Experimental study of the influence of structural planes on the mechanical properties of sandstone specimens under cyclic dynamic disturbance

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Abstract
Discontinuity-controlled slopes constitute potentially fatal risks during exploitation activities in open-pit mines and severely threaten the safe mining of collieries. To investigate the dynamic mechanical properties of rock with structural planes under periodic cyclic dynamic disturbance, this study took the north side of the Anjialing open-pit mine as an example. Uniaxial compression tests were carried out on rock samples containing structural planes with different dip angles (θ). The impacts of θ on the strength, deformation characteristics, energy dissipation characteristics, and fracture rules of rock samples under different disturbance stress amplitudes (Δσ) are studied. The test results show that the structural plane affects the mechanical properties of rocks under dynamic perturbation. A closed hysteresis loop curve of rock is formed in each loading and unloading cycle, thus forming three stages: a sparse stage, a dense stage, and another sparse stage. θ, Δσ, and the number of disturbance cycles (n) impact the rock hysteresis loop curve and deformation modulus. The plastic deformation of rock mainly includes an initial deformation stage, an isokinetic deformation stage, and an accelerated deformation and overall failure stage. Moreover, there is a disturbance threshold for Δσ′ of the rock failure, and the evolution law of plastic strain accumulation and n near the disturbance threshold conform to the negative exponential function and the inverse function of the Langevin function, respectively. Based on damage theory, a constitutive model of the structural plane considering a disturbance threshold is proposed.

KEYWORDS
constitutive model, cyclic dynamic disturbance, mechanical property, rock, structural plane, uniaxial compression test

1 | INTRODUCTION

In recent years, with the increasing demand for mineral deposit resources and continuous consumption of shallow resources, many mines are gradually entering the deep mining state so that open-pit mines on high and steep slopes can be excavated.1-4 The ultra-deep mine of TauTona in South Africa is known to extend at least 3500 m.5 The Asturian coal basin in Spain has already reached depths exceeding 4000 m.6 According to statistical analysis using the protocol...
of the International Society for Rock Mechanics, there are hundreds of deep mines with mining depths over 1000 m in production or being constructed in China (see Table 1).

Specifically, in the open-pit combined mining environment, rock slopes are not under monotonic loading conditions but may experience the combined action of open-pit mining and underground mining. In the process of deep mining, high-stress hard rock is generally subjected to blasting operations, mechanical shock earthquakes, overburden fracture, and other dynamic disturbances around the deep underground rock. The distribution and change of the stress field and other dynamic disturbances around the deep underground mechanical shock earthquakes, overburden fracture, and stress hard rock is generally subjected to blasting operations, mechanical shock earthquakes, overburden fracture, and other dynamic disturbances around the deep underground rock.

The distribution and change of the stress field and displacement field in the slopes are complicated, which degrades the stability of the slopes. In reality, the rock mass is in a combined stress state of prestatic load and dynamic disturbance, which leads to a completely different mechanical response from the pure static or dynamic loading state of shallow rocks. It is difficult to give a reasonable explanation for the above failure phenomenon using the classical rock statics or dynamics theory based on shallow mining. Most rock engineering accidents around the world are due to the lack of research on structural characteristics.

On 10 December 2006, the north side of the Anjialing open-pit mine in Shanxi Province, China, experienced a massive area landslide, with a drop of approximately 90 m and a volume of approximately 600,000 m³. The trigger factors for this landslide can be classified into two categories: internal master control factors (such as the development of joints and structural planes in the slope) and anthropogenic activities (such as disturbances to the workbench under the open pit due to underground mining and frequent blast dynamic disturbance). Therefore, special attention should be paid to the structural characteristics of prestatic rock subjected to dynamic disturbance. Under dynamic disturbance, further damage occurs in the structural planes of high steep rock slopes, resulting in a change in the slope stress state and extension and intersection of cracks in the slopes, causing the deterioration of the mechanical properties of surface planes.

Open-pit mining will cause secondary damage to the slope and will intensify the cumulative damage to the structure of high and steep slopes, with microcracks gradually reaching the critical state, resulting in the large-scale instantaneous dynamic expansion of cracks on the basis of long-term deformation and causing the instability failure of the slope.

Generally, joints, which constitute weak structural surfaces, are widely distributed throughout rock slopes and negatively affect the integrity of the rock mass. The joints can be distributed complexly due to their uncertain geometric and mechanical parameters, such as direction, dip angle, and location. Given the abovementioned analysis, the dynamic mechanical properties of rock slopes with structural planes are very complicated under dynamic disturbance.

To date, many kinds of studies have been carried out on the impact of combined dynamic and static loading or dynamic disturbance on the failure of deep surrounding rocks. Feng et al. investigated the stress state of joints in the progressive damage process using single cyclic loading tests. It was concluded that, even though the test results show that rock damage is directional and anisotropic, the dissipated energy is linearly related to the equivalent irreversible strains and plays a pivotal role in the evolution of rock damage. Lei and Wang carried out laboratory investigations on the deformation and fracture of sandstone when applied to multi-level dynamic cyclic loading with treatment loading frequencies, various amplitudes, and a series of uniaxial compressive strength tests and found that loading frequencies and amplitude, as well as the waveform, were of great significance to and helpful for understanding the sandstone process from both micro- and macroscale perspectives. Nie et performed a comparative study on the mechanical behavior and permeability of coal by using parameters from CTC and UCP-RAS tests. However, recent studies by Chen et al. and Zhou et al. indicated that peak stresses and dynamic loading could induce significant rock specimen failure patterns controlled by structural effects. Su et al. studied the damage and failure rules of rock under three kinds of dynamic disturbances (mild, modest, and weak) by using an AE-controlled testing system and established a new constitutive model of elastic-plastic damage. These studies all indicate that rock may experience a complex loading path during the excavation of underground openings, including the unloading of confining stress, highly confined compression before excavation, and further disturbance of dynamic loading after excavation. In addition, Niu et al. studied the rock failure mechanism induced by excavation according to a simulation.

### TABLE 1 Typical deep mines with depths over 1000 m in China

| No. | Mine name               | Province   | Depth (m) |
|-----|-------------------------|------------|-----------|
| 1   | Huize lead-zinc mine    | Yunnan     | 1000      |
| 2   | Changba lead-zinc mine  | Gansu      | 1050      |
| 3   | Yingxin gold mine       | Henan      | 1500      |
| 4   | Erdaogou gold mine      | Liaoning   | 1020      |
| 5   | Xiangxi gold mine       | Hunan      | 1060      |
| 6   | Hongtoushan copper mine | Liaoning   | 1300      |
| 7   | Dongguashan copper mine | Anhui      | 1120      |
| 8   | Lingbao gold mine       | Henan      | 1200      |
| 9   | Linglong gold mine      | Shandong   | 1000      |
| 10  | Hongxin gold mine       | Henan      | 1000      |
| 11  | Fushun open-pit mine    | Liaoning   | 2500      |
| 12  | Yiminie open-pit mine   | Inner Mongolia | 2000   |
| 13  | Shengli East open-pit mine | Inner Mongolia | 1000 |
| 14  | Baiyinhua open-pit mine | Inner Mongolia | 1500 |
of the failure rock sequentially subjected to this complex loading path, such as the unloading of confining stress, highly confined compression before excavation, and further disturbance of dynamic loading after excavation, by using RFPA-Dynamics. Li et al.\textsuperscript{60-62} and Xu et al.\textsuperscript{63} investigated the dynamic compressive and tensile behaviors of rocks under different coupled static and dynamic loads by using the custom-fabricated SHPB device. Yin et al.\textsuperscript{64} analyzed the failure mechanism of highly stressed rocks under unloading and dynamic disturbance. Rakhimzhanova et al.\textsuperscript{65} studied the compression strength of sandstone under dynamic disturbance with coupled dynamics and a static loading split Hopkinson pressure bar system. However, the dynamic characteristics of the prestatic load and cyclic dynamic disturbance need to be further studied. The dynamic deformation and energy change law of rocks containing structural planes are especially worth further study.

Therefore, to gain insight into the dynamic mechanical characteristics and failure characteristics of a rock mass under dynamic disturbance, a uniaxial rock system was used to evaluate rock mass specimens. In this study, the rock mass specimens were taken from Anjialing open-pit mine in Shanxi Province, China. Samples with structural planes of different angles were taken by using a microcontrol electro-hydraulic servo fatigue testing machine. The dynamic mechanical characteristics and failure characteristics of rock mass under dynamic disturbance are discussed, including the stress-strain law, plastic strain evolution law, and energy principle of strength weakening under periodic dynamic disturbance. Moreover, according to the relationship between cumulative plastic strain and disturbance cycle times based on the damage theory, a constitutive model of rock with structural planes under the influence of a disturbance threshold was constructed, and the model was verified by using test data. This typical case study can provide a reference for open-pit combined mining rock slopes with structural planes controlled under dynamic disturbance.

2  |  METHODOLOGY

2.1  |  Test equipment

A PA-100 microcontrolled electro-hydraulic servo fatigue testing machine is used as the loading system, as shown in Figure 1. The maximum static load of the testing machine is 100 kN, and its maximum dynamic load is ±100 kN. The disturbance frequency ranges from 0.01 to 30 Hz, and the dynamic load capacity is strong, with high measurement accuracy. Low-frequency fatigue test software is used to simulate the dynamic stress wave loading of rock materials. A DH5922D dynamic strain gauge produced by JSDH (CHINA) Testing Technology Development Co., LTD, is adopted for dynamic data collection and recording.

2.2  |  Sample preparation

It is difficult to sample a natural jointed rock mass, and some structural plane conditions can hardly meet the test design requirements. Therefore, the joint rock samples are prepared by prefabrication or manual preparation in this study. The test samples were taken from the 1300-m platform sandstone block of the north slope of Pingshuo Anjialing, Shanxi Province, China (Figure 2). First, rock samples were drilled into cylinders with a diameter of 50 mm, and then, the rock

![FIGURE 1] Dynamic disturbance test system: A, Schematic diagram of the testing machine; B, Picture of the testing machine
samples were made into coal-rock samples with a height of approximately 100 mm. Then, after grinding the cutting surface, the two parts are glued together with epoxy resin adhesive, to make composite samples with different inclination angles. The nonparallelism of the two ends is less than 0.05 mm, the diameter deviation of the upper and lower ends is less than 0.03 mm, and the axial deviation is less than 0.25°. Processed sandstone specimens with different dip angles are shown in Figure 3. The basic parameters of samples with different inclination angles were tested. During the measurement, the test position was changed 3 times for each parameter, and the average value was taken as the final result. The test results of each sample are shown in Table 2. It is worth noting that secondary joints widely distribute in natural rock masses generally. In the process of rock sample collection, it is impossible to guarantee that all rock samples collected are homogeneous. Hence, in the process of preparing artificial rock samples with structural planes in the laboratory, the homogeneous natural rock samples should be selected as far as possible. In the analysis of the test results, we choose the test data of rock samples without natural joint for this study.

2.3 Test plan

During the cyclic dynamic disturbance tests of rocks under a certain static load, all the test samples in this work were automatically controlled via computer by means of continuous loading. First, the static load is applied to the rock. Load control increases the load at a certain loading speed. When the static stress ($\sigma$) reaches a certain predetermined value, $\sigma_m$ is taken as the average stress, and the cyclic dynamic load is applied. Load control maintains a constant loading rate during the cyclic disturbance. To simulate the elastic wave during vibration propagation, the cyclic disturbance waveform is in the form of a sinusoidal wave, and the testing machine loads the rock with constant upper and lower loads until the rock is damaged.

The loading process and characteristics are shown in Figure 4. $\sigma_{\text{max}}$ and $\sigma_{\text{min}}$ are the upper and lower limit stresses of cyclic load. $\Delta \sigma = \sigma_{\text{max}} - \sigma_{\text{min}}$ is the dynamic disturbance amplitude. $T$ is the loading period. During the test, the testing system can collect the axial load, axial deformation, transverse deformation, and time data and draw the corresponding parameter relationship curve.

To obtain reasonable values of $\sigma_{\text{max}}$ and $\sigma_{\text{min}}$, a uniaxial compression test is conducted on the samples using the TAW-2000 uniaxial servo-hydraulic testing machine. The uniaxial compression stress-strain curve is obtained, and it is shown in Figure 5. The yield strength of the rock containing structurally plane rock is calculated as the average value of cyclic loading stress, and an axial cyclic disturbance load of the sinusoidal waveform with a fixed frequency of 2 Hz is applied to the samples. The specific steps of the periodic loading test are as follows: (a) Samples are placed on the low actuator of the testing machine, and a 100 mm $\times$ 50 mm $\times$ 20 mm rigid backing plate is placed on each side of the samples; and (b) test parameters are set, and the loading waveform is a sinusoidal wave with an amplitude that is set to exceed the estimated strength value of the samples. During the cyclic loading tests, the yield strength is taken as the centerline of the test, that is, $\sigma_m$. The rock specimen is placed on the lower actuator, and the upper actuator is slowly moved down until it touches the specimen and applies a force of approximately 2 kN. The position of the lower actuator is then adjusted by force control on the test control interface, and it is slowly loaded to the centerline position, which is the yield strength of rock specimen. This state is held for a period of time before the experiment begins. The upper actuator remains fixed, and the lower actuator applies a cyclic load according to preset conditions. This experiment is divided into 12 groups, which contains 3 pieces. Rock samples with dip angles of 0°, 15°, 30°, and 45° are selected to conduct disturbance tests at different amplitudes. The test sample number consists of three parts, which are $\theta$, the average load of the cyclic load, and the disturbance amplitude of the cyclic load. The average cyclic load and disturbance amplitude of the second and third parts are expressed according to the actual applied loads. For example, A25-4 means that $\theta$ is 0°, the average cyclic load is 25 MPa, and $\Delta \sigma$ is 4 MPa. The specific test scheme is shown in Table 3.

3 RESULTS AND DISCUSSION

3.1 Analysis of the stress-strain curve of rock under dynamic disturbance

The corresponding yield strength is applied to samples containing structural planes with different dip angles, and then, cyclic perturbations under different stress amplitudes are carried out. The cracks generated by the destroyed rocks developed along the axial direction and eventually formed many vertical cracks, which then fragmented into relatively thin sheets or strips of debris, showing a tendency to expand along the structural plane. The stress-strain curves under cyclic loading of samples with different dip angles under different amplitudes are shown in Figure 6. Due to the nonlinear characteristics of the rocks, the deformation of samples lags behind the change in external stress, forming a closed hysteresis loop curve under each loading and unloading cycle, whose area reflects the energy consumed by the sample during cyclic loading. In the first five cycles of loading, the distribution of the stress-strain hysteresis ring is sparse, and the inelastic strain is large (see Figure 6). After the sixth cycle, the hysteresis loop curve becomes denser as the number of
FIGURE 2  A, Location of the study area; B, Sample collection in combined mining area
cycles increases, the spacing between each loop gradually decreases, and the inelastic strain decreases. The area of the hysteresis loop decreases as \( n \) increases and finally stabilizes. This trend occurs because there are many microcracks in the rock; these cracks close and cause large deformations when subjected to dynamic load. When the microcracks in the rock are rammed to a certain extent, the deformation rate of the rock tends to be stable, with plastic strain occurring slowly and uniformly. In the 2-3 cycles before rock failure, the area of the hysteresis loop curve increases rapidly due to the complete penetration of cracks caused by cyclic loading. Overall, the curve forms three stages: a sparse stage, a dense stage, and another sparse stage.

Figure 6A shows that the plastic strain accumulation value of the samples after 80 loading cycles tends to be stable, and the samples are not damaged when the \( \theta = 0^\circ \) under different disturbance stress values. Figure 6B shows that the accumulated plastic strain values change significantly when \( \theta = 15^\circ \) under \( \Delta \sigma \) of 4 MPa and 6 MPa after 80 cycles, but no damage occurs. When \( \Delta \sigma \) reaches 8 MPa, damage begins to accrue in the samples after 70 loading cycles. Figure 6C shows that, when \( \theta = 30^\circ \), the samples began to sustain damage after 70 and 60 loading cycles under \( \Delta \sigma \) values of 6 MPa and 8 MPa, respectively; however, no damage occurs after 80 loading cycles when \( \Delta \sigma = 4 \) MPa. Moreover, when \( \theta = 45^\circ \), the samples are damaged after 65, 60, and 45 loading cycles under \( \Delta \sigma \) values of 4 MPa, 6 MPa, and 8 MPa, respectively (Figure 6D). Therefore, for samples with the same \( \theta \) values, the larger the value of \( \Delta \sigma \), the larger area of the hysteresis loop curve, and the larger the corresponding rock deformation. Thus, the increase in \( \Delta \sigma \) causes the rock to accumulate consumable energy within each cycle, which is consumed by the bond-slip between mineral grains and new cracks, resulting in damage in the rock and decreasing the total amount of cumulative irreversible deformation required for failure, which will significantly degrade the load resistance of the samples. In contrast, the smaller the hysteresis loop curve area, the closer the rock strength characteristic to the strength under simple continuous loading. There is a disturbance threshold for the disturbance stress amplitude when the rock contains structural planes.

In addition, the cyclic loading stress-strain curves of different samples under different \( \Delta \sigma \) values are as given in

**FIGURE 3** Rock containing structural planes with different dip angles: A. Test rock samples with different \( \theta \); B. Schematic diagram of sample.
Figure 7, which shows that, with an increase in $\theta$, the change in amplitude of the hysteresis loop curves and the amount of deformation of the rock increase, but the failure intensity of the rock decreases, and the rock is more prone to failure. During cycle loading, when $\sigma_{\text{max}}$ is smaller, the slope of the hysteresis loop curve is small, indicating that cyclic loading expands the cracks inside the sample and reduces the stiffness of the sample. With the increase in $\Delta \sigma$, the amount of rock deformation in each cycle dynamic disturbance period increases, and $n$ decreases. When $\sigma_{\text{max}}$ is close to the rock static strength, the variation in the average stress and cyclic stress amplitudes will have a great influence on the fatigue life of rock under dynamic disturbance.

The relationship of rock mass specimens with different dip angles of the structural plane under different dynamic disturbance amplitudes and the number of cycle times is presented in Figure 8, which shows the value of the dynamic disturbance amplitude when rock mass specimens with different structural dip angles are damaged under different numbers of cycles and dip angles of structural planes. From the perspective of the dip angles of structural planes, the rock mass specimens are prone
### TABLE 3  Coupled static and dynamic loading compression test program

| Dip angle θ (°) | No. | Static stress σ (MPa) | Frequency (Hz) | Amplitude (MPa) | σ<sub>max</sub> (MPa) | σ<sub>min</sub> (MPa) | σ<sub>m</sub> (MPa) |
|-----------------|-----|-----------------------|----------------|----------------|---------------------|---------------------|-------------------|
| 0               | A25-4 | 25                    | 2              | 4              | 29                  | 21                  | 25                |
|                 | A25-6 | 25                    | 2              | 6              | 31                  | 19                  | 25                |
|                 | A25-8 | 25                    | 2              | 8              | 33                  | 17                  | 25                |
| 15              | B22-4 | 22                    | 2              | 4              | 26                  | 18                  | 22                |
|                 | B22-6 | 22                    | 2              | 6              | 28                  | 16                  | 22                |
|                 | B22-8 | 22                    | 2              | 8              | 30                  | 14                  | 22                |
| 30              | C18-4 | 18                    | 2              | 4              | 22                  | 14                  | 18                |
|                 | C18-6 | 18                    | 2              | 6              | 24                  | 12                  | 18                |
|                 | C18-8 | 18                    | 2              | 8              | 26                  | 10                  | 18                |
| 45              | D16-4 | 16                    | 2              | 4              | 20                  | 12                  | 16                |
|                 | D16-6 | 16                    | 2              | 6              | 22                  | 10                  | 16                |
|                 | D16-8 | 16                    | 2              | 8              | 24                  | 8                   | 16                |

**FIGURE 6**  Cyclic loading stress-strain curves of rock samples under different disturbance stress amplitudes: A, θ = 0°; B, θ = 15°; C, θ = 30°; D, θ = 45°
The damage of rock mass specimens occurs 1 time, 2 times, and 3 times when the \( \theta \) values are 15°, 30°, and 45°, respectively. Moreover, the corresponding \( \Delta \sigma \) values on the damage point are 8 MPa, 8 MPa, 6 MPa, 8 MPa, 6 MPa, and 4 MPa. As can also be observed in Figure 8, with the increase in \( \theta \), the number of loadings required for the corresponding damage to the rock mass specimens is decreased. The relationship between the dip angles and dynamic disturbance amplitudes with peak stress is calculated as shown in Figure 9. Figure 9A shows that, as the dynamic disturbance amplitude increases, the required peak stress first increases and then decreases. The relationship between dynamic disturbance amplitude and peak stress becomes sinusoidal, conforming to an exponential relationship. Figure 9B shows that, as the dynamic disturbance amplitude increases, the required peak stress first increases and then decreases. The relationship between dynamic disturbance amplitude and peak stress becomes sinusoidal, conforming to a sinusoidal relationship. Then, we fit the relationships of dip angles and dynamic disturbance amplitude with peak stress. The coefficients of determination \( (R^2) \) are both greater than 0.97.

Through the above analysis, it can be concluded that the strength of rock with structural planes during cyclic loading and unloading is related to its dip angle, the amplitude of disturbance stress, and the number of cycles.

The area of the hysteresis loop curve reflects the maximum elastic strain energy consumed and stored during a cycle, as shown in Figure 10. Generally, the plastic characteristics of the rock can be analyzed through the ratio of hysteresis energy and damping ratio \( \gamma \) and the deformation modulus \( E \) and \( \gamma \) of a single cycle can be defined as

\[
E = \frac{\sigma'_{\max} - \sigma'_{\min}}{(\varepsilon'_{\max} - \varepsilon'_{\min})} \quad \text{and} \quad \gamma = A/(4\pi A_b)
\]

where \( \sigma'_{\max} \) and \( \sigma'_{\min} \) are the maximum and minimum stresses of the hysteresis loop curve. The values of \( \varepsilon'_{\max} \) and \( \varepsilon'_{\min} \) are their corresponding strains. \( A \) is the area enclosed by the hysteresis loop curve. \( A_b \) is the area of triangle \( ABD \).

The cyclic loading experiment results are calculated according to the above equations. The change curves of \( E \) with \( \theta \) under different disturbance stresses are shown in Figure 11, and the change curves of \( \gamma \) with the cycles of different samples are shown in Figure 12. Figure 11 shows that \( E \) decreases with the increase in \( \theta \) under a certain \( \Delta \sigma \), which is because \( \theta > 0 \) leads to the occurrence of macroscopic cracks. The cracks become larger with \( \theta \), leading to a higher degree of fatigue for samples under cyclic load and a decrease in \( E \). When \( \theta \) is fixed, larger values of \( \Delta \sigma \) and \( E \) indicate that \( \Delta \sigma \) is positively correlated with the fatigue degree under cyclic loading. Figure 12 shows that the \( \gamma \) value of the rock decreases first with the increase in the number of cycles and then tends to stabilize, indicating that cyclic loading strengthens the rock. Then, the \( \gamma \) value of the rock shows an increasing trend because the strength of the rock gradually deteriorates with the increase in the number of cycles, causing cracks to expand and resulting in softening. According to Figure 12, the plastic deformation (fracture expansion) of rock can be divided into three stages: the initial deformation (pore compaction) stage, the isokinetic deformation (microfracture stable development) stage, and the accelerated deformation (fracture expansion).
penetration) and overall failure stage. Figure 12 also shows that, as $\theta$ increases, $n$ decreases when $\gamma$ increases, indicating that the rock is easier to break with an increase in $\gamma$.

**FIGURE 8** Heat map of rock mass specimen status vs dip angles and number of cycles

3.2 Plastic strain evolution law of rock under dynamic perturbation

Taking $\theta = 15^\circ$ as an example (Figure 6B), the axial effective plastic deviation strain of $\sigma'_\text{max}$ is taken as the parameter for irreversible deformation, and the relationship between cumulative plastic strain ($\varepsilon^D$) and $n$ is as shown in Figure 13. Figure 13 shows that their relationship can be described as follows: (a) in curve I, the rock is not damaged when $\Delta \sigma$ is small; and (b) in curve II, the rock is damaged when $\Delta \sigma$ is large. To describe the curves of two kinds of stress states, a disturbance threshold $\Delta \sigma'$ should be introduced. When $\Delta \sigma < \Delta \sigma'$, with an increase in the loading and unloading cyclic period, $\varepsilon^D$ tends to be stable, and the samples are not damaged. In contrast, when $\Delta \sigma > \Delta \sigma'$, the relationship curve between $\varepsilon^D$ and $n$ during the disturbance cyclic loading process presents an S shape.

**FIGURE 9** Correlation analysis: A, dip angle vs peak stress; B, dynamic disturbance amplitude vs peak stress

**FIGURE 10** Stress-strain hysteresis loop curve

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In the initial acceleration stage, the increasing rate and increment in $\varepsilon^D$ are large, but with increasing $n$, they gradually become small. When $\varepsilon^D$ increases a certain extent, the increasing rate has a sudden increase, leading to the failure of the samples. The fitting curve of $\varepsilon^D$ and $n$ is shown in Figure 13, which shows that the evolution law of $\varepsilon^D$ and $n$ around $\Delta \sigma'$ conforms to the negative exponential function$^{66,67}$ and the inverse function expression of the Langevin function$^{68-71}$:

$$\varepsilon^D = A - \exp \left( -\frac{n}{B} \right) \left( \Delta \sigma < \Delta \sigma_B \right)$$

$$n = c + a \coth(\varepsilon^D - b) - \frac{1}{\varepsilon^D - b} \left( \Delta \sigma \geq \Delta \sigma_B \right)$$

$\coth(\varepsilon^D - b) = \frac{\varepsilon^{D_0} - b}{\varepsilon^{D_0} - b + \varepsilon^{\theta}}$

(1)

where the $A$, $B$, $a$, $b$, and $c$ are undetermined parameters.

Given the abovementioned analysis, the $\Delta \sigma'$ value for the permanent deformation of rock under dynamic disturbance is related to both $\sigma$ and $\theta$. According to the above method, the relationship between $\varepsilon^D$ of samples with different dip angles and $n$ under different stress amplitudes was analyzed. $\Delta \sigma'$ was fitted with the two-parameter curve of static load and $\theta$, and the fitting results are shown in Figure 14. Based on the original data of $\sigma_m$, $\theta$, and $\Delta \sigma'$ in Figure 14, we use the nonsurface fit function to obtain the relationship between the dynamic disturbance threshold, the dynamic disturbance amplitude, and the dip angles. $\Delta \sigma'$ is expressed as:

$$\Delta \sigma' = 42.73 \exp \left( -\sqrt{\frac{\theta^2 + \sigma_m^2}{19.26}} - 2.42 \right)$$

(2)

In addition, the physical significance of the parameters ($A$, $B$, $a$, $b$, and $c$) is discussed to analyze their effects on the plastic strain evolution law of rock under dynamic perturbation. Some typical values of these parameters are selected, and the influence of parameters on the $n$-$\varepsilon^D$ curve can be obtained (Figure 15). For example, the physical significance of parameter $A$ is as follows. In Equation (1), to describe curve I, parameter $B$ was fixed to 32, and $A$ was set to 3.0, 3.5, and 4.0 (Figure 15A). Figure 15A shows that $\varepsilon^D$ increases with the increase in $A$, and when the three curves are approximately equal to their corresponding $A$ values, $\varepsilon^D$ will reach a stable stage. The parameter $A$ can basically determine the stable value of the $\varepsilon^D$ of rock during cyclic loading, which has a controlling effect on the $\varepsilon^D$ value of the stable phase in curve I. Therefore, Parameter $A$ is defined as the axial plastic strain accumulation rate factor in the initial stage. Figure 15B shows that the $\varepsilon^D$ values of the three curves tend to be stable with an increase in $n$, but as the value of parameter $B$ increases, the $n$ value before the stable stage increases gradually, which indicates that the change rate of $\varepsilon^D$ has a negative correlation with the parameter $B$. Therefore, the physical meaning of the parameter $B$ is the I characterization curve slope size, indicating that curve I epsilon $D$ rate and the size of the $B$ value show a negative correlation. Therefore, the physical significance of the parameter $B$ is represented by the slope of curve I. Figure 15C shows that the three curves will intersect at some point with increasing $n$, and the values of $n$ and $\varepsilon^D$ at that point are approximately equal to $c$ and $b$, respectively. The value of $n$ when the rock is damaged increases with the increase in $a$, but the slope of curve II decreases gradually in the initial stage, indicating that the change rate of $\varepsilon^D$ has a negative correlation with $a$. Hence, Parameter $a$ is defined as the change rate factor of $\varepsilon^D$ in the initial stage when $\Delta \sigma$ is large, representing the slope of curve II in the initial stage, and controlling $n$ before rock damage. Figure 15D shows that the $\varepsilon^D$ values of rock increase with the increase in $b$, and the rock is ultimately damaged when $n$ reaches 85. Parameter $b$ is defined as the intensity factor of the rock itself, which can better reflect the $\varepsilon^D$ values of rocks with different strengths under the same cyclic loading condition. Moreover, Figure 15E shows that $n$ before the rock is damaged increases with $c$, and the hysteresis phenomenon will occur in the acceleration phase, but the value of $\varepsilon^D$ decreases. Based on this characteristic, Parameter $c$ is defined as the plastic cumulative strain rate factor in the acceleration stage.
FIGURE 12  Change curve of damping ratio and disturbance cycle number of rocks with different dip angles: A, $\theta = 0^\circ$; B, $\theta = 15^\circ$; C, $\theta = 30^\circ$; D, $\theta = 45^\circ$
3.3 Energy failure mechanism of rock under dynamic perturbation

The loading and unloading processes of the rock are driven by the increase and release of the internal energy of rock, respectively. Figure 16 shows the energy change process of the rock containing the structural plane during cyclic loading and unloading. When the upper limit stress of cyclic loading is $\sigma_1$, the strain of the rock is $\varepsilon_1$, and the increased energy inside the rock is $W$, as shown in Figure 16A. When the lower limit stress of the cyclic load is $\sigma_2$, the strain decreases to $\varepsilon_2$ under unloading, with the energy released by the rock being $W_1$, as shown in Figure 16B. When the rock is subjected to secondary loading, the stress will reach the maximum limit of cycle load $\sigma_1$, and the strain also changes to $\varepsilon_1$; furthermore, the work energy increases to $W_2$ in this process, as shown in Figure 16C. Due to the rock deformation hysteresis characteristics, the two loading stress-strain curves and the unloading curve do not overlap; therefore, when the stress level is $\sigma_1$, $W_1$ is larger than $W_2$, and the newly increased energy in the rock $W_3 = W_2 - W_1$. That is, after the loading and unloading cycle reaches the upper limit stress of the cyclic load, the increased energy inside the rock is $W_3$, as shown in Figure 16D. As long as the stress-strain curves of loading and unloading do not coincide and there is a hysteresis ring curve, the new energy $W_3$ always exists and gradually increases with an increase in loading time. When the accumulated energy inside the rock exceeds its ultimate bearing capacity, weak local links will be destabilized, and the energy will be released. This is the energy failure mechanism of rock with structural planes under cyclic loading.

Given the abovementioned analysis, the larger the hysteresis loop curve area, the larger the new increased energy $W_3$ inside the rocks containing structural planes, and the more prone to failure the rocks under cyclic loading. In contrast, the smaller the hysteresis loop curve area, the closer the rock strength to the strength value under uniaxial compression. The cumulative area of the hysteresis loop curve increases, but the intensity of rock decreases with the increase in the dip angle, the cycle number and the disturbance stress amplitude. According to the analysis of the cyclic loading-unloading process of rock mass specimens with dip angles of 0°, 15°, 30°, and 45° under different disturbance amplitudes, it can be observed that the cumulative value of new increased energy also increases as a result of cyclic loading-unloading with the increase in $\theta$. Specifically, when $\theta$ is 45°, the difference between the loading-unloading curves is noticeable, and the hysteresis loop curve area is large, which leads to the reduction in the rock strength. Moreover, according to the comparison of the stress-strain curves of cyclic loading and unloading.
with \( \Delta \sigma \) values of 4 MPa and 8 MPa, the newly increased energy in the rock under a \( \Delta \sigma \) of 8 MPa is significantly larger than that under a \( \Delta \sigma \) of 4 MPa, indicating that the larger the value of \( \Delta \sigma \), the lower the rock strength, and the more prone the rock is to failure during cyclic loading and unloading.
4 | CONSTITUTIVE MODEL

4.1 | Derivation of damage evolution equation

The nonuniform failure of local microelements of rock material causes material damage; that is, the material is transformed from the linear elastic stress state to the nonlinear stress state. To consider damaged material at the microcosmic level, Lemaitre\(^{72}\) hypothesized the following. Assume that the number of damaged elements inside the rock material under a certain load is \(N\), and the statistical damage variable \(D_s\) is the ratio of \(N\) to the total number of damaged elements \(N_m\), that is, \(D_s = N/N_m\).\(^{73,74}\) The number of infinitesimal bodies generated in any interval \([\varepsilon, \varepsilon + d\varepsilon]\) is \(NP(\varepsilon)\,d\varepsilon\). When loading to a strain value, the number of microelements that have been destroyed is\(^{73}\):

\[
N(\varepsilon) = \int_0^\varepsilon NP(\varepsilon)\,d\varepsilon = N\left\{1 - \exp\left[-\left(\frac{\varepsilon}{F}\right)^m\right]\right\} \quad (3)
\]

On this basis, to reflect the nonlinear deformation characteristics of rock materials, a damage threshold parameter \(D_t\), also known as plastic strain, is introduced:

\[
N(\varepsilon) = \int_0^\varepsilon NP(\varepsilon)\,d\varepsilon = N\left\{1 - \exp\left[-\left(\frac{\varepsilon - D_t}{F}\right)^m\right]\right\} \quad (4)
\]

The relationship between \(D_s\) and the probability density of microelement failure can be expressed as:

\[
\int \frac{dD_s}{d\varepsilon} = N(\varepsilon) \quad (5)
\]

The damage evolution equation is as follows\(^{75}\):

\[
D_s = \begin{cases} 
0 & (\varepsilon < D_t) \\
1 - \exp\left[-\left(\frac{\varepsilon - D_t}{F}\right)^m\right] & (\varepsilon \geq D_t)
\end{cases} \quad (6)
\]

When \(D_s = 0\), there is no damage inside the rock, but when \(D_s = 1\), all the microelements inside the rock have been destroyed. The value of \(D_s\) reflects the degree of microelement damage inside the rock.\(^{75}\)

During rock material compression, compressive stress and shear stress are transferred to other microelement bodies after the failure of internal microelements, the effective area of compressive stress, and shear stress are the same, and the damage variables in all directions are \(D_s\). Therefore, the effective stress after compression can be assumed to be\(^{76}\):

\[
\sigma = E_s(1 - \delta D_s) \quad (7)
\]

where \(\delta\) is the damage scale coefficient of rock material, reflecting the residual strength of rock material, with a value
of $0 < \delta = \sqrt{\frac{\sigma_r}{\sigma_c}} \leq 1$, $\sigma_r$ is the residual strength of rock material, and $\sigma_c$ is its peak strength. $\delta = 1$, $\varepsilon > \gamma$, and substituting Equation (6) into Equation (7), we have Equation (8):

$$\sigma_s = E_\varepsilon \left\{ 1 - \delta + \delta \exp \left[ - \left( \frac{\varepsilon - D_s}{F} \right)^m \right] \right\}$$  \hspace{1cm} (8)

Without considering the size effect of rock material, $\delta$ is obtained, and the parameters $F$ and $m$ are determined by $\sigma_c$ and its corresponding strain $\varepsilon_c$. Since the slope at $\sigma_c$ during loading is 0,

$$\begin{align*}
\frac{d\sigma_c}{d\varepsilon} &= (1 - \delta)E + \delta E \left[ 1 - \left( \frac{\varepsilon - D_s}{F} \right)^m \right] \exp \left[ - \left( \frac{\varepsilon - D_s}{F} \right)^m \right] = 0 \\
\sigma_c &= (1 - \delta)E \varepsilon_c + \delta \varepsilon_c \exp \left[ - \left( \frac{\varepsilon - D_s}{F} \right)^m \right] \\
\end{align*}$$

According to Equation (9), the following equations can be obtained:

$$\begin{align*}
F &= \frac{\sigma_c}{\varepsilon_c \left[ \frac{\sigma_c}{(\delta - 1)E \varepsilon_c} \ln \frac{1}{\delta - 1} \right] + m \sigma_c + (\delta - 1)E \varepsilon_c} \\
\end{align*}$$

4.2 Deformation and failure constitutive model of structural plane

Under the action of an external stress level, the damage of rock with a structural plane is composed of undamaged materials and damaged materials, and the total stress is borne by these two parts. Suppose the nominal stress on the rock is $\sigma_i$ and the nominal area is $A^0$. The stress on the undamaged part is $\sigma'_i$, and the corresponding area is $A'$. The stress on the damaged part is $\sigma''_i$, and the corresponding area is $A''$; then:

$$\begin{align*}
\sigma_i A^0 &= \sigma'_i A' + \sigma''_i A'' \\
\sigma_i &= \frac{\sigma'_i A'}{A^0} + \frac{\sigma''_i A''}{A^0} \\
\end{align*}$$

Take $Ds = A'_i / A''$ as the damage variable of rock material, and substitute it into Equation (12):

$$\sigma_i A^0 = (1 - D_s)\sigma'_i + D_s \sigma''_i$$  \hspace{1cm} (11)

Assume that, under static loading, the stress-strain relationship of undamaged rock obeys the linear elastic relationship $\sigma'_i = E_i \varepsilon_i$, where $E_i$ is the elastic modulus of rock. Additionally, assume that the stress-strain relationship of the damaged part of the structural surface of the rock material under dynamic and static loads obeys the deformation law during the deformation process. The deformation of the structural plane of rock mass under dynamic disturbance includes the deformation of intact rock, the deformation amount of the structural plane under pressure, and the deformation amount caused by shear $\frac{\sigma''}{\eta \frac{d\sigma'}{dt}}$, where $\eta$ is the viscosity coefficient of the structural plane. When the structural plane is subjected to a compression load, the deformation of the structural plane includes the shear strain generated along the structural plane and the closed strain generated by the compression of the structural plane.

$$\begin{align*}
\sigma_i j_1 &= \sigma'' \sin^2 \alpha \cos \alpha \\
\sigma_i j_2 &= \frac{\sigma'' \cos^2 \alpha}{E_j \varepsilon_j} \cos \alpha \\
\end{align*}$$

Here, $k_i$ is the tangential stiffness of the structural plane; $L$ is the length of the monomer; $\varepsilon_j 1$, $\varepsilon_j 2$, and $\varepsilon_j 0$ are the shear strain, closure strain, and maximum closing strain of the structural plane, respectively; $E_j$ is the closed modulus of the structural plane; and $\alpha$ is the dip angle of the structural plane.

From the above equations, the deformation and failure constitutive model of the structural plane under the periodic dynamic disturbance of uniaxial compression considering a disturbance threshold can be obtained as follows:

$$\begin{align*}
\sigma_i &= (1 - D_s)\sigma'_i + D_s \sigma''_i \\
\sigma'_i &= E_i \varepsilon_i \\
\sigma''_i &= \eta \frac{d\sigma_i}{dt} \\
\varepsilon &= \varepsilon_1 + \varepsilon_2 \\
\sigma''_i &= \frac{\sigma'' \cos^2 \alpha}{E_j \varepsilon_j} \cos \alpha \\
\varepsilon_1 &= \frac{k_i L}{E_j \varepsilon_j} \sin^2 \alpha \cos \alpha \\
\varepsilon_2 &= \frac{\sigma'' \cos^2 \alpha}{E_j \varepsilon_j} \cos \alpha \\
\end{align*}$$

4.3 Constitutive model regression inversion

To verify the rationality of the elastic-plastic constitutive model of rock with a structural plane established, a cyclic addition-unloading constitutive model of rock is established by using the least squares method of origin nonlinear
FIGURE 17  Contrast curve of loading section test data and fitting value: A, 1st load cycle; B, 10th load cycle; C, 20th load cycle; D, 50th load cycle; E, 75th load cycle.
fitting, and fitting is realized by regression and inversion. Taking the working condition ($\theta = 15^\circ$, $\Delta \sigma = 6$ MPa) as an example, the experimental data at 1, 10, 20, 50, and 75 cycles are compared with the theoretical fitting curve, as shown in Figure 17. Figure 17 shows that the theoretical fitting curve is consistent with the experimental data. Thus, the established constitutive model can accurately describe the stress-strain curves for each cycle during the cyclic loading-unloading dynamic disturbance test, the upper and lower limit of the cycle disturbance load, and the corresponding axial strain value. This model can reflect the changing trend of the loss modulus during the disturbance process and fully reflects the feasibility and accuracy of the modeling method in this work.

5 | CONCLUSION

By carrying out experimental testing, the effect of periodic dynamic disturbance on the mechanical properties of a rock mass with different structural planes during uniaxial compression tests was investigated. For the north side slope in the Anjialing open-pit mine, different prestatic loads were applied to rock mass specimens under different dynamic disturbance amplitudes. The impacts of $\theta$ on the strength, deformation characteristics, energy dissipation characteristics, and fracture rules of rock mass specimens under different disturbance stress amplitudes were discussed systematically in this paper. Experimentally, closed hysteresis loop curves were formed during each loading and unloading cycle. Moreover, the relationship between plastic accumulated strain and the number of disturbance cycles was analyzed. Eventually, a constitutive model of rock under cyclic loading-unloading was established and verified. The specific conclusions of this paper are as follows:

1. The obtained hysteresis loop curves go through three stages: a sparse stage, a dense stage, and another sparse stage. The variation in the cumulative area of the hysteresis loop curves reflects the change law of the strength of rock with structural planes during cyclic loading and unloading. The cumulative area of the hysteresis loop curve increases with increasing $\theta$, $\Delta \sigma$, and $n$, but the intensity of rock decreases. The area of the hysteresis loop curve increases with $\Delta \sigma$ for rock with the same $\theta$. Specifically, $E$ decreases with increases in $\theta$ under a certain $\Delta \sigma$, but it increases with $\Delta \sigma$ when $\theta$ is fixed. The strength of rock with structural planes is related to $\theta$, $\Delta \sigma$, and $n$.

2. There is a disturbance threshold of the disturbance stress amplitude when the rock contains a structural plane, and the disturbance threshold is a function of $\sigma$ and $\theta$. The physical significance of the parameters ($A$, $B$, $a$, $b$, and $c$) was discussed to analyze their effects on the plastic strain evolution law of rock under dynamic perturbation.

3. Based on damage theory, the damage constitutive equation of rock with a structural plane under dynamic disturbance is deduced, and the constitutive model of rock under cyclic loading-unloading is established and verified by test data.

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NOMENCLATURE

- $T$: Load periodic (s)
- $n$: Number of disturbance cycles
- $A$: Area enclosed by hysteresis loop curve
- $E$: Deformation modulus of the rock
- $W$: Increased energy inside the rock
- $D_t$: Statistical damage variable
- $D_s$: Damage threshold

Greek alphabets

- $\sigma$: Static stress
- $\sigma_m$: Target set average stress
- $\sigma_{\text{max}}$: Upper limit stress of cyclic load
- $\sigma_{\text{min}}$: Lower limit stress of cyclic load
- $\sigma_{\text{max}}$: Maximum stress of the hysteresis loop curve
- $\sigma_{\text{min}}$: Minimum stress of the hysteresis loop curve
- $\Delta \sigma$: Dynamic disturbance amplitude
- $\Delta \sigma'$: Dynamic disturbance threshold
- $\sigma_r$: Residual strength of rock material
- $\sigma_c$: Peak strength of rock material
- $\sigma_i$: Effective stress
- $\varepsilon_{\text{max}}$: Maximum stress corresponding to maximum strain
- $\varepsilon_{\text{min}}$: Minimum stress corresponding to minimum strain
- $\varepsilon^D$: Cumulative plastic strain
- $\varepsilon_c$: Peak strength corresponding to strain
- $\theta$: Dip angle of the structural plane
- $\gamma$: Damping ratio
- $\delta$: Damage scale coefficient of rock material

CONFLICT OF INTEREST

None declared.
REFERENCES

1. Xia KZ, Chen CX, Deng YY, et al. In situ monitoring and analysis of the mining-induced deep ground movement in a metal mine. *Int J Rock Mech Min Sci*. 2018;109:32-51.

2. Li XB, Gong FQ, Tao M, et al. Failure mechanism and coupled static - dynamic loading theory in deep hard rock mining: a review. *JRMGE*. 2017;9(4):767-782.

3. Cao AY, Jing GC, Ding YL, Liu S. Mining - induced static and dynamic loading rate effect on rock damage and acoustic emission characteristic under uniaxial compression. *Saf Sci*. 2019;116:86-96.

4. Liu GW, Song DQ, Chen Z, Yang JW. Dynamic response characteristics and failure mechanisms of coal slopes with weak intercalated layers under blasting loads. *Adv Civ Eng*. 2020;5412795:1–18.

5. Joghnna LP, Jörg E, Martin Z, et al. Geogas transport in fractured hard rock - correlations with mining seismicity at 3.54 km depth, TauTona gold mine, South Africa. *Appl Geochem*. 2011;26(12):2134-2146.

6. Quiroz AG, Álvarez JPF. Conceptualization and finite element groundwater flow modeling of a flooded underground mine reservoir in the Asturian Coal Basin, Spain. *J Hydrol*. 2019;578:124036-124046.

7. Bieniawski ZT, Bernede MJ. Suggested methods for determining the uniaxial compressive strength and deformability of rock materials. *Int J Rock Mech Min Sci*. 1979;16(2):137-138.

8. Chen X, Tang CA, Kong XY. Study on progressive damage and failure of sandstone samples subjected to cyclic disturbance loads using a modified triaxial test system. *KSCE J Civ Eng*. 2019;23(5):2371-2383.

9. Fan YB, Li SH, Zhou Y, Zhiyong F, Xiaoou L. Lessons learned from the landslides in Shengli east open-pit mine and north open-pit mine in Xilinhot City, Inner Mongolia province, China. *Geotech Geol Eng*. 2016;34(2):425-435.

10. Li ZR, Colinet G, Zu YQ, et al. Species diversity of *Arabis alpina* L. communities in two Pb/Zn mining areas with different smelting history in Yunnan Province, China. *Chemosphere*. 2019;233:603-614.

11. Guo MS, Liu T, Zhao LJ, Wu ZY, Liu Y. Main frequency band of blast vibration signal based on wavelet packet transform. *Appl Math Model*. 2019;74:569-585.

12. Wang JQ. Rock burst criticality evaluation and prevention countermeasures in gold deposits. *Gold*. 2007;28(6):24-28 (in Chinese with English abstract).

13. Ren FY, Hang H, Ren GY, Liu ZY. Experimental study on stope rockburst control in Erdaoqou gold mine. *J Northeast Univ*. 2012;33(6):891-894 (in Chinese with English abstract).

14. Zhang HP. Ideas, techniques and methods for prospecting replacement resources in crisis gold mines: a case study of gold exploration breakthroughs in western Hunan. *World Nonferrous Met*. 2019;2:89-90 (in Chinese with English abstract).

15. Wang LF, Wu XB, Zhang BY, et al. Recognition of significant surface soil geochemical anomalies via weighted 3D shortest-distance field of subsurface orebodies: a case study in the Hongshtoushan copper mine, NE China. *Nat Resour Res*. 2019;28(3):587-607.

16. Liu ZF, Shao YJ, Zhang Y, Wang C. Geochemistry and geochronology of the Qingshangjiao granites: implications for the genesis of the Dongguashan copper (gold) ore deposit in the Tongling ore district, Eastern China. *Ore Geol Rev*. 2018;99:42-57.

17. Li P, Guo SH, Zhao JT, Gao YX. Human biological monitoring of mercury through hair samples in China. *Bull Environ Contam Toxicol*. 2019;102(5):701-707.

18. Weng L, Huang LQ, Taheri A, Li XB. Rockburst characteristics and numerical simulation based on a strain energy density index: a case study of a roadway in Linglong gold mine, China. *Tunn Undergro Space Technol*. 2017;69:223-232.

19. Zhao SR, Li JW, Lentz D, Bi SJ, Zhao XF, Tang KF. Discrete mineralization events at the Hongtuling Au-(Mo) vein deposit in the Xiaoqinling district, southern North China Craton: evidence from monazite-U-Pb and molybdenite Re-Os dating. *Ore Geol Rev*. 2019;109:413-425.

20. Zhang F, Yang TH, Li LC, Wang Z, Xiao P. Cooperative monitoring and numerical investigation on the stability of the south slope of the Fushun west open-pit mine. *Bull Eng Geol Environ*. 2019;78(4):2409-2429.

21. Wang SH, Zhou W, Cai QX, Shi X, Lu X, Luan B. The coal mining model under slipper slope in Yiminhe open pit coal mines. *GeoTech Geol Eng*. 2019;37(5):3727-3737.

22. Wu ZH, Lei SG, Lu QQ, Bian ZF. Impacts of large-scale open-pit coal base on the landscape ecological health of semi-arid grasslands. *Remote Sens*. 2019;11(15):1820.

23. Wang BY, Qin Y, Shen J, Wang G, Zhang Q, Liu M. Experimental study on water sensitivity and salt sensitivity of lignite reservoir under different pH. *J Pet Sci Eng*. 2019;172:1202-1214.

24. Dowding CH, Andersson CA. Potential for rock bursting and slabbing in deep caverns. *Eng Geol*. 1986;22(3):265-279.

25. Li C, Hu YQ, Meng T, Jin PH, Zhao ZR, Zhang CW. Experimental study of the influence of temperature and cooling method on mechanical properties of granite: implication for geothermal mining. *Energy Sci Eng*. 2020;8(5):1716-1728.

26. Tonkovich A, Li ZB, DiCecco S, Altenholf W, Banting R, Hu H. Experimental observations of tyre deformation characteristics on heavy mining vehicles under static and quasi-static loading. *J Terramech*. 2012;49(3–4):215-231.

27. Xu H, Wang G, yang Guo Y, Chang B, Hu Y, Fan J. Theoretical, numerical, and experimental analysis of effective extraction radius of coalbed methane boreholes by a gas seepage model based on defined criteria. *Energy Sci Eng*. 2020;8:880-897.

28. Nie W, Krautblatter M, Leith K, et al. A modified tank model including snowmelt and infiltration time lags for deep-seated landslides in alpine environments (Aggenalm, Germany). *Nat Hazards Earth Syst Sci*. 2017;17(9):1595-1610.

29. Kaiser PK, Yazici S, Maloney S. Mining - induced stress change and consequences of stress path on excavation stability - a case study. *Int J Rock Mech Min Sci*. 2001;38(2):167-180.

30. Zhao Z, Gao X, Chen S, Zhang M, Sun W. Coupling model of disk splitting for expansive rock mass in deep storage considering water infiltration. *Energy Sci Eng*. 2020;4:1-17.

31. Fan XY, Nie W, Xu FL, et al. Dimension analysis-based model for prediction of shale compressive strength. *Adv Mater Sci Eng*. 2016;2016:1-12.

32. Pasculli A, Calista M, Sciarra N. Variability of local stress states resulting from the application of Monte Carlo and finite difference methods to the stability study of a selected slope. *Eng Geol*. 2018;245:370-389.
33. Zhu WC, Li S, Niu LL, Liu K, Xu T. Experimental and numerical study on stress relaxation of sandstones disturbed by dynamic loading. *Rock Mech Rock Eng*. 2016;49(10):3963-3982.

34. Su GS, Hu LH, Feng XT, et al. True triaxial experimental study of rockbursts induced by ramp and cyclic dynamic disturbances. *Rock Mech Rock Eng*. 2018;51(4):1027-1045.

35. Song DQ, Al C, Zhu RJ, Ge XR. Dynamic response characteristics of a rock slope with discontinuous joints under the combined action of earthquakes and rapid water drawdown. *Landslides*. 2018;15(6):1109-1125.

36. Song DQ, Che AL, Zhu RJ, Ge XR. Seismic stability of a rock slope with discontinuities under rapid water drawdown and earthquakes in large-scale shaking table tests. *Eng Geol*. 2018;245:153-168.

37. Iverson RM, George DL. Basal stress equations for granular debris masses on smooth or discretized slopes. *J. Geophys Res.-Earth Surf*. 2019;124(6):1464-1484.

38. Zhu WC, Li SH, Li S, Niu LL. Influence of dynamic disturbance on the creep of sandstone: an experimental study. *Rock Mech Rock Eng*. 2019;52(4):1023-1039.

39. Zhou CT, Xu CS, Karakus M, Shen JY. A particle mechanics approach for the dynamic strength model of the jointed rock mass considering the joint orientation. *Int J Numer Anal Methods Geomech*. 2019;43(18):2797-2815.

40. Song DQ, Chen Z, Hu C, Ke YT, Nie W. Numerical study on seismic response of a rock slope with discontinuities based on the time-frequency joint analysis method. *Soil Dyn Earthq Eng*. 2020;133:106112.

41. Song DQ, Chen Z, Ke Y, Nie W. Seismic response analysis of a bedding rock slope based on the time-frequency joint analysis method: a case study from the middle reach of the Jinsha River, China. *Eng Geol*. 2020;274:105731.

42. Liu Z, Ni F, Wei C, Li H. Experimental and numerical investigation of roadheader for breaking rock containing predrill holes. *Energy Sci Eng*. 2020;8:2511-2526.

43. Liu Y, Ba Q, He L, Shen K, Xiong W. Study on the rock-breaking effect of water jets generated by self-rotatory multinozzle drilling bit. *Energy Sci Eng*. 2020;8:2457-2470.

44. Shao ZL, Wang DM, Wang YM, Zhong X, Tang X, Hu X. Controlling coal fires using the three-phase foam and water mist techniques in the Anjialing Open Pit Mine, China. *Nat Hazards*. 2015;75(2):1833-11852.

45. Song DQ, Chen JD, Cai JH. Deformation monitoring of rock slope with weak bedding structural plane subject to tunnel excavation. *Arab J Geosci*. 2018;11(11):251-261.

46. Li ZL, Wang LG, Lu YL, et al. Experimental investigation on true triaxial deformation and progressive damage behaviour of sandstone. *Sci Rep*. 2019;9:1-19.

47. Vyzazmensky A, Stead D, Elmo D, Moss A. Numerical analysis of block caving-induced instability in large open pit slopes: a finite element/Discrete element approach. *Rock Mech Rock Eng*. 2010;43(1):21-29.

48. Song DQ, Che AL, Zhu RJ, Ge XR. Natural frequency characteristics of rock masses containing a complex geological structure and their effects on the dynamic stability of slopes. *Rock Mech Rock Eng*. 2019;52(11):4457-4473.

49. Jiang MJ, Jiang T, Crosta GB, et al. Modeling failure of jointed rock slope with two main joint sets using a novel DEM bond contact model. *Eng Geol*. 2015;193:79-96.

50. Song DQ, Liu XL, Huang J, Zhang JM. Dynamic response characteristics of a rock slope with discontinuous joints under the combined action of earthquakes and rapid water drawdown. *Landslides*. 2020;17(1):1-19.

51. Song DQ, Liang SY, Wang ZQ. The influence of reservoir filling on a preexisting bank landslide stability. *Indian J Geo-Marine Sci*. 2018;47(2):297-300.

52. Du H, Song D, Chen Z, Shu H, Guo Z. Prediction model oriented for landslide displacement with step-like curve by applying ensemble empirical mode decomposition and the PSO-ELM method. *J Clean Prod*. 2020;270:122248.

53. Gao YH, Feng XT. Study on damage evolution of intact and jointed marble subjected to cyclic true triaxial loading. *Eng Fract Mech*. 2019;215:224-234.

54. Lei RD, Wang Y, Zhang L, et al. The evolution of sandstone microstructure and mechanical properties with thermal damage. *Energy Sci Eng*. 2019;9:3058–3075.

55. Zhang QG, Nie W, Fan XY, et al. Mechanical behavior and permeability evolution of reconstituted coal samples under various unloading confining pressures-implications for Wellbore stability analysis. *Energies*. 2017;10(3):292.

56. Chen SJ, Jiang TQ, Wang HY, Feng F, Yin D, Li X. Influence of cyclic wetting - drying on the mechanical strength characteristics of coal samples: a laboratory-scale study. *Energy Sci Eng*. 2019;7(6):3020-3037.

57. Zhou ZJ, Ren CN, Xu GJ, Zhan H, Liu T. Dynamic failure mode and dynamic response of high slope using shaking table test. *Shock Vib*. 2017;2019:1-19.

58. Luo DN, Su GS, Zhang GL. True - triaxial experimental study on mechanical behaviours and acoustic emission characteristics of dynamically induced rock failure. *Rock Mech Rock Eng*. 2019;53(3):1205-1223.

59. Niu LL, Zhu WC, Cheng Z, Guan K, Qin T. Numerical simulation on excavation - induced damage of rock under quasi - static unloading and dynamic disturbance. *Environ Earth Sci*. 2017;76:614-628.

60. Zhou ZL, Cai X, Li XB, Cao W, Du X. Dynamic response and energy evolution of sandstone under coupled static-dynamic compression: insights from experimental study into deep rock engineering applications. *Rock Mech Rock Eng*. 2019;53(3):1305-1331.

61. Wang SF, Li XB, Yao JR, et al. Experimental investigation of rock breakage by a conical pick and its application to non-explosive mechanized mining in deep hard rock. *Int J Rock Mech Min*. 2019;122:104062-104076.

62. Wu QH, Li XB, Weng L, Li Q, Zhu Y, Luo R. Experimental investigation of the dynamic response of prestressed rockbolt by using an SHPB-based rockbolt test system. *Tunn Undergr Space Technol*. 2019;53:103088-103101.

63. Xu Y, Dai F. Dynamic response and failure mechanism of brittle rocks under combined compression -shear loading experiments. *Rock Mech Rock Eng*. 2018;51(3):747-764.

64. Yin ZQ, Li XB, Jin JF, He XQ, Du K. Failure characteristics of high stress rock induced by impact disturbance under confining pressure unloading. *Trans Nonferrous Met Soc China*. 2012;22(1):175-184.

65. Rakhimzhanova AK, Thornton C, Fok SC, Michael ZY. 3D DEM simulations of triaxial compression tests of cemented sandstone. *Comput Geotech*. 2019;1:1-17.

66. Grubbs FE. Approximate fiducial bounds on reliability for the two parameter negative exponential distribution. *Technometrics*. 1971;13(4):873-876.
67. Tuckey Z, Stead D. Improvements to field and remote sensing methods for mapping discontinuity persistence and intact rock bridges in rock slopes. *Eng Geol*. 2016;208:136-153.

68. Cohen A. A Padé approximant to the inverse Langevin function. *Rheol Acta*. 1991;30(3):270-273.

69. Johal AS, Dunstan DJ. Energy functions for rubber from microscopic potentials. *J Appl Phys*. 2007;101(8):84917.

70. Jedynak R. New facts concerning the approximation of the inverse Langevin function. *J Non-Newton Fluid Mech*. 2017;249:8-25.

71. Babaei H, Basak A, Levitas VI. Algorithmic aspects and finite element solutions for advanced phase field approach to martensitic phase transformation under large strains. *Comput Mech*. 2019;64(4):1177-1197.

72. Lemaitre J. A continuous damage mechanics model for ductile fracture. *J Eng Mater Technol-Trans ASME*. 1985;107(1):83-89.

73. Zhu ZN, Jiang GS, Tian H, Wu WB. Study on statistical thermal damage constitutive model of rock based on normal distribution. *J Cent South Univ: Nat Sci Ed*. 2019;50(6):1411-1418 (in Chinese with English abstract).

74. Xiao JQ, Ding DX, Jiang FL, Xu G. Fatigue damage variable and evolution of rock subjected to cyclic loading. *Int J Rock Mech Min Sci*. 2010;47:461-468.

75. Lin Y, Gao F, Zhou K, Gao R, Guo H. Mechanical properties and statistical damage constitutive model of rock under a coupled chemical-mechanical condition. *Geofluids*. 2019;2019:17.

76. Zhao GM, Xie LX, Meng XR. A damage-based constitutive model for rock under impacting load. *J China U Min Techno*. 2014;24:505-511.

77. Rossmanith HP. *Fracture and Damage of Concrete and Rock FDCR-2*, 1st ed. Florida: CRC Press; 1993.

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