The role of fabric in evaluating the failure mode of the stiffened Bringelly shale

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Abstract: A series of stress path tests have been carried out for natural clay shale specimens from a geological formation in the Sydney basin, Australia. The specimens have been reloaded to a range of stress levels in a triaxial cell, and subjected to standard drained compression tests at a constant strain rate until approaching failure. The test results of the natural shale are compared to the results of tests on the same material that has been reconstituted and compressed to different porosities. The results indicate that at effective confining stresses <1000kPa, the reconstituted material behaves like other clays with an ultimate frictional strength identical to its residual strength, with a value of $\phi' = 28^o$. At higher stress levels, the ultimate friction angle of the reconstituted material is reduced owing to the fabric created by the high stress, which, in turn, leads to a reduction in the strength.

To investigate the effects of cementation and de-structuring, the behaviour of the reconstituted and natural materials have been compared. It is found that the strengths of the natural and reconstituted specimens (at the same void ratio) are similar, with both showing friction angles significantly less than the reconstituted material at higher void ratio. This paper presents some results from the reconstituted material showing the important influence of porosity, and hence confining stress level, on the observed frictional response. Data from the natural samples are then compared with the reconstituted samples to show the relatively minor influence of structure, which is consistent with micro-structural observations.

Résumé: Un feuilleton de tests de chemins de tension a été exécuté pour les spécimens de shale d'argile naturels d'une formation géologique dans le bassin de Sydney, Australie. Les spécimens ont été rechargés à une gamme de niveaux de tension dans une cellule triaxiale, et exposé à la norme a drainé les tests de compression à un taux de tension constant jusqu'à ce qu'approche l'échec. Les résultats de test du shale naturel sont comparés aux résultats de tests sur le matériel pareil reconstitué et serré aux porosités différentes. Les données indiquent qu'à limiter les tensions efficaces <1000kPa, le matériel reconstitué se comporte comme les autres argiles avec une force de friction ultime identique à sa force résiduelle donnée par $\phi' = 28^o$. Aux plus hauts niveaux de tension, l'angle de friction ultime du matériel reconstitué est réduit dû au tissu créé par l'haute tension, qui, dans le virage, mene à une réduction dans la force. Pour examiner les effets de cimentation et de-structurer, le comportement des matériels reconstitué et naturels a été comparé. Il est trouvé que les forces des spécimens naturels et reconstitué (à la proportion vide pareille) sont similaire, avec les deux friction de démonstration incline significativement moins que le matériel reconstitué à la plus haute proportion vide. Ce papier présente quelques-uns résulte du matériel reconstitué montrant l'influence importante de porosité, et limitant donc la tension nivelle, sur la réponse de friction observée. Les données des échantillons naturels alors comparé aux échantillons reconstitué pour montrer l'influence relativement mineure de structure, qui est conforme aux observations micro-structuraux.

Keywords: clay minerals, microcracks, shale, shear strength, saturation, triaxial tests.

INTRODUCTION

As the urban sprawl of the Sydney metropolitan area reaches westward, much of the new residential, commercial and industrial development is founded on the Bringelly shale formation, part of a major geological group of Late Triassic period known as the Wianamatta Group. The formation is composed of different lithologies with a maximum thickness of 257m. Bringelly Shale has a very low porosity and average unconfined compressive strengths of about 25 MPa. However, there is little evidence of induration and only apparently weak bonding due to recrystallisation of mica at particle contacts. It also shows a tendency to swell and can disintegrate on immersion in water. The shale is believed to have been deposited in an alluvial flood basin (Herbert, 1979) resulting in a range of facies, the majority being comprised of claystone-siltstone, with significant components of laminite and sandstone. This paper is concerned with the claystone-siltstone material which comprises 70% of the shale and has been found to be of similar mineralogy and particle size distribution across the Sydney Basin (William & Airey, 2004). On average the shale is composed of 55% clay minerals, 40% quartz, 3% siderite and 2% other minerals. The clay minerals consist of mixed layer clays, illite and kaolinite.

The high unconfined strength, coupled with low cementation and water sensitivity have made it difficult to obtain core samples of Bringelly Shale (William & Airey, 1999a), and even when suitable core specimens have been taken the extent to which they are representative of the shale mass is difficult to assess. To further investigate the behaviour of the shale a series of tests on reconstituted samples of crushed shale have been performed. These tests enable lower bound values for strength and stiffness to be determined. In this paper tests on reconstituted soil specimens at conventional soil mechanics stress levels and at elevated stress levels are presented. The behaviours of these specimens at different porosities are compared with the behaviour of intact shale. This paper focuses on the significance of the internal structure in influencing the stiffness and strength of Bringelly shale using available data on the mineralogy, micro-structure, swelling, stiffness and strength of the shale.

The results are presented within a critical state framework, and the relevance of this framework to natural shale is considered. At the pressures considered in conventional soil mechanics critical state models provide reasonable predictions, however their applicability to soft rocks has been debated. Recent developments have enabled models to account for structure and fabric due to bonding and particle arrangement, however, their applicability to soft rocks requires further verification.

TESTING ARRANGEMENT

The triaxial test arrangement is shown schematically in Figure 1. Internal instrumentation was used in some tests at confining stresses < 1000 kPa to enable the soil stiffness to be more accurately determined. This included both on sample measurements using Hall effect transducers (HET) to measure axial and radial strains and equipment to measure the travel time of shear waves to estimate the shear modulus, G. In all tests an external load cell and an external LVDT were used to measure the axial force and deformation of the specimen, and two digital GDS controllers were used to supply the cell and back pressures and measure volume changes. All data have been logged automatically by a *PC* using a 16 bit *A/D* plug in card. A computer program written in Visual Basic was used to fully control the test.

The samples tested had diameters ranging from 38 *mm* to 51 *mm* and effective confining stresses varying from a minimum of *20 kPa* to a maximum of *60MPa*. Due to the wide range of the hydrostatic pressure used, it was necessary to employ different sets of triaxial testing equipment to suit the pressure and the subsequent deviator load during the test. However, in all tests the equipment was schematically similar to that shown in Figure 1.

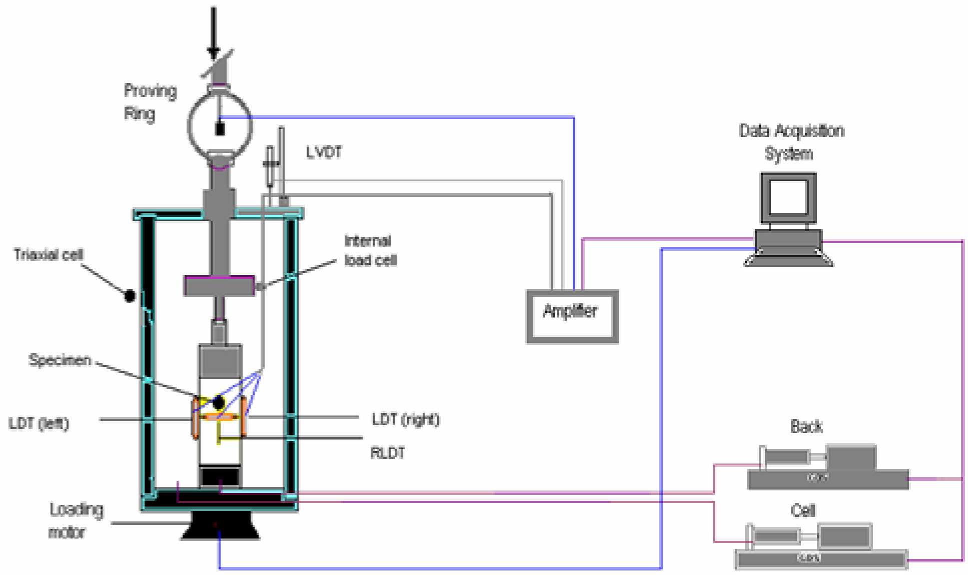


Figure 1. General layout of the testing arrangement

TESTING PROGRAM

Fifty five triaxial compression tests were performed on reconstituted and core specimens. In order to prepare reconstituted specimens block samples of shale were pulverized to break down any particle agglomerations. The resulting material was mixed with water to form a slurry and compressed one-dimensionally in a cylindrical mould to form specimens for triaxial testing. After transfer to the triaxial cell specimens were saturated, isotropically consolidated and then sheared under drained or undrained conditions to establish the stress-strain-strength behaviour. Two series of triaxial compression tests were carried out on reconstituted samples. The first series was performed on specimens with a maximum confining pressure of *1000kPa* and will be referred to as the low pressure test series. The second series of tests was performed on specimens compressed to a confining pressure of *60MPa,* and will be referred to as the high pressure test series. For both series specimens were tested with a range of over-consolidation ratios.

Natural core samples were obtained from various locations using conventional diamond drilling using water flush and from block samples cored in the laboratory. Natural shale specimens were tested at their sampled moisture content and after saturation at a range of confining pressures up to 60 *MPa*.

ISOTROPIC COMPRESSION

Typical isotropic compression responses are shown in Figure 2 for reconstituted specimens. Representative data are shown in $e : \ln p'$ space. It is often assumed that the isotropic normal compression line (INCL) may be described by an equation of the form:

$$v = 1 + e = N - \lambda \ln p'$$

where v is the specific volume, λ is the slope of the compression line, and N is the specific volume at mean effective stress p' = 1 *kPa.* Similarly, the unload-reload response can be written as:

$$v = v_\kappa - \kappa \ln p'$$

where v_κ is a constant and κ is the slope. It has been suggested (Butterfield, 1979) that improved linearity in the isotropic response occurs when plotting logarithms of both v and p', but this was not observed for the reconstituted shale, which gave non-linear responses in all the various representations investigated. However, it was found that the equations above gave a good representation of the data for stresses from 100 *kPa* to 10,000 *kPa*. It was also found that the *INCL* could be described well for the range 10 *MPa* $\leq p' \leq$ 60 *MPa* by a hyperbolic function given by

$$e = \frac{A}{(\ln p' - B)}$$

where A and B are constants that can be chosen so that *INCL* is continuous with no change of slope at *p'=10,000 kPa*. No data on the *INCL* were available for pressures > 60 *MPa*. It may be noted that responses of slurry specimens were repeatable with only a small range of v for a given p' as shown in Figure 2. The parameters λ, N, and κ are constant within the range of p' from 100 *kPa* to10,000 *kPa* and are given by 0.07, 1.85, and 0.009 respectively. A and B are constants and equal to 1.4 and 1.2 respectively. It may also be noted that the value of κ reduces from 0.009 at 1000 *kPa* to 0.0013 at 60 *MPa*.

The isotropic response has been reported to be linear over a wide range of stress for many other clays and clay shale (Skempton, 1970; Morgenstern, 1977; and Yang et al. 2004). However, data on the compression response at high stress is very limited. For example, Skempton's data is based on interpretation of void ratio with depth from in-situ deposits. Yang et al. (2004) reported the slope of the void ratio, logarithm of vertical effective stress response for clay shales from the North Sea, showing no reduction in compression coefficient between 10 *MPa* and 40 *MPa* even though the void ratio was significantly reduced. It is thus unclear whether the curvature in the $e : \ln p'$ response from this study is a consequence of the high stress level or the unusually low void ratios achieved for this material. It is shown below that the low void ratio has required substantial particle alignment and this limits further compression, and hence it is believed that the low void ratio has a significant effect on the flattening of the compression response for this material.

The saturation of natural specimens was performed under an effective stress of 600 *kPa* as this stress was sufficient to prevent disintegration of the specimens, although not to prevent swelling. The responses of the saturated core specimens during isotropic swelling and compression after saturation at 600 *kPa* are shown in Figure 3 together with the response of the reconstituted material.

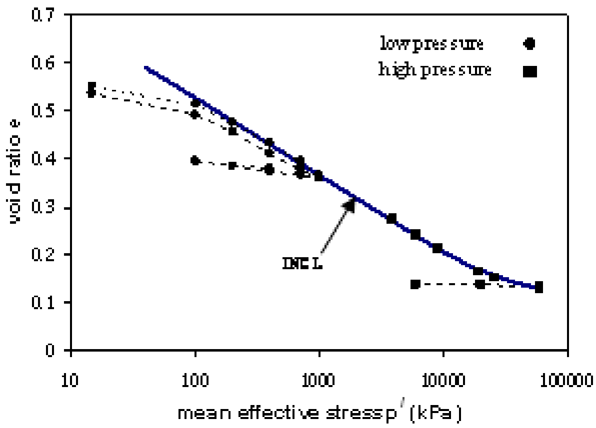


Figure 2. Isotropic compression of reconstituted Bringelly shale

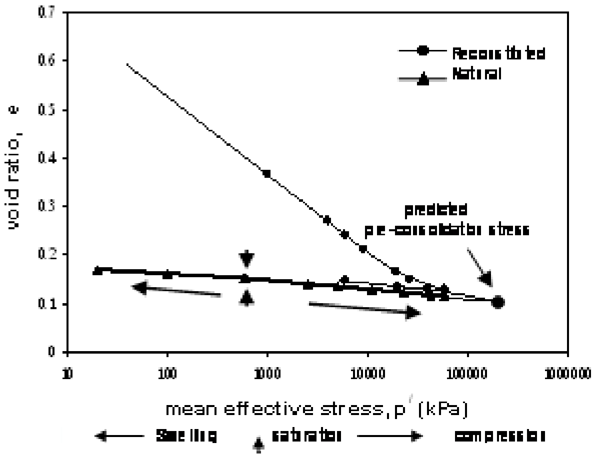
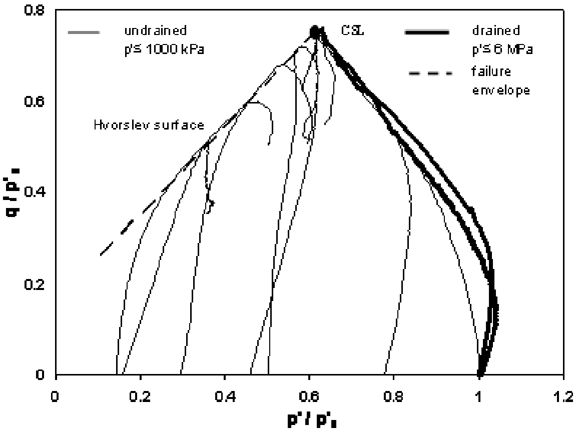


Figure 3. Isotropic compression response of natural and reconstituted material

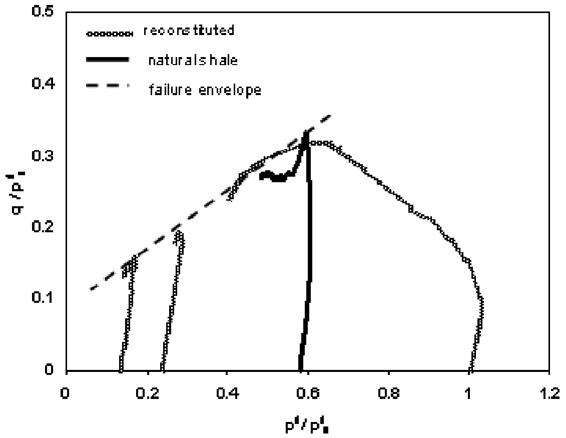
Comparison of the isotropic responses of the reconstituted material and the natural shale indicate an effective confining stress of 60 *MPa* produces a void ratio of 0.15 and 0.10 for the reconstituted and natural shale respectively. These values are equivalent to the average porosity of ~10% previously measured for the natural shale (William, 1999b). As the shale appears to be only weakly cemented and not to have undergone significant diagenetic changes, the compression responses suggest that the natural shale has experienced a maximum effective stress of >60 *MPa* as indicated on Figure 3. It can be seen that the void ratio increases from 0.10 at a maximum effective stress of p'_c = 60 *MPa* to 0.18 at a minimum effective stress, p'_c = 20 *kPa*, and that the slope of the compression response of the natural shale is similar to that of the reconstituted material allowed to swell from a maximum stress of 60 MPa.

TRIAXIAL COMPRESSION TESTS

A series of standard drained and undrained triaxial tests were performed on specimens with over-consolidated ratios of up to 10. The data was presented in two graphs of deviator stress, q (= σ'_1- σ'_3) versus mean effective stress, p' (= (σ'_1 + 2 σ'_3) / 3) normalised by p'_e, the pressure on the INCL at the same specific volume.. One for specimens tested to the maximum effective confining stress of ≤ 6 MPa (Fig. 4a) and the other for those tested to the maximum effective confining stress of 60 MPa (Fig.4b).



(a)



(b)

Figure 4. Normalised responses at different maximum stress levels

The slope of the strength envelope (Fig. 5) for specimens that had experienced a maximum effective stress of 1 *MPa* was significantly higher than for specimens that had a maximum effective stress of 60,000 *kPa.* For the low pressure tests an ultimate, or critical state, stress ratio of M = 1.14, corresponding to an effective friction angle of ϕ' = 28.5° was obtained. The specimens which had been highly compressed to a maximum effective confining stress of 60 *MPa* show a much lower ultimate stress ratio, with M = *0.67* and a cooresponding lower effective friction angle of 16°.

This different behaviour was unexpected as reconstituted normally consolidated specimens are expected to behave similarly once allowance is made for differences in confining stress level (e.g. Atkinson and Bransby, 1978). Figure 6 shows the change in the deviator stress (normalised by effective consolidation pressure), axial strain responses as confining stress increases for three normally consolidated specimens subjected to drained shearing. For effective confining stresses between 0.4 *MPa* and 6 *MPa* the normalised behaviour is similar, as observed for many clays and consistent with the assumptions of critical state soil mechanics.

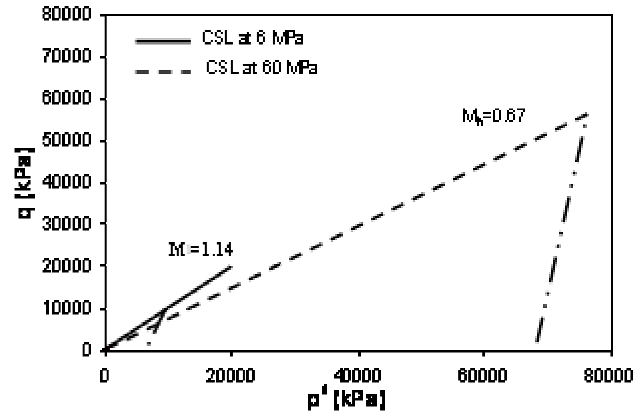


Figure 5. Strength envelopes of low (M_l) and high (M_h) of pressure test series

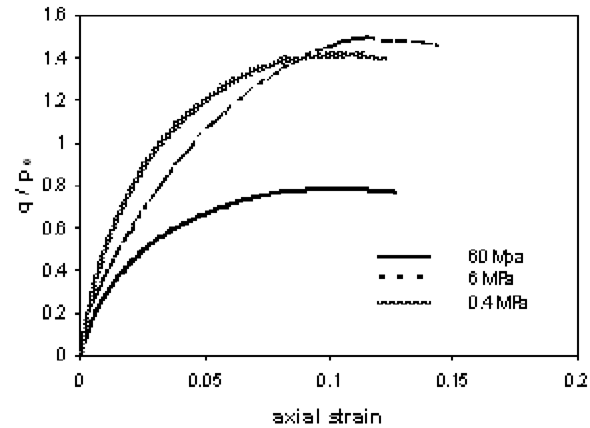


Figure 6. Normalised stress-strain responses reconstituted drained specimens

The response of the specimen with $p_c' = 60$ *MPa* lies significantly below the responses of the specimens that have experienced a maximum stress of ≤ 6 *MPa*. However, it still shows a ductile response with compressive volume strains and reaches a peak at relatively large axial strain of 10% as do the other normally consolidated specimens.

It is an assumption of many simple critical state soil mechanics models (e.g. Atkinson and Bransby, 1978) that sections through deviator stress, *q,* mean effective stress*,* p' , *e* space at constant void ratio, *e* are similar. This would imply that the curves in Figure 6 should all be identical. The data suggest that some of the critical state concepts break down at low void ratio, presumably because of the increasing importance of fabric in controlling the material response.

Tests were also performed on fresh core specimens from two sites at effective confining stresses that varied from zero to 60 *MPa*. The peak strengths from typical tests for saturated and unsaturated conditions are shown in Figure 7. These tests indicate the very dramatic effect of saturating the specimens on the strength. At a confining stress of 600 *kPa* the peak deviator stress drops from 15 *MPa* to 5 *MPa* as the material is saturated. The influence of saturation was more pronounced when the effective confining stress was reduced to 20 *kPa*, with the peak deviator stress of the saturated specimen dropping to 0.8 *MPa*, while the unsaturated shale strength was approximately 15 *MPa*. Based on the data shown in Figure 7, two possible factors can be suggested that may contribute to the large drop in strength following saturation, one is the reduction in effective stress that occurs because of the removal of suctions, leading to reduction in frictional strength, and the other is that the strains associated with saturation and effective stress reduction cause the material cementing the rock to break down.

The influence of saturation was also investigated through comparison with saturated reconstituted specimens at an effective confining stress, p_c’ of 6 *MPa* (Fig. 8). Figure 8 shows the deviator stress, axial strain responses for a reconstituted specimen that had been pre-consolidated to 60 *MPa*, and for the intact shale at its natural moisture content and after saturation. At this confining pressure the difference between the saturated and unsaturated shale is small, and surprisingly the stiffness and strength of the intact shale are only slightly greater than for the reconstituted material. This is surprising because standard rock mechanics procedures classified the shale as a strong rock. It also shows that almost linear elastic responses are observed for deviator stresses up to 6 *MPa* and that all specimens reached a peak (*q*) at an axial strain of about 2% and this was followed by a reduction in the deviator stress with further strain beyond the peak.

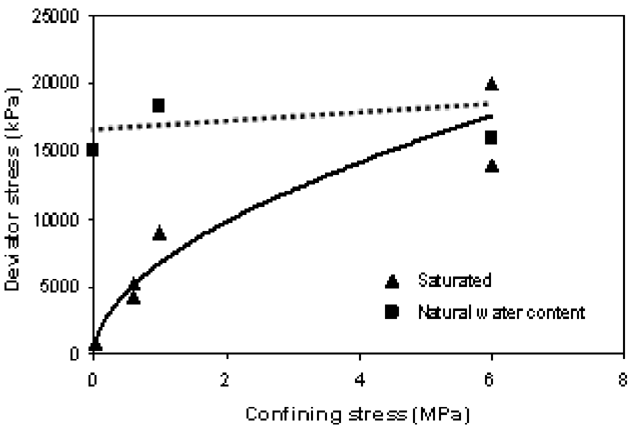


Figure 7. Influence of saturation on failure of the natural shale

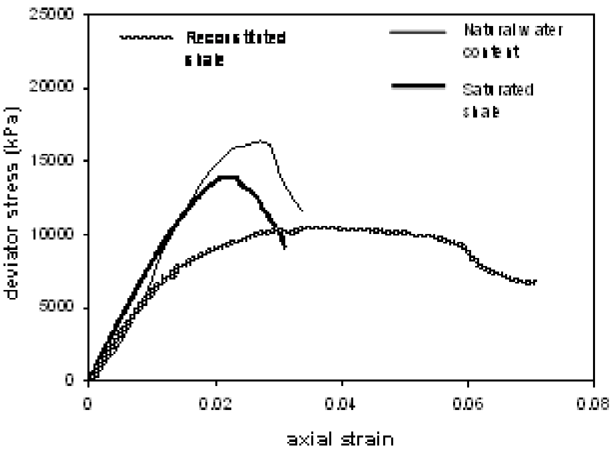


Figure 8. Effect of saturation on natural rock a *6 MPa* effective stress

Monitoring of the specimen deformation during saturation and effective stress reduction showed that both lead to significant strains and these are likely to have led to the breakdown of cementation if it were present. This result, however, does not help in determining whether the low strength of the saturated shale is simply due to the reduction in effective stress, or it is due to a combination of destructuring and effective stress reduction.

SHALE STIFFNESS

The stiffness of the reconstituted shale in drained and undrained conditions was determined from internal and external measurements during the triaxial tests. The influence of effective confining stress on the variation of the secant stiffness can be detected from Figure 9, which shows the stiffness normalised by the effective consolidation stress versus the axial strain. It can be seen that the plot shows that a large drop in normalised stiffness occurs as the confining stress increases. However, it is well known that the correlation between p_c’ and *E,* such as proposed by Kulhawy (1975), is not linear and has the form:

$$E = K' p_a \left(\frac{\sigma_3}{p_a} \right)^n$$

where *E* = Young’s modulus, *K* and *n* are constants and σ_3 = confining pressure

The exponent n was estimated from the relationship between p' and G_{max} determined from shear wave velocity measurements on reconstituted specimens for effective confining stresses up to 400 *kPa* as shown in Figure 10. This gave the relation between p' and G_{max} as:

$$G_{\max} = 6p'^{0.45}$$

where $G_{\max}$ is the shear modulus at very small strain

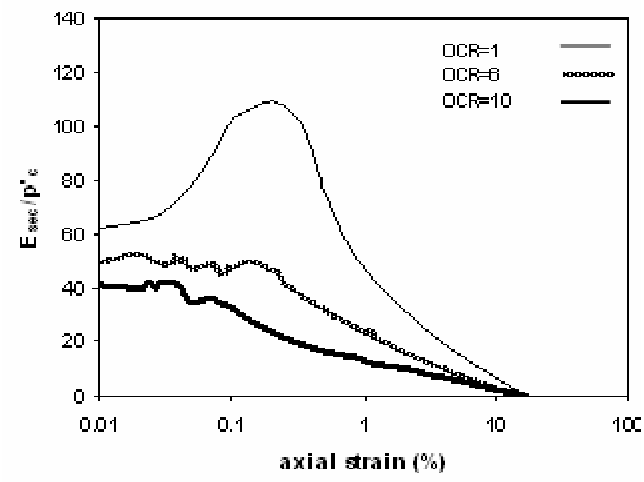


Figure 9. Drained secant modulus at low and high effective stresses

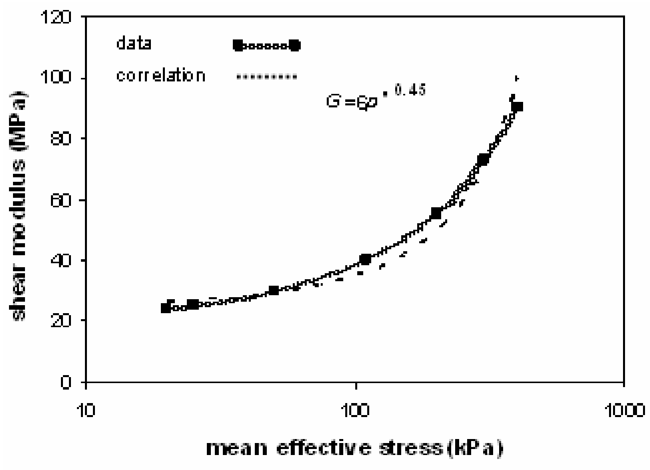


Figure 10. Influence of stress levels on the shear modulus

When the maximum value of the E at very small strains is divided by the $p'^{0.45}$, it gives normalised values of 430, 550, and 480 at 0.4 *MPa*, 6, *MPa*, and 60 *MPa* respectively. These values are close enough to bring the curves within a significantly small range of normalised stiffness and suggest that the above equation is valid for reconstituted specimens over the complete stress range of 0.4 *MPa* to 60 *MPa*.

The stiffness of the natural shale at zero confining pressure was determined for specimens at their natural water content from shear wave velocity measurements. Data from tests on saturated specimens were also analysed and the tangent modulus of elasticity, measured at 50% of the peak strength across a range of confining stresses between 0.02 to 60 *MPa,* determined. These values are not directly comparable with the small strain values determined from shear wave velocity, however, only external displacement measurements could be used to estimate the modulus of the material at high confining stresses. The data will have been influenced by end effects because of the difficulty of preparing specimens.

The influence of saturation on natural and reconstituted material has also been investigated using similar external measurements. Table 1 shows values of E_{50} determined from tests on intact and reconstituted Bringelly shale. The data shows that an increase in the effective confining stress is accompanied by an increase in the stiffness and strength of both natural and reconstituted shale in the saturated condition. Unsaturated shale, on the other hand, has shown an opposite trend in which the increase in the material stiffness occurs as a result of reducing the effective confining stress, and as noted above the strength is largely independent of confining stress. At 6 *MPa*, where direct comparison is possible, the saturated and unsaturated materials have shown similar stiffness. At 1 *MPa* effective stress, the saturated stiffness of the Bringelly shale is reduced to about 50% of that of the same material in unsaturated condition.

Table 1. Values of E_{50} for Bringelly shale

Effective confining stress (*MPa*)	**Bringelly shale Unsaturated (*MPa*)**	**Bringelly shale saturated (*MPa*)**	**Reconstituted void ratio ~ 0.15 (*MPa*)**
0	2300	-	-
0.02	-	45	-
0.6	-	250	-
1	1500	738	-
6	751	823	627
60	-	2800	1500

These observations are consistent with the weak cementation, and low siderite contents, in the Bringelly shale and also with the presence of mixed layer clay minerals which have potential for swelling and physico-chemical changes as moisture content changes. It is believed that cementation is relatively weak in this material, and that pore water suctions are largely responsible for the high strength and stiffness of the natural shale at low confining stresses. Some support for this suggestion is provided by the very large total suctions that have been measured (William&Airey, 2005). Further, as the confining stress increases the suctions will reduce and it is postulated the effective stress could remain approximately constant, accompanied by some loss of cementation so that stiffness reduces and strength stays constant.

DISCUSSION

Stiffness is sensitive to the presence of cementation and this can play an important role in assessing the nature of the cementing agent of the material (Atkinson et al. 1993). In this shale it is, however, difficult to separate the effects of cementation and saturation, as saturation results in sufficient strain to cause some breakdown of any cementation that may be present. It was observed that the stress, strain curves of saturated and unsaturated specimens at different effective confining stress tended to show different trends (e.g. Fig. 8), with the saturated tests giving a linear response and the unsaturated tests indicating increasing stiffness with strain. The initial stiffness is, however, not greatly affected by either the effective stress or the degree of saturation suggesting some cementation is present in the shale.

A related study, concerned with the behaviour of highly plastic intensely fissured clay shales from Italy, has been presented by Picarelli et al (1998, 2003). They showed that the normalised state boundary surface of the intact shale lies below the surface of the reconstituted material, as occurs for the highly compressed Bringelly shale. Picarelli et al. related this behaviour to the effects of fissuring in the natural soil and suggested that the mechanism of deformation could be described by the classical model used for fractured rocks, where deformation and strength are controlled by movements along joints and fissures. They also reported the insignificance of *OCR* in influencing the strength of the material. The low frictional resistance of the highly compressed reconstituted shale suggests that the fabric associated with the low porosity, created by the high stress, is also contributing to the reduced frictional strength and different deformation mechanisms. At low porosity there must be locally a high degree of alignment of the plate-like clay particles. It is possible that failure surfaces could develop that pass through regions where the particles are highly aligned.

The mechanism suggested is illustrated in Figure 11 and is identical to that proposed by Picarelli et al (1998) for their fissured shale. The effective friction angle is controlled by the interparticle friction angle between the particles, ϕ_μ , and the effective dilation angle, which will depend on the roughness of the failure surfaces. It is postulated here that this mechanism is controlling the behaviour of the low porosity reconstituted material even though fissures are not present. The natural Bringelly shale has a very low porosity, similar to that produced by the high stresses in this study.

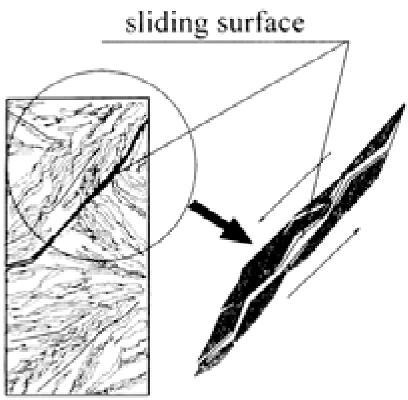


Figure 11. Mechanism of shear deformation and rupture (after Picarelli et al, 1998)

A comparison of the drained stress-strain responses of the reconstituted material and the intact shale at an effective stress of 6 *MPa* has revealed that the stiffness and strength of the natural shale are only slightly greater than the reconstituted material which is contradictory to the rock mechanics procedure that would classify the shale as a strong rock. In addition, at confining stresses of 6 *MPa* and above there is no difference in the ultimate friction angles of the reconstituted and natural material. At lower confining stress, the strength of the intact shale is a function of the saturation condition of the material.

Scanning electron micrographs have shown that the intact shale has highly aligned clay particles and this suggests a mechanism for the low strength and stiffness based on sliding between agglomerations of clay particles. *SEM* pictures were taken before and after shearing in an attempt to identify the fabric and mechanisms responsible for the low frictional strength in the compacted shale. Examples of images taken after shearing are shown in Figures 12 and 13.

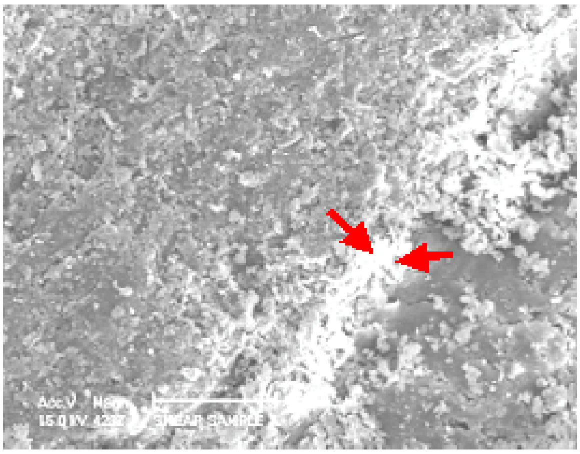


Figure 12. Shear plane of rock specimen at high stress (Magnified at 25x)

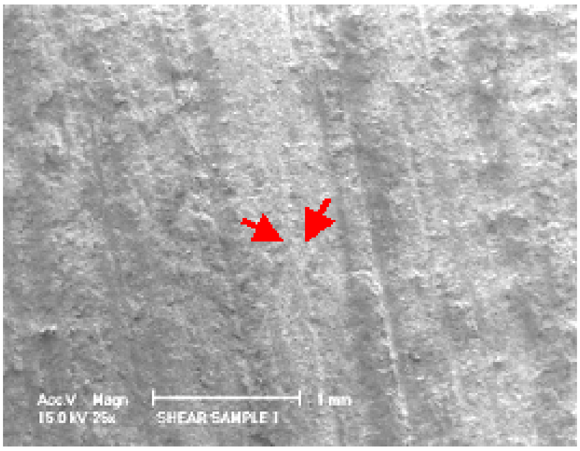


Figure 13. Parallel bands of clay platelets at high stress (Magnified at 1000x)

Figure 12 shows an overview of particle alignment for a specimen at 60 *MPa* as demonstrated by the parallel bands. When magnified, particle alignment along the failure plane is produced as indicated by the arrows (Fig. 13). The micrograph shows diagonally aligned parallel bands resulted from a strong alignment of the clay platelets due to the high stress level. This pattern was not clearly defined when the specimens were subjected to maximum effective stresses less than 1 MPa prior to shearing, and no parallel bands can be detected (Fig. 14). Micrographs were also taken of sections in the specimens that were free of shear bands (e.g. Fig. 15). The figure shows no particular pattern that might suggest particle movements in any one direction. This may affirm the heterogeneity of the material when subjected to high stresses.

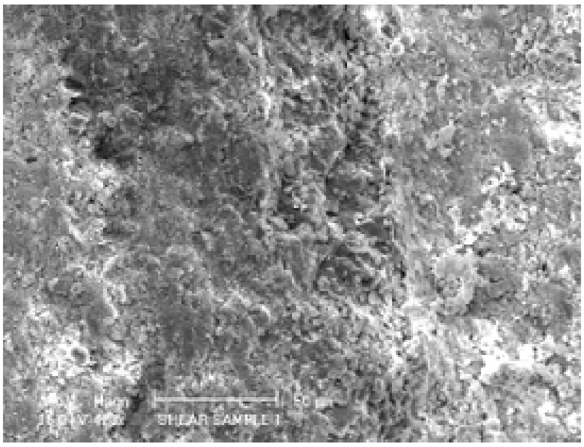


Figure 14. Undefined pattern of clay platelets at low stress (Magnified at 1000x)

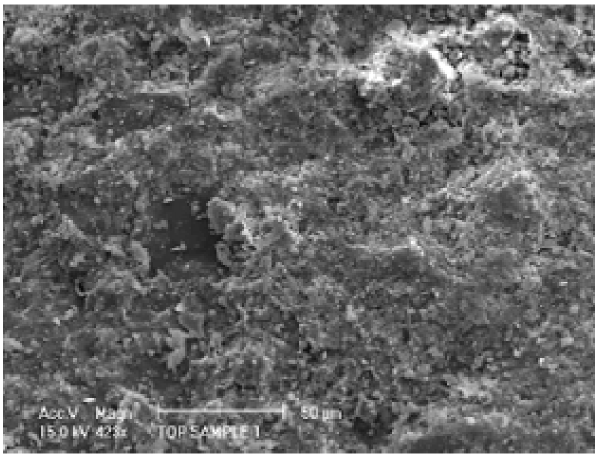


Figure 15. Rupture free section at 60 *Mpa* effective stress (Magnified at 1000x)

The poor alignment of the plate-like clay particles at a stress level of 6 *MPa* can be seen in Figure 16. This poorly identified pattern of alignment is also similar to that for specimens subjected to even lower stresses (Fig. 17). In order to obtain a wider exposure to figure, the specimen was tilted at different angles prior to scanning. It appears that no identifiable pattern of alignment can be observed at stresses ≤ 6 *MPa* prior to the triaxial compression of the natural shale specimens.

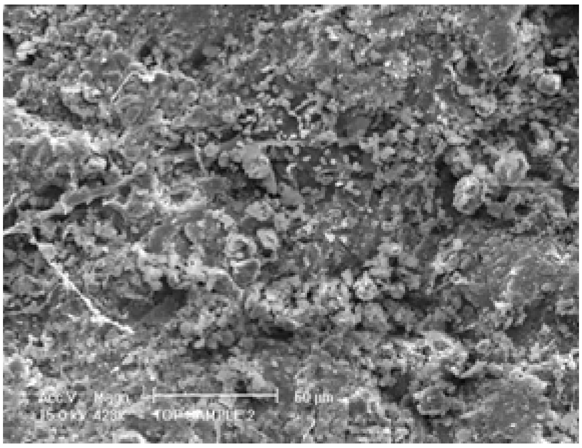
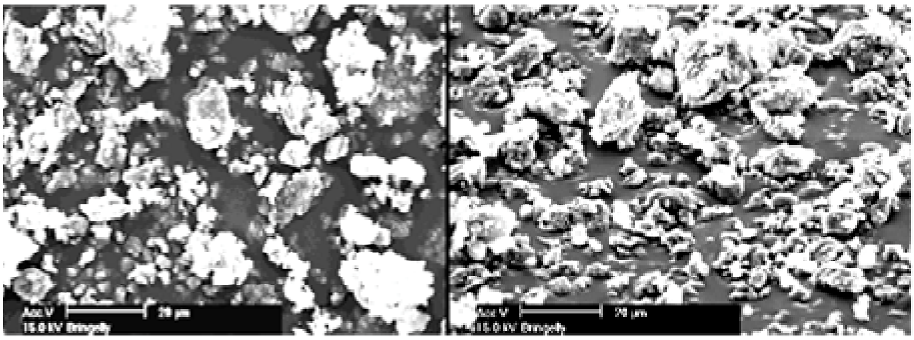


Figure 16. Photomicrograph of a specimen subjected to 6 MPa (Magnified at 1000x)



Specimen oriented at zero degree Specimen oriented at 45 degree

Figure 17. Stacking of platelets of natural shale prior to isotropic consolidation tests

CONCLUSIONS

Bringelly Shale has proven to be a difficult material to characterise due to its high strength at its natural moisture content, but when immersed in water it tends to disintegrate.

Reconstituted specimens were prepared to assess the extent of saturation and fabric in the natural shale after saturation. It was found that reconstituted specimens had to be compressed to the same density as the natural shale to reproduce the strength and stiffness of the saturated shale. When saturated the difference in stiffness between natural and reconstituted specimens was sufficiently small to indicate no cementation was present. However, very high stiffness was measured when the shale was at its natural moisture content suggesting that cementation is present.

The high compressive stresses needed to produce the low void ratio (high density) of the shale lead to fabric having an important role in controlling strength and possibly stiffness.

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