The role of cementation and destructuring in the mechanical behaviour of Bringelly shale

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ABSTRACT

A series of tests have been performed to investigate the effects of cementation and de-structuring on the behaviour of a claystone, which constitutes the largest member of the Bringelly shale formation found in the Sydney Basin. The program of tests has included triaxial tests on the shale at its natural moisture content, triaxial tests on core specimens after saturation, and saturated direct shear tests on specimens reconstituted from crushed claystone. A wide range of isotropic confining stresses, from 0 to 60 MPa, were used, before specimens were subjected to standard drained compression tests at a constant strain rate until approaching failure. It has been found that the strengths of the natural and reconstituted specimens, when compressed to the same void ratio, are similar, with both showing friction angles significantly less than residual strength of the reconstituted material at higher void ratio. It is postulated that it is the fabric, created by the high stress, which, leads to the reduction in the strength. The claystone is only weakly cemented, and its strength and stiffness are greatly affected by the presence of suctions in the partially saturated natural material.

1 INTRODUCTION

The characterisation of shales presents challenges for both engineers and engineering geologists, because these materials have both soil and rock like properties. Assessment of the suitability of shales for use as foundation support or as engineered fill materials requires knowledge of both material properties and the proposed application. One of the critical parameters in any such assessment is the degree of cementation (hardness) and the sensitivity of this is to changes in environmental conditions. This would normally be investigated by a combination of strength and durability tests, but the index tests used to determine these parameters provide little insight into the factors controlling the behaviour.

This paper is concerned with the Bringelly Shale, a material which is the uppermost rock over large areas of the Sydney Basin. This material has unconfined strengths of 20 to 30 MPa, and yet small lumps swell and disintegrate when placed in tap water for 24 hours (Wiliam and Airey, 2004). The UCS results suggest a well cemented material, but the significant swelling which accompanies immersion in water suggests, on the contrary, that the cement must be weak. To investigate the reasons for this behaviour a series of triaxial tests have been conducted using both natural core specimens, and specimens created by reconstituting the shale after first crushing it to a fine powder.

Core specimens have been tested in their natural partially saturated state and after saturation. The degree of saturation was considered to be an important variable in this study, because Bringelly shale is partially saturated above the water table, which is typically 20-30m below the ground surface. Although below this depth, Bringelly shale is fully saturated (MacRae and Paterson, 1990), it is partially saturated material which is encountered in most engineering projects, and which then has to be disposed, or used as a fill material.

By comparing the reconstituted crushed shale and the natural specimens an indication of the importance of fabric and cementation can be obtained. This approach has been widely used to investigate structure in soils and weak rocks. This paper compares the mechanical responses of natural and reconstituted Bringelly shale and uses this data to investigate the role of cementation and fabric in controlling the stiffness and strength of the material.

2 APPARATUS AND PROCEDURE

2.1 Materials

This paper is concerned with the claystone which is the dominant material in Bringelly shale. Only salient properties of the claystone are reported here as the characterisation of Bringelly shale has been discussed in previous papers (e.g. William and Airey, 2004). The claystone is comprised predominantly of clay minerals (52%) and quartz (38%), and the clay minerals include reactive mixed layer illite-smectite. The claystone has been highly compacted with typical porosity of about 10%. There is little evidence of induration although petrographic analysis indicates there is bonding derived from a small amount (3%) of siderite and some recrystallisation of mica at particle contacts. The unconfined compressive strength (UCS) is typically between 20 and 30 MPa, but the material has only medium to low durability. For this study large block and core specimens were obtained from several locations in the Sydney Basin and all were found to have similar properties.

2.2 Triaxial tests

Specimens of natural and reconstituted shale have been subjected to conventional CID triaxial tests with confining pressures varying from zero to 60 MPa. To cope with the large stress range three different triaxial cells with capacities of 2, 20 and 70 MPa were used. The equipment was essentially the same for each cell, with only the capacity of the instrumentation, pressure source and load frame changing to match the stress level. In some tests at low stress levels additional on-sample deformation measurements and shear wave velocities were recorded to provide more reliable stiffness measurements.

Specimens with diameters between 38 mm and 55 mm were used in the triaxial tests. To prepare saturated core specimens the specimens were subjected to a confining pressure of 600 kPa before permeation with carbon dioxide and then with water. Then maintaining an effective cell pressure of 600 kPa the cell and back pressure were raised together until the back pressure reached 500 kPa. A high cell pressure was necessary during saturation to minimise swelling and prevent loss of structural integrity. To prepare reconstituted specimens the shale was crushed to a fine powder and then mixed with distilled water to form a slurry. This was placed in a cylindrical mould and allowed to consolidate one-dimensionally under a stress of 80 kPa. After extrusion from the mould the specimens were set up in the triaxial apparatus under a confining stress of 30 kPa. An elevated back pressure was used to ensure specimen saturation.

Specimens were then isotropically consolidated in stages to the required stress level. The majority of the specimens were then subjected to conventional drained triaxial shear tests to failure. For the high pressure tests on the reconstituted specimens only one-way drainage could be used, and because of the relatively low coefficient of consolidation, c_v = 1.2 m^2/yr due to the clay content and mineralogy, each test took several months to consolidate and shear.

2.3 Ring and direct shear tests

Ring shear and cyclic direct shear box tests were performed to investigate the residual friction angle at low stress levels. The soil specimens used in this investigation were prepared from crushed shale, which was reconstituted to form a slurry at the liquid limit. Specimens were compressed one-dimensionally in the apparatus to stress levels of 50, 100 and 200 kPa at which stages shear tests were performed. A small (Bromhead 1979) ring shear apparatus with an annular ring shaped specimen was used in this study. The lower half of the device was rotated whilst the upper half was held stationary. The direct shear tests were performed in an automated square shear box with an area of 3600 mm^2. The methods of preparation and procedure of the ring and direct test series were carried out in accordance with recommendations of Bromhead (1979; BS 1377: 1990) and ASTM Standard D30-90, 1996 respectively

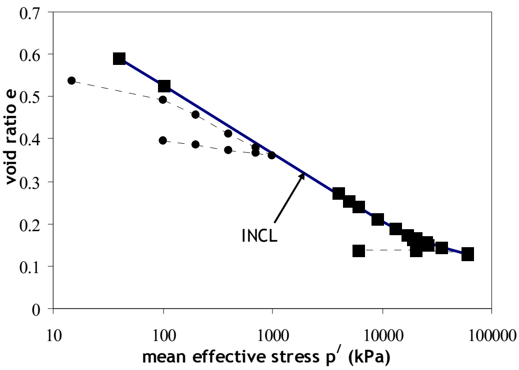

Figure 1: Isotropic response of reconstituted Shale

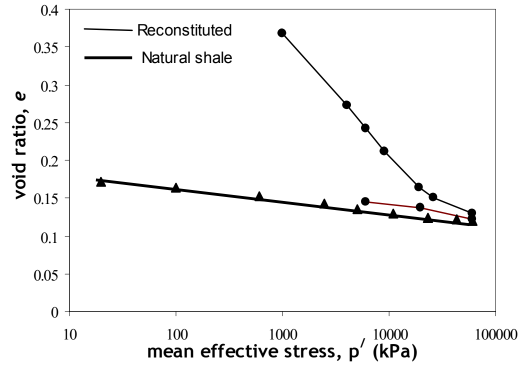

Figure 2: Isotropic response of natural shale

3 RESULTS

3.1 Mechanical responses

3.1.1 Isotropic compression

Figure 1 and Figure 2 show typical isotropic compression responses of reconstituted and saturated natural shale respectively in terms of void ratio, e and mean effective stress, p′. Figure 1 shows that the reconstituted shale has a normal response with a linear normal compression line up to a stress of 10 MPa. However, for higher stresses, and low void ratios, the normal consolidation line flattens and can be described by a hyperbolic relation (William, 2006). The natural shale was saturated at p' = 600 kPa. Figure 2 shows that a linear e, log p′ is obtained both on swelling to 20 kPa, and on reloading up to 60 MPa. The data suggest that the shale had experienced a pre-consolidation stress significantly higher than 60 MPa, and this is consistent with the known geology of the Sydney Basin.

3.1.2 Drained shear responses

Figure 3 shows the influence of effective confining stress level (p'_c) on the deviator stress (q), axial strain response of normally consolidated reconstituted material. It is very clear that the normalised shear strength reduces significantly as the effective confining stress increases from 6 MPa to 60 MPa. This reduction in normalised strength represents a reduction in effective friction angle from $\phi' = 28.5^o$ to 17^o. Reconstituted specimens that had experienced maximum stresses less than 6 MPa behaved according to conventional critical state soil mechanics (Atkinson and Bransby, 1979) with a consistent ultimate frictional strength of 28.5^o. When a reconstituted specimen that had been

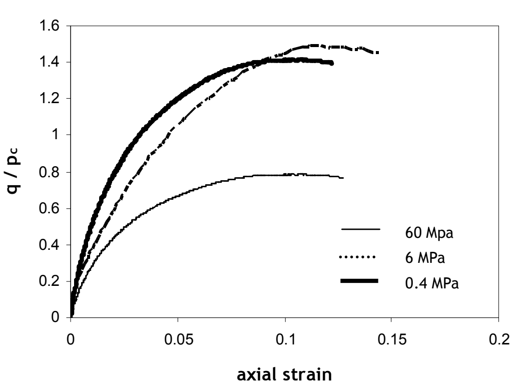

Figure 3: Normalised deviator stress, strain responses for reconstituted shale

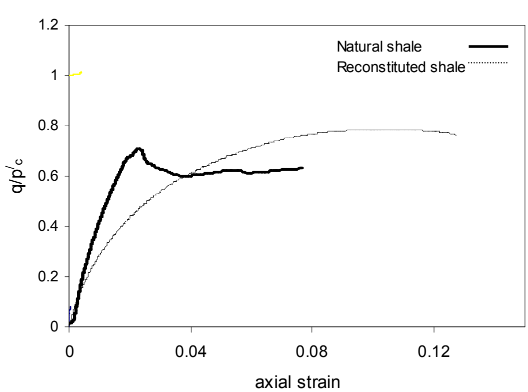

Figure 4: Comparison of natural and reconstituted shale (p'_c = 60 Mpa)

compressed to 60 MPa was unloaded only a slight increase in frictional strength was observed. This result strongly suggests that it is the low void ratio rather than the high stress which is responsible for the friction angle reduction. Figure 4 shows a comparison of the normalised deviator stress, axial strain responses of reconstituted and natural shale at a confining pressure of 60 MPa. The natural specimen shows a slightly stiffer and more brittle response, but both specimens achieve similar strengths. Similar observations were made when comparing over-consolidated reconstituted and natural specimens at lower stresses (William, 2006) indicating that the low friction angle applies to all specimens with low void ratio.

3.2.3 Residual strength

The residual friction angle obtained from Figure 5 for both ring shear and shear box tests was 28.2^0, while c_r was approximately zero. These values agreed with the critical state strength parameters obtained from the triaxial tests on reconstituted specimens with low confining pressures (≤ 6000 kPa). The similar residual and ultimate friction angles were surprising for several reasons: first it has been reported that soils with clay contents > 40% show significant differences between their peak and ultimate ϕ' (Skempton, 1985), and for this material clay content was >50%; second, as noted above, significant reductions in ϕ'_{ult} down to 17^o were measured at low void ratio; and third when residual strengths lower than ultimate strengths have been measured these have been related to particle alignment (e.g. Lupini et al., 1981). It was expected from the high pressure tests that the residual strength of this material would be given by $\phi'_r \approx 17^o$, rather than 28^o. However, the correlation proposed by Wesley (2003), based on the vertical distance from the A-line in a plasticity chart, correctly predicts the residual friction angle as $\phi'_r = 28^o$. This agreement may be fortuitous as Lupini et al. (1981) have noted that attempts to correlate friction angles ϕ'_r, ϕ' of soils to their index parameters have not been very successful, but it does indicate that the lack of a reduced residual strength appears reasonable. These results imply that only at very low void ratios is there sufficient alignment of the particles to enable the sliding mode of failure between clay particles to develop.

3.2 Fabric studies

Scanning electron microscope pictures were taken before and after shearing in an attempt to identify the fabric and mechanisms responsible for the reduced frictional strength at low void ratio. Figure 6 shows the shear plane of a specimen isotropcally compressed to 60 MPa with a void ratio of about 0.1. This figure shows the high alignment of the particles caused by compression to low void ratios, and a clear shear plane that developed during shearing, indicated by the white band. This distinct shear zone was not observed in the less compacted specimens. Comparisons of the highly compacted reconstituted specimens and the natural shale did not reveal any obvious difference in fabric or the presence of cementing material (William, 2006).

3.3 Stiffness

Figure 7 shows data on the maximum shear modulus, G_{max}, as a function of mean effective stress. Data obtained from the triaxial tests are shown for saturated core specimens and the reconstituted soil. In both cases the data may be described by a relation of the form:

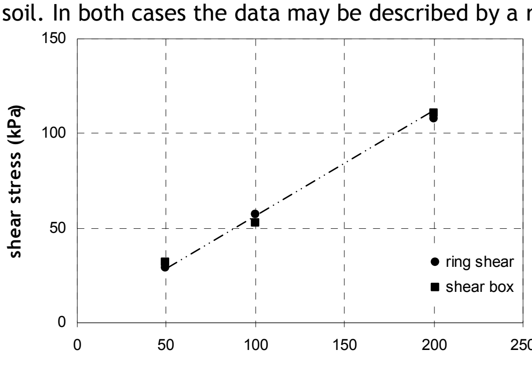

Figure 5: Residual strength envelopes of reconstituted material

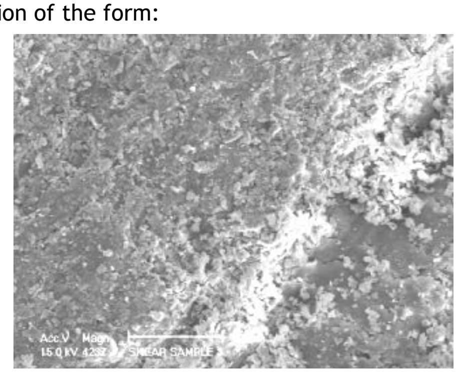

Figure 6: Fabric of highly compressed shale

$$G_{max} = A\,p'^{0.5} \quad (1)$$

where A is a constant. An additional point is shown on Figure 7 indicating the shear modulus measured on core specimens at their natural moisture content from shear wave velocity measurements. This value has been plotted against the total stress, rather than the effective stress, because the true effective stress in this specimen is unknown. However, measurements of total suction indicated values of 150 MPa, which would indicate that there are significant effective stresses, and would be sufficient to align the natural moisture content sample with the saturated data. The similar response of the saturated and reconstituted specimens suggests that any cementation is ineffective. The difference between them can in part be explained by the different load histories, with the natural shale being primarily one-dimensionally compressed, whereas the reconstituted specimens were compressed isotropically

3.4 Response of specimens at natural moisture content

It has been noted above that core specimens at their natural moisture content have very high shear wave velocities and stiffnesses at small strain, and this has been suggested to be primarily due to suctions. Comparisons of the stress, strain responses of saturated and unsaturated specimens are shown in Figure 8 at a confining stress of 1 MPa. The saturated strength is only half the unsaturated value. The UCS strength of this material is about 20 MPa (William, 2006) and the strength of the unsaturated specimens is essentially constant for confining stresses from 0 to 6 MPa. In contrast the strength of the saturated specimens drops from 15 MPa at an effective confining stress of 6 MPa down to 0.85 MPa at an effective confining stress of 20 kPa. Figure 8 does not show a large difference in stiffness between the unsaturated and saturated specimens, in contrast to the data shown in Figure 7. Figure 8, and other tests at low confining stress, shows the stiffness of the unsaturated specimens increasing until close to failure. This pattern of behaviour is normally associated with closure of micro-cracks, which were widely observed in SEM studies. As the shear wave passes through the intact material it may overestimate the effective modulus of the shale.

4 DISCUSSION

It has been argued that there is little indication of cementation in the claystone. The saturated core specimens are slightly stiffer than the reconstituted material, and this can be explained by differences in the geological history and deposition affecting the fabric. It is possible that some loss of cementation occurred during saturation as a volume strain of 0.5% was recorded. A strain of this magnitude would be sufficient to significantly degrade any existing bonds, however, such large strains are an indication that any bonding is weak. Specimens at their natural moisture content have not been affected by swelling, and hence any cementation would be expected to be more significant for these specimens. The observation of a constant failure stress, independent of stress level, could be explained by strong cement bonds, but this does not seem plausible considering the saturated test data. It is thus believed that the high strength of the natural core derives from its

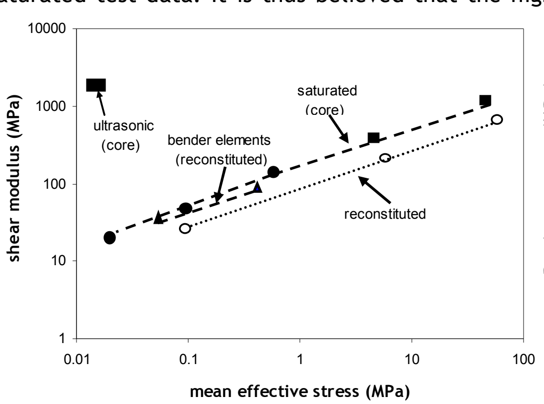

Figure 7: Shear modulus of intact and reconstituted shale

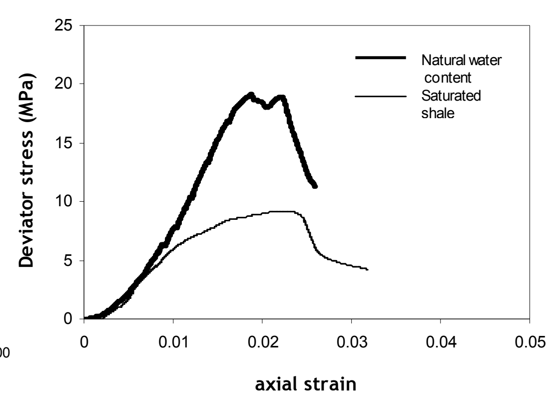

Figure 8: Comparison of stress, strain responses at natural moisture content and saturated

high total suction. The high suctions are themselves a consequence of the low void ratio as the degree of saturation was typically 50%. It is also observed that specimens at their natural moisture content will break along horizontal planes, again providing evidence of weak cementation and of the alignment of the particles.

5 CONCLUSIONS

Index tests of Bringelly Shale indicate an unconfined strength of at least 20 MPa and a medium durability. However, a series of triaxial tests of natural and crushed reconstituted Bringelly Shale show that there is apparently little cementation. The strength and stiffness of the natural and reconstituted materials are essentially the same when the natural material is fully saturated.

At its natural moisture content the shale is unsaturated and possesses significant strength and stiffness. This strength derives principally from pore water suctions. When stress levels are low and water is available the shale is likely to swell and disintegrate.

The mechanical response of the shale is controlled by the particles, which are highly aligned and constrained because of the low porosity. The reconstituted material behaved similarly to the natural shale when compressed to the same low void ratio as the natural shale. The residual friction angle for the reconstituted material at low stress was 28.2^o, significantly higher than the 17^o friction angle of the highly compressed material. Even after swelling back to low stress the strength was affected by the particle alignment, and was much lower than expected from the reconstituted material. This indicates the importance of accounting for fabric in assessing the frictional parameters for argillaceous materials.

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