Research Article

Influence of Unloading Conditions of Confining Pressure on the Compressive Strength and Permeability of Deep Mudstone

Tianyu Xin ¹, Yashengnan Sun ², Junguang Wang ¹, and Weiji Sun¹

¹Institute of Mechanics & Engineering, Liaoning Technical University, Fuxin 123000, China

²College of Safety Science and Engineering, Liaoning Technical University, Fuxin 123000, China

Correspondence should be addressed to Junguang Wang; 307446930@qq.com

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To investigate the compressive strength and permeability of deep mudstone under stress disturbance, a triaxial rheometer is used to conduct seepage experiments on mudstone specimens with different buried depths under triaxial compression and unloading conditions. The experimental results show that the compressive strength of mudstone specimen with a depth of 1000 m is much lower than that of specimen with a depth of 200 m, and the compressive strength of mudstone increases with the increase in confining pressure. Under constant axial pressure and unloading of the confining pressure, the mudstone with a depth of 200 m exhibits brittle failure, and the strain fluctuates in a pointwise manner with the increase in axial stress. In this case, the mudstone with a depth of 1000 m exhibits a transition from brittle failure to ductile failure, and the strain fluctuates linearly with the axial stress. Further, when the volumetric strain change reaches 0.01, it shows an oblique “Z” fluctuation. During the initial stage of unloading of confining pressure, the permeabilities of both the mudstone specimens (with depths of 200 and 1000 m) decrease gradually. As the confining pressure is unloaded, the permeability of mudstone with a depth of 1000 m increases. Until the specimen is completely destroyed, the permeability of mudstone increases rapidly. Overall, this study can serve as a useful reference for analyzing the engineering disasters associated with deep rock mass, tunnel ventilation, and gas storage.

1. Introduction

During the process of underground mining, roadway excavation and heading face cause the surrounding rock to produce stress disturbance and break the equilibrium state of coal and rock, which is usually manifested as axial loading and confining pressure unloading. This phenomenon is more obvious during the deep mining of coal and rock.

Traditionally, the mechanical properties of coal and rock are primarily investigated using experimental triaxial compression test [1–3]. However, the damage characteristics of rock masses after direct loading and after loading-unloading cycle are quite different. Several researchers have intensively studied the unloading characteristics of rock. Qiu et al. [4] used a new loading path and operation method to conduct confining pressure unloading tests under different unloading rates for clearly understanding the effect of unloading rate on the mechanical properties of rock. It was

found that the unloading rate of the confining pressure had a significant impact on the deformation of marble, and the initial confining pressure level mainly controlled the corresponding influence law. Liang et al. [5] continuously loaded argillaceous limestone specimens to the residual stage under different initial confining pressures in the conventional triaxial compression test. The influence of confining pressure and loading-unloading cycles on the mechanical properties of residual rocks was examined through the stress and strain characteristics and acoustic emission monitoring. In addition, the yielding prediction information about the rock in the loading stage of confining pressure was analyzed. Huang et al. [6] used unloading tests to examine the deformation and fracture characteristics of rocks under unloading. Chen et al. [7] investigated the influence of unloading process on the permeability and damage deformation of coal by X-ray computed tomography (CT). Kang et al. [8] performed X-ray diffraction (XRD) measurements,

longitudinal wave velocity measurements, uniaxial compression tests, and dynamic impact tests on granite from Daliuhang Gold Mine, Yantai, Shandong, China, at various burial depths to reveal the static and dynamic mechanical properties of the rock specimens. Based on the energy theory, Zhao et al. [9] examined the strain energy conversion process of rocks with initial confining pressures of 10, 20, and 30 MPa under different unloading paths.

The differences in mechanical properties can lead to variation in permeability characteristics. To this end, the permeability characteristics of coal rock under unloading conditions have been widely investigated in the recent years. Yin et al. [10], Zhang et al. [11], and Xue et al. [12] studied the impact of unloading rate of confining pressure on the damage and seepage characteristics of gas-bearing coals and suggested that the increase in unloading rate causes a decrease in the peak strength and an increase in the permeability. Ren et al. [13] studied the dynamic behavior and permeability characteristics of coal rocks under strong disturbance and indicated that the permeability of sample with largest disturbance stress before failure was generally higher than those of the other samples. Besides, several other researchers (Pan and Connell [14], Wang and Du [15], Xie et al. [16], and Zhang et al. [17]) systematically explored the mechanical or permeability properties of coals under confining pressure unloading through triaxial compression tests.

Although the existing studies provided useful insights into the mechanical and permeability characteristics of coal and rock under unloading and simulated the stress environment of deep coal and rock, the difference between deep and shallow rocks has been mostly ignored [18, 19]. In this study, based on the difference between deep and shallow rocks, the compressive strength and permeability characteristics of deep mudstone under confining pressure unloading are investigated through comparative experiments. The results can serve as an effective guide for the analysis of engineering disasters associated with deep rock mass, roadway ventilation, and gas storage.

2. Experimental Device and Scheme

2.1. Experimental Device. The experimental device is a triaxial rheometer developed by Wuhan University of Science and Technology, China, as shown in Figure 1. The device includes a numerically controlled pressure system, osmotic system, triaxial pressure chamber, data acquisition system, and auxiliary system. It can be used to realize multifield coupling of stress field, seepage field, gas pressure field, and so forth. The maximum confining pressure, maximum axial load, and accuracy of the device are 50 MPa, 650 kN, and ± 0.2 MPa, respectively. A high-precision axial linear variable differential transformer (LVDT) and special circumferential strain sensor are used to measure the axial strain and circumferential strain simultaneously.

2.2. Experimental Method. The mudstone used in the test was taken from the 12th mining area of Pingmei coal field. Due to the special geological structure of Pingmei mining

area, the same geological layer shows different buried depth in different working faces. Two mudstone specimens with different buried depths of 200m and 1000m were taken from working faces 17120 and 31030 of the 12th mining area of Pingmei coal field, respectively. The specific sampling locations are shown in Figure 2. According to the requirements of the national standard, the diameter (ϕ) and height (h) of a standard cylindrical specimen are 50 mm and 100 mm, respectively. The cut specimen is shown in Figure 3(a).

- (1) Triaxial compression test: Three values of confining pressure are considered: 0, 15, and 30 MPa. First, $\sigma_1 = \sigma_2 = \sigma_3$ are loaded to a hydrostatic pressure of 30 MPa at a rate of 0.01 mm/s. Keeping $\sigma_2 = \sigma_3$ constant at 30 MPa, loading of σ_1 is continued at a rate of 0.02 mm/s until the specimen is destroyed.
- (2) Seepage with confining pressure unloading: First, $\sigma_1 = \sigma_2 = \sigma_3$ are loaded to hydrostatic pressure of 30 MPa at a loading rate of 0.01 mm/s. $\sigma_2 = \sigma_3$ are kept constant at 30 MPa, and σ_1 is further loaded to 80 MPa at a rate of 0.02 mm/s. Keeping σ_1 unchanged, $\sigma_2 = \sigma_3$ are unloaded at a rate of 0.02 mm/s, 3 MPa per discharge. The variation in permeability of each mudstone specimen at each unloading (~ 30 min) is measured until the specimen is completely destroyed. The specimen is replaced, and all the steps are repeated until the end of the test. The stress path during the confining pressure unloading test is shown in Figure 3(b).

2.3. Principle and Method of Gas Permeability Test. In the quasistatic method, the inlet end of the pressure chamber is used to control the gas pressure on the lower surface of the specimen. This method is used for examining the gas permeability. The outlet end of the pressure chamber (the upper surface of the specimen) is connected to the atmosphere [20]. After the pressure difference between the upper and lower end surfaces allows steady-state seepage through the specimen, the change in gas pressure at the inlet end is recorded as a function of time to determine the gas permeability of the specimen. A schematic of test is shown in Figure 4.

The general form of the partial differential equation for gas percolation is

$$\nabla \cdot \left(\delta \frac{K}{\mu} \frac{p}{Z} \nabla p \right) = \frac{\partial (\phi p / Z)}{\partial t}, \quad (1)$$

where δ is the inertial-turbulence correction coefficient; K is the gas permeability (m^2); μ is the viscosity of the gas (Pa·s); p is the gas pressure (MPa); ϕ is the porosity; Z is the deviation factor, the modifier of the ideal gas equation of state; T is the seepage time (s).

In the quasistatic method, it is assumed that the gas seepage satisfies the following conditions: (1) Darcy's law is established, $\delta = 1$; (2) the seepage gas is an ideal gas, $Z = 1$.

Therefore, the one-dimensional gas percolation equation is simplified as follows:

FIGURE 1: Schematic and actual image of triaxial rheometer.

FIGURE 2: Sampling locations.

$$\frac{K}{2\mu} \frac{\partial^2 P^2}{\partial x^2} = \phi \frac{\partial P}{\partial t}. \quad (2)$$

During the test, the following initial condition is satisfied: when $t=0$, $P|_{x=0} = P_1$. Further, the boundary condition is that when $t \geq 0$, $P|_{x=L} = P_0$. Since the permeability of the

rock $< 10\text{--}17 \text{ m}^2$ [21], it can be assumed that $\phi \approx 0$. Thus, equation (2) can be solved, and the gas pressure in the pressure chamber can be obtained as follows:

$$P(x) = \sqrt{P_1^2 \left(1 - \frac{x}{L}\right) + P_0^2 \left(\frac{x}{L}\right)} \quad (0 \leq x \leq L), \quad (3)$$

FIGURE 3: Mudstone specimens and stress path during the confining pressure unloading test.

FIGURE 4: Principle of quasistatic method and the corresponding experimental system. (a) Principle of quasistatic method; (b) apparatus for measuring the permeability by transient method.

where P_1 is the inlet pressure of the pressure chamber (MPa); P_0 is atmospheric pressure (0.1 MPa); x is the distance between the cross section at any point inside the test piece and the test-piece cross section at the inlet (m); L is the length of the test piece (m).

The gas seepage in the test satisfies Darcy's law, so

$$Q = -\frac{kA}{\mu} \frac{\partial P}{\partial x}. \quad (4)$$

Substituting equation (3) into (4) and setting $x = L$, the expression of the gas flow rate at the outlet end of the pressure chamber can be obtained as

$$Q_1 = \frac{kA(P_0^2 - P_1^2)}{2\mu LP_1}. \quad (5)$$

It is assumed that, at $t = 0$ and $t = \Delta t$, the pressure at the intake end is P_1 and $P_1 - \Delta P_1$, respectively. Therefore, average pressure $P_{\text{moy}} = P_1 - \Delta P_1/2$, where ΔP_1 is the pressure change at the intake end.

Within Δt , the gas quality in the pressure chamber changes as $\Delta m = \nu \times \Delta \rho_1$. According to assumption (2), from the ideal gas state equation $p/\rho = c$, we get

$$\Delta m = \nu \times \frac{\rho_1 \times \Delta P_1}{P_1}, \quad (6)$$

$$\Delta m = \rho_{\text{moy}} \times Q_{\text{moy}} \times \Delta t, \quad (7)$$

$$Q_{\text{moy}} = \frac{\nu \Delta P_1}{P_{\text{moy}} \Delta t}. \quad (8)$$

According to equations (5) and (8), the gas permeability of rock sample can be expressed as follows:

$$k = \frac{2\mu L \Delta P_1}{S(P_{\text{moy}}^2 - P_0^2)\Delta t}, \quad (9)$$

where Q is the gas flow rate (m^3/s); S is the cross-sectional area of the rock sample (m^2); v is the volume of rock sample; ρ_{moy} and Q_{moy} are the indoor average gas density (kg/m^3) and average gas flow rate at the chamber inlet end (m^3/s) in the interval Δt , respectively.

3. Results and Discussion

3.1. Effect of Confining Pressure on the Stress-Strain Curve and Compressive Strength of Deep Mudstone. Through triaxial compression test, compressive strength of mudstone specimens with different buried depths is obtained, as shown in Table 1, and the stress-strain curves of some mudstone specimens are shown in Figure 5. It is clear that the compressive strength of mudstone with a depth of 1000 m is much smaller than that of mudstone with a depth of 200 m. This is because the mudstone with buried depth of 1000 m remains in a high stress state for a long time. During mining, the stress on the mudstone is released in a short time, which causes the internal structure to exhibit plastic deformation, and the compressive strength is reduced. The mudstone buried at a depth of 200 m contains a small amount of white SiO_2 mineral particles, which enhances the compressive strength [22]. The composition of mudstone is quantitatively analyzed based on fluorescence spectroscopy, and the results are shown in Table 2. Compared to the conventional hard rock [19], the specimen also experiences sudden release of ground stress, leading to the initial damage of internal structure, but it shows completely opposite results. It is considered that the initial damage of mudstone will not have a decisive influence on its strength. Here, the mudstone with a burial depth of 200 m has a higher SiO_2 content, which mainly results in its higher strength.

Figure 6 shows the relationship between peak strength of mudstone and confining pressure. The peak strength of the two types of buried mudstone increases linearly with the increase in the confining pressure. It can be seen from the slope of the fitting curve in Table 3 that the variation rates of the peak strength of the two buried mudstone specimens are slightly different. When the confining pressure rises from 0 MPa to 30 MPa, the average peak strength of mudstone buried at a depth of 200 m increases by 78.95 MPa (from 70.21 MPa to 149.16 MPa), while that of mudstone buried at a depth of 1000 m increases by 73.7 MPa (from 42.29 MPa to 115.99 MPa). It is clear that the variation rate of peak strength for mudstone buried at a depth of 200 m is higher. On the other hand, from the perspective of strain (left side of the axis in Figure 5), when the confining pressure increases, the initial strain of mudstone with a buried depth of 200 m is smaller than that of mudstone with a buried depth of 1000 m. This implies that, compared to the mudstone specimen with buried depth of 1000 m, the effect of

confining press on the deformation is stronger for the mudstone specimen with buried depth of 200 m.

3.2. Influence of Confining Pressure Unloading on the Volume Strain of Deep Mudstone. Due to the anisotropy between mudstone specimens, it is crucial to select the maximum value of axial pressure during the confining pressure unloading test. After several experiments, two representative deep mudstone specimens were selected for the analysis.

The relationship between strain of mudstone specimen and axial stress (confining pressure) is shown in Figure 7 (Figure 8). The three important stages of the test process can be clearly identified from the diagram, namely, axial compression loading stage, confining pressure unloading stage, and “fracture” stage. The variation trend of strain in the axial compression loading stage is basically the same for the two specimens; the loading strain gradually increases with the increase in axial pressure. Unlike triaxial compression tests, the two deep buried mudstones exhibit significantly different response in the confining pressure unloading stage. The volume strain of mudstone with buried depth of 200 m is negligibly changed during this stage, and the axial stress remains basically constant at its maximum value and fluctuates in a pointwise manner until the confining pressure is unloaded to 6 MPa, which indicates fracture of the specimen, where the volume strain drops rapidly, and the mudstone exhibits extremely strong brittle characteristics. The volume strain ($\Delta \epsilon$) of the mudstone specimen with buried depth of 1000 m is close to 0.02 at this stage. During the initial stage, the axial stress linearly fluctuates with the increase in strain. When the value of $\Delta \epsilon$ approaches 0.01, axial stress fluctuates with strain in an oblique “Z” fashion. Further, it shows a gradual decline and is unable to return to the initial level. This indicates that the unloading of confining pressure causes circumferential expansion and axial compression. Therefore, the axial compression causes a small decrease of 63.12 MPa in stress when the confining pressure is unloaded to 12 MPa, which indicates destructive strength, where the volume strain drops rapidly. In the confining pressure unloading phase, the failure form of deep mudstone changes from brittle fracture of shallow mudstone to fracture growth controlled by lateral stress, that is, from shallow dynamic failure to deep quasistatic failure. The mechanical response is changed, and the fragility of the shallow part is transformed into potential ductile failure of the deep part.

3.3. Permeability Characteristics of Deep Mudstone. The permeability of mudstone is closely related to its porosity and structural characteristics. The better the pore connection, the higher the permeability. By comparison, the pore characteristics of the two deep buried mudstones are similar. Further, the permeabilities of the two mudstones are nearly similar before confining pressure unloading, but they become different during the unloading process. This indicates that the structural characteristics of the two deep buried mudstones vary differently during the unloading process.

The relationship between mudstone permeability and confining pressure is shown in Figure 9. In conventional

TABLE 1: Compressive strength of different specimens.

Specimen number	Buried depth (m)	Confining pressure (MPa)	Diameter (mm)	Compressive strength (MPa)
MS-2-0-1	200	0	49.97	70.85
MS-2-0-2	200	0	50.21	71.20
MS-2-0-3	200	0	48.97	68.58
MS-2-15-1	200	15	49.91	121.50
MS-2-15-2	200	15	51.24	123.52
MS-2-15-3	200	15	50.37	117.26
MS-2-30-1	200	30	50.01	148.00
MS-2-30-2	200	30	49.87	153.26
MS-2-30-3	200	30	49.93	146.21
MS-10-0-1	1000	0	49.87	42.96
MS-10-0-2	1000	0	50.33	38.56
MS-10-0-3	1000	0	51.02	45.36
MS-10-15-1	1000	15	48.98	81.23
MS-10-15-2	1000	15	49.51	86.58
MS-10-15-3	1000	15	50.97	78.26
MS-10-30-1	1000	30	49.49	107.21
MS-10-30-2	1000	30	50.87	119.54
MS-10-30-3	1000	30	50.49	121.23

FIGURE 5: Stress-strain curves of mudstone specimens.

TABLE 2: Quantitative analysis of composition of mudstone based on fluorescence spectroscopy.

Buried depth of mudstone	200 m	1000 m
Ignition loss	9.280	14.620
SiO ₂	62.480	53.360
Al ₂ O ₃	16.450	24.560
TFe	5.240	3.870
FeO	4.460	2.680
K ₂ O	1.458	0.580
Other	0.632	0.330

FIGURE 6: Peak strength versus confining pressure curve for the mudstone specimen.

TABLE 3: Relationship between peak strength and confining pressure.

Buried depth (m)	Fitting equation	Correlation coefficient
200	$y = 2.571x + 74.875$	0.98406
1000	$y = 2.529x + 43.682$	0.99708

FIGURE 7: Axial stress versus strain curve for the mudstone specimens. (a) Depth = 200 m. (b) Depth = 1000 m.

triaxial pressure test of coal-rock seepage, the permeability of coal rock increases with the decrease in the confining pressure (within failure strength regime), but, during the early stage of confining pressure unloading in the seepage test, the permeability of both deep mudstone specimens decreases with the increase in confining pressure. With

further unloading of confining pressure, unlike the mudstone with buried depth of 200 m, the permeability of mudstone with buried depth of 1000 m gradually increases.

The permeability of mudstone is strongly correlated with its volume strain. According to Figure 10, after the axial compression stage, the specimen is subjected to different

FIGURE 8: Confining pressure versus strain curve for the mudstone specimens. (a) Depth = 200 m. (b) Depth = 1000 m.

FIGURE 9: Mudstone permeability versus confining pressure. (a) Depth = 200 m. (b) Depth = 1000 m.

volume strains. During the initial stage of unloading, the deformation of two deep buried mudstones is close to the volume strain. Overall, the mudstone specimen begins to expand, and this process represents axial compression and circumferential expansion. According to the observed tri-axial failure of mudstone, it is clear that the two sedimentary structures of deep buried mudstone resemble horizontal bedding. Before reaching peak intensity, with the unloading of the confining pressure, lower lateral bound of mudstone specimens is achieved. Because the axial stress remains basically constant, longitudinal compaction occurs, and the joint opening decreases gradually. The gas seepage is effectively related with the continuous blockage of channels. The fracture connection rate is reduced, and the gas

permeability is also reduced. Due to the high compressive strength and low deformation of mudstone with buried depth of 200 m, after unloading to 6 MPa, direct brittle failure is observed, and the mudstone transitions from expansion stage to instability failure stage. Further, the variation trend of permeability changes from decreasing to rapidly increasing. The mudstone with a depth of 1000 m shows different response. Figures 8 and 9 show that, after reaching the peak failure strength without rapid destruction, the mudstone exhibits deformation resistance. Consequently, the ductile failure stage of mudstone with buried depth of 1000 m occurs after the volume strain recovers to 0. The deformation of mudstone continuously increases in the ductile failure stage. Further, the internal structure changes

FIGURE 10: Volume strain versus permeability and pressure of mudstone. (a) Depth = 200 m. (b) Depth = 1000 m.

dramatically. The holes and cracks gradually collide, and the permeability increases. When the confining pressure is unloaded from 21 MPa to 12 MPa, the permeability changes by $4 \times 10^{-19} \text{ m}^2$, which is five times higher than the decrease in permeability during unloading from 30 MPa to 21 MPa. As the confining pressure continues to unload, mudstone with buried depth of 1000 m becomes completely unstable, and the permeability increases rapidly.

4. Conclusion

To investigate the compressive strength and permeability characteristics of deep mudstone, conventional triaxial compression test and seepage test under confining pressure unloading were conducted on two kinds of deep buried mudstone from the 12th mining area of Pingmei, China. The main results of the study are summarized as follows:

- (1) Unlike conventional hard rock, the triaxial compressive strength of mudstone with buried depth of 1000 m was much smaller than that of mudstone with buried depth of 200 m. The peak strength of both specimens increased with the increase in confining pressure. For the mudstone specimen buried at a depth of 1000 m, the effect of confining pressure on the peak strength was weak, but the effect of confining pressure on the deformation was strong.
- (2) The confining pressure unloading test clearly discerned the axial pressure loading stage, confining pressure unloading stage, and fracture stage. The strain of mudstone buried at a depth of 200 m exhibited pointwise fluctuation with the increase in axial stress during the unloading of confining pressure. This mudstone exhibited fracture failure when the confining pressure was unloaded to 6 MPa, where it showed extremely strong brittle characteristics. The strain of mudstone buried at a depth of 1000 m fluctuated

linearly with the increase in axial stress during confining pressure unloading. After the volume strain ($\Delta\epsilon$) reached 0.01, it showed an oblique “Z” fluctuation with the increase in axial stress. The mudstone exhibited fracture failure when the confining pressure was unloaded to 15 MPa, and the failure characteristics changed to brittle failure. The strain of mudstone buried at a depth of 1000 m decreased with the increase in confining pressure. The total strain at this stage was 74.44% of the volume strain variation in each stage of confining pressure unloading. Further, the confining pressure showed a power function relationship with the unloading pressure difference.

- (3) The permeability of mudstone was found to be closely related to the volume strain. During the initial stage of confining pressure unloading, the volume strain of the two deep buried mudstones was close to 0, and the permeability decreased gradually. With further unloading of confining pressure, the volume strain of mudstone increased gradually, and the permeability of the mudstone with buried depth of 1000 m was enhanced.

Data Availability

The data that support the findings of this study are available from the corresponding author upon reasonable request.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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