Some Influences of Anisotropy in Clay Soils and Soft Rocks

R. H. G. Parry

ABSTRACT : Anisotropic behaviour in soils and soft rocks may be either fabric or stress related and in practice is invariably a combination of both. Theoretical studies in the paper include the influence on undrained strength of assuming both the critical state and Mohr-Coulomb concepts to hold, and the influence of elastic anisotropy on predicted undrained effective stress paths. The predictions stemming from these theoretical concepts are examined in the light of evidence from triaxial compression and extension tests on laboratory prepared, compacted and natural clays and from triaxial compression tests on clay shales. The experimental studies also show the influence of sample orientation on undrained shear strength, as well as the influence of anisotropy on the effective stress angle of shearing resistance and of stress path on measured stiffness.

1. INTRODUCTION

All soils behave anisotropically when deformed, but it is not usual to take specific account of this in design or analysis. This is so despite the large influences anisotropy can have on the deformation characteristics of the soil. For example, both theoretical considerations and observed behaviour show that it is possible to get a two to one ratio in undrained shear strength of a clay depending upon the applied stress path. One of the main problems in studying anisotropy in soils is that both the structure or fabric of the soil and the applied stress path induce anisotropic behaviour and it is impossible to completely separate these. The purpose of this paper is to pesent some of the writer's observations of anisotropic behaviour in clays and soft rocks and to discuss, where possible, the influences of stress and structure. Summaries of published anisotropic behaviour have been presented (for example, Duncan and Seed 1966, Mayne 1985) and it is not intended to repeat this here.

2. BASIC CAUSES OF ANISOTROPIC BEHAVIOUR

Anisotropic deformational behaviour in soil and soft rocks arises from :

(a) Physical anisotropy in the fabric of the soil or soft rock.
(b) The stress history experienced by the soil or soft rock, and stress conditions obtaining immediately before loading.
(c) The applied stress path.

These different causes of anisotropic behaviour interact strongly with each other making it impossible to separate the influence of each ; but in some cases at least it can be shown that one particular form of anisotropy is dominating the strength and deformation behaviour.

Fabric or structural anisotropy arises both from preferred particle orientation and from the packing of the particles. The former will be most influential where the particles are elongated or flat, having one dimension either much less or much greater than the other two while the latter will be the dominant influence in soils consisting of bulky particles. Even with pure single sized spheres anisotropic packing will occur.

In attempting to distinguish the effects of anisotropy in a soil or soft rock it would be convenient to have as a starting point the observed behaviour of an isotropic specimen, but this is not possible because a specimen would not be isotropic even with random orientation or random packing of particles. Nevertheless, such a specimen represents the best starting point to assess anisotropic effects, but with the realisation that preferred particle orientation or packing will be initiated as soon as shear deformation of the specimen commences.

As this paper is concerned with clay soils and soft argillaceous rocks, fabric anisotropy is mainly related to particle orientation. In their natural state such orientations will have been produced both during deposition and under the stresses subsequently imposed by overburden pressures and perhaps other field events.

3. ELASTIC BEHAVIOUR

Although soils do not behave elastically in the sense of deforming linearly and reversibly under imposed stress, it is recognised that within prescribed limits assumed elastic behaviour can be usefully employed in analysis or design. For example, it is common to calculate stresses at a point below a loaded area by assuming an elastic medium. Some soft rocks may exhibit reversible strain behaviour over a limited range of applied stress.

While accepting that soils and most soft rocks do not deform in a fully elastic manner, saturated specimens tested undrained in the triaxial cell often exhibit a pore pressure response over a limited stress range similar to that which would be predicted by assumed elastic behaviour. It is useful, then to explore pore pressure responses (and thus resulting stress paths) predicted for undrained triaxial tests on isotropic and anisotropic elastic soils and soft rocks.

Skempton (1948, 1954) related volume change to effective stress change in triaxial test specimens by assuming isotropic elastic behaviour. Putting volume change equal to zero for an undrained test on saturated clay be derived the well known expression :

$$\triangle u = B\,[\,\triangle\sigma_3 + A(\triangle\sigma_1 - \triangle\sigma_3)\,] \quad \text{------------------------------------} \quad (1)$$

where

$\triangle u$	is the pore pressure response under applied principal total stress increment $\triangle\sigma_{1,}\ \triangle\sigma_3$
B	is the pore pressure response under applied isotropic stress, usually assumed to be unity
A	is a constant found to be equal to ⅓ for an isotropic elastic soil

For an undrained triaxial compression test on an isotropic elastic soil, if B=1

$$\triangle u = \triangle\sigma_r + \tfrac{1}{3}(\triangle\sigma_a - \triangle\sigma_r) \quad \text{--} \quad (2)$$

where σ_a, σ_r are the axial and radial total stresses respectively.

Equation (2) can also be written :

$$0 = \triangle\sigma_a' + 2\triangle\sigma_r' \quad \text{--} \quad (3)$$

from which the slopes of stress paths on q vs p′ and t vs s′ can be found, where q and p′ are the 'Cambridge" stress parameters $q = (\sigma_a' - \sigma_r')$, $p' = \tfrac{1}{3}(\sigma_a' + 2\sigma_r')$ and t, s′ are the 'MIT' stress parameters $t = \tfrac{1}{2}(\sigma_a' - \sigma_r')$, $s' = \tfrac{1}{2}(\sigma_a' + \sigma_r')$. The effective stress paths given by equation (3) are linear and have the slopes

$$\triangle q / \triangle p' = \text{infinite}$$

$$\triangle t / \triangle s' = +3.0$$

as shown in Figure 1 ($n'=1$).

In a triaxial test specimen a cross-anisotropic soil or rock has isotropic stiffness in radial directions, but a different stiffness in the axial direction. The effective stress elastic moduli are E_r', and E_a' respectively, and Poisson's ratios are ν_{ra}', the ratio of strain in the radial direction to an applied strain in the axial direction, and ν_{rr}', the ratio of strain in one radial direction to an applied strain in the orthogonal radial direction.

From superposition :

$$\nu_{ra}' / \nu_{ar}' = E_r' / E_a' = n' \qquad \text{-- (4)}$$

After setting up the elastic equations relating strains to effective stress changes and assuming zero volume change, the effective stress paths for undrained triaxial tests on saturated soil or soft rock are given by (Parry and Wroth 1976) :

$$\frac{\triangle q}{\triangle p'} = \frac{3(2\nu_{rr}' + 4n'\nu_{ar}' - n' - 2)}{2(\nu_{rr}' - n'\nu_{ar}' + n' - 1)} \qquad \text{-------------------------------- (5)}$$

or

$$\frac{\triangle t}{\triangle s} = \frac{2\nu_{rr}' + 4n'\nu_{ar}' - n' - 2}{2\nu_{rr}' - n' - 2} \qquad \text{-------------------------------- (6)}$$

The pore pressure parameter A is given by :

$$A = \frac{n'(1 - 2\nu_{ar}')}{n'(1 - 4\nu_{ar}') + 2(1 - \nu_{rr}')} \quad \text{------------------------------- (7)}$$

Thus, if a saturated soil or soft rock is assumed to be behaving elastically the ratio n' of horizontal to vertical stiffness can be obtained from the pore pressure response on the slope of the effective stress path, if ν_{rr}' and ν_{ar}' are known. In general these are not known, but some indication of the influence of stiffness anisotropy can be obtained by adopting the Henkel (1971) assumption :

$$\nu_{rr}' = \frac{1}{2}(\nu_{ar}' + \nu_{ra}') = \frac{\nu_{ar}'}{2}(1 + n') \quad \text{------------------------------ (8)}$$

If it is assumed that $\nu_{rr}'=0.2$ the stress paths in Fig. 1 are obtained for $n'=0.5$, 1.0 and 2.0.

Some soils and soft rocks do show substantially linear effective stress paths over an initial range of loading. In Figure 2, q vs p′ plots are shown for three undisturbed soils for which the effective stress paths showed some initial linearity. Details of the soils are :

Fulford Clay : Soft clayey silt, sample depth 2m, overconsolidation ratio approximately 2.0, w_L=65%, w_p=39%, w=55%.

London Clay : Stiff blue, sample depth 13m, OCR approximately 20, w_L=84%, w_p=29%, w=31.5%.

North Sea Clay : Very hard sandy silty clay, sample depth 18m below sea level, OCR approximately 9, w_L=38%, w_p=15%, w=14%.

It can be seen that stiffness ratios implied by these stress paths comply with values that might be expected for soils with these overconsolidation ratios. That is, the horizontal stiffness is less than the vertical stiffness in the lightly overconsolidated Fulford clay, but greater in the heavily overconsolidated London clay. The moderately overconsolidated North Sea soil is behaving initially in an isotropic manner.

4. CRITICAL STATE AND MOHR-COULOMB

The concept of critical state simply states that a mass of soil under continuing shear strain will achieve an ultimate state in which the mean effective stress is uniquely related to the voids ratio for the particular soil. The shear strength in turn is governed by the effective stress.

Although the concept of critical state itself makes no statement about the soil structure, it is reasonable to assume that if a unique ultimate state is to be achieved the shear strain should be uniform throughout the soil mass and that particles should not have a preferred packing or orientation. These conditions can never be strictly achieved even in laboratory prepared specimens. Nevertheless, the concept provides an excellent vantage point from which to view the behaviour of specific soils and, indeed, it is often found that even if a soil does not reach a true critical state the behaviour conforms to some degress with the principles embodied in the critical state concept.

One of these principles relates to the influence of stress path on undrained shear strength. It can be shown (Parry 1971) that by combining the critical state and Mohr-Coulomb strength criteria, the ratio of undrained shear strength in triaxial extension $c_u(E)$ to that in triaxial compression $c_u(C)$ is given by the expression :

$$c_u(E) \,/\, c_u(C) \;=\; (3 - \sin\phi') \,/\, (3 + \sin\phi') \quad \text{--------------------} \; (9)$$

where ϕ' is the effective stress angle of shearing resistance for the soil.

Equation (9) gives the ratios shown below for $\phi'=20^\circ$ and 30° :

ϕ'	$c_u(E) \,/\, c_u(C)$
20°	0.80
30°	0.71

A considerable amount of data is available in the literature supporting undrained strength ratios broadly of these magnitudes, which gives credence to the above theory and, in turn, implying that different stress paths generate anisotropic behaviour.

5. LABORATORY PREPARED CLAY SPECIMENS

Although soil can never be structurally isotropic or even have completely random orientation of particles, specimens mixed at high water content in the laboratory and consolidated under isotropic pressure in the triaxial cell can be expected to demonstrate behaviour as close to isotropic as can be achieved. Even these specimens, however, can exhibit markedly different behaviour under different stress paths. Triaxial tests (Parry 1960) on Weald Clay (w_L=46%, w_P=26%) showed ratios of undrained shear strengths in extension to those in compression to be consistently in the range 0.75 to 0.85, independent of overconsolidation ratio, which values conform with those expected from the combined critical state Mohr-Coulomb concept discussed above. For normally consolidated specimens, ϕ' values are given below ($c'=0$) :

	ϕ'	ϕ' (E)
Undrained	22.6°	21.3°
Drained	21.0°	22.3°

These small differences suggest that stress path has little influence on the effective stress angle of shearing resistance for isotropically consolidated laboratory specimens with low values of ϕ'. It may not be true for soils with higher ϕ' values.

Different consolidation histories combined with different applied stress paths can have a significant influence on ϕ', as shown by tests (Nadarajah 1971) on laboratory prepared specimens of kaolin (w_L=72%, w_P=40%) consolidated from a high moisture content (approx. $2w_L$) under both isotropic and K_o conditions. The resulting undrained values of ϕ'(C) and ϕ'(E) for normally consolidated and lightly overconsolidated specimens were found to be :

	ϕ' (C)	ϕ' (E)
Isotropically consolidated	22.6°	20.6°
Ko consolidated	20.8°	28.0°

As for Weald clay above, isotropically consolidated specimens showed ϕ'(E) values lower than ϕ'(C) values, which may have been due to the instabilities which can arise near failure in extension tests ; but the most striking feature is the much higher value of ϕ'(E) for the K_o consolidated specimens.

6. NATURAL SOFT CLAYS

Soft clays in the field usually exhibit a small degree of overconsolidation brought about by ground water movements of ageing. In most cases the ratio K_o of horizontal to vertical effective stresses will be slightly greater than the normally consolidated value, but less than unity. That is, it is likely to be in the range 0.65 to 0.9. As the vertical effective stress exceeds the horizontal effective stress, and has always done so, this will have a strong influence in establishing the anisotropic characteristics of the soil. The principle manifestation of this will be a lateral stiffness less than the vertical stiffness.

There is evidence that for some soft clays the lateral stiffness is about one-half the vertical stiffness. This is shown clearly in Figure 3 for Mucking Flats clay (Wesley 1975). Figure 3a shows a plot of consolidation pressure against settlement for laboratory specimens cut vertically (V) and horizontally (H) from undisturbed block samples, and in Figure 3b where the corresponding coefficients of compressibility (m_v) are plotted against consolidation pressure. It can be seen that for stress increments in the pre-yield phase, just in excess of the field stresses, the compressibility in the horizontal direction is about double that in the vertical direction.

Other features of these plots are that the maximum compressibility actually occurs in the vertical direction at a stress slightly higher than the preconsolidation pressure σ_{vm} and, as the consolidation pressure is increased beyond yield, the compression curves for the vertical and horizontal specimens approach each other and eventually merge at a consolidation pressure equal to about three times the preconsolidation pressure. That is, at a sufficiently high consolidation pressure the effects of initial structural anisotropy are subjugated. Identical behaviour to that shown in Figure 3a, b has been demonstrated for two other soft clays (Parry and Wroth 1981).

Although the lateral stiffness of a soft clay, as shown above, will normally be less than the vertical stiffness under compressive effective stresses in both directions, a complication arises in undrained triaxial compression tests where there is an increase in axial effective stress and, because of the positive pore pressure change, a decrease in radial effective stress. The stiffness measured under a decreasing effective stress may, in some soft clays, exceed that under increasing effective stress. If this increase is by a ratio of two to one, than a typical specimen with a two to one vertical stiffness under compressive stresses in both directions would behave as an isotropic soil giving, for example, a vertical stress path (V) on a q vs p′ plot as shown in Figure 4a. In a horizontal specimen the effect of the anisotropy would be exaggerated, as the decreasing effective stress would be partly in the direction of the maximum stiffness. Making simple assumptions (Parry and Wroth ibid) showed that radial to axial stiffness will be in the ratio of about three to one, giving the stress path (H) in Figure 4a. By similar reasoning expected stress paths for undrained triaxial extension tests can be determined. These are also shown in Figure 4a. In Figure 4b the results of such tests on Mucking Flats clay are shown (Wesley ibid) and it can be seen that there is a remarkable measure of agreement in the patterns of the theoretical and experimental stress paths. This type of behaviour does not seem to be universal for all soft clays, as witness the stress path for Fulford clay shown in Figure 2a.

In order to study the effects of both structural anisotropy and stress path induced anisotropy in a soft clay, undisturbed samples of soft clay were taken in a trench at a depth of 2m at Fulford (near York UK). Undisturbed samples 38mm in diameter were taken vertically, horizontally and at an angle of 45° to the horizontal. Two samples were taken at each inclination, one submitted initially to an undrained triaxial compression test at its natural moisture content and the other to an undrained extension test. Multi-stage consolidation undrained tests with pore pressure measurement were then carried out on each specimen at increasing cell pressures to measure the effective stress strength parameters c', ϕ'. The undrained shear strength in compression of the vertical specimen at its natural moisture content $c_u(CV)$ was 14 kPa. The relative undrained strengths of the other specimens are given below :

	cu/cu(CV) V	I	H
Compression	1.0	0.70	0.83
Extension	0.70	0.61	0.72

It can be seen from these results that :

(a) The inclined specimens (I) gave the lowest strength in both compression and extension. A similar trend to this has been reported for San Francisco Bay mud (Duncan and Seed 1966). Other workers have shown a progressive decrease in undrained strength from the vertical to the horizontal (e. g. Lo 1965 and Delory and Lai 1971 for Wellan clay and Wesley 1975 for Mucking Flats clay).

(b) The undrained strength in extension is 0.70 times the compressive strength for vertical samples and 0.87 times the compressive strength for inclined and horizontal samples.

(c) The undrained strength for horizontal samples is less than for vertical samples in compression, but slightly higher in extension.

(d) Depending on stress path and sample orientation the measured undrained shear strengths ranged from 8.5 kPa to 14 kPa, clearly illustrating that there is no unique undrained shear strength for clay.

Values of c′, ϕ' from the consolidated undrained tests on Fulford clay are given below :

	Compression		Extension	
	c′ kPa	ϕ'	c′ kPa	ϕ'
Vertical	8	26°	9	34°
Inclined	3	27°	8	34°
Horizontal	4	29°	11	35°

Sample orientation had comparatively little influence on ϕ' values, but with a slight tendency towards higher values for horizontal specimens in both compression and extension. On the other hand the stress path had a substantial influence on ϕ', with extension values exceeding compression values by 6° to 8° . It is difficult to reach any specific conclusions regarding c′ values as these are small and sensitive to slight sample variations and data interpretation ; but in all cases extension values exceeded compression values. These differences may have been influenced by the fact that small effective stress tensile strengths of up to 8 kPa were measured in these tests (Parry and Nadarajah 1974).

7. GAULT CLAY

Gault clay is one of a series of stiff heavily overconsolidated clays occurring in South-east England. In the Cambridge area it has a maximum depth of about 30 metres and overlies very compact Greensand. It is thought to have been overlain in the past by a maximum of about 520 metres of sediments. The clay is strongly fissured, particularly in the top five to ten metres depth. Watertables as high as 1 to 2 metres below ground level are often measured in this clay, but this probably reflects water in the fissures rather than in the intact lumps of clay. Although apparent K_o values as high as 5.0 have been deduced from self-boring pressuremeter tests, the true values are probably in the range 1.5 to 2.5.

Measured values of saturation on undisturbed tube samples are usually 96% to 98%, but occasionally dropping to 93% near the surface of a profile with a deep watertable. Index properties are in the ranges w_L=68% to 95%, w_P=26% to 33% and clay content 50% to 65%, mainly smectite with illite and kaolinite. Undrained triaxial compression tests in the laboratory typically show undisturbed shear strengths of 100 kPa or higher, while effective stress strength parameters from undrained and drained triaxial compression tests are commonly c'=8 kPa to 20 kPa and ϕ'=21.5° to 24°

A profile of gault clay taken from a borehole put down to a depth of 20 metres is shown in Figure 5. An indication of the degree of anisotropy of the clay in-situ to a depth of nearly 8 metres is given by the plot in Figure 5 of undrained moduli against depth, from screw plate tests and pressuremeter tests. In both cases the moduli were calculated as if the soil had isotropic elastic properties, so the plotted values are weighted averages of the vertical and horizontal stiffnesses. However, the pressuremeter values will be most strongly influenced by the horizontal stiffness, whereas the screw plate values will be most strongly influenced by the vertical stiffness. The difference in the values to a depth of 3.5 metres is relatively small, probably because the watertable fluctuates within this depth imposing, at times, high (isotropic) negative pore pressures from capillary rise and plant extraction of water, whereas below 3.5 metres the horizontal stiffness is clearly substantially in excess of the vertical value, mainly as a result of K_o value well in excess of unity and probably a greater horizontal fabric stiffness as well.

A series of undrained triaxial compression and extension tests performed on undisturbed and remoulded specimens 38 mm by 76 mm to a depth of 20 metres, gave the strength shown in Figures 6a, b respectively. It can be seen that the remoulded strengths are generally higher than the undisturbed strengths, reflecting the influence of fissures on the latter. It can also be seen that extension values are less than compression values, both for undisturbed and remoulded specimens. On average, in both cases, undrained extension strengths are about 0.72 times the undrained compression strengths, which agrees closely with ratios predicted by the combined critical state Mohr-Coulomb concept.

It is also of interest to look at the stiffnesses of these specimens. In Figure 6c, d the deviator stress q at an axial strain of 0.5% is shown in compression and extension for, respectively, undisturbed and remoulded specimens. It can be seen that for the undisturbed specimens most of these q values in extension are less than in compression, whereas for the remoulded specimens the reverse is true. The conclusions to be drawn from this are that where the heavily overconsolidated soil is essentially isotropic in its structure, the stiffness in extension exceeds that in compression, but where the soil is behaving anisotropically in its natural state, this, together with the presence of fissures, leads to vertical stiffnesses in extension slightly less than the compression values.

Gault clay has been extensively used for compacted motorway embankments up to 10 metres high, thus creating an interest in the strength characteristics of this compacted clay. Under standard Proctor compaction optimum water content is 25% and dry density γ_d=15 kN/m^3. Anisotropy in compacted clays has been reported (Parcher 1965, Fourie 1991), the former measuring higher lateral than vertical swelling in free swelling tests while the latter measured horizontal swelling pressures equal to twice the vertical values.

After compacting block samples at Proctor compaction and w=28.6% to

28.8%, triaxial test specimens 38 mm dia. by 76 mm long were trimmed from the block at various inclinations from vertical to horizontal. These were submitted to undrained compression and extension tests, the former performed unconfined and the latter at a cell pressure of 350 kPa. In addition, a number of undrained compression tests were carried out on vertical specimens at different cell pressures, giving the strength envelope shown in Figure 7a, for which $\phi_u=8^\circ$ and $c_u=83$ kPa.

The failure circle for the corresponding extension test is also shown and it can be seen that the undrained strength in extension is 0.75 times that in compression, again agreeing well with the critical state Mohr-Coulomb prediction. Corresponding initial axial modulus values were found to be 5.1 MPa in compression and 7.7 MPa in extension.

The undrained strengths of specimens at different orientations are shown in Figure 7b. For compression tests the minimum strength or 75 kPa was shown by a specimen oriented, at 42° to the vertical, which compared to strengths of 95 kPa for the vertical specimen and 92 kPa for the horizontal specimen. Extension tests showed a minimum strength of 73 kPa for a specimen oriented at 55° to the vertical, compared to strengths of 88 kPa for the vertical specimen and 80 kPa for the horizontal specimen. These results confirm that there is structural anisotropy in the compacted clay, which can influence mobilised undrained shear strengths by at least 20%. The compression and extension tests in Figure 7b cannot be directly related as the compression tests were performed unconfined and the extension tests at a cell pressure of 350 kPa.

8. OXFORD CLAY

Oxford clay ($w_L=70\%$, $w_p=25\%$), like the Gault clay, is one of the series of heavily overconsolidated clays outcropping in South-east England. It is an upper Jurassic deposit estimated to have had some 900 metres of overburden in the past (Smith 1978), as a result of which it is strongly laminated in the horizontal direction (Figure 8). Pieces of the clay defoliate like the leaves of a book on drying.

Investigations were carried out on brick pits near Bedford, having excavated slopes up to 20 metres high, to assess their possible suitability as cooling water reservoirs (Parry 1972). In order to design suitable excavated slopes for the reservoirs it was essential to determine the likely influence of the extreme structural anisotropy of the clay on its strength characteristics. Drained direct shear box tests were performed to measure peak and residual strengths of specimens with the shear plane vertical (V), horizontal (H) and at 30° to the horizontal (I). The results for tests with a normal applied stress of 350 kPa are shown in Figure 9.

It can be see that whereas the H and I tests gave almost identical result, the strengths were much higher for the V tests. The resulting peak (ϕ_p', c_p') and residual (ϕ_r', c_r') strength parameters are given below :

	ϕ_p'	c_p' kPa	ϕ_r'	c_r' kPa
V	33°	10	24°	0
I	21°	30	12.5°	20
H	21.5°	20	17°	0

As a consequence of these results stability analysis were performed using composite slip surfaces consisting of an "active" circular portion with peak strength parameters and a horizontal portion, daylighting at the base of the slope, with residual strength parameters.

9. CLAY SHALES

Piezometers installed some nine months after the failure of Waco Dam in Texas, showed high excess pore pressures to be still present in the underlying Pepper clay shale formation (Beene 1967). This prompted the US Army Waterways Experiment Station to initiate a research project into the strength and deformation characteristics of a number of clay shales occurring in the United States. Much of this programme was directed towards investigating pore pressures generated in triaxial test specimens under axial loading, which led to an investigation of the influence of anisotropy on the development of pore pressures in clay shales. The results of this work have been published in two reports (Parry 1976 ; Leavell, Peter and Townsend 1982).

A preliminary programme of consolidation tests and undrained triaxial was performed on Taylor clay shale (w_L=65%, w_P=21%, w=17%) from Laneport Damsite in Texas, sampled from a depth of about 17 metres. Consolidation-swelling observations, under isotropic stress changes, showed the ratios of axial strain ε_a to volumetric strain ε_v to be :

$\varepsilon_a / \varepsilon_v = 0.472$ in consolidation

$\varepsilon_a / \varepsilon_v = 0.536$ in swelling

If the clay shale had behaved isotropically these ratios would each have been equal to 0.33. Applying elastic theory to the above ratios and assuming Equation 8 to hold and ν_{rr}'=0.2, indicated the lateral stiffness of the clay shale to be 1.5 to 1.7 times the vertical stiffness which, when substituted into Equation 7, gives A values of 0.48 to 0.52.

An undrained triaxial test was performed on specimen of Taylor clay shale 36 mm dia by 76 mm long, originally at equilibrium under cell pressure of 827 kPa and a back pressure of 551 kPa. An increment of 207 kPa in lateral pressure was then applied, holding the axial total stress constant, followed by an increment of 207 kPa in axial stress, holding the lateral total stress constant. This sequence was then repeated. Under these small stress increments, which were much less than the failure deviator stress of 3.1 MPa, the clay shale behaved elastically and reversibly. The measured A values ranged from 0.55 to 0.6 and averaged 0.58, which, when substituted into Equation 7, gives $n' = 2.0$. This is probably a more accurate measure of n' than that obtained from the direct strain measurements under isotropic consolidation and swelling, which were very sensitive to small errors.

A programme similar to the above was performed on four more clay shales (Leavell, Peters and Townsend ibid), having the following characteristics :

Clay Shale	Location	w_L %	w_P %	w %
Bearpaw	Montana	58	24	17
Kincaid	Texas	85	20	19
Pierre	Colorado	58	18	23
Quivira	Kansas	51	24	15

A plot of measured radial strain against axial strain for smal isotropic consolidation stress increments on Bearpaw clay shale is shown in Figure 10, and a plot of undrained effective stress path in Figure 11. It can be seen in Figure 10 that the ratio of axial to radial strain is 2.4. Similarly, measured ratios for all four clay shales are given below, together with pore pressure A values predicted from these measurements, and the directly measured A values. Values of n'

deduced from the measured strains and from the measured A values are also given :

Clay Shale	ε_a / ε_r	n′ from strains	A predicted	A measured	n′ from A
Bearpaw	2.4	1.8	0.54	0.60	2.3
Kincaid	2.5	1.9	0.55	0.53	1.8
Pierre	3.0	2.1	0.58	0.57	2.0
Quivira	4.0	2.4	0.62	0.70	3.3

It is again clear from these figures that the ratio of radial to axial stiffness for these clay shales is consistently around 2.0. The high figure of 3.3 deduced for Quivira clay shale from measured A values is suspect because wide variations occurred in measured A values, ranging from 0.47 to 0.87. Referring to Figure 11 the initial slope of the stress path is -5.5 for Bearpaw clay shale, giving n′ =1.7 assuming ν_{rr}' and adopting Equation 8.

10. CONCLSIONS

1. All soils and soft rocks exhibit anisotropic properties to some degree, which may be either fabric or stress related or both.

2. Combining the critical state concept and the Mohr-Coulomb failure criteria leads to the prediction that undrained shear strengths for clays in triaxial extension will be commonly in the range 0.7 to 0.8, which is found to be the case for many natural, laboratory prepared and compacted clay.

3. Samples taken at different orientations in one-dimensionally consolidated clays exhibit different undrained strengths. In undisturbed Fulford clay inclined

samples showed strengths less than either the vertical or horizontal samples. This was so for both compression and extension tests. It was also found to be true for laboratory compacted specimens of Gault clay.

4. In laboratory prepared, one-dimensionally consolidated kaolin, and in undisturbed soft clays the effective stress angle of shearing resistance was found to be several degrees higher in triaxial extension than in compression.

5. Drained shear box tests on highly anisotropic Oxford clay showed effective stress angles of shearing resistance, both peak and residual, to be much higher for specimens with the shear plane oriented vertically with respect to their natural state than for those with the shear planes oriented horizontally, or inclined to the horizontal.

6. As predicted by elastic theory, undrained effective stress paths, and hence pore pressure magnitudes, can be strongly influenced by anisotropic stiffness in a clay or clay shale specimen. Pore pressure predictions using elastic theory may need to take account of different stiffnesses under increasing and decreasing effective stress.

7. In undisturbed gault clay, which was heavily overconsolidated, the undrained moduli measured in triaxial compression were higher than in extension ; but th same samples when remoulded showed higher values in extension than to compression. This behaviour may be attributed, at least partly, to fissures in the natural clay.

8. Clay shales from a number of locations in the United States showed, under isotropic consolidation and in undrained triaxial tests, lateral effective stress stiffnesses equal to about twice the vertical values.

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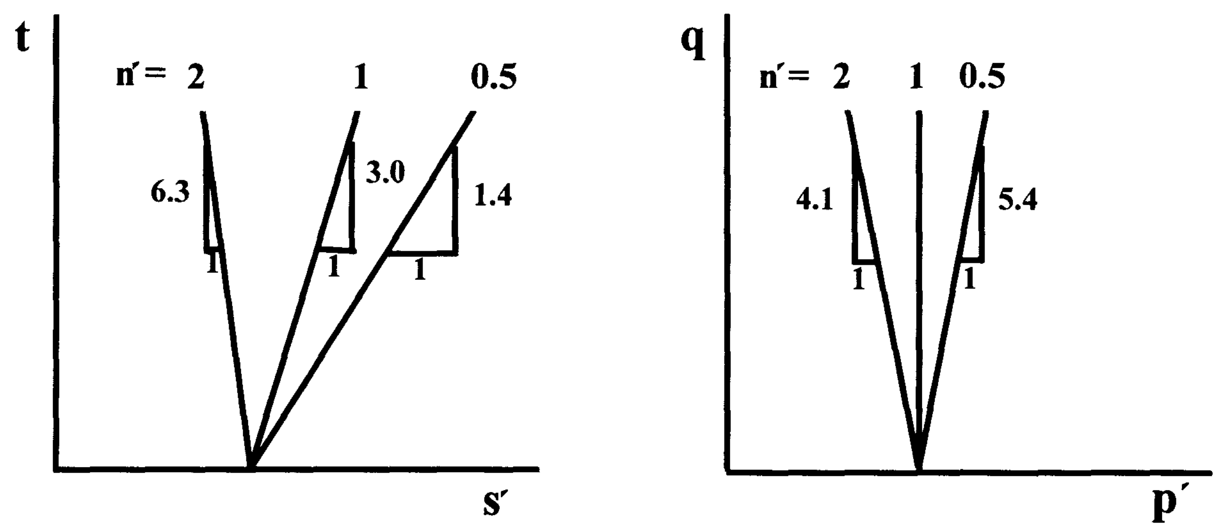


Fig. 1 : Elastic stress paths assuming Eq. (8) and $\nu'_{rr} = 1.2$

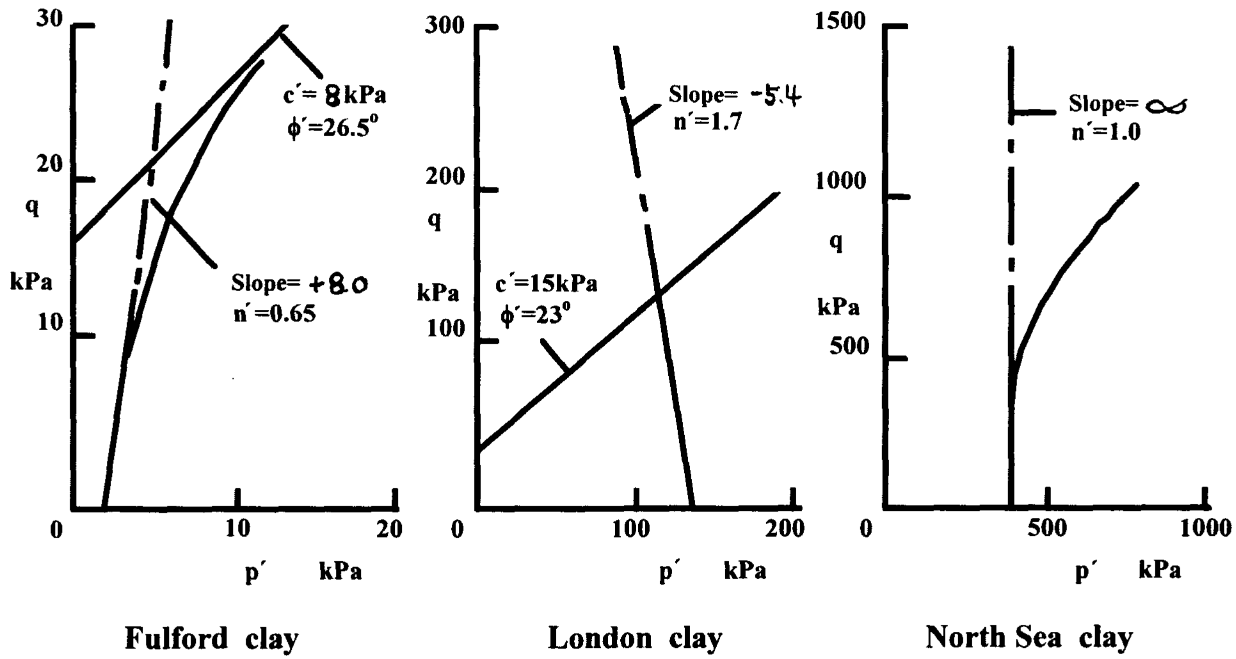
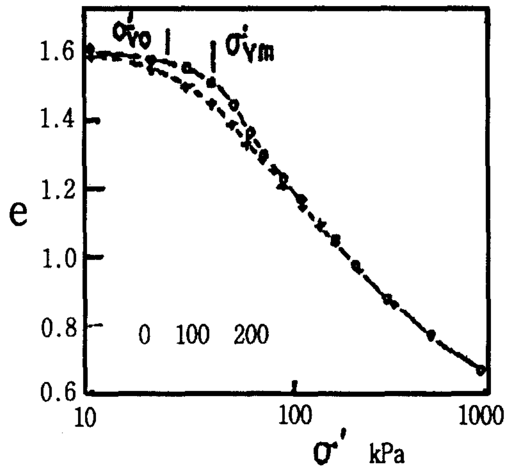
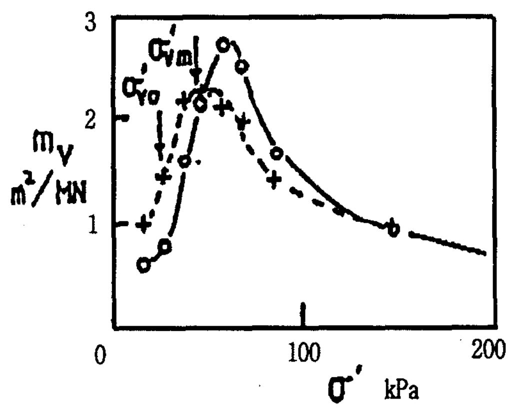


Fig. 2 : Undrained effective stress paths for three soils

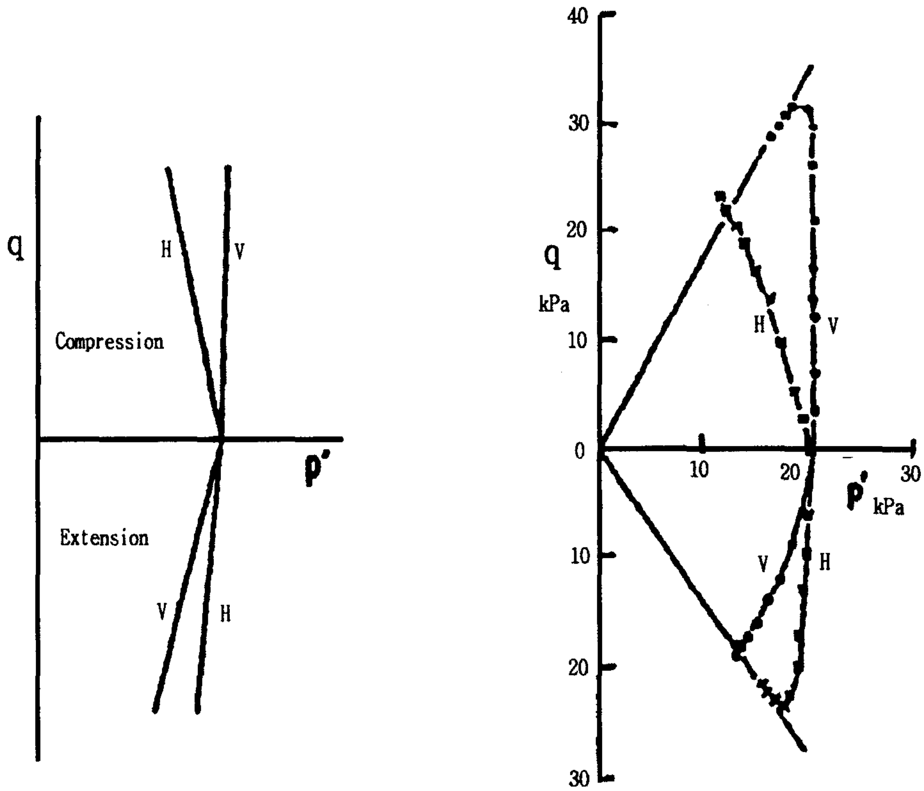


(a)



(b)

Fig. 3 : Consolidation characteristics of vertical and horizontal specimens of Mucking Flats clay (after Wesley 1975)



(a) (b)

Fig. 4 : showing (a) Theoretical undrained effective stress paths for a soft clay

(b) corresponding stress paths for Mucking Flates clays (after Wesley 1975)

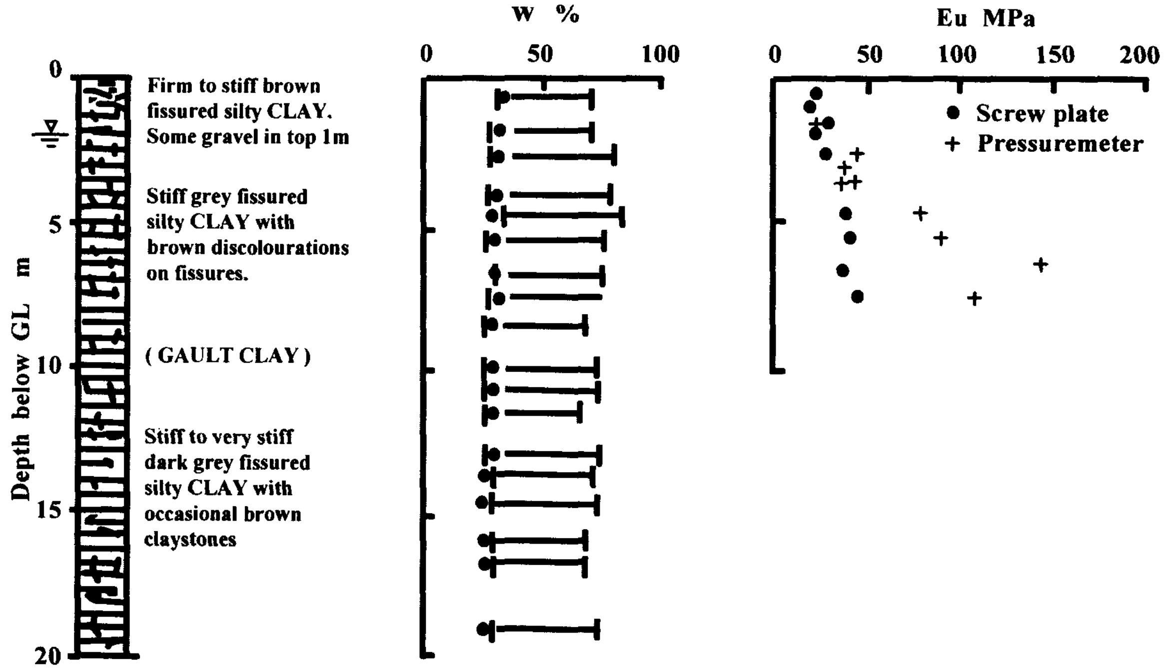


Fig. 5 : Gault Clay profile

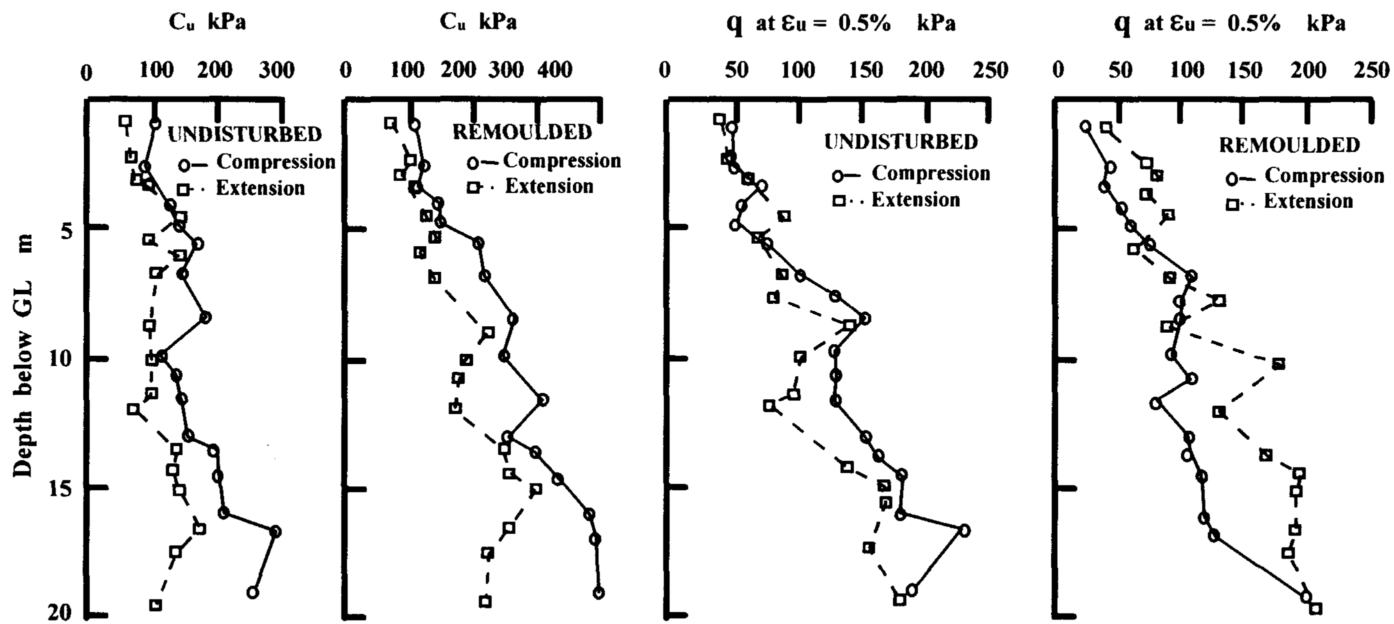
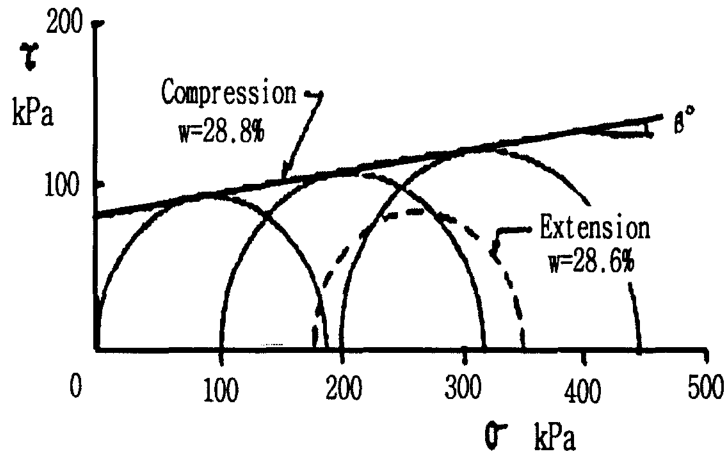
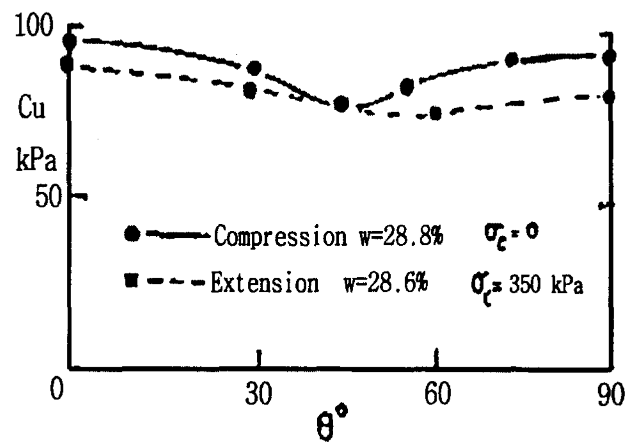


Fig. 6 : Measured undrained strength and stiffnesses of Gault clay specimens



(a)



(b)

Fig. 7 : Undrained triaxial tests on compacted Gault clay

Fig. 8 : Oxford Clay

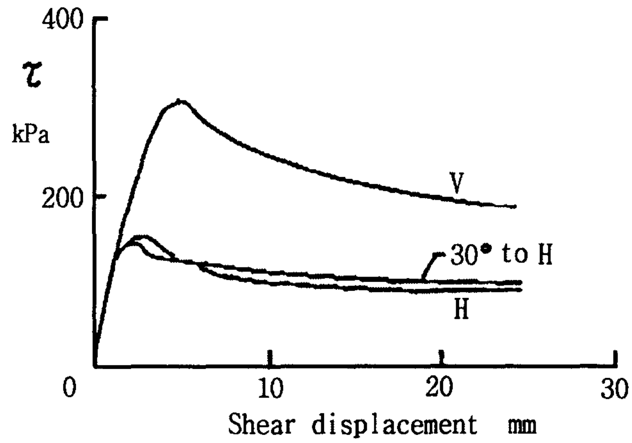


Fig. 9 : Shear box tests on Oxford clay

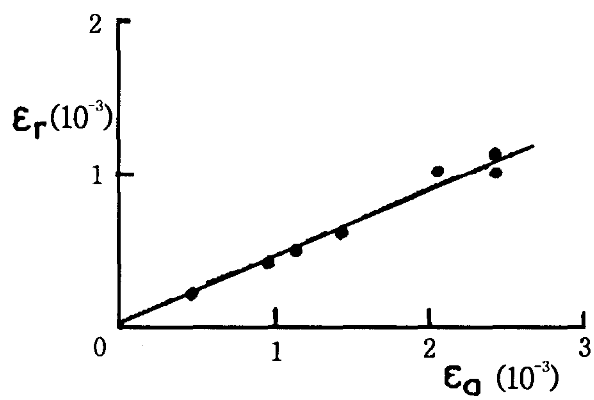


Fig. 10 : Radial and axial strains for Bearpaw clay shale under isotropic consolidation

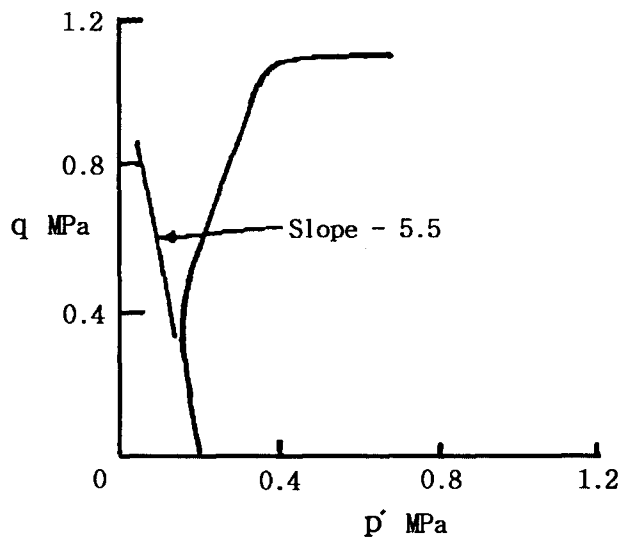


Fig. 11 : Undrained effective stress path for Bearpaw clay shalc