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An Experimental Investigation into the Deformation Characteristics of Sandstone under Mine Dewatering

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ABSTRACT

This paper reports the findings of an investigation carried out to evaluate the deformation characteristics of saturated sandstone aquifers in the Collie Coal Basin under conditions anticipated during dewatering operations.

A drained triaxial test technique for various combinations of confining pressure and back pressure has been designed to examine the applicability of the effective Stress Thoery suggested by Terzaghi (1936), with particular reference to the Collie Sandstone. Further, the results obtained have been utilised to characterise the strength and deformability properties of Collie Sandstone.

The influence of stress ratio and confining pressure on deformability modulus has been examined and a mathematical function for the non-linear stress-dependent characteristics of the Collie sandstone is proposed.

This paper contains a description of the equipment commissioned, test techniques, results, analysis and interpretation of the data obtained.

Because the testing evaluation techniques are general in nature, they can be applied to field situations in locations where similar weak sandstones occur.

INTRODUCTION

The Collie Coal Basin is located approximately 200 km south-east of Perth in Western Australia. It contains extensive reserves of good steaming coal which is currently being mined by both open cut and underground methods.

The Collie Coalfield has a long history of strata control problems. They manifest themselves in the form of localised poor roof control, surface subsidence, slope instability and mine abandonment (due to a sand-slurry inrush). Major sources of these problems include the very extensive, weak, saturated, sandstone aquifers. As a result, underground operations have been limited to room and pillar extraction, presently carried out by continuous miners and road-heading machines. Approximately 30-40% recovery by volume is being achieved by this method.

In order to increase the recovery to approximately 70%, the Wongawilli method of short-wall mining has been introduced. Caving of the immediate roof is integral with this method. Extensive aquifer de-watering was carried out to enable this mining method to be applied. The porous and weak nature of the aquifers provides a potential source of subsidence (due to pore closure), and strata failure (due to increasing the effective stress), as a result of pore pressure reduction upon de-watering. The proposed development of multiple seam extraction below areas sensitive to surface subsidence has increased the need to establish the strata mechanical properties. This will assist in confident application of rock mechanics principles for predictive modelling of strata behaviour. The importance of a knowledge of the effects of induced stress changes as a result of de-watering a weakly-cemented sandstone aquifer has been previously referred to by Kawecki et al. (1988). The two aspects that are of primary interest are (1) deformations produced by changes in the state of stresses of the aquifer as a result of de-watering, and (2) an erosional effect due to the flow of water induced by de-watering, which would tend to exacerbate any detrimental effect of pore pressure reduction.

Although seepage and drainage of both soils and hypothetical media have been investigated theoretically and experimentally by geotechnical engineers, drainage of poorly-consolidated rock has been neglected. There is little reference in the literature to the basic properties of rock deformation caused by migration of rock particles. Furthermore, most failure theories and experimental work are related to non-porous and low-porosity rock types, while major problems during aquifer de-watering relate to relatively weak formations such as loose sand and/or poorly-consolidated sandstone, as is the case in the Collie Basin.

The role of fluid and fluid pressure on the deformation behaviour of rock is often an important consideration in studies of rock mass stability. The influence of pore pressure, particularly that of pore pressure on the strength of rocks has been reported by Handin, and Hager (1957), Robinson (1959), Heard (1960), Serdengecti et al. (1962), Handin et al. (1963), Jaeger (1966), Vutukuri et al. (1974), and others. These investigations have often produced conflicting results and have tended, particularly in the region of less porous rocks, to cause confusion as the validity of the effective stress principle.

A drained triaxial compression test technique for various combinations of combining pressure and back pressure has been designed to examine the applicability of the effective Stress Theory suggested by Terzaghi (1936), with particular reference to the Collie Sandstone. Further, the results obtained have been utilised to characterise the strength and deformability properties of Collie Sandstone.

The influence of stress ratio and confining pressure on deformability modulus has been examined and a mathematical function for non-linear stress-dependent characteristics of the Collie sandstone is proposed.

The paper contains a description of the equipment commissioned, test techniques, results, analysis and interpretation of the data obtained.

The testing evaluation techniques are general in nature and can be applied to field situations in locations where similar weak sandstones occur.

GEOLOGY

The geological formation, in the Collie Basin is the sedimentation of an impervious igneous basin with the water trapped within the sediments leaving the sediments heavily saturated from basement to water table. The water table lies in some proximity to the river drainage system, and so is reasonably close to the surface.

The ground water is contained within the Collie Coal Measures, which constitute a multi-layer of sandstone beds separated by confining beds of shale, mudstone and coal beds. Sandstone and grits in the Collie Basin constitute approximately 65 to 75 percent of the coal measures. The water influence has prevented the normal consolidation of the sediments and the sandstone and siltstone are very weak, with little adhesive characteristics. The resultant hydraulic pressure with an absence of consolidation presents a formidable mining problem, which greatly increases with depth.

TRIAXIAL EQUIPMENT

The scope of the research required unique triaxial testing of rock. As no commercial system was available, a system with the appropriate capabilities was designed.

An automated data capture system utilising transducers and dynamic recording were designed and commissioned. The overall system was designed to withstand a maximum predicted hydraulic pressure of 14 MPa.

While the equipment developed for use in this and in other associated programmes of research was similar in principle to that described by Bishop and Henkel (1962), a number of refinements had to be introduced because of the markedly higher strength and stiffness characteristics of soft rock which required the use of significantly higher testing pressures.

The system developed for this study consisted of six integrated units :

1. A triaxial cell
2. A confining pressure system
3. A pore pressure system
4. A stiff load frame and servo-controlledloading ram
5. Monitoring equipment for applied loads, water flows, and specimen deformation, and
6. Data capture and display system

Full details of the equipment design may be found in Nikraz (1991).

DRAINED TRIAXIAL COMPRESSION TEST

Thirty two specimens of Collie sandstone were prepared and tested at various combinations of cell pressure and pore pressure to evaluate the applicability of the effective stress theory and also to characterise the strength and deformability properties of the sandstone.

The drained triaxial compression tests consisted of four stages, namely :-

1. specimen mounting in triaxial cell
2. specimen saturation
3. back pressure adjustment
4. load application

Only a brief description of test procedures will be presented herein but full details may be found in Nikraz (1991).

After the specimen was mounted in the triaxial cell and saturated with water, a predetermined confining pressure was applied and the back pressure was adjusted to the desired level. Vertical load was only applied after the confining pressure had been adjusted. During the test the confining pressure and the back pressure were maintained at a constant level. The specimen then tested to failure by application of a constant vertical loading rate (3 x 10^{-4} strain/min).

The back-pressure connected to the drainage outlet, the function of which can be described as to equate levels of the test's pore water pressure to in situ pore water pressure. However, in absence of detailed information with regard to in situ pore water pressure, different levels of pore water pressure and thus, back pressure were adopted.

Using the back pressure two further advantages were envisaged:

(1) to dissolve any residual air in the pipework connections and in the specimen, thus ensuring full saturation, and

(2) to prevent negative gauge pressures and cavitation effects of specimens that tend to dilate during shearing, thus putting the pore water into tension.

RESULTS ANALYSIS

The stress/strain relationships typical of sandstones tested in drained triaxial compression at various combinations of cell pressure and pore pressure are shown in Figures 1. These curves emphasise the significant increase in triaxial strain at failure as the effective confining stress (σ_3') increases. The variation of axial strain at failure $\varepsilon_{1(F)}$ is shown in Figure 2.

The data in Table 1 indicates that the maximum differential stress is essentially the same for tests with equal effective confining pressures and increases with an increase in effective confining pressure. Thus, it may be concluded that the effective stress theory is valid for the specimens tested.

This theory states that the effective stress is the difference between the total stress and the pore pressure and is the controlling factor influencing frictional strength of rock, other parameters remaining constant.

Comparing the stress/strain behaviour, as shown in Figure 1, with that of strong rocks, as is documented by Jaeger & Cook (1976), and Brady & Brown (1985), it can be noted that for the sandstone tested no clear transition is apparent due to their increased plasticity. In contrast in competent strata, the transition from elastic to plastic behaviour is readily recognisable.

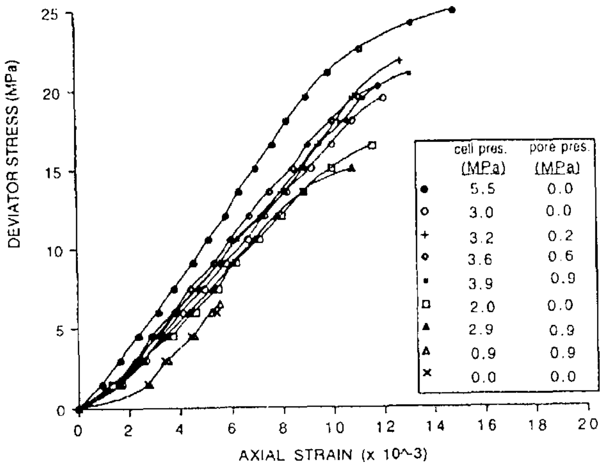

FIGURE 1 - Typical average stress-strain curves for drained test specimens with the same effective confining pressure.

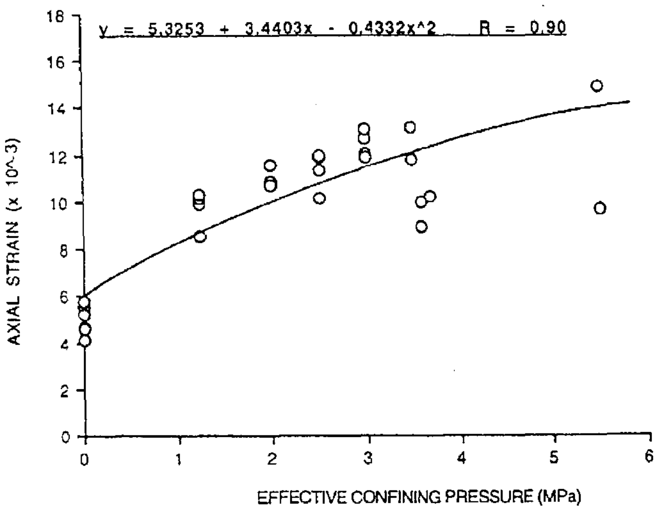

FIGURE 2 - Variation of axial strain at failure with effective confining pressure.

The source of this plasticity is the non-recoverable strain arising from the closure of pores and fissures and the onset of microfractures with increased stress. The magnitude of plastic deformation depends on rock type, stress level and rock environment (Brace, 1965; Walsh, 1965; Simmons et al., 1974; Podneiks et al., 1968; Van Eeckhout & Pong, 1975; and Priest & Selvakumar, 1982).

Classification of the rocks according to the method of Deere and Miller (1966) showed them to have a lower modulus ratio than that defined for weak rocks (see Figure 3). The modulus ratio is relatively constant (between about 160 and 360) irrespective of the strength or the modulus. This, however, means that the modular ratio is less suitable for characterising and classifying the sandstone in the Collie Basin.

The influence of effective confining stress and deformability modulus is illustrated in Figure 4. The deformability modulus varies considerably with the increase in effective confining pressure (σ_c'). The tangent modulus was obtained from triaxial stress/strain curves at 50% of the peak strength. Volumetric strain was not monitored and therefore it is not known if these values are above the onset of dilatancy.

Furthermore, typical variations of strains with stress ratio, $K = \sigma_3'/\sigma_1'$ under different confining pressures are shown in Figure 5. This indicates a parabolic trend with strain becoming excessive as the stress ratio approaches the failure ratio, K_f. To analyse the data mathematically a function of type :-

$$\varepsilon_{1\,(K)} = a \ln \frac{1}{(K - K_f)} \qquad (1.1)$$

was found to be reasonably adequate (Figure 6).

Where :-

$\varepsilon_{1(K)}$ = Axial strain at any stress ratio

K = Stress ratio, $K = \sigma_3'/\sigma_1'$

K_f = Stress ratio at failure

σ_1' = Effective axial pressure

σ_3' = Effective confining pressure

a = Constant depends on the observed stress/strain characteristics of the rock. Typical values for 'a' are presented in Table 2.

TABLE 1 - Comparison of drained tests at equal effective confining pressures.

Specimen Location	Depth (m)	Effective Confining pressure (MPa)	Cell Pressure (MPa)	Pore Pressure (MPa)	Maximum Deviator Stress at Failure (MPa)
Western No. 6	125.00	5.50	5.50	0.00	24.84
		3.00	3.00	0.00	19.37
		3.00	3.20	0.20	21.66
		3.00	3.60	0.60	20.11
		3.00	3.90	0.90	20.87
		2.00	0.00	0.00	16.36
		2.00	2.90	0.90	14.96
		0.00	0.00	0.00	5.44
		0.00	0.90	0.90	6.50
D156	282.76	3.70	3.70	0.00	14.89
		3.70	4.60	0.90	15.12
		2.50	2.50	0.00	11.77
		2.50	3.40	0.90	12.57
		1.25	1.25	0.00	8.73
		1.25	2.15	0.90	8.21
		0.00	0.00	0.00	1.022
		0.00	0.90	0.90	1.80
D157	272.89	5.50	5.50	0.00	23.61
		3.50	3.50	0.00	23.61
		3.50	4.40	0.90	25.12
		2.00	2.00	0.00	18.26
		2.00	2.90	0.90	16.30
		0.00	0.00	0.00	7.43
		0.00	0.90	0.90	6.23
D158	255.54	3.60	3.60	0.00	14.89
		3.60	4.50	0.90	15.53
		2.50	2.50	0.00	14.12
		2.50	3.40	0.90	13.35
		1.25	1.25	0.00	8.47
		1.25	2.15	0.90	9.26
		0	0	0.00	2.65
		0	0.90	0.90	3.10

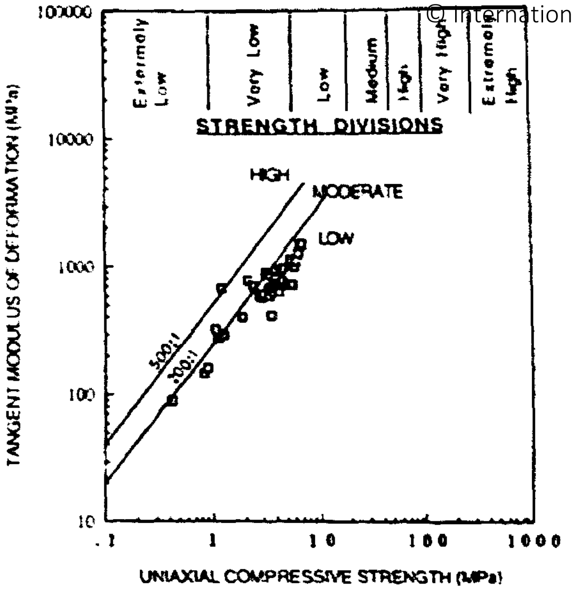

FIGURE 3 - Summary of strength and modulus data for Collie Sandstone (after Deere and Miller, 1966).

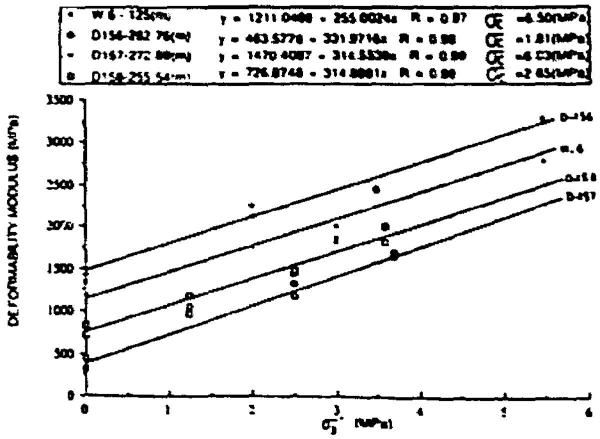

FIGURE 4 - Variation of deformability modulus with effective confining pressure.

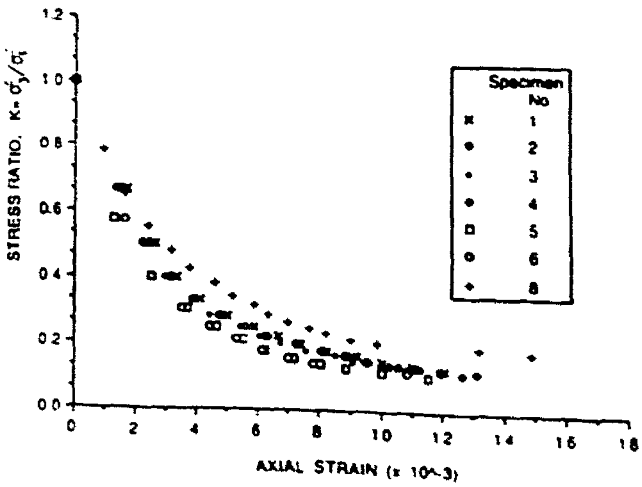

FIGURE 5 - Typical variation of axial strain with stress ratio (bore hole No. - WD.6 - 125.00)

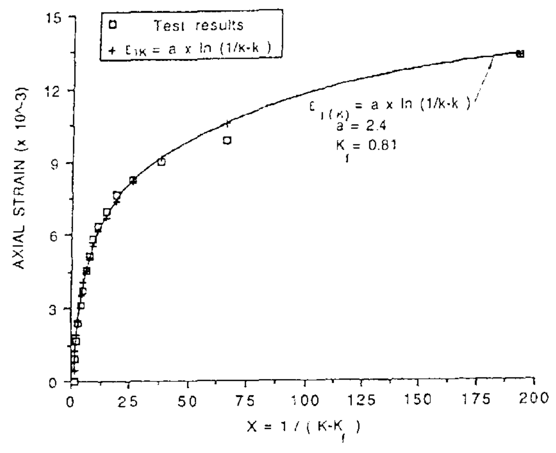

FIGURE 6 - Example of transformed stress-strain curve to determine parameter "a" (specimen No. 8)

TABLE 2 - Parameters 'a' and 'K_f' for W6 - 125 specimens

Specimen No.	Constant Effective Confining Pressure, $\sigma_3 - \sigma_o$ (MPa)	a	K_f
1	3.0	2.6689	0.131
2	3.0	2.6188	0.122
3	3.0	2.5462	0.130
4	3.0	2.6091	0.126
5	2.0	2.3100	0.109
6	2.0	2.2390	0.188
8	5.5	2.4500	0.181

Substitution of the type :

$$x = \frac{1}{(K - K_f)} \qquad (1.2)$$

reduces to :

$$\varepsilon_{1\,(K)} = a \ln x \qquad (1.3)$$

The stress and strain at a point are dependent upon the elastic constants, deformability modulus E and Poisson's Ratio ν, (Timoshenko & Goodier, 1951) and the symmetry attainable with triaxial apparatus, yields the relationship :

$$E\,\varepsilon_1 = \sigma_1' - \nu\, 2\, \sigma_3' \qquad (1.4)$$

and

$$E\,\varepsilon_3 = \sigma_3 - \nu\,(\sigma_1' + \sigma_3') \qquad (1.5)$$

in which ε_1 and ε_3 = the axial and lateral strains respectively.

Rearranging equations (4) and (5) provides :

$$E = \frac{(\sigma_1' + \sigma_3')\,\sigma_1' - 2\,(\sigma_3')^2}{(\sigma_1' + \sigma_3')\,\varepsilon_1 - 2\,\sigma_2'\,\varepsilon_3} \tag{1.6}$$

Equation (1.6) is valid when deformability modulus, E, is material constant. However, as shown in Figure (1) the non-linear relationship between stress and strain for the sandstone tested indicates that deformability modulus is stress-dependent. To account for the non-linear and stress dependent behaviour of the rock, it is assumed that deformability modulus is constant in infinitesimal stress and strain changes and Equation (1.6) can be written as :-

$$E = \frac{(\Delta\sigma_1' + \Delta\sigma_3')\,\Delta\sigma_1' - 2\,(\Delta\sigma_3')^2}{(\Delta\sigma_1' + \Delta\sigma_3')\,\Delta\varepsilon_1 - 2\,\Delta\sigma_3'\,\Delta\varepsilon_3} \tag{1.7}$$

Value σ_3' = constant = σ_0; and failure of the sample with σ_1' increasing, in which σ_0 = initial confirming pressure.

Under constant effective confining pressure the term $\Delta\sigma_3'$ is zero and :

$$\Delta\sigma_1' = \sigma_{1\,(K+\Delta K)} - \sigma_{1\,(K)} = \frac{\sigma_0}{K + \Delta K} - \frac{\sigma_0}{K} \tag{1.8}$$

Rearranging equation (1.8) yields :

$$= -\frac{\sigma_0\,\Delta K}{K\,(K + \Delta K)} \tag{1.9}$$

Also the increment in axial strain, $\Delta\varepsilon_{1\,(K)}$ can be written as :

$$\Delta\varepsilon_{1\,(K)} = +\,\varepsilon_{1(K+\Delta K)} - \varepsilon_{1\,(K)} \tag{1.10}$$

From Equations (1.7), (1.9) and (1.10), and using differential difference equation technique :

$$E = -\frac{\sigma_0}{K^2}\,\frac{1}{\varepsilon'_{1(K)}} \tag{1.11}$$

in which $\varepsilon'_{1(K)}$ is the first derivative of $\varepsilon_{1(K)}$.

From Equation (1) :

$$\varepsilon'_{1(K)} = -\,\frac{a}{\bar{x}} \tag{1.12}$$

in which $\bar{x} = (K - K_f)$.

Values ε_1 expressed as percentage and σ_0:

$$E = \frac{\sigma_0\,\bar{x}}{K^2}\,\frac{1000}{a} \tag{1.13}$$

The variation of deformability modulus of W6 - 125 specimens with stress ratios for various confining pressures are shown in Figure (7). The deformability modulus, E, increases at or near, K = 1, to its peak value and reduces to zero at the stress ratio, $K = K_f$. Figure (7) indicates that the peak deformability values are stress ratio dependent.

More studies are necessary to confirm these results for other types of weak sandstones. However, deformability modulus obtained from a uniaxial or triaxial compression tests must be regarded with reservation.

CONCLUSIONS

The results, analysis and discussions derived in the course of this study allow the following conclusions to be drawn.

(a) A drained triaxial compression test at various combinations of confining pressure and back pressure has been developed to attempt to characterise the deformability properties of the Collie sandstone.

(b) Test results indicated that the effective stress theory as given by the original Tarzaghi Equation (1.1) is applicable to the Collie sandstone.

(c) The influence of stress ratio and confining pressure on deformability modulus has been examined and a mathematical function for the non-linear stress-dependent characteristics of the Collie Sandstone is proposed (Equation 1.13):

$$E = \frac{\sigma_c\,\bar{x}}{K^2}\,\frac{1000}{a} \tag{1.13}$$

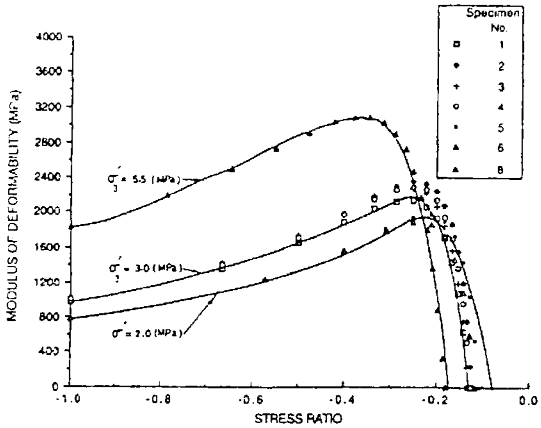

FIGURE 7 - Variation of modulus of deformability with stress ratio, $k = \sigma_3'/\sigma_1'$.

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