Research Article

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Damage constitutive model of jointed rock mass considering structural features and load effect

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Abstract: Rock masses in underground engineering are usually damaged, which are caused by rock genesis and environmental stress. Studying the constitutive relationship between rock strength and deformation under loading is crucial for the design and evaluation of such scenarios. The new damage constitutive model considering the dynamic change of joint damage was developed to describe the behavior of rocks under loading in this work. First, considering the influence of jointed rock mass structural features in their entirety, the Drucker–Prager criterion and the Hoek–Brown criterion were combined. Second, based on the idea of macro–micro coupling, the calculation formulae of damage variables were derived. Finally, the damage constitutive model of the jointed rock mass was established, and the proposed model was fitted and compared with the test data. Results show that the variation rules for damage value and peak strength are opposite, and the stress–strain is highly sensitive to changes in the parameter *s* of the model. Moreover, the proposed model can accurately describe the effect of joint deterioration on the entire process of rock mass compression failure, which shows that the damage constitutive models are useful for evaluating the strength characteristics of jointed rock mass in engineering practice.

Keywords: jointed rock mass, structural features, load effect, coupling effect, damage constitutive model

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1 Introduction

Long-term geological evolution has led to the widespread development of random joint surfaces in rocks. The existence of various scale defects in jointed rock mass from micro to macro makes the mechanical behavior and failure characteristics of rock materials extremely complicated, which has great influence on the safety and long-term stability of rock engineering. Frequently, water conservancy, slopes, and mining engineering are affected by environmental stress – for instance, after the foundation of a hydropower dam is excavated, the stress relaxation of the rock mass of the dam foundation, or even the failure of the dam owing to seepage. Natural and constructed slopes are subject to landslides and cave-ins due to the influences of gravity and vibration. The surrounding rock mass fractures and deforms as a result of the redistribution of stress caused by the excavation of the mining route. Since the constitutive relationship of rock masses is the basis for rational design and assessing the stability of engineering constructions. Consequently, using the damage constitutive relationship to describe the deformation and failure characteristics of the jointed rock mass under stress is one of the hottest research topics. Taking into account the macroscopic with a mesoscopic damage coupling mechanism, the staged deformation characteristics of the jointed rock mass are essential to solve this issue.

The emergence and development of damage mechanics gives a new research concept for examining the mechanical properties and failure mechanism of rocks [1]. Kyoya *et al.* [2] applied damage theory for the first time to the analysis of initial joint damage of rock mass and established its constitutive relationship. Rocks with different initial damage states can be considered as materials of different properties; based on damage continuum and statistical theory, the constitutive model affected by the loading capacity of the damaged elements is derived [3]. Chen *et al.* [4,5] created the concepts of damage rate and damage index and discovered that the Weibull distribution can more accurately reflect the transition of rock from brittleness to ductility than power function. Therefore, the micro-unit strength is generally assumed to follow the Weibull distribution [6]. In order to consider joint geometry, strength, and deformation parameters simultaneously, the macro- and mesoscopic damage coupling viewpoint is put forward, and the damage constitutive model of intermittently jointed rock mass is developed [7,8]. In the analysis of the damage evolution law, it is shown that joint inclination has a major effect on initial damage [9–11]. Considering the interaction between joints and surrounding rock, a damage-plasticity cohesive-frictional model is established, which enables the model to capture the key characteristics of jointed rock mass responses at different spatial scales [12]. With the advancement of strength theory, the micro-element strength of rock mass is primarily described by the strength criterion connected to the factor of the rock mass strength – the Hoek–Brown (H-B) criteria, which can consider the various factors of the rock quality and the rock strength associated with the surrounding rock [13,14]; the Griffith criterion, which can characterize the fracture failure of brittle materials from an energy perspective [15]; and the Drucker–Prager (D-P) criterion, which can consider the effect of intermediate primary stress and hydrostatic pressure [16,17]. However, the D-P criterion normally produces larger damage zone in numerical simulations as it is a distribution parameter of mesoscopic element strength, so the Mohr–Coulomb (M-C) criterion is used as the distribution parameter of damage-softening statistical constitutive model [18]. Due to the presence of a fissure compaction mechanism in the initial process of rock failure, Wen *et al.* [19] derived a calculation formula considering the crack closure effect and proposed a statistical damage model of structural surfaces based on the M-C criterion to define the micropore strength of rock structural surfaces [20]. With the deep development of engineering environment, its stress environment is gradually complicated. Since M-C criterion is only suitable for predicting the stress characteristic values of rock at low confining pressure level, after the basic concept of critical confining pressure was introduced, and a nonlinear strength criterion of rock that can consider the transformation of brittle–ductile characteristics was proposed [21]. Based on the relationship between the rock deformation and the deformation of the initial void and the skeleton, a rock deformation analysis model was established [22] and the idea of void strain ratio was introduced [23]. The internal cracks of a rock grow constantly under load, and its strength also fluctuates continuously [24]; in this context, the constitutive model of fissured rock mass was constructed based on the deformation characteristics of compressive micro-cracks under compression [25]. Considering that crack closure and development are accompanied by energy dissipation, Liu *et al.* [26] and Ma *et al.* [27] proposed the damage variable based on energy dissipation, respectively. The lateral strain of rock exhibits nonlinear properties throughout the damage and failure processes. Under the circumstance that rock exhibits dilatancy in the compression process, a damage evolution equation considering dilatancy characteristics and post-peak shape was derived [28]. The residual strength after the peak of stress–strain curve plays an important role in the stability of geotechnical system. With the increase of the confining pressure, the residual strength gradually becomes the main factor affecting the posterior section of the rock full stress–strain curve [29]. A theoretical approach to simulate the brittle rock stress–strain curve was proposed by considering the residual strength response of rock materials [30].

Strength criteria are frequently used to determine the micro-element strength of rock masses in previous studies, while the definition of macroscopic damage variables is mostly based on changes in elastic modulus. Some existing constitutive models simulate the stress and deformation of rock mass in stages for a better estimate, but their complexity and number of parameters limit their practicality. Although these studies have made significant advances in the theory of joint fracture damage, they have not considered the effect of joint deterioration on the rock mass failure process under pressure. Moreover, the presence of joint surfaces in actual rock mass engineering will cause varying degrees of deterioration of the rock mass under random stress and will become the primary inducing factor of rock mass failure under certain conditions. To address shortcomings in existing research and characterize jointed damage accurately during loading, the H-B criterion and the D-P criterion are combined to account for the structural features of the rock mass. On the micro-level, the rock mass strength obtained by Griffith criterion is used to establish the micro-damage variable through the idea of element subdivision. Then, a damage constitutive model of jointed rock mass under the coupling action of structural features and load effect is proposed, and the rationality of the model is verified.

2 Damage constitutive model of jointed rock mass considering load and structural features

The natural rock mass contains various scale defects from macro to micro. Various damage defects of different scales will have different effects on the physical and mechanical properties of rock mass. Due to the large number and incomplete penetration of rock mass structural plane, it cannot be considered individually. For engineering rock mass, the distributed joint cracks are regarded as macroscopic defects. If the initial damage state of intact rock is taken as the benchmark damage state, the macroscopic defects can be regarded as macroscopic damage.

2.1 Macroscopic damage variable

The accumulation of micro-cracks and pores in rock mass under loading will affect the mechanical properties of rock and eventually lead to its failure. Failure criteria of rock mass elements can be stated as a function of their effective stress and material parameters [31]:

$$f(\sigma^*) - k_0 = 0, \tag{1}$$

where k_0 is the material parameter and σ^* is the effective stress.

The presence of a joint surface modifies the macroscopic structural features of a rock mass structure and causes varying degrees of damage to the rock mass under different stress levels. According to the failure criterion adopted in the theory of damage and the introduction of a micro-mechanical expression to characterize the deterioration process of the action effect of rock mass structure [32], the following equation can be obtained as:

$$f(\sigma^*) = \frac{f(\sigma)}{(1 - D_1)}, \tag{2}$$

where D_1 is the macroscopic damage and σ is the nominal stress.

Due to the fact that the D-P criterion may simultaneously reflect the influence of volume stress, shear stress, and intermediate principal stress on rock strength, it can more precisely describe the actual condition. Consequently, $f(\sigma)$ can be represented in accordance with the D-P criterion as follows:

$$f(\sigma) = \alpha_0 I_1 + \sqrt{J_2}, \tag{3}$$

where α_0 is the material constant, φ is the internal friction angle, I_1 is the first invariant of the stress tensor, as shown in Eq. (4), and J_2 is the second invariant of the stress deviatoric, as shown in Eq. (5):

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3 = \sigma_x + \sigma_y + \sigma_z, \tag{4}$$

$$J_2 = [(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2]/6. \tag{5}$$

Moreover, the H-B criterion considers the surrounding rock strength, and it is more applicable to rock materials. The H-B criterion of effective stress-invariant modification is therefore introduced, as shown in the following equation:

$$\begin{aligned} f(\sigma^*) = \frac{m\sigma_c I_1^*}{3} + 4J_2^* \cos^2\theta_\sigma + m\sigma_c\sqrt{J_2^*}(\cos\theta_\sigma \\ + \sin\theta_\sigma/3) = \sigma_c^2 s, \end{aligned} \tag{6}$$

where the H-B criterion parameter m reflects the hardness of the rock and its value ranges from 1×10^{-7} to 25, take a small value when it is badly disturbed and a large value when it is entirely hard; parameter s reflects the degree of rock fragmentation and its value range is 0–1, the value of broken rock mass is small and the value of intact rock mass is large; σ_c is the uniaxial compressive strength of the intact rock; θ_σ is the Lode angle; $\theta_\sigma = 30°$; I_1^* is the first stress invariant that is effective, such as Eq. (7); and J_2^* is the second effective deviatoric stress invariant, such as Eq. (8):

$$I_1^* = E\varepsilon_1(\sigma_1 + \sigma_2 + \sigma_3)/[\sigma_1 - \nu(\sigma_2 + \sigma_3)], \tag{7}$$

$$\begin{aligned} J_2^* = E^2\varepsilon_1{}^2(\sigma_1^2 + \sigma_2^2 + \sigma_3^2 - \sigma_1\sigma_2 - \sigma_2\sigma_3 - \sigma_1\sigma_3) \\ /\{3[\sigma_1 - \nu(\sigma_2 + \sigma_3)]^2\}, \end{aligned} \tag{8}$$

where E is the elastic modulus, ε_1 is the axial strain, and ν is the Poisson ratio.

The definition of damage variable based on elastic modulus variation can adequately explain the initial damage of jointed rock mass [33], but the degradation of jointed rock mass during the compression failure process is not taken into account. In consideration of the applicability of the H-B criterion and the D-P criterion in anisotropic rock mass materials, this work combines them to account for the macroscopic damage of rock mass induced by joints during the loading failure process. Substituting Eqs. (3) and (6) into Eq. (2) yields the following expression for the macroscopic damage produced by joints to a rock mass:

$$\begin{aligned} D_1 = 1 - (\alpha_0 I_1 + \sqrt{J_2})/[m\sigma_c I_1^*/3 + 4J_2^*\cos^2\theta_\sigma \\ + m\sigma_c\sqrt{J_2^*}(\cos\theta_\sigma + \sin\theta_\sigma/3)]. \end{aligned} \tag{9}$$

2.2 Microscopic damage variable

All kinds of micro-defects are randomly distributed inside the rock, so the internal defects can be regarded as random damage and studied from the idea of statistical damage mechanics. It quantifies the degree of damage inside the rock by the strength of the micro-element and according to the characteristics of random distribution of damage inside the rock. It is assumed that the internal defects of rocks obey certain distribution, and then, the corresponding statistical damage constitutive model of rocks is established [14–16]. The modified Griffith criterion is used in building rock mass element strength. Cracks in the rock will close during compression [34]; stress is transmitted to crack surfaces and causes friction between them. Brace [35] believed that the compressive stress necessary for the crack to close was negligible. The resulting equation for the simplified modified Griffith criterion is obtained:

$$\sigma_1((f^2+1)^{0.5}-f)-\sigma_3((f^2+1)^{0.5}-f)=4\sigma_t, \quad (10)$$

where σ_t is the ultimate tensile strength of materials and f is the coefficient of internal friction.

Assuming that the micro-element strength and other relationships of rock under load correspond to the Weibull distribution function, the micro-element probability density function can be obtained [36,37]:

$$\psi(F)=\{n(F/F_0)^{n-1}\exp[-(F/F_0)^n]\}/F_0, \quad (11)$$

where n and F_0 are the shape parameter and the size parameter of the Weibull distribution function, respectively.

Failure of jointed rock mass is the macroscopic manifestation of the partial failure of its internal micro-elements; the relationship between the damage variable D_2 under load and the failure probability of the micro-element is described by the following equation:

$$\frac{dD_2}{dF}=\psi(F). \quad (12)$$

According to Eqs. (11) and (12), the damage variable D_2 of rock mass under load can be obtained as:

$$D_2=\int_0^F\psi(F)dF=1-\exp[-(F/F_0)^n]. \quad (13)$$

By substituting Eq. (10) into Eq. (13) to replace the strength of rock micro-element approximatively, the damage variable of rock micro-element can be obtained as:

$$D_2=1-\exp\{-[(((f^2+1)^{0.5}-f)-\sigma_3((f^2+1)^{0.5}-f))/(4F_0)]^n\}. \quad (14)$$

2.3 Establishment of the damage constitutive model

According to the theory of damage mechanics, the definition of damage variables is the premise and basis of damage model establishment, and the coupling of damage defects at different scales is concentrated as the coupling of damage variables [8]. The total damage of jointed rock mass under load can be equivalent to the coupling of structural features and load effect. One is based on the definition of the structural features of the loaded rock mass, which characterizes as macroscopic damage the changes in the structural features of the rock mass induced by cracks and joints during the loading process. The other is loading damage, which is the micro-damage induced by the micro-element strength reduction during the loading process. According to Lemaitre's strain equivalence hypothesis, the damage constitutive relationship of jointed rock mass can be obtained as:

$$\sigma=E_0(1-D)\varepsilon+\nu(\sigma_2+\sigma_3). \quad (15)$$

Liu *et al.* [38] believed that the following equation represents the total damage variable of jointed rock mass under load:

$$D=D_1+D_2-D_1D_2. \quad (16)$$

By substituting Eq. (16) into Eq. (15), the constitutive relationship of the jointed rock mass expressed by the joint damage variable and the load damage variable can be obtained as:

$$\sigma=E_0(1-D_1)(1-D_2)\varepsilon+\nu(\sigma_2+\sigma_3). \quad (17)$$

The damage variable is a measure of the damage degree of the rock mass under force, and the damage degree is related to the defects within the rock mass, which have a direct impact on the mechanical characteristics of engineering rock mass. By substituting Eqs. (9) and (14) into Eq. (16), the total damage evolution equation of jointed rock mass under loading can be obtained as:

$$\begin{aligned}D=&1-[f(\sigma)/f(\sigma^*)]\exp\\&\{-[(((f^2+1)^{0.5}-f)-\sigma_3((f^2+1)^{0.5}-f))/(4F_0)]^n\}.\end{aligned} \quad (18)$$

As mentioned earlier, the fractured rock mass has both macro- and mesoscopic defects. The mesoscopic defects of the rock mass deteriorate under the action of load, and the micropores (cracks) inside the rock mass may expand, resulting in mesoscopic damage and new cracks. The combination of these cracks may lead to the formation of macroscopic cracks. The existence of macroscopic defects such as cracks will also weaken the strength and stiffness of rock mass [39]. By substituting Eq. (18) into Eq. (15), the final damage constitutive model of jointed rock mass can be obtained as:

$$\begin{aligned}\sigma=&E_0\varepsilon[f(\sigma)/f(\sigma^*)]\exp\\&\{-[(((f^2+1)^{0.5}-f)-\sigma_3((f^2+1)^{0.5}-f))/(4F_0)]^n\}\\&+\nu(\sigma_2+\sigma_3).\end{aligned} \quad (19)$$

When the rock mass is subjected to triaxial stress, the three principal stresses $\sigma_1\neq\sigma_2\neq\sigma_3\neq0$, and the constitutive model was established as follows:

$$\sigma = E_0\varepsilon \frac{\alpha_0 I_1 + \sqrt{J_2}}{m\sigma_c \frac{I_1^*}{3} + 4J_2^* \cos^2\theta_\sigma + m\sigma_c\sqrt{J_2^*}\left(\cos\theta_\sigma + \frac{\sin\theta_\sigma}{3}\right)} \times \exp\left\{-\left[\frac{\sigma_1((f^2+1)^{0.5} - f) - \sigma_3((f^2+1)^{0.5} - f)}{4F_0}\right]^n\right\} + \nu(\sigma_2 + \sigma_3). \quad (20)$$

When $\sigma_1 \neq 0$ and $\sigma_2 = \sigma_3 = 0$, the rock mass is under uniaxial stress, and the constitutive model can be obtained as:

$$\sigma = E_0\varepsilon[(\alpha_0 + 1/\sqrt{3})\sigma_1 /(\sigma_c{}^2 s)] \exp\{-[(\sigma_1((f^2+1)^{0.5} - f))/(4F_0)]^n\}. \quad (21)$$

3 Verification and analysis of the model

3.1 Experiment

The experiment used yellow sandstone from Hunan, China. According to the method recommended by the international society of rock mechanics, the cylinder rock samples with the diameter of 50 mm and the height of 100 mm were generated [40]. In order to limit the impact of rock dispersion, rock samples with equal wave velocities and good homogeneity were chosen using ultrasonic testing. The common dip angle joints with a length of 20 mm were sawed with an emery wire, and the specimens were sorted into six groups based on the total rock mass and the dip angle α of the joints, as shown in Figure 1.

The RMT-150B rock mechanics testing system was applied, which was designed by the Chinese Academy of Sciences' Institute of Rock and Soil Mechanics, and the jointed rock mass specimens were subjected to uniaxial compression failure testing, as shown in Figure 2. The main parameters of the testing machine are as follows: the vertical cylinder piston stroke is 0–50 mm, the force sensor has a range of 1,000 kN

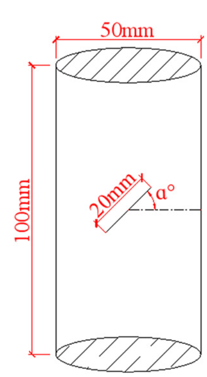

Figure 1: Specimen parameters (α = 0°, 30°, 45°, 60°, 90°).

with an accuracy of 5‰, the axial and lateral displacement sensors have respective ranges of 5 and 2.5 mm, with an accuracy of 1.5‰. Before the uniaxial compression experiment, butter was applied to the end face of the sample to reduce the influence of friction on the contact surface between the apparatus and the sample. During the test, the loading rate was applied at 0.5 MPa·s^{-1}. The mechanical properties of the rock mass with different inclinations are shown in Table 1.

3.2 Damage characteristic analysis

3.2.1 Model validation

The damage constitutive model of jointed rock mass under uniaxial stress (Eq. (21)) was validated by the uniaxial compression test. Herein, the internal friction angle φ was 30°, $\alpha_0 = \sin\varphi/\sqrt{9 + 3\sin^2\varphi} = 1/\sqrt{30}$, $f = \tan\varphi = 1/\sqrt{3}$, the peak strength of intact rock mass according to experimental data σ_c = 70.77 MPa, E_0 = 9.52 GPa, and Eq. (21) can be reduced to the following equation:

$$\sigma = [0.76E_0E_\beta\varepsilon^2/(70.77^2 s)]\exp[-(0.144E_0\varepsilon/F_0)^n], \quad (22)$$

where E_β is the elastic modulus of the rock mass after joint degeneration, and its value is presented in Table 1.

Curve fitting is a typical method for determining the parameters of a constitutive model, which can make full use of experimental data to achieve the smallest possible total discrepancy between theoretical and experimental results [41]. Based on mechanical parameters of a rock mass with various inclination joints, Eq. (22) is used to perform nonlinear least-squares fitting, as shown in Figure 3, and the values of the model parameters are shown in Table 2.

As can be seen from Figure 3, the stress–strain curves can be separated into five stages. During the early loading phase, the joint surface and initial micro-cracks gradually close, the friction between the closed surfaces restricts the development and beginning of micro-cracks, and the strain develops rapidly, which is the compaction stage (I). In the concave portion of the curve, it enters the nonlinear elastic stage (II). The slope of the curve increases gradually and tends to be linear, and the rock mass enters the linear elastic stage (III) from the nonlinear elastic stage as its deformation velocity slows and its stress–strain increases in equal steps. When stress redistribution gradually builds an effective load-bearing structure, the specimen structure continues to absorb external energy and store energy for later damage and failure, as seen by an upward concave curve, and the rock mass enters the yield stage (IV). After reaching the ultimate bearing capacity, the bearing structure of rock mass is continuously softened and enters the brittle failure stage (V).

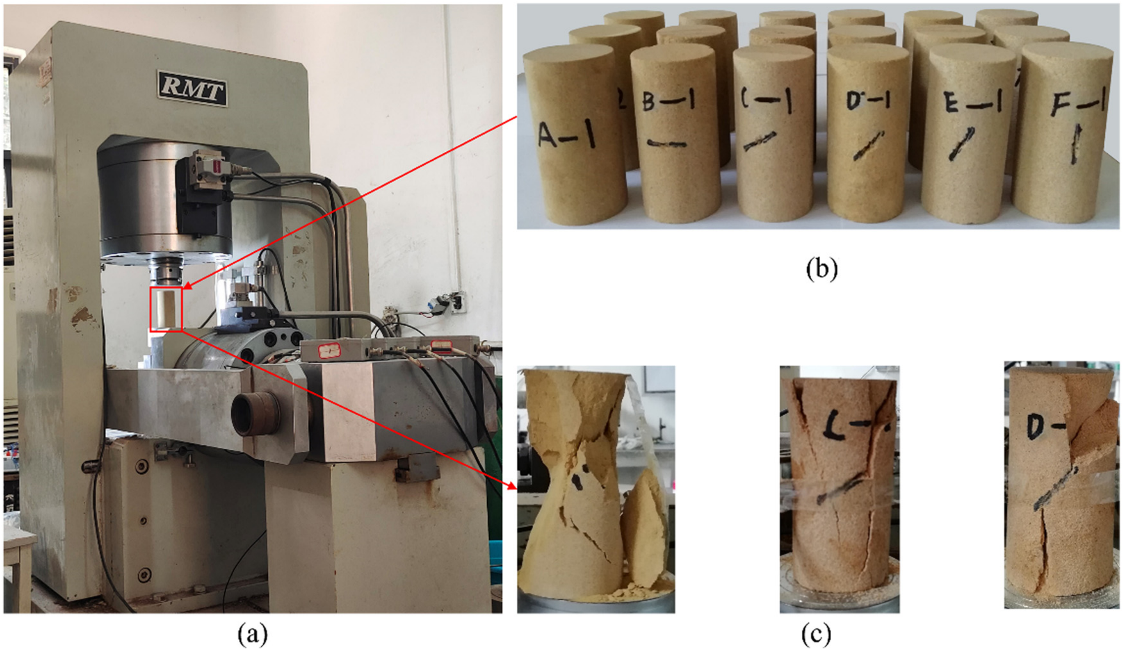

Figure 2: Uniaxial compression testing: (a) RMT-150B, (b) before compression, and (c) after compression.

Table 1: Mechanical properties of jointed rock mass with different inclinations

Joint inclination (°)	Elastic modulus (GPa)	Peak stress (MPa)	Peak strain (10^{-3})
Intact rock mass	9.52	70.77	11.21
β = 0°	7.48	53.18	8.49
β = 30°	7.49	46.17	9.18
β = 45°	7.67	44.19	8.23
β = 60°	7.20	37.80	8.98
β = 90°	7.78	59.38	9.89

All correlation coefficients between stress–strain curves and fitting curves of jointed rock mass with varying inclinations are higher than 0.98. The result proves the constitutive model developed in this study can describe the compression, elastic, yield, and brittle failure stages of the stress–strain curve of a jointed rock mass during loading and indicates that the mechanical behavior, strength, and deformation law of rock mass under varying jointed inclinations could accurately predict.

3.2.2 Law of damage evolution

The jointed structural surface substantially impacts the deformation properties and failure law of rock mass. In order to further verify the rationality of the established model, the damage at the peak value is calculated by Eq. (18) according to the parameter values in Table 2, and its damage evolution characteristics are analyzed and verified through normalization. The peak strength and damage changes induced by jointed inclination are shown in Figure 4.

As can be seen in Figure 4, under the coupling effect of structural features and load effect, the experimental peak value of jointed rock mass is the least at 60° and reaches the maximum at 90°, showing a "U"-shaped distribution rule in general. When the joint inclination angle increases from 0° to 30°, the absolute deviation marginally increases from 1.05 to 1.78 MPa, and the maximum deviation rate is 3.86%. The variation can be decreased through parameter adjustment, and it is vital to assess the parameter sensitivity in order to improve the precision of theoretical results. As the angle of inclination increases, the absolute deviation decreases to less than 1.0 MPa, the peak stress calculated theoretically corresponds to the experimental peak stress level, and the degree of coincidence is high.

The damage value at the peak of a mass of jointed rock with varying inclination is plainly distinct, and the variation rules for the damage value and peak strength value are diametrically opposed. The damage value is lowest when the joint inclination angle is 0° and increases to 0.88 from 0.61 as the joint inclination increases to 30°. While the joint inclination increases to 45°, the damage value increases to 0.79, which indicates that the compressive resistance of the rock mass structure has been

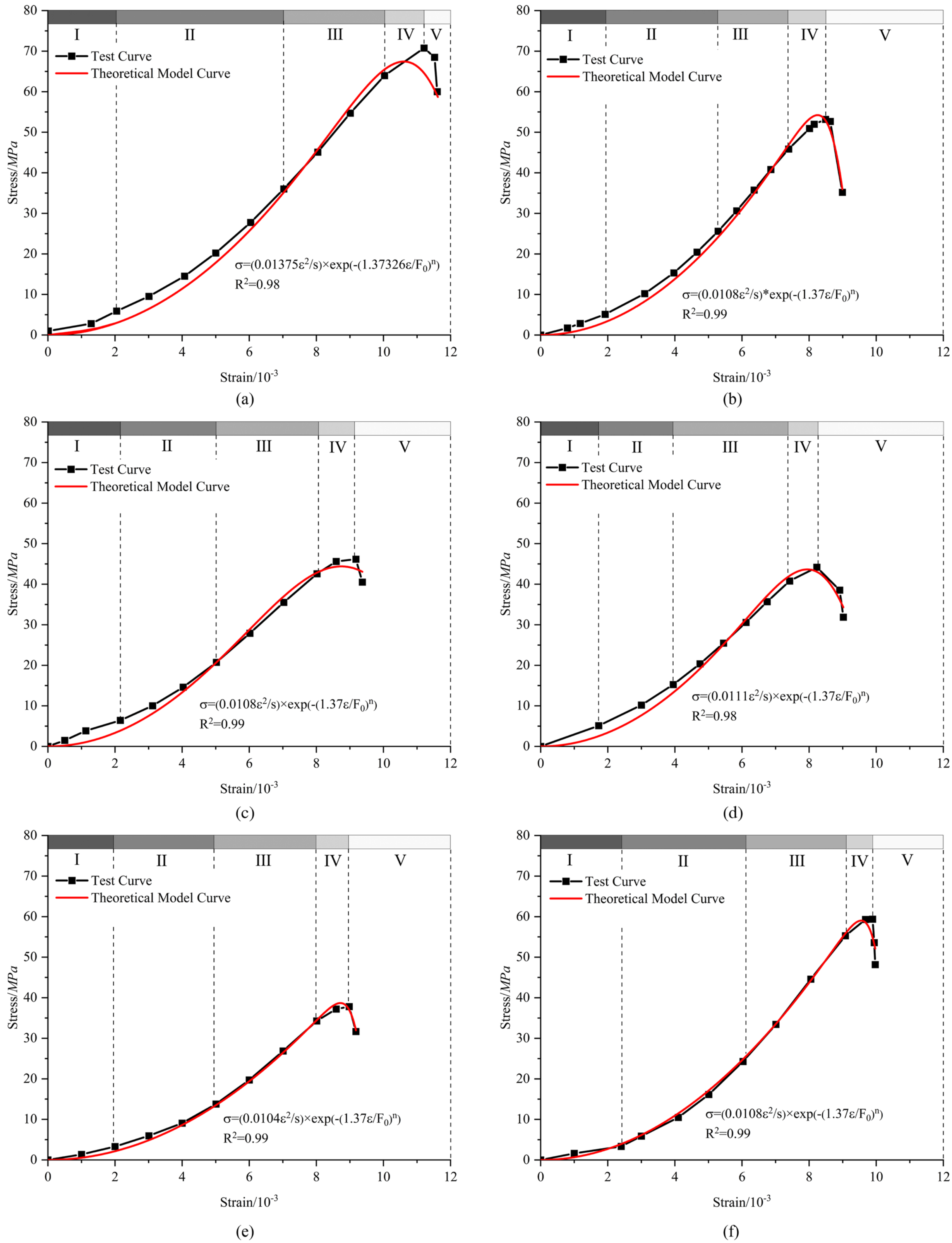

Figure 3: Comparison of experimental and theoretical values of stress–strain curves: (a) intact rock mass, (b) $\beta = 0°$, (c) $\beta = 30°$, (d) $\beta = 45°$, (e) $\beta = 60°$, and (f) $\beta = 90°$.

Table 2: Model parameters s, F_0, and n

Joint inclination (°)	s	n	F_0
Intact rock mass	0.0193	11.480	16.965
β = 0°	0.0125	24.499	12.554
β = 30°	0.0129	5.487	14.432
β = 45°	0.0130	9.580	12.850
β = 60°	0.0193	35.391	12.985
β = 90°	0.0165	36.896	14.194

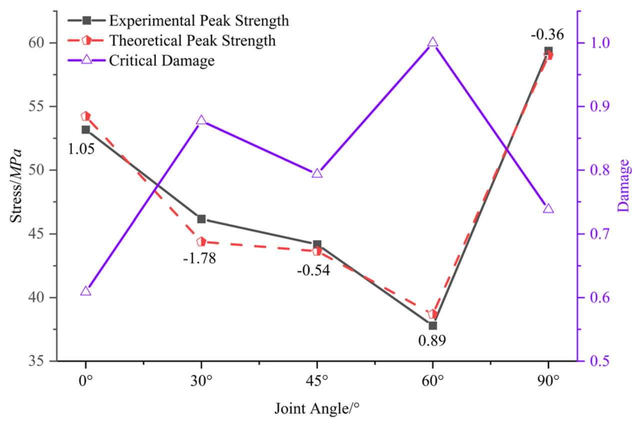

Figure 4: Relationship of peak stress and critical damage with jointed inclination.

enhanced. However, when the joint inclination is 60°, the damage value is maximum, and the peak strength of the jointed rock mass is lowest. The rock mass experiences shear slip failure along the joint surface and has a low failure resistance, which indicates that the shear force on the joint surface is more than the sum of friction force and cohesion force. In theory, this is consistent with the rock mass failure angle ($\beta = \pi/4 + \Phi_W/2$, wherein Φ_W is the friction angle in the structural surface) calculated by the single structural plane theory. It demonstrates that the damage constitutive model built for the jointed rock mass and the damage evolution equation based on the coupling of structural features and load effect are reasonable. When the joint inclination is 90°, the specimen does not shear slide along the joint surface, the damage value is reduced, and the peak strength is increased significantly.

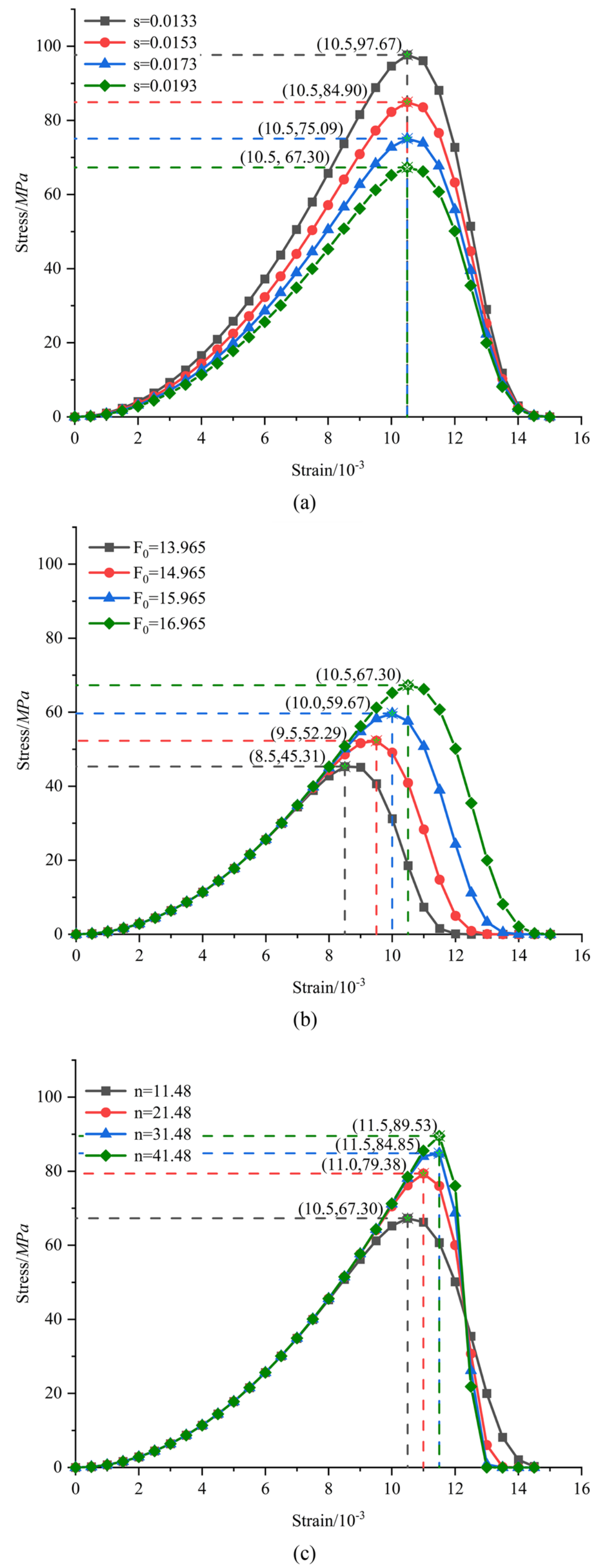

Figure 5: Stress–strain curve under different values of the parameters s, F_0, and n. (a) $s(F_0 = 16.965, n = 11.48)$, (b) $F_0(s = 0.0193, n = 11.48)$, and (c) $n(s = 0.0193, F_0 = 16.965)$.

4 Discussion

The parameters s, F_0, and n of the damage constitutive model impact the geometric scale and shape of the fitting curve, which are of major significance in the applicability of the model. Therefore, a sensitivity analysis is undertaken based on the shape of the stress–strain relationship obtained using the control variable approach. Taking into account the representativeness of the values, the complete rock specimen group is used as the criterion for analysis, and the parameters are chosen by gradient based on the results (Table 2) of the fitting. In Eq. (22), let σ_c = 70.77 MPa, E_0 = 9.52 GPa, and the stress–strain curves based on different parameter values, as shown in Figure 5.

The macro-damage parameter s of the model governs the overall shape of the stress–strain curve and exerts varied degrees of influence on the shape of the stress–strain curve at each stage, as shown in Figure 5(a). The parameter s increases with the gradient from 0.0133 to 0.0193, while peak stress declines by 13.07, 11.55, and 10.37%, respectively. The magnitude of the peak stress variation decreases, although the peak strain remains constant, and the rate of reduction during the post-peak phase is slowed. The microscopic damage parameter F_0 determines the peak stress and strain of a jointed rock mass, as shown in Figure 5(b). The parameter F_0 increases with the gradient from 13.965 to 16.965, the compressive strength increases, and the variation range of peak stress is 15.4, 14.1, and 12.79%, respectively. However, the variation range of peak strain is 11.76, 5.50, and 5%, respectively. The increase in F_0 has little effect on the variation amplitude of peak stress, but has a great effect on the variation amplitude of peak strain. It is shown that the curves in the pre-peak phase overlap and the slope remains unchanged, and the curves in the post-peak phase are basically parallel and move to the right. The compressive strength of the stress–strain curve increases as the micro-damage parameter n increases, as can be seen in Figure 5(c). The parameter n increases with the gradient from 11.48 to 41.48, and the peak stress changes are 17.95, 6.89, and 5.52%, respectively. However, peak strain changes are 12.29, 4.55, and 0%, respectively. As the influence of an increase in parameter n on the peak point is substantially reduced and tends to be stable, the post-peak stage curve shows a tendency toward a faster rate of drop.

Based on the change features of the stress–strain curve for various parameter values, it is evident that the stress–strain curve is highly sensitive to parameter s. Both stress–strain curves in Figure 5(b) and (c) exhibit evident bifurcation points. Simplify Eq. (22) into a negative exponential function whose value is close to 1 when ε is close to 0. Currently, the stress–strain curve is only related to the coefficient of the function, resulting in a coincident segment. A bifurcation point emerges as the value of the function without the coefficient component deviates from 1 as strain increases. Therefore, in the gradient change of parameter F_0, the curves are clearly separated only when the strain is 0.007, and the strain at the bifurcation point in Figure 5(c) is 0.009. The parameter s in Figure 5(a) is related to the coefficient of the function. Hence, it determines the overall shape of the stress–strain curve.

5 Conclusions

As basic elements in mines, the study of the relationship between strength and deformation under load is all important for the rock mass. In this study, the damage constitutive model of jointed rock mass under the influence of structural features and load effect is developed, and the main conclusions are as follows.

1) The results calculated by the model correspond well with the stress–strain experimental results obtained by uniaxial compression for jointed rock masses with differing inclinations, which accurately described the stress–strain relationship of jointed rock masses with varying inclinations, thereby validating the rationality of the model.
2) The rock mass has the lowest resistance to failure and the highest damage value when the joint inclination is 60°. In this condition, the shear force on the joint surface exceeds the sum of friction and cohesion, resulting in shear slip failure along the joint surface. Therefore, this form of jointed and fractured rock mass requires special consideration in engineering.
3) The theoretical curve of the model is highly sensitive to the macroscopic damage parameter s, and the microscopic damage parameter n controls the peak stress and peak strain of the jointed rock mass, as well as the drop velocity during the brittle failure stage. In addition, increasing the model parameter F_0 has no effect on the variation range of the peak point, and the post-peak curves are essentially parallel and have no effect on the shape of the theoretical curve.

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