Shear strength and deformation properties of clays in direct shear tests at high strain rates

Propriétés de résistance au cisaillement et de déformation des argiles en essais de cisaillement direct aux grandes vitesses de déformation

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SYNOPSIS

In the conventional slip surface plastic failure approach in slope stability analysis only one single strength parameter - the shear strength of the soil - is used. Time enters into the analysis only in so far as we discern between drained and undrained conditions.

In references |3|,|4| Bernander and Olofsson have shown quantitatively how a minor disturbance acting in a slope of strain-softening material may trigger a progressive failure which eventually propagates into vast areas of - may be - even horizontal ground. The object of the present investigations has been to study in particular the effect of time on the stress-deformation properties of some soft Swedish clays. The data serve as input in the progressive failure analysis. Cf |3|, |4| and |10|.

Shear tests described in this paper show that the limit state - large strain strength in direct shear may be highly dependent on strain rate and on the over-consolidation ratio (OCR). Strain rate and OCR also affects the peak shear strength (critical state) as is generally known from field vane tests. Thus according to the present investigation not only is the peak strength dependent on strain rate but so is also the large strain (limit state) strength. Lower strain rates tend to give lower peak-strengths and higher limit shear strengths whereas higher strain rates yield higher peak-strengths and lower limit shear strengths. It should be observed, however, that the tests have been carried out without using the usual rubber membrane around the probe as this is believed to reflect the real conditions more accurately.

INTRODUCTION

Many of the extensive landslides which have occurred in Sweden cannot be readily understood by means of conventional analysis based on the concept of plastic failure. In many cases the slides have been triggered by man-made operations of unknown intensity (piling, fills etc), but even in hindsight it has often turned out to be very unrewarding to attempt to explain the slides - and in particular to correlate the actual extent of a slide - by back-analysis in accordance with current procedures.

The Tuve slide (1978) in the Western part of Sweden has been analyzed by conventional approach. Safety factors with respect to slides of some length were evaluated from = 1,1 |18| to 2,2 à 2,6 |22| depending on the investigators' approach to the problem of choosing a shear strength value for back analysis.

Bernander and Olofsson |4| have shown that the Tuve slide may be phenomenologically understood with a progressive failure model according to |3|, an analytical approach which has been further developed in a contribution to the Toronto symposium on landslides 1984 |10|. It was also demonstrated in |4| that - applying the current plastic failure analysis - it is possible to prove with a considerable margin of safety ($f \approx 3,5$) that what actually happened in the slide could not have taken place. This fact disqualifies analysis based on plastic failure in long slopes in sensitive clays especially for predicting the extent and degree of catastrophy of a potential slide.

Long term stability and the effect of slowly applied loads in a slope may be checked by drained analysis in which case the conventional approach with plastic failure ought to be applicable. However, when failure occurs in a natural slope the triggering agent is usually of temporary nature (increased pore water pressure due to rainfall, fills, piling etc.). But even in the case when a local failure is of a drained character the subsequent rupture will take place under growing strain rates and increasingly undrained conditions.

A landslide such as that at Tuve starts as a gradual accelleration of an ongoing creep deformation. The final phase takes place within one or a few minutes, which gives the timing of the events that govern the development, propagation and final extent and morphology of a slide. It is therefore the response of the soil at the actual strain rates that is relevant and only laboratory tests and analysis related to the actual timing of a slide can tell us something about the consequences of e.g. a local failure in major slope.
Thus when analyzing failure mechanisms, failure propagation and the ultimate extent (the degree of disaster) of potential slides it is necessary to use undrained parameters which are relevant to the strain rates at stake.

The conclusion of the aforesaid is obviously that stress-strain properties of soils are of great interest. Strain softening is a well known phenomenon in geotechnical literature. E.g. Torstensson |24| and Aas |1| have shown how peak strength and the large strain strength (Aas) are strain rate dependent in field vane tests. Lefebvre and La Rochelle |19| have demonstrated the dependence of both critical state and limit state strength of OCR in Champlain clays. In order to study the effect of strain rate on the stress-strain proper-

ties of soft Swedish clays various kinds of shear box tests have been conducted at the SKANSKA geotechnical laboratory (Gothenburg).
Tests with a Casagrande shear box on some soft clays by Bernander, S and Svensk, I |6| indicated that both the critical and limit state parameters were markedly dependent on strain rate and OCR.
Further tests in the course of a graduation work by Bernander, J and Isacsson, K |11| - where the soil specimen were contained by sets of rings without an interconnecting membrane - showed similar tendencies. Continued testing at SKANSKA has been carried out in order to accumulate background experience of how to perform new types of routine tests designed to reveal the behaviour of soft clays at high strain rates relevant to the timing of possible actual events in the course of large planar landslides.

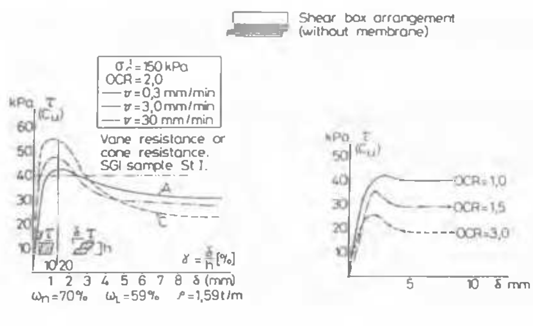

Fig. 1 Typical test results from shear tests of some Swedish soft clays. Stress-deformation properties depend on strain rate and OCR.

STRESS-STRAIN RELATIONSHIP IN DIRECT SHEAR

The failure mechanism in a potential failure zone of a planar slide is taken to be as follows:

Stage 1

Initially the material in the potential failure zone is sheared more or less uniformly as shown in fig. 2b.

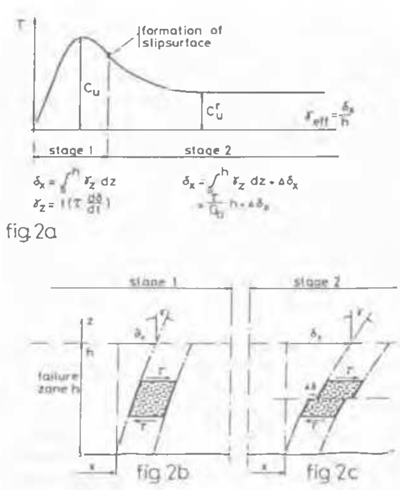

Fig. 2 Stress-deformation relationship and mode of failure.

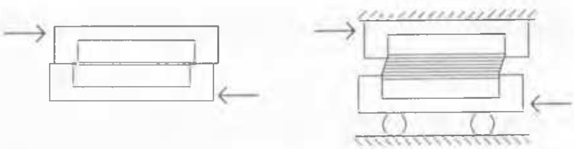

Fig. 3 Shear box tests without and with rings.

A plastic membrane, however flexible it may be, will namely resist excessive sliding between adjacent rings (fig. 4) and in a general way prevent or postpone the formation of slipsurfaces. Thus spiral-reinforced membranes of the type commonly used in geotechnical laboratories will tend to distort or modify the performance of the clay sample in shear, particularly at large strains. This will tend to affect the test results with regard to the limit state shear strength. Therefore - in this investigation - the normally used reinforced membranes were replaced by tightly fitting rings according to fig. 5 permitting the probe to deform uniformly as in 5a (Stage 1) or by forming a slip surface as in fig. 5b (Stage 2).

The total displacement in the failure zone is then

$$\delta_x = \int_0^h \gamma_z \cdot dz = \gamma_m \cdot h \quad \text{...........(1)}$$

where γ_z is a function of the shear stress and the strain rate. (Other notations are defined in fig. 1 and 2).

Stage 2

At some point a slip surface is formed - an event usually accompanied by a gradual loss of shear strength. The displacement now has two components - one due to deviatoric strain in the material and one due to the slip in the slip surface itself ($\Delta\delta_x$). (Fig. 2c).

$$\delta_x = \int_0^h \gamma_z \cdot dz + \Delta\delta_x \quad \text{...............(2)}$$

The residual shear resistance in the slip surface, however, will tend to be highly dependent on strain-rate owing to the interaction between pore pressure build-up in the slip surface and the rate of dissipation of these excess pore water presssures. It should be noted that the magnitude of these high local pore water pressures cannot be studied in conventional apparatus for undrained testing as with such gear only the mean pore pressure rise in the sample can be registered.

DESCRIPTION OF TESTS

Some of the tests carried out in this investigation are shear box tests (fig. 3a). This was found appropriate as our interest in this case was mainly focused on the limit state shear strength properties in the slip surface proper at high strain rates.

In most of the tests, however, the sample was contained in rings or in an extremely flexible spiral (fig. 3b). It is important to note that the shear tests have been carried out without the usual membrane of plastic or rubber containing the probe thus permitting the failure mechanism described above Stage 1 with uniform shear and - Stage 2 with a formed slip surface - to take place.

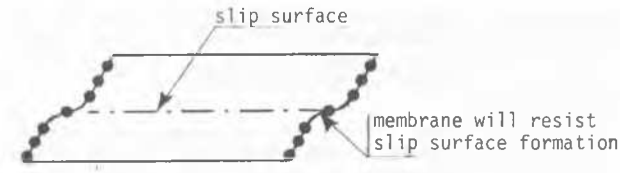

Fig. 4 Effect of membrane on slip surface formation.

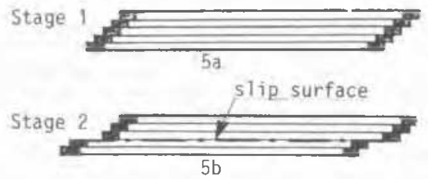

Fig. 5 Deformation of sample contained in tightly fitting rings without membrane.

The samples were reconsolidated to σ'_c in the shear-box and - after deloading to the required OCR - allowed to swell before testing.
The clays have been tested for various values of OCR and at different strain rates. Different types of clay have been tested, the properties of which are seen in table 1.

TABLE 1

Type:	C_u^{cone} kN/m^2	S_t	W%	W_L%	$\rho \cdot g$ kN/m^3	OCR 1 2 4
Lilla Edet	30	-	75	60	16,0	◺ ⊢ ◖
Linnarhult	40-50	20-40	45-65	45-65	16,5-18,9	□ + ◗
Rollsbo	16	30-80	100	70	15,5	◇ × ●

TEST RESULTS

The tests show that the limit state shear strength may be highly dependent on strain rate and on the overconsolidation ratio (OCR).
The peak shear strength tends to increase with strain rate ($d\gamma/dt$) and decrease with augmenting OCR values (i.e. decreasing effective stress). Fig. 7. (C.f.Ladd & Foot |17| and Lefebvre & La Rochelle |19|). In contrast the large strain (or limit state) shear strength tends to decrease with both growing strain rates ($d\delta/dt$) and increasing OCR.
This general trend is very pronounced and appears to be a more crucial parameter than the inherent clay characteristics themselves - including sensitivity, fig. 6.
The brittleness ratio ($B = C_u^r/C_u$) is here defined as the ratio between the limit state shear strength and the critical state shear strength such as this may be evaluated from standard tests (field vane, cone, unconfined compression etc.). In the present investigation C_u has been determined by cone tests on reconsolidated samples, the results of which compared well with calculated values by Hansbo´s formula (C_u=0,45 W_L σ'_c). (In principle - for Swedish clays - these values should also agree with the results from standard field vane tests.)

Very low limit state shear strengths have been found for soft clays at some of the tested strain rates, which - although they be considerably higher than those commonly used in standard direct shear tests - are still very far from the strain rates actually occurring during the evolution and in the final phase of a landslide. (Fig. 6).

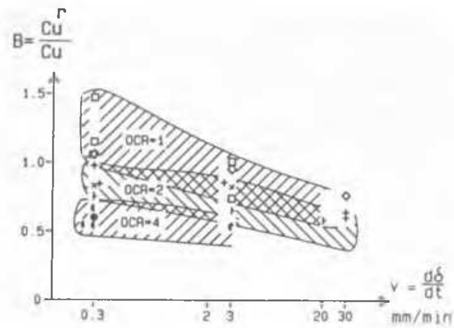

Fig. 6 Brittleness ratio C_u^r/C_u as a function of strain rate. The value of C_u^u is related to standard tests for OCR 1.

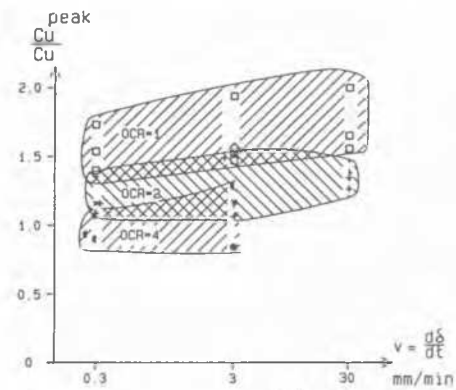

Fig. 7 Relationship between C_u^{peak} and C_u as a function of strain rate. The value of C_u^u is related to standard tests for OCR = 1.

CONCLUSIONS

The stress-deformation characteristics derived from the tests described in this article indicate that the brittleness ratio C_u^r/C_u may attain values as low as 0,3 for soft clays at strain rates, which are relevant to landslides. Brittleness ratios of this order readily explain the large planar landslides in Sweden if progressive failure analysis is applied as is e.g. shown for the Tuve slide in |4|.

A model for progressive failure analysis was presented at ICSMFE 81 by Bernander & Olofsson |3|. A further development of this approach has been published by Bernander and Gustaas, Toronto 1984 |10|. By these methods the limit criteria for stability of slopes can be evaluated for any non-linear stress relationship that an investigator of a potential (or a finished slide) may regard as appropriate. In general low strain rate curves will be applicable for in situ conditions or long term loading, whereas high strain rate relationships will be relevant for rapid load application (e.g. pore water pressure generation due to piling) and rapid creep as well as for prediction of failure propagation, failure mechanisms and failure events. It is considered to be of utmost importance for the geotechnician not only to be able to predict a local failure but also to be able to forecast the potential and likely consequences of such a local failure. This can only be accomplished by means of failure analyses based on stress-deformation data which are relevant to the actual strain rates and OCR - conditions valid for the anticipated slide. Thus in strain rate softening soils slope stability analysis will in principle be an iterative trial and error process in which the dynamics and timing of the slide should be involved. It seems, however, for the time being sufficient to estimate the timing of slide movement by experience. The proposed procedures in |3|,|10| will all the same constitute a major improvement to the understanding of landslide mechanisms in sensitive clays and to the ability of forecasting the scope of disaster of a potential landslide event.

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