Rock strength reduction and its potential environmental consequences as a result of groundwater rebound

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ABSTRACT

In abandoned mines, subsidence and geological faults may be reactivated because of the rock strength reduction resulting from groundwater recovery, leading to surface environmental hazards and damage. This paper presents the laboratory results carried out on two types of UK coal measure rocks with multistage triaxial compressive tests under various degrees of saturation, aiming at quantifying the effects of water on rock properties, with particular emphasis on already disturbed or failed rocks as found post mining. The effect of the degree of saturation is examined and compared for intact rock and fractured or jointed rock respectively. It is observed that the jointed rock is more reactive to water than intact rock because of the high sensitivity of the residual friction angle to the presence of water. Additionally, the presence of soluble minerals plays a significant role. Probable underlying mechanisms are explored.

INTRODUCTION

The systematic and rapid closure of coal mines throughout the UK has resulted in a gradual recovery of the water table once the mine pumps are switched off. This groundwater rebound has been documented for a number of UK coalfields as well as areas in Germany, France and Poland. It is anticipated that, as well as the environmental impacts in terms of surface water pollution as the rising groundwater intersects the ground surface, there is also a geomechanical response in the rock mass which may in turn influence the surface environment in terms of residual or renewed subsidence. Problems with the movement of old shafts, the resettlement of collapsed workings and fill materials may result in serious damage to surface properties.

Due to the porous nature of rock, moisture is potentially absorbed into the voids and cracks because of the surface free energy. As a result, a thin water film is formed on the crack surface, which in turn facilitates further crack propagation and deteriorates the material strength (West, 1994). Saturated rocks can as a result reach a failure state at a relatively low stress level compared to dry rock. With respect to rock joints or fractures, which are quite commonly found in subsided strata post mining, water exists not only in the pore voids of intact parts but also on the failure surfaces in the form of a thin water film. Hence, the mechanical and geometrical parameters of the failure surface such as Joint Roughness Coefficient (JRC), dilation angle and basic friction angle etc are inevitably influenced. Accordingly, not only the intact rock but also the disturbed rock mass is at the risk of influence by water. Water continues to be a major concern to both operating and abandoned mines both on the surface and underground (Swift & Reddish, 1998).

The detrimental effect of water has been observed on intact rock by many researchers (Ballivy, et al. 1976; Broch, 1979; Colback & Wiid 1965; Dube & Singh, 1972; Masuda, 2001; Parate, 1973; Van Eeckhout, 1976). Many relevant tests have been conducted on different types of rocks. Previous results showed that up to 90% of the unconfined compressive strength could be lost at approximately one third of the moisture content at saturation (Colback & Wiid, 1965; Hawkins & McConnel, 1992). In addition, Chenevert (1970) found that the triaxial compressive strength of rocks decreases after the adsorption of water and the yielding strength varies almost linearly with the water content, so do the other strength parameters such as stiffness, cohesive strength and internal friction angle etc. Most of these analyses, however, are qualitative descriptions based on laboratory observations rather than quantitative expressions. Meanwhile, although various hypotheses have been put forward such as fracture energy reduction, capillary tension decrease, frictional reduction, chemical and corrosive deterioration and effective stress decrease due to the pore pressure increase (Van Eeckhout, 1976) in attempts to interpret the water effects, none of them provides a reliable and general quantitative measure of the problem.

Additionally, little attention has been paid to the already disturbed rock under the consideration of water effects even though the disturbed rock is often the main composition of the overburden in the abandoned mine context. A great number of stress related rock joints and fractures will be generated in the overlying strata during and after mining. The shear strength of these rock joints and fractures will control the bearing capacity and deformation of the disturbed rock strata. In addition, under saturated conditions rock joints fail more easily than intact rock because the main resistance force of failed rocks is from the friction and cohesion of the already existing failure surface, which is more sensitive to the effects of water. Consequently, in order to assess the effect of water on rocks post mining, research needs to be focused on the rock joints or fractures.

In view of the above, a series of triaxial compressive tests were carried out on coarse grained sandstone and silty sandstone as intact and broken samples, under various saturation conditions. Corresponding results are presented and discussed. The variations of the strength property parameters due to the different degrees of saturation are analysed subsequently. A significant difference in the deteriorative effect of water is observed between the intact samples and the rock joint. Mechanisms of strength reduction are also proposed.

EXPERIMENTAL TECHNIQUE

SAMPLE DESCRIPTION

Since sandstone is one of the major types of underground water bearing strata, the rock samples employed in this study were coarse grained sandstone and silty sandstone, which were collected from an opencast coal mine in Derbyshire, England. The coarse grained sandstone consists of quartz and a little feldspar, whilst the silty sandstone contains more silt and clay. The constituent grains are angular to sub-granular, which are demonstrated in Figure 1 observed from SEM scanning. Groups of samples were drilled out in the same vertical orientation from the same rock block. This approach allowed a high level of replication to be obtained and similar mechanical and physical properties to be shared among the group of samples. Corresponding mean physical properties of these two types of rock are listed in Table 1.

Table 1. Physical properties of samples tested.

Sample type	Dry density (kg/m^3)	Saturated density (kg/m^3)	Porosity (%)
Coarse Grained Sandstone	2277±2.2%	2390±1.3%	12.95±8.7%
Silty Sandstone	2393±0.3%	2468±0.3%	7.40±9.3%

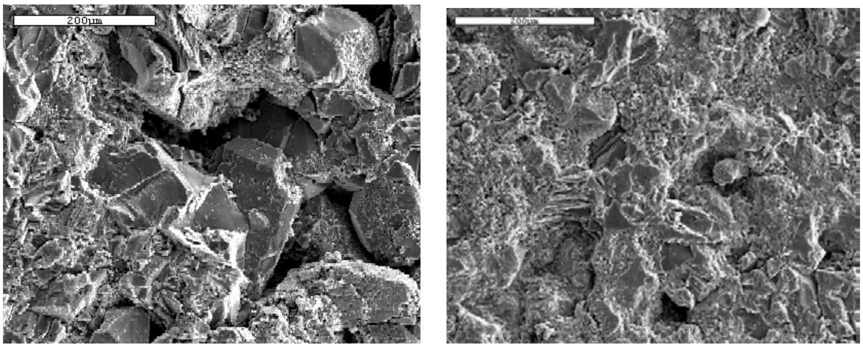

Figure 1: Topography of Coarse grained sandstone (a) and Silty sandstone (b) from SEM scanning

Test specimens were prepared according to the standards outlined in the ISRM suggested specifications. The specimen was a nominal cylinder, 38 mm in diameter and 76 mm in height.

Self-sticking cling film was used to protect samples against moisture loss after the samples were saturated. Additionally, samples were placed into airtight plastic bags and stored in a room with constant relative humidity and constant temperature before testing.

The rock joints or fractures were produced by crushing the intact samples through triaxial compression, as shown in Figure 2. Although the degree of saturation decreases in the process of loading, especially after dilatancy occurs (Masuda, 2001), the change is not big enough to result in an evident influence on the mechanical properties. Therefore, this effect can be neglected. The rock joint or fracture can be assumed to have the same moisture content as the intact samples, from which they were produced.

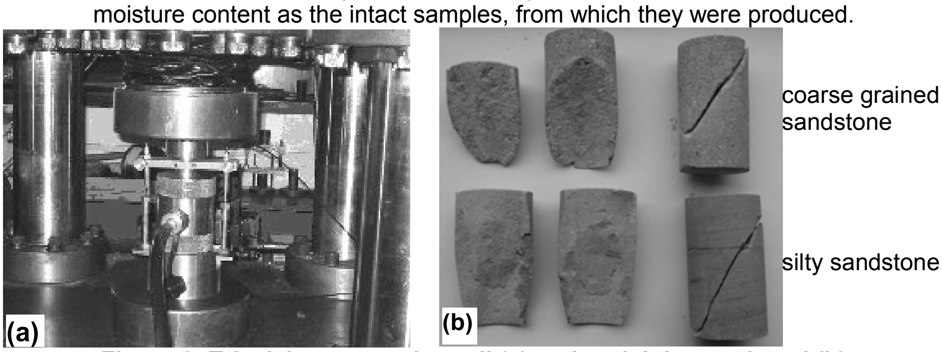

Figure 2: Triaxial compressive cell (a) and rock joint produced (b)

In order to obtain a controlled degree of saturation, a periodical weighting process was employed when samples were being conditioned. Specimens were dried first in a ventilated oven at 105±3°C until a constant mass was reached and then allowed to cool in an airtight desiccator for 30-60 minutes. Constant mass was assumed when successive mass determinations at intervals of 4 hours yielded values differing by less than 0.1%.

After the dimensions and dry masses of the specimens were measured they were saturated in a vacuum by immersing into water at a constant vacuum of 1000Pa. Periodical stirring was engaged to release bubbles trapped in pores or voids. Additionally, the samples were weighed every 4 hours to monitor the saturation process. When constant mass was reached the specimens were considered fully saturated. Care needed to be taken to prevent any loss of loose particles while the specimens were surface dried by means of a moist cloth before being weighed.

TESTING PROCEDURE

After sample preparation, multistage triaxial compressive testing was carried out with a constant loading rate, 0.005mm/sec. The intact sample was compressed uniaxially first to a state just before its peak strength. Before the sample actually fails, confinement is increased with the starting confining pressure of 0, then 10.5 MPa and then 21 MPa respectively. At the last stage of the confinement process the sample is compressed till fully broken. The subsequent unloading process is conducted in displacement control by reducing the confining pressure in a 3 MPa interval, axial load is also decreased accordingly in each step to avoid a sudden collapse of the sample. The testing method is illustrated in Figure 3 by plotting a full loading-unloading curve against axial displacement. The numbers appearing on Figure 3 are the confining pressures at different test stages. In this study, failed rock is sheared at the unloading phase. We define the peak values and post failure stable values as peak shear strengths and residual shear strengths. Corresponding friction angles are peak friction angle and residual friction angle respectively.

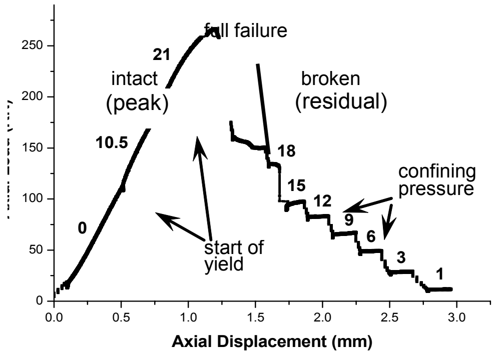

Figure 3: The multistage triaxial compressive testing method

EXPERIMENTAL RESULTS AND ANALYSES

EFFECT OF WATER ON THE SHEAR STRENGTH

Herein, the recorded principal stresses are transformed into corresponding shear stresses and normal stresses on the potential or already existing failure surface based on the assumption that further sliding of the failed rock is quasi static. Equations for the stress transformation can be expressed as:

$$\tau = \frac{1}{2}(\sigma_1 - \sigma_3) sin 2\beta$$
$$\sigma_n = \frac{1}{2}(\sigma_1 + \sigma_3) + \frac{1}{2}(\sigma_1 - \sigma_3) cos 2\beta \quad (1)$$

where τ and σ_n are the shear strength and normal stress on the failure plane, σ_1 and σ_3 are the axial stress and corresponding confinement respectively and β is the failure angle which can be determined by:

$$tan 2\beta = -\frac{1}{tan \phi} \quad (2)$$

where ϕ is the internal friction angle that can be measured from inclination of the strength envelopes of intact samples.

Relevant results regarding the strength reduction due to water saturation are presented in Figure 4 (a) and Figure 4 (b) for the coarse grained sandstone and silty sandstone respectively. It is observed that the strength envelopes of saturated samples are underneath the dry samples for both intact and failed conditions, which means the shear strength decreases when the rock mass is saturated. Additionally, the intact strength envelopes keep monotonously increasing with the confinement no matter if the samples are dry or saturated, i.e., the strengths of intact rock can be represented by straight lines with the assumption of the linear Mohr-Coulomb yielding criterion. However, with respect to the failed rock the strength envelopes are a series of curves. Additionally, the strength envelopes of intact samples have nonzero intercepts on the vertical axis, whilst those curves for failed rock always start from the origin, which indicates that the rock sample loses its cohesive strength completely after being broken at the end of the loading process, The intact rocks however still hold some cohesive strength even though it tends to be reduced by water.

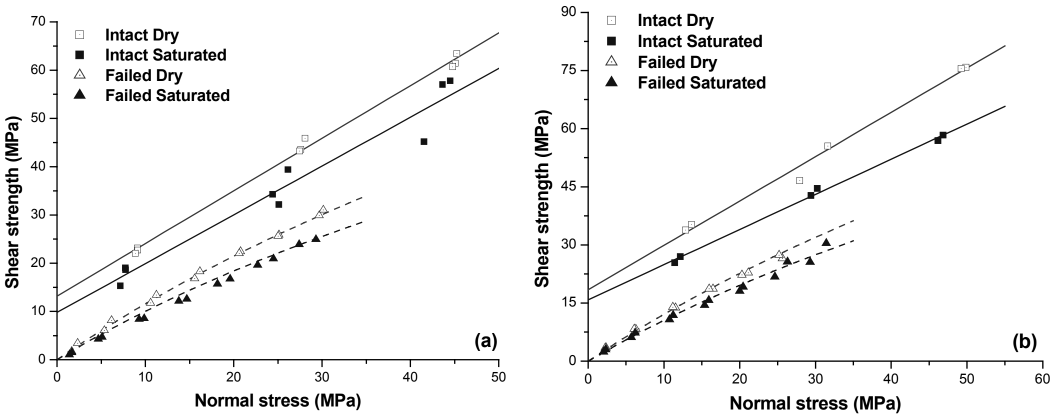

Figure 4: Strength envelopes of intact and failed (a) Coarse grained sandstone (b) Silty sandstone when dried and saturated

After saturation the cohesive strength reduces from 14.1 to 10.3 MPa and from 18.7 to 15.9 MPa, for the coarse grain sandstone and silty sandstone respectively. Corresponding reduction rates are 27% and 16%. As for the failed rock, the initial cohesive strength drops down to zero no matter whether dry or wet.

Another characteristic in Figure 4 (a) is that, for coarse grained sandstone the two envelopes of intact samples are nearly parallel with each other, whilst, in Figure 4 (b) the counterparts of silty sandstone are different in their inclinations after saturation. This indicates that the internal friction angle of the intact coarse grained sandstone sample is not affected by water but the friction angle of the silty sandstone has been reduced by the water's presence. Once the intact silty sandstone sample is saturated the internal friction angle decreases 12.4°, from 48.6° to 36.3°. For the coarse grained sandstone, no reduction happens on this parameter.

DETRIMENTAL EFFECT ON THE FRICTION ANGLE OF ROCK JOINT

As has been illustrated in Figure 4, the internal friction angles are almost unaffected by the confining pressure for dry samples and saturated samples as long as the samples are kept intact. The angles remain constant with confinement even though there is some reduction for silty sandstone after saturation.

When the samples are failed the residual friction angle is variable as well, reducing with both water saturation and confining pressure, as shown in Figure 5. The testing values reduce from 57.6° to 42.7° and from 57.8° to 52.4° for coarse grained sandstone and silty sandstone respectively when samples change from completely dry state to fully saturated at zero normal stress. The corresponding reduction rates are 25.8% and 9.2%. More details are listed in Table 2.

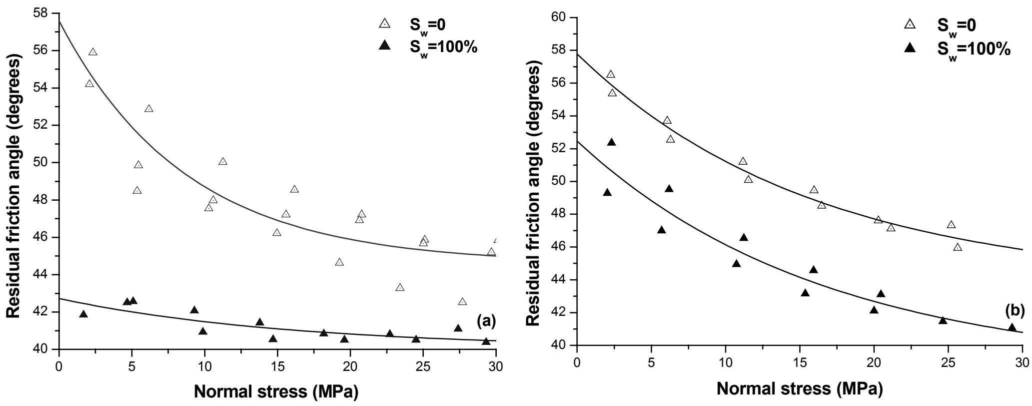

Figure 5: Developing tendency of the residual friction angle due to water, (a) Coarse grained sandstone (b) Silty sandstone

	Coarse Grained Sandstone			Silty Sandstone		
	Dry	Saturated	RS	Dry	Saturated	RS
0MPa	57.6°	42.7°	25.8%	57.8°	52.4°	9.2%
30MPa	45.0°	40.5°	10.1%	45.8°	40.8°	11.0%
RN	21.9%	5.3%		20.6%	22.2%	

Table 2. Testing values of residual friction angle and corresponding reduction rate due to saturation (RS) and normal stress (RN).

Note: RS: Reduction due to Saturation at each normal stress level
RN: Reduction due to Normal stress rising

This observation can be explained as follows. According to Patton's rock joint model, the apparent shear stress before the failure of surface asperities can be simplified and expressed as Equation 3 by assuming the shape of asperities is identical.

$$\tau = \sigma_n \tan(\phi_r + i) \qquad (3)$$

where τ is the shear strength, σ_n is the normal stress, ϕ_r and i are the residual friction angle and the dilation angle of the rock joint respectively. Before the surface asperities failure the shear strength of the rock joint is dominated by the sum of ϕ_r and i. Consequently, the water deteriorates the rock joint strength mainly by reducing the residual friction angle and the dilation angle. However, the rock joint is produced with low initial dilation angle because of high confinement in this study. Therefore, the residual friction angle can be regarded as a dominate factor in the strength development.

When the confining pressure reaches a certain level, the surface asperities will undergo large deformation and eventually fail. Under this state the reduced surface asperities are not easily interlocked. Accordingly, the failure surface tends to be smoother, resulting in a smaller friction angle. With regard to the saturated sample, the corresponding deformation of the surface asperities is larger than that in the dry sample at the same normal stress levels. Consequently, the relevant apparent friction angles are smaller, as demonstrated in Figure 6.

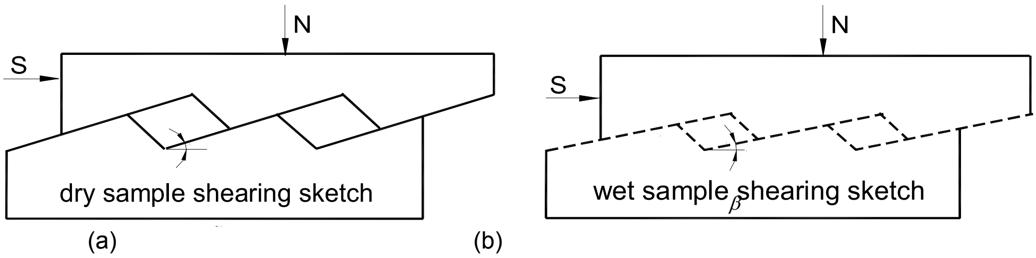

Figure 6. Schematic graph showing the deformation of surface asperities and different sliding surface because of saturation, $\alpha > \beta$

INFLUENCES ON THE INSTANTANEOUS COHESION OF ROCK JOINT

Not only the residual friction angle but also the instantaneous cohesion of rock joint could be influenced by water. The definition of the instantaneous cohesion of a rock joint is given in Figure 7. After the shear strength envelopes of rock joints for dry and saturated samples have been obtained, a series of instantaneous cohesions is calculated out for various normal stress levels with the method illustrated in Figure 7. As is shown in Figure 8, for the coarse grained sandstone and silty sandstone, the value of the instantaneous cohesion for both dry and saturated samples is increasing with the normal stress, essentially with the confining pressure. The instantaneous cohesion of the dry coarse grained sandstone is higher than the value of saturated, but for the silty sandstone it is contrary. This might be attributed to the chemical weakening of the silt cementations contained in the silty sandstone once saturated (Morrow et al, 2000).

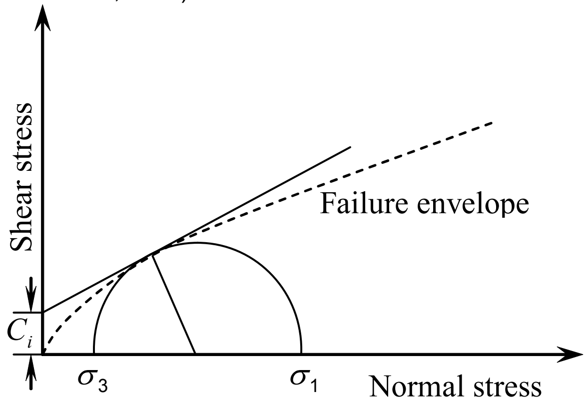

Figure 7: Schematic graph showing the definition of the instantaneous cohesion of rock joint (C_i) in terms of axial stress σ_1 and confining pressure σ_3

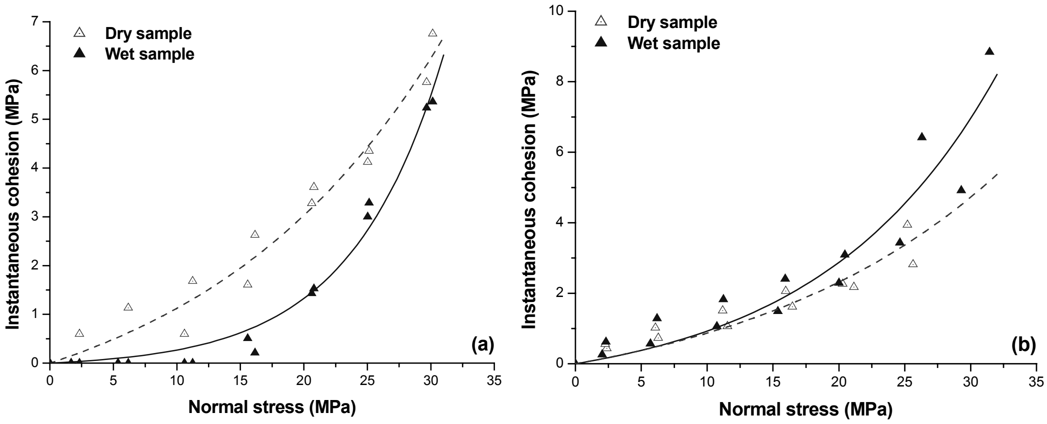

Figure 8: Instantaneous cohesion of the rock joint against the normal stress for (a) Coarse grained sandstone, (b) Silty sandstone

CONCLUSIONS

The experimental results and analyses presented in this paper show that the significance of the detrimental effect of water rests on the constitution of the rock. The more water sensitive constituents such as clay and silt it contains, the more deterioration effect the water has.

For the intact rock mainly consisting of quartz and feldspar the water effect is reflected by the reduction of cohesive strength only. The basic friction angle is almost unaffected. If rocks contain certain amounts of clay or silt, not only the cohesive strength but also the basic friction angle will be altered by the saturation.

Once the sample fails, the cohesive strength is lost completely. The presence of water does not play an adhesive role significantly on the two failure surfaces. The shear strength reduction of rock joints or failed rocks is mainly from the water's serious deterioration of the residual friction angle. Residual shear strength dominates the stability of the entire disturbed or jointed rock mass.

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