Uniaxial Compression Experiments And Numerical Simulations of Artificial Rock Samples With Regular Dentate Discontinuity

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Abstract

To study the uniaxial compression performance of artificial rock samples with symmetrical and asymmetrical regular dentate discontinuities, uniaxial compression tests and Particle Flow Code (PFC) numerical simulation are conducted on cement mortar specimens, and the combined effects of dip angle, undulation angle, and number of undulating structures of cracks on the compressive strength, peak strain, elastic modulus, and crack propagation in the specimens are studied. Among these parameters, undulating structure is defined as a single regular dentate structure in the dentate discontinuity; dip angle is the angle between the bottom line of the undulating shape and the horizontal line; undulating angle is the angle between the bottom line of the undulating structure and the left line of the undulating shape; and the number of undulating structures is the number of undulating structures in the dentate discontinuity. The experimental and numerical simulation results show that when the number of the undulating structures and undulating angles remain unchanged, the uniaxial compressive strength of the specimens peak at a dip angle of 90° . In addition, when the dip and undulating angles remain unchanged, the compressive strength, peak strain, and elastic modulus of the specimens decrease with an increase in the number of undulating structures. Moreover, when the number of undulating structures and the dip angle remain unchanged, the compressive strength, peak strain, and elastic modulus of the specimens decrease with an increase in the undulating angle. Further, almost all of the new cracks in the specimens initiate at the tip of the prefabricated cracks.

Introduction

The numerous intricate joints and cracks present in natural rock reduce the mechanical properties of the rock mass, which creates challenges in geotechnical engineering practices.

Scholars worldwide have conducted extensive research on crack initiation, propagation, and coalescence in fractured rock mass under compression. In particular, related studies based on indoor modeling began at the end of the last century. Brace et al. (1963) conducted compression experiments on a single prefabricated rock mass specimen containing inclined fractures. They found that an increase in loading under no or low confining pressure caused micro-cracks to appear at the tip of the prefabricated fracture that gradually bent toward the loading direction along the prefabricated crack and finally approached the principal stress direction. Some scholars also verified the law of wing crack initiation and propagation through theory and experiments and proposed a wing crack model based on their results (Horii et al. 1985, 1986; Ashby et al. 1986). Wong et al. (2009) used uniaxial compression tests to study the propagation of a single crack at different angles and obtained the propagation law of tension cracks and shear cracks caused by compression. Robet et al. (1998) conducted uniaxial and biaxial compression tests to study crack propagation using prefabricated rock mass specimens containing dentate discontinuity. Wong et al. (1998) conducted a uniaxial compression test in sandstone materials containing two prefabricated cracks under different crack dip and bridge angles to obtain the strength law of dentate discontinuity in prefabricated rock mass specimens and the merge mode of three types of the main cracks. Yang et al. (2011) studied the fracture process of a single fracture and two parallel

fractures in sandstone specimens based on the fracture dip angle, length, and rock bridge dip angle. Lee et al. (2011) studied the crack initiation, propagation, and coalescence of three materials containing prefabricated cracks under uniaxial compression and used the two-dimensional Particle Flow Code (PFC2D) program to develop a numerical simulation model shown to be in good agreement with the experimental results. Zhang et al. (2012) studied the strength, crack propagation, and fracture process of single-crack rock materials based on the PFC method using parallel bonding models.

Dentate cracks are common types of fissures that are widely developed in natural slopes of hard rock masses and excavation engineering slopes. Many scholars have conducted experiments and numerical simulations based on the shear mechanical properties of rock masses containing dentate cracks. As early as 1966, Patton (1966) used plaster to simulate the climbing angle effect of dentate discontinuity. Kodikara and Johnston (1994) studied the shear mechanical properties of regular and irregular dentate discontinuities in rock–concrete joints under constant normal stress and constant normal stiffness and found that the brittle damage of regular jagged joints under normal stiffness is stronger than that of irregular jagged joints. Haberfield and Johnson (1994) detailed the failure mechanism of rock mass specimens with dentate discontinuity during the shearing process and found that joint roughness plays a major role in the shear behavior of rock joints. Seidel JP et al. (1995) conducted a direct shear test on a regular dentate discontinuity and subsequent energy analysis of the discontinuity and explained the relationship among the normal strain, tangential strain, and effective normal stress of the dentate discontinuity during the direct shear process. Yang et al. (2000) used a plaster dentate discontinuity model to conduct a shear test and analyzed the combined influence of shear direction and dentate protrusion angle on the shear characteristics of the model. Homand et al. (2001) conducted a cyclic direct shear test on granite with prefabricated dentate discontinuity by analyzing the morphological evolution of the dentate discontinuity area of before and after shearing to quantitatively determine that the damaged area of the discontinuity increased with an increase in normal stress under low normal stress cyclic shear. Park and Song (2009) used the three-dimensional PFC (PFC3D) program to study the shear mechanics characteristics of a rock mass discontinuity affected by the microscopic parameters of the medium including the particle size ratio, particle elastic modulus, and other features. Kwon et al. (2010) used plaster materials with rectangular discontinuity specimens of different undulation heights to conduct shear tests to obtain the relationship between the shear strength of the rough structure of the unit and the shear displacement. Bahaaddini et al. (2013, 2014) used PFC2D to study the shear behavior of rock with dentate discontinuity and analyzed the influence of joint length on the peak shear behavior of the rock mass. The final numerical simulation and physical experiment were in good agreement with consistent shear intensity.

Although the literature includes abundant compression tests and numerical simulation studies on crack propagation in fractured rock masses as well as direct shear tests and numerical simulation on rock masses with dentate discontinuity, few researchers have conducted uniaxial compression tests on rock masses with dentate discontinuity. In this study, indoor uniaxial compression tests and PFC numerical simulations are conducted on mortar specimens with symmetrical and asymmetrical regular dentate discontinuities. The influence of the dip angle, undulation angle, number of undulating structures, and

symmetry of the dentate discontinuity on the compressive strength, peak strain, elastic modulus, and crack propagation in the rock mass is evaluated.

Experimental Design

Whereas conventional test-based designs involve numerous specimens and require pre-cracking, cracks are easily developed and propagated in mortar. Both types of fabrication create specimens that are similar in brittleness, homogeneity, and isotropy; thus, the present study adopted cement mortar to create rock-like specimens.

2.1 Specimen preparation

This study conducted uniaxial compression testing on rock mass specimens containing symmetrical and asymmetrical dentate discontinuities. The specimens were composed of cement mortar with a water-to-cement ratio of 0.65 containing 325-type cement and ISO standard sand with particle sizes of 0.5–1.0 mm. Many of the complete specimens were subjected to uniaxial compression testing, and the relevant mechanical properties of the material were obtained as listed in Table 1.

Table 1

Physical and mechanical parameters of intact rock-like specimens under uniaxial compression

Material	Uniaxial compressive strength σ (MPa)	Elastic Modulus E (GPa)	Density ρ (g cm ⁻³)
Rock-like cement mortar	25.14	1.73	2.05

The dimensions of the specimens used in this study were 100 mm × 100 mm × 100 mm. The mortar was poured into the mold, and after sufficient vibration, the required plastic plates of various widths and 1 mm thickness were inserted into the bottom of the mold. Then, the plastic plates were removed before the mortar initial was set, and the specimens were cured at room temperature for 24 h before demolding. After the production was completed, the specimens were placed in water for 28 days to cure before being dried for testing. Figures 1 and 2 present schematic diagrams of uniaxial compression for the specimens with symmetrical and asymmetrical regular dentate discontinuities, respectively.

In the figures, n and γ are the number of the rock masses with symmetrical and asymmetrical dentate discontinuities and their dip angle, respectively, and β and β_1 are the undulation angle of the rock mass with symmetrical and asymmetrical regular dentate discontinuity, respectively. Among them, the two sides of each undulating structure in the rock mass with symmetrical regular dentate discontinuity were equal, and the vertex angle of the undulating structure with an asymmetrical regular dentate discontinuity was a right angle. The length of the bottom edge of each undulating structure in the rock mass with both regular dentate discontinuities was 20 mm. Figures 1 shows the case in which γ and β are unchanged, and Fig. 2 shows that in which γ and β_1 are unchanged; n assumes 1, 2, 3, and 4 in both cases.

This test was conducted to determine the influence of the combined changes in the dip angle, undulation angle, and number of undulations on the compressive strength, peak strain, elastic modulus, and crack propagation in the specimens.

2.2 Test grouping

In this test, according to the combined changes of n , β , and γ and those of n , β_1 and γ , the compression tests of the specimens with symmetrical and asymmetrical regular dentate discontinuities were grouped, with 144 and 108 cases in total, as shown in Tables 2 and 3, respectively. When β was 45° , the shape of prefabricated fractures in the symmetrical dentate discontinuity rock mass was exactly the same as that in the asymmetrical dentate discontinuity rock mass when β_1 is 45° .

Table 2
Test grouping of specimens with symmetrical regular dentate discontinuity

n	$\beta(^{\circ})$	$\gamma(^{\circ})$
1	30, 45, 60, 75	0, 30, 45, 60, 90, 120, 135, 150, and 180
2	30, 45, 60, 75	0, 30, 45, 60, 90, 120, 135, 150, and 180
3	30, 45, 60, 75	0, 30, 45, 60, 90, 120, 135, 150, and 180
4	30, 45, 60, 75	0, 30, 45, 60, 90, 120, 135, 150, and 180

Table 3
Test grouping of specimens with asymmetrical regular dentate discontinuity

n	$\beta_1(^{\circ})$	$\gamma(^{\circ})$
1	30, 45, 60	0, 30, 45, 60, 90, 120, 135, 150, and 180
2	30, 45, 60	0, 30, 45, 60, 90, 120, 135, 150, and 180
3	30, 45, 60	0, 30, 45, 60, 90, 120, 135, 150, and 180
4	30, 45, 60	0, 30, 45, 60, 90, 120, 135, 150, and 180

2.3 Experiment procedure

This experiment was conducted using a CSS-44300 microcomputer-controlled electronic universal testing machine. During the experiment, the computer screen directly displayed the curves and the measured values of the test force, deformation, and displacement. The maximum axial pressure was 300 kN. The

loading method of this test adopted displacement loading at a rate of 1 mm/min. For each case in the test group, three identical specimens were created; therefore, 648 specimens were fabricated in total.

Test Results

3.1 Analysis of stress–strain curves

Figure 3 shows the uniaxial compression stress–strain curves of the rock mass specimens with dentate discontinuity with various γ values under each selected β and β_1 when $n = 1$.

As shown in the figure, the compression failure processes of the rock masses with symmetrical and asymmetrical dentate discontinuities include stages of slowly developing compaction, stably developing linear elasticity, peak, and rapid stress decrease following the peak. When γ and n of the rock mass with symmetrical regular dentate discontinuity remained unchanged and β increased from 30° to 75° , the peak stress for $\beta = 30^\circ$ was higher than that for $\beta = 75^\circ$. However, when γ and n of that with asymmetrical regular dentate discontinuity remained unchanged and β_1 increased from 30° to 60° , the peak stress for $\beta_1 = 30^\circ$ was close to that for $\beta_1 = 60^\circ$.

3.2 Influence of γ on the compressive strength of the specimens

Figure 4 shows the influence of various γ values on the uniaxial compressive strength of the rock mass specimens with dentate discontinuity when n , β , and β_1 remained the same. The compressive strength assumed the peak point of the stress–strain curves.

Figure 4 (a)–(d) show the uniaxial compressive strength results of the rock mass specimens with symmetrical dentate discontinuity, and (e) and (f) show those for asymmetrical dentate discontinuity. When n , β , and β_1 of the specimens remained unchanged, the compressive strength of the rock mass reached the maximum, at $\gamma = 90^\circ$. The uniaxial compressive strength of the rock mass with dentate discontinuity increased (decreased) with $\gamma = 0^\circ - 90^\circ$ ($\gamma = 90^\circ - 180^\circ$). When γ increased from 0° to 180° , the peak stress showed a symmetrical phenomenon, with $\gamma = 90^\circ$ as the axis of symmetry.

3.3 Influence of β on the compressive strength, peak strain, and elastic modulus of the specimens

Figure 5 shows the influence of various β values on the uniaxial compressive strength of the rock mass specimens with dentate discontinuity when n and γ remained unchanged.

As shown in the figure, when γ and n of the specimens remained unchanged, the uniaxial compressive strength of the rock mass specimens with dentate discontinuity decreased gradually as β increased from 30° to 75° .

Figure 6 shows the influence of various β values under each selected γ and n on the peak strain of the rock mass specimens with dentate discontinuity. The peak strain assumed the strain corresponding to the peak stress in the stress–strain curves of uniaxial compression.

The figure shows that when γ and n of the specimens remained unchanged, the peak strain of the rock mass specimens with dentate discontinuity decreased gradually as β increased from 30° to 75° .

Figure 7 shows the influence of various β values under each selected γ and n on the elastic modulus of the rock mass specimens with dentate discontinuity. The elastic modulus is the ratio of stress to strain occurring at 50% of the peak stress of the specimens.

The figure shows that when γ and n of the specimens remained unchanged, the elastic modulus of the rock mass specimens with dentate discontinuity decreased gradually as β increased from 30° to 75° .

3.4 Influence of n on the compressive strength, peak strain, and elastic modulus of the specimens

Figure 8 shows the influence of n on the uniaxial compressive strength of the rock mass specimens with dentate discontinuity when γ , β , and β_1 remained the same.

Figure 8 (a)–(d) show the uniaxial compressive strength test results of the specimens with symmetrical dentate discontinuity; (e) and (f) show those for asymmetrical dentate discontinuity. The uniaxial compressive strength of the specimens with dentate discontinuity gradually decreased with an increase in n from 1 to 4 when γ , β , and β_1 of the specimens remained unchanged.

Figure 9 shows the influence of n on the peak strain of the rock mass specimens with dentate discontinuity when γ , β , and β_1 remained unchanged.

Figure 9 (a)–(d) show the peak strain test results of the specimens with symmetrical dentate discontinuity; (e) and (f) show those for asymmetrical dentate discontinuity. The peak strain of the specimens with dentate discontinuity gradually decreased with an increase n from 1 to 4 when γ , β , and β_1 of the specimens remained unchanged.

Figure 10 shows the influence of n on the elastic modulus of the rock mass specimens with dentate discontinuity when γ , β , and β_1 remained unchanged.

Figure 10 (a)–(d) show the elastic modulus test results of the specimens with symmetrical dentate discontinuity; (e) and (f) show those for asymmetrical dentate discontinuity. The elastic modulus of the specimens with dentate discontinuity gradually decreased with an increase of n from 1 to 4 when γ , β , and β_1 remained unchanged.

Uniaxial Compression Experiment And Numerical Simulation Of The Rock Mass With Symmetrical And Asymmetrical Joint Fractures

4.1 Numerical simulation

In the PFC program, the particles are round, rigid bodies of particular masses. In the simulation process, the particles are not limited by deformation variables and can effectively reflect the mechanism, process, and results of material damage. Therefore, this method has been widely used by researchers to compensate for mechanical problems associated with discontinuous phenomena such as rock bursting.

In this study, PFC2D was used to conduct numerical simulation of uniaxial compression tests on rock samples with symmetrical and asymmetrical dentate discontinuities, and a square model with dimensions of 100 mm × 100 mm was established that included 6468 circular particles of various sizes. The PFC materials are composed of rigid balls or clusters accumulating at the contact point; the contact between particles adopts a parallel bonded model. Before the simulation, the meso-parameters of the complete rock mass test block were calibrated, as listed in Table 4.

Table 4
Partial parameters of PFC material

Minimum particle radius (mm)	Maximum particle radius (mm)	Density (g m^{-3})	Effective modulus (GPa)	Stiffness ratio	Coefficient of friction
1.0	1.66	1.96	1.4	1.0	0.5

The microscopic parameters of the modified particles were written into the 2D compression test model, and numerical simulation was then conducted through compression testing of the 2D parallel bonded model. The experimental and numerical results of the uniaxial compressive strength of the rock mass specimens with symmetrical dentate discontinuity with various β values when $n = 1$ and $\gamma = 0^\circ$ are shown in Fig. 11.

Figure 11 (a)–(d) show the results of the uniaxial compressive strength experiment and numerical simulation of the rock mass specimens with symmetrical dentate discontinuity when the β values were 30° , 45° , 60° , and 75° , respectively. When n and γ remained unchanged, the numerical strength of the rock mass specimens with different β values was about 80% of the experimental strength.

Figure 12 shows the experimental and numerical results of failure characteristics of the rock mass specimens with dentate discontinuity when $n = 1$, $\gamma = 60^\circ$, and $\beta = 75^\circ$. The results were essentially consistent in terms of crack initiation, crack propagation, and crack coalescence.

In summary, the test results of the uniaxial compressive strength of the rock mass specimen were consistent with the change trend of the numerical results, as were the failure characteristics. Therefore, numerical simulation is essential for uniaxial compression specimens with dentate discontinuity. According to the test grouping, $\beta = 15^\circ$ was added to the compression experimental grouping of the original regular dentate discontinuity as its numerical simulation grouping, and $\beta_1 = 15^\circ$ and $\beta_1 = 75^\circ$ were added to that of the original irregular dentate discontinuity as the numerical simulation grouping.

4.2 Influence of γ on the compressive strength of the specimen numerical simulation

Figure 13 shows the influence of various γ values on the numerical simulation results of the uniaxial compressive strength of the rock mass specimens with dentate discontinuity when n , β , and β_1 remained unchanged.

Figure 13 (a)–(e) show the numerical results of the uniaxial compressive strength of the rock mass specimens with symmetrical dentate discontinuity; (f)–(i) show those for asymmetrical dentate discontinuity. When n , β , and β_1 remained unchanged, the compressive strength of the irregular dentate discontinuity rock mass specimens was maximized at $\gamma = 90^\circ$. The uniaxial compressive strength of the specimens assumed $\gamma = 90^\circ$ as the symmetry axis and increased (decreased) with an increase in γ from 0° to 90° (90° to 180°). These results are consistent with the experimental results.

4.3 Effect of β on the numerical simulation results

Figure 14 shows the influence of various β and β_1 values on the numerical simulation results of the uniaxial compressive strength of the rock mass specimens with dentate discontinuity when n and γ remained unchanged.

Figure 14 (a)–(d) show the numerical results of the uniaxial compressive strength of the rock mass specimens with symmetrical dentate discontinuity; (e)–(h) show those for asymmetrical dentate discontinuity. When γ and n remained unchanged, the simulated strength of the rock mass specimens with symmetrical dentate discontinuity decreased gradually as β increases from 15° to 75° , which is consistent with the experimental results of those with symmetrical dentate discontinuity. When γ and n of rock mass with asymmetrical dentate discontinuity remained unchanged, the simulated strength at $\beta_1 = 15^\circ$ was close to that at $\beta_1 = 75^\circ$, and the simulated strength at $\beta_1 = 30^\circ$ was close to that at $\beta_1 = 60^\circ$. However, the simulated strength of the rock mass with asymmetrical dentate discontinuity showed no obvious change when β_1 increased from 15° to 75° .

Figure 15 shows the influence of various β and β_1 values on the numerical simulation results of the peak strain of the rock mass specimens with dentate discontinuity when n and γ remained unchanged.

Figure 15 (a)–(d) show the numerical results of the peak strain of the rock mass specimens with symmetrical dentate discontinuity; (e)–(h) show those for asymmetrical dentate discontinuity. When γ and n remained unchanged, the simulated peak strain of the rock mass specimens with dentate discontinuity decreased gradually as β increased from 15° to 75° , which is consistent with the experimental results of those with symmetrical dentate discontinuity. When γ and n of the rock mass with asymmetrical dentate discontinuity remained unchanged, the simulated peak strain at $\beta_1 = 15^\circ$ was close to that at $\beta_1 = 75^\circ$, and the simulated peak strain at $\beta_1 = 30^\circ$ was close to that at $\beta_1 = 60^\circ$. However, the

simulated peak strain of the rock mass with asymmetrical dentate discontinuity showed no obvious change when β_1 increased from 15° to 75° .

Figure 16 shows the influence of various β and β_1 values on the numerical simulation results of the elastic modulus of rock mass specimens with dentate discontinuity when n and γ remained unchanged.

Figure 16 (a)–(d) show the numerical results of the elastic modulus of the rock mass specimens with symmetrical dentate discontinuity; (e)–(h) show those for asymmetrical dentate discontinuity. When γ and n remained unchanged, the simulated elastic modulus of the rock mass specimens with dentate discontinuity decreased gradually as β increases from 15° to 75° , which is consistent with the experimental results of rock mass specimens with symmetrical dentate discontinuity. When γ and n in the rock mass with asymmetrical dentate discontinuity remained unchanged, the simulated elastic modulus at $\beta_1 = 15^\circ$ (30°) was close to that at $\beta_1 = 75^\circ$ (60°). However, the simulated elastic modulus of rock mass with asymmetrical dentate discontinuity showed no obvious change when β_1 increased from 15° to 75° .

4.4 Effect of n on the numerical simulation results

Figure 17 shows the influence of various n values on the numerical simulation results of the uniaxial compressive strength of the rock mass specimens with dentate discontinuity when γ , β , and β_1 remained the same.

Figure 17 (a)–(e) show the numerical results of the uniaxial compressive strength of the rock mass specimens with symmetrical dentate discontinuity; (f)–(i) show those for asymmetrical dentate discontinuity. When γ , β , and β_1 remained unchanged, the simulated strength of the rock mass specimens with dentate discontinuity decreased gradually as n increased from 1 to 4, which is consistent with the experimental results.

Figure 18 shows the influence of various n values on the numerical simulation results of the peak strain of the rock mass specimens with dentate discontinuity when γ , β , and β_1 remained the same.

Figure 18 (a)–(e) show the numerical results of peak strain of the rock mass specimens with symmetrical dentate discontinuity; (f)–(i) show those for asymmetrical dentate discontinuity. When γ , β , and β_1 remained unchanged, the simulated peak strain of the rock mass specimens with dentate discontinuity decreased gradually as n increased from 1 to 4, which is consistent with the experimental results.

Figure 19 shows the influence of various n values on the numerical simulation results of the elastic modulus of the rock mass specimens with dentate discontinuity when γ , β , and β_1 remained the same.

Figure 19 (a)–(e) show the numerical results of the elastic modulus of the rock mass specimens with symmetrical dentate discontinuity; (f)–(i) show those for asymmetrical dentate discontinuity. When γ , β and β_1 remained unchanged, the simulated elastic modulus of the rock mass specimens with dentate

discontinuity decreased gradually as n increased from 1 to 4, which is consistent with the experimental results.

4.5 Comparative analysis of failure characteristics of test and numerical simulation

To obtain more complete uniaxial compression stress–strain curves, the maximum load was decreased 80% at the end of the loading process in this study compared with that used in the PFC numerical simulation; thus, the failure characteristics in the test were more obvious than those in the simulation. In both the experiments and the numerical simulation, almost all new cracks were initiated at the tip of the prefabricated crack, with a few initiated at the middle of or far from the prefabricated crack. The types of crack propagation include tensile, shear, and tension–shear composite cracking.

Figure 20 shows the influence of various γ values on the failure characteristics of the specimens in the experiments and numerical simulations when $n = 2$ and $\beta = 75^\circ$.

Figure 20 (a)–(i) show failure characteristics of the experiment specimen, and (a₁)–(i₁) show those observed in numerical simulation. When γ was 0° , 30° , 150° , and 180° , most of the new cracks originated at the tip of the prefabricated crack and propagated at an acute angle from the direction of the hypotenuse of the prefabricated crack. When γ was 90° , most of the new cracks originated at the middle of the prefabricated crack and propagated at a right angle from the direction of the hypotenuse of the prefabricated crack. When γ was 45° , 60° , 120° , and 135° , most of the new cracks began at the tip of the prefabricated crack and propagated a right angle from the direction of the hypotenuse of the prefabricated crack. The failure of the rock mass specimens was mainly tensile crack type, and the coalescence direction of cracks eventually tended to follow the axial direction. When γ was 30° , 45° , 60° , 90° , 120° , 135° , and 150° , several new cracks were initiated at a distance from the prefabricated crack.

Figure 21 shows the influence of various β and β_1 values on the failure characteristics of the specimens in the experiments and numerical simulations when $n = 4$ and $\gamma = 180^\circ$.

Figure 21 (a)–(f) show the failure characteristics in the experiment specimens, and (a₁)–(f₁) show those of the numerical simulation. When β was 30° , 45° , and 60° and β_1 was 30° and 60° , the new cracks initiated mainly at the tip of the prefabricated crack. The failure of rock mass specimens was dominated by tensile cracks; the new cracks were distributed evenly and symmetrically. A few shear cracks formed at the prefabricated crack tip owing to the stress concentration at occurring at both ends of the prefabricated crack. When $\beta = 75^\circ$, the failure form of the specimens was relatively simple, with only a small number of cracks initiating at the tip of the prefabricated crack.

Figure 22 shows the influence of various n values on the failure characteristics of the specimens in the experiments and numerical simulations when $\beta = 75^\circ$ and $\gamma = 60^\circ$.

Figure 22 (a)–(d) show the failure characteristics of the experiment specimens, and (a₁)–(d₁) show those of the numerical simulation. When n was 1, 2, and 3, the tensile cracks were initiated mainly at the tip of the prefabricated crack, and the shear cracks began at the edges of the specimens far from the prefabricated crack. The new cracks propagated at a right angle from the direction of the hypotenuse of the prefabricated crack. When $n = 4$, the coalescence of the new cracks was not obvious.

Conclusion

- (1) When the number of the undulating structures and the undulating angles remain unchanged, the uniaxial compressive strength of the rock mass specimens with symmetrical and asymmetrical regular dentate discontinuities peaks at a dip angle of 90° . As the dip angle of the rock mass specimens increases from 0° to 180° , their uniaxial compressive strength first increases and then decreases, and the uniaxial compressive strength is distributed along an axis of symmetry at 90° .
- (2) When the dip angle and undulating angle remain unchanged, the compressive strength, peak strain, and elastic modulus of the rock mass specimens with symmetrical and asymmetrical dentate discontinuities all decrease with an increase in the number of undulating structures.
- (3) When the number and dip angle of undulating structures are known, the compressive strength, peak stress, and elastic modulus of the rock mass specimens containing symmetrical dentate discontinuity decrease with an increase in the fluctuation angle. The compressive strength of the rock mass with specimens with asymmetrical dentate discontinuity depends more on the smaller angle of 90° for β_1 and β_2 than that with symmetrical dentate discontinuity with the same undulating angle.
- (4) Almost all of the new cracks in the rock mass specimens with symmetrical and asymmetrical regular dentate discontinuities are initiated at the tip of the prefabricated crack, with a few beginning at the middle of or far from the prefabricated crack. The failure characteristics of the rock mass are associated mainly with tensile cracks accompanied by a few shear cracks.
- (5) When the undulation angle and the number of undulating structures remain unchanged and the dip angle is 45° – 135° , the crack in the rock mass with symmetrical dentate discontinuity propagates first at a right angle from the direction of the hypotenuse of the preformed crack and finally propagates axially. When the dip angle and the number of undulating structures remains unchanged and the crack propagation of the rock mass is rich when the undulating angle is less than or equal to 60° , the undulation angle and dip angle remain unchanged, and the number of undulating structures is less than four, the crack propagation is abundant, and shear cracks are initiated at the edge of the specimens far from the prefabricated crack.

Declarations

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Figures

Figure 1

Schematic diagram of uniaxial compression for specimens with symmetrical regular dentate discontinuity (unit: nm).

Figure 3

Influence of γ on the stress–strain curves of the specimens for each selected β and β_1 when $n = 1$

Figure 4

Influence of various γ values on the uniaxial compressive strength of the specimens for each selected n , β , and β_1

Figure 5

Influence of various β values on the uniaxial compressive strength of the specimens for selected γ and n

Figure 6

Influence of β values on the peak strain of the specimens for selected γ and n

(a) $n = 1$

(b) $n = 2$

(c) $n = 3$

(d) $n = 4$

Figure 7

Influence of β values on the elastic modulus of the specimens for selected γ and n

Figure 8

Influence of n on the uniaxial compressive strength of the specimens for selected γ , β , and β_1

Figure 9

Influence of n on the peak strain of the specimens for selected γ , β , and β_1

Figure 10

Influence of n on the elastic modulus of the specimens for selected γ , β , and β_I

(a) $\beta = 30^\circ$

(b) $\beta = 45^\circ$

(c) $\beta = 60^\circ$

(d) $\beta = 75^\circ$

Figure 11

Comparison of experimental and numerical simulation of the uniaxial compressive strength of the rock mass specimens with symmetrical dentate discontinuity

Figure 12

Comparison of experiment results and numerical simulation of the failure characteristics of the rock mass specimens with symmetrical dentate discontinuity

Figure 13

Influence of various γ values on the numerical simulation results of the uniaxial compressive strength of the rock mass specimens for each selected n , β , and β_1 .

Figure 14

Influence of various β and β_1 values on the numerical simulation results of the uniaxial compressive strength of the rock mass specimens for selected n and γ

Figure 15

Influence of various β and β_1 values on the numerical simulation results of the peak strain of the rock mass specimens for selected n and γ

Figure 16

Influence of various β and β_1 values on the numerical simulation results of the elastic modulus of the rock mass specimens for selected n and γ

Figure 17

Influence of various n values on the numerical simulation results of the uniaxial compressive strength of the rock mass specimens for selected γ , β , and β_1

Figure 18

Influence of various n values on the numerical simulation results of the peak strain of the rock mass specimens for selected γ , β , and β_1

Figure 19

Influence of various n values on the numerical simulation results of the elastic modulus of the rock mass specimens for selected γ , β , and β_1

Figure 20

Influence of various γ values on the failure characteristics of the rock mass specimens in the experiments and numerical simulations when $n = 4$ and $\beta = 75^\circ$. In the panels, T and S represent tensile and shear cracking,

Figure 21

Influence of various β and β_1 values on the failure characteristic of the rock mass specimens in the experiments and numerical simulations when $n = 4$ and $\gamma = 180^\circ$

Figure 22

Influence of various n values on the failure characteristic of the rock mass specimens in the experiments and numerical simulations when $\beta = 75^\circ$ and $\gamma = 60^\circ$.