REPEATED LOADING OF NORMALLY CONSOLIDATED CLAY

REPEATED LOADING OF NORMALLY CONSOLIDATED CLAY

BY

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A Thesis

Submitted to the School of Graduate Studies

in Partial Fulfilment of the Requirements

for the Degree

Master of Engineering

McMaster University

Sept. 1970

MASTER OF ENGINEERING (1970)
(Civil Engineering)

McMASTER UNIVERSITY
Hamilton, Ontario,
Canada.

TITLE: Repeated Loading of Normally Consolidated Clay

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SUPERVISOR: Mr. N.E. Wilson

NUMBER OF PAGES: x, 104

SCOPE AND CONTENTS:

The effects of repeated loading on a normally consolidated, saturated silty clay, are compared to the effects of sustained loading and standard strength tests on the same material. Attention is given to the axial strains and pore water pressures generated under the different loading conditions.

ACKNOWLEDGEMENTS

The author wishes to express his sincere appreciation to Mr. N.E. Wilson for his assistance and encouragement throughout the research program.

The financial support of the experimental program by the National Research Council of Canada, and the Defence Research Board, is gratefully acknowledged.

The author would like to thank Dr. D.A. Sangrey, of Cornell University, for the valuable discussion and comments on the thesis topic.

The assistance of Mr. William Sherriff with the experimental apparatus, and Mr. Paul Anderson with the preparation of the diagrams for this thesis, is very much appreciated.

Special thanks are due to Glynis J. Greenwood, wife of the author, for her encouragement throughout the research program.

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TERMINOLOGY

ε = Total axial strain

Δu = Total excess pore water pressure

$\Delta\varepsilon_e$ = Component of axial strain recovered on removal of the applied deviator stress.

$\Delta\varepsilon_p$ = Component of axial strain not recovered on removal of the applied deviator stress.

Δu_e = Component of excess pore water pressure recovered on removal of the deviator stress.

Δu_p = Component of excess pore water pressure not recovered on removal of the deviator stress.

σ_1 = Major principal total stress

σ_3 = Minor principal total stress

σ_1' = Major principal effective stress

σ_3' = Minor principal effective stress

$\dfrac{\sigma_1'}{\sigma_3'}$ = Effective stress ratio

DEVIATOR STRESS (STRESS DIFFERENCE) = $(\sigma_1' - \sigma_3') = (\sigma_1 - \sigma_3)$

σ_s = Compressive strength of a sample = Maximum deviator stress ('R'Test)

σ_r = Repeated or sustained deviator stress

ISOTROPIC CONSOLIDATION - $\sigma_1' = \sigma_3'$ during consolidation.

'K_o' CONSOLIDATION - $\sigma_1' \neq \sigma_3'$ during consolidation, $K_o = \dfrac{\sigma_3'}{\sigma_1'}$

'R' Test - Triaxial compression test in which the sample is consolidated then sheared in the undrained state by increasing the axial strain.

EFFECTIVE STRESS STATE - State of effective stresses at a given time.

EFFECTIVE STRESS PATH - Line in effective stress space followed by a soil sample under a particular loading condition.

CHAPTER 1

INTRODUCTION

Highway Engineers have had many problems as a result of repeated loading caused by the intermittent passage of vehicles along a road. Much of the early research on repeated loading was carried out on compacted highway subgrade materials. The concept was introduced of a critical repeated stress level above which the soil would fail. The repeated critical stress was found to be considerably lower than the static load which caused shear failure in the soil.

Other cases of repeated loading may be quoted:- the moving of loads across a factory floor, wind on a structure causing a pressure variation across the foundation, waves and tidal action against waterfront structures,or traffic passing over a support of a bridge. The soils to which these repeated loads are applied may well be saturated. If the soil is undrained the development of pore water pressure is critical to the stability of the soil. In this thesis, repeated loading of clay soils in the undrained state is considered, as the undrained state is generally the the most critical for the stability of the soil.

The development of pore water pressures due to an applied load on a saturated clay has long been a source of discussion, as often pore water pressures develop which are of much greater magnitude than is predicted by Elastic Theory.

Bjerrum in 1961 illustrated an example of slope failure due to excess pore pressures developed during Pile driving. This is a repeated loading effect combined with a forced straining of the soil. In chapter 5 it is shown that the pore water pressures are related to the strain of the soil.

In this thesis a series of tests results is presented which show the effects of repeated load compared with a sustained and gradually increasing load on saturated, undrained clay.

Two different soils were used in the investigation. One is a natural clay from the Hamilton Bay. 4" Piston tube samples were taken from 40ft. depth, and test samples were prepared from these tubes. The clay was reasonably uniform but there were inevitable variations from sample to sample. Because of this, another series of tests was run on a commercial Kaolin clay. These samples were prepared in the laboratory from a Kaolin powder, and there was consequently more consistency between samples.

All testing was carried out on a triaxial testing machine, which was modified for repeated loading. The repeated load was typically of one minute dureation and one minute interval. It was applied instantaneously but without impact.

It was originally intended that a dynamic load (duration 0.2 secs.) would be applied to the soil, but many points of interest arose from the low frequency loading and the author decided not to extend the thesis to dynamic loading until the behaviour of clay under single and repeated load applications is understood.

The effect of repeated loading is studied by comparing the compressive strength of the samples under repeated load, sustained load and gradually increasing load ('R' Test). Pore water pressures and Axial strains are considered with time under repeated and sustained load. The strength of samples strained to failure after a period under repeated loading is compared with the compressive strength of samples not subjected to repeated loading.

An attempt is made to relate the findings of this investigation to cases of practical interest, but the author is very much aware that his experience in soil mechanics practise is limited.

CHAPTER 2

LITERATURE REVIEW

2.1 Repeated Loading Apparatus

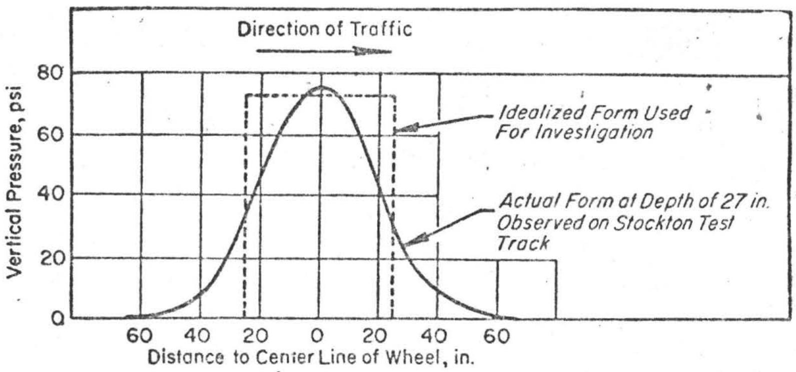
The requirements of a repeated loading apparatus are basically that a repeated stress may be applied to a soil sample in a controlled manner such that there are no impact effects.

The triaxial testing machine is recognised as being the most convenient tool for simulating the effects of repeated loading in the ground. All the previous repeated loading apparatus has utilized a triaxial cell, and the various apparatus is described with reference to a triaxial cell but it might be used equally well on a consolidometer.

H.B. Seed did much of the early research into repeated loading. He developed an apparatus (Seed & Fead/1959) which applied a load to the soil sample by means of air pressure acting on a piston. The flow of air was controlled by an solenoid valve, and with this apparatus he was able to simulate wheel loads moving at about 30 m.p.h. (About 0.1 sec. load duration).

The major disadvantage with this apparatus is that there is very little control of the slope of the loading curve.

The changes in stress on a soil element in the ground due to a moving load have been shown to be of a sinusoidal nature (Seed and Fead 1959) but this apparatus must use the idealized form of a square wave (Fig.1).



(After Seed and Fead 1959)

FIG. 1.—Changes in Stress on Soil Element due to Moving Load.

Normally, when an element of soil is subjected to an increase in deviator stress there is a simultaneous increase in the confining pressure acting on the element. This may be simulated in the laboratory by pulsing the cell pressure as the repeated deviator stress is applied. Although laboratory results have been obtained using a pulsed cell pressure this complicates the apparatus considerably. Seed had problems in controlling the relative rates of increase of the confining pressure and the deviator stress so as to keep the effective stress ratio $(\frac{\sigma_1'}{\sigma_3'})$ constant.

It was decided that no attempt would be made to pulse the cell pressure in the current research, the effect of not pulsing the cell pressure is described in chapter 6

Grainger and Lister (1962) developed a mechanical apparatus to study the behaviour of soils under repeated applications of stresses of the same magnitude as those which occur in road subgrades. The load was applied by a lever arm/spring arrangement. The lever arm was tilted by a rotating cam to apply the load to the cell piston. The duration, interval and loading pattern were controlled by the dimensions of the cam. A compensating device was required to ensure that the lever arm/spring device remained in contact with the sample as deformation of the sample occurred.

The apparatus was bulky, and each of the mechanical components had to have its characteristics taken into account. There would inevitably be a certain amount of 'play' between components.

From past experience it was decided that the only way to provide accurate control over the applied load would be to use a hydraulic ram to apply the force to the piston. The use of a hydraulic oil rather than compressed air would give the system more rigidity and better control. The load applied by the ram could be controlled electronically through a load cell in line with the ram.

Unfortunately there was not enough time available in the current research program to set up the apparatus described, and therefore a simpler mechanical device was constructed as an interim measure (chapter 3).

2.2 Effects of Repeated Loading on Compacted Materials

Much research in the field of repeated loading has concerned the subgrade materials for highways. Most of the early work concentrated on typical compacted materials at various degrees of saturation.

A review of the findings of the researchers is presented here so that a comparison may be made between the results of tests on compacted soil and tests performed by the author on natural saturated clays.

Seed et al (1958) carried out an investigation on the increased resistance to deformation of clay caused by repeated loading. They tested silty clays at various degrees of saturation. The results indicated that a large number of stress applications (deviator stress only) may cause an increase in the resistance to deformation. However it was impossible to predict the amount of settlement that would occur after a particular stress history and no values could be put to the increase in strength. A typical strength increase is shown in Fig.2(a).

Seed thought the increase in shearing resistance might be attributed to changes in moisture distribution in the sample, or structural arrangement of the soil by the extrusion of absorbed water. The author suggests another mechanism which may cause the strength increase observed by Seed (Chapter 5.6.).

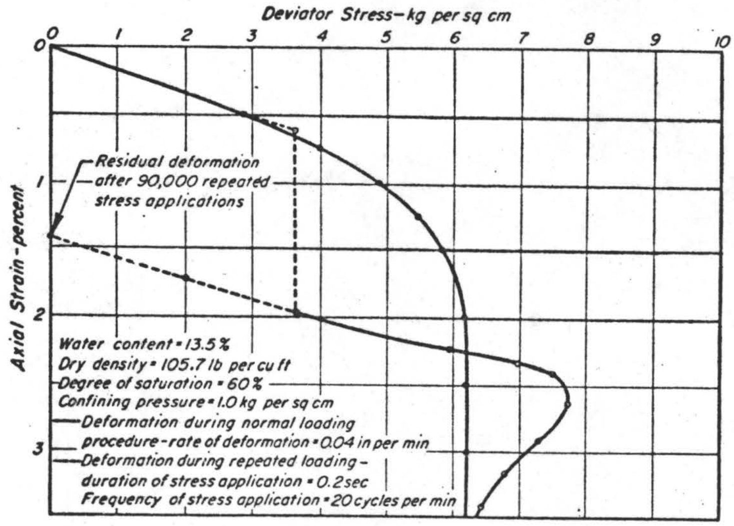


Fig.2 -DEFORMATION OF SPECIMENS OF SILTY CLAY IN NORMAL STRENGTH TESTS BEFORE AND AFTER REPEATED LOADING.

(After Seed et al, 1958)

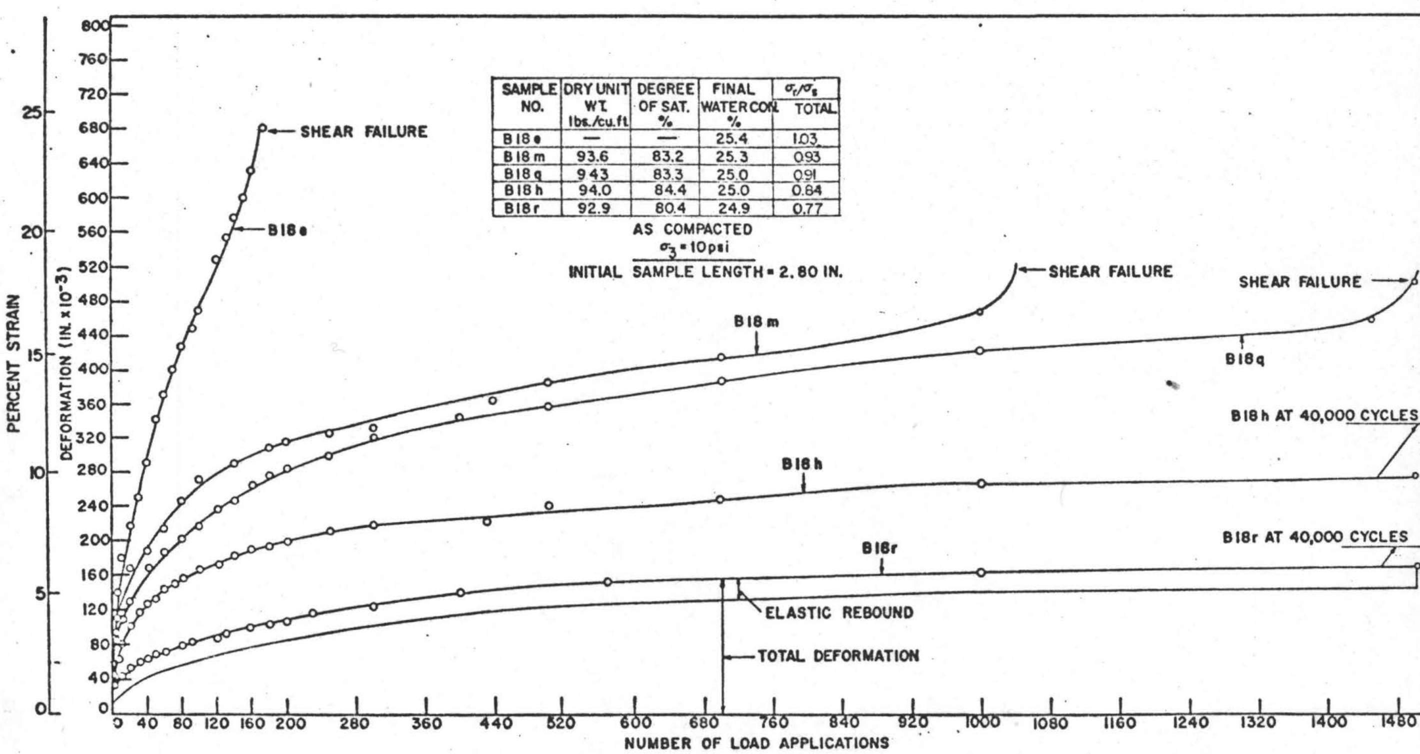


Figure 3. Deformation vs number of load applications, Soil B.

(After Larew and Leonards, 1962)

Seed and Chan (1961) studied the effect of duration of stress application on soil deformation. The effects of duration varied widely both for sands and clays. Seed concluded that the deformations of compacted clay under repeated loading was influenced by a) increase in deformation with time under load due to creep effects; b) increase in resistance to deformation due to thixotropy (as the interval between loads increases, clay will regain some of its thixotropic strength lost during deformation under the first stress application); c) increase in resistance to deformation due to changes in structure induced by load application (Extrusion of Absorbed water); d) reseparation of particles during periods of unloading giving a decreased resistance to deformation. It was not possible for Seed to put any quantitive values to the various phenomena he suggested.

The concept of a critical stress level for soils subjected to a repeated load, was developed by Larew and Leonards (1962). Two criteria for the strength of fine grained soils subjected to repeated loading were postulated and studied, one of the proposed criteria was verified.

A typical set of curves of deformation versus number of load applications is shown (Fig.3). Samples subjected to loads above a certain (critical) level eventually fail after a number of load applications. Samples tested below the critical level reach an almost constant state of deformation. The elastic rebound observed on removal of the load appears to be essentially constant for samples tested below the critical level of repeated stress (see 5.3).

It should be noted that the load application in the papers reviewed in this section is of a dynamic nature to simulate vehicle traffic. It is included because the results and conclusions bear a certain similarity with those of the current research.

2.3 Effects of Repeated Loading on Saturated Clays

Until recently little research had been carried out on the effects of repeated loading on saturated clays. The main concern with saturated clays in the undrained condition is that any increase in pore water pressure increases the effective stress ratio and brings the soil closer to the failure condition.

In previous research on saturated clays the loads have generally been applied slowly over a period of hours (Knight & Blight 1965, Sangrey 1969) so that accurate pore pressure measurements could be taken. The repeated load tests on compacted clays were generally of a dynamic nature (with load durations 0.1→5 secs.) to simulate vehicle traffic.

The testing by Knight and Blight was limited and they concluded only that there is a critical range of stress in which the effects of repeated loading are significant. Sangrey (1969) performed a series of repeated loading tests on saturated samples of clay. A cycle of loading and unloading required approximately 10 hours to enable satisfactory measurement of pore pressures to be made. As in the compacted clays (Larew 1962), Sangrey showed a critical level of repeated stress considerably lower than the compressive strength obtained by continuous loading to failure.

For repeated stresses below the critical level, pore water pressures climbed to an equilibrium position after a certain number of loading cycles. For repeated stresses greater than the critical, the pore water pressures continued to rise until the sample failed.

Sangrey represented his results by stress paths in effective stress space. A typical stress path is shown for a sample reaching an equilibrium condition. The build up of pore water pressure leads to the migration of the stress path towards the origin until non failure equilibrium, shown by the closed loop n-m, is reached. (Fig.4).

By carrying out tests at various stress levels a series of points such as m representing the stress peaks at equilibrium were obtained. Sangrey found that these points lay on a straight line termed the equilibrium line. Reference is made to this in Chapter 5.4. (Fig.5).

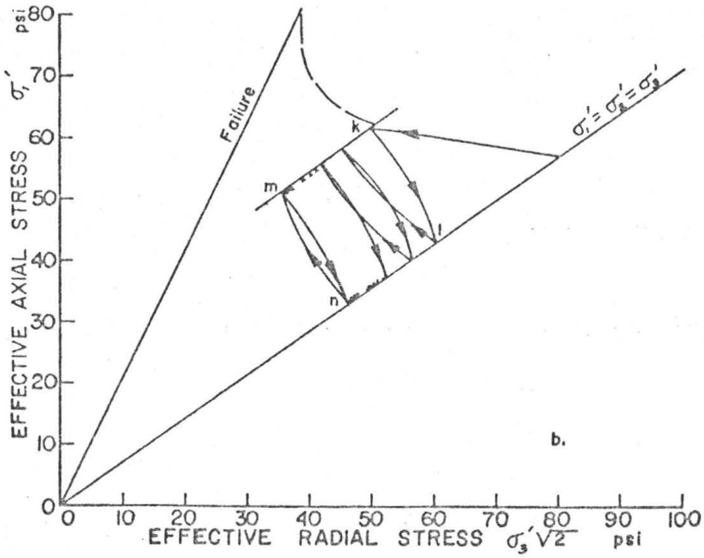


Fig. 4. Stress paths for sample T2

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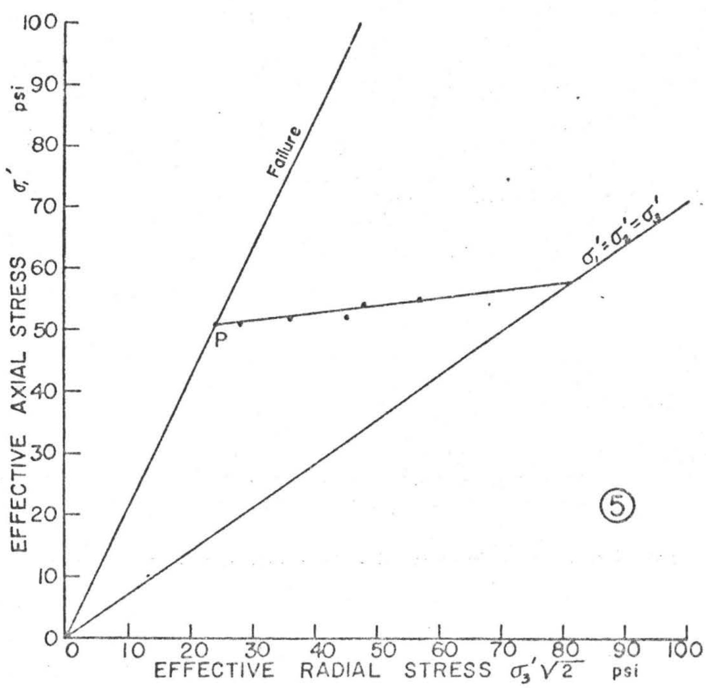


Fig. 5. The equilibrium line.

(After Sangrey et al 1969)

CHAPTER 3

DESCRIPTION OF TESTS

3.0 The initial tests were of an exploratory nature. Samples of Kaolin were prepared and consolidated in a triaxial cell. A repeated load was applied to the sample by manually lifting a dead weight on to the cell piston. With the experience of the initial tests and the problems involved, a test program was planned for Kaolin and Natural clay samples.

For a particular test series the consolidation pressure was constant. Two main test series were run. One on Kaolin samples prepared in the laboratory and isotropically consolidated to 25 p.s.i., and one on a Natural silty clay normally consolidated to a net isotropic pressure of 50 p.s.i.

The general procedure was to test a number of samples under identical conditions, except for a change in the repeated stress value. Some undrained creep tests were run, and also a number of tests on remolded samples. Other special tests are described.

3.1. Sample Preparation

Kaolin Samples

The Kaolin samples were prepared from a Kaolin powder known as hydrite U.F. This particular Kaolin has uniform grading. It is a fine silt with 8 per cent clay sizes.

The Atterberg limits change depending on the time after mixing with distilled water. (Table 1). This was an indication that the Kaolin required a certain time to stabilize perhaps because of absorbed water effects. The Kaolin was therefore mixed with water at least two weeks before the samples were prepared. A consolidation test on the material gave the parameters shown in Table 1.

The Kaolin has a reasonably high permeability because of its uniform grading and predominance of [illegible] silt grain size. This is important for ensuring equilibrium of pore water pressures within the soil sample. (Appendix)

A quantity of Kaolin powder was handmixed with distilled de-aired water to form a slurry at a water content of approximately 200 per cent. The slurry was allowed to stand for two weeks for any time effects to occur. It was then poured into a 6" dia. consolidometer from which drainage was allowed to the top and bottom.

The soil was loaded vertically in increments 0.45, 3.0, 6.0, 12.8, 24.6 p.s.i. Primary consolidation was allowed to finish before the next load increment was added. The soil was left under the final load of 24.6 p.s.i. for at least five days. After this period all the load was removed and swelling was allowed for one day. The water content after swelling was 58 per cent. 1.4" diameter piston tube samples were taken from the soil cake without removing it from the consolidometer. Each sample was ejected from the stainless steel sampling tube and cut to 3" in length, with a wire saw. 1.4" diameter saturated porous stones were placed on the ends of the sample and a slit, saturated filter paper was wrapped around.

Although the prime purpose was to form a radial drain during testing (Appendix 1.5), the addition of the filter paper also made the samples easier to handle. Extreme care was taken when handling the samples as they were soft and easily disturbed.

The samples were stored by wrapping in a sheet of polythene covered by a sheet of aluminium foil and waxed. For further protection the wrapped samples were wax sealed inside a 1.5" diameter plastic tube and stored in a humid room for at least one week before testing.

TABLE 1

KAOLIN - SOIL CHARACTERISTICS

Atterberg Limits	L.L.	P.L.	P.I.
At time of mixing from dry powder	67	38	29
After 2 days	70	39	31
After 2 months	73	40	33

Specific Gravity of Soil Grains = 2.58

Grain Size D_{60} = .0066"

Grain Size D_{10} = .0022"

Coefficient of Uniformity $\frac{D_{60}}{D_{10}}$ = 3

CONSOLIDATION CHARACTERISTICS (Load increment 10 psi.to 20 psi.)

Coefficient of Consolidation $C_v = 4.2 \times 10^{-3}$ cm²/sec

Coeft. of Volume Compressibility $M_v = 8.0 \times 10^{-5}$ cm²/g

Coefficient of Permeability $K = 3.2 \times 10^{-7}$ cm/sec

3.2 Natural Clay Samples

The natural clay was obtained from a borehole in the Hamilton Bay - $4\frac{3}{4}$" diameter piston tubes were used for the sampling. All the test samples were taken from the middle section of one sample tube. The clay was identical to that described and tested by Lopes (1970). The properties are listed in Table 2.

The test samples were obtained by cutting a 4" length from a zone near the centre of the sampling tube. The ends were not used because they were more likely to be disturbed by the sampling process. The clay was extruded from the 4" length of tube by means of a hydraulic extruder. The resulting soil cake was cut into quarters and each quarter was wrapped in polythene and aluminium foil then waxed for storing.

When a sample was required, the protective wrappings were removed and the sample was cut to 1.4" diameter on a soil lathe. There were a few small stones in some samples and if they caught on the lathe the hole was patched with someof the cut clay. Any samples with stones over $\frac{1}{4}$" were discarded. The samples were finally trimmed to 3" in length. All work on the samples was done in a humid room to prevent drying of the soil.

Saturated porous stones were placed on the ends of the sample and a saturated, slit filter paper was wrapped around. The sample was carefully placed on the pedestal of the triaxial cell in readiness for the testing procedure.

TABLE 2

NATURAL SILTY CLAY Normally consolidated, sample depth 40ft.

Atterberg Limits	L.L.	P.L.	P.I.
	34	20	14

Classification - Glacial clay, inorganic, medium plasticity (Casagrande 1932)

Natural Water Content	= 25%
Specific Gravity of Soil Grains	= 2.73
Sensitivity	= 3 to 4
Grain Size D_{60}	= .003"
Grain Size D_{10}	= .0001" (Appx.)
Coefficient of Uniformity $\frac{D_{60}}{D_{10}}$	= 30 (Appx.)

Consolidation Characteristics (Load increment 20 psi. to 40 psi.)

Coefficient of Consolidation	C_v	$= 3.0 \times 10^{-4}$ cm²/sec
Coeft. of Volume Compressibility	M_v	$= 1.2 \times 10^{-5}$ cm²/g
Coefficient of Permeability	K	$= 3.6 \times 10^{-9}$ cm/sec

3.3 Remolded Samples of Kaolin and Natural Clay

The remolded samples were prepared by a similar method to that suggested by Bishop and Henkel (1957). The clay was thoroughly mixed with water until it was wetter than the liquid limit. The clay was spooned into a 1.4" diameter stainless steel piston sampler using a spatula. Clay was added in small amounts whilst the piston was gradually withdrawn. By this method very little air was trapped in the sample. The piston was marked so exactly 3" of soil could be placed in the tube. This eliminated any need to handle the soft sample. The sample was allowed to remain in the tube for a few hours so that thixotropic hardening would give the soil more rigidity. After this time the sample was extruded directly on to a saturated porous stone on the triaxial cell pedestal. The upper porous stone was placed and a saturated filter paper wrapped around.

3.4 Test Apparatus

It was originally intended that an apparatus would be developed which could apply a repeated load to a soil sample at values of duration and intervals from 0.1 sec. upwards. An electronically controlled hydraulic oil system was planned, but the inevitable complications of setting up and operating such a system made it impractical for this research program.

Many types of mechanical repeated loading apparatus were considered, and rejected mainly because of the uncertainty of the function of load applied with time.

The preliminary tests in which a load was applied manually suggested that a great deal could be learnt from a test in which the duration (time for which the load is applied) and interval (time between loadings) were of the order of 15 seconds or more rather than the 0.1 seconds originally considered. A mechanical apparatus to give such a loading was relatively simple to construct. The tests were carried out on a standard Wykeham Farrance triaxial testing machine. The cell and back pressures were applied by means of a mercury pot and spring compensating system. Rotating brushings were employed on all the triaxial cells used to reduce ram friction. Preliminary tests without rotating bushings showed evidence that the ram was sticking under constant load and therefore rotating bushings were essential for an accurate determination of the applied load, and control over the strain rate.

For the shearing stage of testing the ram load was measured on a 300 lb. proving ring. Pore water pressures were measured at the base of the sample by means of a pressure transducer located in the base of the cell. Statham pressure transducers were used of working range 0-100 lb/sq. in. A typical transducer housing is illustrated in Fig. 6. The pressure transducers were excited by a Fylde D.C. power supply. The power supplies were allowed to stabilize for one day before use. After this time the fluctuations in excitation voltage observed were of the order 0.01 per cent of the correct setting and were therefore insignificant. The signal from the transducers was recorded on a Solatron digital voltmeter. A direct reading of pore water pressures in lb/sq.in. was obtained by choosing the appropriate value of excitation voltage.

The pore pressure measuring system employed had the distinct advantage that the pore water pressure could be observed at any time during the testing. The direct readout in lb/sq.in. enabled close monitoring of the procedure as tests progressed.

A check was made of the pressure measured by the transducer by comparing with the pressure recorded on a bourdon gauge. The maximum difference in readings was 0.4 p.s.i. The differences may be due to faults in either the transducer or the bourdon gauge although the latter is suspected. The differences are small and insignificant, as compared with the variability between samples, and the effects of temperature (Ap,2).

The main problem encountered with pore pressure measurement was variation due to temperature effects. The laboratory was not temperature controlled and there were fluctuations of 3-4°C over a 24hr. period. An investigation into temperature effects (Appendix 2) indicated a variation in the measured pore water pressure of approximately 0.5lb/sq. in./deg. centigrade change in temperature for Kaolin samples. As far as possible the laboratory was held at constant temperature but when excessive temperature changes occurred it was necessary to apply a correction to the measured pore water pressure (Appendix A). Temperature corrections were based on the temperature at the time when the drainage valve was closed after consolidation.

The top cap was a special design to ensure that the samples remained verticle during the testing. (Fig.7). It was constructed of perspex for lightness, with a stainless steel button to distribute the load. The bottom of the ram was conical to accommodate a ball bearing, this ensured a ring of contact with the top cap, thereby maintaining stability.

For repeated loads and creep tests under constant load, a force was applied to the ram. The loads required for the tests on Kaolin samples were of the order 0-10 lb. and this was applied directly to the ram (Fig.8). Larger loads were required for the natural soil sample and a lever arm device was required (Fig.9). The lever arm was calibrated by applying the load to a proving ring. The factor for the lever arm was 3.24.

On both loading devices a permanent static load was required to counteract the upthrust on the ram due to the cell pressure.

The repeated load was raised and lowered by a mechanical device driven by a General Electric motor through a Zero-Max variable speed gearbox (Fig.10). The number of load applications was set by changing the speed of rotation of the wheel.

The one eighth HP. motor was more than adequate to lift the weights with no reduction in speed. The addition of the pulley on the weight carrier reduced the tension in the chain and halved the speed of application of the load. The impact force was calculated for a typical wheel speed of 1 rev. in 2 minutes and a load of 6lbs. assuming a time of application of 0.1 sec. The impact force was equivalent to .006lbs. The time of application was somewhat greater than 0.1 sec. due to the elasticity of the mechanical system and the elasticity of the sample, therefore impact forces were negligible.

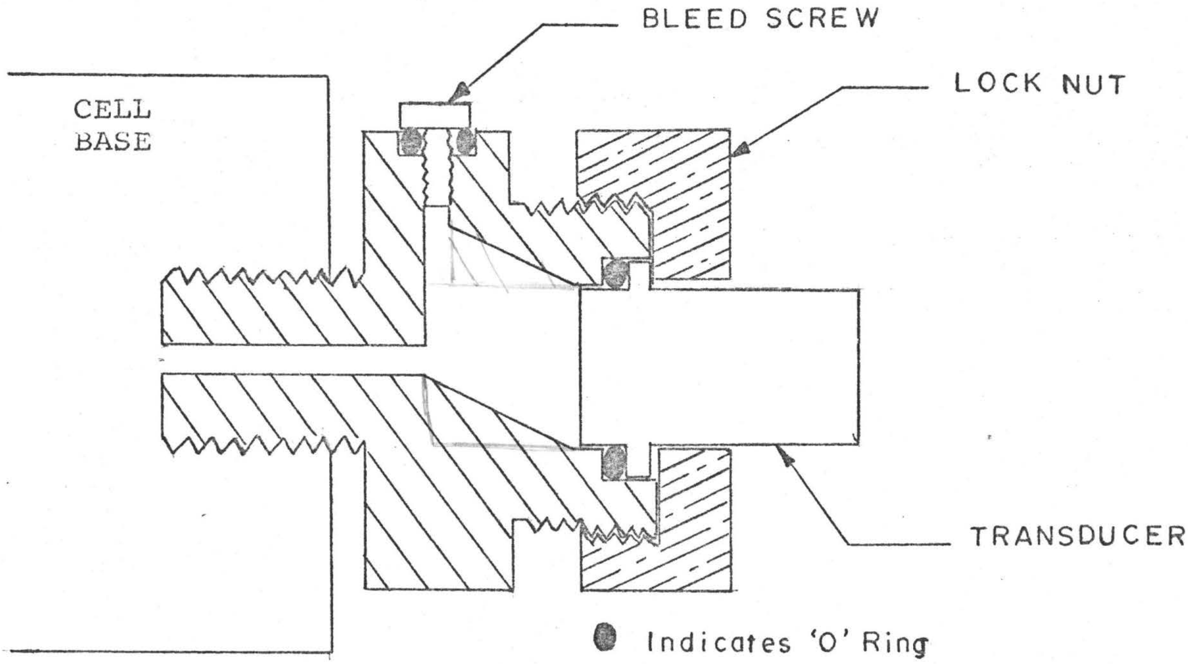


FIG. 6 TRANSDUCER HOUSING.

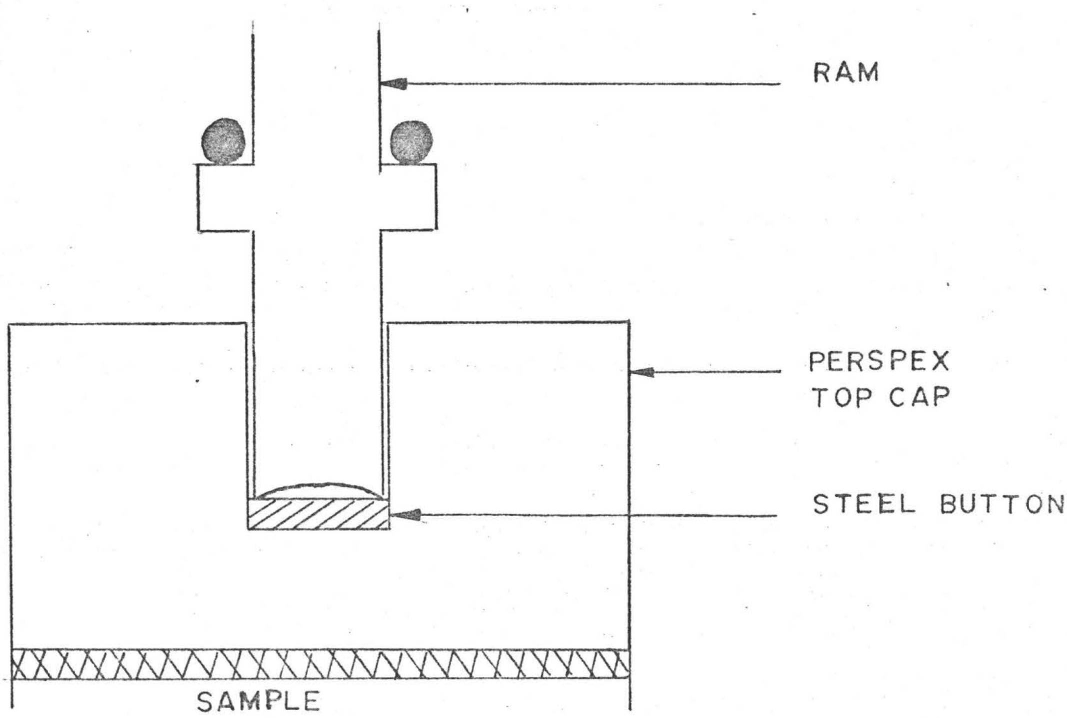


FIG. 7 TOP CAP

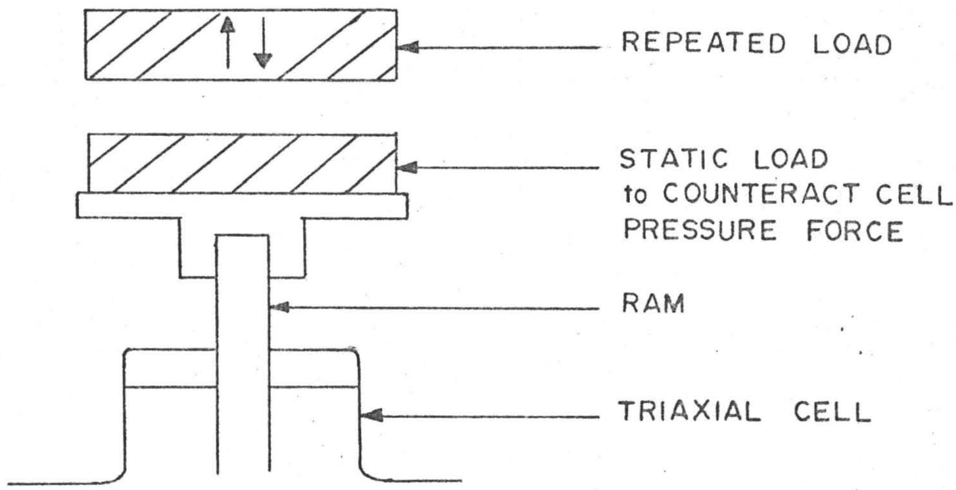


FIG. 8 REPEATED LOADING (SMALL LOAD)

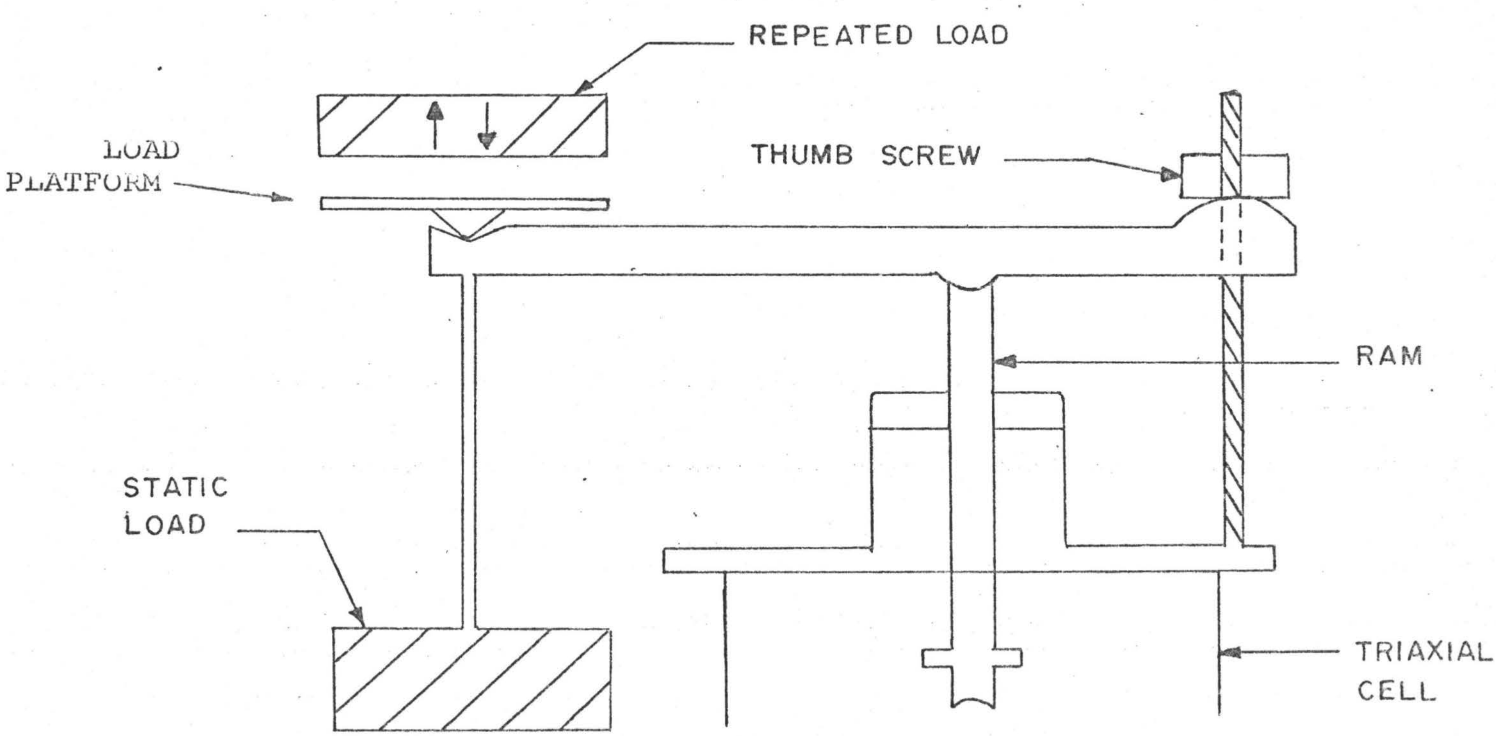


FIG. 9 REPEATED LOADING (LARGE LOAD)

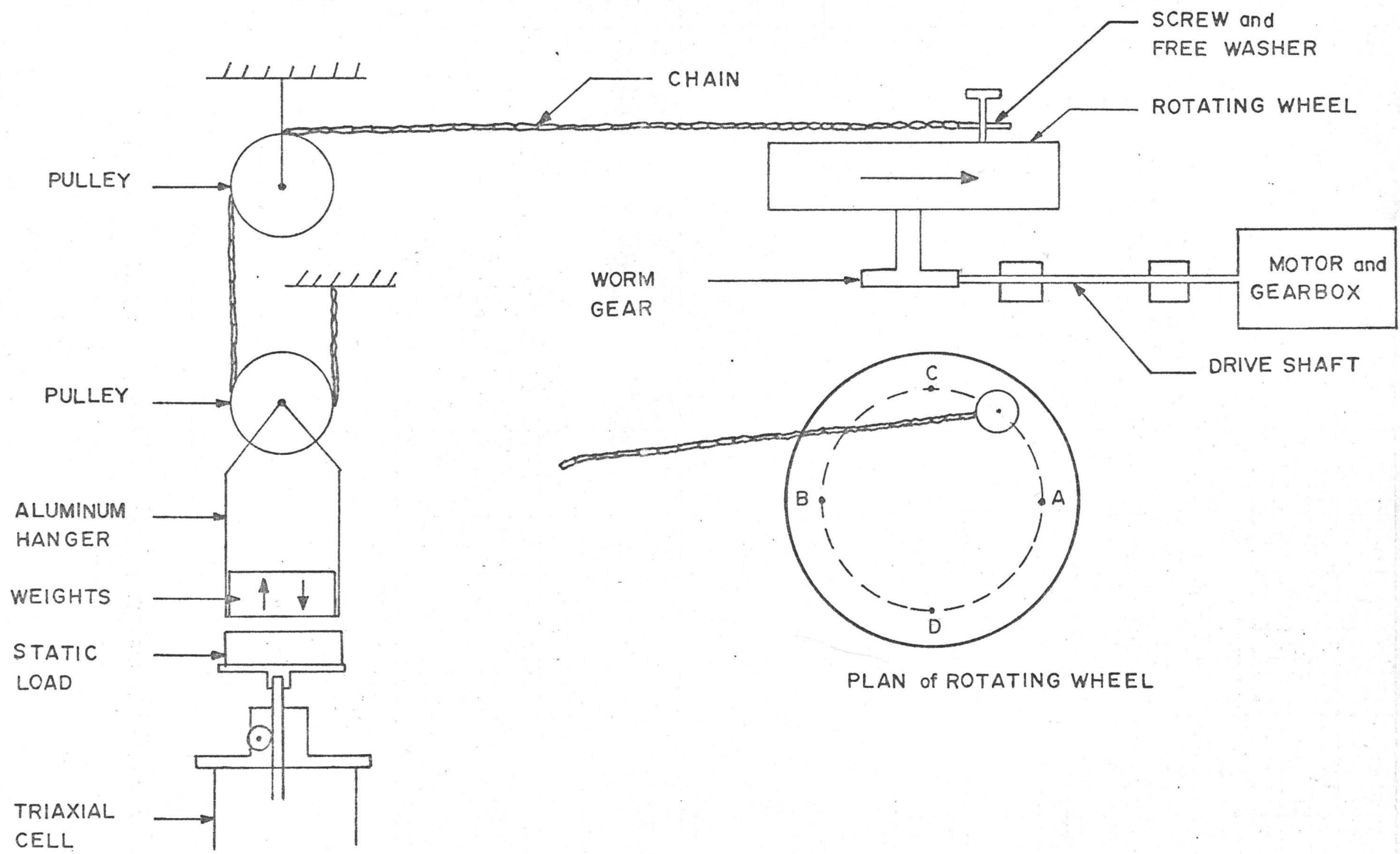


FIG. 10 REPEATED LOADING DEVICE

CHAPTER 4

TEST PROCEDURE

Membrane Application

The method employed for setting up a sample was similar to the accepted practice suggested by Bishop and Henkel (1957). The sample was placed on the pedestal of the triaxial cell with the two end porous stones and saturated slit filter paper in place. A perspex top cap was positioned and a rubber membrane placed around the sample and secured with O rings. The cell was filled with de-aired water and pressurised to 10 p.s.i. for 3 or 4 hours. During this time any air trapped beneath the membrane passed through the membrane into the cell water. The cell was then drained and a layer of silicone grease was smeared over the membrane, a second membrane was applied and secured with O rings.

The membranes used throughout the tests were Trojan Prophylactics of thickness .002". Tests by Lopes (1970) and others have indicated that over time periods greater than one day, one membrane is permeable to water. However, the use of two membranes separated by a coating of silicone grease has been shown to virtually eliminate the passage of water for tests up to one week. (Appendix 1.4).

The cell was refilled with de-aired water and again the sample was allowed to stand under a small pressure to give air trapped between the membranes a chance to pass through the outer membrane into the cell water. Fig. 11 shows a diagram of the sample as set up ready for testing.

Consolidation

The Kaolin samples were consolidated with a cell pressure of 35 p.s.i. against a back pressure of 10 p.s.i. giving a net consolidating pressure of 25 p.s.i.

The natural samples were consolidated with a cell pressure of 70 p.s.i. against a back pressure of 20 p.s.i. giving a net consolidating pressure of 50 p.s.i.

The back pressures used were high enough to give complete saturation of the soil samples. Experimental evidence indicated that all air was dissolved into the pore water at these pressures; An increase in cell pressure, with no sample drainage, produced an equal increase in the pore water pressure, indicating that there was no air in the voids of the soil.

Before placing the sample in the triaxial cell, the pressure transducer was adjusted for 'zero' reading and all lines and values were checked for the presence of air bubbles and leaks. Axial deformation and volume change readings were recorded on some of the samples to indicate the progress of consolidation. The slit filter paper fastened the consolidation process by providing radial drainage. Water

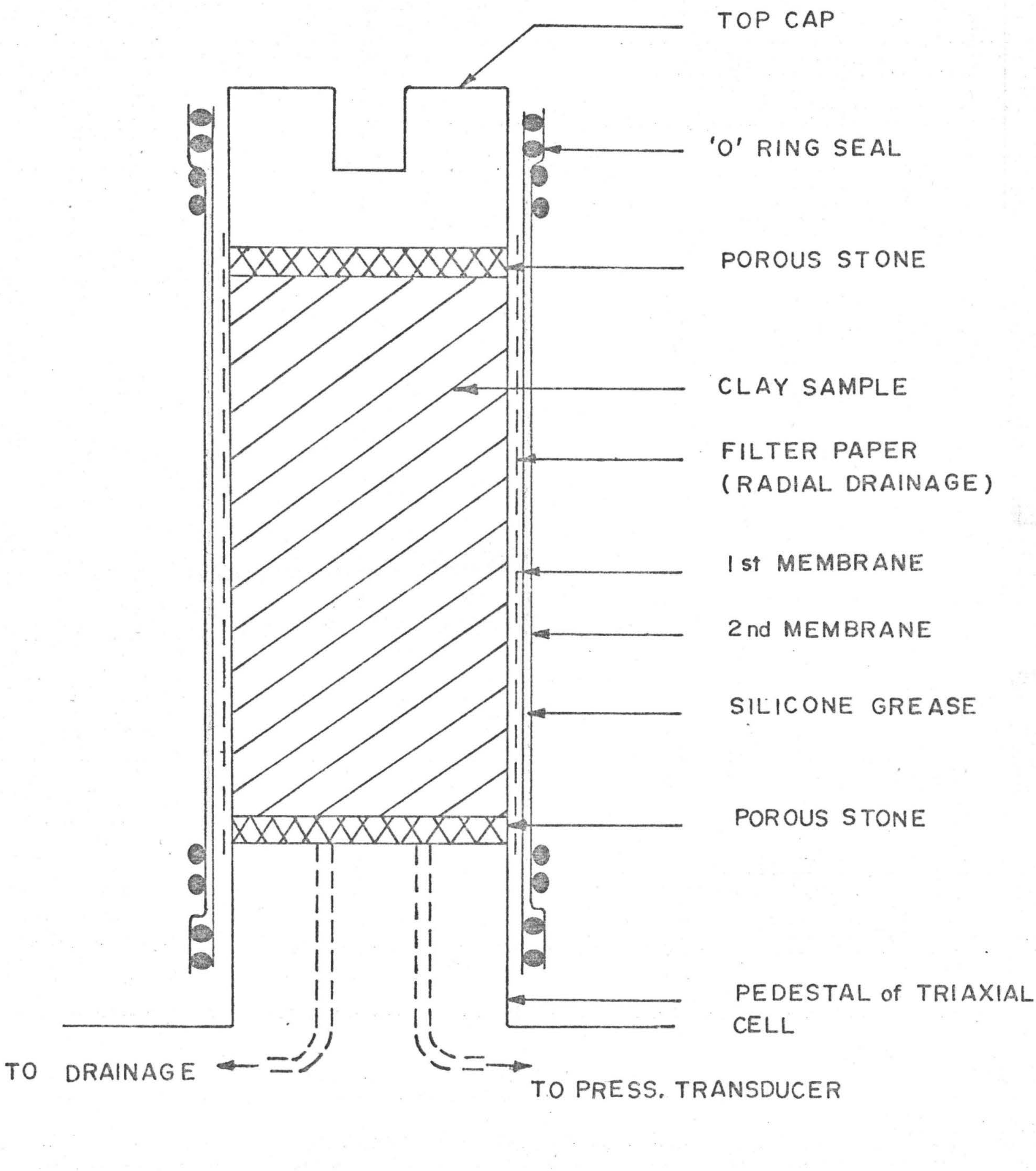


FIG. II SAMPLE READY for TESTING

outlet was from the base pedestal only. Axial deformation was recorded by means of a dial gauge (.0001 inch), attatched to the ram, which recorded ram movement relative to the top of the triaxial cell. A dead weight was applied to counterbalance the upward force on the ram due to the cell pressure; the ram therefore remained in contact with the sample at all times. Volume changes were recorded on a Wykeham Farrance volume change gauge (.05cc calibration).

Consolidation was allowed to continue for 4 days to eliminate a major proportion of the secondary consolidation and obtain better agreement between samples. At the end of this period the drainage valve was closed (maintaining the back pressure within the sample) and the undrained phase of the tests commenced.

Strength Tests

The standard consolidated-undrained (R) tests were performed in the manner described by Bishop and Henkel (1957). Pore water pressures were measured by means of the pressure transducer, the deviator stresses were measured on a proving ring, and the deformations on a dial gauge reading to .0001".

The rate of testing for the natural clay was .0016ins/min. This speed was considered to be suitable for obtaining reasonable equalization of pore water pressures (see Appendix 1.5). A rate of .008ins/min. was used for the Kaolin samples which had a higher permeability.

The tests were run to approximately 15 per cent strain; the sample was removed from the cell and weighed in a 'soil weight' tin for determination of the moisture content.

Repeated Loading

The repeated load was applied to the samples by means of the device described. The interval and duration used throughout the test were one minute.

The static load which was used to counteract the upward ram force during the consolidation stage, was maintained on the ram. The speed of the rotating wheel was set to give one revolution in two minutes, and the height of the repeated load was adjusted, to give equal duration and interval, by raising or lowering the cell. The bracket supporting the repeated load was adjusted so that the load would land in a central position on the load platform (Fig.9). The lever arm device (Fig. 9) was maintained in a horizontal position by the thumb screw. The elastic deformation under each load application was small (.005"max.) and the effect on the inclination of the lever arm was negligible. Any permanent deformations were followed by lowering the thumb screw.

The selected value of the repeated deviator stress was calculated as a proportion of the maximum compressive strength of the soil as obtained from the standard consolidated-undrained test. If the sample did not fail under the action of the selected repeated load, the test was stopped after a period of time (usually 24 hrs.); the repeated loading device was removed and the soil was loaded to failure in the manner described previously.

The sustained loads were applied with the same apparatus but the rotating wheel was stopped with the load on the sample. Readings of time, axial deformation, pore water pressure, and temperature were taken throughout the loading stages.

General

No corrections were applied for membrane strength or the strength of the filter drain. The total errors are small (Appendix 1) and as the errors are consistent for all the samples tested, they do not affect any comparisons made between samples subjected to repeated loads, sustained loads, or standard strength tests.

The average cross sectional area of the consolidated sample was calculated by considering the changes in height and volume during the consolidation stage (Appendix 1.1). Distortion of the sample occurred due to friction between the sample and the porous stones. Lubricated end discs were considered for the tests but rejected because of the extra complication in setting up the sample. Although the variation in sectional area of the sample was small, it inevitably produced an irregular stress distribution of unknown form. However this does not invalidate any comparisons between samples, as every sample had similar dimensions after consolidation.

A computer program was prepared to calculate the total and effective stresses acting on a sample during an undrained triaxial shear test.

CHAPTER 5

RESULTS AND DISCUSSION

5.0 General

Results are presented for the natural clay samples and the laboratory compressed Kaolin samples. The responses of both soils to repeated and sustained loads are very similar and therefore only the results obtained from the natural clay samples are presented in full. Results of the Kaolin tests are given where there may be some doubt as to the accuracy of the recorded pore water pressure in the natural clay (Appendix 1.5).

A summary of the tests is given in tables 3 and 4. The repeated or sustained deviator stresses (σ_r) are expressed as a proportion of the compressive strength (σ_s) of the soil as obtained from a standard consolidated-undrained (R) test under the same confining pressure.

The results of a typical repeated load test are shown in Figs. 12a, 12b. The first application of stress produces an immediate strain and excess pore water pressure. The strain and excess pore water pressure continue to increase until the stress is removed; on removal there is a residual strain ($\Delta\varepsilon_p$) and residual pore water pressure (Δu_p). With reapplications of stress the residual strain and residual pore pressure continue to increase. The recoverable portions of the strain and pore pressure ($\Delta\varepsilon_e$ and Δu_e) remain almost constant with subsequent reapplications of stress.

The total strain ($\Delta\varepsilon$) at any time with the load applied is comprised of a recoverable (elastic) component ($\Delta\varepsilon_e$) and a permanent (plastic) component ($\Delta\varepsilon_p$).

thus $$\Delta\varepsilon = \Delta\varepsilon_e + \Delta\varepsilon_p$$

Similarly the excess pore water pressure (Δu) with the load applied is comprised of a recoverable component (Δu_e) and a non recoverable component (Δu_p).

thus $$\Delta u = \Delta u_e + \Delta u_p$$

The results of the Kaolin test indicate that the pore pressure trend in the natural clay tests is correct, although there may be a small error in the recorded values due to non equalization of pore pressures. The calculated time for 95 per cent equalization of the natural clay is 50 minutes as compared with 3 minutes for the Kaolin (Appendix 1.5); experimental evidence supported the validity of these calculated equalization times.

Fig. 13 shows the axial strain, with the load applied, plotted versus log. time for samples under repeated loads at various stress levels (σ_r). At low values of σ_r (ie.,zero to $0.37\sigma_s$), the curves are approximately linear after 100 minutes.

At higher levels of repeated stress the curves show a definite tendency for increased strain with log. time. There is a critical level of repeated deviator stress below which the soil is reasonably stable, and above which the strain increases under each stress application until failure occurs.

However the critical level is time dependant. For instance, at 60 minutes after commencing repeated loading, a sample fails if the repeated deviator stress is greater than $0.80\sigma_s$. After 1000 minutes of repeated loading, a sample fails at $0.60\sigma_s$. At very large times (e.g. 10^4 minutes), the critical level may be as low as half of σ_s. These figures indicate that for a repeated loading of the type used (Duration = Interval = 1 Minute), the soil sample can withstand 30 applications at $0.80\sigma_s$, 500 applications at $0.60\sigma_s$, and perhaps 5,000 load applications at half of σ_s before failure occurs in the undrained state.

If drainage occurs, the factor of safety against normally consolidated clays usually increases. For practical purposes it is necessary to choose a time under undrained conditions which is related to the likely drainage conditions existing in the ground. In this discussion the deformation and critical stress level are considered after one day and seven days of repeated loading in the undrained state. The deformation and critical stress level are considered after half a day and three and a half days for sustained load tests, in order that the same time under load may be studied.

The curves of pore water (load applied) versus log. time (Fig. 14) are similar to the curves for axial strain versus log. time. After 100 minutes the curves are approximately linear. It was necessary to correct some of the points (marked 'o') for temperature changes. Corrections are based on the temperature at the time the drainage valve was closed (Appendix 2).

At low levels of σ_r (i.e., zero to $0.37\sigma_s$), a small increase in the sustained or repeated stress is observed to produce, after one day, mainly a recoverable increase in the strain and pore water pressures, i.e., $\Delta\varepsilon_e > \Delta\varepsilon_p$. This is termed, for convenience in this discussion, the region of 'Elastic' behaviour of the soil.

For repeated or sustained stresses greater than $0.37\sigma_s$, a small increase in the sustained or repeated load produces mainly a non recoverable increase in the strain and pore water pressures after one day of loading, i.e., $\Delta\varepsilon_e < \Delta\varepsilon_p$. This is termed the region of 'Plastic' behaviour of the soil.

It should be emphasised that the 'elastic' range is by no means exclusively elastic and there is no clear cut dividing line between the two ranges.

The 'elastic' region would probably be the desired working range of a soil, as the critical stress level is in the region of plastic behaviour.

5.1 Comparison Between Repeated and Sustained Loads

It is possible to make a direct comparison between repeated and sustained loading by considering strains and pore water pressures for the time under load only (Fig. 15 and 16).

For tests (N7 and N9) at values of deviator stress within the 'elastic' region (i.e., zero to $0.37\sigma_s$) no difference was observed between repeated and sustained loading; the axial strain and pore water pressures were entirely dependant on the time under load. However, at higher values of deviator stress (region of 'plastic' behaviour), repeated loading gives a definite increase in the axial strains and pore pressures over those produced by a sustained load. This may be seen in tests N6 and N13 at $0.52\sigma_s$ and tests N4 and N 10 at $0.80\sigma_s$.

Test N 11 shows the effect of changing from repeated loading to sustained loading at the same applied stress level. The strain increases at a greater rate than log. time under a repeated load, but under a sustained load the curve becomes linear. Under a sustained load (creep) the axial strain is linear with log. time for all levels of stress. This has been shown previously by Walker (1969).

Test N 10 shows the effect of changing from sustained loading to repeated loading; the axial strain increases rapidly under repeated loading.

At stress levels within the 'plastic' region, repeated loading increases the rate of axial strain and pore pressure generation. The critical stress level under sustained load is therefore higher than the critical stress level under repeated loading after the same period of time under load. The rate of loading, duration, and interval are all likely to affect the deformation characteristics (Seed and Chan 1961). If the applied stress is in the plastic range, individual tests are necessary to predict the response of a particular clay to a given loading condition.

5.2 Pore Water Pressure versus Strain Relationship

A relationship may be seen between the pore water pressures and the axial strain. Figs. 17a and 17b show the pore water pressure plotted against axial strain for natural clay and Kaolin samples. The lines on the graph represent the behaviour after a certain number of load applications (n). The lines joining the 'load off' to the 'load on' points represents the recoverable components of strain and pore water pressure. The slope of the lines is reasonably constant after the first two cycles of loading. The value of the recoverable component of the pore pressure (Δu) is approximately equal to 1/3 x the applied deviator stress, as predicted in the elastic theory for saturated soils (Skempton 1954).

The exact path of the 'elastic' curve cannot be traced with the equipment used; only the end points are known.

There is probably some form of hysteresis curve between the points (Sangrey 1969) but for this investigation it is reasonable to assume that the line is straight.

The 'load on' points are seen to lie in a atraight line; likewise the 'load off' points lie in a straight line. This indicates a linear relationship between the residual pore water pressure component (Δu_p) and the axial strain ($\Delta \varepsilon_p$).

For values of repeated load within the region of elastic behaviour, it has been shown that the pore water pressures may be divided into two distinct components. The recoverable and non recoverable component are both directly proportioned to axial strain, although with different constants of proportionality.

A pore water pressure (Δu) versus strain ($\Delta \varepsilon$) relationship may be drawn for all repeated loading tests by considering the pore water pressures with the load applied (Fig. 18). The pore water pressures are proportional to strain for repeated loads within the elastic range. For higher values of repeated stress, the strain increases at a greater rate than the pore pressures. The greater the value of applied stress, the sooner the curve tends to break from the linear.

The lines are not colinear because the recoverable pore pressure component does not bear the same constant of proportionality with strain as does the non recoverable component of pore pressure. (Fig. 17a).

A linear pore water pressure versus axial strain relationship has been observed previously in undrained creep tests (Walker 1969), and the tests of Sangrey (1969) show an apparent linear relationship for repeated loading tests. Lo (1969) postulated that there exists a linear relationship between pore water pressure and axial strain, and the results are in accordance with these predictions. Lo explained the linear relationship as due to the collapse of the soil structure causing proportional strains and pore water pressure increases (see 5.5).

5.3 Rebound Characteristics

The changes in strain and pore water pressure that are observed immediately on removal of the load represent the recoverable components of strain ($\Delta\varepsilon_e$) and pore water pressure (Δu_e). Fig. 19 shows the variation in rebound characteristics for various repeated stress levels (σ_{fr}) and after a number of stress applications (n).

For small repeated stresses (i.e., within the 'elastic' region), the elastic rebound remains at a constant value for a given repeated stress; the Modulus of Elasticity (represented by the gradient of the curve) is constant. At higher repeated stress levels (i.e., within the 'plastic' region), the Modulus of Elasticity decreases; at a given stress level a further decrease in the modulus is observed with increasing numbers of load applications.

The recoverable pore pressure (Δu_e) also varies linearly with repeated stress (σ_r) for small values of stress. The recoverable pore water pressure (Δu_e) is approximately one third of the applied deviator stress, as would be expected from elastic behaviour of a two phase system (Skempton 1954). At higher repeated stress levels where the sample approaches failure, a reduction in the recoverable pore water pressure is observed. This is probably due to dilatancy in the soil as shearing takes place.

With larger times after the removal of the stress there is a tendency for a further decrease in strains and pore water pressures due to the swelling of the soils. This effect is very small compared to the magnitude of the permanent strains and pore water pressures for the clays tested. Fifty minutes after the load was finally removed in test N3 the pore water pressures had changed by only 0.2 p.s.i. from the value at the time of stress release. The permanent component of pore pressure (Δu_p) at this time was 8.0 p.s.i. The swelling over this period was 0.0003", as compared with the permanent deformation of 0.0033" under the action of the repeated load. Swelling may be of greater importance in clays which are susceptible to swelling effects.

5.4 Stress Paths

A stress path is a graph of effective principal stresses which describes the behaviour of a soil under load; the stress path is plotted in the principal stress space. In recent years a number of effective stress diagrams have been used. The most suitable for following the behaviour of a soil under repeated loading is the Rendulic plot (Henkel 1960) in which the vertical effective stress is plotted against √2 times the horizontal effective stress.

The stress path of a repeated loading test within the 'elastic' region is shown in Fig. 20. After isotropic consolidation to 50 p.s.i., the stresses are represented by point 'a' on the $\sigma_1' = \sigma_1'$ line; this line is the space diagonal and all samples return to a state on this line if the applied deviator stress is removed.

When a deviator stress is applied, the stress state moves to b. Under constant deviator stress the pore water pressures continue to rise and the stress path moves in a direction $\overline{bc}$. The path is parallel to the space diagonal; the generated pore pressures cause equal changes in the vertical and horizontal effective stresses. Line $\overline{bc}$ is a typical path obtained with no change in total stress. The pore pressures generated under constant applied stress are most likely due to a partial collapse of the soil structure (see 5.5).

If the application of deviator stress produces only an 'elastic' recoverable pore water pressure response then the pore water pressure is one-third of the applied deviator stress. (see 5.3). The stress path is then perpendicular to the space diagonal.

If dilatancy or swelling occurs the pore water pressures decrease and equal increases in the vertical and horizontal effective stresses result. A stress path away from the origin parallel to the space diagonal indicates dilatancy or swelling. The three major components of generated pore pressures are represented by the axes shown on Fig. 20.

The pore pressures generated due to absorbed water effects also produce stress path movement parallel to the space diagonal. These effects are thought to be small compared with the other effects mentioned, although they may be important in clays susceptible to swelling.

The stress path shows that a soil subjected to repeated loading, after the first load application, behaves in an almost truly elastic manner with each load application. There is however a tendency for residual pore water pressures to increase such that after 750 load applications (i.e., 1 day) the stress path between the load off and load on conditions is $\overline{dc}$. Point c represents the stress state (load applied) after repeated loading at $0.30\sigma_s$ for one day. The stress state after 7 days (point e) is predicted by extrapolating the excess pore pressure from Fig. 14.

The stress path of a sample strained to failure in a standard 'R' test (N 16) is shown in the diagram for comparison with the repeated loading case.

In Fig. 21. the stress states after 1 day and 7 days are shown for all repeated loading and sustained loading tests.

Test results have been published in which a repeated load was applied gradually over a period of 5 hours and removed over a similar period (Sangrey 1969); this slow rate of loading ensured pore water pressure equalization. Sangrey stated that the pore water pressures tended towards an equilibrium position after a number of cycles of loading. The plots presented here of pore pressure and axial strain with log. time (Figs. 13 and 14) indicate that there is no true equilibrium position. The pore water pressures and strains continue to increase with time and therefore the state of the sample after a given time interval must be considered.

The line joining the states of stress of samples subjected to repeated loading for a given length of time will be termed the equilibrium line for continuity of terminology, but the time to reach this condition will always be quoted, i.e., equilibrium line (7 days).

The equilibrium line obtained by Sangrey was straight and passed through the point 'A' representing the isotropic consolidation pressure (Fig. 5). The equilibrium lines after 1 day and 7 days obtained in this test series are not straight (Fig. 21).

Sangrey did not show any data points for low values of repeated stress, and it may be for this reason that his equilibrium lines appeared to be straight. If the points for repeated stresses lower than $0.39\sigma_s$ are ignored, such a straight line could be drawn on Fig. 21. The equilibrium line for sustained loading tests appears to be identical with that for repeated loading tests (assuming same time under load) for stress values within the 'elastic' region. The equilibrium line for sustained loading is different from that for repeated loading for applied stresses within the 'plastic' region. A repeated load causes greater strains and pore water pressures than a sustained load of the same magnitude. The critical stress level is represented by the point at which the equilibrium line meets the failure line, and it may be seen that the critical stress level is reduced by the application of a repeated load rather than a sustained load.

It is observed that the shape of the equilibrium line (1 day or 7 days) is similar to the stress path of a remolded sample (Test N 14), isotropically consolidated such that the final water content was similar to that of the natural clay after consolidation. A net consolidation pressure of 45 p.s.i. was found to give the remolded, reconsolidated, sample a similar water content to the natural clay (consolidated to 50 p.s.i.).

The rate of testing for the remolded soil and the natural clay samples was .055 per cent/min.

The similarity between the equilibrium lines and the remolded stress path suggests that, under the action of a repeated or sustained load, the sensitive natural clay structure is gradually broken down until it approaches the insensitive (remolded) state (see 5.5).

The remolded soil is perhaps an indication of the final equilibrium position of a sensitive clay subjected to sustained loads. The critical stress level for sustained loads is similar to the maximum strength of the remolded clay. For repeated loads the critical stress level is reduced to slightly below the compressive strength of the remolded soil. A repeated load hastens the process of shearing if the applied stresses are within the region of plastic behaviour. It is likely that the repeated load can counteract the reserve of strength available because the soil grains must dilate before failure.

It should be noted that the shape of the stress path for the remolded soil is itself governed by the rate of loading. A remolded sample was subjected to a sustained load of $0.30\sigma_s$ to investigate how much the remolded clay stress path would creep towards the origin. The values of σ_1' and σ_1' after 1 day were approximately 2 p.s.i. lower than the value obtained under the same load in the 'R' test (remolded clay). 'Creep' of the remolded stress path is therefore small compared with the variation in the natural clay stress path under different applied stress conditions.

In Fig. 22 the strains under repeated loading are compared with the stress/straincurves for the standard 'R' test and the remolded 'R' test.

It is seen that again the remolded test is a reasonable approximation to the 'equilibrium' positions after 1 day and 7 days. The critical level of repeated and sustained stress is seen to approximate to compressive strength of the remolded clay.

5.5. Mechanistic Picture

The behaviour of a sensitive clay under repeated or sustained loading may be explained by considering a simplified mechanistic picture of the clay grains. Fig. 23 is a two dimensional representation of a group of clay particles in random distribution.

Over thousands of years it is likely that the natural clay structure has developed strong bonds at the points of contact between the soil grains (Fig. 23A). When the soil is loaded, there is an elastic compression of the bonded soil structure. The contacts between the soil grains are able to transmit both normal and shear stresses. The permanent deformations come about by the failure of the bonds at the contact points of the grains. The breaking of the bonds is time dependant and occurs only under repeated or sustained loading.

When the bond fails, the load previously carried by the soil structure is transferred partly to the adjacent contact points, and partly to the pore water giving an increase in the pore water pressure (Fig. 23C) (Lo 1969). This accounts for the linearity of the relationship between pore water pressure and strain under constant load. The strain depends directly on the number of bonds failing, and the failing bonds produce a corresponding pore water pressure increase.

In the case of the remolded soil the bonds at the contact points are weak (Fig. 23B). The soil structure collapses almost immediately on application of the load and immediate pore water pressures and strains result corresponding to those obtained for the natural sensitive soil after 1 day or more under load.

This simple picture does not account for changes in the shape of the soil structure on remolding, and it is intended only as a general indication of the processes that are occurring.

The large strength discrepancy between samples tested under a sustained load and those tested under a constantly increasing load may be explained in terms of collapse of the clay structure. Under a constantly increasing load some structural collapse occurs, shear cannot take place between the collapsed grains without a further increase in stress to counteract the effects of dilatancy, hence a 'false' peak strength is obtained. The rate of loading is critical in determining the peak strength obtained (6.4).

In the case of samples under a repeated or sustained loading, if the applied stress is above the critical value the soil structure continues to collapse with time until the resulting pore water pressure increases bringing the sample to failure.

There is a small reserve of strength even at the failure condition because dilatancy must occur before shear can take place. In such a situation the tests show that a repeated load overcomes this reserve of strength whereas a sustained load cannot. However, for practical purposes, it is undesirable to have the soil at the state of failure.

5.6 Strength After Repeated Loading

The samples which did not fail under the action of a repeated load were tested for strength by increasing the strain as in a standard undrained test. The stress paths for such tests are shown in Fig. 24. There is no loss in strength due to the repeated loading, and there was definite evidence of a gain in the peak strength of the soil; this gain in strength is also shown in the shear strength characteristics (Fig. 25).

Up to the point when the continuously increasing stress is equal to the repeated stress that was employed, the soil behaves perfectly elastically. Above the repeated stress level the pore water pressures do not increase as rapidly with increasing strain. In the case of Test N6 an actual decrease was observed; this suggests that dilatancy occurred. Under the action of the repeated load the soil structure partly collapses giving the grains a slightly denser packing. In order that shear can occur the grains must dilate thereby producing a negative pore pressure. The soil is then able to withstand a higher deviator stress. In the standard 'R' test, there is not enough time under constant load to allow as great a degree of structural collapse to occur. If the applied stress is large enough to cause shear within the soil mass, most grains can slide freely without having to dilate.

The increase in strength under undrained conditions is comparible with that found by Seed (1958) (Chapter 2.2) for compacted materials. It is likely that a similar partial collapse of the structure occurs in non saturated materials making a certain degree of dilatancy necessary before shearing can occur.

TABLE 3

SUMMARY OF TESTS:- NATURAL SILTY CLAY

TEST No.	TYPE OF TEST	PROPORTION OF COMPRESSIVE STRENGTH	FINAL WATER CONTENT	REMARKS
N 1	'R' Test	1.0	24.8	Compr. strength = 34.8 p.s.i.
N 16	'R' Test	1.0	23.0	Compr. strength = 33.1 p.s.i.
N 8	Rep. Load	0.16	24.8	Strength test after rep. load
N 7	Rep. Load	0.3	24.2	Strength test after rep. load
N 3	Rep. Load	0.37	22.8	Strength test after rep. load
N 6	Rep. Load	0.52	24.8	Failure under rep. load
N 4	Rep. Load	0.8	24.6	Failure under rep. load
N 9	Creep	0.3	25.0	Strength test after creep
N 13	Creep	0.52	26.5	
N 10	Creep and Rep. Load	0.8	25.5	Failure under rep. load
N 11	Rep. Load and Creep	0.59	23.4	Strength test after creep
N 12	Remold 'R' Test	-	22.5	Consolidation Pressure = 45p.s.i.
N 14	Remold 'R' Test	-	22.5	Consolidation Pressure = 45p.s.i.

Notes to Table 3

1. Dimensions of natural samples after consolidation are 2.9" x 1.37"
2. Dimensions of remolded samples after consolidation are 2.7" x 1.35"
3. Consolidation pressure = 50 p.s.i. except for remolded samples.
4. All testing in undrained state after initial consolidation.
5. For Repeated Loading, Interval = Duration = 1 minute.
6. Rate of shear testing = .0016 ins/min (.055% per min)

TABLE 4

SUMMARY OF TESTS:- KAOLIN SAMPLES

TEST No	TYPE OF TEST	PROPORTION OF COMPRESSIVE STRENGTH	FINAL WATER CONTENT	REMARKS
K 5	'R' Test	1.0	48.5	Compr. strength = 21.5 p.s.i.
K 2	Rep. Load	0.3	51.5	Strength test after rep. load
K 3	Rep. Load	0.4	-	Strength test after rep. load
K 6	Rep. Load	0.5	50.0	Failure under rep. load
K 7	Creep	0.4	48.5	Strength test after creep

Notes to Table 4

1. Sample dimensions after consolidation are 2.9" x 1.37 '
2. Consolidation pressure = 25 p.s.i.
3. All testing in undrained state after consolidation.
4. For Repeated Loading, Interval = Duration = 1 minute
5. Rate of shear testing = .008 ins/min (.275% per min.)

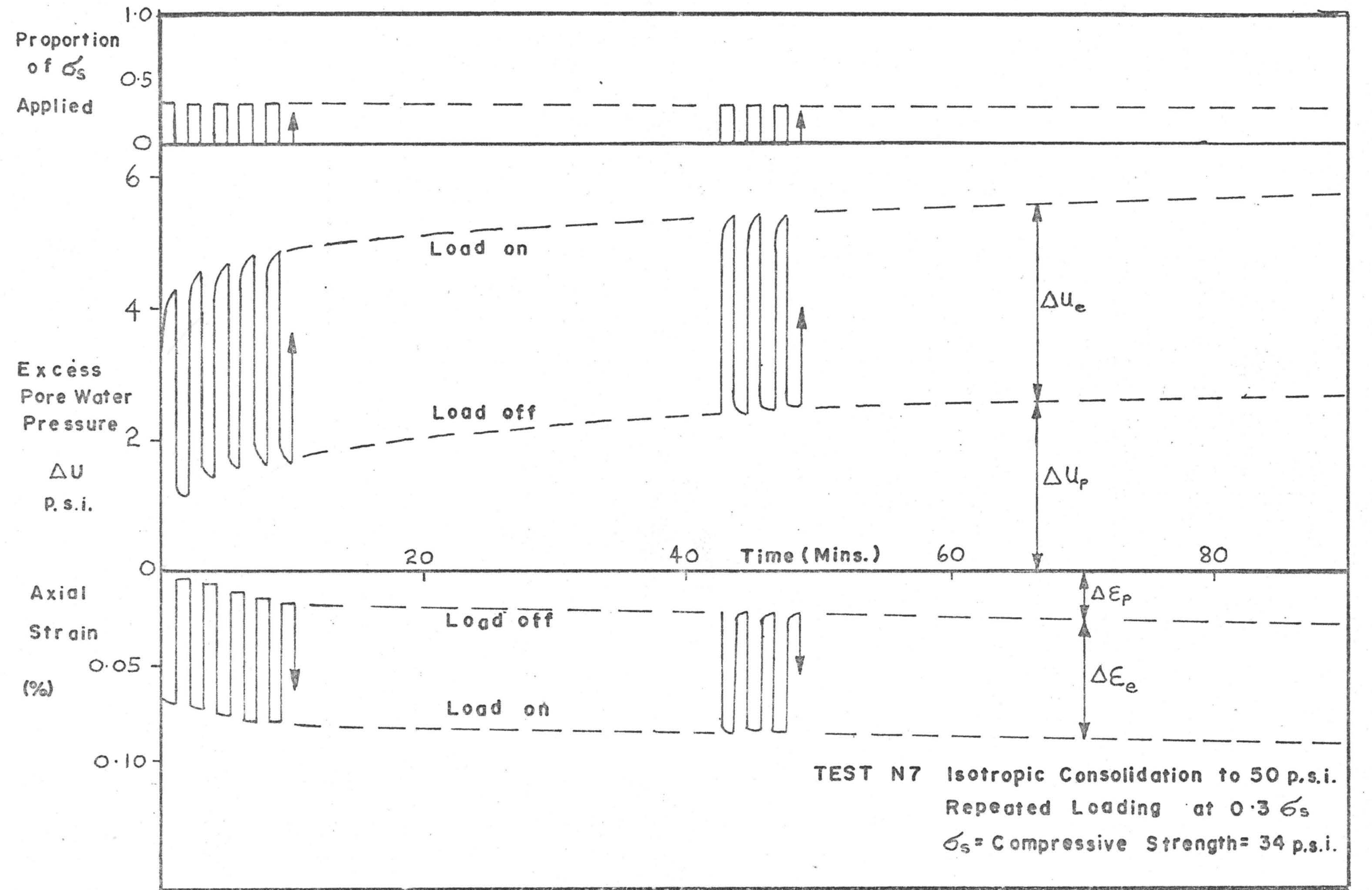


FIG. 12 a TYPICAL CURVES of LOAD, EXCESS PORE PRESSURE and STRAIN vs TIME for REPEATED LOADING TEST on NATURAL SILTY CLAY

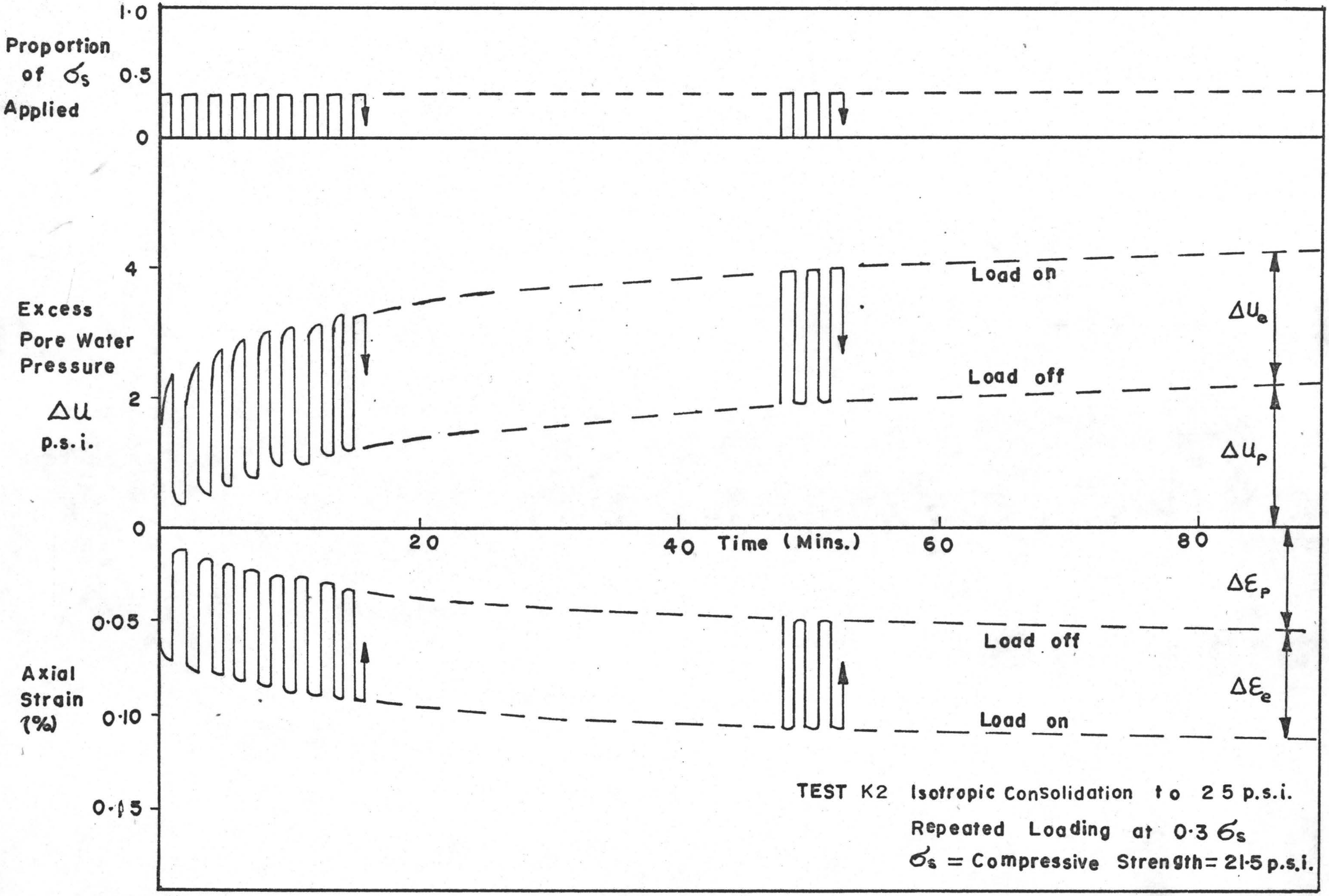


FIG. 12b TYPICAL CURVES of LOAD, EXCESS PORE PRESSURE, STRAIN vs TIME for REPEATED LOADING TEST on KAOLIN

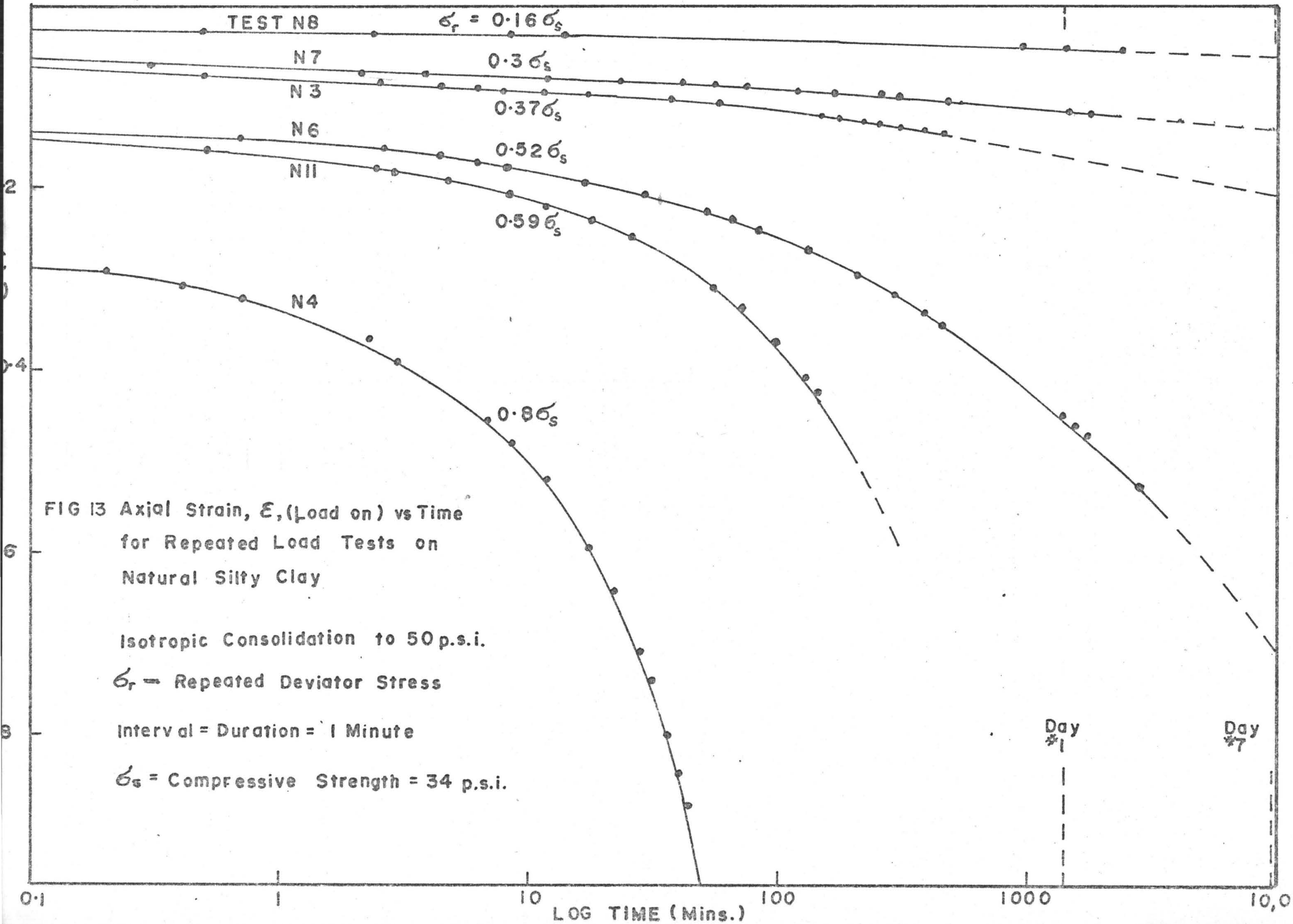


FIG 13 Axial Strain, ε, (Load on) vs Time for Repeated Load Tests on Natural Silty Clay

Isotropic Consolidation to 50 p.s.i.

σ_r — Repeated Deviator Stress

Interval = Duration = 1 Minute

σ_s = Compressive Strength = 34 p.s.i.

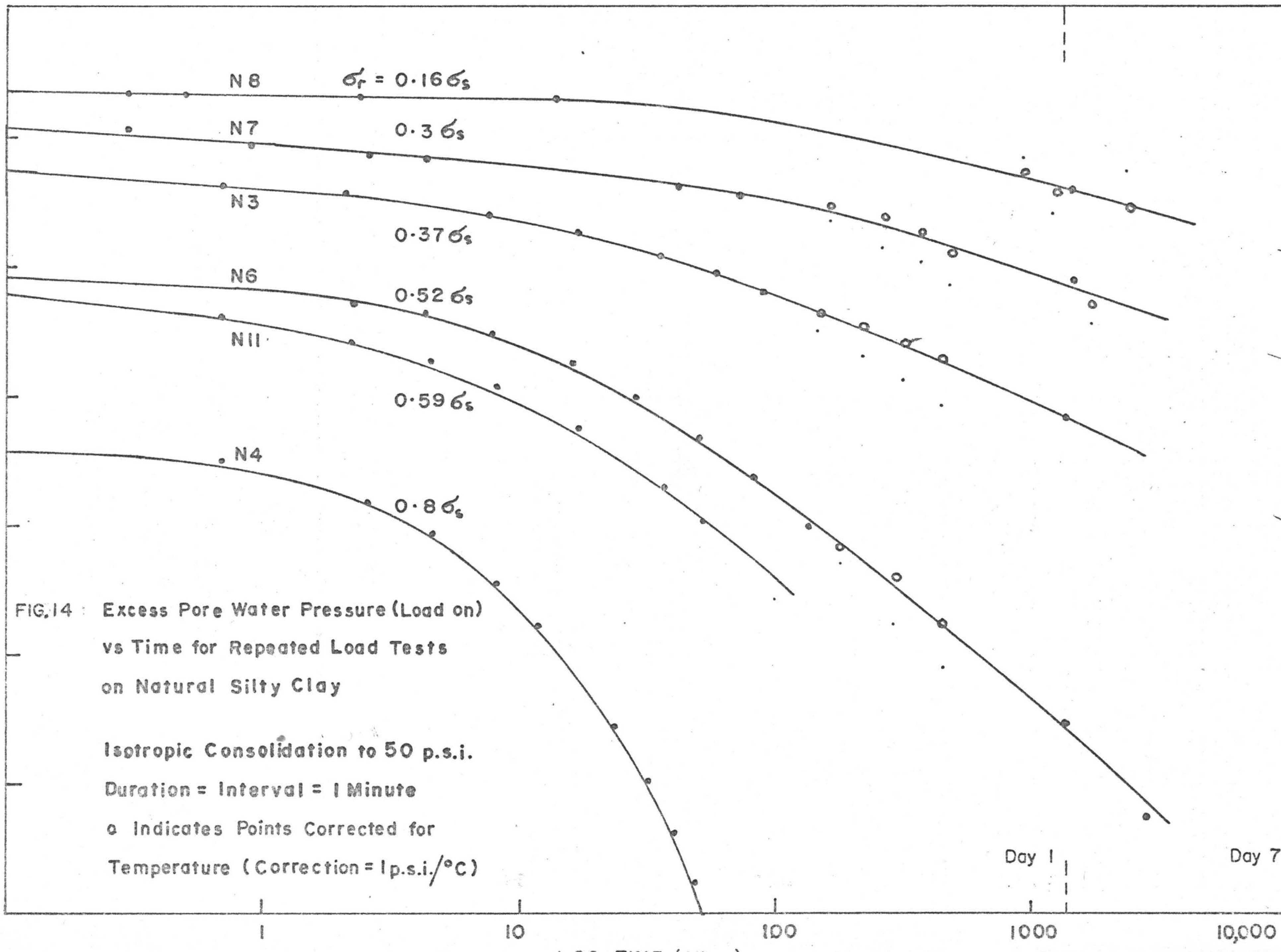


FIG. 14 : Excess Pore Water Pressure (Load on) vs Time for Repeated Load Tests on Natural Silty Clay

Isotropic Consolidation to 50 p.s.i.
Duration = Interval = 1 Minute
o Indicates Points Corrected for Temperature (Correction = 1 p.s.i./°C)

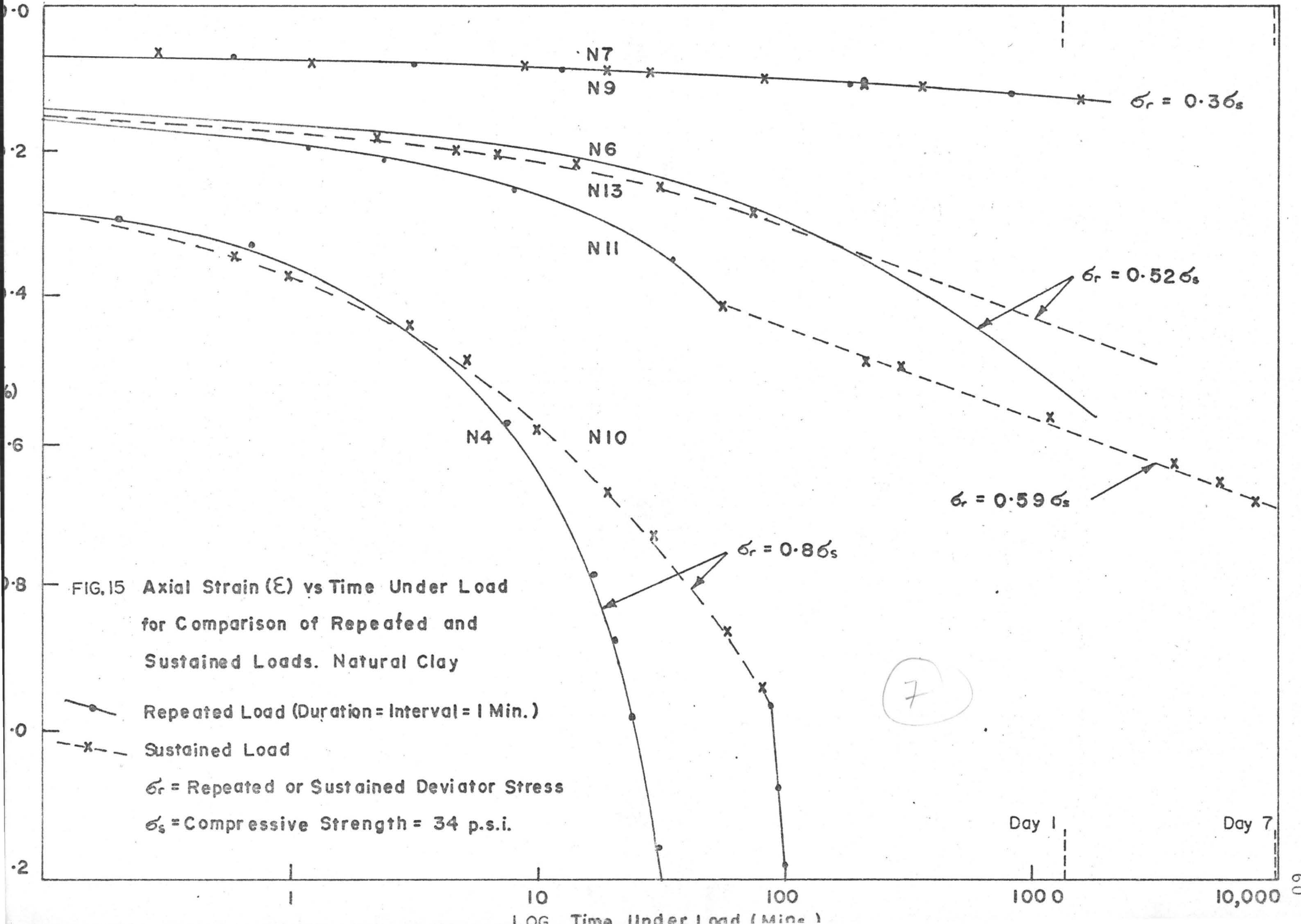


FIG. 15 Axial Strain (ε) vs Time Under Load for Comparison of Repeated and Sustained Loads. Natural Clay

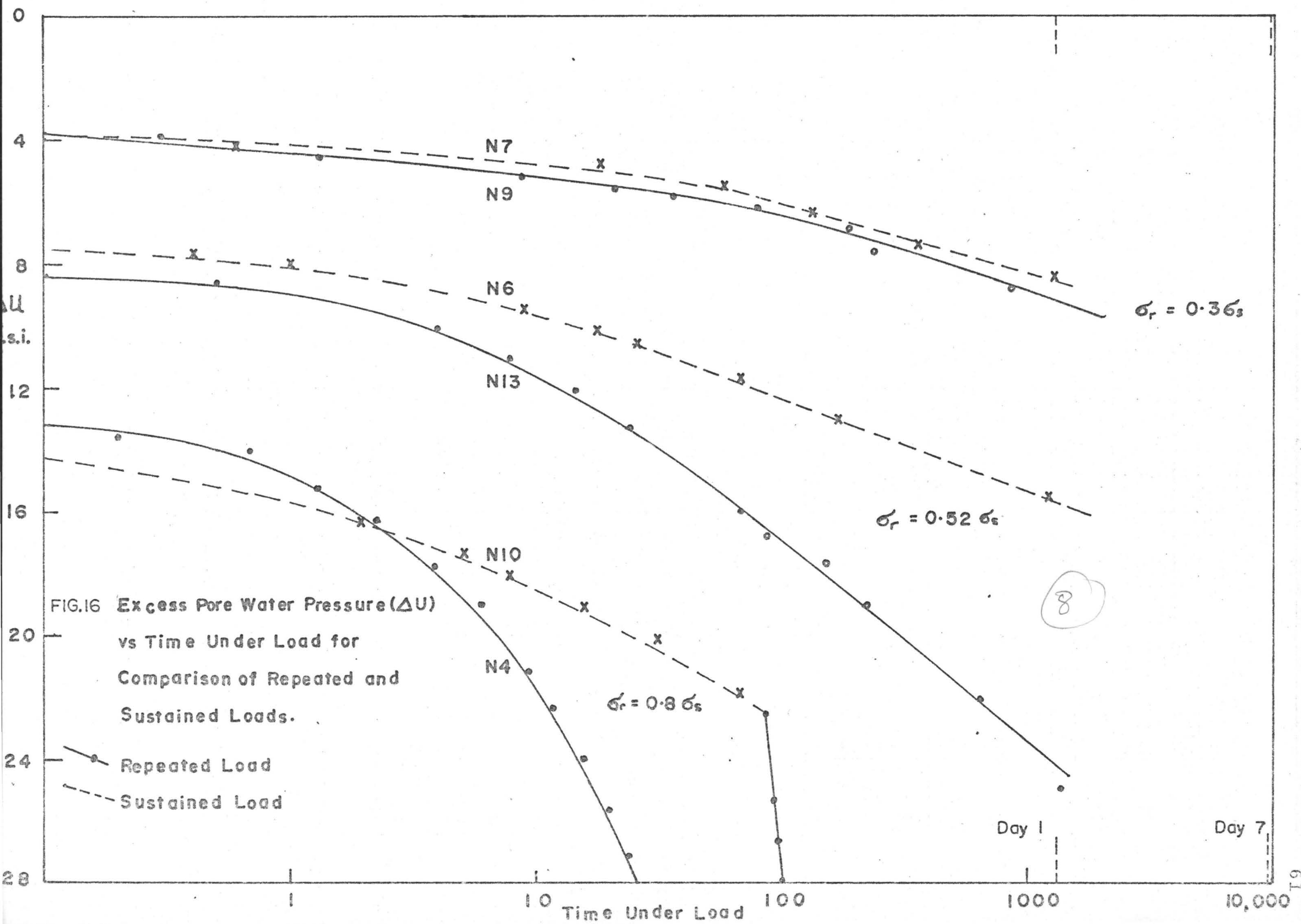


FIG.16 Excess Pore Water Pressure (ΔU) vs Time Under Load for Comparison of Repeated and Sustained Loads.

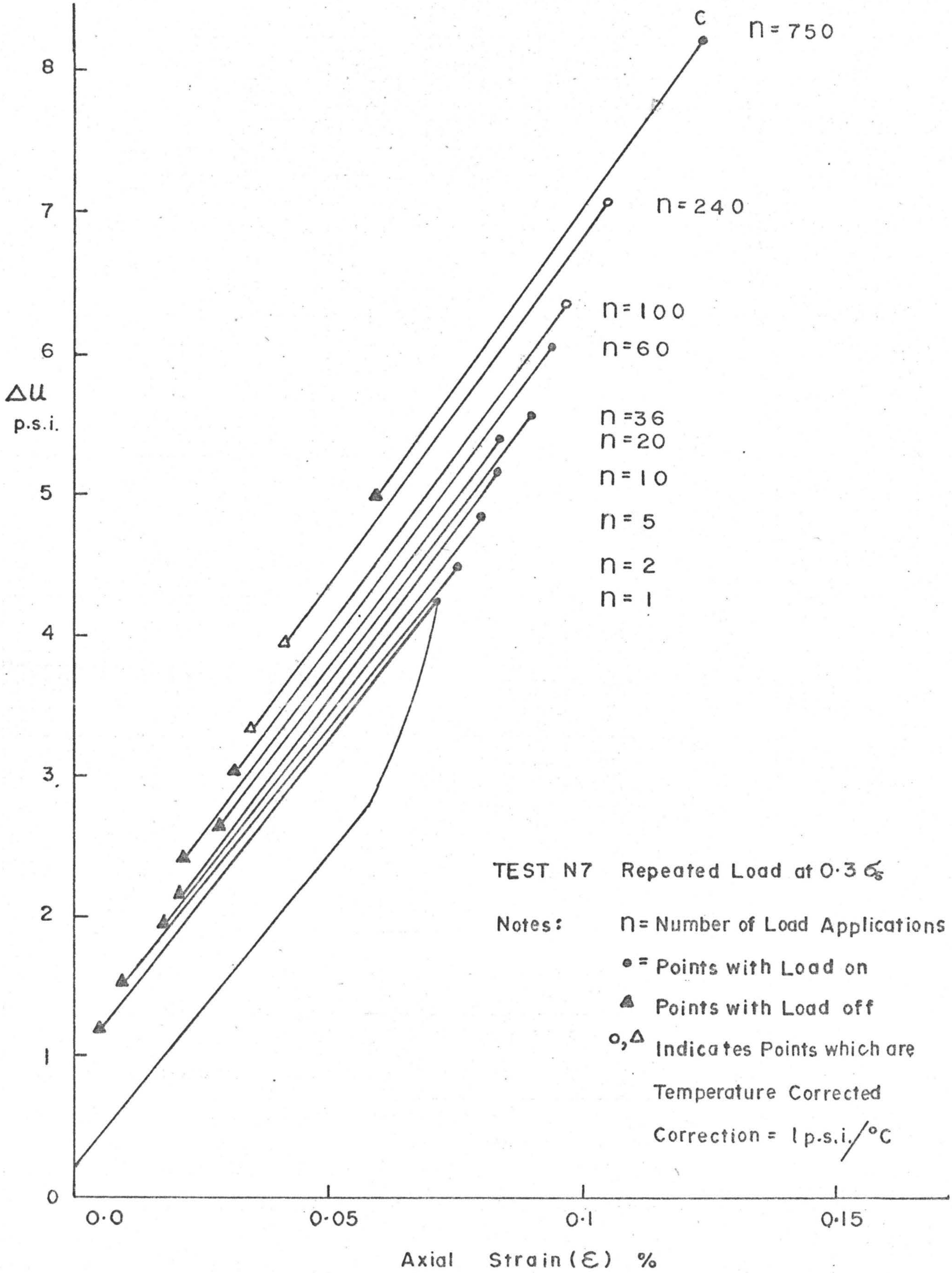


FIG. 17a EXCESS PORE WATER PRESSURE (ΔU) vs AXIAL STRAIN (ε) for TYPICAL REPEATED LOAD TEST on NATURAL CLAY

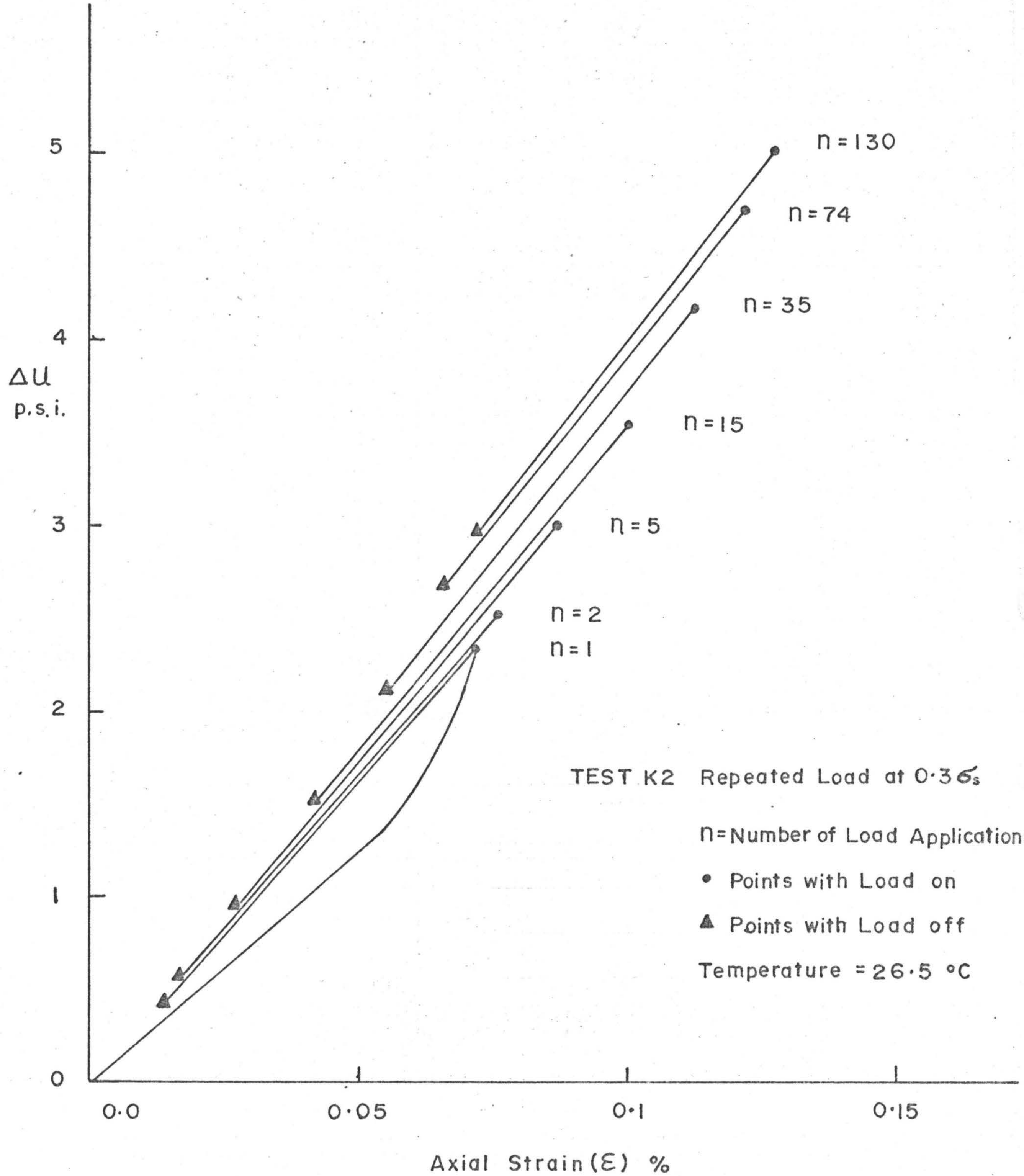


FIG. 17b EXCESS PORE WATER PRESSURE (ΔU) vs AXIAL STRAIN (ε) for TYPICAL REPEATED LOAD TEST on KAOLIN

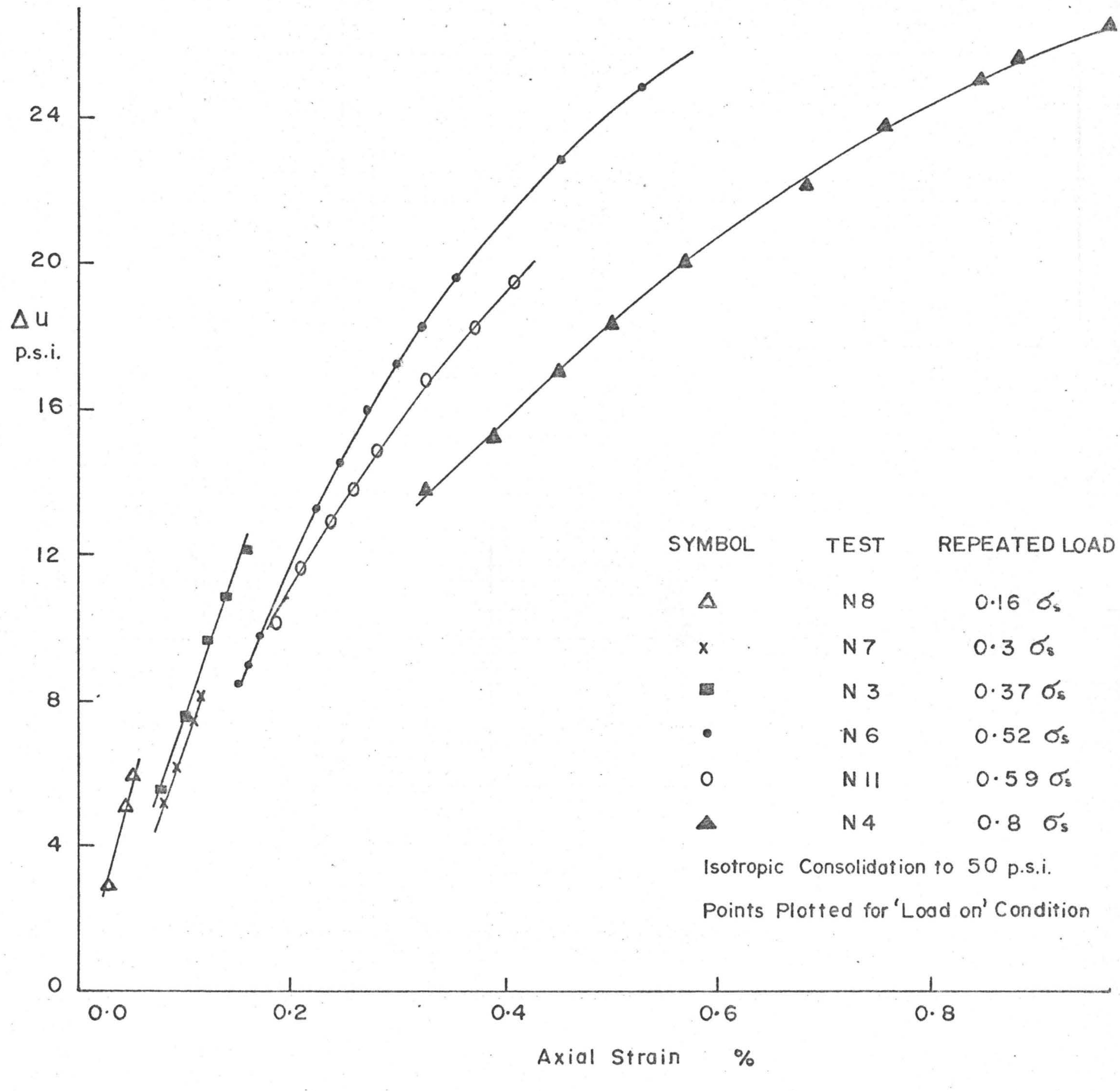
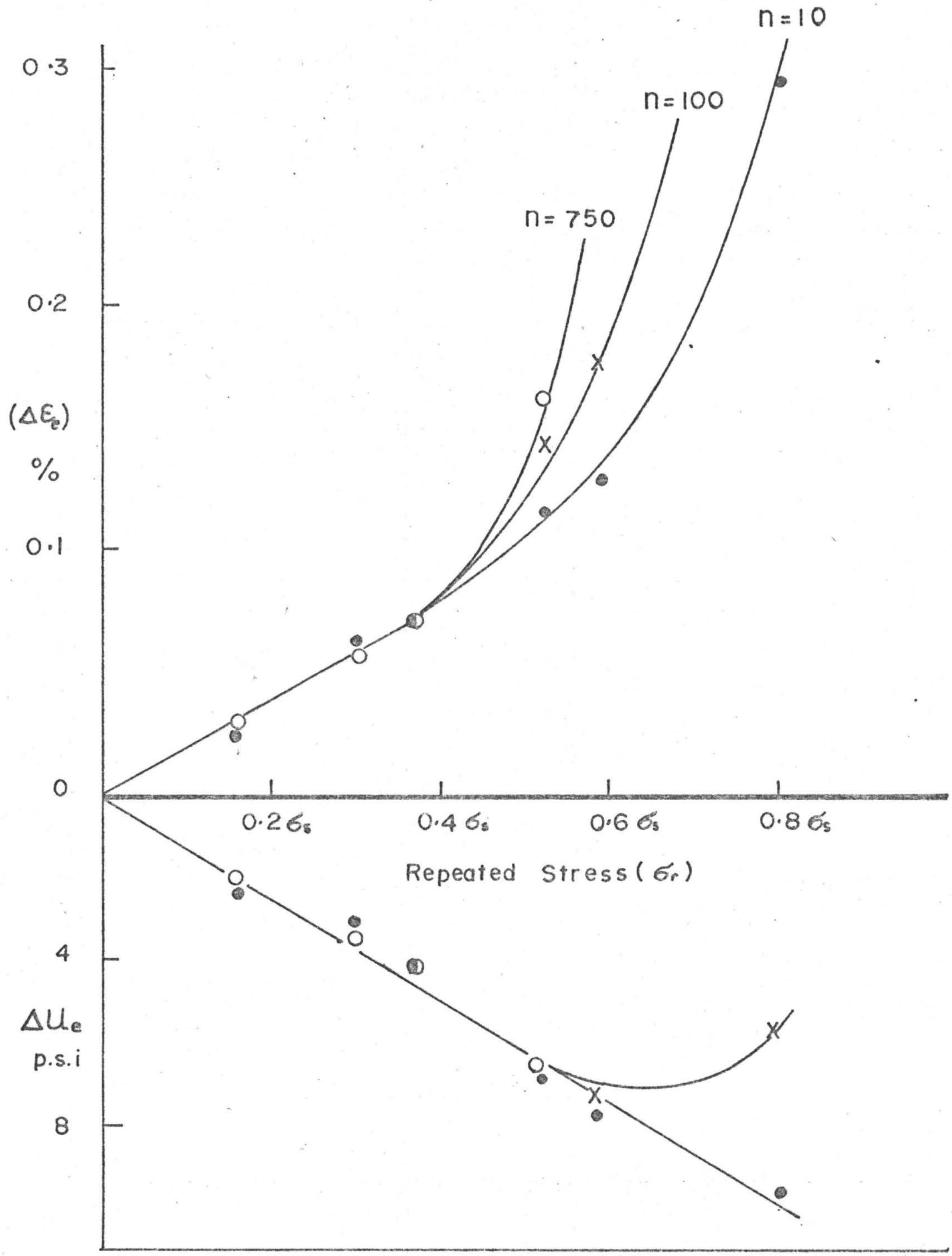


FIG. 18

PORE PRESSURE (LOAD ON) vs AXIAL STRAIN for REPEATED LOAD TESTS



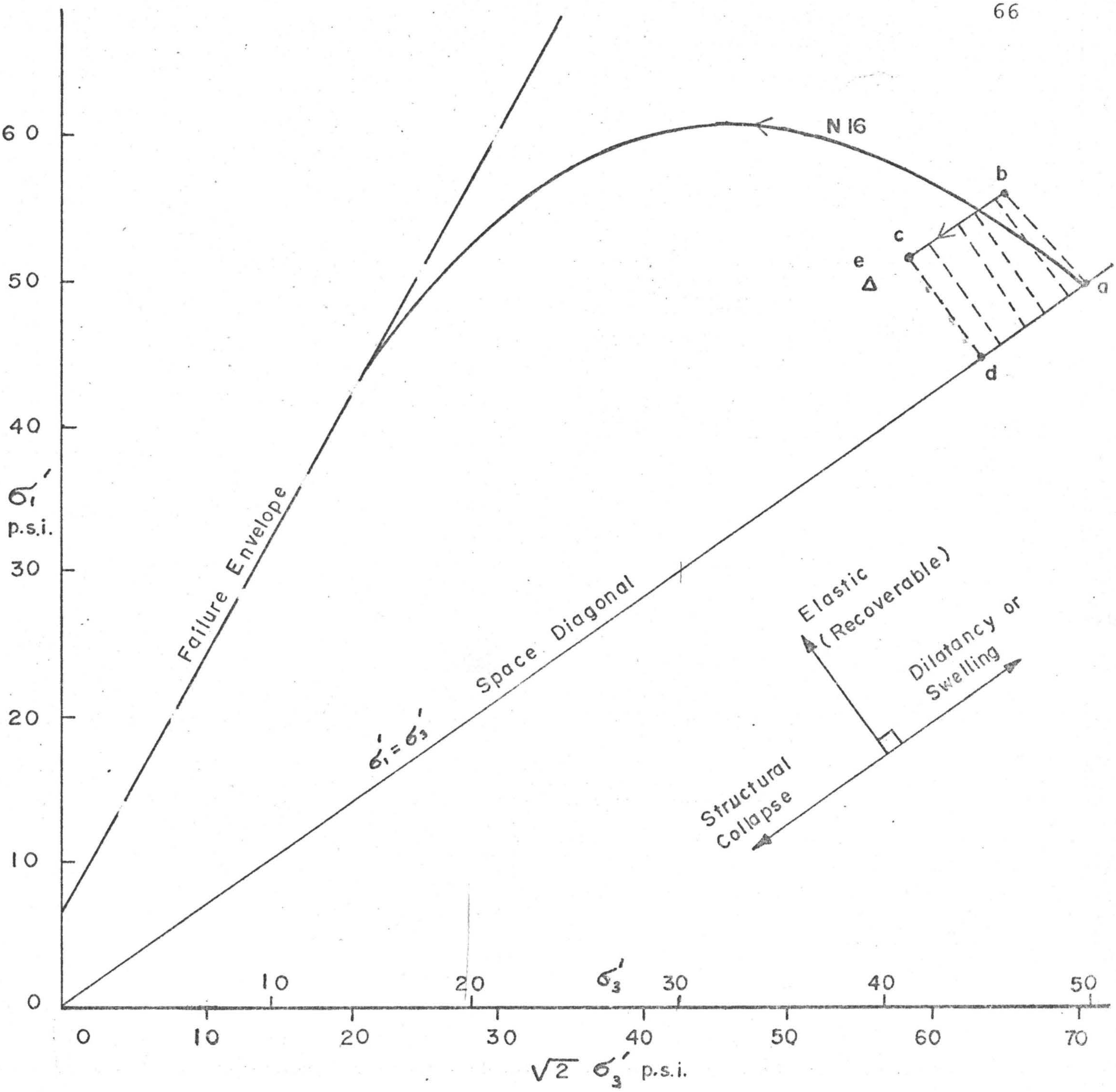
• After 10 Stress Applications

X After 100 Stress Applications

O After 750 Stress Applications

σ_s = Compressive Strength = 34·0 p.s.i.

FIG. 19 RECOVERABLE STRAIN ($\Delta\varepsilon_e$) and PORE PRESSURES (ΔU_e) vs LEVEL of REPEATED DEVIATOR STRESS



'R' TEST (Rate of Straining = 0·055 % per Minute)

Repeated Load Test, ($\sigma_r = 0{\cdot}3\ \sigma_R$, Duration = Interval = 1 Min.)

State of Stress After 1 day, Load Applied (Measured)

Δ State of Stress After 7 days, Load Applied (Predicted)

FIG. 20

EFFECTIVE STRESS PATHS for a TYPICAL REPEATED LOADING TEST and STANDARD 'R' TEST on NATURAL SILTY CLAY

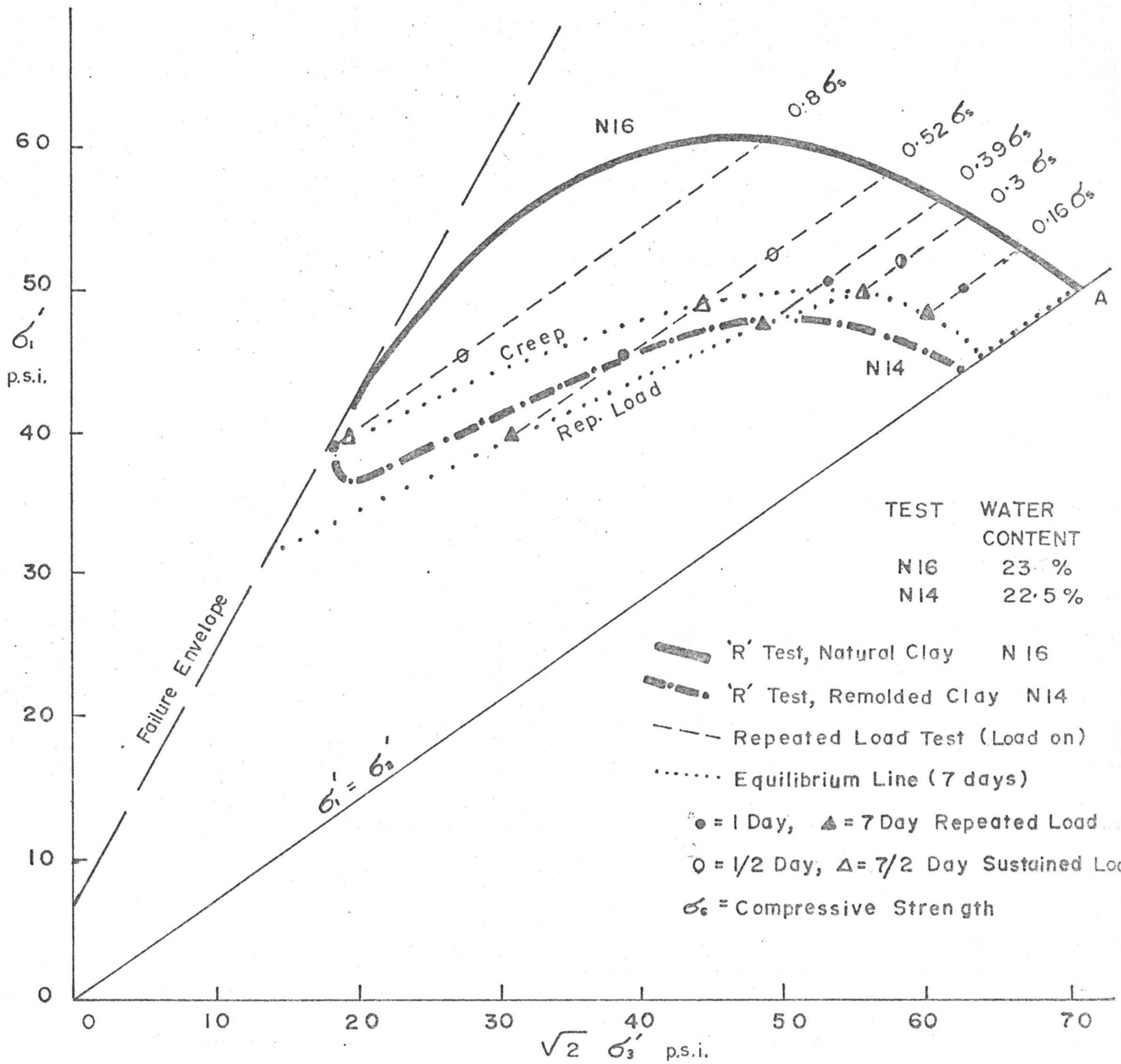


FIG. 21 STRESS PATHS and STATES of STRESS after REPEATED LOADING and CREEP of NATURAL CLAY

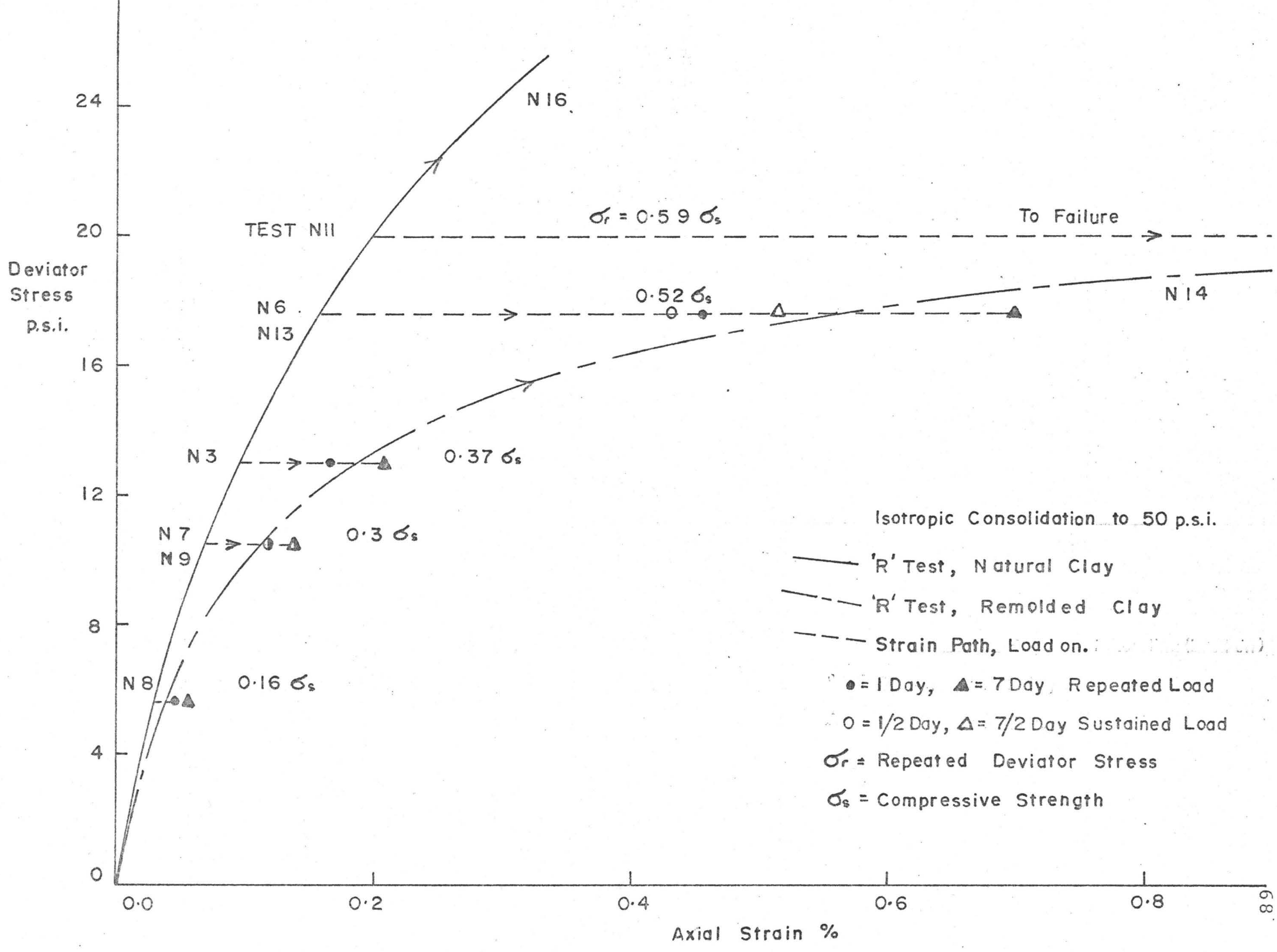


FIG. 22 DEVIATOR STRESS vs STRAIN for REPEATED LOADING, CREEP and 'R' TESTS

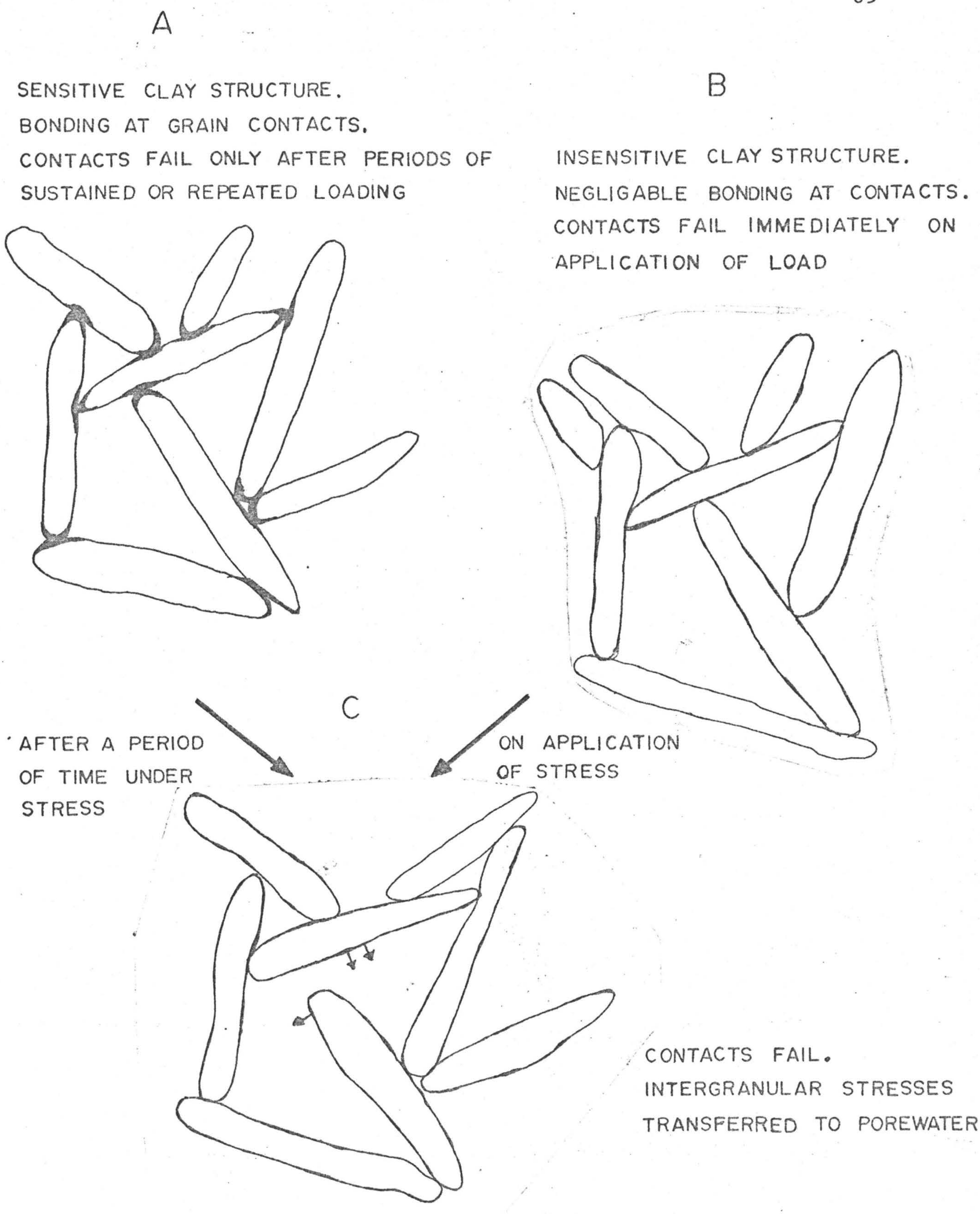


FIG. 23

SIMPLIFIED MECHANISTIC PICTURE for the BEHAVIOUR of
NORMALLY CONSOLIDATED CLAY UNDER APPLIED STRESS

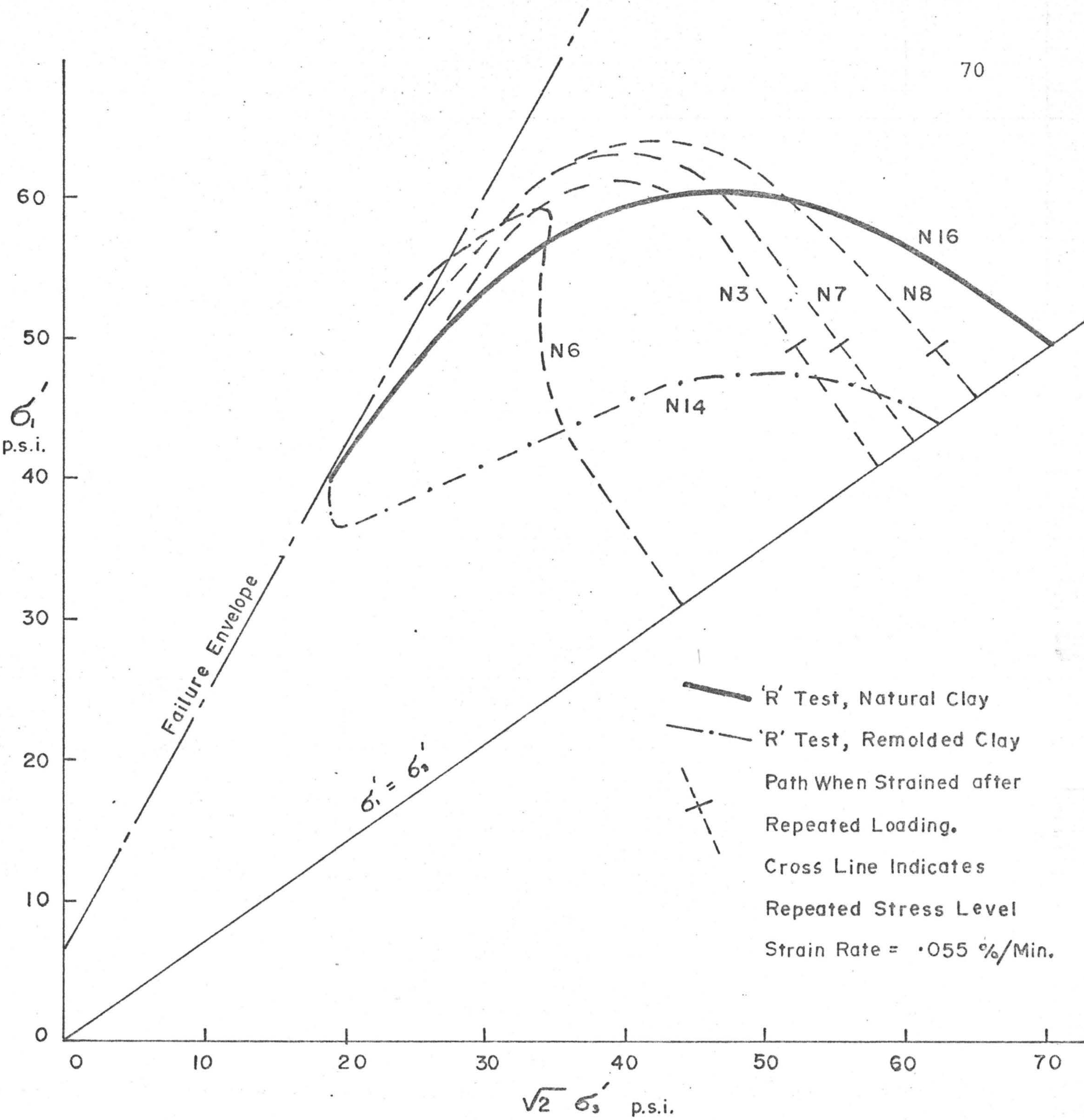


FIG. 24

STRESS PATHS for NATURAL CLAY SAMPLES STRAINED to FAILURE after REPEATED LOADING

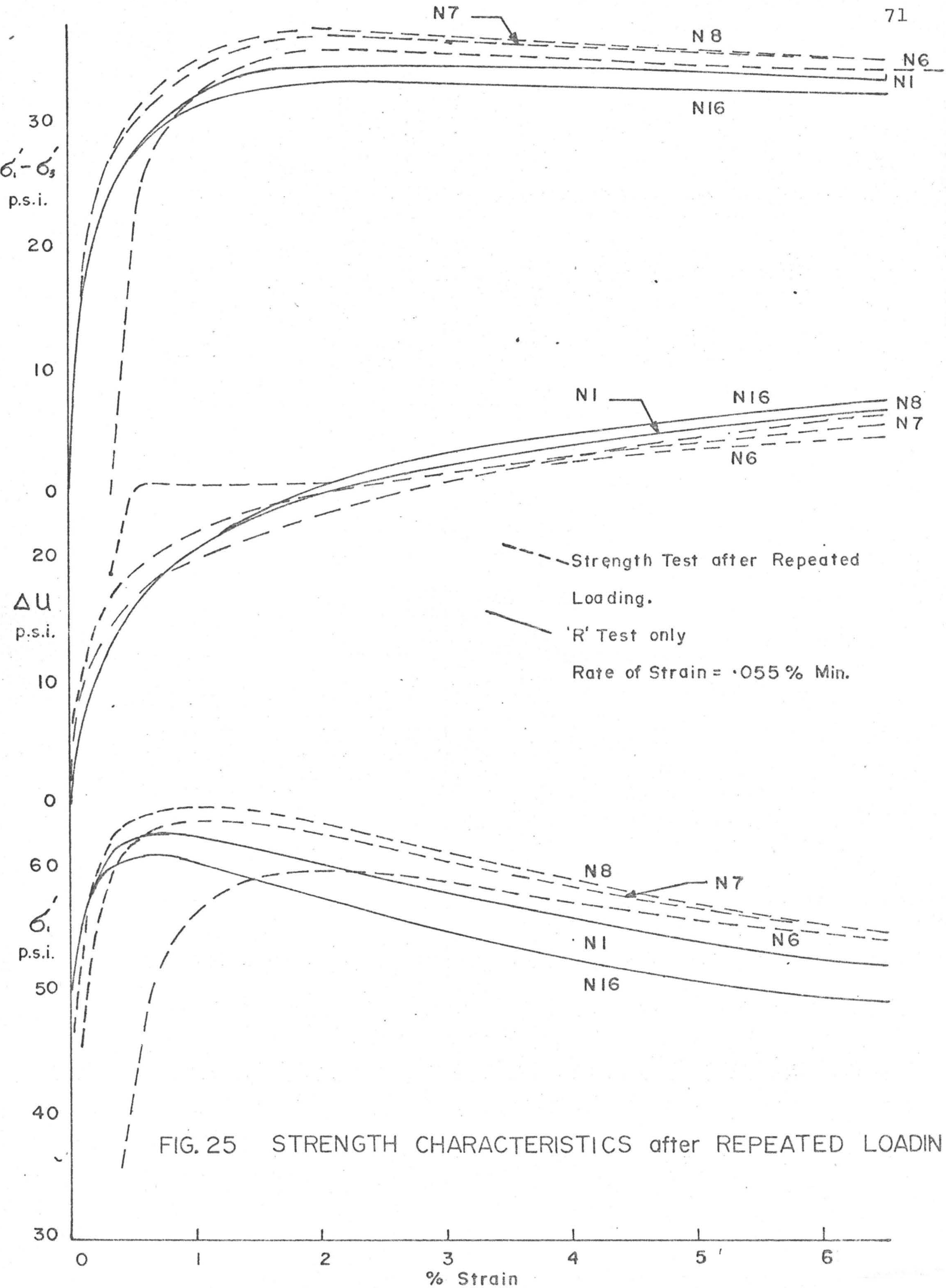


FIG. 25 STRENGTH CHARACTERISTICS after REPEATED LOADING

CHAPTER 6

PRACTICAL CONSIDERATIONS

6.1 Pore Water Pressure/Strain Considerations

Testing of soils is limited both in the laboratory and in the field, by the testing equipment available. The simple soil strength tests (unconfined compression or vane test for example) are widely used because of their simplicity, and they provide the engineer with an indication of the soil strength available. For a more complete study of the behaviour of soils under load, a better simulation of ground conditions is required. Triaxial compression tests may be used to simulate the horizontal effective and total stress in the ground before a shearing stress is applied. In Fig. 26 an element of soil in the ground is considered.

Initially, the element is assumed to be normally consolidated at a state of equilibrium. Upon application of a load, $\Delta\sigma_V$, an increase in the vertical total stress results (assumed to also equal $\Delta\sigma_V$), and a change in the horizontal total stress occurs ($\Delta\sigma_H$). The initial induced pore water pressure is due to the elastic compression of the soil, and must be taken as the average of the applied total stresses (Lo 1969).

$$\text{i.e.} \quad \Delta u_e = \frac{\Delta\sigma_V + 2\Delta\sigma_H}{3}$$

The value of $\Delta\sigma_H$ is difficult to predict in the field since it depends on the compression characteristics of the adjacent soil elements.

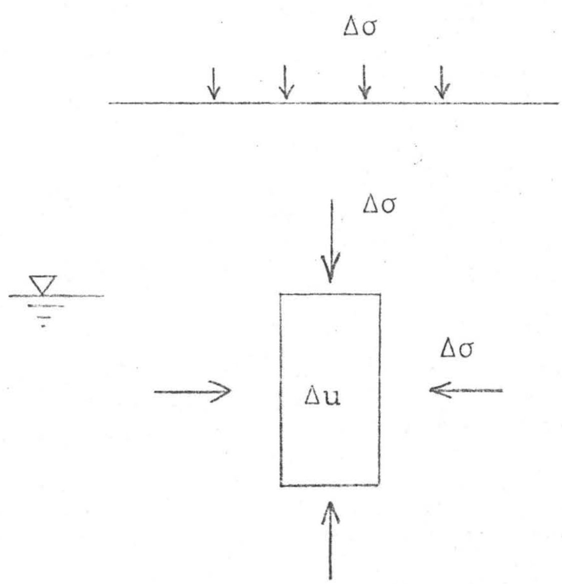


Fig.26 Typical Element of Saturated Clay

If the adjacent soil elements are assumed to be rigid then $\Delta\sigma_H$ will be equal to $\Delta\sigma_V$, and the elastic induced pore pressures will be $\Delta u_e = \Delta\sigma_V$. This is the case of the one dimensional consolidometer.

If the adjacent soil elements offer no resistance then $\Delta\sigma_H = 0$ and the induced pore pressure will be $\Delta u_e = \frac{1}{3}\Delta\sigma_V$. This is the case of the triaxial compression under constant cell pressure. A vertical excavation in clay, in which the horizontal stress is released, is the classic example of soil with no lateral restraint.

In most practical cases the adjacent soil elements can offer varying degrees of resistance depending on the depth of the element and the properties of the soil under passive soil pressures. There will usually be some deformation in the horizontal direction allowing a corresponding amount of vertical strain.

The experimental data presented in this thesis shows that for small strains in the non failure condition the induced permanent pore water pressures are proportional to the major principal strain of the sample. On this basis it should be possible to predict the pore water pressures induced in a clay layer by obtaining the linear pore water pressure versus axial strain relationship from a laboratory test sample, and relating this to the predicted or measured strains. Lo (1969) has suggested that the results obtained in the triaxial sample may be directly related to the field element by comparing the strains. The effects of different consolidation pressures may be taken into account by dividing the pore water pressure by the consolidation pressure to obtain a normalized relationship (Lo 1969).

The author is of the opinion that a clay layer in the undrained state may be analysed for a given load by considering separately the immediate and the time dependant strain distribution due to that load.

A well documented case history of the construction, failure and reconstruction of a highway embankment on varied clay was reported by Lo and Stermac (1965) and Stermac et al (1967). The field recordings of pore water pressures and settlements indicated that after construction had ceased, the pore water pressures and settlements continued to increase (linearly on a log. time scale). Walker used the example to illustrate the importance of creep effects under a sustained load. For embankments of this type the available drainage must be assessed to prevent a build up of pore pressures under sustained load.

6.2 Effective Stress Plots

The effective stress paths under condition of sustained or repeated loading, show large variations from the stress paths obtained from a standard undrained 'R' test. It is therefore most important that a stress path is chosen which is applicable to the particular field conditions.

In considering the case of an open excavation in which the total horizontal stress is reduced, the total and effective stresses may be drawn on a stress path plot (Fig. 27). Henkel (1970) used a similar plot to illustrate the stress behaviour of a soil consolidated under K_o condition, and sheared under conditions of plane strain; but Henkel overlooked the variation in stress path with different loading conditions. The results used in Fig.27 were obtained from the tests reported in chapter 5; the soil is isotropically consolidated. This is not true of practical conditions but the general behaviour is indicative of the behaviour under the field condition of K_0 consolidations. a'b' represents the stress path of a sample which is sheared by increasing the strain at a constant rate of 0.055 per cent/min. ('R' test). a'c'c" and a'd'd" are the paths of samples which are subjected to a constant applied deviator stress of $0.5\sigma_s$ and $0.8\sigma_s$ respectively.

We may assume that the soil element is initially at total stress state A with a ground pore water pressure of U_0. As the total horizontal stress is reduced by excavation, the total stress state moves along the path ab. If the effective stress path a'b' is used it would appear that the total stress may be reduced to the point B before failure occurs. However, if the effective stress path of a sample subjected to sustained loading is used, it is seen that the horizontal stress may only be reduced to the point D before failure occurs after a period of time. Movement of the state of effective stress along the stress path d'd" depends on time. If the soil remains undrained with the total stresses at point D then failure will eventually occur. Time under constant total stress is very important in the stability of earth works. In the truly undrained state pore water pressures continue to increase with time. In practise however, after a period of time, pore water pressures dissipate at a faster rate than they are generated and the factor of safety therefore increases.

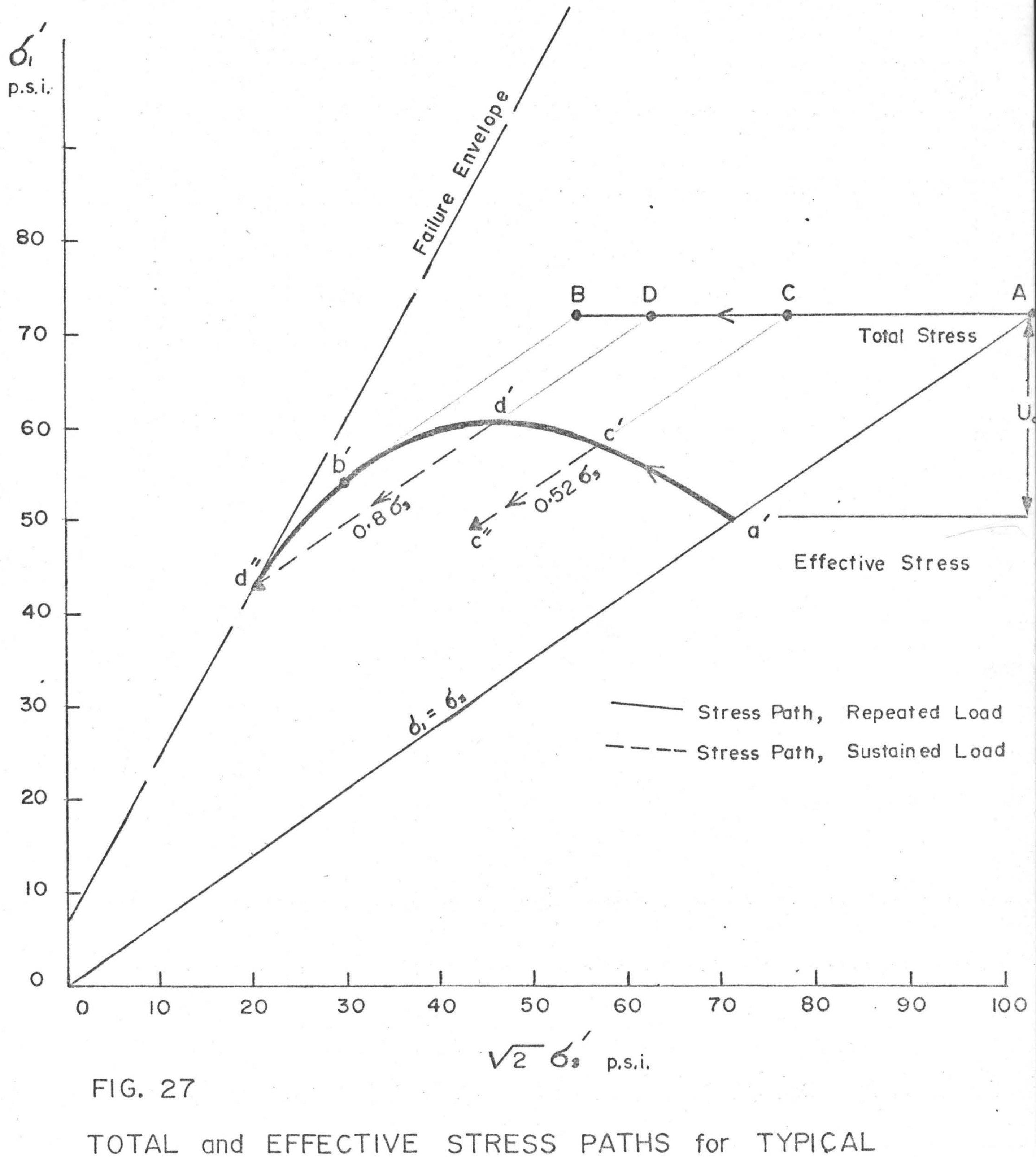


FIG. 27

TOTAL and EFFECTIVE STRESS PATHS for TYPICAL OPEN EXCAVATION

The soil in most field cases behaves under conditions of plane strain in which the intermediate principal strain is equal to zero (i.e., under a continuous footing). Plane strain tests are difficult to perform in the laboratory. In this discussion only the stress paths under conditions of axial symmetry are considered. The stress paths under plane strain conditions will be different. The elastic component of pore pressures will probably be greater for plane strain due to the resistance offered by the intermediate principal strain; and for a given axial deformation, greater permanent pore pressure will be produced because more disturbance of the soil will take place. However, the general shape and behaviour of stress paths for plane strain and axial symmetry will be similar. The results of loading under different total stress conditions have been shown to be related (Lo 1969).

6.3 Sampling of Sensitive Clays

Whenever a sample of soil is removed from the ground the release of in situ stresses causes a certain amount of sample disturbance. Disturbance to a greater degree occurs when a sampling tube is pushed into the ground. The process of sampling induces strains within the sample which may be of the order of 2 per cent with a sample recovery of 98 per cent. This inevitably produces collapse of the soil grains structure (Rochelle 1970) with the accompanying induced pore water pressure.

The experimental results (Fig. 18) show that soil strain of only 0.3 per cent can produce an excess pore water pressure of 18 p.s.i. Very small strains give an irreversible structural change to a sensitive clay. The free water often seen at the ends of sample tube may be due to the flow of water because of pore water pressures set up during the disturbance of the sampling process. It would be of interest to record pore water pressures during the sampling of a sensitive clay.

The partial collapse of the soil structure will not greatly influence the strength of the soils unless a large quantity of water is allowed to drain out of the sample tube. The strength tests after repeated loading (Figs. 24 and 25) indicate that the soil would initially behave in a characteristic elastic manner if some previous disturbance had occurred to the soil structure. The ultimate strength may be increased slightly due to the dilatancy effects after particle collapse (ref. ch. 5.6).

It would be unwise to base stress/strain predictions on tube samples of sensitive clay unless disturbance can be kept to an absolute minimum. It would be very misleading to attempt to determine excess pore water pressures from such a tube sampling where collapse of the soil structure has already occurred. A verbal comment was made in a discussion by Morgenstern at the A.S.T.M. Symposium on sampling of soil and rock (Toronto 1970). He suggested that in certain cases, perhaps the remolded strength of sensitive soils is what engineers should be using. The author would agree that for the long term undrained case the sensitivity of a soil cannot be relied upon. The peak strength of the sensitive soil may be utilized only if immediate drainage is ensured to avoid the increase of pore water pressure with time which would bring the soil to the failure state.

6.4 Rates of Shear Testing

It has been shown that the permanent, or plastic strains under load are time dependant (Chapter 5). In the standard triaxial test ('R' or 'Q') the strain is increased at a reasonably constant rate, and the deviator stress on the sample gradually increases until failure occurs.

If the rate of testing is very high, collapse of the soil structure will not occur to any great degree, and the soil will behave almost perfectly elastically (Fig. 28). A typical path might be a-c.

If the rate of testing is slow, the soil structure partially collapses and a typical stress path a-b results. The stress path moves towards the origin with increasing time to failure. Casagrande and Wilson (1951) showed that the undrained shear strength decreases linearly with log.

time to failure.

For large times to failure, the stress path for constantly increasing loads, tends to a constant position because, as more structural collapse occurs, more dilatancy is necessary before shearing can take place between the soil grains (see Chapter 5.6). For this reason the stress path for constantly increasing loads can never coincide with the stress path under sustained load.

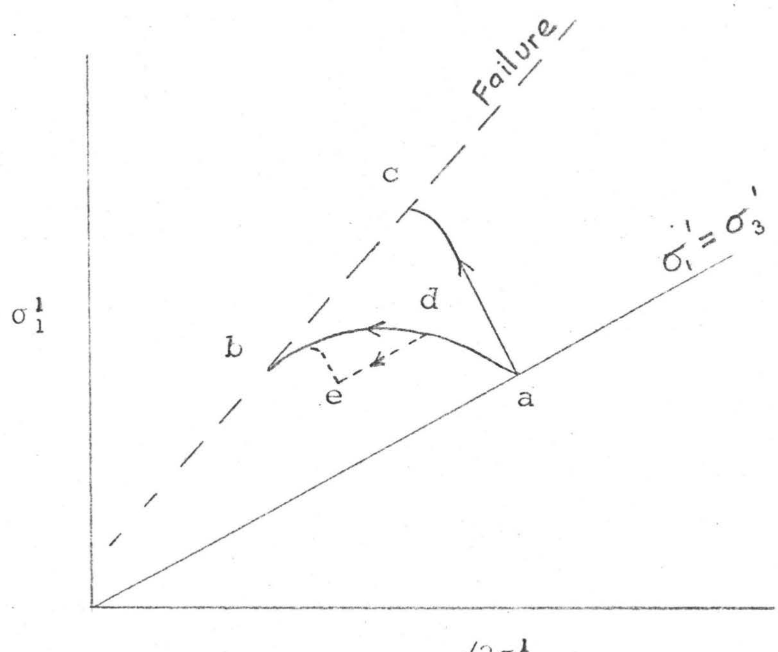


Fig. 28 Effect of Rate of Loadings on Stress Path

Under sustained loads the pore water pressures gradually increase with time (path d.e.) until failure occurs. However, if the load is increased before failure, the sample dilates and the negative pore water pressures result in an apparent increase in the strength of the soil. (path e.b.)

This is illustrated by the samples which are tested for strength after repeated loading. They all returned to the original stress path. (Fig. 24). The author considers this stress path to be typical of a constantly increasing load only. The strength of a soil in the undrained condition may be determined from a test at a sufficiently slow rate of loading, that the major proportion of possible collapse of the grain structure occurs at a given applied stress level. This would mean test rates to give failure in say 7 days or longer. Sustained load tests are preferable for accurate determination of the true undrained strength.

Triaxial test rates are therefore limited more by the characteristics of the soil structure under load than by the necessity to equalize pore water pressures. The sensitivity of a clay soil may be taken advantage of providing the type and rate of loading are carefully controlled, and an appropriate stress path is used for the study. There is no unique effective stress path for a particular sensitive soil sample.

SUMMARY AND CONCLUSIONS

1. When a repeated or sustained deviator stress is applied to an undrained, normally consolidated, clay sample, under triaxial conditions, the axial strains and pore water pressures continue to increase with time.

2. The sample subjected to a repeated or sustained deviator stress fails at a stress level considerably lower than the compressive strength of the soil as obtained from a standard strain controlled test ('R' test).

3. The level of repeated or sustained deviator stress at which the soil fails is termed the critical stress level.

4. The research data shows that the level of repeated or sustained stress at which the soil fails (Critical Level) varies with the time of loading; for example, for a duration and interval of 1 minute, the natural silty clay samples failed at $0.80\sigma_s$ after 60 minutes of repeated loading, $0.59\sigma_s$ after 1000 minutes of repeated loading, and $0.50\sigma_s$ after approximately 10,000 minutes of repeated loading.

5. For small deviator stresses (zero to $0.37\sigma_s$ for the natural silty clay) a small increase in the repeated or sustained deviator stress produces mainly an elastic, (and recoverable) increase in the strains and pore water pressure after 1 day under load. This may be termed the region of 'elastic' behaviour of the soil.

6. For repeated and sustained deviator stresses greater than $0.37\sigma_s$ a small increase in the repeated or sustained deviator stress, after 1 day, produces mainly a plastic (and permanent) increase in the strain and pore water pressures. This may be termed the region of 'plastic' behaviour of the soil.

7. Repeated loading at stress levels within the region of elastic behaviour has no effect on the clay. The deformations and pore water pressures depend only on the time under load.

8. Repeated loading within the region of plastic behaviour produces axial strains and pore water pressures greater than those under a sustained load of the same magnitude. The critical stress level under repeated loading is lower than that under a sustained load.

9. The axial strain due to repeated loading may be divided into two components;

   a) An elastic or recoverable component ($\Delta\varepsilon_e$) due to the elastic compression of the soil structure under load.

   b) A plastic permanent component ($\Delta\varepsilon_p$).

10. The generated pore water pressures can be divided into two distinct components:-

    a) A recoverable component (Δu_e) due to the elastic compression of the soil structure under load. The recoverable pore water pressure is equal to $\frac{1}{3}$ of the applied deviator stress, as is predicted by elastic theory.

    b) A permanent component (Δu_p) due to the collapse of the soil structure under load.

11. For values of repeated and sustained deviator stress within the 'elastic' range the recoverable component of pore pressure (Δu_e) is proportional to the recoverable strain ($\Delta\varepsilon_e$), and permanent pore water pressure (Δu_p) is proportional to the permanent strain ($\Delta\varepsilon_p$).

12. The linear relationship between Δu_p and $\Delta\varepsilon_p$ is explained by a time dependant collapse of the bonded grain structure, resulting in a transfer of grain to grain stresses to the pore water with a corresponding straining of the soil.

13. For greater values of stress (0.37 to $1.0\sigma_s$) shearing occurs between the soil grains. Axial deformation occurring without further pore pressure increase results in a non linear pore pressure versus axial strain relationship.

14. The effective stress paths for samples under repeated or sustained loading differ greatly from the stress path of a strain controlled test. With time under load, the states of effective stress tend towards an equilibrium position.

15. After a certain time under load, the equilibrium points for different values of sustained or repeated stress form lines in effective stress space known as the equilibrium lines.

16. In the elastic range of stress, the equilibrium line is the same for both repeated and sustained stresses. In the plastic range of stress, where repeated loading increases the strains and pore pressure, the equilibrium lines are different for repeated and sustained stresses.

17. The axial strain and the effective stresses after a period of sustained loading of a natural clay sample are similar to the effective stresses, and axial strains developed in an 'R' test on a remolded sample of the same soil at the same water content.

18. The behaviour of a sensitive saturated clay under load may be explained by the time-dependant breakdown of the bonds between the grains of the soil, causing partial collapse of the soil structure and a transfer of inter-granular stress to the pore water.

19. The strength of a clay sample which does not fail under repeated or sustained loading is 5 to 10 per cent greater than the strength of a sample which is not subjected to a repeated load. It is likely that the partial collapse of the soil structure under repeated loading results in more interlocking between the soil grains, therefore, more dilatancy is necessary before shearing can occur.

APPENDIX 1

SOURCES OF ERROR IN THE TESTING PROCEDURE

1.1 Area Correction

The dimensions of all samples before consolidation were 3.0" high x 1.4" diameter. During isotropic consolidation of the samples, approximately 7c.c. of water were expelled and the sample dimensions decreased. The usual end effects were observed due to friction between the sample and porous stones (Bishop and Henkel 1957). The average cross sectional area used in determining the value of the applied deviator stress was calculated by the formula :-

$$a = a_o(1 + \frac{\Delta v}{v}) \qquad \text{(Bishop and Henkel 1957)}$$

where

$\frac{\Delta v}{v}$ = volumetric strain

a = new cross sectional area

a_o = original cross sectional area

The dimensions of samples after consolidation were 2.9" high x 1.37" diameter. No area corrections were applied for the repeated load stage oftesting as strains were usually small. There would however be a reduction in applied deviator stress for the samples which approached failure under higher values of repeated stress.

1.2 Membrane Strength

Two Trojan Prophylactics were used to seal the soil samples against water leakage. The correction to the compressive strength of a sample on account of the rubber membranes has been shown to be very small, (Bishop and Henkel 1957). A typical correction increases with increasing strain to a value of 0.2 p.s.i. at 10 per cent strain which is clearly negligible compared with the measured compressive strength of 34 p.s.i.

1.3 Filter Drain Strength

Filter paper drains ensure fast equalization of pore water pressures. Drains for the test series were cut from Cenco Brand No. 13250 filter paper which has a low wet strength The drain was constructed from one piece of filter paper wrapped around the sample in such a way as to overlap the upper and lower porous stones. Vertical slits were cut in the paper every ¼" around the sample circumference.

The effect of the filter paper on the compressive strength is rather uncertain. It is conceivable that a rigid cylinder of paper could withstand a high stress, but in the tests it was observed that buckling of the paper occurred during the consolidation stage. It is not conceivable that such a paper could support any significant proportion of the applied deviator stress therefore no correction was applied.

Any error from this cause would be constant for all tests and would not invalidate comparisons made between different modes of load application. The advantages of fast equalization of pore pressures (2.5) outweigh any disadvantages due to errors in compressive strength, and filter drains were used in all tests.

1.4 Membrane Leakage

For many years it has been accepted that a certain amount of leakage occurs through a rubber membrane. It has been shown (Bishop and Henkel 1957) that over a period of hours air passes through the rubber. This is used to advantage for removing air bubbles trapped between the membrane and the soil sample. (see Chapter 4).

A certain amount of water will also pass through the membrane; tests have indicated that the rise in pore water pressures due to this effect may be of the order of 2 p.s.i. per hour under a net cell pressure of 50 p.s.i. (Lopes 1970). However, if two membranes are used with a layer of silicone high vacuum grease between them, the leakage is reduced considerably. In tests carried out on Trojan Prophylactics around a stack of saturated porous stones (Lopes 1970) the leakage resulted in a pore pressure increase of 5 p.s.i. after 5 days under a net cell pressure of 90 p.s.i.

The net cell pressures used in the repeated load test series were 50 p.s.i. therefore a maximum pore pressure increase of 2 or 3 p.s.i. could be expected over a five day period. This effect is small compared with the variations in pore pressure due to temperature fluctuations. No corrections were necessary for membrane leakage as the undrained tests were generally of only 1 or 2 days duration.

1.5 Pore Water Pressure Equalization and Rates of Testing

When a stress is applied to an ideal triaxial sample under undrained conditions, an equal distribution of pore pressure is found within the sample. Often this does not occur because of the irregular stress distribution caused by the friction between the porous stones and the soil sample. If differences in pore pressures develop, migration of the pore water from one part of the sample to another may occur giving rise to a non uniform sample.

Bishop and Henkel (1957) rightly stated that the plotting of test results on the basis of a single measurement of pore pressure is justified only if the testing rate is chosen so that relatively complete equalization can occur.

Gibson obtained a theoretical relationship between the 'per cent equalization of pore pressures' and the time factor $T = C\ t/h^2$ where C is the coefficient of consolidation, t is the time to reach the degree of equalization required and $2h$ is the sample height.

For fully efficient all round drainage the time factor for 95 per cent equalization would be;

Kaolin $3\frac{1}{2}$ mins.

Natural silty clay 50 mins.

In the tests run in the research program, filter drains were used to shorten the drainage path. The measured pore water pressure is always an average of values over the length of the sample, but to prevent excessive pore water pressure differences and the resulting migration of water the following rates of testing were chosen:-

Kaolin	Natural silty clay
.008 cm/min	.0016 cm/min
(.275 % /min)	(.055 % /min)

APPENDIX 2

TEMPERATURE EFFECTS IN UNDRAINED SOILS

2.0 Effect of Temperature Variation on Measured Pore Water Pressure and Axial Strain

During the preliminary investigations into repeated loading effects, it was noticed that the results were showing a tendency to vary with the laboratory temperature. An investigation into the effects of temperature variation was made on a Kaolin sample which was consolidated isotropically to 25 p.s.i. for 1 day, then a sustained load of 8.2 p.s.i. was applied. After 10 days the pore water pressures had risen from the initial back pressure of 10 p.s.i. to 21.3 p.s.i. (at 21.5°C). The sample was cooled by opening the laboratory to the winter air and readings of pore water pressure and deformation were recorded.

The effects of the temperature change may be seen in Fig.A1. The thermometer was hanging freely next to the triaxial cell. When the temperature was reduced, an immediate decrease in dial gauge reading was recorded. The pore pressure change was negligible during this period. The dial gauge error was attributed to the sensitivity of the light metal parts of the dial gauge to temperature change. It is unlikely that the heavier parts of the

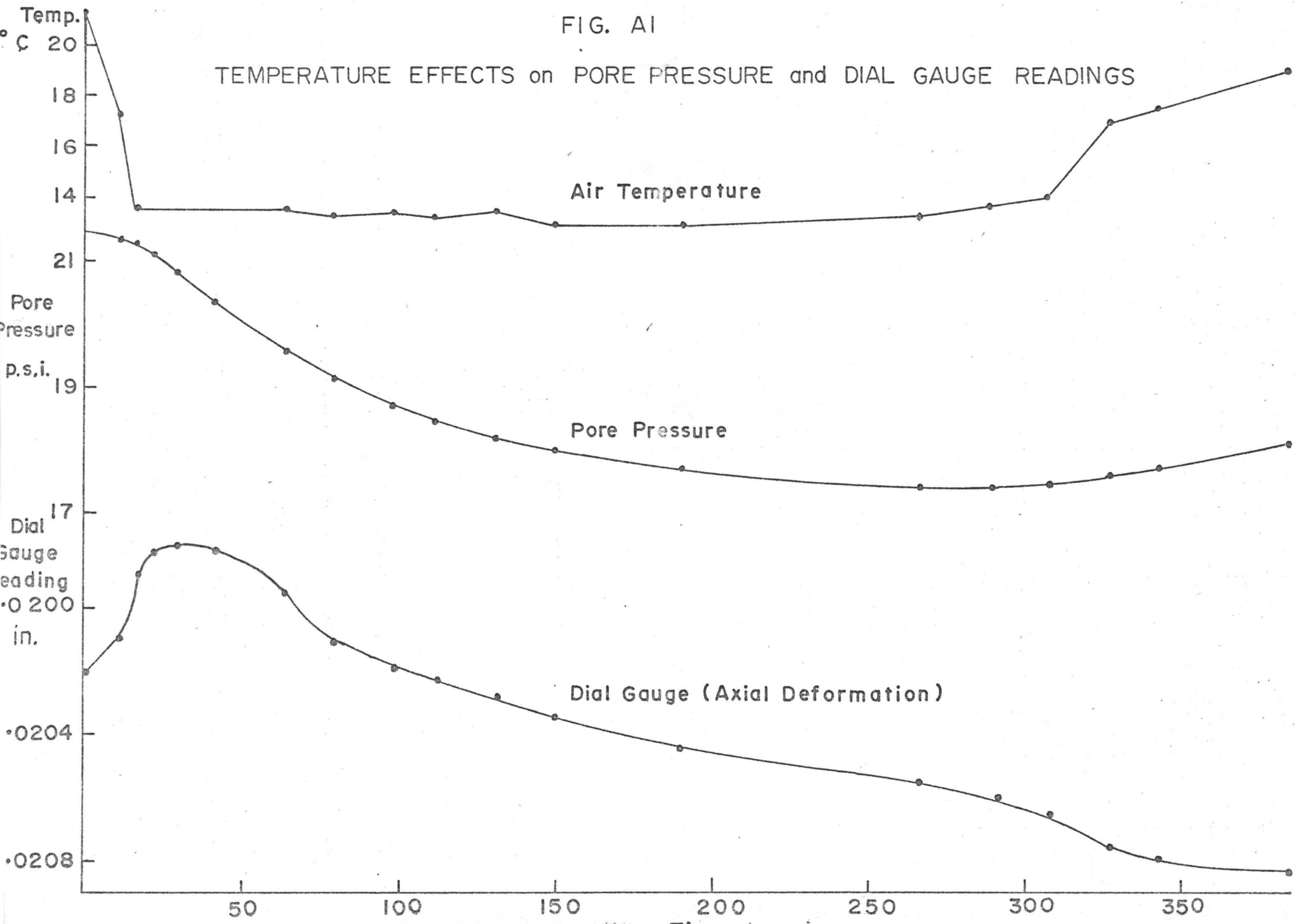


FIG. A1

TEMPERATURE EFFECTS on PORE PRESSURE and DIAL GAUGE READINGS triaxial cell would react so quickly to the change in temperature. A decrease of 1°C. caused an apparent swelling of the sample of 0.00005".

Under constant reduced temperature the dial gauge reading started to increase (apparent compression of the sample) and the pore water pressures decreased. At this stage the cell,water, and sample are cooling down causing a pressure and volume change within the sample. The relationship between temperature, volume, and pressure is difficult to predict theoretically because the soil is neither at constant volume (expansion is allowed even though the sample is undrained) nor at constant pressure. The deformations and pore pressure changes recorded depend on the nature of the soil and the thermal properties of the testing equipment. Dial gauge changes recorded were approximately .0001"/C (apparent compression of the sample).

The initial and final effects of temperature on the dial gauge cause errors which are in opposite directions giving a net error of .00005"/deg.C. It is not known whether the error is due to a contraction of the soil sample or a change in length of parts of the test apparatus.

The temperature fluctuations encountered during regular testing were ± 2°C and therefore dial gauge error was negligible.

The pore water pressure response to temperature is delayed because of the mass of the cell and water. Eventually a decrease of 1°C. gave a pore pressure decrease of 0.5 p.s.i. This is important for the accuracy of the test results, and it was decided that for all tests a thermometer would be placed inside the triaxial cell as close as possible to the sample. Corrections were applied to the measured pore water pressure whenever the temperature varied by more than $\frac{1}{2}$°C. from the temperature at the time the sample drainage was closed.

The pore pressure correction necessary for a natural clay sample consolidated to 50 p.s.i. was 1.0 p.s.i. /deg.C. D.A. Sangrey (1968) carried out a study on pore water pressure variations with temperature for clays consolidated at 57 p.s.i. He obtained experimentally a factor of 0.82 p.s.i./°F. (1.47 p.s.i./°C).

Sangrey suggested four major components which contributed to this overall factor.

1. The pore water (including the water in the porous stone and in the cell base).
2. The solid soil matrix.
3. The porous stones.
4. The triaxial cell base and transducer connection.

By considering a suitable coefficient of volumetric thermal expansion for each component he was able to arrive at a theoretical figure close to the true one. However, many assumptions were made and Sangrey concluded that for accurate results the laboratory temperatures must be controlled. It would seem that if large temperature variations are to be encountered, a temperature control test must be run to measure the fluctuations. Every set of apparatus will have its own characteristics and must be calibrated accordingly.

The few results available suggest that the correction is proportional to the net stresses acting on the sample, and therefore the correction would change as the pore pressures change.

The author would agree with Sangrey that the only sure way of avoiding error in the tests is to work under temperature controlled conditions.

The temperature in the McMaster Laboratory was maintained at 24°C. by means of an air conditioning unit. Fluctuations were of the order $\pm$ 2°C. Temperature variations less than $\frac{1}{2}$°C. were ignored. Corrections were applied to the measured pore water pressures for any variations greater than this.

2.1 Increase in Residual Pore Pressure (Δu_p) With Temperature Fluctuations

Sangrey (1968) presents an excellent review of literature cincerning the pore water pressures induced in a soil due to temperature change. Sowa (1963) showed a mechanism whereby an increase in temperature causes an increase in the pore water pressure which reduces the effective stress on the sample; there is therefore a tendency for the soil to swell. Sowa suggests that the soil will follow a typical rebound curve a-b (Fig. A.2) Presumably the change in void ratio is accounted for by the expansion of the water as the sample is undrained at all times.

When the temperature falls again, the effective stress rises, and the sample follows a reload curve such as b-c. Although the void ratio is the same at c, the soil is not in its original state and there will be a residual, positive pore water pressure. If the temperature is reduced further (below that at the time of consolidation) the effective stresses will be greater than at the time of consolidation. Subsequent temperature cycles may cause further pore pressure increases, depending on the range of temperatures involved. Sangrey modified Sowa's model to account for the limit of pore pressures due to cycled temperatures (Sangrey 1968).

The author suggests that pore water pressures induced by cyclic temperature changes may also be due to the mechanism of structural collapse. When the temperature is reduced, the pore water pressures decrease and the effective stresses increase. If the increase causes the effective stress to be greater than that during consolidation, the

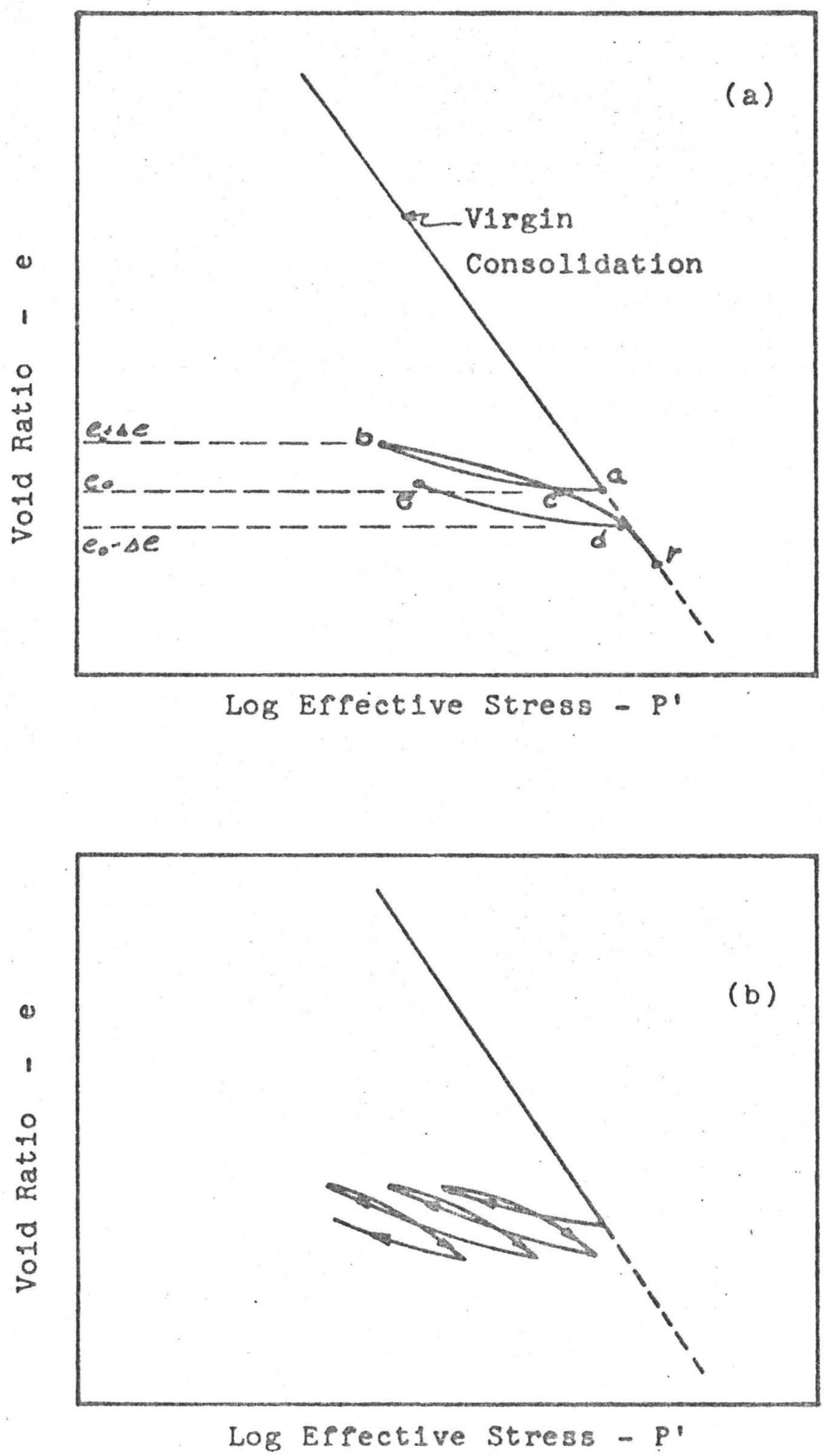


Figure #A2 The Irreversibility of Pore Water Pressure Change due to Temperature Variation

from Sowa (1963)

soil is effectively loaded, and there will be a tendency for more structural collapse to occur between the soil grains.

The author is of the opinion that structural collapse is a process continuing indefinitely with time. A sample consolidated isotropically under a given pressure will continue to experience secondary consolidation indefinitely. Secondary consolidation (drained state) is considered to be a process of structural collapse. In the drained state, drainage generally occurs at a rate faster than the pore pressures develop by the process of structural collapse. If the drainage is closed at any time during the secondary consolidation phase, a gradual increase of pore water pressures and axial strain occurs with time. The process of secondary consolidation is not observable in natural clays in situ because of the large times since sedimentation. It is impossible to simulate the consolidation process in the laboratory because of the time processes involved. When laboratory testing is carried out it is inevitable that some secondary consolidation (structural collapse) is still occurring, with a corresponding increase in pore water pressures and strain (undrained samples). This may account for some of the pore water pressures noticed during cyclic temperature tests.

The author concludes that there are three possible mechanisms by which residual pore water pressures could increase during cycling of temperature:-

1. Difference between rebound and loading p-e curves (Sowa).
2. Collapse of the soil structure due to higher effective stress.
3. Incomplete secondary consolidation (structural collapse) giving observable pore water pressure changes during the period of fluctuating temperature.

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