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The Response of Shanghai Clay in Multi-Axial Tests

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SUMMARY Preliminary test data for undisturbed Shanghai Clay samples measured with a new multi-axial cell is presented. Although the influence of $\sigma_2 \neq \sigma_3$ on strength and stiffness characteristics of Shanghai Clay conformed in broad terms with the findings based on re-constituted laboratory soils, existing theories cannot satisfactorily predict the test results from $\sigma_2 = \sigma_3$ data. However, the test results appear to indicate that interference between approaching adjoining platens has been reduced to an insignificant level. The new multi-axial cell is simple to operate and is a viable testing tool.

1 INTRODUCTION

It has been recognized that laboratory tests need to model the stress state experienced by typical soil elements under prototype conditions in order to provide data for predicting, reliably, the performance of a geotechnical structure. Thus it is necessary to examine the effect of $\sigma_2 \neq \sigma_3$ on the stress-strain-strength characteristics of soils by the use of multi-axial testing where all three principal stresses can be varied independently.

The challenge in the design of multi-axial cells is to devise a loading system which

(i) can impose uniform normal stress along the boundaries, including the corners/edges,

(ii) can accommodate displacement of adjoining boundaries without deviation from cubic in geometry or uniform in normal stress,

(iii) can eliminate interference between approaching adjoining boundaries.

A range of cells of various degree of complexities (Hambly 1969, Ko & Scott 1967, Sutherland 1969, Green 1972, Lade 1978 & Sture 1979) have been developed and used in research with re-constituted samples with varying degrees of success or lack of success. However, the application of multi-axial testing to undisturbed samples retrieved from a construction site is relatively limited. Furthermore, "...[existing] data from complicated laboratory apparatus in which the principal stress can be independently controlled [ie multi-axial cell] are notoriously suspect..." (Wroth 1984). It is also recognized that the observed response of soil is influenced by detailed experimental technique such as sample preparation details (Green 1972). Thus an equally important factor in the application of multi-axial testing to undisturbed samples is simplicity. This paper presents preliminary test data for undisturbed Shanghai Clay samples tested under a range of multi-axial stress conditions using a new multi-axial cell.

2 MULTI-AXIAL TESTING

The multi-axial cell (referred to as Mk 1) utilizes the conventional triaxial apparatus to provide the major and minor principal stresses but the intermediate stress is applied via a specially designed loading unit referred to as "composite boundary" (Fig 1) developed in the UNSW (Lo 1885). The "composite boundary" consists of an undersized rigid platen set in the loading face of a pressure bag. Special attention was paid to the design of the pressure bag to ensure σ_2 is equal to the pressure inside the bag - a characteristic that cannot be taken for granted (Arthur 1972). The "composite boundary" combines the advantages of both flexible and rigid boundaries.

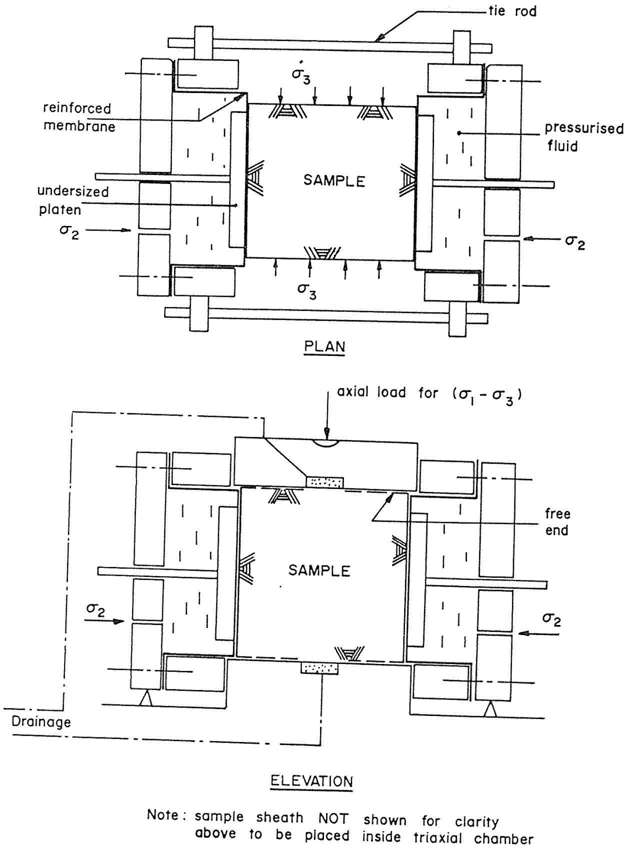

Figure 1 Multi-axial Cell

The application of σ_2 via pressure bags contributes to uniformity of boundary stress. The flexible corners eliminate corner/edge imperfections and avoid interference between approaching adjoining platens. The undersized rigid platen permit accurate monitoring of boundary displacement by LVDT and provide some "damping" against instability due to minor non-uniformity. Tests conducted on Sydney sand using the UNSW-cell where all three stresses were applied by the "composite boundary" mounted on a cubic spacing frame indicated satisfactory performance (Lo 1985, Lo & Lee 1987). Mk1 was developed with the objective of simplification in accommodating undisturbed samples. Mk1 can also accommodate high strain which is a desirable characteristic for testing clay samples.

3 TESTING PROGRAMME

Samples of the Shanghai Clay were handcut from undisturbed block samples retrieved from construction site. The clay was interspersed with thin fine sand layers (Termed layer 3 - a shallow sea deposit). Typical properties were:

Water content - 37%
Bulk density - 1.84 t/m^3
Void ratio - 1.01
Specific gravity - 2.70

Drained strength parameters measured in direct shear tests were Ø = 30° and c = 9 kPa

Samples were initially isotropically consolidated under a pressure of either 100kPa or 200 kPa. Drained constant σ_3 tests were then completed by incrementing σ_1 and σ_2 such that $b = (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)$ was maintained constant.

A series of five tests were completed for b of 0.0, 0.37, 0.58, 0.77 and 0.95 using a σ_1 increment of 15kPa. The tests were conducted such that σ_1 and σ_2 were always provided by the axial load and the "composite boundary" respectively for the complete range of b-values. Failure was defined as the peak deviator stress.

The setting up of the samples can be based on techniques similar to that of conventional triaxial testing. The samples deformed very uniformly throughout the tests except a shear plane was observed for test at b = 0.37.

4 STRENGTH DATA

Failure Criteria

The variation of drained friction angle ϕ with b was first studied, where ϕ is defined by the Mohr-Coulomb failure function given by

$$\sigma_1(1 - sin\phi) = \sigma_3(1 + sin\phi) + 2c.cos\phi \qquad (1)$$

In computing ϕ corresponding to the failure stress state, it was assumed that the effective cohesion c is independent of b. This assumption is considered to be reasonable and any errors arising from this assumption is likely to be insignificant. The variation of ϕ with b is plotted in Fig 2. It is evident that ϕ is dependent on b and the Mohr-Coulomb failure criterion (with constant ϕ) is not satisfactory for the drained strength of Shanghai Clay.

Three other failure criteria were then examined.

(1) The Drucker Praeger (sometimes referred to as extended Von Mises) criterion can be expressed as

$$\sqrt{J_{2D}} - \alpha.I_1 - k = 0 \qquad (2)$$

where $I_1 = \sigma_1 + \sigma_2 + \sigma_3$
$J_{2D} = (\sum(\sigma_1 - \sigma_2)^2)/6$

α and k represent the frictional and cohesive components respectively. If eqn (2) is made to agree with eqn (1) at

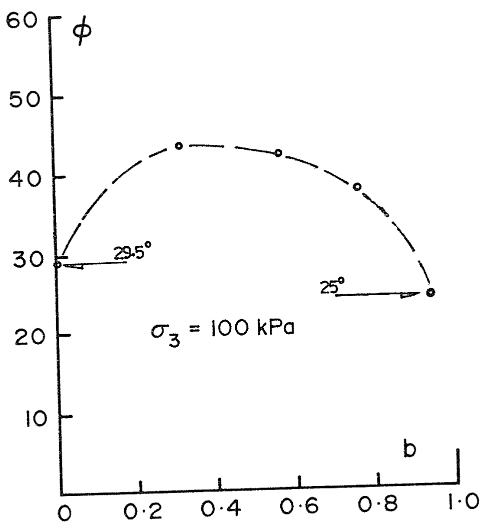

Figure 2 $\phi - b$ Curves

b=0, then

$$k = \frac{6c.cos\phi}{\sqrt{3}.(3 - sin\phi)} \qquad (2a)$$

Thus $M = \sqrt{J_{2D}} - k/I_1$ is a constant.

(2) The failure criteria by Lade (1979), if the curvature of the failure envelop in the meridian plane is neglected, can be expressed as

$$I_1^3/I_3 = n = constant \qquad (3)$$

where $I_3 = I_1.I_2.I_3$
Eqn (3) is based on empirical fitting to a wide range of test data.

(3) The SMP failure criterion (Matsuoka 1976) is given by

$$r = \sqrt{\frac{I_1.I_2 - 9I_3}{9I_3}} = constant \qquad (4)$$

where $I_3 = \sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_3\sigma_1$
The SMP failure criterion is based on the theoretical postulate that the stress-strain-strength properties of soils are characterized by the shear and normal stresses acting on a stress-state dependent plane called Spatial Mobilized Plane.

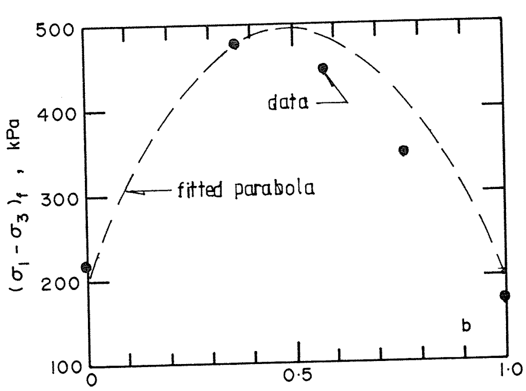

Figure 3 Failure Stresses Versus b

Eqn (3) and (4) inherently assumed effective cohesion is zero. For Shanghai Clay tested at a consolidation stress of $\sigma_3 \geq 100kPa$, the error involved in neglecting the effective cohesion is small. The values of m, n, and s are presented in Tab. 1. It is evident that none of the above criteria can satisfactorily model the drained strength of Shanghai Clay for the complete range of b-values. A parabolic function given by eqn (5) was proposed to fit, empirically the test data (Fig 3).

$$(\sigma_1 - \sigma_3)_f = (\sigma_1 - s_3)_{f,max} - 4(b - 0.5)^2. \quad [(\sigma_1 - \sigma_3)_{f,max} - (\sigma_1 - \sigma_3)_{f,o}] \tag{5}$$

where subscript f - failure
max - maximum value at b=0.5
o - minimum value at b=0

Note that eqn (5) requires the strength data at two b-values to define the parabolic function.

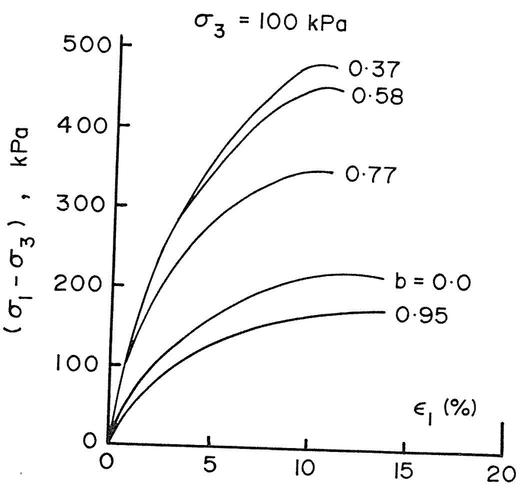

Figure 4 Stress-Strain Curves

Strength at b = 1

The drained strength at b close to unity is then examined in further detail. This aspect of strength characteristic has still not been satisfactorily resolved even for remolded "laboratory soil". Pearce (1972) reported that ϕ at triaxial compression (b = 0) was the same as that at triaxial extension

TABLE I

Parameters for Failure Functions

b	m	n	r
0.0	0.224	44.05	0.581
0.37	0.243	54.74	0.797
0.58	0.213	52.04	0.799
0.77	0.188	46.75	0.725
0.95	0.131	35.80	0.483

(b = 1) for remoulded Kaolin using the Cambridge 'True Triaxial Apparatus'. But Lade (1979) reported test data that support the use of eqn (3), which give ϕ at b=1 greater than that at b=0, for remoulded Gunite Clay. Similar conflicting results were also reported for loose to dense sand with ϕ ranging from 30 to 45 degree. Sutherland (1969) and Matsuoka(1982) suggested that the higher value of ϕ at b=1 compared with that at b = 0 could be due to interference between approaching adjoining platens. The series of test conducted on Toyoura sand and Fujinomoric clay (Matsuoka & Nakai 1982) appeared to support such a hypothesis. The test data for undisturbed Shanghai Clay is not inconsistent with this hypothesis and that the "composite boundary" can satisfactorily eliminate interference between approaching σ_1 and σ_2 platens. However, the observation that ϕ at b=0.95 was 4.5 degree lower than that at b=0 cannot be explained by existing theories although this could be due to inherent anisotropy and/or variability of Shanghai Clay.

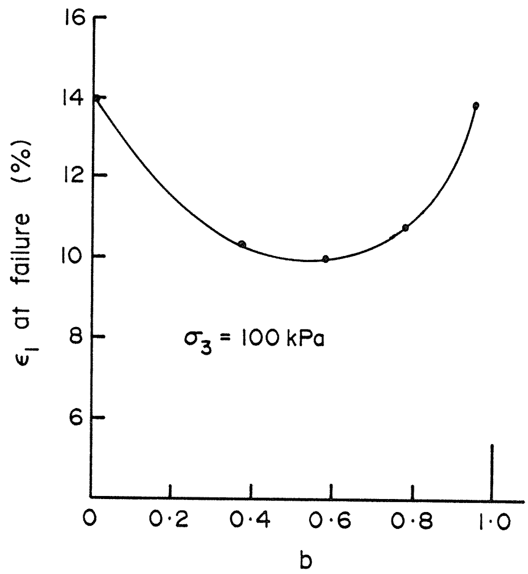

Figure 5 Axial Strain at Failure

5 STIFFNESS CHARACTERISTICS

The dependence of stiffness on σ_2 was clearly illustrated by the stress-strain curves presented in Fig 4. The stiffness of the $(\sigma_1 - \sigma_3)$ versus ϵ_1 curves increases with b in the range of b = 0 to b = 0.37 and then reduces with b. This was consistent with the axial strain at failure shown in Fig. 5. However, if the stiffness data in the major and intermediate principal stresses direction were interpreted based on the assumption of isotropic elasticity, it was found that the initial tangent "Young's modulus" increased with b over a larger range of b = 0 to b = 0.77 (Fig 6). Thus a simplified model used in an analysis needs to be consistent with the method of interpreting multi-axial test data.

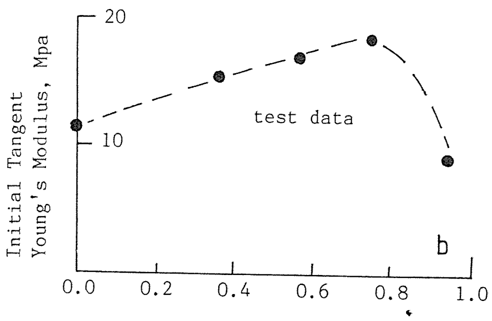

Figure 6 Initial Tangent Young's Moduli

The $\epsilon_1 - \epsilon_2$ curves are presented in Fig 7. Plane strain condition occurs approximately at b = 0.37. It is evident that ϵ_2 to ϵ_1 ratio increases with b except for the test at b = 0.95 which exhibit some anomaly. This may be due to variability of natural soil. However, it can still be be concluded that the stiffness characteristics of Shanghai Clay are anisotropic since ϵ_1 and ϵ_2 is evidently different at $\sigma_1 \approx \sigma_2$. This is consistent with test results from conventional triaxial compression tests using both vertical and horizontal samples (Lo et al 1988).

6 CONCLUSIONS

Preliminary strength and stiffness characteristics of undisturbed Shanghai Clay samples measured in a new multi-axial cell are presented. Existing failure functions did not appear to predict the test data satisfactorily and a parabolic function was proposed to fit the strength data. The question of drained friction angle at b close to unity was still not satisfactorily resolved. However, interference between approaching adjoining platens appeared to be eliminated by the new cell since a monotonic increase of ϕ with b was not observed. Experience gained from the presented testing confirmed that the new multi-axial cell satisfy the important criterion of simplicity in operation and thus can readily be used in measuring more appropiate parameters for predicting the performance of geotechnical structures.

7 ACKNOWLEDGMENTS

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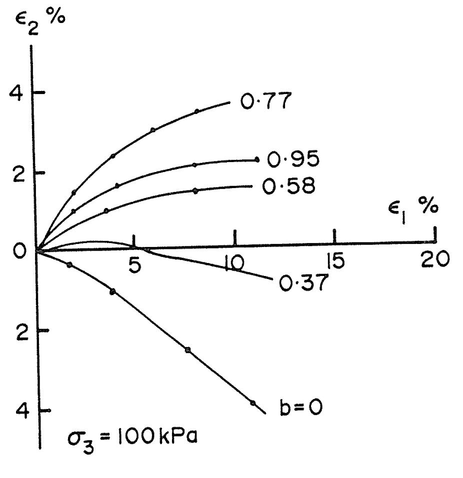

Figure 7 $\epsilon_1 - \epsilon_2$ Curves

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