In order to study the weakening mechanism of mechanical properties and the evolution of fracture of water-bearing rocks, cylindrical standard siltstone samples with four moisture contents (0, 2.85, 3.87, and 4.25%) were prepared, and the mechanical properties, damage mode, AE characteristics, and fractal law of water-bearing rock samples were studied by means of uniaxial compression test and acoustic emission (AE) monitoring technique, and based on the test findings, a constitutive model of the entire process of rock deformation and damage under uniaxial compression was built with different moisture content of rock. The results show that with the increase of moisture content, the peak stress, the stress threshold of void compaction stage, and the stress threshold of elastic stage of the rock samples decreased linearly, elastic modulus decreased exponentially as a function, and the peak strain, the strain threshold of void compaction stage, and the strain threshold of elastic stage increased linearly. The higher the moisture content, the weaker the AE signal intensity and the smaller the AE count value. From dry to saturated, the damage form of rock samples gradually transitioned from predominantly tensile damage to predominantly shear damage. The fractal dimension of the broken block linearly decreases as the moisture content rises. The model constructed in this paper has good applicability to the deformation characteristics of water-bearing rocks under uniaxial compression before the peak stress; it especially can express the rock void compaction stage, but it cannot accurately describe the postpeak deformation characteristics.

1. Introduction

Water-rock interaction is one of the important factors that cause the geological hazards, and the processes of deformation, destabilization, and damage of rock masses often involve water, such as rainfall leading to slope sliding [1, 2], water level rise and fall causing dam base landslides [3, 4], water-rich roadway and tunnel roof destabilization [5, 6], and groundwater flow-induced earthquakes [7]. Water-rock interaction in coal mining can have a negative impact on engineering rock stability and constitute a severe threat to coal mine safety production. For example, (1) in the study of water-rich roadway or water-rich working face surrounding rock control, the rock support parameters of the top and bottom slabs are different from those of general roadways or working faces because of the influence of water infiltration; (2) coal mining on or under pressurized water should maintain the dynamic balance between the water pressure of the aquifer and the water barrier capacity of the water barrier to prevent sudden water [8]; (3) in the study of ecological, the study of underground reservoir water storage or water retention mining in fragile mining areas should consider the influence of water on the boundary coal rock columns.
stability, the underground reservoir water storage coefficient, and the development pattern of water-conducting fracture zones [9].

The physical, chemical, and mechanical processes that occur in rocks under certain water pressure are the root cause of deformation and damage in engineering rocks, not only the mechanical effects that are simply considered from the effective stress principle [10–12]. The principle of water-rock action is shown in Figure 1. Water changes the mechanical properties of rocks by changing the structure and stress state of defects and is a key factor in controlling rock damage [13]. Roy et al. [14] carried out uniaxial compression test and Brazilian splitting test of sandstone and shale with different saturation and came to the conclusion that the mechanical characteristics of the rocks—including their uniaxial compressive strength, tensile strength, modulus of elasticity, tensile strength, and fracture toughness—decreased with increasing saturation. Daraei and Zare [15] looked into the role of moisture content on the critical and breaking strains of rocks by uniaxial compression tests, and the rocks used in the tests included eight types of sandstone, conglomerate, mudstone, limestone, siltstone, schist, sillimanite, and amphibolite, and found that the critical strain increased after most of the rocks reached saturation, and as the moisture content increased, the maximum value of rock-breaking strain is 2.14 times of the critical strain. Wang et al. [16] conducted mechanical and permeability tests on limestone and coarse sandstone before and after fracture under hydraulic coupling and found that with increasing water pressure differences, strength, deformation modulus, and permeability decreased, while Poisson’s ratio increased, changing the physical and mechanical characteristics of the samples by modifying the stress conditions and microstructure of the samples. Sha et al. [17] studied the creep properties of water-rich roadway perimeter rock under the action of seepage flow through a homemade seepage-creep coupled test system and found that the maximum creep deformation of saturated specimen samples under the same perimeter pressure of sandstone is greater than that of natural water-bearing samples.

The degree of fracture extension at different damage stages is directly correlated with the strength of rocks. Different methods have been used by researchers to explore the spatial distribution of rock fractures. The energy mechanism can reflect the damage and failure process of water-bearing rocks and provide information on damage prognosis [18]. The energy storage and release capacity of rock samples both decrease with the increases of moisture content [19, 20]. Acoustic emission (AE) tests are useful for characterizing the extent of rock damage [21], locating damage locations, and determining fracture types [22, 23]. The complexity and evolutionary mechanism of interior rock fractures are related to block fractal properties after rock destruction, and they play a crucial statistical role in exposing the method of rock destruction [24, 25].

The construction of rock intrinsic constitutive equations is the basis for the research and development of rock mechanics. To represent the link between stress and strain in rocks, researchers frequently create statistical damage intrinsic equations based on the damage mechanics of continuous media and statistical strength theory [26, 27]. However, fewer are able to describe void compaction stage of rocks. For uniaxial compression or triaxial compression tests with small circumferential pressures, most nondense rocks have a significant void compaction stage. The water-rock interaction causes the mechanical properties of rocks to be reduced and internal pores and microfractures (collectively referred to as voids) to be more developed, and void compaction stage is more pronounced in water-bearing rocks under uniaxial compression or lower envelope pressure than in dry rocks [28], and it is impossible to ignore the void compaction stage of the constitutive model.

Previous studies on water-rock interaction have concentrated on the variation of rock mineral structure and the effect of loading methods on rock mechanical properties [29]. However, there are relatively few studies on the fracture types and damage forms of rocks under the effect of moisture content during loading, the spatial evolution law, and the construction of rock ontological equations considering water-rock interaction. In this paper, four kinds of cylindrical siltstone samples with different moisture contents, including dry and water-saturated states, were prepared based on the self-developed water immersion device, and uniaxial compression tests were carried out by servo testing machine, and at the same time, AE signals were monitored simultaneously. Based on the test results, a rock constitutive model considering the effect of moisture content under uniaxial compression was constructed. The research results provide useful references for the water-rock interaction problems involved in the calculation of water-conducting fracture zones in water conservation mining, the design of underground reservoir dams, the retention of water-resistant coal rock columns, and the stability control of water-rich roadway surrounding rocks.

2. Experimental Design

2.1. Preparation of Rock Samples. The siltstone samples were taken from the direct roof of the 12306 working face of the Shangwan coal mine in the Shandong mining area, located in northwestern China. The retrieved rock was processed into 32 standard cylinders of 50 mm in diameter and 100 mm in height according to the International Commission on Rock Mechanics (ISRM) [30, 31]. It is required that the nonparallelism of the two ends of the rock samples should not be greater than 0.001 mm, the deviation of the upper and lower diameters should not be greater than 0.02 mm, and the top surface and the side surfaces should be perpendicular to each other with a maximum deviation of not more than 0.25°. 12 rock samples were selected for uniaxial compression test, and the remaining rock samples were used for physical properties test and as backup samples. The sampling locations and the prepared rock samples are shown in Figure 2.

A dried rock sample was cracked into a mortar and ground into a powder with less than 38 μm sized particles, and XRD tests were performed using an X-ray diffractometer (D8 ADVANCE, Bruker, Germany).

Table 1 and Figure 3 display the XRD test outcomes for the rock sample. The siltstone’s primary mineral components are quartz, albite, orthoclase, illite, kaolinite, and chlorite. The majority of the mineral makeup, at 27.3%, is made up of quartz. The clay minerals (illite, kaolinite, and chlorite) account for 48.4% of the overall mineral composition, with illite, kaolinite,
and chlorite accounting for 22.4%, 8.2%, and 17.8% of the overall mineral composition, respectively. Illite has a high water absorption capacity, which causes the water film that mineral particles have absorbed to thicken when it comes into contact with water and cause the rock’s volume to expand, squeezing the surrounding structures and other minerals. Rock’s mechanical characteristics will be weakened as a result of unequal volume expansion, which will cause uneven stress in the rock. After coming into contact with water, kaolinite becomes plastic, which enhances a rock’s capacity for plastic deformation. The most weathering-sensitive material is chlorite.

Table 1: Mineral content of rock sample.

| Mineral     | Quartz | Albite | Orthoclase | Illite | Kaolinite | Chlorite | Other |
|-------------|--------|--------|------------|--------|-----------|----------|-------|
| Content/%   | 27.3   | 12.9   | 9.5        | 22.4   | 8.2       | 17.8     | 1.9   |

2.2. Test Equipment and Scheme. The automatic nondestructive water immersion device (ANDWID) [32, 33] is shown in Figure 4(a). The device is independently developed by the team and is composed of ultrasonic humidifier, high-precision stress sensor, data acquisition and transmission system, and pumping system. It can realize nondestructive immersion of samples, recycling of water and automatic monitoring and recording of moisture content.

The calculation formula of moisture content is

$$w_i = \frac{1}{3} \sum_{i=1}^{3} \left( \frac{m_{it} - m_{i0}}{m_{i0}} \times 100\% \right),$$

where $w_i$ is the average moisture content of rock sample at immersion $t$ time (%), $m_{i0}$ is the mass of the $i$th rock sample when dried (kg), and $m_{it}$ is the mass of immersion $t$ time of the $i$th rock sample (kg).

The loading and monitoring system is shown in Figure 4(b). The loading system was an electrohydraulic servo-controlled universal testing machine (C64.106, MTS Industrial Systems (China) Co., Ltd.) with a maximum load of 1000 kN and a measurement accuracy of ±0.5%. The monitoring system included a static resistance strain gauge.
The water immersion test of rock samples was carried out using ANDWID to obtain the variation pattern of moisture content of rock samples with immersion time, and based on the test results, 4 rock samples with different moisture contents were created. Uniaxial compression tests were performed on the four rock samples with different moisture contents using displacement control (0.2 mm/min), and during the loading procedure, the strain and AE signals were collected simultaneously. The sampling threshold of the AE system was set to 40 dB with a gain of 40 dB, and the AE transducers (PCI-2, Physical Acoustic Corporation, USA) operated in the frequency range of 125-750 kHz with a resonance frequency of 20 kHz.

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140 kHz. The number of AE transducers was 6, distributed crosswise at the top and bottom of the rock sample. Table 2 contains a list of the main mechanical parameters of the rock samples.

3. Results and Discussion

3.1. Relationship between Moisture Content and Water Immersion. According to Figure 5, the curves were divided into 3 stages according to the change of slope: (I) fast growing stage (0-12 h), (II) slow growing stage (12-24 h), and (III) stable stage (24-60 h). The moisture content increased rapidly with immersion time from 0 to 12 h. The water was immersed from the surface of the rock sample into the internal pores and dispersed in different positions of the rock sample, and with the continuous immersion of water, the water gradually connected at each position and moved toward the center of the rock sample. From 12-24 h, the moisture content continues to increase, but the rate of increase decreases. The complete seepage channels inside the rock sample gradually formed, and the water had already infiltrated into most of the pores inside the rock sample. From 24-60 h, the moisture content of the rock sample remained almost constant, which means that the rock sample has reached saturation.

According to the water immersion pattern of rock samples, we selected four representative time points, 0 h (dry), 8 h (rapid growth stage), 16 h (slow growth stage), and 60 h (saturation), representing rock samples with moisture content of 0, 2.85%, 3.87%, and 4.25%, respectively.

The following is an expression for the relationship between the moisture content of the rock sample and the immersion time:

\[ w_t = -4.26e^{-t/7.2} + 4.27, \]

\[ R^2 = 0.99, \]

where \( t \) is the immersion time (h) and \( R^2 \) is the correlation coefficient.

The test data fit the curve in formula (2) quite well. This formula can be used to determine the moisture content through the immersion time.

The rock samples for uniaxial compression tests were labeled based on the 4 chosen moisture contents. The abbreviations “W0,” “W1,” “W2,” and “W3” stand for rock samples with moisture contents of 0, 2.85, 3.87, and 4.87%, respectively. The subsequent serial numbers “1,” “2,” and “3” were then used to denote the first, second, and third rock sample, respectively. For instance, W1-2 denotes the second rock sample with a moisture content of 2.85%, W2-3 denotes the third rock sample with a moisture content of 3.87%, etc.
3.2. Effect of Moisture Content on Mechanical Properties

3.2.1. Relationship between Stress and Strain. It can be seen from Figure 6 that the stress-strain curves of rock samples with different moisture contents are consistent, and they can all be categorized into four stages: void compaction stage, elastic stage, plastic stage, and postpeak stage [34]. In void compaction stage, the curve is concave and the slope gradually increases, which is due to the existence of voids inside the rock sample, which are gradually compacted under the action of stress. When the rock sample is in the elastic stage, the stress rises linearly with the strain and causes recoverable elastic deformation. In the plastic stage, the curve is concave, the slope gradually decreases, and part of the curve starts to fluctuate (W-0-1), and the rock sample produces plastic deformation. The initial stress in the postpeak stage is the peak stress of the rock sample (i.e., uniaxial compressive strength), and the curve begins to decline, indicating that the rock sample has undergone damage and gradually lost its bearing capacity. The curve of the postpeak stage of dry rock samples decreases rapidly, producing a violent brittle damage. With the increase of moisture content, the postpeak curve of the rock sample begins to appear “stepped” characteristics, indicating that water reduces the brittleness of the rock sample, reducing the degree of damage of the rock sample. The water-rock interaction weakened the cohesive force inside the rock sample and enhanced plasticity of the rock sample. After the rock sample entered the postpeak stage, the water-bearing rock sample with strong plastic deformation ability was not instantly destroyed completely but continued to be damaged by the force and gradually.

3.2.2. Effect of Moisture Content on Peak Stress, Peak Strain, and Elastic Modulus. Figure 7 depicts the impact of moisture content on the peak stress, peak strain, and elastic modulus of the rock sample. The following is the elastic modulus calculation formula:

\[
E = \frac{\sigma_E - \sigma_{com}}{\varepsilon_E - \varepsilon_{com}},
\]

where \(E\) is the elastic modulus (GPa), \(\sigma_E\) and \(\sigma_{com}\) are the end point stress of elastic stage and void compaction stage, respectively (MPa), and \(\varepsilon_E\) and \(\varepsilon_{com}\) are the end strain of elastic stage and void compaction stage, respectively.

As can be seen from Figure 7, the effect of moisture content on the peak stress, peak strain, and elastic modulus of the rock samples is obvious. The peak stress of the rock samples reduced linearly as the moisture content increase, while the peak strain increased linearly. From dry to saturated, the average value of peak stress of rock samples decreased from 46.42 MPa to 21.93 MPa, with a decrease of 52.8%, and the average value of peak strain increased from 0.012 to 0.019, with an increase of 58.3%. The elastic modulus of rock samples with different moisture contents decreased approximately as an exponential function. The average elastic modulus of dry rock samples was 4.93 GPa, while the average elastic modulus of other three moisture content rock samples were 2.56, 2.37, and 2.18 MPa, respectively, with a decrease of 48.1%, 51.9%, and 55.8%, respectively, compared with that of dry samples.

After water immersion of the rocks, water is distributed in the internal voids of the rocks, weakening the connection forces between the structures. The scouring impact of water causes the mobility of free particles in the rock voids, which in turn produces changes in the microstructure of the rocks. The presence of water causes the illite in the rocks to swelling and disintegrate. The lubricating effect of water reduces the coefficient of friction between particle contact surfaces and between microfracture surfaces, decreasing the relative sliding difficulty and therefore reducing the elastic modulus of the rock. The weak coupling effect of hydroxide ions in water on the rock changes the microstructure of the rock, enhances plasticity of the rock, reduces the stiffness of the rock, and increases the susceptibility to deformation of the rock.

3.2.3. Effect of Moisture Content on Threshold of Void Compaction Stage and Elastic Stage. The effects of moisture content on the stress threshold and strain threshold of the rock sample in void compaction stage and the stress threshold and strain threshold of elastic stage were obtained according to the four stages of the division of the stress-strain curve of the rock sample, as shown in Figure 8, where the stress threshold and strain threshold are the stress and strain values at the end of the stage, respectively.

As shown in Figure 8, the stress thresholds of both void compaction and elastic stages decrease linearly, and strain threshold increase linearly as moisture content increases. From dry to saturation, the average values of stress threshold of void compaction stage and elastic stage decreased from 9.84 MPa and 29.35 MPa to 7.04 MPa and 13.36 MPa, respectively, with a decrease of 28.5% and 54.5%, respectively. The mean values of strain threshold of void compaction and elastic stages increased from 0.005 and 0.009 to 0.011 and 0.014, respectively, with an increase of 120% and 55.6%. The scouring effect of water will enlarge the internal void of rock and
increase the deformation of void compaction stage. The softening effect of water will enhance the plasticity of rock, reduce the stress required for the rock to enter the plastic stage, and increase the strain threshold of elastic stage. The relatively large dispersion of stress threshold of void compaction stage is related to the anisotropy of the rock. The mineral composition, particle morphology, and microstructural connectivity within the rock can have some effect on the results.

3.3. Effect of Moisture Content on Energy Storage and Dissipation. The mechanical energy generated by the external load on the rock is transformed into the input energy of the rock, part of energy is elastic strain energy that can be released and stored in rock, and the other part is the dissipative energy generated by the friction of the rock rupture surface, which is the intrinsic driving force of the rock damage and leads to the generation, development, and extension of rock fractures. Rock fracture closure, production, and extension are caused by the transformation of input energy, elastic strain energy, and dissipation energy, which ultimately results in rock damage.

According to the first law of thermodynamics, and assuming that the loading process of rock is a closed system without heat exchange with the outside environment, the following result is reached:

\[ U = U^e + U^d, \]  

where \( U \) is the total input energy density, also known as the total input energy per unit volume (\( J/m^3 \)); \( U^e \) is the elastic strain energy density, also known as elastic strain energy per
unit volume (J/m³); and $U_d$ is the dissipative energy density, also known as the dissipative energy per unit volume (J/m³).

The following are the formulas used to calculate the input total energy density, elastic strain energy density, and dissipated energy density [35]:

$$ U = \int_0^\epsilon \sigma \, d\epsilon, \quad (5) $$

$$ U^e = \frac{1}{2} \sigma_1 \epsilon_1^e + \frac{1}{2} \sigma_2 \epsilon_2^e + \frac{1}{2} \sigma_3 \epsilon_3^e, \quad (6) $$

$$ U^d = U - U^e. \quad (7) $$

For uniaxial compression test, $\sigma_2 = \sigma_3 = 0$, equation (6) is simplified as

$$ U^e = \frac{1}{2} \sigma_1 \epsilon_1^e = \frac{\sigma_1^2}{2E}. \quad (8) $$

Multiply the total input energy density, elastic strain energy density, and dissipated energy density (equations (5), (7), and (8)) by the volume of the rock. The following are the calculating formulas:

$$ U_V = U V_r = V_r \int_0^\epsilon \sigma \, d\epsilon, \quad (9) $$
\[ U^e_V = U^e V_r = \frac{\sigma^2 V_r}{2E} \]  \hspace{1cm} (10)

\[ U^d_V = U^d V_r = (U - U^e) V_r \]  \hspace{1cm} (11)

where \( U_V \) is the total input energy (J), \( U^e_V \) is elastic strain energy (J), \( U^d_V \) is the dissipated energy (J), and \( V_r \) is the rock volume (m\(^3\)).

The total energy, releasable elastic strain energy, and dissipated energy of rock samples are calculated by equations (9)-(11).

The changes of total energy, releasable elastic strain energy, and dissipated energy of rock samples with different moisture content are shown in Figure 9.

As can be seen from Figure 9, the energy change pattern of rock samples with different moisture content during loading is similar. The total energy-strain curve and the elastic strain energy-strain curve are roughly parallel before the peak stress and continue to grow with increasing strain, and the slope of the curve gradually increases. The dissipative energy-strain curve of void compaction stage (OA) is above elastic strain energy-strain curve, indicating that the dissipative energy in this stage is larger than elastic strain energy. The dissipative energy-strain curves of elastic stage (AB) and plastic stage (BC) are below elastic strain energy-strain curve, indicating that elastic strain energy is greater than the dissipative energy. The dissipative energy is greater than the elastic strain energy from a certain strain after the peak because the elastic strain energy-strain curve decreases rapidly and the dissipative energy-strain curve increases rapidly after the peak stress. This suggests that the most of the energy absorbed by the rock sample prior to damage is stored as elastic strain energy. This energy is then converted into dissipative energy and released when the stress reaches the peak instantly, the internal fractures expand and penetrate quickly, and the rock sample is destabilized and damaged. The total input energy at the peak stress was 45.9 J and 24.4 J for dry and saturated samples, respectively, and elastic strain energy was 41.4 J and 19.4 J. The decrease of total input energy and elastic strain energy from dry to saturated was 45.4% and 40.9%, respectively. The water-rock interaction damaged the internal structure of the rock sample containing water, the water weakens the cohesion and strong mechanical properties can accumulate a large amount of energy under load and release it at the moment of destruction, producing a strong AE signal. After the rock sample contains water, the water weakens the cohesion and structure of the rock sample, lowers the friction coefficient between the fracture surfaces, and decreases the energy accumulation, and the energy released after the destruction of the rock sample is smaller than that of the dry rock sample. The higher the moisture content, the greater the effect of water-rock interaction, the weaker the AE signal intensity generated, and the smaller the AE count value. The weakening of AE signal intensity and AE count value will lead to the decrease of cumulative AE count. The slope of the curve increases significantly after entering the plastic stage. The cumulative AE count-time curve can reflect the mechanical properties of the rock sample at each loading stage, but the location of the peak point of the stress is difficult to judge and needs to be analyzed in combination with the stress-time curve.

3.4. Effect of Moisture Content on Fracture Evolution Law

3.4.1. Effect of Moisture Content on AE Count. The number of oscillations of the signal that exceeding the threshold is known as the AE count. The degree of internal rock damage is positively correlated with the AE count inside the rock. The larger the AE activity, the higher the energy released, and the more serious the rock damage. In order to analyze the internal damage, fracture progression, and energy release of rock samples with different moisture contents at different loading stages, the stress-AE count-time curve and cumulative AE count-time curve were plotted, as shown in Figure 10.

As can be seen from Figure 10, there is good correlation between the stress-time curves and the AE count-time curves of rock samples with different moisture contents, and the AE signals are distributed in each stage of the loading process, and there is an AE count maximum near the peak stress, indicating that the AE activity and internal damage of the rock samples are most intense near the peak stress. In void compaction stage (OA), the AE signal of the rock sample is less, AE is inactive, and the internal primary void is gradually compacted and closed with little energy released. In elastic stage (AB), compared with void compaction stage, the AE signal is more active, the rock sample undergoes elastic deformation, and the internal fracture edges rub. In the plastic stage (BC), compared with the first two stages, the AE signal is further enhanced, the internal fracture of the rock sample is continuously expanded and penetrated, and the fracture surface is extruded, sliding and misshapen, which makes the degree of damage intensify.

The AE count value gradually decreases at the same stage as the moisture content rises. The maximum value of AE count for dry rock samples was 5006, and the maximum values of AE count for moisture content 2.85%, moisture content 3.87%, and saturated rock samples were 3493, 2779, and 2466, respectively, with the decreases of 30.2%, 44.5%, and 50.7%, respectively, compared with dry rock samples. The production of an AE signal in rock samples is inhibited by water, and the inhibitory effect is more pronounced with higher moisture content. Dry rock samples with structural integrity, high cohesion, and strong mechanical properties can accumulate a large amount of energy under load and release it at the moment of destruction, producing a strong AE signal. After the rock sample contains water, the water weakens the cohesion and structure of the rock sample, lowers the friction coefficient between the fracture surfaces, and decreases the energy accumulation, and the energy released after the destruction of the rock sample is smaller than that of the dry rock sample. The higher the moisture content, the greater the effect of water-rock interaction, the weaker the AE signal intensity generated, and the smaller the AE count value. The weakening of AE signal intensity and AE count value will lead to the decrease of cumulative AE count. The slope of the curve increases significantly after entering the plastic stage. The cumulative AE count-time curve can reflect the mechanical properties of the rock sample at each loading stage, but the location of the peak point of the stress is difficult to judge and needs to be analyzed in combination with the stress-time curve.

3.4.2. Effect of Moisture Content on Fracture Development Type and Failure Form. The failure form of rock is the result of the comprehensive action of structural strength and stress distribution [36]. Rising angle (RA value) and average frequency (AF value) in AE, where RA value is the ratio of rising time to amplitude and AF value is the ratio of ringing count to duration, can reveal the type of fissures in the rock [37]. Research shows that [38, 39] shear fracture production or development is represented by acoustic emission signals with low AF and high RA values and vice versa. Therefore, RA-AF relationship can be used to distinguish the fracture development type and failure form of rock.
The RA-AF relationship density clouds plotted for rock samples with different moisture content are shown in Figure 10 for void compaction stage, elastic stage, plastic stage, and the whole process from the start of loading to the peak stress, respectively. The red area and blue area in the figure represent the maximum and minimum (0) data densities, respectively, the red-dashed line is the line of tensile and shear fracture division, and the data density of the area within the black-dotted line is higher than 6, indicating the concentrated distribution of RA-AF. The data in the range of RA values of 0-100 ms/v and AF values of 0-1000 kHz account for more than 98% of the total data and are representative [40], and the small amount of discrete data beyond the above range is not analyzed.

From Figure 11, it can be seen that the evolution of RA-AF relationship density cloud diagrams of rock samples with different moisture contents in void compaction stage, elastic stage, and plastic stage shows similarity. At the early stage of loading, the high-density nucleation zone is mainly distributed above the division line of tension and shear fractures, indicating that tension fractures and shear fractures are generated and developed simultaneously during the loading process, and tension fractures are dominant. The rate of microfracture emergence and development increases, the AE signal grows significantly, and the range of RA-AF high-density core area gradually expands and tends to move below the division line of tensile and shear fractures as load increase and damage inside the rock sample increase. It shows an increase in the shear fissures caused by the late loading stage.

The macroscopic damage form of the rock sample may be reflected by the RA-AF relational density cloud map of the whole loading process. With the increase of moisture content, the RA-AF high-density nucleation zone gradually...
shocks below the division line of tension and shear fracture, and the data of shear fracture zone gradually increases, indicating that the damage form of rock samples changes from mainly tension damage, to compound damage of tension-shear, to mainly shear damage from dryness to saturation. There are a large number of primary closed fractures inside the rock, and the fracture surface produces a misalignment or misalignment trend under the action of load, and the

Figure 10: Corresponding relationship between AE count, cumulative AE, and stress of rock samples with different moisture content. (a) Stress AE count time curve of W0-1, (b) stress AE count time curve of W1-1, (c) stress AE count time curve of W2-1, (d) stress AE count time curve of W3-1, and (e) cumulative AE count time curve.
friction between the fracture surfaces will hinder this misalignment or misalignment trend. When the rock contains water, the lubricating effect of water will reduce the friction coefficient between fracture surfaces and reduce the relative sliding difficulty of fracture surfaces. Water will also weaken the cementation between rock particles and change the connection state between mineral particles, thus increasing the possibility of rock shear damage.

3.5. Effect of Moisture Content on Fractal Law of Broken Block. In order to study the influence of moisture content on the fractal characteristics of rock fragmentation, the fragmentation of rock samples is divided into 0-2.5, 2.5-5, 5-10, 10-15, 15-20, 20-25, 25-30, 30-40, 40-50, and 50-100 mm intervals by using the grading screen and scale with the aperture of 2.5, 5, 10-5, 10-15, 15-20, 20-25, 25-30, 30-40, 40-50, and 50-100 mm; by using an electronic balance, the mass of fractured blocks with different fragmentations is measured. Figure 12 displays the distribution of broken blocks of rock sample.
According to the relevant research on fractal theory of fragmentation of broken rock samples [41], the ratio of mass $M_d$ of broken blocks with fragmentation less than $d$ to total mass $m$ of broken blocks is

$$\frac{M_d}{M} = \frac{d^{3-D} - d_{\text{mix}}^{3-D}}{d_{\text{max}}^{3-D} - d_{\text{mix}}^{3-D}},$$  \hspace{1cm} (12)$$

where $d_{\text{mix}}$ is the minimum fragment size of broken block (mm) and $d_{\text{max}}$ is the maximum fragment size of broken rock block (mm).

Assuming $d_{\text{mix}} = 0$, equation (12) is simplified as

$$\frac{M_d}{M} = \frac{d^{3-D}}{d_{\text{max}}^{3-D}},$$  \hspace{1cm} (13)$$

Take logarithms on both sides of equation (13) at the same time to obtain the fractal dimension calculation formula of rock broken block size as follows:

$$\ln \frac{M_d}{M} = (3 - D) \ln \frac{d}{d_{\text{max}}}. $$  \hspace{1cm} (14)$$

From equation (14), the slope of the straight line in the $\ln (M_d/M)$ plane right-angle coordinate system is $3 - D$. The relationship of $\ln (M_d/M) - \ln d$ for rock samples with different moisture content is shown in Figure 13(a). Based on the measurement results, the fractal dimension of the rock fractured masses can be calculated by linear fitting. The larger the number of small fractured blocks, the larger the $\ln (M_d/M)$, the smaller the slope of the line, and the larger the fractal dimension $D$. It is clear that the rock sample is more broken with larger fractal dimension.

The fractal dimension of broken blocks after failure of rock samples with different moisture content is shown in Figure 13(b). From Figure 13(b), the average values of fractal dimension after destruction of rock samples with four moisture contents are 2.231, 2.067, 2.044, and 2.008, respectively. The fractal dimension decreases by 10% from dry to saturated conditions, and it decreases linearly with increasing moisture content. It shows that there are more massive blocks and less fragmentation of the rock sample after destruction the greater the moisture content. This is because the presence of water weakens the interparticle connectivity, dissolves and softens the clay minerals in the rock, and makes the internal structure of the rock change. The primary micro pores and fissures are penetrated by dissolution, and under the load, the rock is more likely to expand and develop along the already penetrated micro pores and fissures and eventually form macroscopic fissures, which makes the number of macroscopic fissures produced by the

---

**Table 3: Parameters of rock sample model.**

| Sample number | $m_1$ | $F_{01}$ | $m_2$ | $F_{02}$ |
|---------------|-------|----------|-------|----------|
| W0-1          | 0.718 | 43.692   | 338.356 | 39.490   |
| W1-1          | 1.832 | 48.925   | 142.417 | 22.538   |
| W2-1          | 2.382 | 48.137   | 32.230  | 18.838   |
| W3-1          | 2.400 | 35.993   | 7.759   | 19.607   |

---

**Figure 13:** Calculation of fractal dimension. (a) $\ln x - \ln (M_d/M)$ relationship and (b) relationship between fractal dimension and moisture content.
rock decrease, and eventually, the size of broken rock masses increases after destruction.

3.6. Rock Constitutive Model considering the Influence of Moisture Content

3.6.1. Construction of Constitutive Model. Based on Lemaitre’s strain equivalence hypothesis [42], the strain of the material is equivalent before and after deformation. The effective stress and apparent stress of rock have the following relationships:

\[ [\sigma] = [\sigma^*][I - D], \]

where \( D \) is the damage variable, \( I \) is the identity matrix, \( [\sigma] \) is the apparent stress matrix (MPa), and \( [\sigma^*] \) is the effective stress matrix (MPa).

Microelement failure in rock occurs in a random manner under the action of load. The expression defining \( D \) is

\[ D = \frac{N_d}{N}, \]

where \( N_d \) is the number of damaged microelements and \( N \) is the total number of microelements.

Assuming that the rock macroscopically satisfies the isotropic condition, the strength of rock microelement follows Weibull distribution, the microelement has linear elastic properties before failure, and the generalized Hooke law is obeyed by the deformation characteristics, and the probability density function of a rock microelement is given by

\[ P(F) = \frac{m}{F_0} \left( \frac{F}{F_0} \right)^{m-1} e^{-\left( \frac{F}{F_0} \right)^m}. \]

According to simultaneous equations (15) and (16), the expression of \( D \) based on Weibull distribution can be obtained as follows:

\[ D = \frac{N_d}{N} = \frac{f^{NP}(x)}{N} = 1 - e^{-\left( \frac{F}{F_0} \right)^m}, \]

where \( m \) is the homogeneity degree, \( F \) is the random distribution variable of microelement strength (MPa), and \( m \) and \( F_0 \) are Weibull distribution parameters.

Mohr-Coulomb criterion is used to express the strength of microelements [43], and the expression is as follows:

\[ F = f(\sigma^*) = \sigma_1^* - \frac{1 + \sin \phi}{1 - \sin \phi} \sigma_3^*, \]

where \( \phi \) is the internal friction angle of microelement (°).

The primary stress of generalized Hooke’s law is expressed as follows:

\[ \sigma_1^* = E\varepsilon_1 + \nu(\sigma_2^* + \sigma_3^*). \]

According to simultaneous equations (15) and (18)–(20), the statistical damage constitutive equation of rock based on Mohr-Coulomb criterion and expressed by apparent stress is

\[ \sigma = E\varepsilon e^{-\left( \frac{F}{F_0} \right)^m}. \]

For rock uniaxial compression test, \( \sigma_2 = \sigma_3 = 0 \), equation (21) is simplified as

\[ \sigma = E\varepsilon e^{-\left( \frac{F}{F_0} \right)^m}. \]

The aforementioned formula represents the statistical damage constitutive equation for rock taking the effect of moisture content during uniaxial compression into consideration.

According to the above analysis, under uniaxial compression, siltstone samples with different moisture content exhibit a clear compaction stage, and the stress-strain curve is concave. The greater the moisture content, the longer the compaction stage. This is because the rock is a porous medium material composed of pores, microcracks, and matrix. Under the action of load, the internal primary voids are closed first. After the rock is immersed by water, water-rock interaction will take place and change the interior structure of rock, wash and dissolve rock particles, cause particle migration, and then increase the pores in the rock. However, the stress-strain curve obtained from the statistical damage constitutive equation is convex. Therefore, when constructing the constitutive equation of the whole process of deformation and failure of rocks with different moisture content under uniaxial compression, the compaction stage in the stress-strain curve should be separated from other stages (elastic stage, plastic stage, and post peak stage).

The shape of rock stress-strain theoretical curve is determined by the properties of the Weibull distribution function based on statistical damage constitutive equation that is mainly affected by exponential function \( \exp \left( -\left( \frac{E\varepsilon}{F_0} \right)^m \right) \). Referring to the method of Deng et al. [44], according to the concave characteristics of the stress-strain curve in the compaction stage, the exponential function in the constitutive equation is changed to \( 1 - \exp \left( -\left( \frac{E\varepsilon}{F_0} \right)^m \right) \), and the constitutive equation of compaction stage is obtained. Let the Weibull distribution parameters in the compaction stage be \( m_1 \) and \( F_{01} \) and the Weibull distribution parameters in other stages be \( m_2 \).
and $F_{02}$. The constitutive equation for the whole process of rock deformation and failure is expressed piecewise as follows:

$$
\begin{align*}
\sigma &= E\left(1 - e^{-\left(E/\sigma_{\text{com}}\right)^{m_1}}\right), & \varepsilon \leq \varepsilon_{\text{com}}, \\
\sigma &= E(\varepsilon - \varepsilon_{\text{com}})e^{-\left(E(\varepsilon-\varepsilon_{\text{com}})/F_{02}\right)^{m_2}} + \sigma_{\text{com}}, & \varepsilon > \varepsilon_{\text{com}}.
\end{align*}
$$

(23)

From the previous analysis, it is clear that $E$, $\sigma_{\text{com}}$, and $\varepsilon_{\text{com}}$ are all functions of the moisture content $\omega$ where $E$ is an exponential function of $\omega$, expressed as $y = a \times b^\omega + c$. $\sigma_{\text{com}}$ and $\varepsilon_{\text{com}}$ are primary functions of $\omega$, expressed as $y = a' x + b'$. $a$, $b$, $c$, $a'$, and $b'$ are all parameters related to the rock itself. Equation (23) becomes

$$
\begin{align*}
\sigma &= E(\omega)\left(1 - e^{-\left(E(\omega)/F_{01}\right)^{m_1}}\right), & \varepsilon \leq \varepsilon_{\text{com}}(\omega), \\
\sigma &= E(\omega)(\varepsilon - \varepsilon_{\text{com}}(\omega))e^{-\left(E(\varepsilon-\varepsilon_{\text{com}}(\omega))/F_{02}\right)^{m_2}} + \sigma_{\text{com}}(\omega), & \varepsilon > \varepsilon_{\text{com}}(\omega).
\end{align*}
$$

(24)

The above formula is the constitutive equation of the whole process of rock deformation and failure considering the influence of moisture content under uniaxial compression.

3.6.2. Determination of Model Parameters. The model parameters were determined by linear regression. When $\varepsilon < \varepsilon_{\text{com}}(\omega)$, equation (24) is deformed as follows.

$$
1 - \frac{\sigma}{E(\omega)\varepsilon} = e^{-\left(E(\omega)/F_{01}\right)^{m_1}}.
$$

(25)
Taking the logarithm twice for both sides of equation (25) yields
\[
\ln \left( \ln \left( 1 - \frac{\sigma}{E(\omega)\varepsilon} \right) \right) = m_1 \ln (E(\omega)\varepsilon) - m_1 \ln (F_{01}).
\] (26)

Let
\[
\begin{align*}
X_1 &= \ln (E(\omega)\varepsilon), \\
Y_1 &= \ln \left( \ln \left( 1 - \frac{\sigma}{E(\omega)\varepsilon} \right) \right), \\
B_1 &= -m_1 \ln (F_{01}).
\end{align*}
\] (27)

Then,
\[
Y_1 = m_1 X_1 + B_1.
\] (28)

By linear fitting the data in the compaction stage of the stress-strain curve and combining with equation (27), the parameters \( m_1 \) and \( F_{01} \) can be found. When \( \varepsilon \geq \varepsilon_{\text{com}}(\omega) \), the deformation equation (24) as follows:
\[
\frac{\sigma - \sigma_{\text{com}}(\omega)}{E(\omega)(\varepsilon - \varepsilon_{\text{com}}(\omega))} = e^{-(E(\omega)(\varepsilon - \varepsilon_{\text{com}}(\omega))/F_{01})^{m_2}}.
\] (29)

Taking the logarithm twice for both sides of equation (29), we obtain
\[
\ln \left( \ln \left( \frac{\sigma - \sigma_{\text{com}}(\omega)}{E(\omega)(\varepsilon - \varepsilon_{\text{com}}(\omega))} \right) \right) = m_2 \ln (E(\omega)(\varepsilon - \varepsilon_{\text{com}}(\omega))) - m_2 \ln (F_{01}).
\] (30)

Let
\[
\begin{align*}
X_2 &= \ln (E(\omega)(\varepsilon - \varepsilon_{\text{com}}(\omega))), \\
Y_2 &= \ln \left( \ln \left( \frac{\sigma - \sigma_{\text{com}}(\omega)}{E(\omega)(\varepsilon - \varepsilon_{\text{com}}(\omega))} \right) \right), \\
B_2 &= -m_2 \ln (F_{02}).
\end{align*}
\] (31)

Then,
\[
Y_2 = m_2 X_2 + B_2.
\] (32)

The parameters \( m_2 \) and \( F_{02} \) can be obtained by linear fitting the data in the compaction stage of the stress-strain curve and combining with equation (31). Taking the test data of W0-1, W1-1, W2-1, and W3-1 as examples, the piecewise linear fitting solution is carried out according to the above process, and the model parameters under uniaxial compression are obtained, as given in Table 3.

3.6.3. Model Verification. The theoretical stress-strain curve for this rock can be obtained by substituting the model parameters from Table 3 into equation (24). Figure 14 shows the comparison of the experimental curves and theoretical curves (including the statistical damage constitutive model and the model of this paper).

As can be seen from Figure 14, the model in this paper is in good agreement with the test curve before the peak stress, indicating that the analytical idea of considering the influence of moisture content on the stress-strain relationship of the rock void compaction stage is reasonable and feasible and can better reflect the deterioration damage effect of water-rock interaction on the rock. The shortcoming of the model in this paper is that it cannot accurately express the intrinsic structure relationship after the peak of the rock sample at present, and further research is needed.

4. Conclusion

ANDWID examined the water absorption properties of the rock samples, and rock samples with 4 kinds of different moisture were prepared. Uniaxial compression tests were performed on rock samples with different moisture contents, and the AE signals during the tests were monitored. The test results and conclusions are as follows.

1. The stress-strain curves were divided into 4 stages: void compression, elastic, plastic, and postpeak. As moisture content increases, the peak stress, the stress threshold of void compaction stage, and the stress threshold of elastic stage of the rock sample decreased linearly, elastic modulus decreased exponentially as a function, and the peak strain, the strain threshold of void compaction stage, and the strain threshold of elastic stage increased linearly.

2. Combining AE count and cumulative AE count curves with stress curves can better analyze the internal damage activity, fracture development, and structural damage of rock samples at each loading stage. With rising moisture content, the strength of the AE signal decreases. Tensile fractures dominate in the early stages of loading, while shear fractures progressively increase with the increase of loading. As moisture content increases, the damage form of rock samples gradually changed from mainly tension damage to mainly shear damage.

3. At the peak stress, the total input energy and elastic strain energy of water-bearing rock samples are smaller than those of dry rock samples. Compared with dry rock samples, the total input energy and elastic strain energy of saturated rock samples at the peak stress are reduced by 45.4% and 40.9%, respectively. As moisture content increases, the fractal dimension of fractured masses in rock samples decreases linearly; that is, the number of large fractured masses increases and the degree of fracture decreases.

4. A constitutive model of the whole process of rock deformation and damage under uniaxial compression considering the effect of moisture content was...
constructed based on the segmentation function, which can better reflect the stage of rock void compaction, but cannot accurately describe the stress-strain relationship after the peak of the rock sample.

Data Availability

No data were used to support this study.

Conflicts of Interest

The authors declare that they have no conflict of interest to this work.

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