROMETER CALIBRATION = 1 mV/g

r - dist. from center of rotation to mid-point of sample.
r′- dist. from center of rotation to acceleromet

Req'd accn. at midpoint of sample	r	r′	Accn at accelerometer	Voltage diff req'd	Voltage Reading	
46.32 g's	43.25"	39.9"	42.73 g's	0.0427 Volts	-65.3 mV	EQUATION: $Y = \frac{r'}{r} X$
						Y - acceleration at accelerometer X - acceleration at mid-point of sample

Centrifugal Test No. 2
Acceleration = 46.3 g's

COMPUTATION OF PRESSURE ON MODEL.

Weight of loading plate, W = 0.945 lb

Area of loading plate, A = 0.196 ft^2

TEST NO.	Height of sand above sample.	Specific Weight of sand	Pressure due to sand surcharge	Additional Weight	W + additional weights	Pressure due to total wt.	Total pressure on sample	Accn. Field g's	Effective Pressure
2^a	1.0"	130 pcf	10.83 psf	5.05 lb	6.0 lb	30.6 psf	41.4 psf	46.3	0.96 tsf
2^b	1.0"	130 pcf	10.83 psf	13.88 lb	14.8 lb	75.6 psf	86.5 psf	46.3	2.00 tsf
2^c	1.0"	130 pcf	10.83 psf	15.86 lb	16.8 lb	85.7 psf	96.6 psf	46.3	2.24 tsf

Feb. 24th, '82

TEST NO: 2 a

ACCELERATION FIELD = 46.3 g's

PRESSURE = 0.96 tsf

LVDT REFERENCE VOLTAGE = 0.542 Volts

LVDT CALIBRATION = 1 Volt/0.21835 in.

$N^2 = 2143.7$

DATE / Pressure on model	TIME OF DAY	ELAPSED TIME	EFFECTIVE TIME (days)	LVDT Reading (Volts)	Voltage Difference (Volts)	DISPLACEMENT	PERSON REPORTING
2/24/82 0.89 tsf	9:32:00 a.m	-	START				
	9:32:30	30 sec.	0.74	-0.500	0.042	92	
		1 min.	1.49	-0.468	0.074	162	
		2 min.	2.98	-0.400	0.142	310	
		4 min.	5.95	-0.304	0.238	520	
		8 min.	11.91	-0.171	0.371	810	
		15 min.	22.33	+0.049	0.591	1290	
		30 min.	44.66	+0.483	1.025	2238	
		1 hr.	89.32	+1.362	1.904	4157	
		2 hr.	178.64	+2.650	3.192	6970	
		3 hr.	267.96	+3.317	3.859	8426	
	1:51:00	4 hr 19 min	385.57	+4.567	5.109	11156	LVDT stopped and readjusted within acceptable range (i.e. ±2 Volts)

CONTINUED

Feb. 24th, '82

PRESSURE = 0.96 tsf

TEST NO: 2 a

ACCELERATION FIELD = 46.3 g's

LVDT REFERENCE VOLTAGE = 1.776 Volts = 1.12 in. displacement

LVDT CALIBRATION = 0.21835 in/1 Volt

$N^2 = 2143.7$

DATE / Pressure on model	TIME OF DAY	ELAPSED TIME	EFFECTIVE TIME (Days)	LVDT Reading (Volts)	Voltage Difference (Volts)	DISPLACEMENT ($\times 10^{-4}$ in)	PERSON REPORTING
2/24/82 0.89 tsf		5 hrs	446.60	-1.176	0.600	12466	
	3:00:00	5.38 hrs	480.54	-0.943	0.833	12975	
	4.00:00	6.38 hrs	569.86	-0.354	1.422	14261	
	5:00:00	7.38 hrs	659.18	+0.106	1.882	15265	
	6:00:00	8.38 hrs	748.51	+0.461	2.237	16040	
	6:07:00 p.m	8.50 hrs	759.22	+0.490	2.266	16104	
		STOPPED					
	CONTINUED ON THURSDAY, FEB. 25th, 1982						
	7:00:00 a.m.	REFERENCE = -1.550 Volts = 16,104 ($\times 10^{-4}$ in)					
	8:00:00	9.5 hrs	848.54	-1.290	0.260	16672	
	9:00:00	10.5 hrs.	937.86	-1.143	0.407	16993	
	10:00:00	11.5 hrs	1027.18	-1.069	0.481	17154	

CONTINUED

Thursday, Feb. 25th, '82

PRESSURE = 0.96 tsf

TEST NO: 2 a

ACCELERATION FIELD = 46.3 g's

LVDT REFERENCE VOLTAGE = 1.550 Volts = 1.51 in.

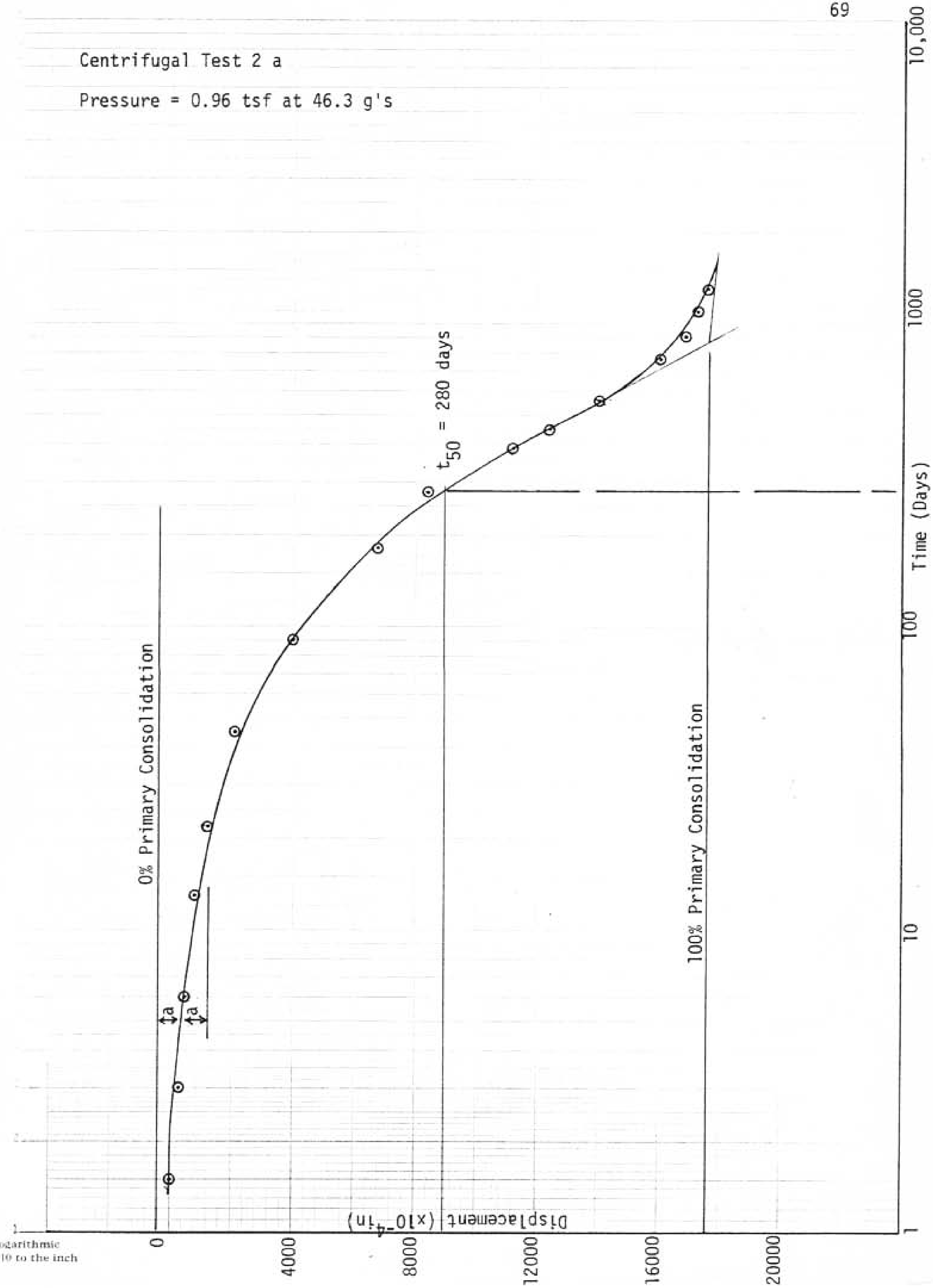
LVDT CALIBRATION =

$N^2 = 2143.7$

DATE / Pressure on model	TIME OF DAY	ELAPSED TIME	EFFECTIVE TIME (Days)	LVDT Reading (Volts)	Voltage Difference (Volts)	DISPLACEMENT	PERSON REPORTING
2/25/82 0.89 tsf	CONTINUED FROM PREVIOUS PAGE						
	11:12:00	12.7 hrs	1134.37	-1.019	0.531	17263	
	12:29:00	14.0 hrs	1250.49	-0.991	0.559	17325	
	1:42:00	15.2 hrs	1357.67	-0.978	0.572	17353	
	2:03:00	15.6 hrs	1393.40	-0.975	0.575	17360	
					END		

Centrifugal Test 2 a

Pressure = 0.96 tsf at 46.3 g's



Friday, Feb. 26th, '82

TEST NO: 2 b ACCELERATION FIELD = 46.3 g's PRESSURE = 2.00 tsf

LVDT REFERENCE VOLTAGE = -1.278 Volts

LVDT CALIBRATION = 1 Volt/0.21835 in. $N^2 = 2143.7$

DATE / Pressure on model	TIME OF DAY	ELAPSED TIME	EFFECTIVE TIME (Days)	LVDT Reading (Volts)	Voltage Difference (Volts)	DISPLACEMENT $\times 10^{-4}$ in	PERSON REPORTING
2/26/82 1.85 tsf	8:31:00		START				
	8:31:15	15 sec	0.37	-1.147	0.131	286	
	8:32:00	1 min	1.49	-1.041	0.237	517	
	8:34:00	2 min	2.98	-0.980	0.298	650	
	8:38:00	7 min	10.42	-0.897	0.381	832	
	8:46:00	15 min	22.33	-0.793	0.485	1059	
	9:01:00	30 min	44.66	-0.664	0.614	1341	
	9:31:00	1 hr	89.32	-0.500	0.778	1699	
	10:31:00	2 hr	178.64	-0.309	0.969	2116	
	11:31:00	3 hr	267.96	-0.207	1.071	2339	
	CENTRIFUGE STOPPED	FOR ADJUSTMENTS	15 minutes				
	1:46:00	5 hr	446.60	-0.110	1.168	2550	CONTINUED

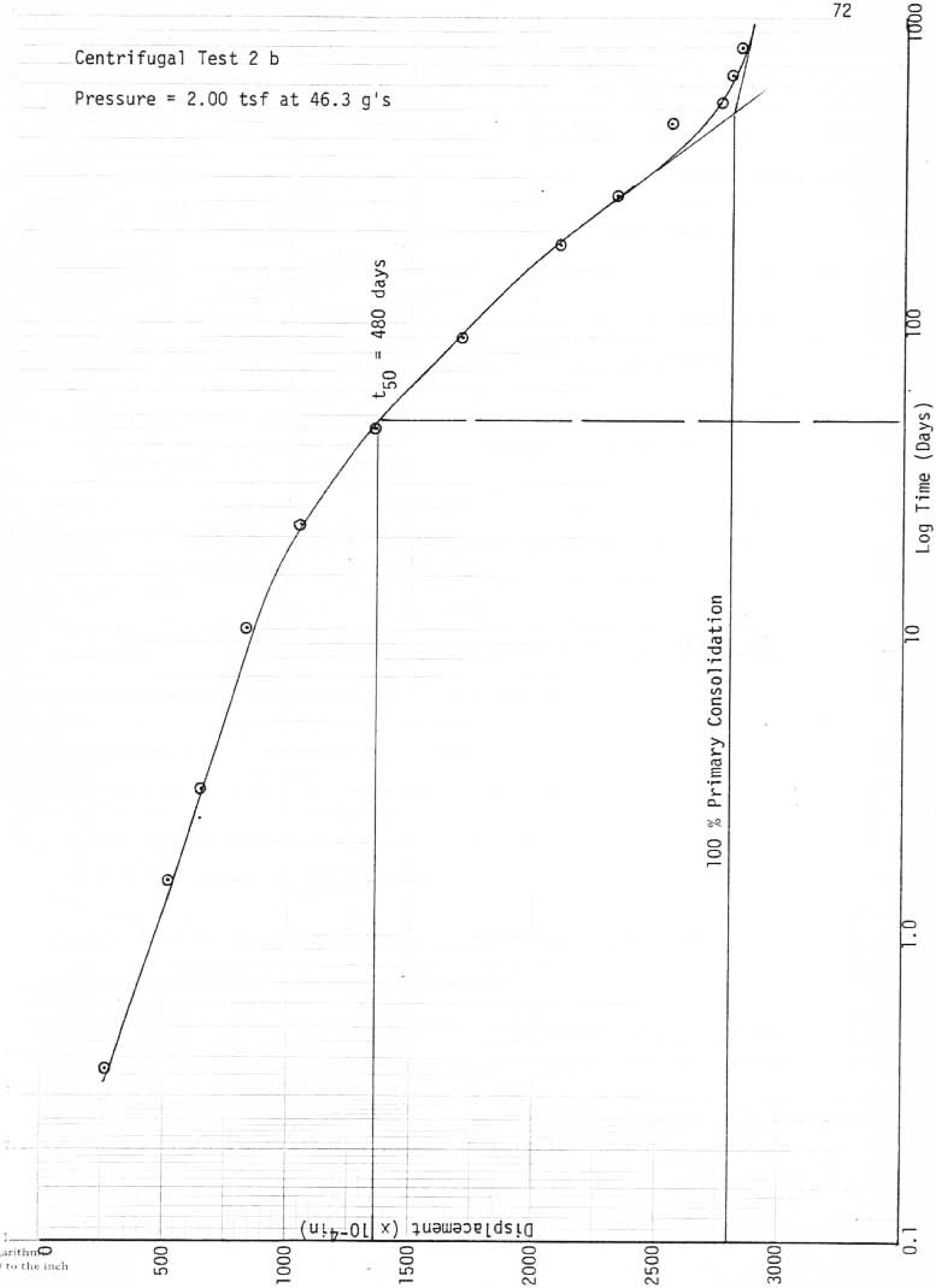
Friday, Feb. 26th, '82

TEST NO: 2 b (continued) ACCELERATION FIELD = 46.3 g's PRESSURE = 2.00 tsf

LVDT REFERENCE VOLTAGE = -1.278

LVDT CALIBRATION = 1 Volt/0.21835 in N^2 = 2143.7

DATE / Pressure on model	TIME OF DAY	ELAPSED TIME	EFFECTIVE TIME (Days)	LVDT Reading (Volts)	Voltage Difference (Volts)	DISPLACEMENT $\times 10^{-4}$ in	PERSON REPORTING
2/26/82 1.85 tsf	CONTINUED						
	2:46:00	6 hr	535.93	-0.026	1.252	2734	
	4:06:00	7 1/3 hr	654.72	-0.004	1.274	2782	
	4:46:00	8 hr	714.57	-0.000	1.278	2791	
	5.21:00	8.58 hr	766.37	-0.002	1.280	2795	


Centrifugal Test 2 b
Pressure = 2.00 tsf at 46.3 g's
t_{50} = 480 days
100 % Primary Consolidation
Log Time (Days)
Displacement (x 10^{-4}in)
0
500
1000
1500
2000
2500
3000
0.1
1.0
10
100
1000

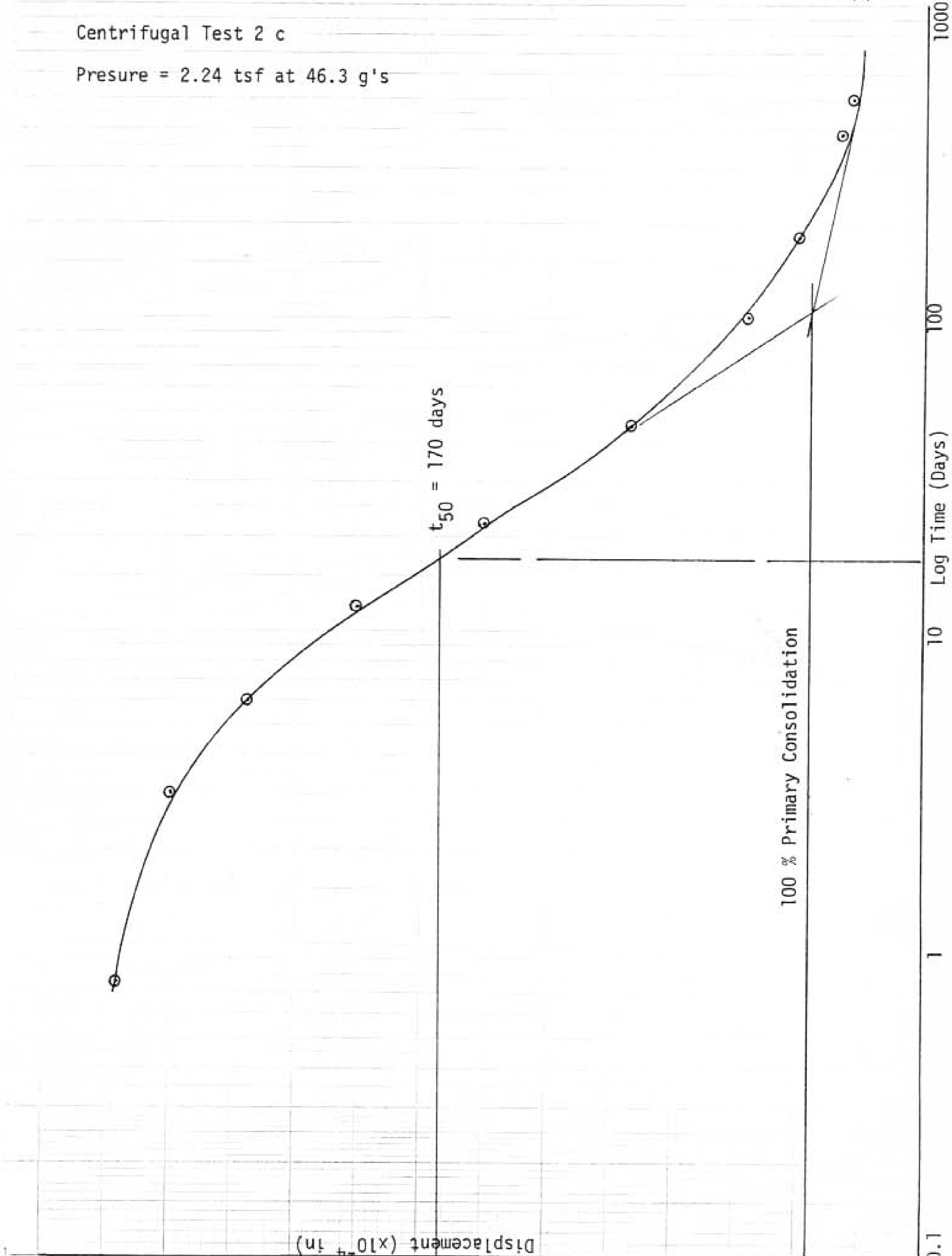
Monday, March 1st, '82

TEST NO: 2 c — ACCELERATION FIELD = 46.3 g's — PRESSURE = 2.24 tsf

LVDT REFERENCE VOLTAGE = +0.410

LVDT CALIBRATION = 1 Volt/0.21835 in. — N^2 = 2143.7

DATE / Pressure on model	TIME OF DAY	ELAPSED TIME	EFFECTIVE TIME (Days)	LVDT Reading (volts)	Voltage Difference (Volts)	DISPLACEMENT x 10^{-4} in.	PERSON REPORTING
3/1/82 2.06 tsf	8.30:00 a.m	START					
	8:30:30	30 sec.	0.74	+0.439	0.029	63	
	8:32:00	2 min	2.98	+0.458	0.048	105	
	8:34:00	4 min	5.95	+0.485	0.075	164	
	8:38:00	8 min	11.91	+0.525	0.115	251	
	8:45:00	15 min	22.33	+0.571	0.161	352	
	9:00:00	30 min	44.66	+0.624	0.214	467	
	9:37:00	67 min	99.74	+0.666	0.256	559	
	10:30:00	2 hr	178.64	+0.684	0.274	598	
	11:30:00	3 hr	267.96	+0.693	0.283	618	
	12:38:00 p.m	4 hr 8 min	369.19	+0.700	0.290	633	
	1:15:00 p.m	5 hr 15 min	468.93	+0.706	0.296	646	



Semi-Logarithmic
Cycles x 10 to the inch

Centrifugal Test No. 2

Acceleration = 46.3 g's

CONSOLIDATION TEST

Computation of void ratio

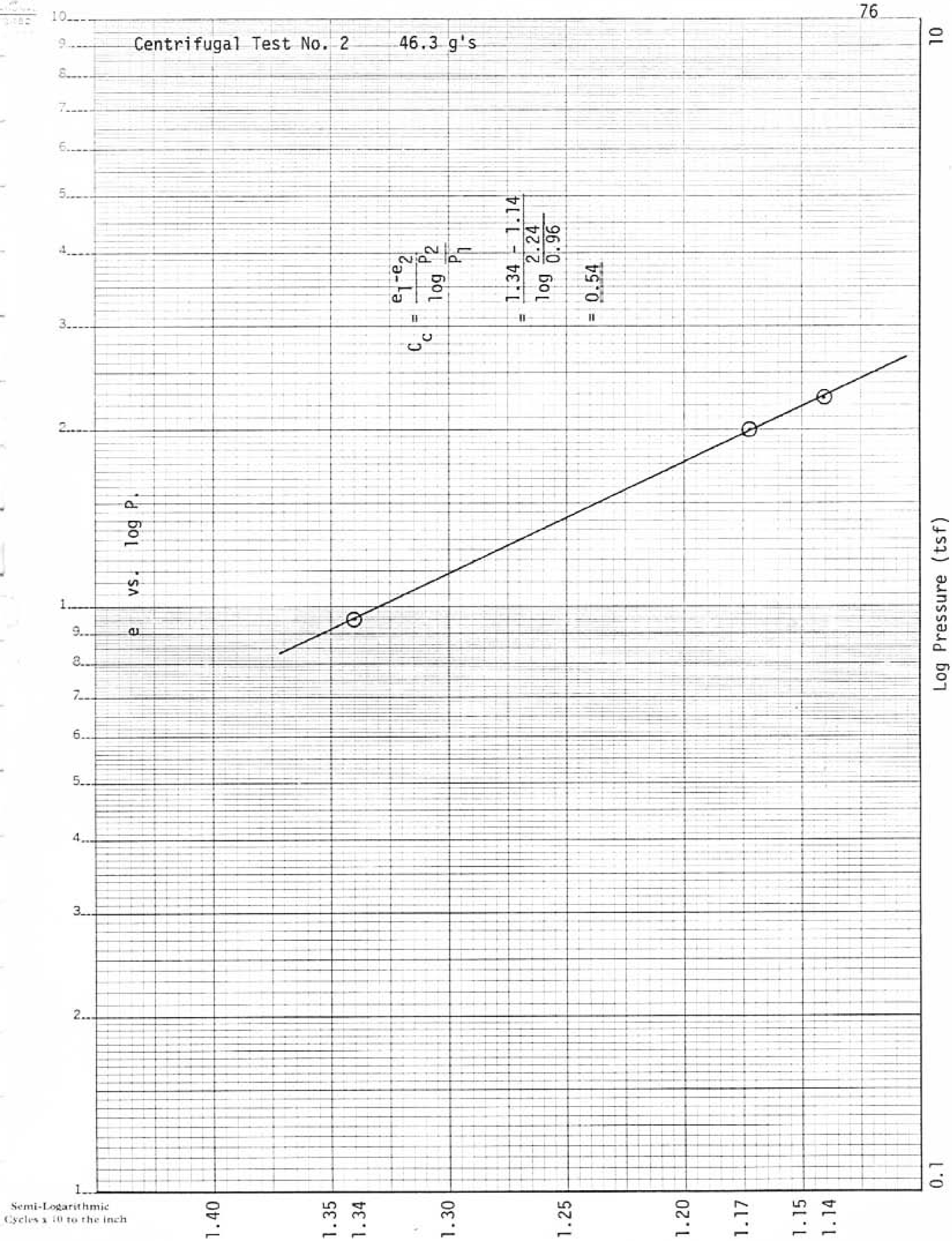
Pressure, tsf	Date increment app.	Time increment effective	Displacement by 10^{-4} in	Change in height, ΔH	Height of voids	Void ratio, e e_{100}
0.96	2/24/82 and 2/25/82	1187 days	15,200	1.52"	1.85"	1.34
2.00	2/26/82	653 days	17,600	1.76"	1.61"	1.17
2.24	3/1/82	400 days	17,980	1.798"	1.57"	1.14

H = 4.75"

H_s = 1.38 in

Height of voids, $H_v = (H - H_s) - \Delta H =$

Void ratio, $e = \dfrac{H_v}{H_s}$


Centrifugal Test No. 2 46.3 g's
e vs. log P.
$C_c = \frac{e_1 - e_2}{\log \frac{P_2}{P_1}} = \frac{1.34 - 1.14}{\log \frac{2.24}{0.96}} = \underline{0.54}$
Log Pressure (tsf)
10
0.1
1.40
1.35
1.34
1.30
1.25
1.20
1.17
1.15
1.14
Semi-Logarithmic
Cycles x 10 to the inch

EVALUATION OF DATA FOR CENTRIFUGAL TESTS

Using the displacement values from LVDT readings to examine the consolidation of the clay layer causes several problems, and this is best illustrated by doing an example.

Consider centrifugal test No. 1;

Initial height of clay layer = 2.75"

Displacement at end of first loading = 1.286" (0.81 tsf)

Displacement at end of second loading = 0.336" (3.1 tsf)

These two displacements were determined from the strain meter (LVDT) readings and represents a total change in height of 1.622 inches (1.286 + 0.336).

The final height of the specimen from this data would be

$$H_f = H - \Delta H = 2.75 - 1.622$$
$$= 1.128 \text{ inches}$$

(Please note that the actual measured height of the clay layer at the end of the test was 1.83 inches).

The height of solids in the clay layer, H_s = 0.93 inches (calculated previously)

also the final height of water, H_{wf} = 1.025 inches

Therefore,

Degree of saturation after test, $S_f = \frac{H_{wf}}{H_f - H_s} = \frac{1.025}{1.128 - 0.93} \times 100\%$

$= 517.7\%$

The maximum theoretical value of the degree of saturation is 100% but a range of 85%-115% is acceptable for practical testing situations. This shows that the LVDT displacement values must be adjusted to represent the actual settlement of the clay layer.

There was one major problem in analyzing the data relating the settlement of the clay layer with time: the displacement computed from the LVDT measured the net change in height of the model and not the actual settlement of the clay layer. In each case the recorded displacement was greater than the actual displacement experienced due to consolidation of clay. There are several reasons proposed to explain this increase in observed settlement and they are presented in the following list:

1. Both sand layers (i.e. top and bottom) shifted into denser configurations on application of the loads, and this accounted for a portion of the settlement. It should be noted that some of the settlement of the sand is permanent while some is due to elastic compression of the sand particles. This means that a portion of the sand settlement rebounds upon removal of the load. The various loadings for each test were applied on different days so the sand and clay had sufficient time to rebound overnight when the centrifuge was stopped. This results in a duplication of observed readings since the elastic compression of the sand would be picked up agian on the strain meter. This effect was reduced in the second centrifugal test (i.e. 46.3 g's) because the reference voltage was not taken immediately upon load application but after a short period of time judged to be sufficient for adequate seating.
2. The LVDT (or strain gauge) had to be reset on several occasions when the voltage output was "out of range" (i.e. greater than or less than ±2 volts for this particular LVDT). There is a good possibility that

rebound and minor discrepancies due to resetting the LVDT accounted for larger observed displacements than actually occurred.

3. A vacuum was not available for the preparation of the clay mixture, and this mixture was also "spooned" into the test bucket; this probably resulted in the introduction of some air spaces which were "flushed out" upon application of the load. However, the author exercised great care in the placement of the saturated clay, and the displacement errors described above were probably kept at a minimum.

The following system was devised from judgement to exclude extraneous readings from the settlement of the clay: Suppose the strain recorded from the LVDT for an entire test was 17000 (x 10^{-4} in), but the actual settlement of the clay layer was 14000 (x 10^{-4} in) or 1.4 inches, this meant that 3000 (x 10^{-4} in) of displacement would have to be removed from the observed data in order to exactly represent the settlement of the clay. It would seem obvious that most of this "seating" settlement took place during the first loading of each centrifuge test, and this occurred in the early stages of the test. The latter portion of each settlement (vs log time) curve was left intact since consolidation of clay would have been the only source of settlement at this stage of the test.

Referring to the example above in which 3000 (x 10^{-4} in) of displacement would have to be "chopped off"; the procedure is as follows:

a. remove <u>70%</u> of this 3000 (x 10^{-4} in) from the upper portion of the <u>first</u> settlement data,
b. remove <u>30%</u> (if only 2 tests were done) from the upper portion of the <u>second</u> settlement data,
c. remove <u>20%</u> (if 3 tests were done) from the upper portion of the <u>second</u> settlement data, and <u>10%</u> from the <u>third</u> loading that produces consolidation.

With reference to the 1st centrifugal test (i.e. 80 g's):

2 loads were used: 0.81 tsf and 3.1 tsf

Settlement of model as recorded from LVDT = 1.2850 + .3363

= 1.622 inches

Actual settlement of clay layer = 0.92 inches

Therefore the settlement to be removed from data to represent the settlement of the clay layer only = 1.62 - 0.92

= 0.70 inches

or 7000 (x 10^{-4} in)

70% of 7000 ≈ 5000

30% of 7000 ≈ 2000

Therefore 5000 (x 10^{-4} in) of displacement would be removed from the upper portion of the first settlement (vs log time) curve.

From the observed data; 100% primary consoldiation occurred at a displacement of 12,800 (x 10^{-4} in).

∴ 100% primary consolidation for calculation purposes = 16160 - 7000

= 9160 x 10^{-4} in

= <u>0.916 in</u>

With reference to the 2nd centrifugal test (46.3 g's):

3 loads were used: 0.96, 2.00, and 2.24 tsf

Total settlement observed from strain gauge (i.e. LVDT) = 20,801 x 10^{-4}in

Actual settlement of clay layer = 18,000 x 10^{-4}in

Therefore settlement not affiliated with clay layer = (20.8-18) x 10^{-2}

= 0.28 inches

70% of 2800 ≈ 2000

20% of 2800 ≈ 600

10% of 2800 ≈ 200

Observed displacement for 100% primary consolidation in 1st test $= 17360 \times 10^{-4}$ in.

Displacement for 100% primary consolidation for computational purposes $= 17,200 - 2000$

$= 15,200 \times 10^{-4}$ in

= 1.52 inches (for 0.96 tsf)

Observed displacement for 100% primary consolidation in 2nd test $= 18,200 \times 10^{-4}$ in.

Displacement for 100% primary consolidation for computational purposes $= 18,200 - 600$

$= 17,600 \times 10^{-4}$ in

= 1.76 inches (for 2.00 tsf)

Observed displacement for 100% primary consolidation in 3rd test $= 18,180 \times 10^{-4}$ in.

Displacement for 100% primary consolidation for computational purposes $= 18,180 - 200$

$= 17,980 \times 10^{-4}$ in

= 1.798 inches (for 2.24 tsf)

SUMMARY

Test No.	Accn in g's	Original height of clay	Load (tsf)	Displacement for 100% primary consolidation
1 a	80	2.75"	0.81	0.78"
1 b	80	2.75"	3.1	0.92"
2 a	46.3	4.75"	0.96	1.52"
2 b	46.3	4.75"	2.00	1.76"
2c	46.3	4.75"	2.24	1.80"

Calculation of the coefficient of consolidation for the centrifugal tests.

CENTRIFUGAL TEST NO. 1

Pressure tsf	Void Ratio e_{50}	t_{50} (days)	t_{50} (sec)	$H'=H-\Delta H$ (in)	H' cm	$C_v = \frac{0.197(H'/2)^2}{t_{50}}$ cm^2/sec x 10^{-3}
0.81 tsf	1.71	450	450x24x 60x60	(2.75-0.23)80	512	0.33
3.1 tsf	1.10	100	100x24x 60x60	(2.75- 0.80)80	396	0.90

CENTRIFUGAL TEST NO. 2

Pressure tsf	Void Ratio e_{50}	t_{50} (days)	t_{50} (sec)	$H'=H-\Delta H$ (in)	H' cm	$C_v = \frac{0.197(H'/2)^2}{t_{50}}$ cm^2/sec x 10^{-3}
0.96	1.89	280	280x24x 60x60	(4.75- 0.76)46.3	469	0.45
2.00	1.25	480	480x24x 60x60	(4.75- 1.64)46.3	366	0.16
2.24	1.15	170	170x24x 60x60	(4.75- 1.78)46.3	349	0.41

SUMMARY OF RESULTS

Test Description	Pressure tsf	Compression Index, C_c	Coefficient of Consolidation C_v ($\times 10^{-3} cm^2/3$)	Time for 50% primary consolidation, t_{50}	Effective Initial height
Centrifugal Test No. 1 at 80 g's	0.81	0.26	0.33	450 days	18.33 feet
	3.1		0.90	100 days	
Centrifugal Test No. 2 at 46.3 g's	0.96	0.54	0.45	280 days	18.33 feet
	2.00		0.16	480 days	
	2.24		0.46	170 days	
Regular consolidation test according to EM 1110-2-1906 Appendix VIII	0.25	0.27	0.862	6.0 min	1.00 inches
	0.50		1.262	4.0 min	
	1.00		1.396	3.5 min	
	2.00		1.860	2.5 min	
	4.00		2.305	1.9 min	
	8.00		4.045	1.0 min	

DISCUSSION OF RESULTS

a. Pressure on model

It was not possible to achieve a wide range of pressures on the centrifuge models because of the lack of clearance necessary to install additional weights. The maximum stress possible was 3.1 tsf, and this occurred in the test at 80 g's. The regular consolidation test was performed on a sample extruded from the first centrifugal test so a preconsolidation pressure of approximately 3.1 tsf was expected. However, the value obtained for the preconsolidation pressure was closer to 2.00 tsf, indicating that there may have been considerable losses (ex. friction) in load transference to the clay layer. The original intention was to use a maximum of 16 tsf in the regular consolidation test, but this idea was abandoned when some soft clay was starting to seep through the sides of the porestones at a pressure of 8 tsf. The result was that the e vs. log P virgin portion of the curve was not properly estimated, and this may have resulted in an erroneous value of the preconsolidation pressure.

b. Compression Index, C_c.

These values (i.e. 0.26 & 0.54 for the centrifugal tests and 0.27 for the regular test) were very consistent when the broad assumptions used in their computation are considered. Compression index values (from Lambe and Whitman) range from 0.21 to 0.60 for Kaolinite, and this is good evidence that the centrifugal tests did provide reliable data on consolidation.

c. Coefficient of consolidation, C_v

Typical values for coefficient of consolidation (U.S. Navy, 1962)

Liquid Limit	Lower Limit For Recompression, cm^2/sec	Undisturbed Virgin Compression, cm^2/sec	Upper Limit Remolded cm^2/sec
30	3.5×10^{-2}	5×10^{-3}	1.2×10^{-3}
60	3.5×10^{-3}	1×10^{-3}	3×10^{-4}
100	4×10^{-4}	2×10^{-4}	1×10^{-4}

The liquid limit of the Kaolinite used in this test was approximately 55% so the range of values should lie between 3.5×10^{-3} and 3×10^{-4} cm^2/sec. The centrifugal tests were performed on undisturbed (virgin compression) samples, and the average value of the coefficient of consolidation was 0.46×10^{-3} cm^2/sec and the value predicted by the chart above for a Kaolinite of this plasticity is 1.00×10^{-3} cm^2/sec. This correlation is good, and enhances the respectability of the centrifugal testing theory that relates the modelling factor N to the primary flow processes of pore water, i.e. $\frac{\text{Time in model}}{\text{Time in prototype}} = \frac{1}{N^2}$

The results of the regular consolidation test shows a steady increase in C_v (coefficient of consolidation) with increasing pressure. The average value in the regular test was 2×10^{-3} cm^2/sec. which is twice as large as the value for the centrifugal tests, but it should be noted that this value is for a preconsolidated sample and therefore relates more to the value of 3.5×10^{-3} cm^2/sec. for recompression. Terzaghi's theory of consolidation assumes that the permeability remain constant, but in actuality, the coefficient of permeability (K) decreases as the void ratio decreases. The value of C_v should decrease as the regular test proceeds with increasing pressures, but the results show that the value of C_v increases leaving some question of doubt. One of the major contributors to this discrepancy may have been the escape of some of the clay which would distort the time for 50% primary consolidation.

d. Time for 50% primary consolidation

Terzaghi's theory also suggests that the time for a certain percent of primary consolidation has no relationship to the applied pressure.

$$t = \frac{T\ (H/2)^2}{C_v}$$

The height of the layer, H, and the time factor, T, are both constants, but the value of C_v is assumed constant with different pressures. This assumption would make the time, t, independent of the pressure (or load), but as shown from the results, the value of C_v varies with pressure, and Terzaghi's theory may not be broad enough to account for these deviations. The times for 50% primary consolidation (hereafter referred to as t_{50}) in the primary consolidation test varies from a high of 480 days to a low of 100 days (occurs at 46.3 and 80 g's). The large t_{50} value in the first test (450 days) may be somewhat exaggerated because a proper procedure to allow for seating and other minor errors had not been used. The second test was more "professionally" handled than the first because of the experience gained during the first few days of testing. The results of the t_{50} values on the second test are more consistent and reflects improved procedural efforts. Nevertheless, a general trend was observed in both the centrifugal and regular tests: the time for consolidation (50% or 100%) decreased with increasing pressure, and this is not supposed to happen under the principles of Terzaghi's theory.

e. Double Drainage

The coefficient of consolidation values in the regular consolidation test was considerably higher than the values in the centrifugal tests. The difference between double and single drainage can have a significant effect on the C_v values;

$$C_{v_1} = \frac{T\,(H/2)^2}{t} \qquad \text{(double drainage)}$$

$$C_{v_2} = \frac{T\;H^2}{t} \qquad \text{(single drainage)}$$

$$\frac{C_{v_1}}{C_{v_2}} = 1/4$$

$$C_{v_2} = 4\;C_{v_1}$$

This shows that C_v values would be 4 times too small if double drainage was assumed to occur but the actual effect was single drainage. There is good reason to believe that single drainage may have occurred in some instances since there was always a considerable quantity of water above the model whereas the bottom drainage port may have been clogged with sand.

CONCLUSIONS

Centrifugal testing is a viable method of researching prototype behavior of clay layers with proper model preparation and testing procedures. The modelling factors proposed to relate the theory of consolidation in large acceleration fields produced by rotation in the centrifuge to the standard laboratory tests under the earth's gravitational field, are fairly accurate. It was not the purpose of this report to suggest any revised theories on modelling factors in the centrifuge, but to critically examine them. Therefore, one of the conclusions is that these factors served the purpose of observing consolidation during centrifugal testing, but in no way verifies them as being 100% accurate. Considering the margin for error introduced by the assumption in data evaluation, the correlation of the parameters, C_v and C_c, were excellent. The time for 50% primary consolidation displayed a general trend that was not accounted for in Terzaghi's theory. This is not meant as a degradation of the theory of consolidation proposed by the "father of soil mechanics", Karl Terzaghi, but it shows that his theory provides a good foundation upon which to develop a more "accurate" theory of consolidation. Such a statement may seem ludicrous when the non-homogeneity of subsurface conditions is considered, and the apparent wasted effort that may go into computing "accurate" parameters which serve only as a guide to the rate and quantity of settlement of a clay layer.

RECOMMENDATIONS

(to anyone who may wish to improve the procedure in this test)

1. The sand surcharge is very inconvenient as a method of adding weight and preventing loss of the clay through seepage along the sides of the bucket. A better model would have consisted of a water-tight piston with some well spaced holes for drainage. The use of a sturdy filter fabric (not filter paper) at every interface is recommended. Every effort should be made to make this piston as frictionless as possible, for instance, a rubber "O" ring can be precision fitted to ensure minimum friction and water-tightness.
2. The clay layer settled unevenly because the bucket did not swing up "perfectly" horizontally, and different parts of the clay layer were subjected to different accelerations. The weights used to exert the pressure on the model shifted the moment of inertia of the bucket and its contents in such a manner that made it difficult to align horizontally during rotation. Air pressure would be considered a more suitable method of increasing stress since it would be easier to measure accurately and adjust the pressures. A device (ex. a rod placed through the housing) should be used to make sure the bucket is horizontal during testing; this procedure should eliminate the uneven settlement.
3. A recorder should be used to collect data since it frees the technician for other duties. A pore pressure transducer connected to the base of the bucket would indicate when 100% primary consolidation has been achieved.

4. A vacuum should be used in the preparation of the clay mixture since this would reduce air voids which introduce errors in the results.

5. The water that collects above the loading plate during consolidation may add a small but significant surcharge if not promptly removed. For example, a 1-inch layer of water at 80 g's creates a pressure of 0.21 tsf which could alter the settlement especially at low pressures. This water can be removed by placing a wick on the loading plate or by using some sort of suction device capable of removing such small quantities of water.

6. The sand layer at the base of the model could be replaced by a rigid porous stone which negates the effect of sand settlement. The outlet at the base of the bucket should be permanently covered with a #200 sieve mesh to prevent clogging with fine particles.

NUMERICAL REFERENCE LIST

1. "Centrifuge Model Testing of Soils: A Literature Review" by Mosaid Al-Hussaini, U.S. Army Engineers Waterway Experiment Station, p. 9.

2. "Centrifuge Model Testing of Soils: A Literature Review" by Mosaid Al-Hussaini, U.S. Army Engineer Waterways Experiment Station, June 1976, p. 27.

3. "Centrifuge Test Results on Bearing Capacity of Layered Soil" - Masters Report by Arody Herdy, Fall 1980, University of Florida, p. 27.

4. Lambe and Whitman, Soil Mechanics, 1969, John Wiley & Sons, Inc. p. 322.