

INDEX PROPERTIES AND THE ENGINEERING BEHAVIOUR OF BRINGELLY SHALE

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SUMMARY

This paper is concerned with the engineering behaviour of Bringelly Shale and how this can be assessed based on laboratory index tests that are widely used for argillaceous rocks. Comparison will be made with data from Ashfield shale to indicate the differences between these two members of the Wianamatta Group. It is shown that Bringelly shale contains reactive clay minerals, absent in Ashfield Shale and, as a result, the shale is more sensitive to changes in environmental conditions. Bringelly Shale is only weakly cemented and its strength and stiffness are lower than Ashfield Shale. Both shales have similar unconfined compressive strengths, typically between 10 MPa and 50 MPa, but in Bringelly Shale a large component of this strength appears to be derived from pore water suctions. When Bringelly Shale is placed in water it disintegrates. The paper concludes with some implications of the data for construction in Bringelly Shale.

1 INTRODUCTION

This paper is concerned with the shales of the Wianamatta group, known as Ashfield Shale and Bringelly Shale. These shales outcrop over a large part of the Sydney metropolitan area as shown in Figure 1. Information on the geology of these sediments has been reported in a number of studies and has been summarized in the Geology of the Penrith Sheet (Jones and Clark, 1991). These studies have suggested that both these shales, and the Minchinbury Sandstone that lies between them, were deposited in a single regressive episode during the Triassic age. The Ashfield Shale is comprised of a sequence of dark-grey to black, sideritic claystone-siltstone which grades upwards into a fine sandstone-siltstone laminite. The Bringelly Shale, which overlies the Ashfield Shale, is a more complex formation composed of different lithologies, which in order of decreasing volumetric significance include: Claystone-Siltstone (70%), Laminite and Sandstone (25%), Coal and highly carbonaceous claystone (3%) and Tuff (2%). The claystone units are composed of several types of fine-grained sediments, namely light-grey leached claystone, dark-grey to black carbonaceous claystone and non-carbonaceous mid to dark-grey claystone and siltstone. The different shale types are believed to reflect different depositional environments, with the Ashfield shale deposited in a marine environment and the Bringelly Shale deposited in an alluvial environment.

There is considerable debate about the post-depositional history of the shales in the Sydney Basin, with estimates for the depth of over-lying sediments ranging from tens of metres to 4 km. Recently Bai et al. (2001) have suggested that the Narrabeen group rocks were buried to a depth of about 3 km before rapid uplift and erosion of about 2 km of sediment occurred with the Tasman Sea rifting in the mid-Cretaceous.

Ashfield Shale has been encountered in many engineering projects and consequently there is a range of data on basic properties and engineering performance for this shale. However there is little information on the engineering geology of the Bringelly Shale. Previous papers (Won, 1985; Chesnut, 1991) have suggested that the properties of the two shales are similar. More recently it has been shown that there are significant differences in mineralogy and durability of the two shales and differences in their engineering performance can be expected (William and Airey, 1999).

This paper will discuss basic laboratory characterization studies of the shales performed at Sydney University over the last 10 years. Data will be provided on the mineralogy, micro-structure, durability, swelling, stiffness and strength of the two shales. These studies have been limited to the claystone-siltstone materials which are the predominant lithology in both shales. Large block samples and cores have been obtained from several locations, indicated on Figure 1, to give a wide areal coverage and to investigate sample variability. Ashfield Shale samples have been obtained from sites at Moorebank, Ryde, and Surry Hills. The Ashfield Shale samples all had a similar interbedded appearance with frequent very thin light grey silty bands in dark grey siltstone. Samples of Bringelly Shale have been obtained from quarries used to extract shale for brick manufacture at Kemps Creek, Badgerys Creek, Horsley Park and Mulgoa. The shale from all these sites could be described as a non-carbonaceous mid to dark-grey claystone. For both shales the data base has been widened by information supplied by several other organisations.

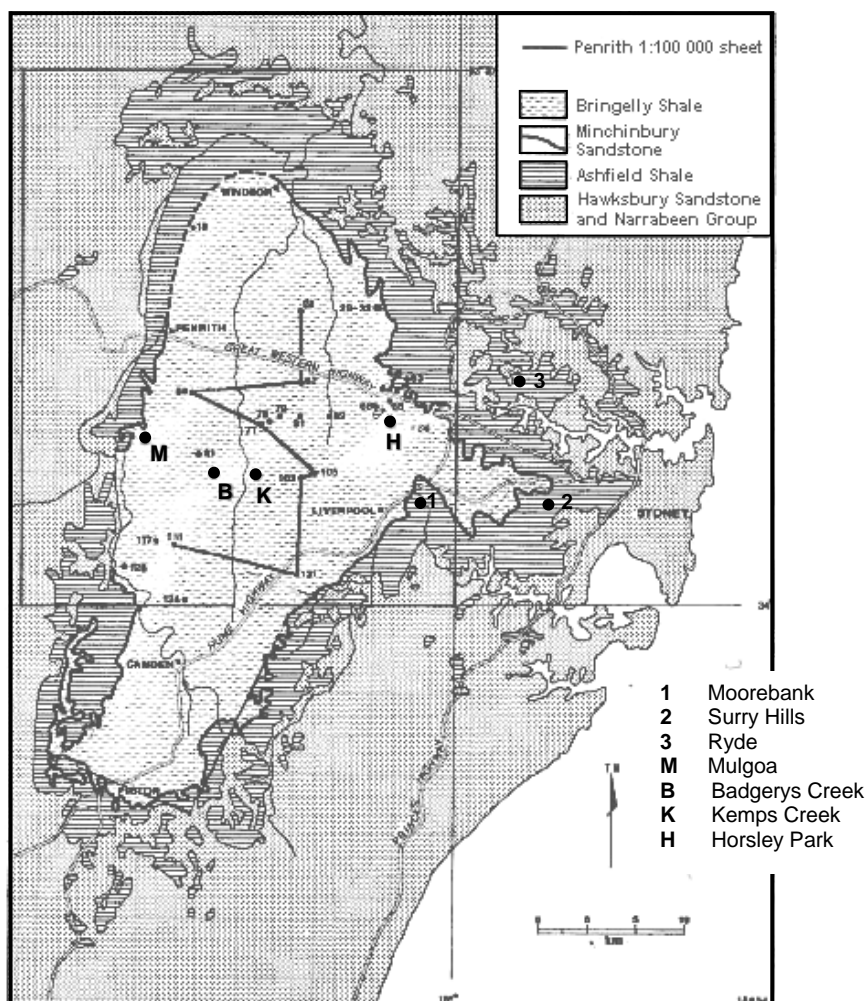


Figure 1: Distribution of Wianamatta Group sediments (after Herbert 1979).

2 CLAYSTONE-SILTSTONE PROPERTIES

2.1 MINERALOGY

The types and quantities of the various clay and other minerals comprising shale can have a significant influence on the engineering behaviour. Herbert (1979) has reported clay mineral contents between 40% and 65% for the claystone-siltstone materials in the Ashfield and Bringelly Shales, with a trend towards higher clay content in Bringelly Shale. For the samples from the sites discussed above, Ashfield Shale was found to have an average clay mineral content of 43% compared with 51.5% for Bringelly Shale. The other major constituents are shown in Table 1. The ranges for the amounts of clay minerals, shown in Table 1, are relatively small and lie within the range previously reported by Herbert (1979). In both shales quartz is the main non-clay mineral with significant amounts of siderite in Ashfield Shale and feldspar in Bringelly Shale. Organic carbon content was measured for the Bringelly Shale, as organic matter appears to contribute to cementation, but for Ashfield Shale this has not been determined as organic matter appears to be negligible. One important difference between the two shales is the amount of siderite present.

Table 1: Mineral composition of fresh Wianamatta Group Shales.

	Clay minerals (%)	Quartz (%)	Siderite (%)	Feldspar(%)	Organic matter (%)
Ashfield Shale	43 (41-45)	47	10	0	?
Bringelly Shale	51.5 (48-55)	38	3	6	1.5

Petrographic studies, comprising optical microscopy and scanning electron microscopy, have been used to investigate the microstructure and nature of cementation in the Wianamatta Group shales. In Ashfield Shale siderite, which is widely dispersed through the shale, acts as the primary cementing agent, while mica acts as a secondary cementing

agent. In Bringelly Shale siderite, organic matter and some recrystallisation of mica could contribute to cementation, but none of these mechanisms is well developed and cementation is expected to be weak.

The distribution of the clay minerals within the shales was investigated by X-ray diffraction. Very similar distributions were obtained from all samples and all sites and average values of our results are presented in Table 2.

Table 2: Percentages of the various clay minerals.

	Ashfield Shale	Bringelly Shale	
	Fresh	Fresh	Extremely Weathered
Kaolin	56	33	30
Illite-Smectite		40	55
Montmorillonite			2.5
Illite	44	21	12.5
Chlorite		6	

Table 2 shows that there are very significant differences in the clay mineralogy of the two shales. The main difference is the large amount of mixed layer, illite-smectite, in the Bringelly Shale. Mixed layer clays have properties intermediate between illite and smectite and thus the shale can be expected to be reactive, susceptible to swelling and changes in pore fluid chemistry. A previous study (Loughnan, 1960) also reported mixed layer clay minerals, but these only comprised 20% of the clay fraction. The greater amounts of mixed layer clay detected in this study could be the result of improvements in analytical techniques or may reflect variability in the shale. For the Bringelly Shale additional tests were performed on extremely weathered material from the four quarry sites and these indicated weathering is associated with changes in mineralogy. It was found that chlorite and some illite had been broken down, with a corresponding increase in mixed layer clay minerals and a small montmorillonite fraction forming. These changes will tend to increase the reactivity of the residual soil compared to the parent shale. This trend is evident in the liquid limit of the crushed shale which increases from 30 for fresh shale to over 50 for extremely weathered shale. In practice residual soils developed over the Bringelly Shale are suspected to be relatively old as they are commonly leached and laterised so that they are not as reactive as might be expected. Chesnut (1991) notes that soils derived from Bringelly Shale can show significant effects from the presence of expansive clays and the problem is most acute on moderate slopes where the soils are younger and show less evidence of laterisation. Tests were not performed on the extremely weathered Ashfield Shale but it is suspected that even if the illite component breaks down, the amount of reactive clay minerals in the residual soil will be considerably less than for soils derived from Bringelly Shale.

The differences in clay mineralogy are surprising as the accepted wisdom is that the two shales were deposited contemporaneously with the same source material (e.g. Herbert, 1979). The differences in mineralogy cannot be easily explained from differences in depositional environment or diagenesis as the estimated maximum sediment burial is insufficient to lead to significant mineral alteration. Thus it is believed that the Bringelly Shale must have had some different source material to the Ashfield Shale.

For Ashfield Shale the microscope studies show that silt-sized quartz particles are present in a clay matrix and that siderite is widely distributed. The shale is highly compacted and this has led to a strong alignment of the clay particles. A similar structure is observed for the Bringelly Shale, however in Bringelly Shale siderite is less significant and planar micro-cracks are observed in the horizontal plane, associated with the clay particle alignment. These micro-cracks are more prevalent in samples with higher porosity and in weathered samples. The extent to which these micro-cracks have been influenced by stress-relief on coring is unclear, but they do support the idea that cementation is weak in Bringelly Shale. In contrast no micro-cracks are evident in the Ashfield Shale.

Both shales have low porosities of between 5% and 12%. However, there is no evidence of any induration of the voids with any cementing agent other than siderite mentioned above. It follows that the low porosities are primarily a consequence of compaction. As noted previously there is some debate about the amount of sediment deposited on top of the shale, and this has been estimated at between 2000 m and 4000 m in some recent studies (Stewart and Adler, 1995, Bai et al., 2001). An estimate of the stress required to produce the observed porosity has been obtained by crushing the Bringelly Shale, reconstituting the material to create a slurry, and then subjecting the slurry to high confining pressures in a triaxial cell. The isotropic compression response, shown in Figure 2, indicates an effective confining stress of about 60 MPa is required to produce a porosity of 10%, similar to the natural material. The isotropic compression response of natural shale is shown for comparison. An effective stress of 60 MPa would require burial depths of the order of 3km to 4km, which is consistent with previous estimates based on geological observations, e.g. Bai et al. (2001).

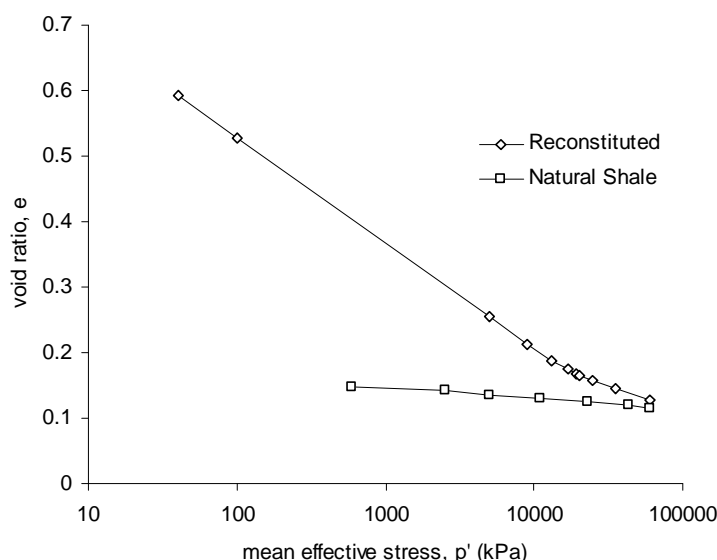


Figure 2: Isotropic compression responses of reconstituted and natural Bringelly Shale.

2.2 SWELLING AND DURABILITY

It has been noted by Chesnut (1991) that both shales weather rapidly on exposure to the atmosphere. The ability to withstand weathering can be assessed through standard slake durability tests. These have been performed for both Ashfield and Bringelly shales following recommended ISRM (1978) procedures. Figure 3 shows the variation of the 2-cycle slake durability with the degree of weathering, estimated using conventional observational methods. It can be seen that the durability of Bringelly Shale is significantly less than for Ashfield Shale and this durability decreases rapidly with the degree of weathering. The classification scheme suggested by Franklin and Chandra (1972) was adopted to assess the durability of Bringelly shale. Based on the test results, the durability of Bringelly Shale varies from medium for fresh intact shale to very low for extremely weathered shale, whereas for Ashfield Shale the durability varies from high to low. Some of the differences between the durability of the two shales can be ascribed to the differences in mineralogy and the different cementing agents.

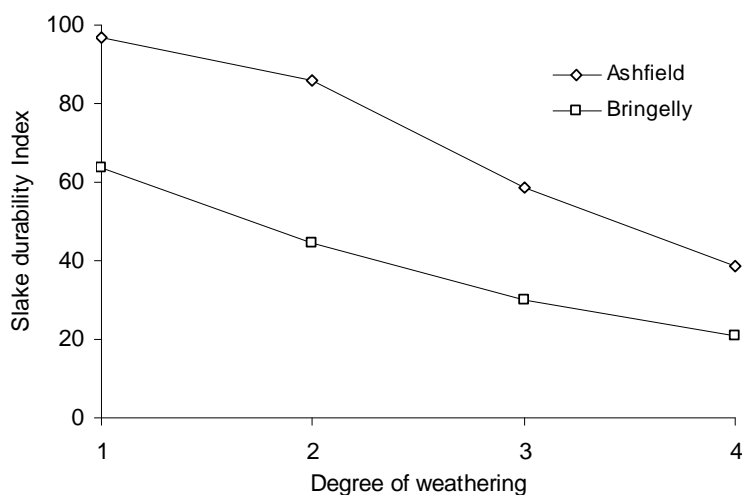


Figure 3: 2-cycle slake durability index for different degrees of weathering (1=fresh, 4 = extremely weathered).

To investigate the mechanisms responsible for the slaking of the shales further tests were performed to assess the free swell potential and the influence of pore fluid chemistry on the swelling of the shale. Figures 4 and 5 show some typical results. Figure 4 shows strains for a period of 1 day in free swell tests on small cubes (3 cm×3 cm×3 cm) of the two shales immersed in tap water. For Ashfield Shale the strains were isotropic and small and no further deformation was observed for a further 5 days after which the test was terminated. For Bringelly Shale, much larger volumetric swelling

strains were measured. Deformation ceased after a few hours until after about 36 hours the sample lost its integrity with some clay dispersing into the water. Figure 6 shows the effect of 48 hours immersion in water on a cylindrical sample.

Figure 4 shows that much greater swelling strains are measured for Bringelly Shale than Ashfield Shale and that for the Bringelly Shale the strains are anisotropic, developing predominantly in the vertical direction. As noted previously Bringelly Shale samples have horizontal micro-cracks and opening of these cracks could explain the greater vertical strains. Other possible reasons for the difference in the behaviour include swelling due to a reduction in effective stress associated with a reduction in pore water suctions and osmotic effects due to differences in chemistry of pore fluid and swelling liquid. The results shown in Figure 4 are typical of swelling tests on Bringelly Shale, however the magnitude of the swelling strain increased as sample dimensions decreased. The influence of sample size on the swelling of Bringelly Shale is believed to result from the opening up of micro-cracks during specimen preparation. If this is the case the amount of swelling in fresh shale exposed to water may be expected to depend on the excavation procedure.

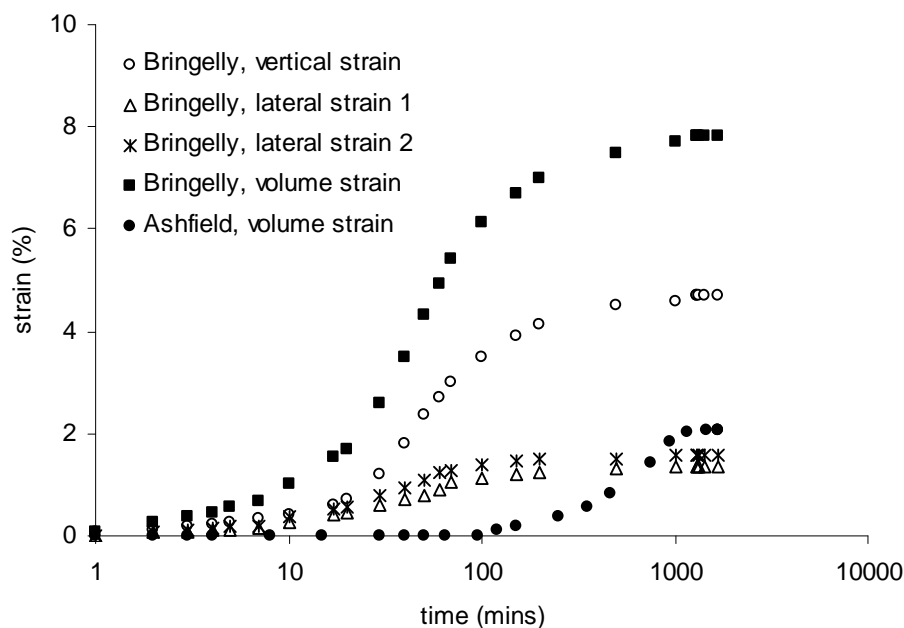


Figure 4: Strains during free swell tests on cuboidal samples immersed in tap water.

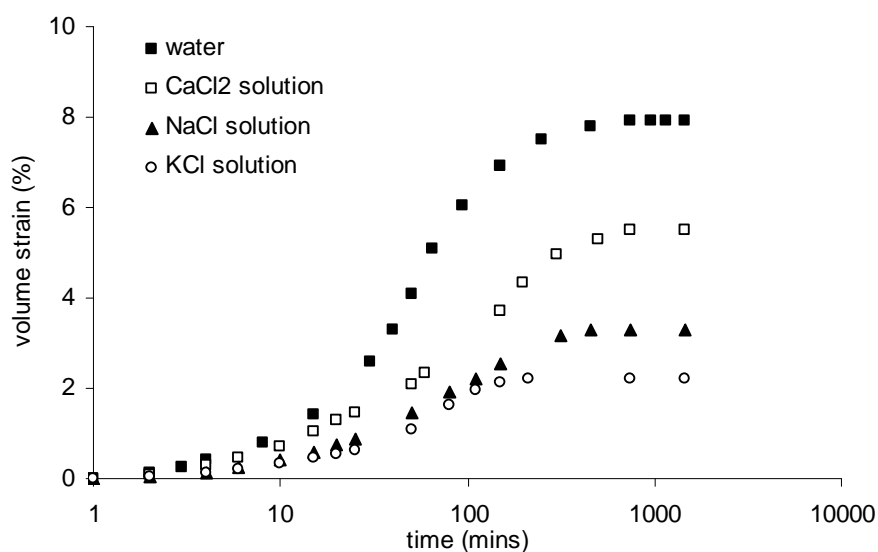


Figure 5: Influence of pore fluid on swelling of Bringelly Shale.

Samples of both shales were obtained from locations above the groundwater table. It was found that Ashfield Shale samples were close to being fully saturated, whereas Bringelly Shale samples had variable degrees of saturation from 50 to 80%. Tests on Bringelly shale from two sites indicated very high total suctions of 150 MPa (6.2 pF). Differences between the shales are related to the presence of micro-cracks which are assumed to facilitate drainage from, and allow air entry to, the Bringelly Shale. The very low porosity and clay fabric ensure relatively high degrees of saturation in both shales. The large suction in Bringelly shale will contribute to the swelling when samples are immersed in water and, as discussed further below, enhance the strength and stiffness of the in-situ shale.

Previous studies of groundwater in Bringelly Shale have indicated that the water is generally saline and unsuitable for water extraction (PPK, 1999). Salinities between 5000 and 26000 ppm were reported by PPK. The pore fluid extracted from one sample indicated a salinity of 1760 ppm, much less than the values in the groundwater.

Because of the presence of reactive mixed layer clays in the Bringelly Shale it can be expected that changes in pore fluid will contribute to volume changes. Figure 5 shows the results of free swell tests with Bringelly Shale in a range of pore fluids with 1 molar concentrations. Tap water and CaCl_2 cause the most swelling and significant deterioration occurs as shown in Figure 6. The least swelling occurs with the potassium chloride solution, and no deterioration occurred when the shale was left in this solution for several days. This is expected as potassium muds are widely used to ensure hole stability in reactive clay shales. The trends in Figure 5 are consistent with double layer interactions, and demonstrate the importance of pore fluid chemistry in controlling the magnitude of any swelling strains. However, the relative contributions of suction and osmosis to the swelling cannot easily be determined from these tests as the NaCl concentration used was 58000 ppm, which was much higher than the pore water concentration.

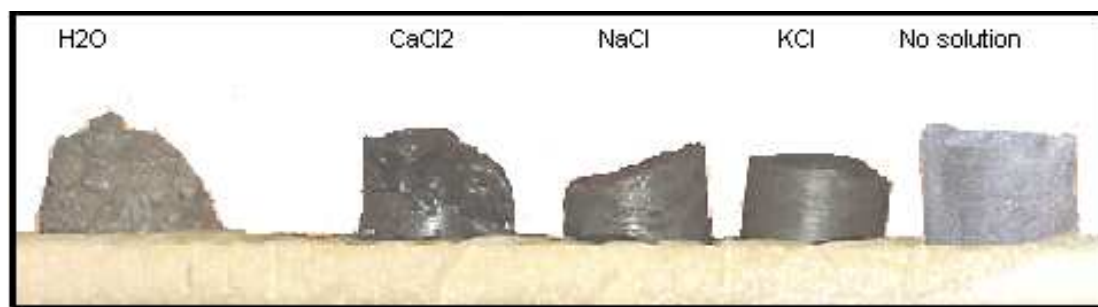


Figure 6: Samples of Bringelly Shale after immersion in different pore fluids,

2.3 SAMPLING THE WIANAMATTA GROUP SHALES

Samples of both shales have been obtained from block samples and by coring *in situ*. To obtain laboratory specimens from the block samples cores were drilled using water flush. With Ashfield Shale these techniques worked and reasonable lengths of core could be obtained *in situ* with very little core loss. In contrast for Bringelly Shale significant amounts of core were lost and very few lengths of core greater than 100 mm were obtained. This appears to be typical of coring operations in Bringelly shale. Attempts were made in the laboratory to use potassium chloride solutions to minimize swelling during coring, but this did not significantly improve core recovery and was not pursued. Nevertheless, it is believed that the use of potassium drilling muds would improve core recovery *in situ* as this would minimize the swelling and deterioration of the shale. The greater ease of sampling of Ashfield Shale can be explained because it is more cemented and lacks the reactive clay minerals present in Bringelly Shale.

3 STRENGTH

3.1 INDEX STRENGTHS

Previous studies (Won, 1985; Chesnut, 1991) have indicated that the strengths of Bringelly and Ashfield Shales are similar. Figure 7 shows some typical data obtained from UCS tests in the current study and from other agencies. Ghafoori et al., (1993a) showed that the data for Ashfield Shale lies in a narrow band and the unconfined compressive strength, σ_c can be described by the equation:

$$\sigma_c = 600 p_a e^{(-0.415m)} \quad (1)$$

where m is the moisture content, and p_a is atmospheric pressure.

For Bringelly Shale the same general trend of decreasing strength with increasing moisture content is apparent but there is much greater variation. This can partly be explained by the differences in degree of saturation of the Bringelly Shale

samples compared to Ashfield Shale, for which all samples were close to full saturation. As discussed further below the apparent similarity of the two shales obscures some important differences in the mechanisms leading to their strengths. One of the difficulties with obtaining UCS data for Bringelly Shale is that core recovery is low, and thus the number of samples suitable for UCS testing is limited. This may have the result that only the more cemented material is tested and the strength is over-predicted.

Because of the difficulty of performing UCS tests there is a widespread practice of relying on point load index tests and relating these to a UCS value on the basis of an empirical correlation. Comparisons between the UCS strength and the axial point load index are shown in Figure 8. Ghafoori (1995) has shown that the relation for Ashfield Shale is given by $UCS = 24 I_{sa(50)}$. For Bringelly Shale the relation shown in Figure 8 can be described by $UCS = 21 I_{sa(50)}$. It should be noted that the data is very limited and as mentioned above may be being biased towards more cemented material.

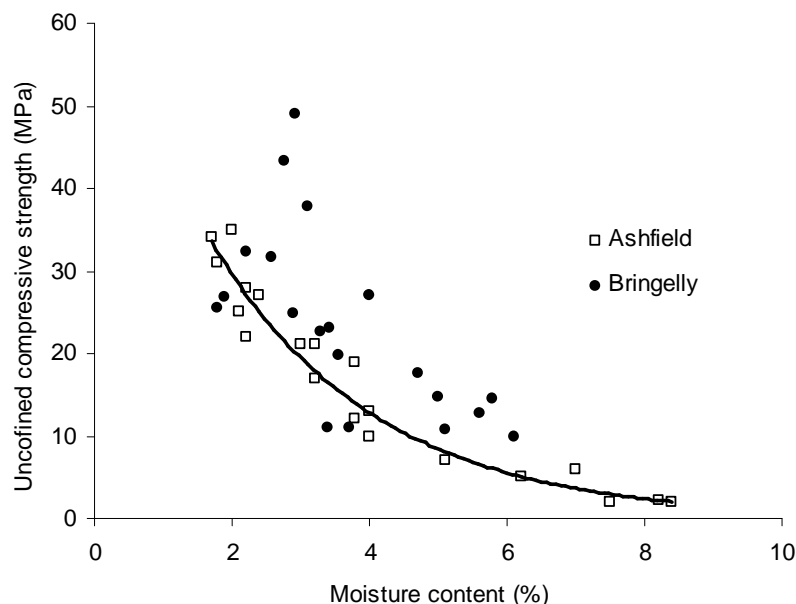


Figure 7: Comparison of UCS versus moisture content for Ashfield and Bringelly Shales.

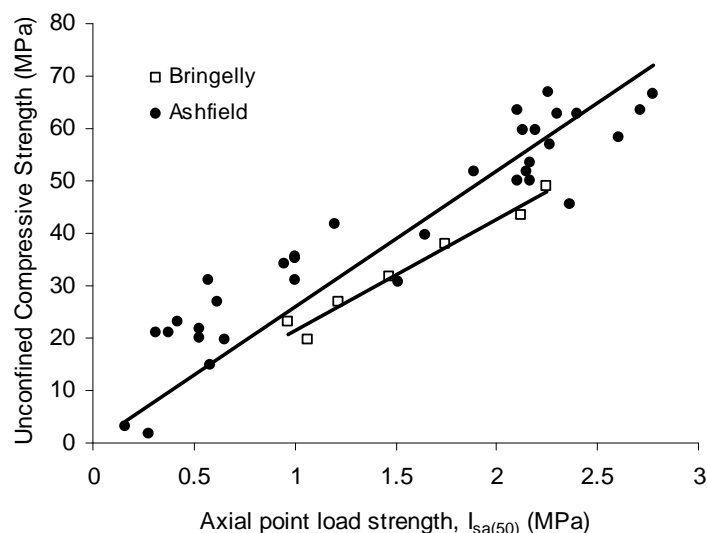


Figure 8: Comparison between UCS and axial point load strengths.

Both diametral and axial point load tests have been performed for the two shales. Broch and Franklin (1972) suggested that the strength anisotropy can be related to the ratio of axial to diametral point load strengths which they termed the anisotropy index I_a . For Bringelly Shale the anisotropy index has a mean of 3.0 and standard deviation of 1.4. For Ashfield Shale the mean is 2 and the standard deviation 0.5. It is evident that there is much greater variability in the diametral point load results for Bringelly Shale, which is believed to reflect the varying extent of micro-cracking which is controlling the diametral strength.

For Ashfield shale a number of direct shear tests have been reported by Ghafoori et al. (1993b). These indicated shear strengths on the horizontal plane, aligned with the silty bands of about 2 MPa, whereas the strength perpendicular to the laminations was about 4 MPa. This is consistent with the strength anisotropy index determined in the point load index tests.

3.2 TRIAXIAL TESTS

Triaxial tests have been performed on both Ashfield and Bringelly Shales. Ashfield Shale shows a conventional soft rock response with a fairly linear stress, strain response up to a peak followed by a pronounced strain softening. Ghafoori (1995) showed that the triaxial strength was well described by Bieniawski's criterion

$$\frac{\sigma_1}{\sigma_c} = 1 + B \left(\frac{\sigma_3}{\sigma_c} \right)^\alpha \quad (2)$$

with $B = 3$ and $\alpha = 0.75$ and where the unconfined compressive strength σ_c is a function of moisture content given by equation 1. Ghafoori (1995) also showed how equation 2 could be simply modified to allow for the anisotropic strength caused by the laminated structure.

A comparable set of tests could not be performed for Bringelly Shale because of the difficulty in obtaining samples large enough to be suitable for triaxial testing. As a result only a limited number of tests have been performed on vertical core samples. The rationale for these tests has been to investigate how the Bringelly Shale, which apparently has little cementation, can give UCS strengths of up to 50 MPa. Triaxial tests have been performed on shale at the *in situ* moisture content, on the natural shale after saturation and on material that has been reconstituted.

In order to perform tests on saturated specimens it was necessary to subject them to an effective confining stress of 600 kPa before saturation. It has already been shown that with no confining stress the specimens will disintegrate on swelling and tests by Itakura (1999) showed partial disintegration occurred during saturation at an effective stress of 200 kPa. Triaxial tests have been performed with effective confining stresses from 600 kPa to 60 MPa. The failure points from some of these tests are shown in Figure 9. This figure shows the very dramatic effect of saturating the specimens. At a confining stress of 600 kPa the peak deviator stress drops from 15 MPa to 5 MPa. There are two possible reasons for this large drop, one is a reduction in effective stress because of the removal of suctions and the other is that the strains associated with saturation and effective stress reduction cause the material cementing the rock to break down. A comparison of the deviator stress, strain responses at the *in situ* moisture content and after saturation is shown in Figure 10a for a confining stress of 1 MPa. This figure shows that the initial stiffness is not greatly affected by the degree of saturation, but the strength is reduced considerably. This suggests some cementation is present in the shale as, if it was uncemented, the stiffness would be expected to vary with effective stress, with the higher effective stresses in the unsaturated specimens giving rise to higher stiffnesses.

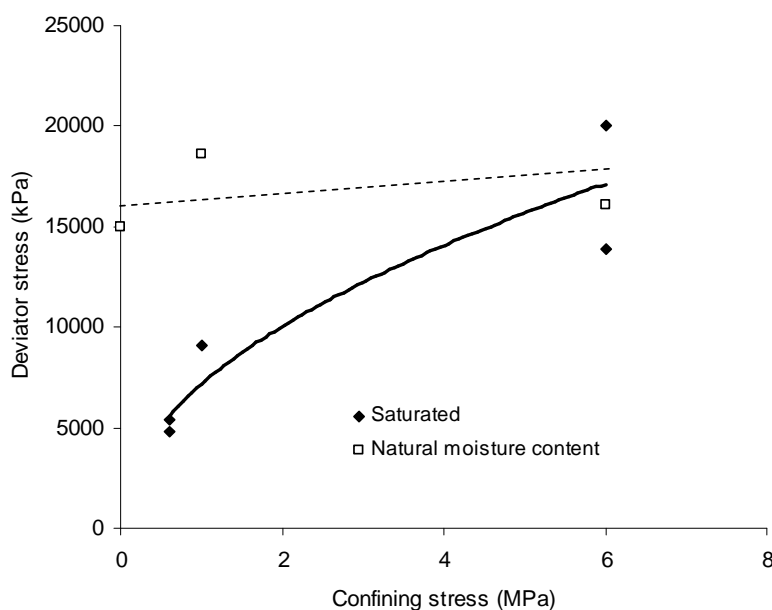


Figure 9: Effect of confining stress and saturation on failure of Bringelly Shale.

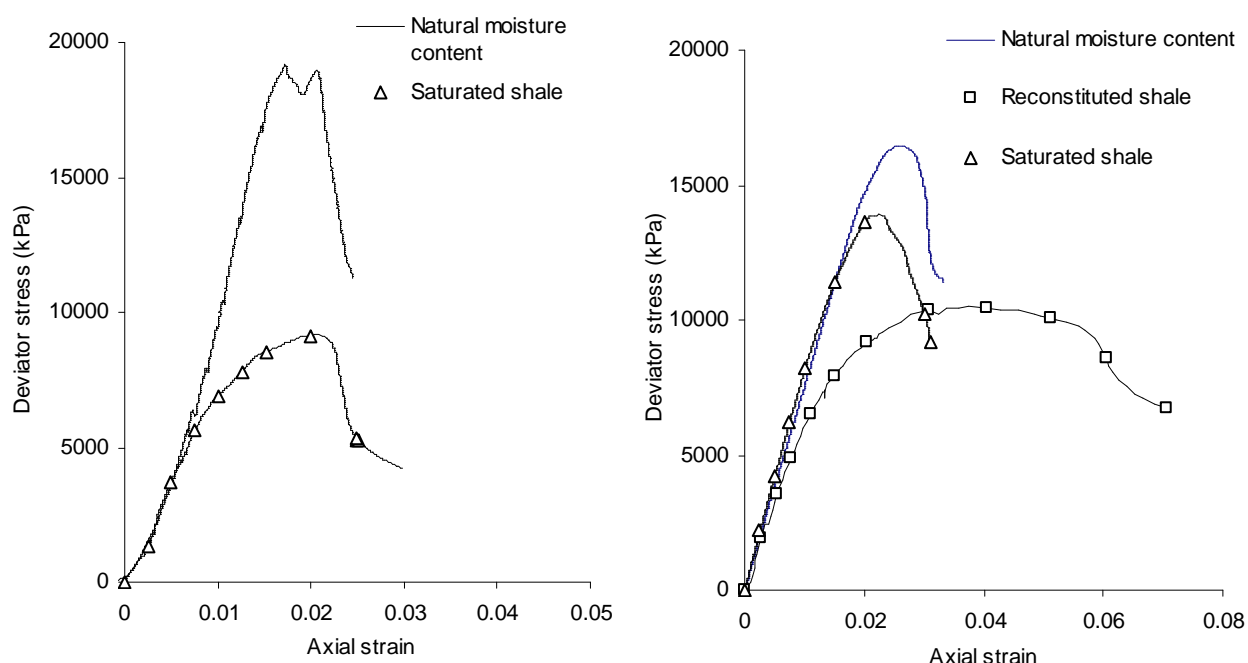


Figure 10 Stress, strain responses from triaxial tests, (a) confining stress = 1 MPa, (b) confining stress = 6 MPa

Because the cementation appears to be weak and because of the difficulty of obtaining large enough samples for triaxial testing, specimens of Bringelly Shale were reconstituted by breaking down the shale and reconstituting it with water. A series of standard drained and undrained triaxial tests were performed on saturated specimens with over-consolidation ratios up to 10 with a maximum effective confining stress of 1 MPa. These gave results consistent with many other reconstituted materials and a normalized response as shown in Figure 11. The ultimate or critical state friction angle estimated from these tests is 28.5° . Tests were performed in shear box and ring shear apparatus and these also indicated a residual friction angle of about 28° . This was much greater than the friction angle of 16.5° determined from tests on the saturated shale. In order to investigate whether this behaviour was a consequence of a low void ratio, the reconstituted material was isotropically compressed to an effective pressure of 60 MPa at which the void ratio was 0.1, similar to the natural shale. Tests were performed on normally and over-consolidated specimens with this pre-consolidation stress.

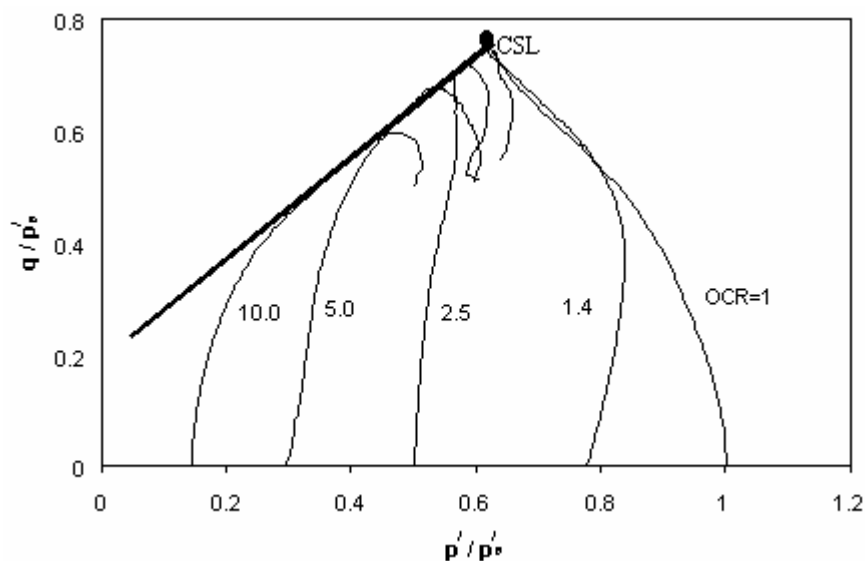


Figure 11: Normalised behaviour of reconstituted material with maximum stress = 1 MPa.

The results of these tests are compared with the saturated shale in Figure 10b. It can be seen that the reconstituted shale now gives strengths comparable with the *in situ* material and the ultimate friction angle reduces to about 17° , similar to

that for the natural shale. Figure 10b also shows that the differences between the intact shale and the reconstituted material are relatively minor when the confining stress reaches 6 MPa. This is another indication that the cementation is relatively weak in this material and that strength is controlled mostly by frictional effects.

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 normalized failure surface of their 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 a mechanism where deformation and strength are controlled by movements along joints and fissures. They also noted that OCR did not seem to significantly affect the strength. The results presented in Figures 9 to 11 show the same pattern of behaviour as reported by Picarelli et al. The observation that the reconstituted shale also has a low frictional resistance 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. 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 12 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_u , 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. In addition it has significant micro-cracking in the plane of the laminations. At confining stresses of 6 MPa and above there is no difference in the ultimate friction angles of the reconstituted and natural material. It is possible that tests on natural shale cores oriented at other angles to the vertical may show even lower frictional strengths when failure surfaces are aligned with the micro-cracks.

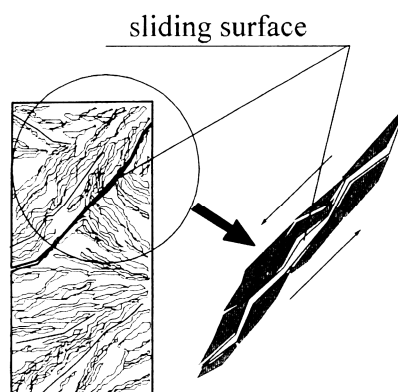


Figure 12: Mechanism of shear deformation and rupture (after Picarelli et al., 1998).

4 STIFFNESS

For Ashfield Shale estimates of modulus were obtained from strain gauges stuck to the shale specimens. For Bringelly Shale the lack of suitably sized samples has meant that only external measurements of deformation have been obtained, and the values are expected to under-estimate the true stiffness of the shale. Consequently, they cannot be directly compared with values reported for Ashfield Shale.

Ghafoori (1995) showed that the stress, strain response of the Ashfield Shale was consistent with a cross anisotropic material with a ratio between modulus in the plane of the laminations and in the plane perpendicular to the laminations of 3. Young's modulus for specimens tested perpendicular to the laminations varied from 3 GPa to 7 GPa in uniaxial compression and these values increased with increasing confining stress according to a power law of the form

$$\frac{E}{P_a} = A \left(1 + \frac{\sigma_3}{P_a} \right)^n \quad (3)$$

where A and n are constants. It is believed A depends on moisture content in a similar way to UCS but insufficient data are available to provide a useful relation: $n = 0.035$ from available data.

Table 3 shows values of E_{50} determined from tests on intact and reconstituted Bringelly Shale. These values are about an order of magnitude lower than those recorded for the Ashfield Shale. It may be noted that at a confining stress of 6

MPa, where direct comparison is possible, the stiffness of the reconstituted and natural shales are similar. These observations are consistent with the weaker cementation in the Bringelly Shale. The highest stiffness is indicated for the shale at the natural moisture content at low confining stress where pore water suctions are most significant.

Table 3: Values of E_{50} determined from tests on natural and reconstituted Bringelly Shale.

Effective confining stress (MPa)	Shale at natural moisture content (MPa)	Shale saturated (MPa)	Reconstituted, void ratio = 0.1 (MPa)
0	2300	-	-
1	1500	738	-
6	751	823	627
60	-	2800	1500

5 DISCUSSION

Both Ashfield and Bringelly Shale have very low porosity and similar unconfined strengths, but in almost all other aspects there are significant differences between the two materials. Bringelly Shale is comprised of mixed layer clay minerals which have greater potential for swelling and physico-chemical changes than the clay minerals in Ashfield Shale. Bringelly Shale contains less siderite than Ashfield Shale in which siderite acts as an effective cementing agent. Weak cementation may be present in Bringelly Shale but this is difficult to detect with certainty in the laboratory. This leads to poor core recovery, particularly when water is used as drilling fluid, as the shale can swell and disintegrate when the confining stress is removed by coring. When exposed to the atmosphere Bringelly Shale degrades more rapidly than Ashfield Shale.

An important factor controlling the strength and stiffness of Bringelly Shale appears to be pore water suction. Despite reasonably high degrees of saturation the shale has a very high total suction. When the shale is saturated some of this strength and stiffness is lost. As the ground-water table is located at depths of 20 to 40 m over much of Western Sydney, when Bringelly Shale is encountered in construction it is likely to be unsaturated. Some caution is recommended, therefore, if it is proposed to use high UCS values and stiffnesses in foundation design.

This study has been limited to the claystone-siltstone materials that comprise the majority of the Wianamatta group shales, and has only considered the intact rock properties. The influence of joints and fissures has not been considered. Within these limitations some general observations can be made concerning the implications of these results for construction on and with these shales.

1. There are significant differences in the engineering behaviour of Ashfield Shale and Bringelly Shale, and identification of the shale type should be required before construction.
2. Bringelly Shale, and residual soils derived from it, contain reactive swelling clay minerals. Construction on residual soil will require special attention, particularly where the soils have not been affected by laterisation.
3. Removal of residual soil to found structures on the underlying shale will not eliminate ground movements because Bringelly Shale has the potential to swell if water is provided to it and stress levels are low.
4. High strength and stiffness derive in part from pore water suction. These values cannot be relied upon if environmental conditions change.
5. Use of Bringelly Shale as a fill material is not recommended as it deteriorates rapidly in the presence of water and is prone to swelling.

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7 REFERENCES

- Bai, G-P., Hamilton, P.J., Eadington, P.J. and Keene, J.B. 2001. Fluid flow histories and diagenesis in permo-triassic sediments of the Sydney basin, SE Australia – Isotope and fluid inclusion constraints, PESA Eastern Australian Basins Symposium, Australian Institute of Mining and Metallurgy, 251-261
- Broch, E. and Franklin, J.A. 1972. The point load strength test, *International Journal of Rock Mechanics and Mining Sciences*, 9, 669-697
- Chesnut, W.S. 1983. "Geology of the Sydney 1:100,000 sheet" New South Wales Geological Survey, Report No. 9130, 182-99

- Chesnut, W.S. 1991. Engineering Geology, Geology of the Penrith 1:100,000 sheet, New South Wales Geological Survey, Report No. 9030, 166-178
- Franklin, J.A. and Chandra, A. 1972. The slake durability test, *International Journal of Rock Mechanics and Mining Sciences*, 9, 325-341
- Ghafoori, M. 1995. Engineering behaviour of Ashfield Shale, PhD thesis, University of Sydney
- Ghafoori, M., Airey, D.W. and Carter, J.P. 1993a. Correlation of moisture content with the uniaxial compressive strength of Ashfield shale, *Australian Geomechanics*, 24, 112-114.
- Ghafoori, M., Airey, D.W. and Carter, J.P. 1993b. Anisotropic behaviour of Ashfield shale in direct shear test, *Geotechnical Engineering of Hard Soils-Soft Rocks*, Anagnostopoulos et al. (eds), Balkema, 1, 509-515
- Herbert, C. 1979. "The geology and resource potential of the Wianamatta Group", New South Wales Geological Survey, Bulletin 25, 203 p
- ISRM, 1978. Suggested methods for determining swelling and slake durability index properties, *International Journal of Rock Mechanics and Mining Sciences*, 16, 151-156
- Itakura, T. 1999. Transport of organic contaminants through natural clayey soils, PhD thesis, University of Sydney
- Jones, D.C. and Clark, N.R. 1991. Editors, Geology of the Penrith 1:100,000 Sheet 9030, New South Wales Geological Survey, Sydney
- Loughnan, F.C. 1960. The origin, mineralogy and some physical properties of the commercial clays of New South Wales, University of New South Wales, Geological Series, 2, 348p
- Picarelli, L., Olivares, L., Di Maio, C. and Urciuoli, G. 1998.. Properties and behaviour of tectonised clay shales in Italy. Proc. 2nd Int. Symp. Geotechnics of Hard Soils – Soft Rocks, Napoli, 3: 1211-1241
- Picarelli, L., Olivares, L., Di Maio, C., Silvestri, F., Di Nocera, S. and Urciuoli, G. 2003. "Structure, properties and mechanical behaviour of highly plastic intensely fissured Bisaccia Clay Shale". Characterisation and Engineering Properties of Natural Soils-Tan et al. (eds). Swets & Zeitlinger, 947-981
- PPK, 1999. Hydrogeological investigations- Final report (unpublished)
- Stewart J.R. & Alder J.D. 1995. New South Wales Petroleum Potential .NSW Department of Mineral Resources, Sydney, Petroleum Bulletin 1, 188p
- William, E. and Airey, D.W. 1999. A Review of the Engineering Properties of the Wianamatta Group Shales, Proc. 8th Australia-New Zealand Conf. on Geomechanics, Hobart, 2, 641-647
- Won, G.W. 1985. "Engineering properties of Wianamatta group rocks from laboratory and in-situ tests", *Engineering Geology of the Sydney Region*, (Ed. P.J.N.Pells), 143-161