Research Article

Experimental Study on the Mechanical Properties and Seepage Characteristics of Red Sandstone with a Single Persistent Joint under Triaxial Compression

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In this research, the conventional triaxial compression experiments for intact red sandstone specimens and the specimens with a single persistent joint at different inclination angles, i.e., 0°, 30°, 45°, and 90°, were conducted at first. Based on the results of the conventional tests, the effects of the confining pressure and the joint inclination angle on the mechanical properties including deformation behavior and strength parameters were summarized and analyzed, respectively. We find that the strength and deformation of jointed red sandstone are enlarged due to the increment of confining pressure, and the mechanical parameters of specimens show a U-shaped development with the rise of the joint angle. Besides, to investigate the effects of the pore pressure on seepage characteristics of rocks with joint angles at 0°, 45°, and 90°, a series of triaxial compression drainage tests on the jointed red sandstone were performed. The results show that the pore pressure has a weakening effect on the strength of jointed specimens, which can reduce the strengthening effect induced by confining pressure. Meanwhile, the tested specimens mostly present shear failure modes. As a result, the mechanical responses, seepage characteristics, and cracking modes in red sandstone containing a single persistent joint under triaxial compression are revealed.

1. Introduction

Rock mass, formed during a long hydrogeological history, is a kind of natural material with complicated internal microstructure, which generally consists of two components, namely, the intact rock and discontinuities. The discontinuities can be either persistent or nonpersistent and appear as various types including primary cracks, joints, weak surfaces, and faults. These discontinuities have a great effect on the determination of the mechanical behavior of rock materials [1–4], which can further lead to the failure along the natural joint surfaces in rock engineering and need to be well treated [5–7]. Meanwhile, seepage properties of jointed rocks have a significant influence on the stability of rock engineering. During the initiation and development of jointed rock mass, the seepage driven by surface water and groundwater is often encountered. For jointed rock mass, the existence of joints

provides channels for water seepage, accelerates the seepage process, causes changes in the internal microstructure of the rock mass, and macroscopically affects its physical properties, which may eventually lead to failure [8]. Consequently, the research on the seepage-mechanical coupling mechanism for jointed rocks has great theoretical and engineering significance.

Experimental studies on mechanical properties and failure modes of rock and rock-like specimens with nonpersistent joints and preexisting cracks have been widely performed. Bobet and Einstein [9] investigated the fracture coalescence of rock-like materials containing double preexisting fractures arranged in different geometries under uniaxial and biaxial loading. Wong and Einstein [10] reported the mechanical and cracking behaviors in modeled gypsum and marble specimens containing a single open crack under uniaxial compression. Lee and Jeon [11] studied the crack

initiation, propagation, and coalescence of preexisting open cracks or flaws in granite and gypsum specimens under uniaxial compression. The deformation and strength behavior of sandstone specimens containing double preexisting closed nonoverlapping flaws under triaxial compression were investigated by Huang et al. [12]. Yang and Huang [13] performed conventional triaxial compression experiments of granite specimens with one preexisting open fissure to study their strength, deformability, and failure characteristics. Furthermore, new techniques such as the posttest X-ray computed tomography scanning technique were used to reveal the fracture behaviors of the marble samples in [14].

Recently, theoretical, experimental, and numerical studies on the mechanical behavior of persistent jointed rocks have been reported. The existing empirical methods for estimating the uniaxial compression strength of persistent jointed rock masses are reviewed and evaluated by Zhang [15]. Serrano et al. [16] proposed a microscopic determination method for the shear strength of persistent rock discontinuities. Li et al. [17] carried out experimental studies to investigate the effect of the initial opening on both the normal and shear deformations of rock joints and then proposed a fractal constitutive model which represents the shear behavior of a large-scale rock with persistent joint [18]. Based on the experiments of jointed rocks, numerical studies were further carried out to analyze the failure behavior of a rock slope by Zhu et al. [19]. Li and Zhu [20] proposed a mesh-free numerical method to model the damage evolution and failure characteristics of fractured rocks. Besides, some pioneering research studies focusing on the seepage characteristics of jointed rocks have been conducted. The effects of shear deformation and normal stress on mechanical and seepage behavior of fractured rocks were experimentally investigated by Chen et al. [21]. Yin et al. [22] analyzed the influence of shear processes on nonlinear flow behavior based on their testing results of the 3-dimensional fractured rock. Wang et al. [23] and Gong et al. [24] carried out experimental investigations on mechanical properties and failure modes of jointed rocks under hydrochemical conditions. Shin and Santamarina [25] proposed an implicit joint-continuum model for the coupled hydromechanical analysis of persistent fractured rock, while Li et al. [26] investigated the permeability evolution in discrete fracture networks during shear.

However, the main contributions in most of the above studies are mainly limited to a single aspect of mechanical properties or seepage characteristics of rocks. Therefore, to comprehensively study the triaxial mechanical properties and seepage characteristics of jointed rocks, in the present paper, the conventional triaxial tests on red sandstone with a single persistent joint are firstly conducted. Then, a series of triaxial compression drainage tests on the jointed red sandstone are performed and the results are analyzed. Finally, we summarize the findings and close the paper with potential perspectives.

2. Experimental Methodology

2.1. Specimen Preparation. To investigate the mechanical properties and seepage characteristics of rock with a single

FIGURE 1: The geometry and types of red sandstone specimens: (a) the intact type of specimen with dimensioning, (b) the type of specimen with a single persistent joint (take $\alpha = 30^\circ$ as an example), and (c) the schematic of the persistent joint in the specimen under triaxial compression condition, where σ_1 denotes the axial stress.

persistent joint under triaxial compression, red sandstone from a certain engineering rock mass in China was chosen as the experimental material for this research. The tested red sandstone is fine-grained and macroscopically homogeneous, with an average bulk density of about 2450 kg/m^3 . A bridge-type infrared cutting machine was chosen to preliminarily cut the rock mass into small suitable-sized cuboids. Then, the cuboids were processed into cylindrical specimens with 50 mm in diameter and 100 mm in height, according to the method suggested by the International Society for Rock Mechanics (ISRM) [27].

In the present research, intact specimens and four types of specimens with various joint inclination angles α , i.e., 0° , 30° , 45° , and 90° , were prepared for studying the effects of the inclination angle on the mechanical properties and failure modes of the red sandstone. The geometry and types of specimens are briefly depicted in Figure 1. The production procedure of the jointed specimens is described as follows: (1) by controlling the direction of the specimens and using the infrared cutting machine, the cylindrical intact specimens were cut into two parts with the desired joint inclination angles; (2) then, cement was used for filling the interface; based on the results from [9], cement could be regarded as a weak material which only provides cementation without affecting rock properties; (3) after 48 hours, the red sandstone can be well bonded by cement; and (4) after the surface of the filling part is polished, the jointed specimens are finally manufactured. The experiments of specimens were carried out under three confining pressures σ_3 , i.e., 0, 10, and 20 MPa, in conventional triaxial tests, to investigate the effect of the confining pressure on the mechanical and cracking behavior. For the triaxial compression drainage tests, the pore pressure p was set as 4 MPa and 8 MPa. Detailed descriptions for red sandstone specimens in conventional triaxial tests with different joint inclination angles are listed in Table 1, while Table 2 shows the detailed geometry of red sandstone specimens in triaxial compression drainage tests.

TABLE 1: The detailed geometry of red sandstone specimens in conventional triaxial tests.

Inclination angles	Specimen label	D (mm)	H (mm)	σ_3 (MPa)
Intact	SY-I-0	50.03	100.18	0
	SY-I-10	50.18	99.92	10
	SY-I-20	49.79	99.88	20
0°	SY-0°-0	50.35	101.19	0
	SY-0°-10	50.38	101.29	10
	SY-0°-20	50.14	100.95	20
30°	SY-30°-0	50.21	100.86	0
	SY-30°-10	49.84	100.71	10
	SY-30°-20	50.02	100.76	20
45°	SY-45°-0	49.78	100.80	0
	SY-45°-10	50.36	100.85	10
	SY-45°-20	49.86	100.56	20
90°	SY-90°-0	50.25	100.05	0
	SY-90°-10	49.89	100.02	10
	SY-90°-20	49.86	100.05	20

Note: D : diameter; H : height.

TABLE 2: The detailed geometry of red sandstone specimens in triaxial compression drainage tests.

Inclination angles	Specimen label	D (mm)	H (mm)	σ_3 (MPa)	p (MPa)
0°	SD-0°-10-4	49.95	98.91	10	4
	SD-0°-10-8	50.02	97.91	10	8
	SD-0°-20-4	49.93	97.43	20	4
	SD-0°-20-8	49.97	98.20	20	8
45°	SD-45°-10-4	50.10	96.64	10	4
	SD-45°-10-8	50.05	99.15	10	8
	SD-45°-20-4	50.28	97.30	20	4
	SD-45°-20-8	50.03	97.86	20	8
90°	SD-90°-10-4	51.10	99.89	10	4
	SD-90°-10-8	50.83	100.02	10	8
	SD-90°-20-4	51.21	99.84	20	4
	SD-90°-20-8	51.11	100.21	20	8

2.2. Testing Apparatus and Procedure. All the triaxial compression experiments were performed on the rock automatic servo-controlled triaxial testing machine (see Figure 2). The equipment includes a triaxial pressure chamber, a loading system, a constant-stability pressure device, an ultrasonic system, a hydraulic pressure system with a pressure transfer device, a pneumatic system, and an automatic data collection system. The three high-pressure pumps in the loading system can control the steady loading and automatic compensation of deviatoric pressure, confining pressure, and water pressure. This equipment is able to perform tests including conventional uniaxial tests, triaxial tests, rheological tests, triaxial drainage tests, undrained tests, hydraulic and pneumatic penetration tests, and ultrasonic tests. The strain

FIGURE 2: Photographs of the rock automatic servo-controlled triaxial testing machine: (a) the overall view of the equipment and (b) the triaxial pressure chamber.

measurement system consists of linear variable differential transformer (LVDT) for measuring the axial strain and circumferential strain measurement ring for measuring the circumferential strain.

To reduce the influence of temperature on the test results, the temperature in the laboratory is strictly controlled at $20 \pm 0.5^\circ\text{C}$. For the conventional triaxial compression experiment, the detailed procedures are listed as follows: (1) first, the confining pressure was applied to the specimen at a constant loading rate of 0.1 MPa/s to ensure that the specimen was under the uniform hydrostatic stress condition; and (2) then, the deviatoric stress was imposed on the surface of the specimen at a constant displacement rate of 0.025 mm/min until the specimen failed.

For the triaxial compression drainage tests, the jointed specimens were saturated before the tests. After the same procedure with step (1) in the conventional test, when the confining pressure was stable, the seepage pressure should be loaded, and it needs to be less than the confining pressure. The rock sample should be saturated until water flows out of the outlet. After the pore pressure reached the predetermined value, the servo-control method was adopted to keep the pore pressure stable. Then, the deviatoric stress was applied by a constant displacement rate of 0.025 mm/min until the specimen failed. Meanwhile, during the process of all the tests, the automatic data collection system can record and digitize the test data in real time, which provides convenience for data analysis.

3. Conventional Triaxial Test Results and Analysis

3.1. Stress-Strain Curves. The axial deviatoric stress-strain curves of intact and jointed red sandstone under different confining pressures are shown in Figure 3. In accordance with Figure 3, for all specimens, a quasi-brittle mechanical behavior can be found and the overall phenomenon demonstrates that the slopes of the curves increase with the confining pressure σ_3 . Likewise, the values of σ_c increase when σ_3 gets higher. Notably, the trends of curves of deviatoric stress ($\sigma_1 - \sigma_3$) versus the strain have (I) the nonlinear stage, (II) the elastic stage, (III) the stable crack growth stage, and (IV) the yielding stage before reaching the peak deviatoric stress σ_c . Thereafter, the axial stress drops sharply after the peak stress with a tiny increase of axial strain. The curves

FIGURE 3: Axial deviatoric stress-strain curves of intact specimens and red sandstone containing a single persistent joint with various joint angles under different confining pressure: (a) intact specimens, (b) $\alpha = 0^\circ$, (c) $\alpha = 30^\circ$, (d) $\alpha = 45^\circ$, and (e) $\alpha = 90^\circ$.

may appear to be serration-type stress drops when near the point of peak strength (see Figure 3(d)), which results from the initiation and propagation of cracks inside the specimens. The location and numbers of the stress drops are closely related to the joint inclination angles. Moreover, the circumferential strains ϵ_3 have an infinitesimal value at the initial loading condition, which are far smaller than their corresponding axial strain ϵ_1 . More detailed analyses of deformation and strength characteristics are explained in the following sections.

3.2. Analysis of the Volumetric Deformation Characteristics. Under triaxial compression conditions, the total volumetric strain ϵ_v is calculated as

$$\epsilon_v = \epsilon_1 + 2\epsilon_3, \quad (1)$$

which can be considered as the composition of two portions: one is the volumetric strain induced by the elastic deformation, namely, the elastic volumetric strain ϵ_v^e , and the other is the portion attributed to the crack development, which is

called the crack-induced volumetric strain and denoted as ε_v^c . Based on Hooke's law, ε_v^c is given by

$$\varepsilon_v^c = \frac{(1 - 2\mu)(\sigma_1 + 2\sigma_3)}{E}, \quad (2)$$

where E is the elastic modulus and μ is Poisson's ratio of the specimen. Following the approximation proposed by Li et al. [28], ε_v^c can be defined as

$$\varepsilon_v^c = \varepsilon_v - \varepsilon_v^e. \quad (3)$$

Figure 4 presents the typical relationship between $(\sigma_1 - \sigma_3)$, ε_v , ε_v^c , and ε_1 of the jointed specimens under the confining pressure at 10 MPa. In Figure 4, four prepeak stages I-IV, i.e., stage I, the nonlinear stage owing to crack closure with a maximum at σ_{cc} ; stage II, the elastic stage with a maximum at the crack initiation σ_{ci} ; stage III, the stable crack growth stage between σ_{ci} and the crack damage threshold σ_{cd} ; and stage IV, the yielding stage with the initiation of macroscopic failure of specimens before the peak point, and a postpeak stage V were distinctively described. Accordingly, the development of prepeak ε_v also shows four stages, i.e., the volumetric compaction stage, the linear volumetric growth stage, the volumetric dilation stage, and the volume rapid increasing stage, corresponding to the compression process of jointed rock specimens. At the compaction stage, the primary transverse crack perpendicular to the axial direction is compacted due to the initial loading, while ε_3 basically has no growth, and hence, the trend of the volumetric compaction is consistent with the change of ε_1 . For the linear volumetric growth stage, ε_1 and ε_3 both grow linearly. The growth amplitude of ε_3 is less than that of ε_1 , which is also reflected as a linear growth stage of ε_v , correspondingly. When ε_3 begins to yield, the growth range of ε_3 increases obviously, while the growth range of ε_1 is relatively slow, and ε_v begins to present dilation behavior. When the specimen is close to failure, ε_3 increases sharply, while ε_1 grows less, and the growth of ε_v is consistent with the circumferential growth.

For the specimens with different joint inclination angles, some common characteristics from the curves of ε_v and ε_v^c versus ε_1 can be obtained from Figure 4. At stage II, ε_v^c increases slowly, while ε_v exhibits a linear growth when ε_1 rises. At stage III, cracks initiate and develop stably, and hence the values of ε_v begins to increase slightly and ε_v^c decreases linearly. At stage IV, both the curves of ε_v and ε_v^c versus ε_1 display a sharp reduction, which implies that the deformation state of specimens turns contraction into dilation and the material begins to perform macroscale failure. In addition, at stage II, it can be found that the slopes of the ε_v^c versus ε_1 curves for joint angles of 45° and 90° are larger than those of 0° and 30°, and the jointed specimens with the dip angles of 45° and 90° present more clearly quasi-elastic behavior, which indicates that the joint angle has a certain influence on ε_v^c .

3.3. Analysis of Strength Characteristics

3.3.1. The Peak Strength. As illustrated above, Figures 3 and 4 have shown the characteristics of the critical stress points; the

crack initiation stress σ_{ci} and the crack damage threshold σ_{cd} are clearly marked in the curves. Table 3 summarizes the values of σ_{cd} and σ_{ci} and their ratios to σ_c of intact specimens and the persistent jointed specimens with different joint angles. It is obvious that σ_c rises with the increment of σ_3 . A similar behavior can be found in σ_{cd} and σ_{ci} with σ_3 . The jointed specimens always have a smaller crack initiation strength and damage threshold than the intact specimens due to the degradation effects of the preexisting joint on the strength. Another significant phenomenon is that, when the joint angle is equal to 45°, the preexisting joint lies on the theoretical weakest slippage surface, which leads to the smallest σ_{cd} and σ_{ci} . In accordance with the experimental results, the values of σ_{cd}/σ_c of intact specimens are approximate to 1, whereas the ratios of jointed specimens range from 0.581 to 0.995. This might reveal that the intact specimens ruptured quickly after the crack damage threshold was reached. Besides, the ratios, σ_{ci}/σ_c , of intact specimens and jointed specimens lie between 0.428 and 0.835, which are independent of σ_3 and show no specific trends when σ_3 rises.

3.3.2. Analysis of Strength Parameters. This study adopted the linear Mohr-Coulomb (M-C) criterion to fit the peak strength and the crack damage strength with confining pressure. The linear M-C criterion can be given as follows:

$$\begin{cases} \sigma_1 = M\sigma_3 + N, \\ \varphi = \arcsin\left(\frac{M-1}{M+1}\right), \\ c = N \frac{1 - \sin \varphi}{2 \cos \varphi}, \end{cases} \quad (4)$$

where M and N are two material parameters. The parameters c and φ are the cohesion and the internal friction angle of rocks, respectively.

Figure 5 presents the fitting results of the peak strength and crack damage threshold of intact and jointed specimens versus confining pressure. Besides, Tables 4 and 5 list the fitting parameters corresponding to the peak strength and crack damage threshold, respectively. It is clear that the peak strength and crack damage threshold exerted a good linear behavior with the increase of confining pressure for the specimens.

For intact specimens, the cohesion c and the friction angle φ in terms of the peak strength are 15.83 MPa and 49.81°, respectively, which are approximately equal to c_d and φ_d in terms of the crack damage threshold, i.e., 15.43 MPa and 49.75°, respectively. However, for the jointed specimen, c ranges from 8.36 MPa to 13.89 MPa, whereas the value of c_d lies between 9.97 MPa and 11.97 MPa. In comparison, the values of c and c_d of jointed specimens are lower than those of intact specimens. Meanwhile, for the jointed specimen, φ is approximately 41.94–50.85°, while φ_d lies between 30.41° and 48.70°. It is clear that φ and φ_d of jointed specimens are lower than those of intact red sandstone specimens expect for the values at 90° angles (namely, showing a

FIGURE 4: A typical evolution of the volumetric strain of red sandstone containing a persistent joint when the confining pressure is 10 MPa: (a) $\alpha = 0^\circ$, (b) $\alpha = 30^\circ$, (c) $\alpha = 45^\circ$, and (d) $\alpha = 90^\circ$.

TABLE 3: The strength and strength ratios of specimens in conventional triaxial compression tests.

Inclination angles	σ_3 (MPa)	σ_c (MPa)	σ_{cd} (MPa)	σ_{ci} (MPa)	σ_{cd}/σ_c	σ_{ci}/σ_c
Intact	0	82.35	79.22	39.78	0.962	0.483
	10	159.55	158.70	107.70	0.995	0.675
	20	211.78	208.20	176.90	0.983	0.835
0°	0	65.03	53.04	39.78	0.816	0.612
	10	134.83	107.35	84.66	0.796	0.628
	20	173.03	142.96	114.84	0.826	0.664
30°	0	57.49	44.37	27.35	0.772	0.476
	10	152.14	88.42	65.12	0.581	0.428
	20	172.37	127.28	80.41	0.738	0.466
45°	0	34.60	33.37	23.76	0.964	0.687
	10	83.67	67.83	50.98	0.811	0.609
	20	115.24	74.43	58.64	0.646	0.509
90°	0	52.94	38.81	24.33	0.733	0.460
	10	146.45	141.52	110.02	0.966	0.751
	20	191.14	159.62	129.78	0.835	0.679

U-shaped trend), which is similar to the conclusion of the specimen containing nonpersistent joints [2].

4. Triaxial Compression Drainage Test Results and Analysis

In this section, triaxial compression drainage tests have been carried out on jointed red sandstone. The mechanical properties and seepage characteristics of specimens with different joint inclination angles are studied under the action of different confining pressures and pore pressures. The effects of the pore pressure and inclination angle on red sandstone hydro-mechanical behavior under triaxial conditions are compared and analyzed.

4.1. Analysis of Stress-Strain Curves. During triaxial compression drainage tests, the mechanical responses of jointed rock specimens will be affected by three factors, i.e., the joint inclination angle α , the pore pressure p , and the confining pressure σ_3 . In the present work, we focus on the effect of the joint inclination angle and the pore pressure on the rock's behavior. Combined with the results of compression tests under different levels of pore pressure and the obtained results in Section 3, the effects of various pore pressures on the characteristics of jointed specimens with the same joint angle and confining pressure are investigated, and the major parameters are listed in Table 6. The statistical results of stress-strain curves with different joint angles are illustrated in Figures 6–8.

Here, we first take the results of the inclination angle at 0° as an example. It can be seen from Figure 6 that under the same σ_3 , with the increase of pore pressure p , the peak strength and the peak strain of the specimens decrease gradually. When σ_3 is equal to 10 MPa, the corresponding peak values of p at 4 MPa and 8 MPa are 113.22 MPa and

FIGURE 5: The fitting results using the M-C criterion: (a) peak stress and (b) peak axial stress with respect to the crack damage.

TABLE 4: Parameters of the peak strength corresponding to the linear M-C criterion.

Joint angles	M	N (MPa)	c (MPa)	φ (°)	R^2
Intact	7.47	86.51	15.83	49.81	0.991
0°	6.40	70.30	13.89	46.86	0.980
30°	6.74	69.89	13.46	47.87	0.908
45°	5.03	37.51	8.36	41.94	0.990
90°	7.91	61.08	10.86	50.85	0.969

108.27 MPa, while the corresponding peak strains are 10.79‰ and 9.62‰, respectively. The peak strength and peak strain of $p = 8$ MPa are 4.37% and 10.88% smaller than those of $p = 4$ MPa, respectively. For σ_3 at 20 MPa, the corresponding peak strength values of $p = 4$ MPa and $p = 8$ MPa are 186.13 MPa and 152.12 MPa, and the corresponding peak

TABLE 5: Parameters of the crack damage threshold corresponding to the linear M-C criterion.

Joint angles	M_d	N_d (MPa)	c_d (MPa)	φ_d (°)	R^2
Intact	7.45	84.22	15.43	49.76	0.987
0°	5.50	56.16	11.97	43.81	0.990
30°	5.15	45.24	9.97	42.44	0.998
45°	3.05	38.01	10.88	30.41	0.935
90°	7.04	52.91	9.97	48.70	0.893

Note: here, we use a subscript “d” for parameters corresponding to the crack damage threshold.

TABLE 6: Strength and deformation parameters of the specimens in triaxial compression drainage tests.

Specimen label	σ_c (MPa)	ε_c (10^{-3})	E (GPa)	E_0 (GPa)
SD-0°-10-4	113.22	10.79	11.40	10.48
SD-0°-10-8	108.27	9.62	12.31	11.66
SD-0°-20-4	186.13	15.25	14.36	14.36
SD-0°-20-8	152.12	12.34	13.95	10.62
SD-45°-10-4	45.10	5.06	11.03	10.16
SD-45°-10-8	21.22	2.74	9.04	8.50
SD-45°-20-4	63.14	6.26	12.88	14.15
SD-45°-20-8	33.99	4.20	11.84	13.55
SD-90°-10-4	126.84	9.11	15.66	14.66
SD-90°-10-8	131.37	13.60	13.64	10.67
SD-90°-20-4	187.31	16.31	20.13	14.54
SD-90°-20-8	185.23	15.68	16.61	14.68

Note: E_0 is the modulus of deformation, namely, the slope of the connection line between the point of the half peak strength and the origin in the stress-strain curve.

strains are 15.25‰ and 12.34‰, respectively. The peak values are reduced by 18% and 8% compared with the previous stage of σ_3 , and the peak strains are reduced by 19% and 7% compared with the previous stage, respectively.

As concluded in the previous section, a larger σ_3 leads to a larger σ_c and a larger ε_c . However, compared with σ_3 , the existence of p may weaken σ_c and decrease ε_c , and the effect of weakening due to p is less obvious than the strengthening of σ_3 . Moreover, for the previous tests under the condition of zero pore pressure, σ_c values of the 0° jointed specimen under σ_3 at 10 MPa and 20 MPa are 148.19 MPa and 207.29 MPa, respectively, while the corresponding peak strength values are reduced by 24% and 10%, respectively, when 4 MPa pore pressure is applied. It can be deduced that the effect of pore pressure is more obvious under the lower confining pressures. The reason for this phenomenon can be described as follows: when under high confining pressure, the mineral microstructure of the specimen is more compact, and hence, the seepage is more difficult to be triggered by the applied pore pressure. Besides, we find that the precompaction effect of the specimens can be more notable under a larger pore pressure condition because the larger pore pressure can weaken the cementation between the mineral components and make the mineral particles easier to be compacted, and

FIGURE 6: The typical stress-strain relationship of specimens with $\alpha = 0^\circ$: (a) $\sigma_3 = 10$ MPa and (b) $\sigma_3 = 20$ MPa.

thus, the precompaction effect is more obvious. The specimen with a 90° joint angle shows the most obvious compaction in this work.

4.2. Analysis of Deformation Characteristics. The relationship of the elastic modulus and the pore pressure is depicted in Figure 9. As the pore pressure increases, the elastic modulus shows a universal and gradual decrease. As a result of the larger pore pressure, it is easier to initiate softening of the mineral composition of the jointed rock and thus results in a larger amount of compression deformation. Therefore, a greater pore pressure leads to a greater compression deformation of the rock under the deviatoric loading, which finally reduces the elastic modulus. In addition, when the pore pressure increases, the difference in the elastic modulus among different confining pressures gradually decreases.

Figure 10 displays the relationship of the elastic modulus and the joint angle. The elastic modulus of the jointed specimen shows a V-shaped trend with the increase of the joint inclination angle, indicating that the existence of the joint

FIGURE 7: The typical stress-strain relationship of specimens with $\alpha = 45^\circ$: (a) $\sigma_3 = 10$ MPa and (b) $\sigma_3 = 20$ MPa.

FIGURE 8: The typical stress-strain relationship of specimens with $\alpha = 90^\circ$: (a) $\sigma_3 = 10$ MPa and (b) $\sigma_3 = 20$ MPa.

induces obvious anisotropy. The minimum values of the elastic modulus all appear at 45° because of the susceptibility of sliding deformation along the 45° joint surface under the action of loading, which leads to a small elastic modulus. Meanwhile, the elastic modulus of the 0° jointed specimen is smaller than the elastic modulus of the 90° jointed specimen; as a result of that, the 0° jointed specimen may produce the compression deformation along the joint surface when the loading is applied, while the 90° jointed specimen can only encounter the deformation of the rock itself. Due to the effect of pore pressure, the softening degree of the joint surface is greater than that of the rock mass, and thus, the axial deformation of the 0° jointed specimen is greater than that of the 90° jointed specimen.

4.3. The Effect of Joint Angles on Strength Characteristics When Considering the Pore Pressure. On the basis of the effect of seepage, the strength of the sandstone specimens will

also show a roughly regular V-shaped change with the increase of the joint inclination angle. Due to the various inclination angles, the seepage paths and seepage pressures are all different on the joint surface, which causes the differences of the strength weakening degree of the rock specimens. Controlling the confining pressure as the same value, the curves of the strength versus the joint inclination angle are shown in Figure 11.

For all the confining pressures and pore pressures, the peak strength of jointed specimens shows a V-shaped change trend with the variation of joint angles. The minimum value is obtained at 45° , the larger values are obtained at 0° and 90° , and the strength of 0° and 90° is almost the same. For instance, under the effect of a confining pressure of 20 MPa and a pore pressure of 4 MPa, the peak values of the 0° , 45° , and 90° jointed specimens are 186.13 MPa, 63.14 MPa, and 187.31 MPa, respectively. Under the action of pore pressure, the peak value of the 45° jointed specimen is only about

FIGURE 9: The relationship of the elastic modulus versus the pore pressure: (a) $\alpha = 0^\circ$, (b) $\alpha = 45^\circ$, and (c) $\alpha = 90^\circ$.

FIGURE 10: The relationship of the elastic modulus versus the joint angle: (a) $\sigma_3 = 10$ MPa and (b) $\sigma_3 = 20$ MPa.

FIGURE 11: The relationship of the strength versus the joint angle: (a) $\sigma_3 = 10$ MPa and (b) $\sigma_3 = 20$ MPa.

FIGURE 12: The failure modes and their corresponding sketch maps of specimens: (a) $\alpha = 0^\circ$, (b) $\alpha = 45^\circ$, and (c) $\alpha = 90^\circ$.

one-third of the peak value of the 0° and 90° jointed specimens, indicating that the oblique joint plane has a great influence on the strength of the rock mass when considering the pore pressure.

4.4. The Failure Mode Analysis. The failure modes and their corresponding sketch maps of specimens are demonstrated in Figure 12. All jointed specimens basically exhibited the shear failure mode, especially the 45° jointed specimen. The

0° joint sample not only has the shear failure surface through the rock but also has the failure surface along the joint surface. For zero pore pressure conditions, the 0° joint sample is dominated by the shear failure surface through the rock, with slight cracks along the joint surface. When the pore pressure is applied, the 0° joint surface will be penetrated by fractures, which has more obvious cracks characteristics than that under the condition of no pore pressure, indicating that the effect of the pore pressure can affect the failure mode of

the 0° jointed rocks to a certain extent. The water flow penetrates into the 0° joint surface, which weakens the degree of cementation between the jointed material and the rock. After a penetrative fracture surface is formed under the deviatoric loading, the fracture surface will also occur along the 0° joint surface. The 90° jointed specimens mainly encounter the shear fracture surface which penetrates the rock, and the joint plane basically does not affect the failure mode of the rock.

Comparing the failure modes under different pore pressures, it is found that the pore pressure will affect the damage degree of the 0° jointed specimen; i.e., the fracture surface along the joint plane can be more obvious. Unlikely, there is basically no effect of the pore pressure on the failure mode of the 45° and 90° jointed specimens, indicating that the pore pressure has relatively little effect on the shear failure modes of the rock. Comprehensively considering the failure modes of the specimens, four main failure modes under the effect of pore pressure are summarized as below: (1) the penetrating shear crack and the crack along the 0° joint plane for the 0° jointed specimens, (2) the shear crack along the joint surface for the 45° jointed specimens, (3) the penetrating shear crack of the whole rock for the 90° jointed specimens, and (4) the shear cracks which first initiate along the shear surface and then penetrate the whole rock for the 90° jointed specimens.

5. Conclusions

In this study, the conventional triaxial compression experiments and triaxial compression drainage tests for intact red sandstone and specimens with a single persistent joint are conducted. Based on the results, the stress-strain relationships, the deformation behavior, the strength parameters, and the failure characteristics are analyzed.

- (1) According to results of conventional tests, the post-peak stress-strain curves of intact and jointed red sandstone all demonstrated the nonlinear stage, the elastic stage, the stable crack growth stage, and yielding stage, while the crack-induced volumetric strain versus axial strain curves of jointed specimens with different joint angles demonstrated a similar four-stage trend. Moreover, the relationship between the peak strength and the crack damage threshold fits the linear Mohr-Coulomb criterion well, and the mechanical parameters of specimens show a U-shaped trend with the raising of joint angle
- (2) The results of triaxial compression drainage tests reveal that the elastic modulus of specimens gradually decreases with the rise of pore pressure and shows a V-shaped trend with the increase of the joint inclination angle. Similarly, the strength parameters show a V-shaped change trend for the enlargement of the pore pressure and the joint inclination angle. Finally, the crack modes in sandstone containing a single persistent joint under triaxial compression are further discussed

The conclusions of this study can be considered a reference for further studies with a wider range of rock and rock-like materials, joint geometries, and stress conditions.

Data Availability

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

Conflicts of Interest

The authors declare no conflicts of interest.

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