On shear strength of normally consolidated and saturated clay

Sur la résistance au cisaillement des argiles saturées normalement consolidées

M.TOPOLNICKI, Technical University of Gdańsk, Poland

SYNOPSIS: Element tests performed in a sophisticated biaxial apparatus indicated important mechanical properties of clay which are beyond the classical concept of the CSSM. Isochoric plane strain paths following different anisotropic consolidations lead to failure, but the stress paths do not terminate reaching the CSL. At failure the clay can have the same specific volume, but different values of the mean effective stress. Shear strength is not only a function of the void ratio at failure and of the path to failure, but also of the consolidation history. Continued isochoric straining causes softening behaviour till a state of constant stress.

1 INTRODUCTION

The purpose of this experimental investigation is to provide a background of information which may help to verify classical concepts concerning the undrained shear strength of normally consolidated clays, and which may be usefull for developing more reliable constitutive relations for soil. For the determination of the stress-strain behaviour element tests are realized with measurements of the effective stress and control of the strain rate.

In the following p' is the mean effective stress, s is the 2nd moment of the stress deviator, referred to as shear stress, and ε denotes natural strain. Compression stresses and contractive strains are negative.

2 EXPERIMENTAL TECHNIQUE

The material used for the tests was a powdered Kaolin, known as a "Karlsruhe" clay, with the following properties: clay fraction ($\leq 2\mu m$) 59%, liquid limit 48%, plastic limit 18%, specific gravity of particles 2.68, organic content 5.6%, lime content 0%. The soil was initially prepared as a slurry, with a moisture content of 81 to 82%, and subsequently pumped into the cuboidal rubber membrane installed between rigid platens of the biaxial apparatus. The possibility of starting the tests from a slurry is an important advantage of this apparatus. In a slurry state the clay behaves like a fluid, i.e. it is homogeneous and no direction is preferred. Consequently, all virgin consolidation paths begin from a state of practically zero effective stresss.

The biaxial apparatus, shown in Fig. 1, has been designed to apply large strains to a rectangular soil sample. The plane strain deformation is assured by two fixed horizontal plates, hold apart at a spacing of 50 mm. Thus the strain component ε_3 along the vertical axis *3* is zero. Four movable vertical platens are designed to change the sample dimensions along the two horizontal axes *1* and *2*. Side friction between the polished face platens and the rubber membrane is reduced with a silicone grease. Each pair of the opposed vertical platens is mechanically linked and is driven by one motor. Sample lengths may vary between 45 and 133 mm. Straining of a sample is controlled by a computer, which also records three normal forces and the pore water pressure. The pore pressure is measured at the vertical *fixed* axis of the specimen by means of a hypodermic needle. This needle has no hydraulic connection to the circular porous stones for drainage centered on the top and bottom of the sample. "Undrained" tests are carried out as drained tests in which the total volume of the specimen is held constant by a suitable movements of the platens.

All tests have been performed as strain controlled. To eliminate the influence of viscosity on the stress response the mean strain rate $\dot{\varepsilon}_m = \sqrt{\dot{\varepsilon}_1^2 + \dot{\varepsilon}_2^2 + \dot{\varepsilon}_3^2}$, set to 0.5% per hour, was maintained throughout the tests. In selecting this strain rate a compromise was made between a too law rate leading to creep deformations, and a too high rate causing excessive pore pressure in the sample. Back pressure of 200 kPa was used to saturate the clay. For further details concerning the apparatus and the testing procedure see Topolnicki (1987 b).

3 INTERPRETATION OF THE OBSERVED BEHAVIOUR

We consider five samples consolidated to the same mean consolidation stress p'_c along four proportional anisotropic strain paths. These paths represent four directions which cover one sector of 60° on the strain deviator plane. Direction I, defined by $\alpha = 0°$, where α is the angle between the strain rate vector and the positive deviatoric principal axis, is an isotropic consolidation at plane strain. Directions II and III are for $\alpha = 30°$ and $\alpha = 45°$, respectively. Direction IV is a one-dimensional consolidation, i.e. $\alpha = 60°$. After normal consolidation all samples were sheared under constant volume (isochoric condition), four to the active and one to the passive side. In active tests (No. 218, 221, 224, 308) the strain rate $\dot{\varepsilon}_1$ was contractant throughout the test, in the passive test No. 307 $\dot{\varepsilon}_1$ changed sign at the moment when isochoric path began. Because the samples were consolidated to $p'_c = -640$ kPa, they reached different void ratios at the end of consolidation, namely 0.656, 0.657, 0.648, 0.638 and 0.633, respectively. Consequently, a direct comparison of the undrained shear strengths with respect to different consolidation histories and paths to failure is not meanigful. To overcome this difficulty we utilize the well established material property that the stress responses for relevant programmes of straining may be normalized by the consolidation stresses. With appropriate equivalent stresses p'_e it is thus possible to replot the stress paths so that they represent the response of five samples having the *same* void ratio e at the beginning of isochoric straining. From the normal consolidation lines of the considered tests (see also Topolnicki, 1987) values of p'_e corresponding to a chosen and unique value of $e = 0.656$ are found. The coefficients $R = p'_c / p'_e$ are then used to evaluate the new values of the stress invariants $s_R = s/R$ and $p'_R = p'/R$.

The results of this procedure are shown in Fig. 2. The s_R vs. p'_R relationships are apparently different at the beginning of

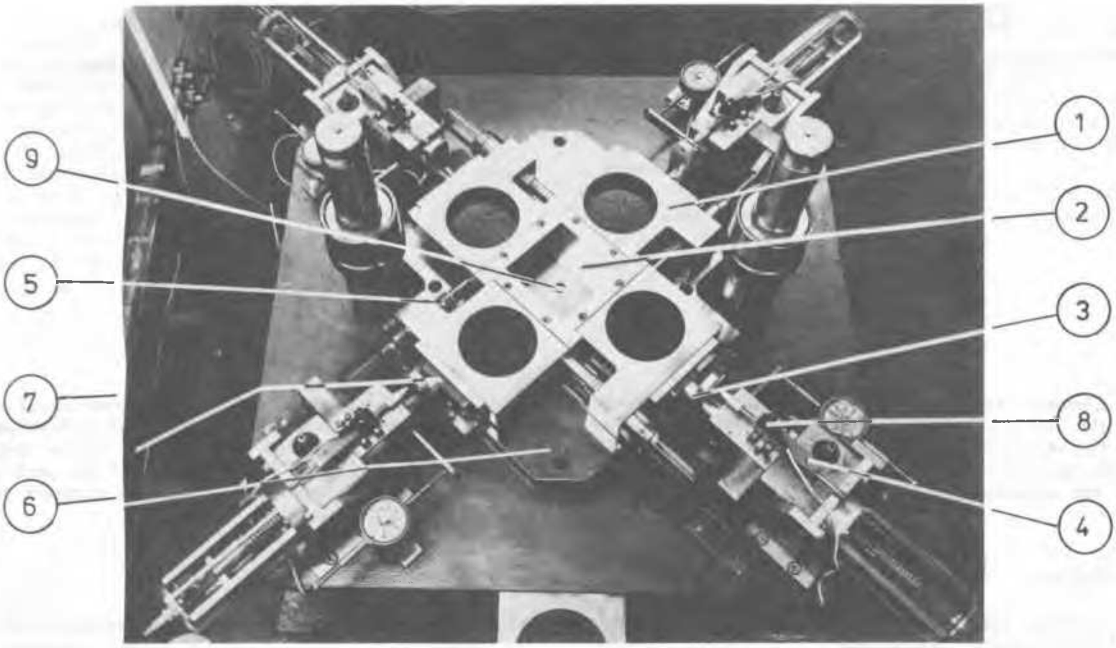

Figure 1. Biaxial apparatus with the top plate removed : 1 - movable platen, 2 - space for slurry to be pumped into rubber membrane, 3 - ram with load cell, 4 - gearing, 5 - translatory sliding guide, 6 - base plate, 7 - slide bearing, 8 - displacement transducer, 9 - hole for bottom sealing disc containing drainage stone and hypodermic needle

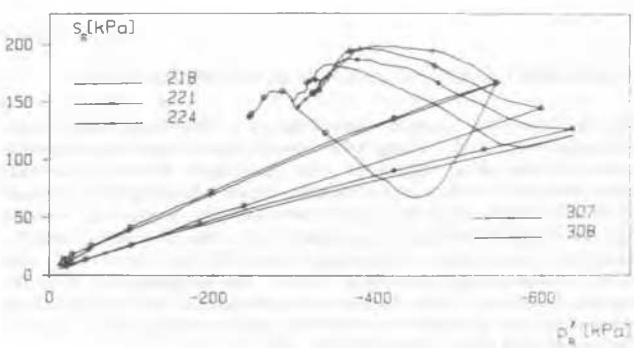

Figure 2. Shear stress versus mean effective stress relationships for five samples normalized to a unique void ratio

isochoric straining, as well as there are *different* shear strengths s_f, while we define the shear strength as the peak value of the shear stress and the corresponding state of the sample as the state of failure. In spite of the same type of the strain paths to failure (i.e. active tests) the initial parts of the curves are convex (test 308) or concave (tests 224, 221, 218). The concavity gradually increases from test 224 to 307, and is extreme in the latter case of a passive test. This characteristic behaviour can be explained concerning the changes of principal stress components in the initial phase of isochoric shear, shown in Fig. 3. In this figure, for each test, the state of the principal stress after completion of the consolidation path corresponds to the abscissa $\varepsilon_{sc} = 0$, where ε_{sc} denotes shear

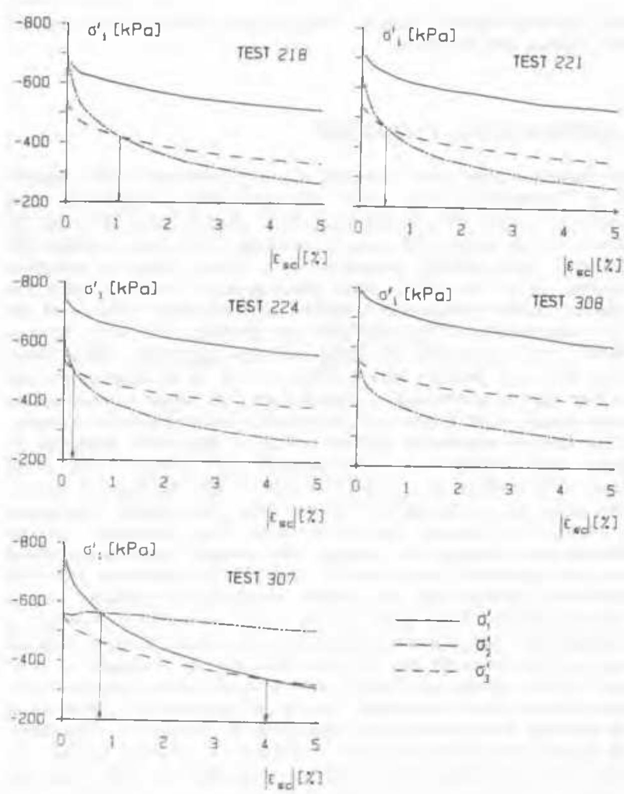

Figure 3. Change of principal stress components during the initial phase of plane strain isochoric paths

strain counted from the beginning of the isochoric path. As seen, in the case of test 308 the initial relations between the stress components $-\sigma_1' > -\sigma_2' = -\sigma_3'$ only change to $-\sigma_1' > -\sigma_2' > -\sigma_3'$ during subsequent straining, while in tests 224, 221 and 218 it comes to a *delayed* interchange of the stresses σ_2' and σ_3' acting in the plane perpendicular to the plane of strain. In the case of test 307 first occurs an interchange in the plane of strain, when σ_1' drops below σ_2', and later an interchange between σ_1' and σ_3'. Consequently, the initial shape of the $s-p'$ curves is directly related to the fact whether and when the interchange of the principal stress directions takes place. The larger is the shear strain to this interchange, the more remarkable is the initial concavity of the $s-p'$ curve.

The influence of the initial stress response is also manifested by different values of the shear strengths s_f. If the interchange of the principal stress directions occurs very early, as in test 224, the drop in s_f is very small as compared with test 308. With increasing deformation accompanying the interchange of principal stress directions the shear strength decreases gradually, as show tests 221, 218 and the test 307 in particular. In other words, clay samples consolidated along various paths may have different shear strengths even if the strain paths to failure are of the same type. Thus the *history of consolidation* has an influence on the undrained shear strength, *provided* the relations between the principal stresses after consolidation change during the shear phase. Note that this statement does not contradict the well established fact that the undrained shear strengths of one-dimensionally and isotropically consolidated samples sheared in compression are the same; this is so because there is *no* interchange of the principal stress directions in these two cases.

Two other important material properties can be formulated by analysing the data shown in Fig. 2.

The first one is connected with the observation that the states of maximum shear stress of five samples are shifted along the p' axis. Consequently, the samples at failure can have the *same* specific volume v_f and *different* values of the mean effective stress p_f'. Thus in the $\ln v$ vs. $\ln(-p')$ plot the pairs v_f, p_f' of related tests form a *family* of parallel lines which represent projections of the failure envelopes depicted in the $s-v-p'$ space. It turns out that these lines are approximately parallel to the normal consolidation lines (viz. Topolnicki, 1987 b), what is one of the key assumptions in the CSSM. It is pointed out that there is a unique projection of the failure envelopes in the $s-p'$ plane for all isochoric tests, but it is not so in the $v-p'$ plane. This is because the actual position of the failure envelope in the $s-v-p'$ space depends on the type of shear test and on the consolidation history of the sample.

The second material property is related to the post-failure behaviour. In Fig. 2 we observe that the samples have *not* reached a steady-state condition at failure, and that the stress paths bend in the vicinity of the Critical State Line (failure envelope) and subsequently move, with a reasonable accuracy, along the CSL with decreasing s and p'. The post-failure decrease of stress, known as softening, was primarily ascribed to overconsolidated samples. In the case of samples normally consolidated from a slurry state softening is commonly not expected. However, the presented data and also recent tests of Airey and Wood (1987) indicate that softening behaviour of NC clay appears in tests where distinct localized zones of deformation occur. Consequently, softening may be attributed to alignment of the platey kaolin particles in the rupture planes. Note that localited zones of deformation are not generally observed in undrained triaxial compression and extension tests when the samples fail due to barrelling or necking.

It is now of interest to demonstrate that the post-failure decrease of stress is *limited*. This conclusion can be drawn from the analysis of Fig. 4, which shows the development of stress invariants s and p' in the course of isochoric straining. It is evident that both invariants reach a state, called an *ultimate state*, where large shear strains can occur with no further change in stress. Furthermore, quite different stress-strain curves for active and passive tests alow to presume that the internal changes of the clay structure in shear bands are different, at least until very large deformations are applied. This is directly connected with the phenomenon that in passive tests an interchange of the principal stress axes in the plane of strain takes place, and consequently much stronger changes of the initial clay structure occur than in active tests.

The reduction of the shear stress from a peak value s_f to an ultimate value s_u is important from a practical point of view (e.g. landslides). The data indicate that the drop of shear stress amounts to about 15% of the peak value at failure, independently on the consolidation history and on the type of test.

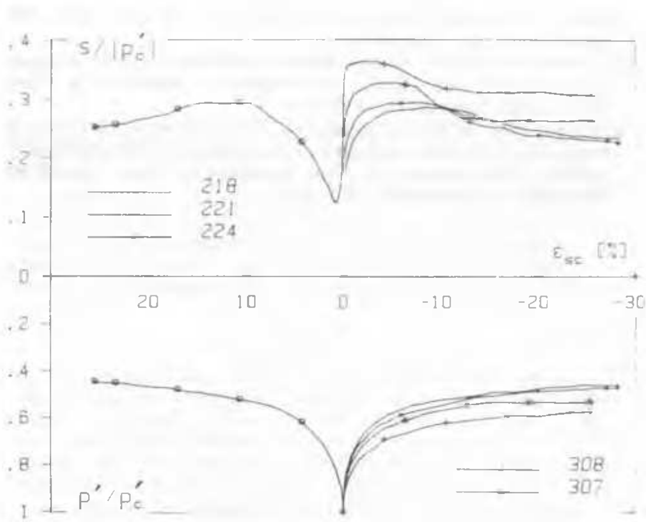

Figure 4. Development of shear stress and mean stress during isochoric shear

4 CONCLUDING REMARKS

The presented interpretation of the experimental data has been based on average principal stresses and strains measured on the sample boundaries. It is important to point out a principal difficulty that average stresses and strains give not a truly representative description of the state within the sample when failure is approached. As soon as localized zones of deformation appear in the sample (compare also Hambly 1969, Airey and Wood 1987) the stress and strain fields become non-uniform. This holds even more for the states following failure when most of the deformation is concentrated in thin rupture zones. These zones were clearly detectable after completion of the considered tests on the interfaces of the sliced samples (viz. Topolnicki, 1987 b). Consequently, also the constant volume condition was maintained only in an average sense after the ruptures had developed.

Since non-uniform conditions of stress and strain appeared in the tests we are unable to interprete material properties at failure and post-failure states with the same degree of accuracy as for the states preceeding failure. Nevertheless, the behaviour at and after failure is of primary importance and cannot be left aside. The observed behaviour determined from the biaxial apparatus after rupture should be analysed as a boundary value problem. The presented data can be then used to draw conclusions with relation to the predictions of constitutive relations for soil.

ACKNOWLEDGEMENTS

This paper is a partial result of a research project conducted in the Institute of Soil and Rock Mechanics of the Karlsruhe University under supervision of Professor Gerd Gudehus. Financial support from the German Research Association is gratefully recognized.

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