The Nature of Anisotropy in Soft Clays

L. D. WESLEY
Tonkin and Taylor, Consulting Engineers, Auckland

1 INTRODUCTION

This paper presents the results of a study of the undrained strength anisotropy of a soft clay and relates the results to measurements of such anisotropy in other similar clays. The clay involved is a soft sedimentary clay which occurs along the north coast of the Thames estuary in the county of Essex. The clay is believed to have been deposited over the last 10,000 years during the gradual rise of the sea level from its low levels during the last ice age. Properties of the clay are as follows:

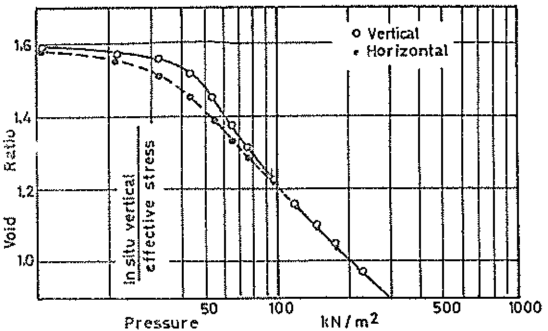
Natural Water Content	51%
Plastic Limit	25
Liquid Limit	54
Clay Fraction	30
Sensitivity	6
Undrained Shear Strength (approximately)	15 kPa

The tests presented in this paper form a small part of a much wider study described fully by Wesley (1975). The tests were performed as part of an investigation of the properties of these coastal clays carried out to assist in the design of higher stopbanks being planned as part of the London flood protection scheme.

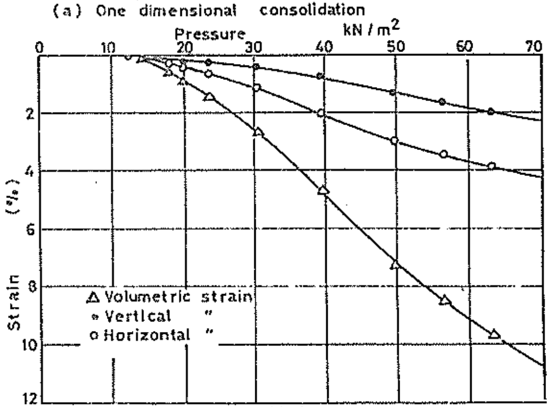
The samples used in the study were taken from a depth of just over 3 m in a wide test pit, using large diameter steel cylinders. These cylinders were 250 mm (10 in) in diameter and 300 mm (12 in) high. The cylinders were pushed into the soil using a dead weight pressing on a plate resting on the upper rim of the cylinder. When full the cylinders were dug out by hand. It is believed that the degree of disturbance using this procedure was considerably less than that which result from normal sampling with a relatively small diameter tube in a borehole.

2 MEASUREMENTS OF SOIL SKELETON STIFFNESS

As a starting point to this study of the anisotropy of soft clays we will look at the results of some measurements of soil skeleton stiffness made in drained tests of various types. The simplest such test is the conventional oedometer test and it is easy to perform these tests on both vertical (i.e. compression axis in the vertical direction) and horizontal samples. The results of such tests on the Mucking clay are given in Fig.1 (a). It is seen that in the initial stages of loading the soil is substantially more compressible in the horizontal direction than in the vertical direction. At higher stress levels the curves become identical. Also shown in Fig.1 (b) are the strains measured in an isotropic (all round) compression test. The initial stress in this test was the pore water tension in the sample. The strain in the horizontal direction is initially much greater than in the vertical direction, but at higher stress levels the strains become the same (and the curves become parallel).



(a) One dimensional consolidation



(b) Isotropic consolidation

Figure 1 Skeleton Stiffness in the Horizontal and Vertical Directions

In Fig.2 the initial portions of stress strain curves from drained triaxial tests on vertical and horizontal samples are shown. These curves also show that the soil skeleton is less stiff in the horizontal direction than in the vertical direction. It is of interest to note that the volume change is almost identical regardless of the direction of stress application.

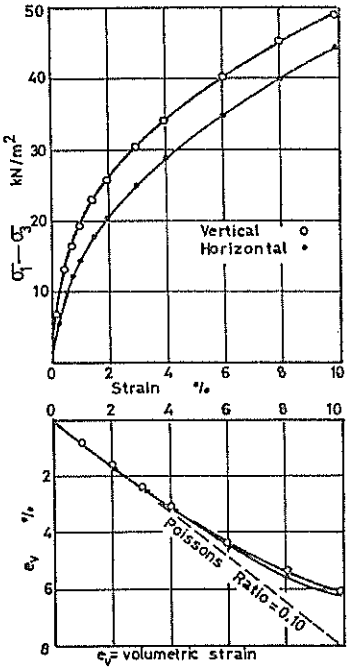


Figure 2 Drained Triaxial Tests on Vertical and Horizontal Samples

A careful analysis of the above test results suggest that the soil skeleton stiffness in the horizontal direction is about half that in the vertical direction. It is not known how representative the above results are of soft sedimentary clays as data from such clays is not available.

3 INFLUENCE OF SOIL SKELETON STIFFNESS ON BEHAVIOUR IN UNDRAINED LOADING

The behaviour of the soil in undrained loading will clearly be influenced by the differences in stiffness of the soil skeleton. To gain some idea of what this influence is likely to be we can make use of elastic theory and the assumption that the soil is transversely isotropic. The simplest means of evaluating the influence which anisotropy may have on undrained strength is by examining its influence on pore pressure response to various types of loading. If the assumption is made that the Mohr-Coulomb failure envelope is isotropic then differing pore pressure responses will lead to differences in undrained strength. We will therefore examine the value of the pore pressure parameter A ($A = \Delta u / \Delta\sigma_1 - \Delta\sigma_3$) for different loading conditions.

Using the same nomenclature as Bishop and Hight (1977) for the elastic parameters we can obtain the following values of A for the following loading conditions:

Loading Condition	A
Triaxial Vertical	$\dfrac{E_x/E_y - 2\mu_{xy}}{E_x/E_y - 4\mu_{xy} - 2\mu_{xx} + 2}$
Triaxial Horizontal	$\dfrac{1 - \mu_{xy} - \mu_{xx}}{E_x/E_y - 4\mu_{xy} - 2\mu_{xx} + 2}$
Plain Strain Vertical	$\dfrac{E_x/E_y - \mu_{xy} - \mu_{xy}^2 - \mu_{xx}\mu_{xy}}{E_x/E_y + 1 - 2\mu_{xy} - 2\mu_{xy}\mu_{xx} - \mu_{xx}^2 - \mu_{xy}^2}$
Plain Strain Horizontal	$\dfrac{1 - \mu_{xy} - \mu_{xy}\mu_{xx} - \mu_{xx}^2}{E_x/E_y + 1 - 2\mu_{xy} - 2\mu_{xy}\mu_{xx} - \mu_{xx}^2 - \mu_{xy}^2}$

The term "triaxial vertical" refers to a triaxial compression test on a sample prepared with its axis in the vertical direction. See Fig.3

Tests carried out on the Mucking clay led to the following values for the elastic parameters.

$\dfrac{E_x}{E_y} = 0.5$, $\mu_{xy} = 0.05$, $\mu_{xx} = 0.15$

and inserting these values in the above expressions leads the following values of A.

Loading Condition	Pore Pressure Parameter A
Triaxial Vertical	0.20
Triaxial Horizontal	0.40
Plane Strain Vertical	0.33
Plane Strain Horizontal	0.66

These values show that higher pore pressures are to to expected from horizontal loading than from vertical loading, and that plane strain loading will result in higher pore pressures than triaxial loading (as with the isotropic case). If we assume that the Mohr-Coulomb failure envelope for the material is defined by $c' = o$ and Ø = 30°, and that the initial state of stress is the same, then the above A values imply a strength for horizontal triaxial loading about 73% of that for vertical triaxial loading, and a similar ratio for the two plane strain loading conditions. The plane strain strengths would be expected to be about 80% of the triaxial strengths respectively.

The above indictation is intended as a guide only to expected behaviour. It is widely recognised that soils do not behave elastically although it is perhaps not unreasonable to expect them to approximate to elastic behaviour at small strains beginning from in situ stress levels.

4 UNDRAINED TRIAXIAL AND PLANE STRAIN TESTS

We will now look at the results of undrained tests to see whether these are in agreement with expectations from the above analysis. These tests include triaxial and plane strain tests on samples prepared at varying inclinations. The full range of test types is indicated in Fig.3. Samples trimmed with their axis vertical, horizontal, and inclined at 45° were tested in both triaxial and plane strain compression. In addition a fourth type of plane strain sample was tested; this was a sample trimmed so that the plane strain axis was the vertical axis in situ.

It should be noted that in the plane strain tests the planes on which failure can occur are much more restricted than in the triaxial tests. For example, in a triaxial test on a horizontal sample failure could occur on planes vertical in situ, or on planes which in situ dip at approximately 30°, or on any intermediate planes. In plane strain tests, however, failure could only occur on planes dipping at approximately 30°; failure on planes vertical in situ, is not possible. This point is of some importance when attempts are made to relate compressive strengths to strengths on particular planes. Each test was carried out several times and the results presented are averaged from the several results. The tests were carried out in a triaxial apparatus or plane strain apparatus and a cell pressure equal to the overburden pressure applied in each case.

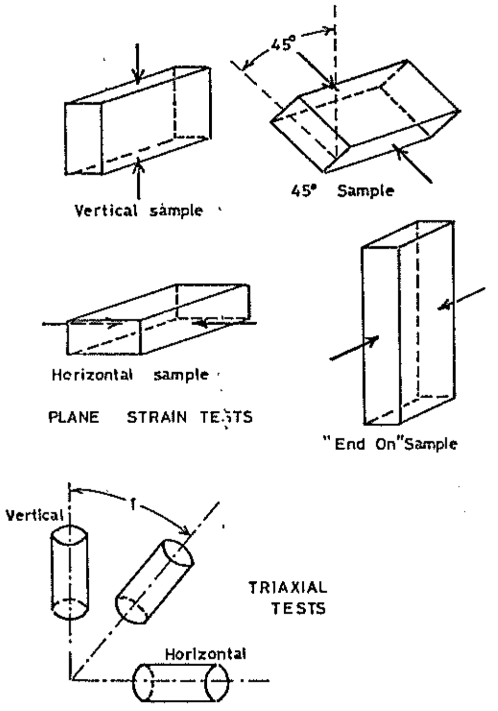


Figure 3 Types of Undrained Tests

The results of these tests are shown in Figs.4 and 5 in the form of stress strain curves. The most significant features of the results are the following:

(a) These is a steady decrease in strength as the angle of inclination changes from vertical to horizontal

(b) The curves from the vertical samples show a much sharper peak than those from the horizontal samples

(c) The initial slope of the curves is reasonably similar in all tests

Similar triaxial and plane strain tests were carried out on a range of samples taken from different depths, with similar results to those in Figs 4 and 5, except for the strength difference between triaxial and plane strain tests. The data in Figs. 4 and 5 suggests that the strength in plane strain tests was significantly lower than in triaxial tests. However, taken as a whole, the test series indicated that the strength in plane strain was only marginally lower than in triaxial compression.

In Fig.6 the data is summarised in the form of a polar diagram. The ratio of the strength in horizontal compression to that in vertical compression varied between 0.64 and 0.78.

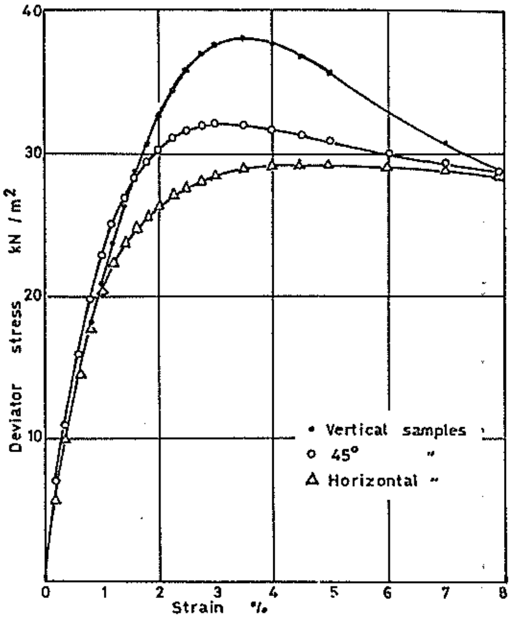


Figure 4 Undrained Triaxial Tests

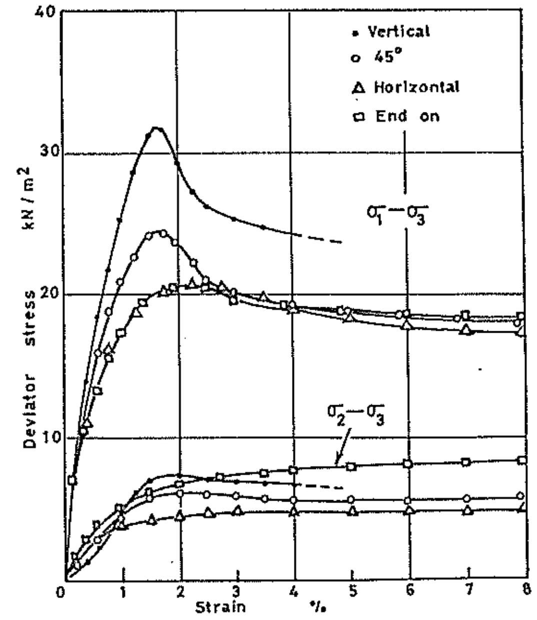


Figure 5 Undrained Plane Strain Tests

These undrained test results are similar to those obtained by Lo (1955) from tests on a similar clay. They are also in agreement with the predictions made earlier in this paper from considerations of the anisotropy of the soil skeleton stiffness. Of particular significance are the results from the plane strain tests on horizontal samples and "end on" samples. Despite the fact that failure is forced to take place on entirely different planes in these two tests, the strength obtained is the same. These suggest strongly that the strength is not related to the orientation of the failure planes, but to the direction of the stress application since the direction of stress application in these two tests is the same.

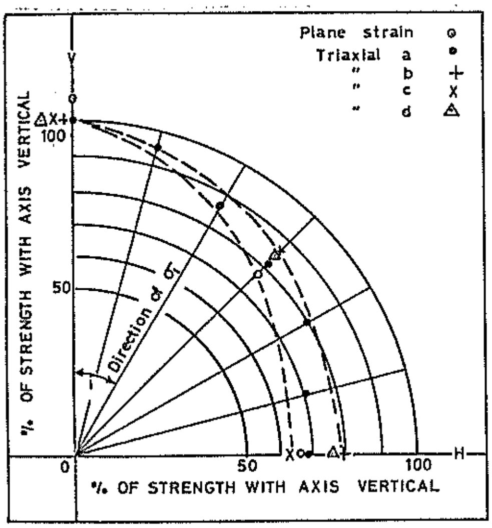


Figure 6 Polar Diagram of Strength With Inclination

5 CONSOLIDATED UNDRAINED TESTS

In the above sections it has been suggested that the anisotropy in undrained strength results from differing pore pressure responses during loading, rather than from anisotropy in the Mohr-Coulomb failure envelope (i.e. in the c' and Ø' values). To check the actual pore pressure response during loading, and the nature of the Mohr-Coulomb envelope, several series of different types of test have been carried out. These have included the following:

(a) Triaxial compression tests on vertical and horizontal samples

(b) Triaxial extension tests on vertical and horizontal samples

(c) Triaxial compression and extension tests on samples prepared at intermediate inclinations to the vertical

(d) Plane strain compression tests on vertical and horizontal samples

Before testing, all samples were consolidated to the mean in situ stress level. This was necessary as sensible comparisons of pore pressure response could only be made if the initial stress states were the same. Full details of these tests are given by Wesley (1975) and only the results of direct relevance to the present paper are presented here.

In Fig.7 all of the failure values are plotted on a conventional plot of $\frac{\sigma_1' - \sigma_3'}{2}$ Versus $\frac{\sigma_1' + \sigma_3'}{2}$. The values plotted in each case are the averages from four identical tests. It is seen that all the values lie close to a single line and demonstrate clearly that anisotropy in the values of c' and Ø is negligible. It was expected intuitively that the c' and Ø values may have been greater on the horizontal plane, since this was the plane of maximum effective stress in situ, and also the plane of bedding, but the tests on inclined samples showed no evidence of this.

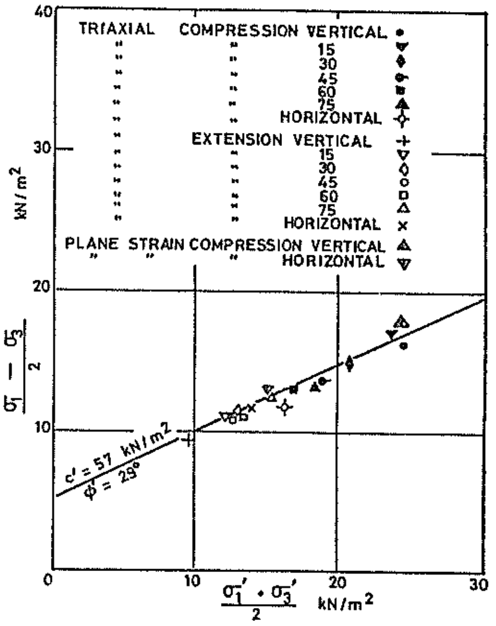


Figure 7 Failure Envelope from Differing Test Types

It is of interest to make a specific comparison of behaviour in undrained tests that predicted from drained tests using elastic theory. In Fig.8 the predicted stress paths and the actual experimental stress paths are shown for four different test types. These test types were:

Triaxial compression and extension on vertical samples

Triaxial compression and extension on horizontal samples

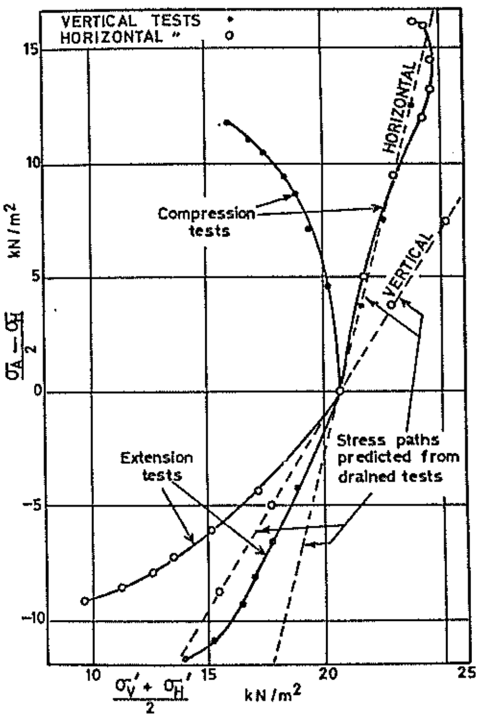


Figure 8 Undrained Stress Paths in Triaxial Tests

It is seen that the stress paths are in agreement with the general trend predicted by elastic theory, but are displaced to the left of the theoretical paths. This means that the actual pore pressures are higher than the theoretical values.

6 OTHER MEASUREMENTS OF UNDRAINED STRENGTH ANISOTROPY

An examination will now be made of measurements of anisotropy reported in the literature and the interpretations which have been placed on such measurements. The simplest, and most common method of measuring undrained strength anisotropy is by carrying out undrained compression tests on cylindrical samples cut at varying inclinations to the vertical. A comprehensive set of such tests was done by Lo (1965) and the results were very similar to those presented in Figs.4 and 6. As Bishop (1966) has pointed out there is considerable ambiguity associated with this type of test when attempts are made to relate particular values of undrained strength to particular planes.

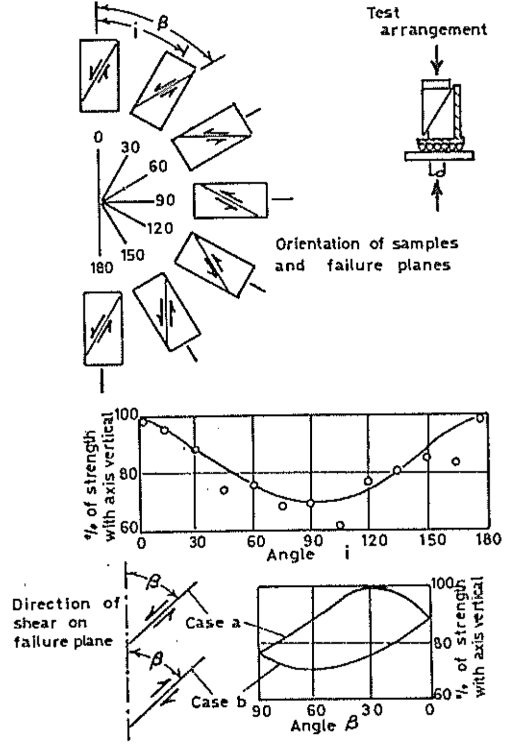


Figure 9 Evidence of Semi-Confined Tests

To overcome this ambiguity, De Lory and Lai (1971) carried out tests on block specimens which they called semi-confined as they used an arrangement (see Fig.9) which restricted failure to one particular plane. Samples were tested at values of i varying from 0 to 180°, and the results are shown in Fig.9. When plotted as strength versus the angle i the results show an identical trend to Lo's tests and the tests presented in this paper, i.e. maximum strength when the axis is vertical and minimum strength when the axis is horizontal. However the results can also be plotted against **β**, the angle of the failure plane to the vertical, and with this plot two curves are obtained as seen in Fig.9. The upper curve (case a) applies when shear is occurring in the same direction as the existing shear stress in the ground, and the other (case b) when the direction of shear is reversed. Dr.Lory and Lai believe their results support a hypothesis put forward by Bjerrum (1972,1973) that the soil structure has a greater resistance to shear in one direction than the other. However it is not at all established that unique values of undrained strength (dependent only on the direction of shear) can be associated with particular planes. A simpler explanation of De Lory and Lai's results is that suggested in this paper, namely that the values of c' and ∅' are not influenced by orientation, but that the pore pressure is, the response increasing as i increases from 0° to 90°.

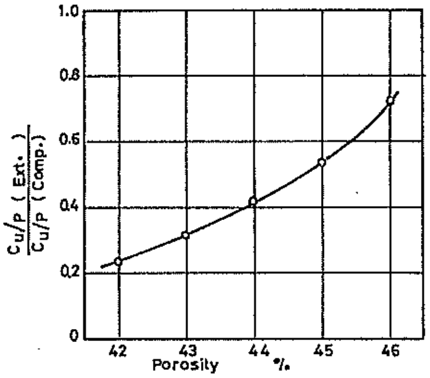
A further method which has been used to measure undrained strength anisotropy is by carrying out field vane tests using vanes of varying height to diameter ratios. Tests by Aas (1965 and 1967) showed the strength on the horizontal plane to be generally greater than the vertical plane and Aas suggests that this results from the differing in situ effective stresses acting on the planes. The results of these vane tests do not appear compatible with the results of compression tests at varying inclinations. If in a normally consolidated clay the minimum strength is found, on vertical planes, then it follows that in tests on cylindrical specimens, the specimen cut with the axis inclined at 30° to the vertical should have the lowest strength as failure would occur on a plane originally vertical in the ground.

Finally, mention should be made of work done by Bjerrum and co-workers at the N.G.I., who have proposed undrained triaxial compression and extension tests as a method of measuring undrained strength anisotropy. Bjerrum and Kenney (1967) present results of compression and extension tests which show a large difference in strength, and put forward their hypothesis to explain this difference. Their hypothesis is that the soil skeleton has developed a greater resistance to shear when the shear stress acts in the same direction as it does in situ than when it acts in the reverse direction (which is the case in an extension test). Bjerrum (1973) and Berre and Bjerrum (1973) further develop this hypothesis and use the ratio of extension to compression strength as a measure of the anisotropy.

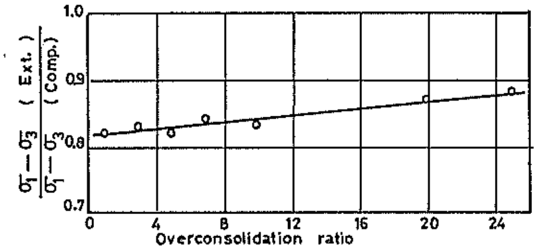
Both the hypothesis and the use of extension and compression tests as a measure of anisotropy are open to serious criticism. There is no clear explanation of what is meant by the hypothesis; in particular whether the "greater resistance to shear" in one direction than the other is because of higher c' and ∅ ' values, or because of lower pore pressures due to some sort of increased resistance to the development of pore pressures in one direction.

The use of undrained triaxial extension and compression tests as a means of measuring anisotropy does not appear valid, as theory predicts and experiment shows that undrained extension tests will give lower strengths than undrained compression tests even with an isotropic soil. Fig. 10 shows the results of undrained extension and compression tests on isotropic sand and clay samples. It is seen that the extension tests give a substantially lower strength than compression tests, the difference being greater with sands than with clays.

This review of other measurements of undrained strength anisotropy has been very brief but sufficient to indicate that such measurements are compatible with the explanation for anisotropy put forward in this paper. The explanation offered in this paper appears to provide a better basis of understanding undrained strength behaviour than do the various hypothesies put forward by the authors.



SAND (Bishop and Eldin, 1953)



REMOULDED LONDON CLAY (Parry, 1960)

Figure 10 Extension and Compression Test on Isotropic Samples

7 CONCLUSION

An explanation has been put forward to account for the undrained strength anisotropy of soft clays. The key to this explanation is the difference in the stiffness of the soil skelton in the vertical and horizontal directions. That such a difference should exist is not surprising since the effective stress to which the skeleton is subjected in situ is about half in the horizontal direction what it is in the vertical direction.

It follows from the above explanation that it is not possible to associate unique values of undrained strength with planes at particular orientations. In taking account of undrained strength anisotropy in stability estimates it is the directions of the principal stresses which should be investigated rather than the angle of the failure plane. Lo (1965) in taking account of anisotropy starts from the position of the slip surface and assumes that the principal stresses are inclined to it at a fixed angle. A more logical procedure would be to determine the principal stress directions (by such a method as finite element analysis) and then to assign strength values to each point throughout the soil mass making use of the laboratory test data. This procedure would lead to some differences with Lo's assumptions.

The purpose of this paper has been to make a contribution to the understanding of undrained strength anisotropy rather than to analyse its effect in practice. It should be mentioned however that the effect in practical situations is not great. For slip circle stability analysis, a very close approximation to the true effect of anisotropy will be obtained simply by using the average of vertical and horizontal strengths in a conventional analysis.

8 REFERENCES

AAS G. (1965) A study of the effect of vane shape and rate of strain on the measured values of in situ shear strength of clays. Proc.6th Int.Conf. on Soil Mech. Toronto.Vol 1, 141-145

AAS G. (1969) Vane tests for investigation of anisotropy of undrained shear strength of clays. Proc. Geotechnical Conf.Oslo. Vol 1, 3-8.

BERRE,T.AND BJERRUM,L (1973) Shear strength of normally consolidated clays. Proc.8th Int.Conf.on Soil Mech. Moscow. 1.1, 39-40.

BJERRUM,L.AND KENNEY T.C. (1967) Effect of [illegible] structure on the shear behaviour of normally consolidated quick clays. Proc.Geotechnical Conf. Oslo. Vol 2, 19-27

BJERRUM (1973) Problems of soil mechanics and construction on soft clays. Proc.8th Int.Conf. on Soil Mech. Moscow.

BISHOP,A.W. AND ELDIN,G. (1953) The effect of stress history on the relation between Ø and porosity in sand. Proc.3rd Int.Conf. on Soil Mech. Zurich Vol 1, 100-105.

BISHOP,A.W. AND HIGHT (1977) The Value of Poisson's ratio in saturated soils and rocks stressed under undrained conditions. Geotechnique 27,No3,369-384

BISHOP,A.W. (1966) The strength of soils as engineering materials Sixth Rankine Lecture. Geotechnique 10:2, 89-129

DE LORY,F.A.AND LAI,H.W. (1971) Variations in undrained shearing strength by semi-confined tests. Canadian Geotech.Journal 8, 538-545

LO,K.Y. (1965) Stability of slopes in anisotropic soils. ASCE Journal Soil Mech and Found.Eng. Vol 91,SM4,85-106

PARRY,R.H.G. (1960) Triaxial compression and extension tests on remoulded saturated clay. Geotechnique Vol 10, No 4, 166-180.

WESLEY (1975) Influence of stress path and anisotropy on the behaviour of a soft alluvial clay. Ph.D. Thesis, Imperial College.