Undrained Behaviour of Soft Clays in Triaxial Tests

Comportement Non Drainé des Argilles Molles dans l'Essais Triaxial

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SYNOPSIS A test program has been carried out on remoulded samples of soft clay consolidated with different stress histories. The samples have been tested in undrained triaxial compression with lubricated platens. The effective stress paths show a bi-linear shape with a break point which does not appear in conventional tests with rough bases. Pore pressure coefficients are related to the previous stress history. Undrained shear strength is also analyzed, checking the validity of a previously presented failure condition.

INTRODUCTION

A comprehensive research is being carried out at the Universidad de Santander on the behaviour of soft clays. Some aspects have been already published regarding theoretical analyses of undrained deformations and consolidation processes. In parallel with these theoretical analyses an experimental study is being performed, whose first results have been previously reported (Sánchez et al., 1979). In the present paper, additional results are exposed, mainly regarding undrained shear strength and pore pressure response.

TEST PROGRAM

It has long been recognized the great influence of previous stress history on the undrained behaviour of soft clays. In the simplest case of a horizontally sedimented deposit, the stress history at any depth can be described by means of the present vertical effective consolidation pressure, p_o', the at-rest earth pressure coefficient, K_o, and the overconsolidation ratio, OCR, due to mechanical overconsolidation and/or aging. In soils exhibiting normalized behaviour the consolidation pressure, p_o', can be used as a scaling factor for parameters having dimensions of stress remaining only two non-dimensional parmeters for the definition of stress history.

In the test program presented herein, a number of samples has been subjected to different stress histories, defined by different values of OCR and K_c (in the following the subscript "c" in the lateral pressure coefficient will mean that consolidation has not taken place under conditions of no lateral strain). The samples have been then subjected to undrained triaxial compression at a strain rate of 0.03 % per minute.

The choice of the triaxial test has been made from the consideration that if a soil element has been unevenly consolidated ($K_c \neq 1$) it will exhibit a cross-anisotropic behaviour. So, if further stresses are applied, the principal directions of stresses and strains will coincide only if they coincide also with the orthotropy axes. If it is not the case, normal stresses will produce shear distortions and vice versa. This fact invalidates the tests in which rotation of principal stresses occurs, unless special care is taken to measure these "parasit" strains (and, of course, provided that they are not inhibited). Even with these requirements, these tests need a complex interpretation if an accurate determination of soil behaviour is desired. Therefore, unconfined compression tests (Bhaskaran, 1975) or direct simple shear tests (Soydemir, 1977) on inclined specimens must be avoided (Saada and Bianchini, 1977; Kousoftas and Fischer, 1977; Sánchez, 1980).

Details of the test procedure followed in this work have been given elsewhere (Sánchez et al., 1979), thus only a brief outline will be presented here. In order to get a whole knowledge of the stress history from the beginning, the samples are consolidated from a slurry with a water content twice the liquid limit. The tests are performed with lubricated platens; this technique together with the use of a sand drain along the specimen axis, ensure that the fields of stresses, strains and pore pressures are uniform within the sample.

In table I the different stress histories given to the specimens are shown. In anisotropically overconsolidated samples (SCA-1 to SCA-9), the specimen is first consolidated to an all around pressure and then unloaded to the desired values of OCR and K_c. In normally consolidated samples (NCI and NCA tests) the value of K_c is mantained approximately constant during all the consolidation process.

For this experimental work, a Keuper clay from a site near Santander has been selected. This clay consists in illite (predominant), kaolinite and chlorite. It has a liquid limit of 55 % and a plasticity index of 29 %, with 49 % in weight of particles finer than 0.002 mm.

The sand for the drain is a very uniform silica sand comprised by the sieves No. 40 and 200, with a uniformity coefficient of 1.40.

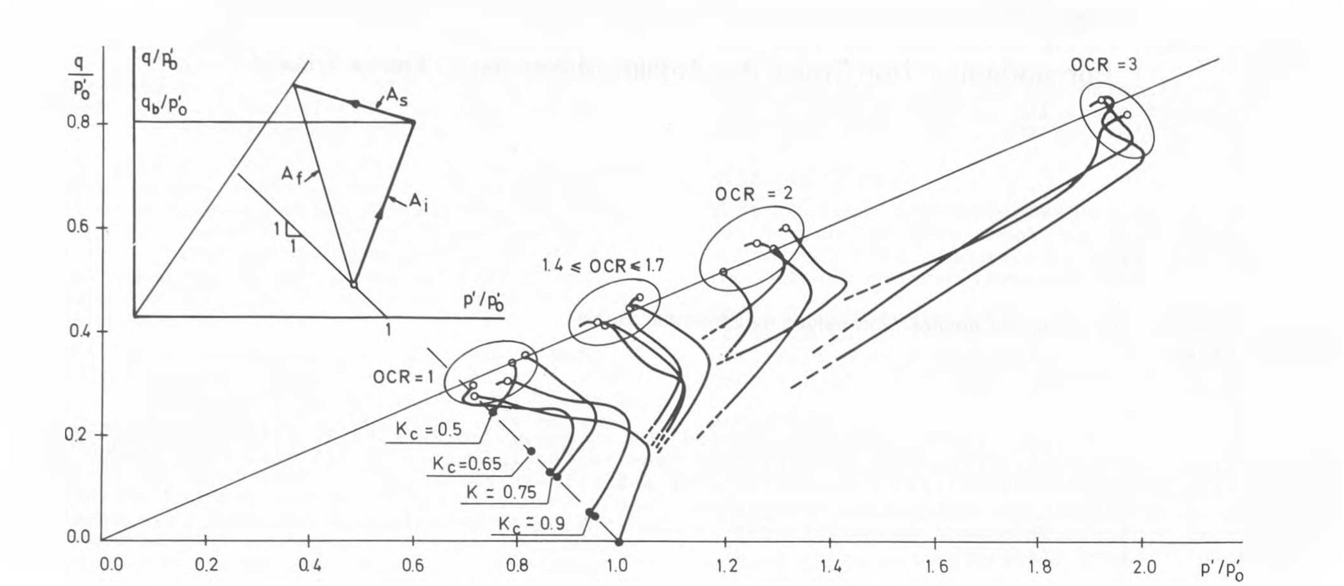

Fig. 1 Observed effective stress paths

TABLE I

Test	p_o' (kPa)	OCR	K_c	q_{cr}/p_o'	ε_{cr} (%)
NCI-5	150	1.0	1.00	0.230	1.3
NCA-1	195	1.0	0.90	0.270	1.4
NCA-2	200	1.0	0.75	0.275	1.1
NCA-3	150	1.0	0.50	0.340	0.8
SCI-1	175	1.4	1.00	0.330	1.5
SCI-2	175	1.7	1.00	0.370	1.9
SCI-3	175	2.0	1.00	0.425	1.1
SCI-4	100	3.0	1.00	0.605	1.2
SCA-1	167	1.5	0.90	0.365	1.3
SCA-2	150	1.5	0.88	0.380	1.8
SCA-3	133	1.5	0.75	0.390	1.8
SCA-4	167	2.0	0.90	0.435	2.1
SCA-5	200	2.0	0.75	0.465	1.6
SCA-6	155	2.0	0.65	0.500	1.1
SCA-7	111	3.0	0.90	0.630	1.2
SCA-8	133	3.0	0.75	0.575	1.7
SCA-9	167	3.0	0.65	0.635	2.3

ANALYSIS OF RESULTS

Undrained effective stress paths

In all the tests carried out, it has been observed that the effective stress paths (ESP) have a bi-linear shape, with a breaking point(Fig. 1). This shape of the ESP has been detected in all the tests, in spite of the variation of the values of OCR and K_c shown in Table I.

It is worth noting that this behaviour is not observed with the standard triaxial test equipment. In Fig. 2 two ESP are compared, corresponding to two identical normally consolidated, isotropic samples (OCR=1, K_c=1), tested with the improvements described above (specimen NCI-5) and in the conventional way (specimen NCI-00). As can be seen, the ESP in the standard test does not show any clearly defined breaking point. This diffe-

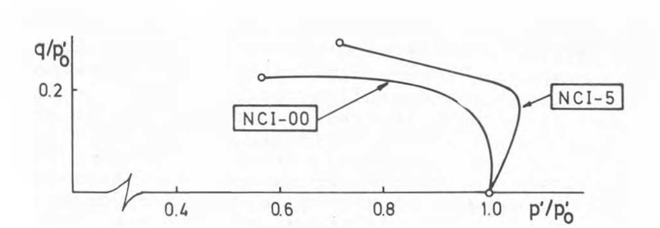

Fig. 2 Influence of lubricated platens on ESP

rence can be attributed to the lack of uniformity of the stress and strain fields within the sample in the standard test, due to base roughness.

For a complete definition of the bi-linear ESP of Fig. 1, four parameters are required: (i) initial pore pressure coefficient, A_i, (ii) normalized deviatoric stress at the break point, q_b/p_o', (iii) incremental pore pressure coefficient along the second linear portion, A_s, and (iv) pore pressure coefficient at failure, A_f. These four parameters are plotted in fig. 3 for the different stress histories. As can be observed, OCR influences all the parameters, whilst the influence of K_c is only clear at the first stages of the loading process, further deformations gradually masking its initial effect. It must be noted that for the analysis of pore pressure development under service loads, only the value of A_i will be required in most cases.

The break in the ESP clearly shows a change in the clay behaviour. As there is no abrupt change of the external forces, there must be a sudden change in the clay structure. This point merits further attention. It must be emphasized that it appears even in the virgin consolidated isotropic samples (NCI), so it can not be regarded as an effect of overconsolidation or prestressing. In fact, changes of this kind have already been observed or postulated (Bjerrum, 1973) at low strains in soft marine clays. In fig. 4 the strain

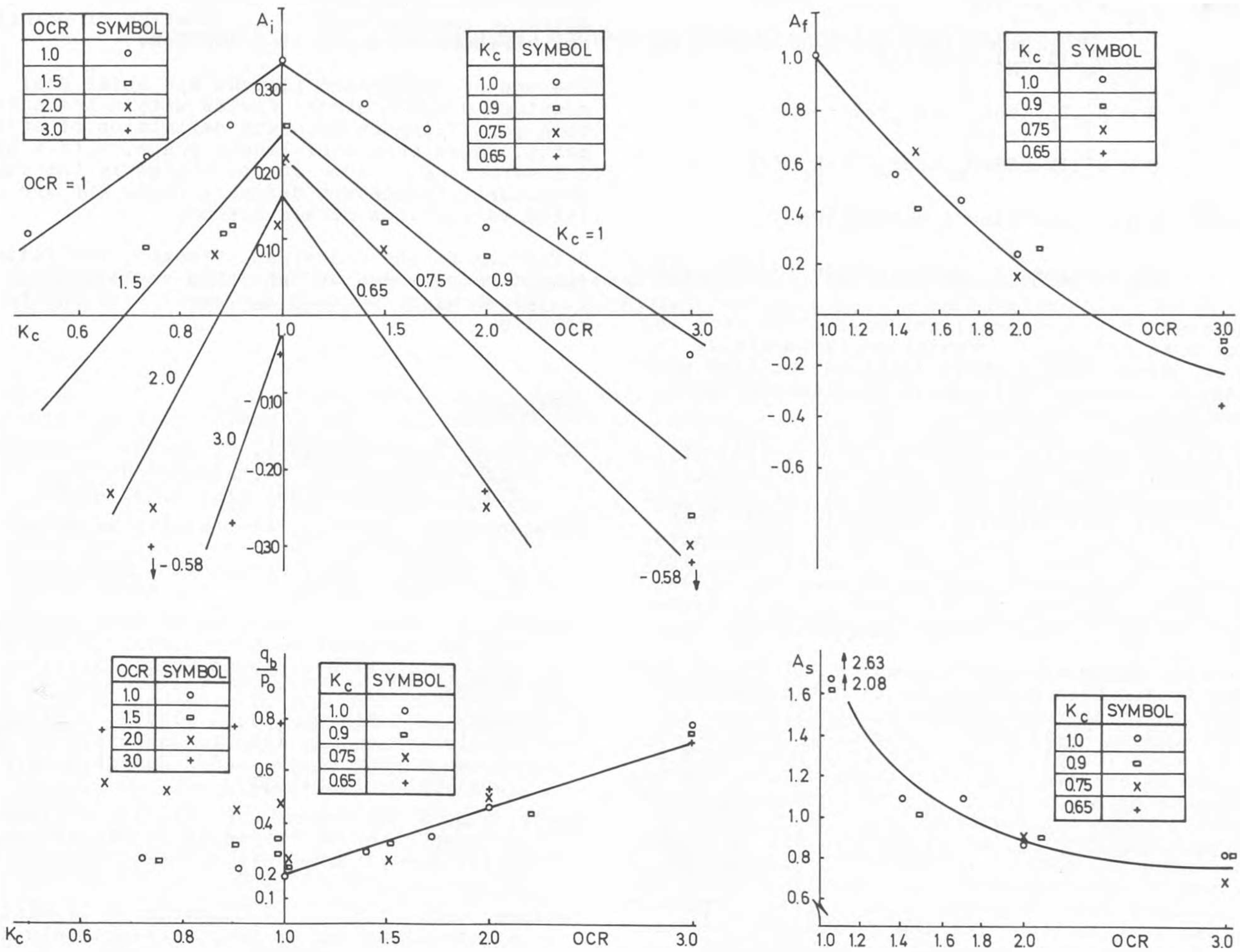

Fig. 3 Parameters of ESP vs. stress history

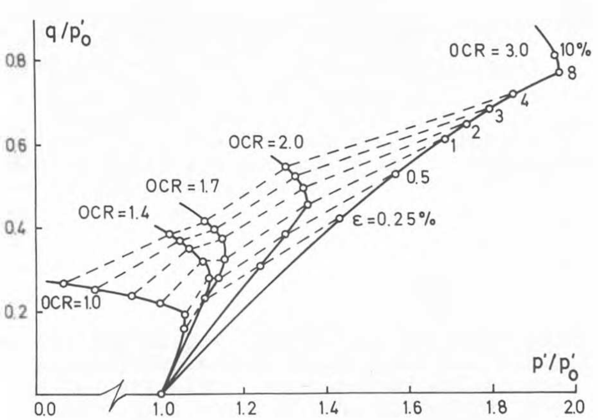

Fig. 4 Strains along ESP. Isotropic samples

levels along the ESP of isotropically consolidated samples are shown. As can be seen, the break occurs at a strain between 0.25 and 0.5 % for normally consolidated samples, from 1 to 2 % for OCR between 1 and 2 and from 8 to 10 % for OCR=3. It seems that in heavily overconsolidated samples the structure is stiff enough to sustain a large amount of strain before "breaking".

Undrained shear strength

One of the main objectives of the experimental work was the assessment of the validity of the failure condition proposed by Ballester and Sagaseta (1979), based on the ideas presented by Bjerrum (1973). The basic assumption of this model is the existence of a critical stress level at low strains at which a change in the clay structure occurs, corresponding to the full mobilization of the strength of the cohesive contacts of the clay particles. The failure condition is anisotropic due to the influence of stress history and is written as:

$$\sqrt{\{(q_{xy}+A)^2\ (|\tau_{xy}|+B)^2\}} = C \qquad (1)$$

where: $q_{xy} = (\sigma_x - \sigma_y)/2$

$A = p_o'.(1-D_M).(1-K_o)/2$

$B = p_o'.D_M.\tan\phi_e.(1-K_o)/2$

$C = p_o'.\{k.OCR+D_M.\tan\phi_e.(1+K_o)/2\}$

k, ϕ_e = Hvorslev's parameters

D_M = degree of mobilisation (Bjerrum,1973)

The need of experimental evidence comes from the fact that Bjerrum's formulation is partially based on direct simple shear tests on inclined samples (Soydemir, 1977), whose shortcomings have been discussed above.

In the tests presented in this paper, the critical stress level has been taken as the point of maximum curvature in the stress-strain curve. It appears at small strains, generally lower than 2 % (see Table I), and it coincides approximately with the break in ESP for normally consolidated or slightly overconsolidated samples, whilst for OCR=3 the break in ESP takes place at larger strains, as it was already discussed (Fig, 4).

The fitting procedure of equation (1) to the test results has been as follows: With the results of tests on anisotropic, normally consolidated samples (OCR=1, $K_c<1$) and isotropically overconsolidated ones (OCR$\geq$1, K_c=1), the soil parameters k, ϕ_e and D_M are deduced; with these values, the predictions of equation (1) for the remaining tests (OCR>1, $K_c<1$) are compared with the observed results. This fitting can be seen in Fig. 5, together with the deduced values of the parameters. As can be seen, agreement is reasonably good. In a previous paper (Sánchez et al., 1979) different values were obtained for the parameters, because at that time only few results were available, as it was then stated, and these former tests were run at a different strain rate.

CONCLUSIONS

A test procedure has been presented which permits the analysis of clay samples subjected to pre-selected stress histories. It can be used for anisotropic samples because no rotation of principal stresses occurs during the test.

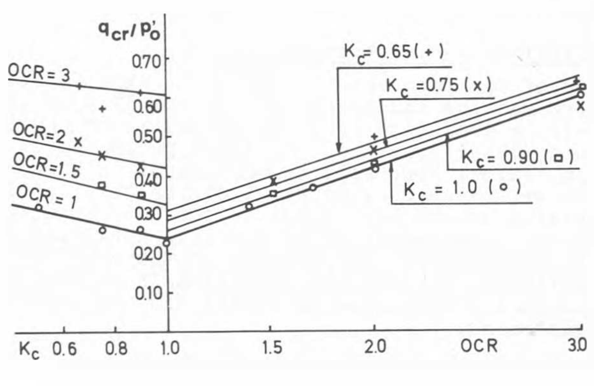

Fig. 5 Undrained strength vs. stress history

The use of lubricated platens and axial drai provides uniform stress fields within the sample, thus permitting an accurate definition of stress paths. These show a bi-linear shape, with a break corresponding to some abrupt change in the clay structure. Parameters defining these ESP are related to previous stress history.

Referring to the undrained strength, the failure condition proposed by Ballester and Sagaseta (1979) shows reasonable agreement with the test results.

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