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Influence of Pressure on the Response of Reconstituted Bringelly Shale

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Summary: Bringelly shale is a major formation of the Wianamatta group that outcrops over a large area of Western Sydney. The shale is comprised predominantly of claystones and siltstones with occasional sandstone layers. It is highly compacted, weakly cemented, and contains significant amounts of swelling minerals. Because of difficulties in obtaining samples from conventional drilling methods tests have been performed on reconstituted samples compressed to the same density as the in-situ shale. The results of triaxial tests on the highly compressed specimens are compared with the behaviour of reconstituted specimens at lower density. It is found that the high stress results in changes in the soil response and reduction in the frictional strength.

INTRODUCTION

Shale, in general is one of the most problematic and least understood geological materials due to the wide variation in its engineering properties. Engineers have often encountered significant problems involving shales and other argillaceous rocks. Bringelly shale is a formation of the Wianamatta group a soft rock of Triassic age within the geological structure known as the Sydney Basin in New South Wales, Australia. In this group, Bringelly shale is the more problematic engineering material because it contains swelling clay minerals. Its geology and mineralogy have been the subject of considerable investigation (Chesnut, 1983, Herbert, 1979) because Bringelly shale is the main source of clay for brick making in the Sydney region. However, there is relatively little data on the engineering performance of Bringelly shale, and before the current study only a few test results had been published by Won (1985).

In the current study the engineering performance of the claystone/siltstone that comprises the majority of the Bringelly shale has been investigated. This investigation has involved characterisation of its microstructure and index physical and mechanical properties (William and Airey, 1999a, 1999b). Obtaining suitable samples for physical property characterisation has been difficult because the shale swells and disintegrates when placed in tap water. In consequence core recovery from conventional diamond core drilling with water flush is very poor, and it might also be expected that when larger core samples are obtained these will necessarily be of stronger material and possibly not representative of the rock mass. The swelling behaviour is a consequence of about 22% of the claystone/siltstone being comprised of swelling clay minerals. The Bringelly shale has a low porosity between 5% and 12% and unconfined compressive strengths of between 10MPa and 50MPa. The reason for these high UCS values is unclear as small lumps disintegrate rapidly when placed in water and microscope studies show the shale is only weakly cemented. There is some evidence of recrystallisation of mica at particle contacts, but there is no evidence of induration and only small amounts of siderite and organic matter that can act as cementing agents are present. Microscopic studies also show that the Bringelly shale contains numerous micro-cracks in the plane of the laminations. To further investigate the behaviour of the shale a series of tests on reconstituted samples of crushed shale have been performed. These tests will enable lower bound values for strength and stiffness to be determined, and because the bonding appears to be weak it is believed that reconstituted specimens of the shale could provide other useful information on the engineering behaviour.

To reproduce the low porosity of the natural shale reconstituted specimens have been subjected to high stress levels. The response of these specimens with low porosity is compared to the behaviour of the same material with higher porosity under stresses more typical of engineering soils. This paper is concerned with the different behaviours shown by the reconstituted material at different porosities, and the implications of the different behaviours for the interpretation of tests on intact shale.

MATERIALS

Figure 1 shows a geological map of the Sydney region indicating the area over which Bringelly shale is the upper rock type. The shale is of Triassic age and of lacustrine origin. Block samples of the rock were obtained from active quarries that provide material for brick making. The quarry locations are marked on Figure 1: M (Mulgoa), B (Badgerys Creek), K (Kemps Creek) and H (Horsley Park).

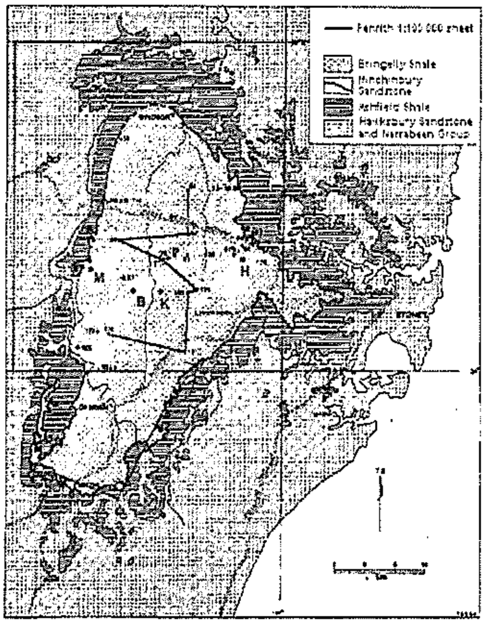

Figure 1. Location map of the study area

The mineralogy and index properties of the claystone/siltstone, the dominant rock type in the shale, are similar at all 4 locations (William and Airey, 1999a). To provide material for the tests, the block samples were broken down and crushed. The grain size distribution is shown in Figure 2, from which it can be seen that the contents of clay, silt, and sand are 55%, 41% and 4%, respectively. The crushed shale has a plasticity index of 10 and liquid limit of 30.

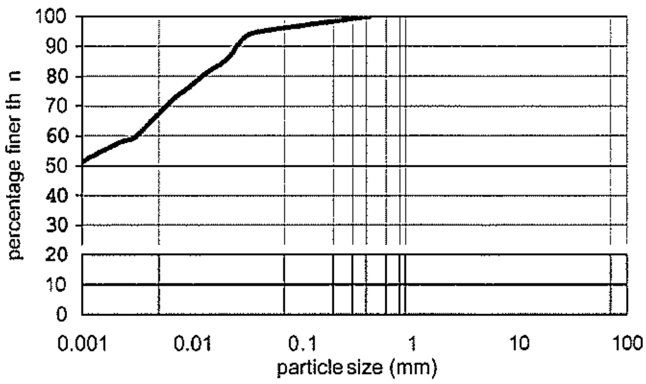

Figure 2. Grain size curve of the tested material

EXPERIMENTAL PROCEDURE

Two methods were used to prepare the samples. The first method involved mixing the crushed shale with water to form slurry at a moisture content close to the liquid limit. The mix was placed into a lubricated cylindrical mould of 38mm diameter, and the soil was then compressed one-dimensionally with drainage from both ends. Loads were applied to a hanger in stages, to give a final vertical stress, σ_v, of 80 kPa. To ensure uniform samples were produced, the mould was turned over occasionally so that the load could be applied at both ends of the sample. After 48 hours samples with height to diameter ratios of approximately 2 were extruded from the mould and placed in a triaxial cell. The samples were subjected to an effective confining stress of 20 kPa at which point the back pressure was raised to 500 kPa to ensure saturation. The second technique was used to rapidly prepare samples with low porosity. A predetermined mass of *dry* crushed shale was placed into a greased split-cylinder steel mould of 38 mm diameter, and axial load was applied to compress the soil to reach a target void ratio of 0.15. The axial load was then removed and the mould split open to extract the soil sample.

The dry samples were then transferred directly to the triaxial cell. After applying a confining pressure of 100 kPa the samples were permeated with carbon dioxide and then saturated with water. An elevated back pressure of 500 kPa was then used to ensure saturation.

Two triaxial cells were used in the tests, a conventional soil mechanics cell for tests with effective confining pressures up to 1 MPa, and a cell designed for tests on rock with confining pressures up to 60 MPa. In both cells the samples were jacketed by latex rubber membranes and water was used to provide the confining stress. Samples were isotropically consolidated to the desired stress in stages. In the rock cell drainage only occurred, through a ceramic porous disk, from the base of the sample. This resulted in long test durations. Compression of the sample to 60 MPa was performed in 6 stages with consolidation taking approximately 6 days for each stage. After reaching equilibrium at the required stress the samples were subjected to standard drained or undrained triaxial tests at constant confining pressure. In the conventional triaxial cell samples were sheared in undrained and drained conditions at approximately constant strain rates of 0.1%/min and 0.01%/min respectively in a 50kN loading machine. The samples subjected to high stress in the rock cell were tested drained at a constant strain rate of 0.0004%/min in a 350kN capacity loading machine. This slow rate, necessary to ensure drainage, meant that the shearing stage of the tests took about 14 days to complete.

RESULTS

The responses in isotropic compression are shown in Figures 3a and 3b. Points representing the equilibrium states of void ratio, e and mean effective stress, p' are plotted in $e : p'$ space. The isotropic normal compression line (INCL) has been drawn on the Figures. It can be seen that this line is linear in $e : \log p'$ (Figure 3a) between p' of 100 kPa and 10,000 kPa. At higher values of p' the curve flattens out, and the log $e : \log p'$ response (Figure 3b) has been included to show more clearly the high pressure response

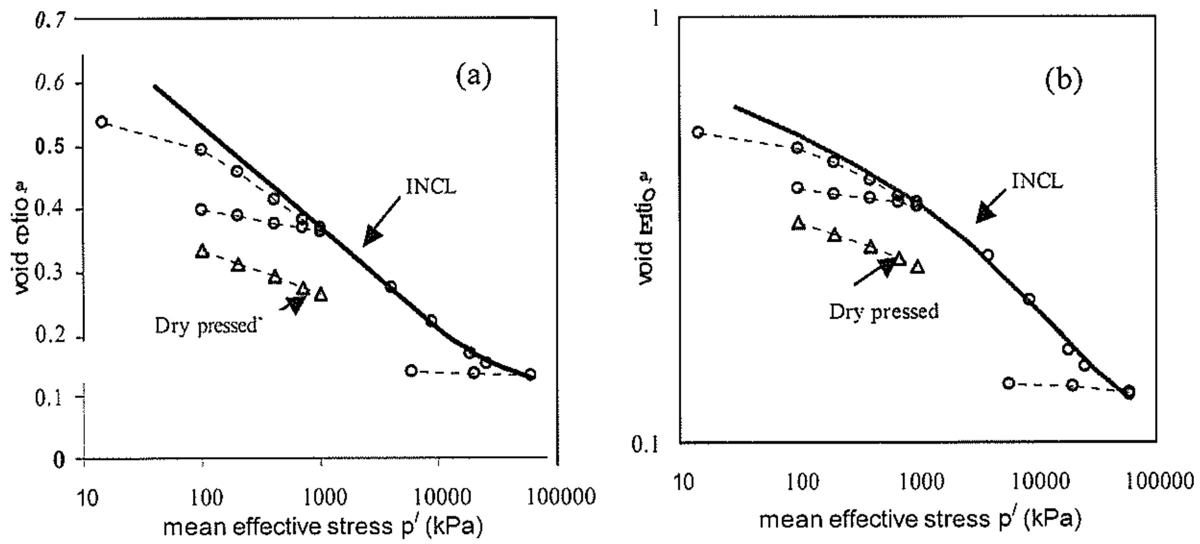

Figure 3. Isotropic compression of Bringelly shale in (a) e : log p', (b) log e : log p'

The INCL can be represented by two equations. For mean effective stresses less than 10,000 kPa it is given by:

$$e = N - \lambda \ln p' \qquad (1)$$

where the slope $\lambda = 0.07$ and the intercept at $p' = 1$kPa, $N = 0.85$. For mean effective stresses higher than 10,000 kPa the INCL may be fitted by a hyperbolic function given by:

$$e = \frac{A}{(\ln p' - B)} \qquad (2)$$

where A and B are constants that can be chosen so that the INCL is continuous with no change of slope at p' = 10,000 kPa.

It may be seen from Figure 3a that the slope of the isotropic unloading curve reduces as the maximum effective stress increases, as does the slope of the INCL. The value of κ, the slope in a conventional $e : \ln p'$ plot reduces from 0.009 to 0.0013. The samples that were prepared by the dry-press method had experienced a maximum effective stress of approximately 30 MPa and a minimum void ratio of 0.15. However, after unloading and extrusion from the mould their void ratio increases to 0.33, primarily due to yield during one-dimensional unloading. On reloading they show a more compressible response than would be expected from an isotropically over-consolidated sample at the same stress and void ratio. It is believed that the loading history has created a heterogeneous material with highly sheared bands of low density surrounding "intact" high density material, and that during isotropic compression the response is dominated by the material in these highly sheared bands. The influence of this imposed structure is discussed further below when looking at the shearing response of these samples.

The effective stress paths from a series of undrained tests on isotropically normally consolidated samples are shown in Figure 4 in a plot of deviator stress, q $(=\sigma_1 - \sigma_3')$ versus mean effective stress, p $(=(\sigma_1 + 2\sigma_3)/3)$. For these samples, which have not experienced effective stress greater than 1 MPa, a critical state line (CSL) can be defined given by $M = 1.14$ and a corresponding effective friction angle $\phi = 28.5°$.

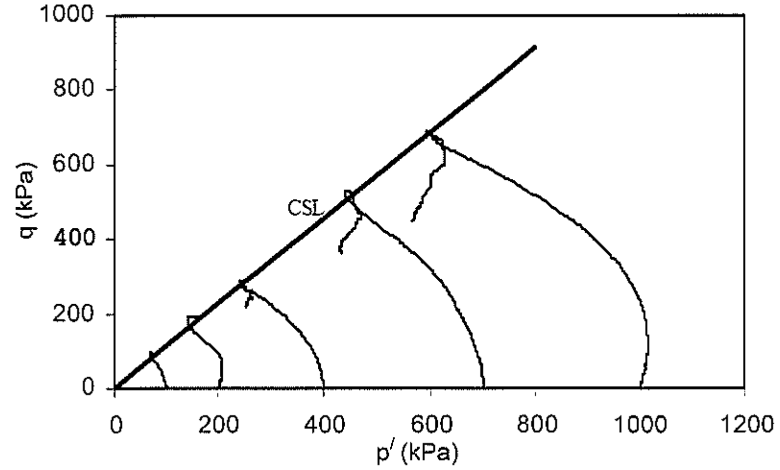

Figure 4 Effective stress paths of normally consolidated samples

It is expected that samples with a given overconsolidation ratio (OCR) will behave similarly once allowance is made for differences in confining stress level (Atkinson and Bransby, 1978). Figure 5 shows deviator stress normalised by the effective consolidation stress, p'_c at the start of shearing for a series of drained tests. The tests have been divided into two groups, those with relatively low pre-consolidation stress, $p'_{max} \le 6000$ kPa, and those with $p'_{max} = 60{,}000$ kPa.

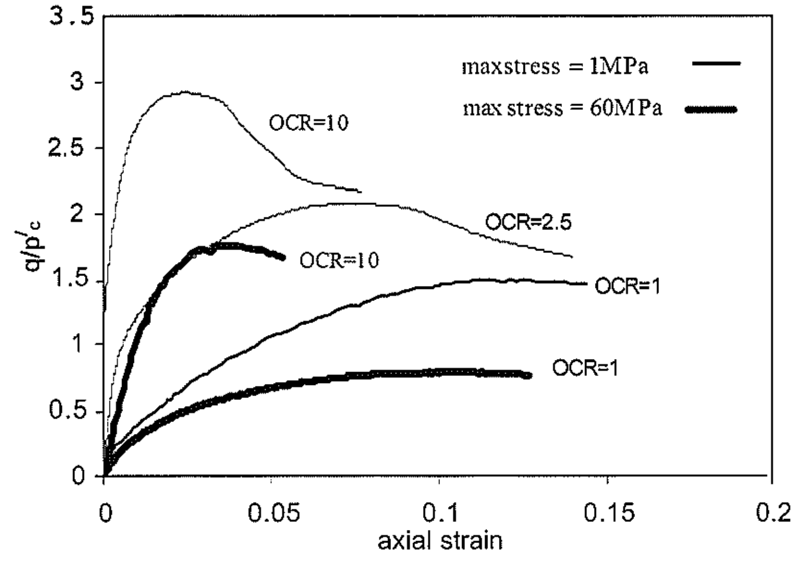

Figure 5 Influence of OCR on normalised deviator stress, axial strain responses from drained tests

From Figure 4 it can be seen that similar and normalisable responses are obtained for effective confining stresses up to 1 MPa. It was also found that the response of a normally consolidated sample with $p'_c = 6$ MPa was similar to that at lower stresses. It is therefore believed that for maximum consolidation stresses of up to 6 MPa the normalised responses for a given OCR are unique, as reported in numerous studies on reconstituted clays (e.g. Atkinson and Bransby, 1978). Figure **5** shows that the responses of the samples that have been compressed with a maximum stress of 60 MPa all lie significantly below the curves of the same OCR where the samples have experienced a maximum stress of 1 MPa. Both the final strength and the normalised stiffness are lower for the highly compressed samples. The ultimate friction angle for the highly compressed samples is only 16.5°;

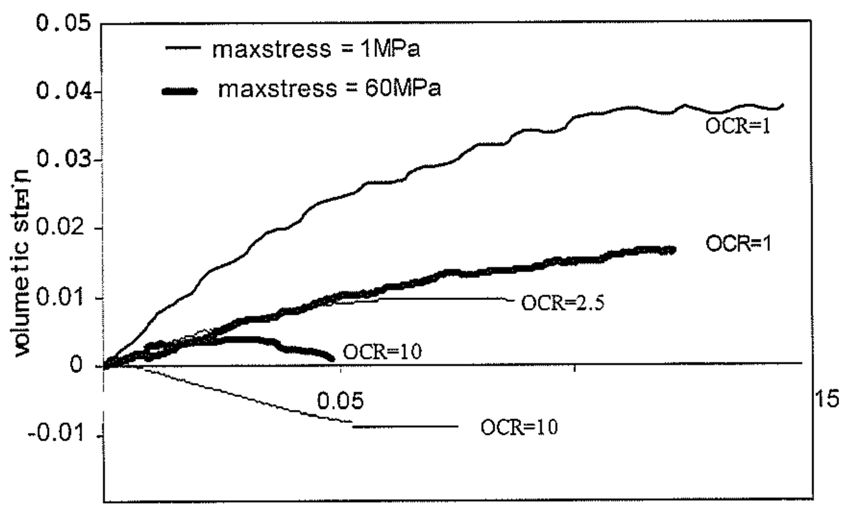

Figure 6 Influence of OCR and pre-consolidation stress on volumetric strain responses

significantly less than the critical state friction angle $\phi' = 28.5°$ measured for the lower density samples. When the confining stress reaches 60 MPa the void ratio is about 0.15. With such low void ratios it can be expected that the potential for further volume change on shearing is reduced and this is shown in Figure **6**. For normally consolidated samples the compressive volume strain for a sample with p_c = 60 MPa is about half that for a sample with p_c = 1 MPa. It may also be noticed that the tendency for volume expansion when OCR = 10 is greatly reduced when p_c = 60 MPa. For low pre-consolidation stresses the material behaves as would be expected from the concepts of critical state soil mechanics, but the patterns of behaviour of the highly compressed samples are not consistent with the existence of a critical state line which would have required much greater dilation from the highly compressed sample with OCR = 10. One observation from these results is that the concept of an OCR as a means of normalising the results has little use for such highly compressed soils. This concept is discussed further below.

One of the assumptions of many models based on critical state soil mechanics is that sections through (q, p, e) space at constant e are similar in shape (Atkinson and Bransby, 1978). These sections can be explored by plotting q and p normalised by the equivalent p_e on the INCL at the same e. To determine p_e the INCL as given by Equations 1 and 2 has been used. The resulting normalised plot is shown in Figure 7. This figure shows clearly the different shape of the sections at constant e, and of the state boundary surface. The highly compressed samples lie significantly below those samples with a maximum stress of 1 MPa. The *dry* pressed samples that have been compressed to a void ratio of 0.15 and then allowed to swell back show intermediate behaviour. This behaviour is believed to be a consequence of the heterogeneous structure created by the sample preparation

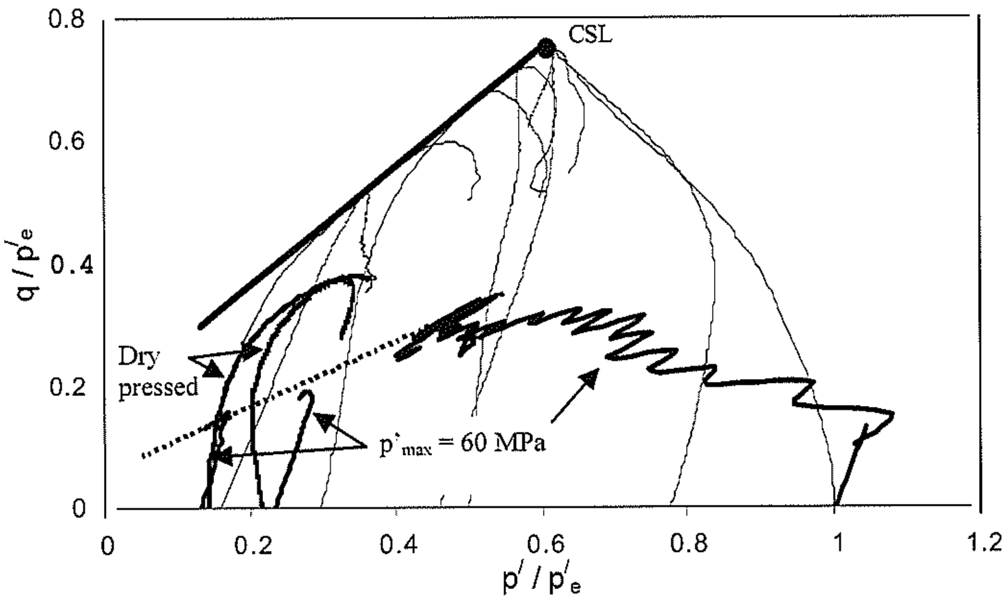

Figure 7 Normalised state boundary surfaces for reconstituted shale at low and high stress

method, with highly sheared bands of low density surrounding "intact" high density material. When sheared these samples reach an ultimate frictional strength identical to the samples that have only been lightly compressed, i.e. M =1.14, ϕ = 28.5". The *dry* pressed samples also have a shear stiffness similar to normally consolidated samples (William et al, 2001). It is apparent that the use of OCR and other conventional means of normalising soil behaviour are not appropriate for the *dry* pressed samples.

DISCUSSION

Direct comparison of the results to other work is difficult since laboratory-induced high pre-consolidation stresses have rarely been reported. However, 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). Picarelli et al show that the normalised state boundary surface of the intact shale lies below the surface for the reconstituted material tested at higher density and lower pre-consolidation stresses. Picarelli et al. interpreted the difference in behaviour as evidence of the effects of fissuring in the natural soil, and suggested that the mechanism of deformation could be described by models used for fractured rocks, where deformation and strength are controlled by movements alongjoints and fissures. They also noted that OCR did not seem to significantly affect the strength. The results presented in Figures 4 to 7 show the same pattern of behaviour reported by Picarelli et al., but in this study only reconstituted soils have been used with no fissures. This result suggests that in addition to fissures, the fabric associated with the low porosity, created by the high stress, is also contributing to reduced strength and different deformation mechanisms. At low porosity there must be locally a high degree of alignment of the plate-like clay particles, although overall no preferential alignment of the clay plates would be expected because of the isotropic compression. 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 8, 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.

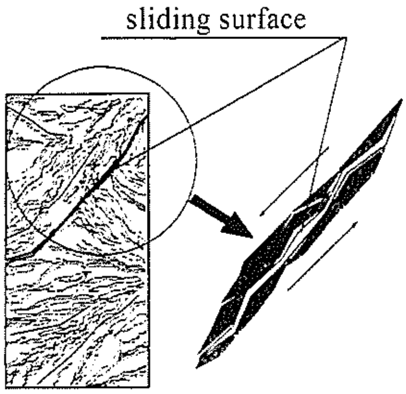

Figure 8 Mechanism of shear deformation and rupture (after Picarelli et al, 1998)

A related mechanism can be proposed for the *dry* pressed samples. As discussed above these samples show many of the characteristics of normally consolidated samples, even though they have been highly compressed and then reloaded to a state where they are clearly over-consolidated. It has been suggested that yielding occurring during 1-D unloading has created softened and highly sheared zones where the highly compressed structure has been broken down. It is believed that movement within these sheared zones is controlling the deformation. It should be noted that there was no visual evidence of heterogeneity and further microscopic investigation is required to confirm the proposed mechanism.

The natural Bringelly shale has a very low porosity, similar to that produced by the high stresses in this study. In addition it has significant micro-cracking in the plane of the laminations. Consideration of these factors would suggest that the effective friction angle controlling the strength of the shale could be 16.5°, determined for the reconstituted material at low porosity, or lower. The actual strength will also be influenced by the degree of bonding in the natural shale. Microscope studies suggest that bonding is relatively weak, and its affect on strength is not anticipated to be large. If this is the case then the reconstituted material may provide useful insights into the behaviour of the natural shale as has been found for structured soils and weak rocks (Leroueil and Vaughan, 1990). Confirmation of these findings will require further mechanical testing of the natural shale, which is in progress.

CONCLUSIONS & RECOMMENDATIONS

Specimens of reconstituted crushed shale have been subjected to standard isotropically consolidated triaxial tests. For material that has experienced a maximum effective stress of less than 6 MPa the behaviour is consistent with the assumptions of critical state soil mechanics. Samples when sheared head towards a unique critical state line, and similar normalised behaviour is shown by samples with the same OCR. However, samples of the same material that have experienced a maximum effective stress of 60 MPa show significantly reduced frictional strength and patterns of behaviour inconsistent with critical state concepts. It has been suggested that the different behaviour of the highly compressed samples is related to alignment of the clay particles and the promotion of sliding on planar surfaces between these aligned particles.

These results are significant for two reasons. First, it may be anticipated that the frictional strength of the natural shale is similar to that of the highly compressed reconstituted material because of the similar void ratio and structure. The frictional strength of the natural shale may therefore be considerably lower than measured from reconstituted samples at stress levels typical of standard soil mechanics practice. Second, attempts to understand the significance of bonding and structure in the natural shale by comparison with the same material in a reconstituted state, can only be achieved if the reconstituted material is compressed to comparable void ratios.

Mechanisms have been proposed to explain the different deformation and strength responses. Microscope studies are required to confirm that different mechanisms are active at low and high stress levels.

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