SHEAR AND K° SWELLING OF OVERCONSOLIDATED CLAY

RESISTANCE AU CISAILLEMENT ET GONFLEMENT K_o D'ARGILE SURCONSOLIDEE

СОПРОТИВЛЕНИЕ СРЕЗУ И НАБУХАНИЕ ГРУНТА ПРИ ЕСТЕСТВЕННОМ НАПРЯЖЕННОМ СОСТОЯНИИ

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SYNOPSIS. The geological history of an overconsolidated clay was reconstructed in the laboratory. Remolded Bearpaw Shale, consolidated under K_o conditions, was allowed to swell under similar conditions in an oedometer apparatus in which lateral stresses, volumetric strains and pore water pressures were measured. A brief description of this apparatus is presented. Direct shear tests on the same material were done with emphasis placed on using low levels of effective normal stress.

Results of the direct shear tests demonstrate that at very low stresses the failure envelope is curved and defines $c' = 0$ at $\sigma_N' = 0$. The changes in stress under K_o unloading led to $\sigma_H' > \sigma_V'$ and ultimately to passive pressure failure at the same failure locus defined by the direct shear. The progressive nature of these failures, particularly pore pressure - time - swelling relationships, are discussed.

INTRODUCTION

Heavily overconsolidated soils, particularly clay soils, result in nature due to a reduction in effective stress from some maximum value associated with the normally consolidated deposit. Usually this stress reduction is a consequence of erosion if, in fact, there is a significant thickness of heavily overconsolidated material. Several major problems have been identified with the engineering use of overconsolidated soil deposits and these can be ascribed to peculiar soil properties.

Foremost among these soil properties is the strength or shearing resistance offered by the soil. Quite generally the locus of peak shearing resistance of overconsolidated clay soil defines an effective stress failure envelope requiring for description both a frictional parameter, ϕ', and a cohesion intercept, c' rather than the single frictional parameter necessary for most normally consolidated soils. The degree to which this peak strength, particularly the "cohesion" part, can be used in design has been debated extensively. There is some consensus that for much long term design the "fully softened strength" or $c' = 0$ is most appropriate, Skempton (1970).

The fact of even lower shearing resistance being available under some circumstances was pointed out by Skempton (1964) in his concept of residual strength. At this residual strength shearing resistance is defined by a single frictional parameter, ϕ'_r, the residual friction angle. Large deformation in thin shearing zones and associated particle orientation are among the arguments offered to explain this phenomenon. Ultimately however, the question in practice is how and why these strength changes occur. Time, stress release, water content changes and deformation, both volumetric and distortional, are recognized as important variables which collectively are termed progressive failure, Bjerrum (1967).

In order to fully understand progressive failure one must know the distribution of stress within the soil and the changes that result when deposits are unloaded. Henkel (1970) addressed this problem as it relates to earth pressure. Several studies, notably Brooker and Ireland (1966) and Skempton (1961), have been concerned with evaluating K_o, the coefficient of earth pressure at rest, for unloaded soils.

In Brooker and Ireland's laboratory studies samples of soil were consolidated in a specially constructed oedometer which permitted measurement of the radial or horizontal stress as the sample was loaded and unloaded. They observed that as samples were unloaded axially the radial stress decreased at a lower rate; this resulted in an increase in K_o with increased overconsolidation ratio, OCR. Skempton had measured a similar phenomenon in a natural field deposit and concluded that the horizontal stresses could become so large, relative to the vertical, that the soil reached a state of passive pressure failure.

Integrating all of these factors (stress release, reduced strength of overconsolidated soils and the progressive nature of the process) would seem to require detailed study of natural deposits. There is a difficulty in that sampling of natural soils, no matter how carefully done, results in a break in the stress history and the full significance of such a break has not been determined. Limited work on such sampling effects has been done by Ladd and Lambe (1963) and Skempton and Sowa (1963) but they were concerned primarily with undrained strengths of softer soils.

In this paper results are presented from a study of the stress changes and shearing resistance measured on samples of soil prepared in the laboratory. The complete stress history leading to heavily overconsolidated soil was reproduced in specially constructed apparatus. At various stages of overconsolidation, and without a sampling break, the drained strength of the soil was determined. Particular emphasis in all testing was placed on studies at very low levels of normal stress.

Several different clay soils, representing the three main clay mineral groups, were tested in this program. All demonstrated the same behavior.

MATERIAL AND TESTING PROGRAM

In view of the above considerations, a testing program was initiated with the objective of defining strength-deformation-time relationships for heavily overconsolidated clays at low stresses. Among other soils tested was a remolded Bearpaw Shale which characterized the extremes of behavior; properties of this soil are listed in Table I. The original sample of Bearpaw Shale was taken from the South Saskatchewan River Dam site and was supplied by the late R. F. Peterson of the Prairie Farm Rehabilitation Administration.

Table I Properties of Bearpaw Shale

Liquid limit (undried)	82%
Plastic limit	18%
Clay content < 2 μ	50%
Specific Gravity	2.76
Mineralogy	
Bulk Minerals	≈30%
Hydrous Mica	≈10%
Montmorillonite	≈60%

The results of two types of test will be discussed in this paper: drained shear, and K_o consolidation and swelling tests. Both tests were done using apparatus and procedures somewhat different from those normally used in soil testing.

In order to reduce problems of sample tipping and the long time necessary for incremental swelling, direct shear tests were done on very thin samples, 2 - 3 mm in thickness and 7.6 cm in diameter (Kenney, 1967). In preparation for shear testing, soil at approximately the liquid limit was placed in the direct shear apparatus within a circular confining ring. Increments of normal load were applied until the specimen was normally consolidated at the desired maximum stress. When overconsolidated specimens were prepared, the sample was unloaded and allowed to swell under reduced stress corresponding to the desired overconsolidation ratio (OCR). The lateral confining ring was then removed and the specimen was sheared at a controlled rate of deformation of approximately 1.3×10^{-4} cm/min. Time to failure was typically several days or more which corresponds to pore pressure dissipation in excess of 90%. Some of the direct shear tests referred to in this paper were done on thicker specimens (approximately 1 cm); except for this, the testing procedure and apparatus were identical to the scheme described.

K_o consolidation testing was done in standard Wykeham Farrance oedometers and in a modified version of the Anteus Self-Loading Back Pressure Consolidation Apparatus, Figure 1. The modification consisted of a system to measure lateral stresses in the sample during consolidation and swelling. This was done using a chamber machined into the confining ring and separated from the sample by a flexible teflon membrane. Fluids in the lateral pressure chamber were connected through a null indicator to a Bourdon gauge. Maintaining null permitted

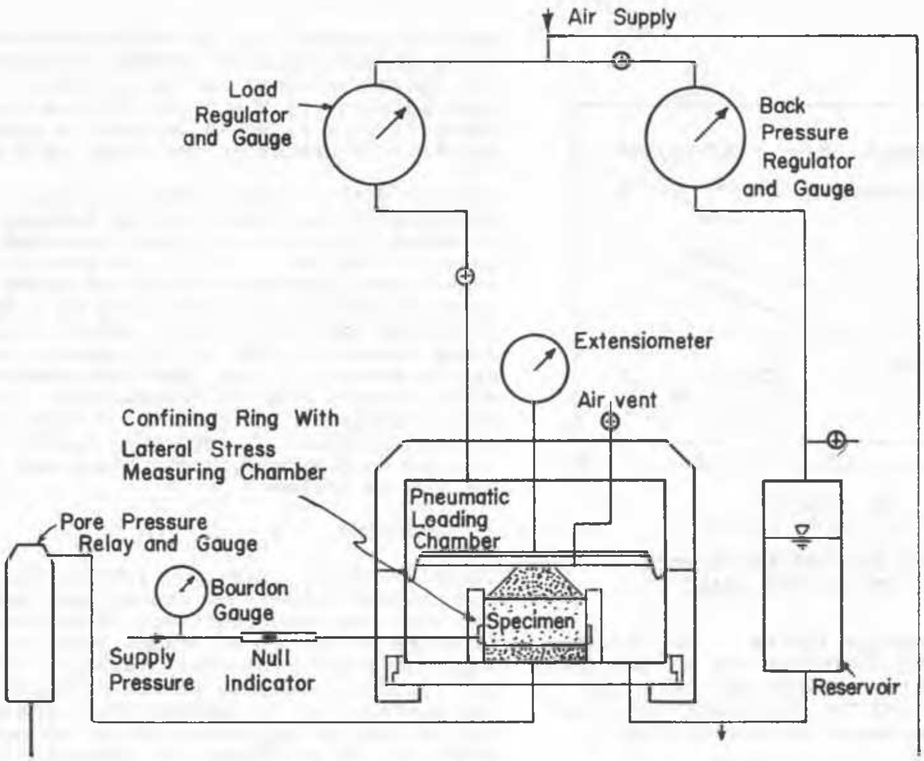

FIG. 1 CONSOLIDATION APPARATUS MODIFIED TO MEASURE LATERAL STRESS

the lateral total stress on the sample to be measured. Pore pressures were measured at the base of the sample, therefore allowing both vertical and horizontal effective stresses to be defined at equilibrium. As in direct shear testing, soil was placed in the oedometer at a water content approximately equal to the liquid limit.

As illustrated in Figure 1, a specimen in the Anteus apparatus is immersed in water under a back pressure. Vertical consolidation loads are applied through a flexible diaphram using air pressure. A porous stone on top of the specimen permits single drainage into the back pressure chamber, while pore pressures are measured through a porous stone at the sample base. Sample dimensions were 6.35 cm. in diameter by approximately 2.5 cm. thick. Vertical stresses up to 35 kg/cm^2 could be applied to specimens of this size.

DIRECT SHEAR TESTS

Drained shear testing of normally consolidated Bearpaw Shale defined effective stress failure parameters $\phi' = 21°$; $c' = 0$ (Figure 2). This is the form and range of results expected from other work on this material and similar tests results were obtained using thin and thick samples.

When overconsolidated soils were tested, different failure envelopes were defined (Figure 2). These results were obtained from tests on specimens which were first normally consolidated under 13.2 kg/cm^2 followed by unloading to the normal stress used in shear testing. Samples were allowed to swell under this reduced load until equilibrium was achieved and then were sheared. Typical stress-deformation response was an increase in shearing resistance which did not achieve a lower asymptote

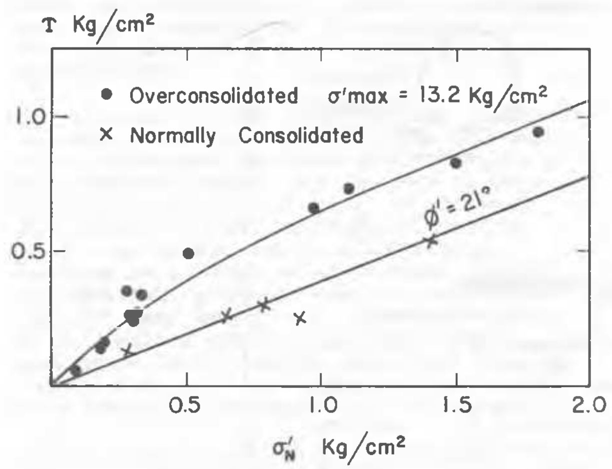

FIG. 2 LOCUS OF FAILURE FOR DIRECT SHEAR TESTS ON BEARPAW SHALE

within the deformation limits of the shear box. Accompanying the shear was an increase in specimen height or thickness; test results were corrected for the energy required for this dilation using the approach of Gibson (1953)*

The significance of these test results is that the failure envelope is distinctly curved and, when extrapolated from the results of tests at the lowest normal stress levels, indicates a cohesion intercept of $c' = 0$. This is quite contrary to the traditional behavior pattern for overconsolidated clays, Hvorslev (1960). The traditional cohesion intercept is defined, however, by extrapolation of test results at somewhat higher normal stresses (0.5 to 1.0 kg/cm^2 or higher) than were used in this testing program. It is only because of the tests at extremely low normal stresses that the curved envelope can be defined. In fact, extrapolation of the higher stress level tests (Figure 2) would indicate a significant cohesion intercept in the order of 0.4 kg/cm^2.

*One of the major advantages of testing heavily overconsolidated clay soils such as Bearpaw Shale in a direct shear apparatus is that the time for dissipation of excess pore water pressure can be reduced due to the short drainage distance for the sample. This is important both during shear and during the consolidation and swelling required for preparation of the samples. In this program testing rates and equilibrium conditions during swelling were determined using Terzaghi's theory and the coefficient of consolidation. In light of the conclusions reached in this paper it may be that some shear tests on the most heavily overconsolidated samples were started while the soil was still undergoing significant swelling, although this would have been after essential pore pressure equilibrium.

Drained direct shear tests were also done on overconsolidated specimens of Bearpaw Shale unloaded from maximum normal stresses other than 13.2 kg/cm^2. It will be useful in subsequent discussion of direct shear results to present all test data in a nondimensional form by dividing normal stress and shear stress at peak by the maximum consolidation stress, σ' max, for each sample. The water content changes accompanying shear, particularly in tests at the lowest stress levels, are also an important aspect of the testing more appropriately discussed in a section to follow.

K_o UNLOADING

Skempton (1961), Terzaghi (1961), Brooker and Ireland (1966) and others have pointed out that the upper sections of natural deposits of overconsolidated clay are in a state of passive pressure failure. Skempton has reported measured values of K_o equal to the coefficient of passive earth pressure, K_p, as much as 12 meters below the ground surface. In an attempt to understand the details of this passive pressure failure phenomenon, specimens of Bearpaw Shale were tested in the Anteus Consolidometer. Vertical stresses as high as 8.1 kg/cm^2 were applied incrementally to consolidate samples which were subsequently unloaded to stresses as low as 0.1 kg/cm^2. During this procedure horizontal stresses and excess pore water pressures were monitored permitting evaluation of K_o and the loading - unloading stress path.

Two typical stress paths are presented in Figure 3. The loading portions of these stress paths, when the sample is normally consolidated, follow a straight line with constant $K_o = 0.65$. This is consistent with the theoretical prediction of Jaky:

$$K_o = 1 - \sin \phi' = 0.64.$$

As the samples were unloaded the rate of decrease of σ_H' was less than for σ_V' and consequently K_o increased, at an OCR of approximately 3.5 $K_o = 1$, and with subsequent unloading $\sigma_H' > \sigma_V'$ and $K_o > 1$. As unloading continued the stress paths approached the passive pressure failure envelope, here illustrated as a range of values because of the different stress histories for the two samples.

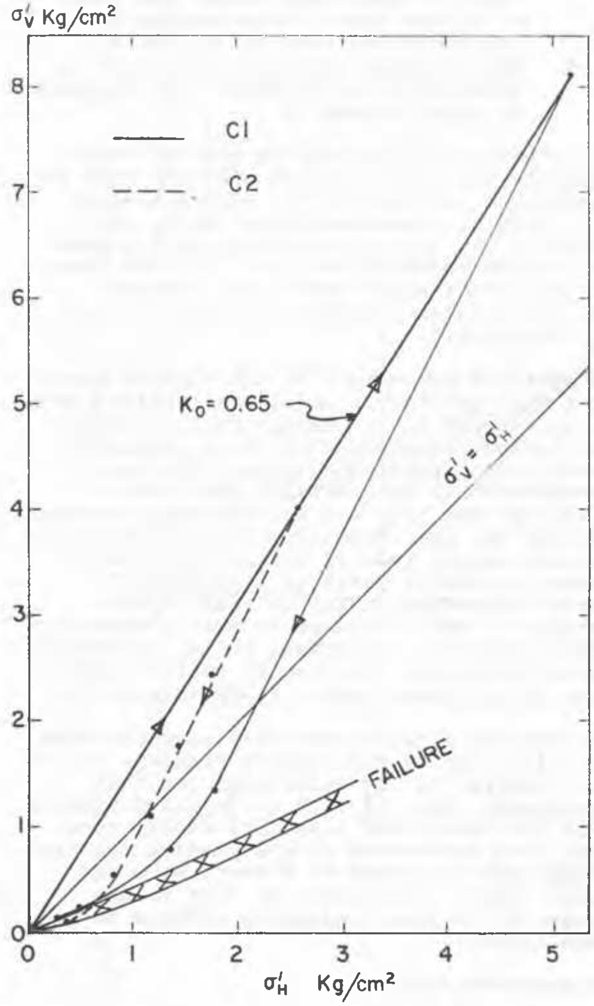

FIG. 3 K_O STRESS PATHS FOR TWO SAMPLES OF BEARPAW SHALE

This passive pressure failure being of particular interest, the data corresponding to the lowest unloading stresses in the K_o tests have been replotted to a large scale, Figure 4. The maximum shear stress $(\sigma_1 - \sigma_3/2)$, and $(\sigma_1' + \sigma_3')/2$ have been normalized in this plot using the maximum consolidation effective stress, σ' max. Also indicated in this figure is the failure envelope defined for Bearpaw Shale including the data presented in Figure 2 and others. At this scale it is clear that both samples C1 and C2 as they were unloaded, came into the locus of failure defined by the direct shear tests. Subsequently the stress paths moved down, more or less along the envelope, in a progressive failure.

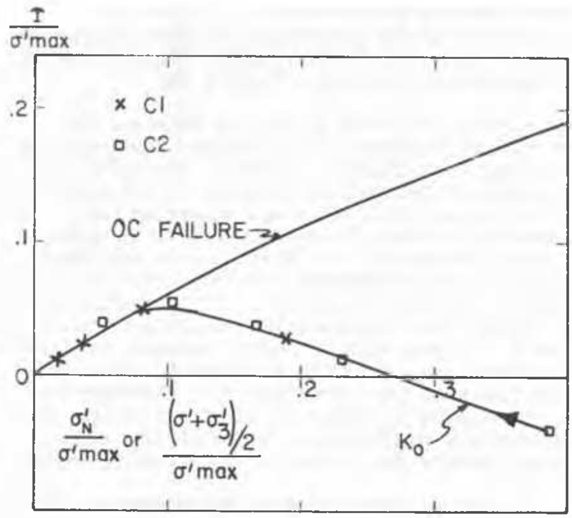

FIG. 4 K_O STRESS PATHS DURING SWELLING AT

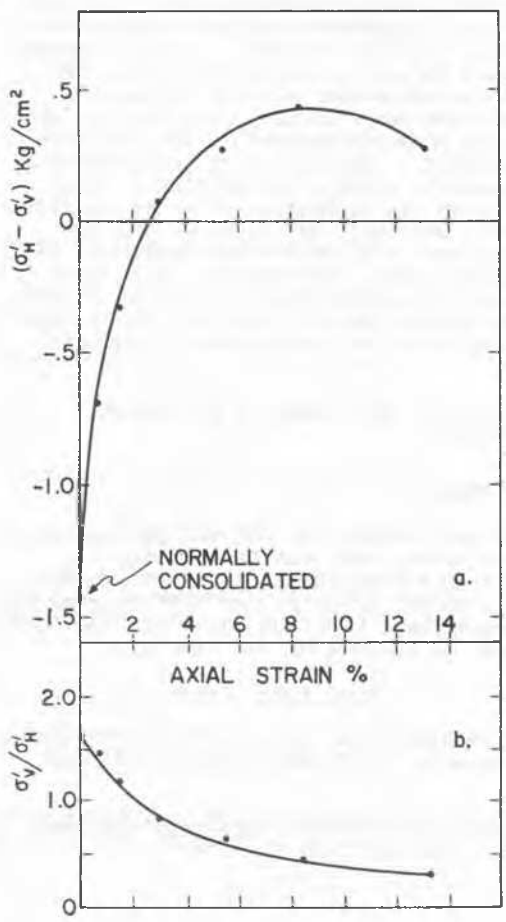

FIG. 5a. STRESS - STRAIN CURVE DURING K_O UNLOADING OF SAMPLE C2

b. STRESS RATIO DURING K_O UNLOADING

This unloading has in fact been a drained extension test with lateral constraint. A meaningful way of presenting the test results is in a plot of principal stress difference versus axial strain, Figure 5a. The corresponding principal effective stress ratio, σ_V'/σ_H', is also useful in discussion of progressive failure, Figure 5b.

It is useful at this point to examine the swelling of Bearpaw Shale during the various unloading increments. Several characteristics might be used to measure this swelling including pore pressure dissipation, volumetric strain or water content change. For test series C2 the dissipation of pore pressures and volumetric strain are presented in Figure 6a and b. It is obvious from these data that while unloading increments 1 - 5 are similar with respect to both parameters, an entirely different behavior is represented by increment 6. Re-examination of Figure 4 provides an explanation for this obvious difference; between the equilibrium points for 5 and 6 the sample failed.

In test series C2 only one increment of loading obviously went past passive failure. The question of subsequent behavior was examined in test series C3 in which a specimen normally consolidated under $\sigma_V' = 31.2$ kg/cm^2 was unloaded to normal stresses as low as $\sigma_V' = 0.068$ kg/cm^2. As illustrated in Figure 7a and b, during the final two unloading increments in this series the specimen was experiencing progressive failure. Two characteristics of this failure are apparent. Firstly, that the magnitude of volumetric strain or swelling is much larger than for unloading prior to passive failure. Secondly, the passive failure swelling does not reach equilibrium in any reasonable time. For example, the final increment of unloading, $\sigma_V' = 0.068$ kg/cm^2, was maintained for 200 days with continued swelling giving no indication of equilibrium.*

DISCUSSION OF TEST RESULTS AND THEIR IMPLICATIONS

DIRECT SHEAR

It has been recognized for the past several years (Bishop, Webb and Lewin, 1965; Skempton and Hutchinson, 1969) that the actual failure envelope for heavily overconsolidated clays might be curved rather than straight as implied by the equation:

$$\tau = c' + \sigma_N' \tan \phi'$$

Having demonstrated that heavily overconsolidated samples of Bearpaw Shale do indeed define such a curved envelope several questions are obvious.

1. Why does the failure envelope curve?
2. What is significant about the state of stress where the curvature begins, and is this a function of the OCR?
3. What changes in the sample, such as water content, accompany the decrease in shear strength?

The results of this testing program would indicate that all of these problems must be considered in light of the stress history of a heavily overconsolidated soil, and recognition that K_o unloading will result in passive pressure failure. In fact, many of the direct shear tests may have been done on samples already at failure due to K_o unloading.*

It would be convenient if the precise point of sharp curvature in a failure envelope were to correspond to K_o passive failure occurring during preparation of the overconsolidated direct shear specimens. The best interpretation of available data would indicate that this was not the case, however. Neither was overconsolidation ratio uniquely related to the reduced strength, although tests on several soils at a variety of stress histories indicated that an OCR between 10 and 15 seemed to mark a change in shear behavior. Likewise, the K_o unloading stress paths met the passive failure envelope in this same range of OCR (Figure 4).

At best one might tentatively conclude that in this range of OCR passive pressure failure occurred during unloading; and that subsequent shear of such an unloaded sample, with its associated principal stress rotation, was influenced by the passive failure. Other data presented by Bishop, Webb and Lewin (1965), also indicate this is the range of OCR where unloading effects become significant. 348

PROGRESSIVE FAILURE

A wealth of discussion of the problem of progressive failure has been offered in recent years (Skempton, 1964; Bjerrum, 1967; Bishop, 1967; Skempton and Hutchinson, 1969) with the common theme, release of strain energy, given to explain the swelling and reduced strength. The results reported in this paper confirm this hypothesis but with the clear conclusion that the release of strain energy is associated with shear failure. This shear failure may be developed by a direct increase in shearing stress (as in the direct shear test) by unloading a soil under K_o constraint, or by a combination of

*It should be noted the drainage distance for this specimen was < 1 cm.

*There would have been a principal stress rotation during the tests however.

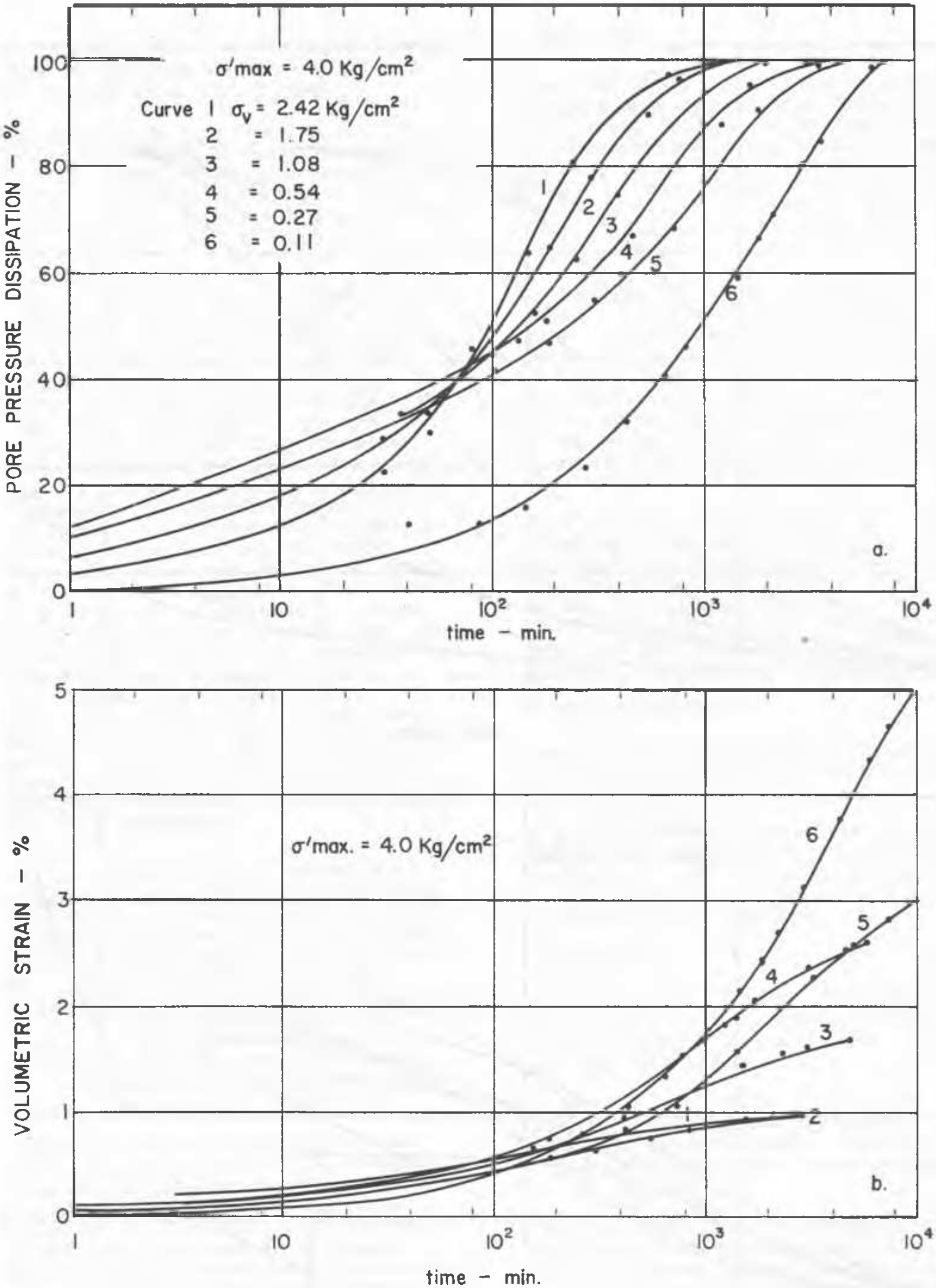

FIG. 6a. PORE PRESSURE DISSIPATION DURING K_o UNLOADING OF SAMPLE C2
b. VOLUMETRIC STRAIN DURING K_o UNLOADING OF SAMPLE C2

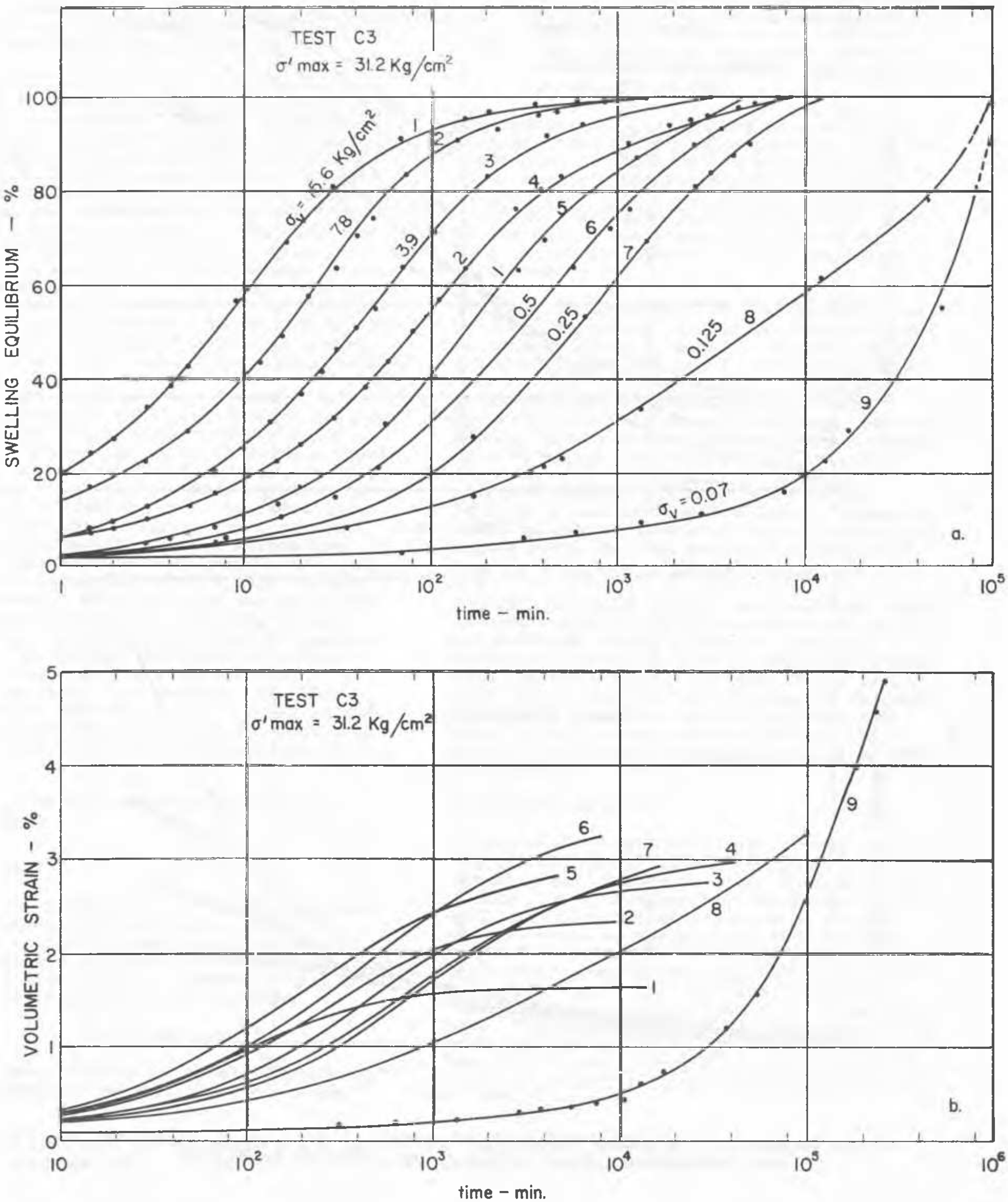

FIG. 7a. SWELLING OF SAMPLE C3 DURING K_o UNLOADING
7b. VOLUMETRIC STRAIN DURING K_o UNLOADING OF SAMPLE C3

factors. Indeed, the K_o unloading produces an increase in shear stress and a failure undistinguishable from direct shear.

It is also clear from the results of these tests that the swelling of heavily overconsolidated clay soils, both in magnitude and rate, is closely related to passive pressure failure. The data presented in Figure 6 and 7 are important in this respect.

The obvious conclusion to be drawn from these data is that both the rate of swelling and the magnitude of volumetric strain change significantly when the unloading soil achieves passive failure. A less obvious point of significance is that the time required for excess pore pressure dissipation agrees with the time required for swelling prior to failure, but not after failure. As indicated by any of the curves 1-4 in Figures 6a and b, volumetric strain and pore pressure dissipation appear to follow Terzaghi, or hydrodynamic, theory. Swelling equilibrium and pore pressure equilibrium occur at essentially the same time. On the other hand, during unloading increment 6 there is no agreement between swelling and pore pressure dissipation; and particularly after excess pore pressure has reached essential equilibrium (approx. 6000 min.) the swelling continues at a significant rate. (When condidered from this standpoint it would appear that unloading increment 5 was more like 6 than the pre-failure increments, indicating that the initial stages of passive failure may have begun with this increment.)

The overall conclusion to be drawn from these data is that regardless of the terms applied to explain excessive swelling of heavily overconsolidated clays (breakdown of diagenetic bonds, reorientation of soil structure, loss of cohesion, etc.) the significant factor in the process is passive failure. As this passive failure takes place (Figure 7) the sample swells, increases in water content and decreases in strength. This strength decrease develops to the extent that at $\sigma_N' = 0$, $c' = 0$.

Time obviously is a major factor if, as indicated by Figures 6 and 7, the swelling during passive failure continues and does not reach an asymptote. Carrying this observation to its logical limit, an overconsolidated soil unloaded to a stress state at the point of passive failure should swell and increase in water content until it softens to the water content of a normally consolidated soil at that normal stress. At this water content the soil would develop the fully softened, $c' = 0$, shearing resistance. This is essentially the same conclusion reached by Skempton (1970) on the basis of his examination of field data.

CONCLUSIONS

The peak shearing resistance offered by heavily overconsolidated clay soils is not adequately defined by extrapolation into low stress ranges ($\sigma_N' < 0.5$ kg/cm^2) of failure parameters defined by tests at higher stress levels. The effective stress failure envelope curves in the lowest stress range defining no coehsion intercept, $c' = 0$, at $\sigma_N' = 0$.

As heavily overconsolidated soils swell under K_o constraints σ_H' becomes larger than σ_V' ultimately resulting in passive pressure failure. This failure occurs at essentially the same failure locus defined by the direct shear test. Further unloading of soils which experienced passive failure leads to relatively large strains, swelling, accompanied by a progressive failure and loss of shearing resistance.

Test results presented in this paper clearly indicate that passive pressure failure in K_o tests separates moderate swelling, in unloading increments below failure, and large strain swelling during progressive failure. The times required for this post-failure swelling are very long and equilibrium limits to strain are difficult to define.

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