

## Article

# Development of Similarity Materials for Jointed Rock Masses in Underground Cavern Model Tests

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## Abstract

Utilizing appropriate similar materials in physical model testing is critical to ensuring an accurate simulation of the prototype mechanical behavior. For example, the physical modeling of jointed and fractured rock masses requires replicating their nonlinear mechanical characteristics. Therefore, this study develops mix proportions for a rock-like material similar to that encountered in the Xiangjiaba Hydropower Project. Based on similarity ratio analysis and mechanical testing, a jointed and fractured rock mass material was developed using a modified thin-sheet stripping method, and its nonlinear behavior was validated through mechanical experiments, numerical simulations, and distortion energy theory. The testing results of the developed similar material are as follows: (1) In specimens with single-jointed and fractured rock mass, the peak uniaxial compressive strength varies according to the joint orientation in the following order:  $90^\circ > 60^\circ > 45^\circ > 0^\circ > 30^\circ$ . In specimens with intersecting joints, the overall strength is primarily controlled by the orientation of the main joint, while the secondary joint plays a lesser role in further reducing the strength. (2) The analysis of distortion energy for both single-jointed and cross-jointed and fractured rock masses indicates that the crack initiation angle generally decreases with increasing joint inclination. Furthermore, at the onset of failure, the crack initiation angle in the lower part of the specimen is consistently larger than that in the upper part. The experimentally observed crack angles are in good agreement with the theoretical predictions. (3) A numerical model of the jointed and fractured rock mass was developed using the PFC2D version 5.0 discrete element software. Comparing the results of the numerical simulations and mechanical tests revealed consistent failure patterns: both exhibit the typical shear-tensile composite failure modes characteristic of jointed and fractured rock masses. These findings confirm that the developed jointed and fractured rock mass similar materials realistically capture the nonlinear mechanical behavior of fractured surrounding rock, providing a reliable material basis for the physical model testing of underground caverns.

**Keywords:** jointed and fractured rock mass similar materials; physical model test; orthogonal experimental design; discrete element numerical simulation



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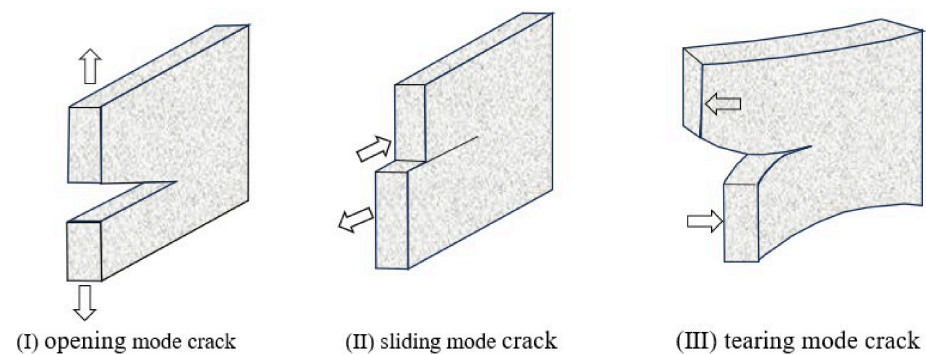
## 1. Introduction

Underground caverns are embedded in rock masses, and their stability is governed by the geological conditions of the surrounding rock. When a cavern intersects faults, fractured zones, or jointed strata, it becomes more susceptible to seismic damage. During the 2008 Wenchuan earthquake, for example, extensive cracking and spalling were observed in the underground caverns of the Yingxiuwan Hydropower Station [1], which were subsequently confirmed to be directly associated with the discontinuous deformation of joint surfaces in the surrounding rock mass [2–4]. Shaking-table model testing is an effective approach for investigating such problems, as it can explicitly capture the dynamic response characteristics of underground structures under earthquake loading. However, owing to constraints in table size, payload capacity, and experimental cost, physical model tests must adopt scaled prototypes in accordance with similarity theory. For underground caverns in jointed and fractured rock masses, this scaling process must satisfy not only geometric similarity and the similarity of key mechanical parameters, but also the ability to reproduce the discontinuous mechanical behavior inherent to jointed media. Consequently, developing suitable similitude materials capable of replicating the nonlinear mechanical response of jointed and fractured rock masses is a prerequisite for conducting reliable shaking-table tests of underground caverns.

In geomechanical model tests of underground engineering, the development of similitude materials for the surrounding rock is a fundamental issue that directly determines the reliability of experimental results. Existing studies on this topic mainly focus on three types of geological media: intact rock masses, fault fracture zones, and jointed and fractured rock masses. Among these, research on similitude materials for intact surrounding rock is the most mature [5–8]. Wang et al. [9], Zhang et al. [10], Guan et al. [11], Li et al. [12], Tang et al. [13], Zhang et al. [14,15], Gao et al. [16], Shen et al. [17], and Liu et al. [18,19] employed different material systems—such as fly ash, river sand, barite powder, quartz sand, and gypsum—to simulate surrounding rock and applied them in model tests of tunnels and underground caverns. These studies established systematic methods for determining material mix proportions; however, the simulated media were essentially treated as continuous materials. Although fault fracture zones belong to discontinuous media, most existing studies have adopted simplified approaches to represent them. In many cases, fracture zones are treated as homogeneous materials with reduced strength and stiffness, typically achieved by mixing sawdust into the surrounding rock materials to lower their mechanical parameters. This method has been adopted by Wang et al. [9], Zhang et al. [10], Li et al. [12], and Tang et al. [13], among others. While such studies provide practical experience in preparing fracture-zone materials, a simplified treatment cannot adequately capture discontinuous deformation behaviors such as sliding and opening along structural planes. Some researchers have attempted to simulate jointed and fractured rock masses by introducing artificial discontinuities into rock-like materials. Li et al. [20], for example, incorporated mica flakes to represent joint planes. Che et al. [21] and Fu et al. [22] assembled prefabricated rock blocks and used polytetrafluoroethylene (PTFE) tape to simulate joint interfaces. These studies successfully reproduced the geometric characteristics of jointed and fractured rock masses and extended the application of similitude materials from continuous to discontinuous media. However, existing studies on jointed and fractured rock mass simulation still present two main limitations. First, most applications have focused on slope engineering, whose failure mechanisms differ fundamentally from those of underground caverns. Second, the depth of verification remains insufficient. Most studies have primarily emphasized geometric representation, without systematically validating whether the developed materials can reproduce the nonlinear mechanical behavior of jointed and fractured rock masses, such as the variation of strength with joint inclination

and the inclination-dependent failure modes. These nonlinear characteristics are precisely the key factors controlling the stability of underground caverns under seismic loading. In summary, existing research indicates that similitude materials developed for intact rock masses cannot represent discontinuous media, simplified treatments of fault fracture zones fail to capture structural-plane-controlled behavior, and, although preliminary attempts have been made to simulate jointed and fractured rock masses, they remain limited by inconsistent application scenarios and insufficient validation.

Rock masses commonly contain numerous joints and fractures. The presence of these discontinuities divides the rock mass into blocky structures and significantly reduces its overall integrity. During seismic events, the relatively weak bonding along joint planes allows adjacent rock blocks to undergo relative movement. Stress concentration may develop near the tips of these discontinuities, causing the joints to experience opening, sliding, or tearing deformation modes (Figure 1). These deformation processes significantly alter the dynamic response of the rock mass and consequently affect the stability of underground caverns. Therefore, in shaking-table model tests of underground caverns in jointed and fractured rock masses, it is essential to employ similitude materials that can adequately reproduce the nonlinear mechanical behavior associated with jointed media. Only by accounting for these characteristics can the dynamic response of underground caverns under seismic loading be more accurately revealed.



**Figure 1.** Failure modes of jointed and fractured rock masses.

Based on previous studies of jointed rock masses, rock strength exhibits a nonlinear relationship with joint inclination [23]. This characteristic is a key criterion for evaluating whether similitude materials can realistically reproduce the mechanical behavior of jointed and fractured rock masses. However, traditional dimensional analysis has certain limitations when applied to similitude materials containing pre-existing joints. Relying solely on similarity theory cannot guarantee that the developed materials will reproduce such nonlinear behavior. Therefore, beyond the design of similitude materials based on similarity principles, it is necessary to verify their mechanical behavior through multiple approaches. In this study, a systematic investigation of similitude materials for jointed and fractured rock masses was conducted. The mix proportions of the matrix material were determined through orthogonal experiments, and discontinuous rock mass models containing joints were fabricated using the thin-sheet stripping method. Unlike previous studies that primarily focused on geometric representation without any systematic validation of nonlinear behavior, the nonlinear failure behavior of the developed materials—including the variation of strength with joint inclination, failure modes, and crack propagation characteristics—was comprehensively validated through laboratory tests, numerical simulations, and theoretical analysis based on fracture mechanics. The results provide a reliable basis for selecting suitable similitude materials for shaking-table model tests of underground caverns in jointed and fractured rock masses.

## 2. Development of Similarity Materials for Jointed and Fractured Rock Masses

To develop similar materials that accurately replicate the nonlinear behavior of jointed and fractured rock masses, it is essential to fabricate models that conform to similarity principles. Additionally, the mechanical behavior and failure patterns of these materials must closely resemble those of the prototype.

### 2.1. Similarity Ratio Design and Material Selection

This section presents the theoretical basis for scaling the prototype to the model, followed by the selection of raw materials for preparing the jointed and fractured rock mass similar material. The similarity ratios are first derived from classical similitude theory, and then the aggregates and binders are chosen based on the target mechanical parameters of the prototype.

#### 2.1.1. Similarity Ratio Design

Similarity conditions form the theoretical basis for physical model testing. When these conditions are satisfied, the physical phenomena observed in the model tests are similar to those of the prototype behavior. The ratio of corresponding physical quantities between the prototype ( $i_p$ ) and the model ( $i_m$ ) is defined as the similarity ratio, expressed as [24]

$$Ci = i_p / i_m \quad (1)$$

Based on the equilibrium equations, geometric relationships, physical laws, and boundary conditions of both the prototype and model, the similarity criteria for the physical model test can be derived as follows.

The relationship among the similarity ratios of stress  $C_\sigma$ , density  $C_\rho$ , and geometry  $C_L$  is given by

$$C_\sigma = C_\rho C_L \quad (2)$$

The similarity ratios of displacement  $C_\delta$ , geometry  $C_L$ , and strain  $C_\epsilon$  satisfy

$$C_\delta = C_L C_\epsilon \quad (3)$$

The similarity ratios of stress  $C_\sigma$ , elastic modulus  $C_E$ , and strain  $C_\epsilon$  relate as

$$C_\sigma = C_E C_\epsilon \quad (4)$$

For geomechanical model tests, all dimensionless physical quantities—such as strain  $\epsilon$ , internal friction angle,  $\varphi$ , and Poisson's ratio,  $\nu$ —must have similarity ratios equal to one, i.e.,

$$C_e = C_\varphi = C_\nu \quad (5)$$

It should be noted that the assumption of  $C_\phi = C_\nu = 1$  in classical similarity theory represents an idealization that is difficult to fully achieve in scaled jointed systems. This study builds upon the existing foundation of similarity materials for continuous media and extends it to the development of similarity materials for discontinuous media. The primary objective is to provide a similarity material for shaking-table model tests of underground caverns that can reproduce the macroscopic nonlinear mechanical behavior of jointed rock masses—such as strength anisotropy and inclination-dependent failure modes—rather than to precisely simulate the meso-scale frictional characteristics of joint interfaces. Under a geometric similarity ratio of 1:100, meso-scale features such as joint surface roughness cannot be accurately reproduced in the physical model. Therefore, retaining the assumption

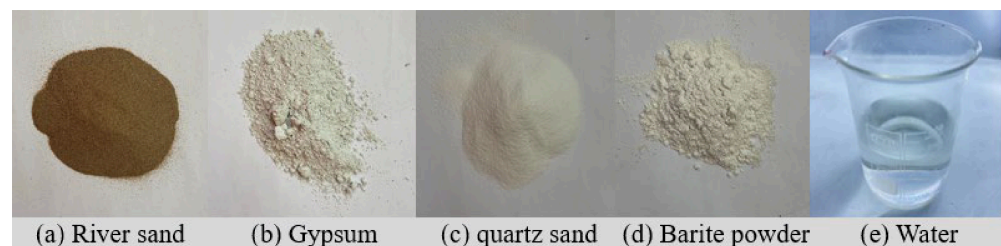
of  $C_\phi = C_v = 1$  in the similarity design, while focusing on the verification of macroscopic mechanical responses (strength, modulus, and failure modes) in the experiments, is a pragmatic approach consistent with common practice in large-scale geomechanical model testing. The physical and mechanical parameters of the prototype and model materials are summarized in Table 1.

**Table 1.** Physical and mechanical parameters of prototype and model materials.

| Material  | Density/(g/cm <sup>3</sup> ) | Compressive Strength/MPa | Elastic Modulus/GPa | Poisson's Ratio | Cohesion/MPa | Internal Friction Angle/° |
|-----------|------------------------------|--------------------------|---------------------|-----------------|--------------|---------------------------|
| Prototype | 2.65                         | 80.00                    | 24.00               | 0.23            | 1.5~2.1      | 50~60                     |
| Model     | 1.77                         | 0.53                     | 0.16                | 0.23            | 0.01~0.014   | 50~60                     |

### 2.1.2. Selection of Similar Materials

Several studies investigated similar materials for rock media in physical model tests. Commonly used material aggregates' combinations include (1) fly ash, river sand, engine oil, and diatomaceous earth; (2) silty clay, fine sand, bentonite, and barite powder; (3) barite powder, iron ore powder, quartz sand, gypsum, and glycerin; (4) iron ore powder, barite powder, and quartz sand. Meanwhile, rosin, alcohol solution, and gypsum are utilized as binders. In this study, the selection and optimization of similar material mixes were based on the established similarity ratios and the actual geological conditions of the project. River sand, barite powder, and quartz sand were chosen as aggregates, while gypsum and water were used as binders to simulate the mechanical behavior of rock. The raw materials used are shown in Figure 2.



**Figure 2.** Raw materials used for similar material preparation.

First, orthogonal tests and range analysis were employed to determine the optimal mix proportions and mechanical properties of the rock-like material. On this basis, jointed and fractured rock mass specimens were fabricated. Mechanical testing, numerical simulation, and theoretical analysis were then performed to verify the mechanical parameters and failure mechanisms of the jointed and fractured rock mass samples. Based on the findings from these tests and analyses, a jointed and fractured rock mass similar material was developed that accurately captures the nonlinear failure behavior observed in the prototype.

## 2.2. Similar Material Mix Design and Mechanical Property Testing for Rock Media

This subsection describes the experimental procedures used to determine the optimal mix proportions for the rock-like material. An orthogonal experimental design is employed to evaluate the influence of key factors (aggregate ratio, gypsum content, and water content) on the density, compressive strength, and elastic modulus of the specimens. The sensitivity analysis and final mix proportion selection are then discussed.

### 2.2.1. Orthogonal Experimental Design

An orthogonal experimental design based on uniaxial compression tests was employed to determine the optimal mix proportions for the jointed and fractured rock mass similar

material. A three-level, three-factor design was adopted. The selected factors are the mass ratio of aggregate components (denoted herein as Factor A); the mass ratio of gypsum to aggregate (denoted as Factor B); and the mass ratio of water to aggregate (denoted as Factor C). The selection of these factors and their corresponding ranges was based on the physical functions of each raw material. Barite powder was chosen for its high specific gravity to regulate density; gypsum serves as the binder to control strength and stiffness; and water is added to ensure adequate hydration. The factor ranges were determined through preliminary trials to ensure that the resulting material properties could cover the target prototype values. The orthogonal experimental design scheme is presented in Table 2.

**Table 2.** Factors and levels for the orthogonal experimental design of similar materials.

| Factor & Level | Aggregate Mass Ratio | Gypsum-to-Aggregate Mass Ratio | Water-to-Aggregate Mass Ratio |
|----------------|----------------------|--------------------------------|-------------------------------|
| Level 1        | 1:1.5:0.38           | 0.12                           | 0.1                           |
| Level 2        | 1:1.8:0.45           | 0.20                           | 0.15                          |
| Level 3        | 1:2.1:0.6            | 0.30                           | 0.20                          |

During the preliminary stage of the orthogonal experiments, specimens were prepared for uniaxial compression tests to determine the fundamental mechanical properties of similar materials, including apparent density, uniaxial compressive strength, and elastic modulus. The specimens were fabricated in strict accordance with standard procedures, using cylindrical samples with a diameter of 50 mm and a height of 100 mm. An orthogonal test design was employed to arrange the experimental scheme, involving nine different mix ratio combinations. To ensure the reliability of the test data, three specimens were prepared for each mix ratio, resulting in a total of 27 uniaxial compression specimens. For data processing, the average value of the three parallel tests was taken as the representative result for each mix ratio group, thereby minimizing the influence of random experimental errors. The orthogonal test scheme for the similar material mix ratios is shown in Table 3.

**Table 3.** Orthogonal test scheme for similar rock material mix ratios.

| Test Group No. | Material Ratio |               |             |        |       |
|----------------|----------------|---------------|-------------|--------|-------|
|                | River Sand     | Barite Powder | Quartz Sand | Gypsum | Water |
| 1              | 1              | 1.5           | 0.38        | 0.35   | 0.29  |
| 2              | 1              | 1.5           | 0.38        | 0.58   | 0.43  |
| 3              | 1              | 1.5           | 0.38        | 0.86   | 0.58  |
| 4              | 1              | 1.8           | 0.45        | 0.65   | 0.66  |
| 5              | 1              | 1.8           | 0.45        | 0.39   | 0.49  |
| 6              | 1              | 1.8           | 0.45        | 0.98   | 0.33  |
| 7              | 1              | 2.1           | 0.6         | 0.74   | 0.37  |
| 8              | 1              | 2.1           | 0.6         | 1.11   | 0.56  |
| 9              | 1              | 2.1           | 0.6         | 0.44   | 0.74  |

The key parameters to be measured for similar rock materials include density, uniaxial compressive strength, and elastic modulus. Density is determined using a high-precision electronic balance, while uniaxial compressive strength and elastic modulus are obtained through uniaxial compression tests. The experimental setup used for these tests is shown in Figure 3.

The development of rock mass analog materials was conducted according to the orthogonal test design. Samples were prepared for nine different mix ratios, followed by weighing, dimensional measurement, and uniaxial compression testing. The obtained data included sample mass, dimensions, density, uniaxial compressive strength, elastic modulus and Poisson's ratio. Based on these test results, the key parameters—density, uniaxial

compressive strength, and elastic modulus—of the analog material samples were analyzed and are presented in Table 4.



**Figure 3.** Experimental setup and testing process.

**Table 4.** Mechanical properties' test results for rock mass analog materials.

| Test Group No. | Density/(g/cm <sup>3</sup> ) | Compressive Strength/MPa | Elastic Modulus/GPa |
|----------------|------------------------------|--------------------------|---------------------|
| 1              | 1.53                         | 0.21                     | 0.06                |
| 2              | 1.61                         | 0.64                     | 0.18                |
| 3              | 1.71                         | 0.85                     | 0.28                |
| 4              | 1.72                         | 0.39                     | 0.16                |
| 5              | 1.78                         | 0.62                     | 0.14                |
| 6              | 1.68                         | 0.51                     | 0.15                |
| 7              | 1.72                         | 0.28                     | 0.09                |
| 8              | 1.78                         | 0.56                     | 0.14                |
| 9              | 1.76                         | 0.31                     | 0.12                |

### 2.2.2. Sensitivity Analysis

Range analysis is commonly used for evaluating the sensitivity of influencing factors in orthogonal design experiments owing to its simplicity [25–28], intuitive clarity, and ease of calculation. By applying range analysis, the primary and secondary factors affecting the mechanical property test results can be identified as follows:

$$R_j = \max\{k_{ij}\} - \min\{k_{ij}\} \quad (6)$$

A larger range value  $R_j$  indicates a greater influence of the corresponding factor on the measured results. Therefore, the relative importance of each factor can be determined by comparing the magnitude of their ranges. In this study, three factors are considered, i.e., Factor A, Factor B and Factor C.

To provide a deeper understanding of the sensitivity results, a micromechanical interpretation is presented here. Barite powder increases density due to its high specific gravity, but excessive amounts weaken the inter-particle bonding and reduce strength and modulus. Gypsum acts as the cementitious phase; its hydration products enhance inter-particle bonding, though excessive gypsum may increase brittleness. Water participates in the hydration reaction—insufficient water leads to incomplete hydration, while excessive water increases porosity. These mechanistic considerations are discussed in the following sensitivity analyses.

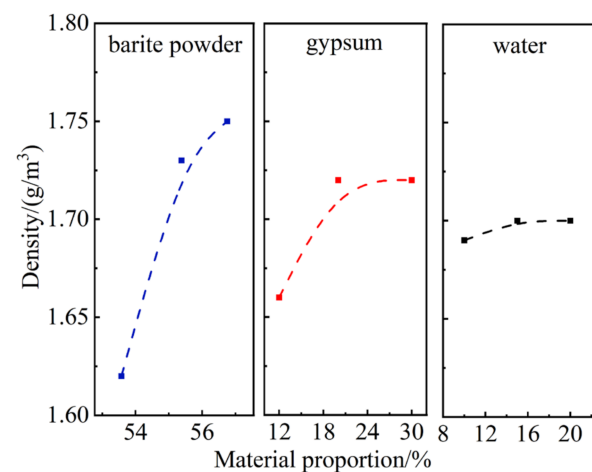
#### (1) Sensitivity Analysis of Factors Affecting Density

The range values and sensitivity analysis for the factors influencing specimen density are presented in Table 5 and Figure 4. The results indicate that the mass proportion of barite powder in the aggregate has the greatest impact on density. As the proportion of barite

powder increases, the specimen density correspondingly increases, which is attributed to the high density of barite powder relative to other aggregates. Since the mass proportions of gypsum and water are relatively small, the barite content in the aggregate predominantly governs the density of the similar material. Therefore, adjusting the amount of barite powder provides an effective means to control the target density.

**Table 5.** Range analysis of density.

| Level             | Density Test Results/(g/cm <sup>3</sup> ) |          |          |
|-------------------|-------------------------------------------|----------|----------|
|                   | Factor A                                  | Factor B | Factor C |
| 1                 | 1.62                                      | 1.66     | 1.69     |
| 2                 | 1.73                                      | 1.72     | 1.70     |
| 3                 | 1.75                                      | 1.72     | 1.70     |
| Range             | 0.13                                      | 0.06     | 0.01     |
| Influence Ranking | 1                                         | 2        | 3        |



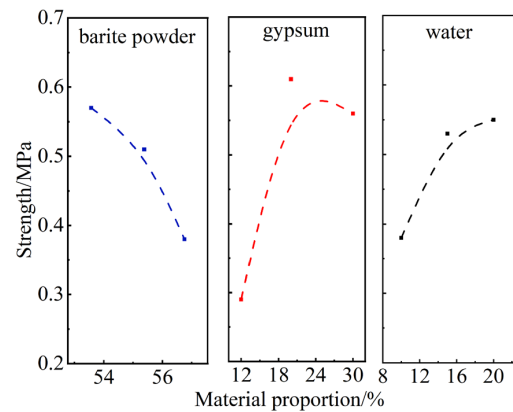
**Figure 4.** Density sensitivity analysis.

## (2) Sensitivity Analysis of Factors Affecting Uniaxial Compressive Strength

The range and sensitivity analysis of the factors influencing the compressive strength of the specimens are presented in Table 6 and Figure 5. The results demonstrate that gypsum, as the primary binder, has a significant effect on the specimen strength. The highest strength is achieved when the gypsum content is controlled at around 0.2 by mass. In contrast, the proportions of barite powder and water have a lesser impact. It is feasible to reduce the barite powder content and slightly increase the water content to enhance the hydration reaction, thereby improving the material's strength.

**Table 6.** Range analysis of compressive strength.

| Level             | Compressive Strength Results/MPa |          |          |
|-------------------|----------------------------------|----------|----------|
|                   | Factor A                         | Factor B | Factor C |
| 1                 | 0.57                             | 0.29     | 0.38     |
| 2                 | 0.51                             | 0.61     | 0.53     |
| 3                 | 0.38                             | 0.56     | 0.55     |
| Range             | 0.19                             | 0.32     | 0.17     |
| Influence Ranking | 2                                | 1        | 3        |



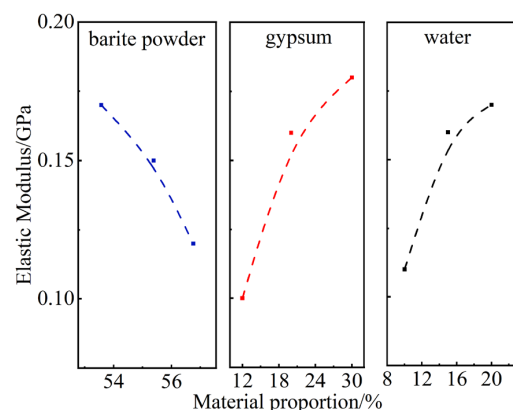
**Figure 5.** Sensitivity analysis of compressive strength.

### (3) Sensitivity Analysis of Factors Affecting Elastic Modulus

The range and sensitivity analysis of the factors influencing the elastic modulus of the specimens are presented in Table 7 and Figure 6. The results illustrate that the elastic modulus of the specimens decreases as the barite powder proportion increases. Similarly, the gypsum content has a significant impact on the elastic modulus: selecting an appropriate gypsum proportion can substantially enhance the modulus. In contrast, the effect of water content on the elastic modulus is relatively minor, as demonstrated by its narrow variation range. The increase in barite powder content reduces the bonding strength between aggregates, resulting in a decline in the elastic modulus. Appropriate gypsum content optimizes the hydration reaction, significantly improving density and modulus. The influence of water content is weak, as it primarily participates in hydration; only excessive amounts of water would significantly degrade performance. From this analysis, within the water content range of 0.1 to 0.2, higher water content promotes enhanced hydration.

**Table 7.** Range analysis of elastic modulus.

| Level             | Elastic Modulus Test Results/GPa |          |          |
|-------------------|----------------------------------|----------|----------|
|                   | Factor A                         | Factor B | Factor C |
| 1                 | 0.17                             | 0.10     | 0.11     |
| 2                 | 0.15                             | 0.16     | 0.16     |
| 3                 | 0.12                             | 0.18     | 0.17     |
| Range             | 0.05                             | 0.08     | 0.06     |
| Influence Ranking | 3                                | 1        | 2        |



**Figure 6.** Sensitivity analysis of elastic modulus.

### 2.2.3. Mix Proportion Selection

Based on the results of the orthogonal mix proportion tests and references from previous studies, a secondary refinement of the test scheme was designed. The orthogonal tests indicated that the proportion of barite powder in the aggregate significantly influences the material's density; higher amounts increase density but decrease strength and elastic modulus. The optimal strength was observed when the gypsum content was around 0.2 by mass, with increasing gypsum content leading to a higher elastic modulus. Water participates fully in the hydration reaction with gypsum; within the water content range of 0.1 to 0.2, adjusting the water proportion can improve both elastic modulus and density. The mix ratio of river sand, barite powder, quartz sand, gypsum, and water at 1:2.1:0.6:1.11:0.56 was found to best approximate the target mechanical parameters. Therefore, the secondary refinement tests used the third-level aggregate proportions while varying the gypsum and water mass ratios appropriately.

In the secondary refinement tests, direct shear tests were performed to determine the internal friction angle and cohesion of the similar materials. The final optimal mix ratio of river sand, barite powder, quartz sand, gypsum, and water was 1:2.1:0.6:1.11:0.76, which most closely matched the target experimental parameters. Detailed test results are presented in Table 8. Overall, the physical and mechanical parameters of the similar materials showed good agreement with theoretical values.

**Table 8.** Final determination of similarity material parameters.

| Material           | Density/(g/cm <sup>3</sup> ) | Compressive Strength/MPa | Elastic Modulus/GPa | Poisson's Ratio | Cohesion/MPa | Internal Friction/° |
|--------------------|------------------------------|--------------------------|---------------------|-----------------|--------------|---------------------|
| Prototype Value    | 2.65                         | 80.00                    | 24.00               | 0.23            | 1.5~2.1      | 50~60               |
| Model Design Value | 1.77                         | 0.53                     | 0.16                | 0.23            | 0.01~0.014   | 50~60               |
| Model Actual Value | 1.76                         | 0.53                     | 0.16                | 0.22            | 0.012        | 57                  |

### 2.3. Preparation of Similarity Materials for Jointed and Fractured Rock Masses

In this study, jointed and fractured rock mass specimens were prepared using the previously described similar material mix proportions. Joints with varying geometric characteristics were created by pre-embedding steel shims (0.2 mm in thickness) of different lengths and inclinations. The shims were carefully removed after the material reached initial setting (approximately 1.5 h) using the thin-sheet stripping method. The specimen molds had dimensions of 50 mm × 50 mm × 100 mm (length × width × height). Uniaxial compression tests were conducted to investigate the mechanical behavior of the jointed and fractured rock mass. The specimen fabrication process is illustrated in Figure 7. It should be noted that the extraction of steel shims may cause local micro-damage or stress redistribution at the joint interface, which could potentially affect the peak strength and crack initiation behavior. To minimize this disturbance, the shims were carefully extracted when the material reached initial setting, at which point the material had gained sufficient strength to maintain the crack geometry while still being soft enough to reduce disturbance to the surrounding matrix. Although some local disturbance is inevitable, this is considered acceptable as it partially reflects the initial damage state of natural jointed rock masses. Importantly, the similarity material developed in this study is specifically designed for physical model tests, and its direct extrapolation to general studies on natural rock masses remains limited.

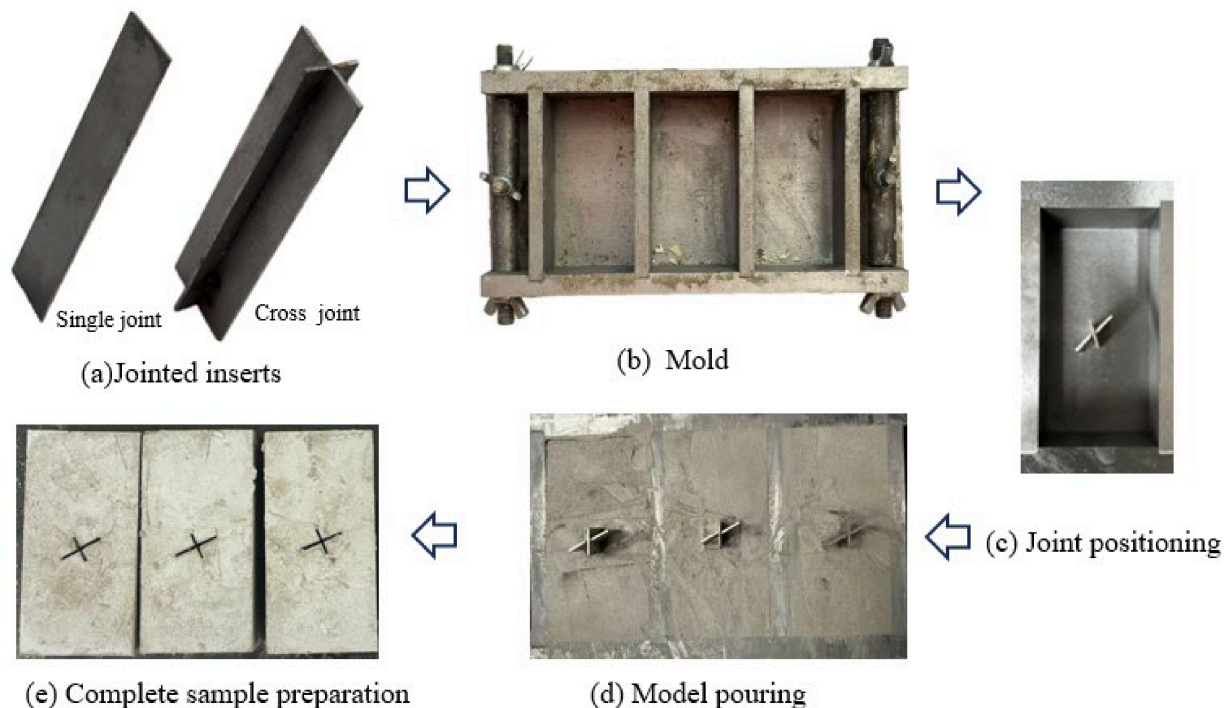


Figure 7. Preparation process of jointed and fractured rock mass specimens.

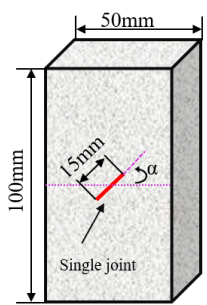
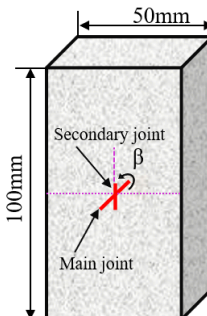
### 3. Experimental Verification of the Mechanical Behavior and Fracture Mechanism of Jointed and Fractured Rock Mass Similar Materials

Published research on the fracture mechanisms of jointed and fractured rock masses primarily involves the use of prefabricated joint cracks [29–31]. In this study, the same approach is employed to fabricate jointed rock mass models. The mechanical parameters and fracture mechanisms of the jointed rock mass similitude materials are analyzed to validate the nonlinear behavior of these similitude materials.

#### 3.1. Experimental Conditions Design

Based on a geometric similarity ratio of  $C_L = 100$ , this study employs a single joint length of 15 mm and a dip angle of  $60^\circ$  to investigate whether the failure patterns of jointed fractures align with those observed in the actual rock mass. To provide comparative data, additional test conditions with joint angles of  $0^\circ$ ,  $30^\circ$ ,  $45^\circ$ , and  $90^\circ$  are included. Furthermore, to explore the failure behavior of intersecting joints, the primary joint parameters are kept consistent with those of the single joint, while the secondary joint length is set at 10 mm. The intersecting joint combinations are denoted as “ $\alpha$ - $\beta$ ”, where  $\alpha$  is the inclination angle of the primary joint and  $\beta$  is the inclination angle of the secondary joint (e.g., “30–60” means primary joint at  $30^\circ$  and secondary joint at  $60^\circ$ ). The design of the intersecting joint combinations referred to the study by Wang et al. [1], with primary joint angles  $\alpha = 30^\circ$  and  $60^\circ$  and secondary joint angles  $\beta = 60^\circ$ ,  $90^\circ$ ,  $120^\circ$ , and  $150^\circ$ . The angle  $\beta = 0^\circ$  was excluded as it does not form an intersecting configuration;  $\beta = 30^\circ$  and  $45^\circ$  were excluded based on Wang et al. [30], who found that these angles correspond to relatively low rock mass strengths. In this study, the joint inclination angle  $\alpha$  is defined as the angle between the joint plane and the vertical loading direction, i.e.,  $\alpha = 90^\circ$ : joint parallel to loading;  $\alpha = 0^\circ$ : joint perpendicular to loading. The test conditions are summarized in Table 9.

**Table 9.** Detailed experimental conditions.

| Case | Joint Type and Schematic                                                          | Joint Length/mm | Thickness/mm | Joint Angle $\alpha$ - $\beta$ (°) |
|------|-----------------------------------------------------------------------------------|-----------------|--------------|------------------------------------|
| 1    |  | 15              | 1            | 0                                  |
| 2    |                                                                                   |                 |              | 30                                 |
| 3    |                                                                                   |                 |              | 45                                 |
| 4    |                                                                                   |                 |              | 60                                 |
| 5    |                                                                                   |                 |              | 90                                 |
| 6    |  | 15/12           | 1            | 30–60                              |
| 7    |                                                                                   |                 |              | 30–90                              |
| 8    |                                                                                   |                 |              | 30–120                             |
| 9    |                                                                                   |                 |              | 60–90                              |
| 10   |                                                                                   |                 |              | 60–120                             |
| 11   |                                                                                   |                 |              | 60–150                             |

### 3.2. Verification of Mechanical Response Characteristics of Similar Materials

As shown in Figure 8, the peak uniaxial compressive strength varies according to the joint orientation in the following order:  $90^\circ > 60^\circ > 45^\circ > 0^\circ > 30^\circ$ . Based on previous studies on jointed and fractured rock masses [29], the observed variation in peak strength aligns well with established engineering prototypes, indicating that similar materials effectively replicate the nonlinear failure behavior of composite jointed and fractured rock masses. The joint angle corresponding to peak strength may depend on material properties such as the internal friction angle of the rock-like material. Further systematic studies are needed to clarify this relationship. The failure process of the specimens can be divided into four stages:

1. Initial Compaction Stage: At this stage, axial stress is relatively low. Microcracks inevitably generated during specimen preparation gradually close, leading to specimen densification
2. Linear Elastic Stage: Axial stress increases steadily, and the specimen exhibits linear elastic behavior.
3. Unstable Crack Propagation Stage: This stage involves elastic deformation and stable microcrack growth, during which the axial stress reaches its peak.
4. Post-Peak Failure Stage: After reaching peak strength, the specimen's internal structure is damaged, causing a rapid decline in stress.

The stress–strain curves of specimens with a constant main joint inclination and varying secondary joint inclinations are presented in Figure 9. The deformation process in all cases can be divided into four stages: initial compaction, linear elastic growth, unstable crack propagation, and post-peak failure. However, subtle differences exist in the failure curves of specimens with different single joints, primarily during the unstable crack propagation and post-peak stages. These differences arise because the specimens are influenced by multiple joints, and the cross-jointed specimens fail along different paths, resulting in variations in post-peak deformation behavior. When the main joint inclination

angle  $\alpha$  is  $30^\circ$ , the influence of the secondary joints on the specimens is minimal, and the peak stresses are relatively similar. In contrast, at  $\alpha = 60^\circ$ , the secondary joints have a more pronounced effect, with the peak strength varying according to the secondary joint angles in the order  $90^\circ > 120^\circ > 150^\circ$ . This indicates that a larger main joint inclination angle significantly affects the strength variation trends among specimens with different secondary joint orientations. In summary, the specimen strength is primarily controlled by the main joint, while the secondary joints contribute to a further reduction in strength. This observed behavior is consistent with the findings summarized by Wang et al. [30].

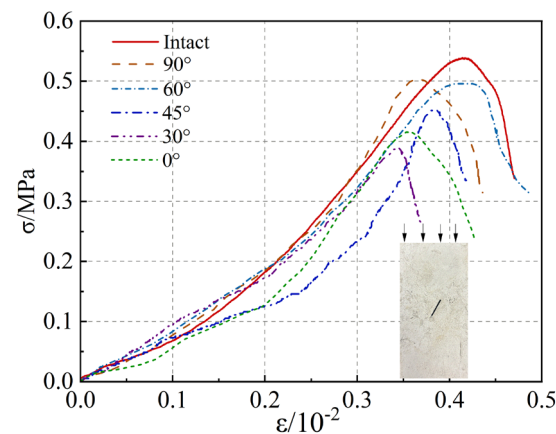


Figure 8. Stress–strain curves of single-joint specimens.

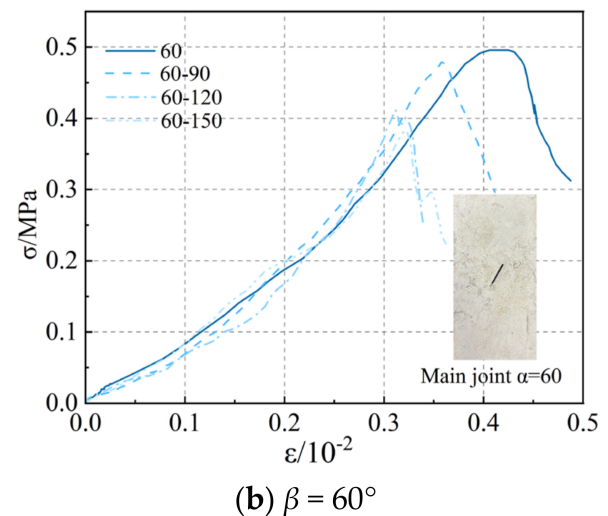
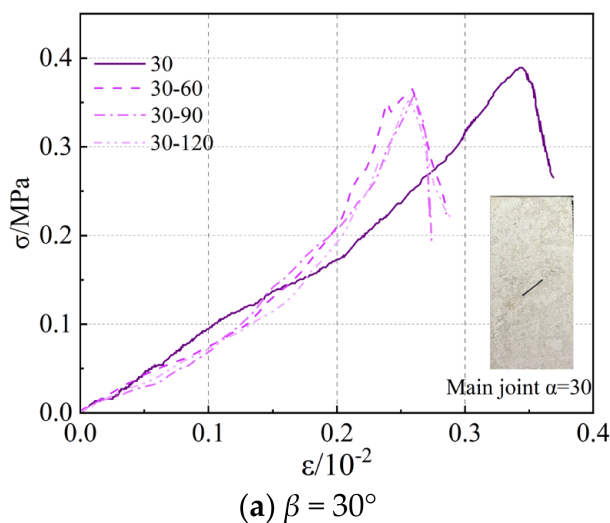
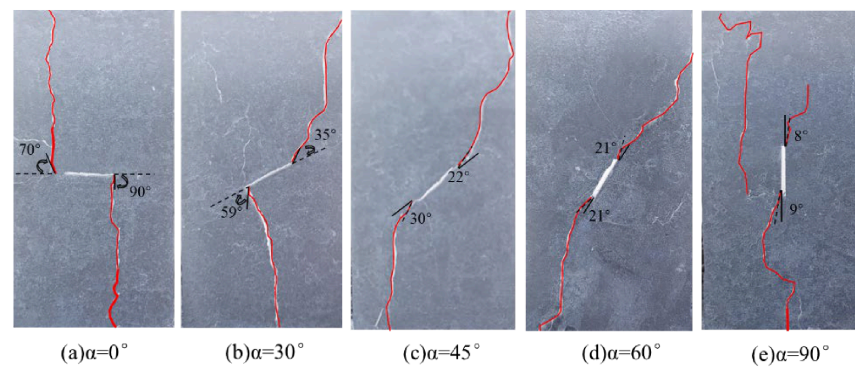


Figure 9. Stress–strain curves of cross-joint specimens.

### 3.3. Verification of Failure Mechanism in Similar Materials

For specimens with five different joint angles, representative scanned images depicting crack damage were selected from each test to illustrate the failure patterns. Penetrating cracks were marked with red lines through these images, and the crack angles were measured, as shown in Figure 10. When the joint inclination angle  $\alpha$  equals  $30^\circ$ ,  $45^\circ$ , and  $60^\circ$ , the failure mode is primarily characterized by tensile-shear cracking, with cracks propagating laterally along both sides of the joint and ultimately penetrating through the entire specimen. Conversely, for  $\alpha = 0^\circ$  and  $90^\circ$ , the failure mode is dominated by tensile splitting, with the main cracks concentrated near the top and bottom of the specimen.



**Figure 10.** Failure patterns in single-joint specimens.

The macroscopic crack angle  $\theta$  is defined as the angle between the primary crack and the extended line of the prefabricated joint in the specimen. When the joint angle  $\alpha = 0^\circ$ , the crack angle  $\theta$  at the upper part is approximately  $70^\circ$ , and at the lower part is  $90^\circ$ . As the joint angle  $\alpha$  increases, the overall crack angle  $\theta$  tends to decrease, with the crack angle at the lower part consistently larger than that at the upper part during cracking.

The distortion energy theory postulates that distortion energy is the primary factor causing fracture. Under uniaxial compressive stress, yielding of the material occurs when the distortion energy at any point within the specimen reaches the critical value corresponding to uniaxial stress conditions. Based on this criterion [23], the functional relationship between the pre-existing joint inclination angle  $\alpha$  and the crack initiation angle  $\theta$  at fracture is expressed as

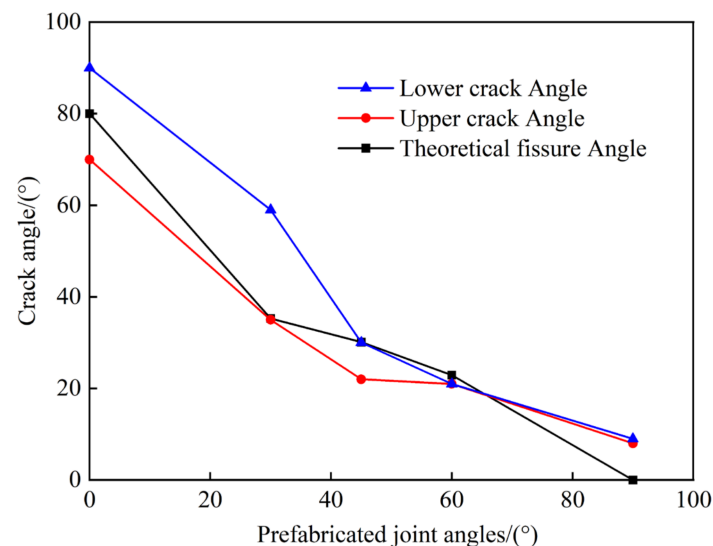
$$\sin\theta \left( 2\cos\theta - \frac{1}{6} \right) \tan^2 \left( \frac{\pi}{2} - \alpha \right) + \left( 4\cos 2\theta - \frac{1}{3} \cos\theta \right) \tan \left( \frac{\pi}{2} - \alpha \right) + \sin\theta \left( \frac{1}{6} - 6\cos\theta \right) = 0 \quad (7)$$

Although Equation (7) is strictly derived for a single joint, Wang et al. [30] demonstrated through 3D-printed intersecting joint specimens that the secondary joint has little influence on the crack initiation angle and fracture propagation mode of the primary joint, and that the primary joint absolutely controls rock failure. Therefore, applying Equation (7) to predict the crack initiation angle along the primary joint in intersecting joint specimens is considered reasonable. The experimental results of the present study (Section 3.3) also support this conclusion.

After simplification, the relationship can be expressed as

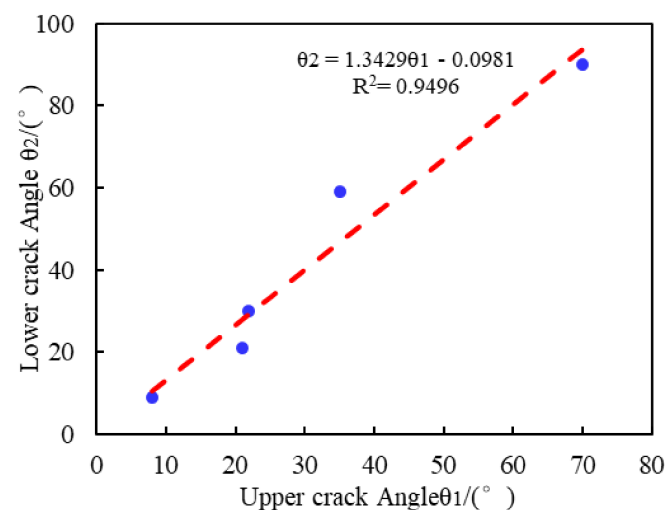
$$\left( 4\cos 2\theta - \frac{1}{3} \cos\theta \right) \tan \left( \frac{\pi}{2} - \alpha \right) - 2\sin 2\theta \quad (8)$$

To investigate the variation pattern of crack initiation angles, quantitative measurements were conducted for each observed cracking angle and compared with theoretical values derived from the distortion energy criterion. Figure 11 compares the observed cracking angles ( $\theta_1$  for the upper part and  $\theta_2$  for the lower part) with the theoretical values. Both sets demonstrate that the crack initiation angles decrease significantly with increasing pre-existing joint inclination angle. This indicates that the development of macroscopic cracks is strongly influenced by the orientation of the prefabricated joint. Additionally, for inclined joints, the lower crack initiation angle ( $\theta_2$ ) is consistently greater than the upper one ( $\theta_1$ ), suggesting that the lower portion of the specimen is more affected by the loading direction during fracture propagation.



**Figure 11.** Relationship between joint angle  $\alpha$  and crack initiation angle  $\theta$ .

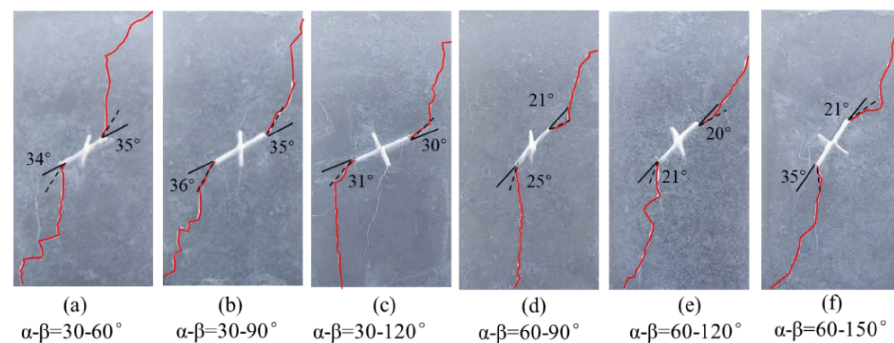
A statistical analysis was performed on the upper and lower crack initiation angles of the same specimen, and the results are presented in Figure 12. There is a clear linear relationship, with the fitted function expressed as  $\theta_2 = 1.3429\theta_1 - 0.0981$  and a coefficient of determination  $R^2 = 0.9496$ . This high degree of fit indicates that a linear function accurately describes the observed data. It also suggests that, under uniaxial compression, the initiation of cracks along prefabricated joints is neither random nor independent. Instead, the upper and lower crack angles are closely related: as the lower crack angle increases, the upper crack angle tends to increase accordingly.



**Figure 12.** Relationship between upper and lower crack initiation angles.

As shown in Figure 13, for cross-jointed specimens with a primary joint inclination of  $\alpha = 30^\circ$ , the observed upper crack initiation angles along the primary joint were  $35^\circ$ ,  $35^\circ$ , and  $30^\circ$ , respectively. When the primary joint angle was  $\alpha = 60^\circ$ , the corresponding crack initiation angles were  $21^\circ$ ,  $20^\circ$ , and  $21^\circ$ . Substituting  $\alpha = 30^\circ$  and  $\alpha = 60^\circ$  into Equation (7) yields theoretical crack initiation angles of  $\theta_{30} = 35.74^\circ$  and  $\theta_{60} = 23.79^\circ$ , respectively. The comparison reveals that, despite the presence of secondary joints at various inclinations relative to the primary joint, the observed crack initiation angles along the primary joint closely match the theoretical predictions. This suggests that the influence of secondary joints on the stress distribution at the tip of the primary joint is minimal, and that crack

propagation is primarily governed by the primary joint. These findings further confirm that the mechanical behavior of jointed and fractured rock masses is predominantly controlled by the orientation and characteristics of the primary joint.



**Figure 13.** Failure characteristics of single-joint specimens.

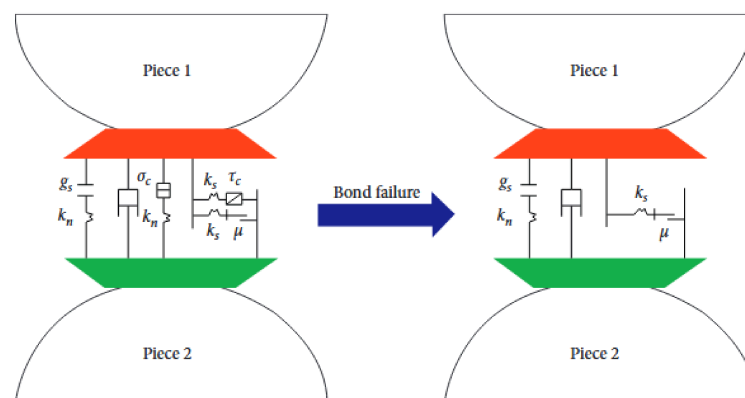
#### 4. Discrete Element Modeling and Analysis of the Nonlinear Mechanical Behavior of Jointed and Fractured Rock Masses

Extensive research has been conducted on the numerical simulation of jointed and fractured rock masses [31–33]. In this section, the particle flow code PFC<sup>2D</sup> is employed to perform discrete element simulations. By constructing numerical models based on actual engineering rock mass parameters and conducting uniaxial compression tests, the simulated results are compared with mechanical tests to further verify whether the failure mechanisms and mechanical response of jointed and fractured rock masses follow nonlinear behavior patterns.

##### 4.1. Numerical Model Construction and Calibration

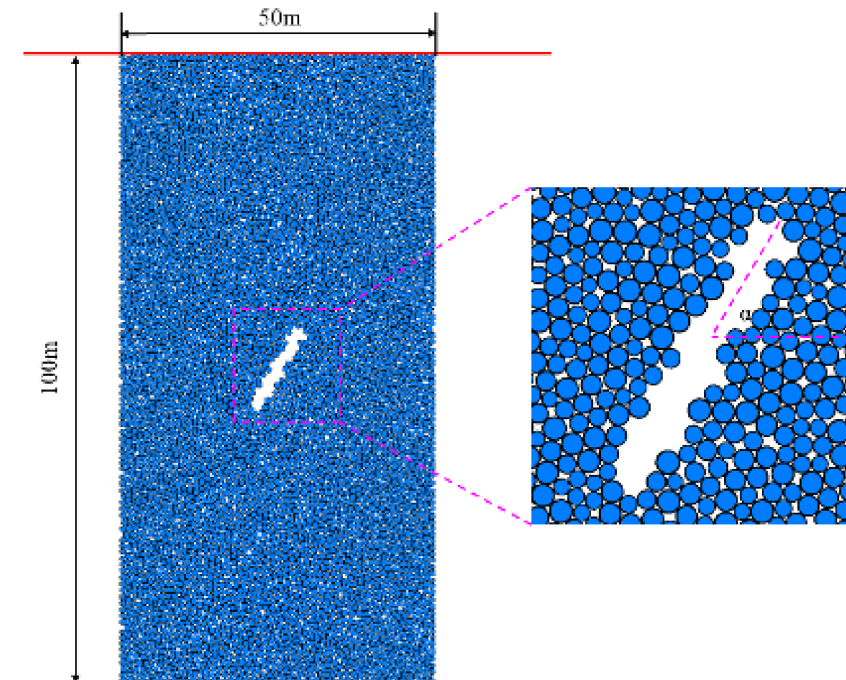
When using PFC<sup>2D</sup> for numerical simulation, certain assumptions regarding particle geometry and contact behavior must be made. The basic assumptions of PFC<sup>2D</sup> include the following: particles are modeled as rigid discs (2D) or spheres (3D); interactions between particles occur at discrete contact points, with bonded contacts possessing specific mechanical properties; small overlaps between particles are permitted at contact points; and the force–displacement relationship defines the constitutive behavior between particles [34,35].

During PFC<sup>2D</sup> simulations, two fundamental laws govern the particle interactions: the force–displacement law, which describes the contact mechanics, and Newton’s second law, which governs particle motion [36]. The parallel bond model is adopted in the present study to simulate the mechanical behavior of bonded rock material. The schematic of this model is shown in Figure 14.



**Figure 14.** Schematic of the parallel bond model.

To investigate the failure mechanisms and mechanical response of jointed and fractured rock masses, numerical specimens were developed in PFC<sup>2D</sup> using parameters representative of actual engineering rock masses. Artificial joints were introduced using the particle deletion method, resulting in numerical models of fractured rock specimens, as shown in Figure 15.

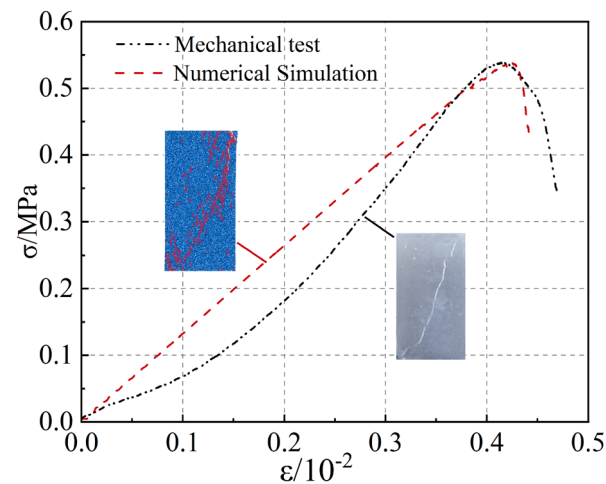


**Figure 15.** Numerical model of a rock specimen with pre-existing joints.

The micro-parameters of the model were calibrated using a trial-and-error approach, with the finalized values presented in Table 10. To validate the rationality of these parameters, numerical uniaxial compression tests were conducted on intact rock specimens for comparison. Additionally, the scaled simulation results were compared with mechanical test outcomes, as shown in Figure 16. A discrepancy is observed in the post-peak response after  $\epsilon \approx 0.37$ : the numerical simulation shows a relatively linear trend, whereas the experimental results exhibit nonlinear behavior. This difference arises from the inherent nature of the PFC2D discrete element model. Once the parallel bonds begin to break, the load-bearing capacity of the specimen decreases rapidly, resulting in a sudden stress drop and a relatively linear post-peak trend in the numerical simulation. In contrast, the physical specimen exhibits progressive microcrack accumulation and frictional sliding along fracture surfaces, producing a more gradual and nonlinear post-peak response.

**Table 10.** Micro-parameters of the jointed rock mass model.

| Parameter                             | Value | Parameter                              | Value |
|---------------------------------------|-------|----------------------------------------|-------|
| Minimum radius of particles/m         | 0.4   | Frictional coefficient of particles    | 0.577 |
| Maximum radius of particles/m         | 0.6   | Parallel-bond modulus/GPa              | 11.76 |
| Particle density/(kg/m <sup>3</sup> ) | 2400  | Stiffness ratio of parallel-bond model | 1.2   |
| Contact modulus of particles/GPa      | 11.76 | Normal bond strength/MPa               | 30.2  |
| Stiffness ratio of particles          | 1.2   | Tangential bond strength/MPa           | 33.5  |

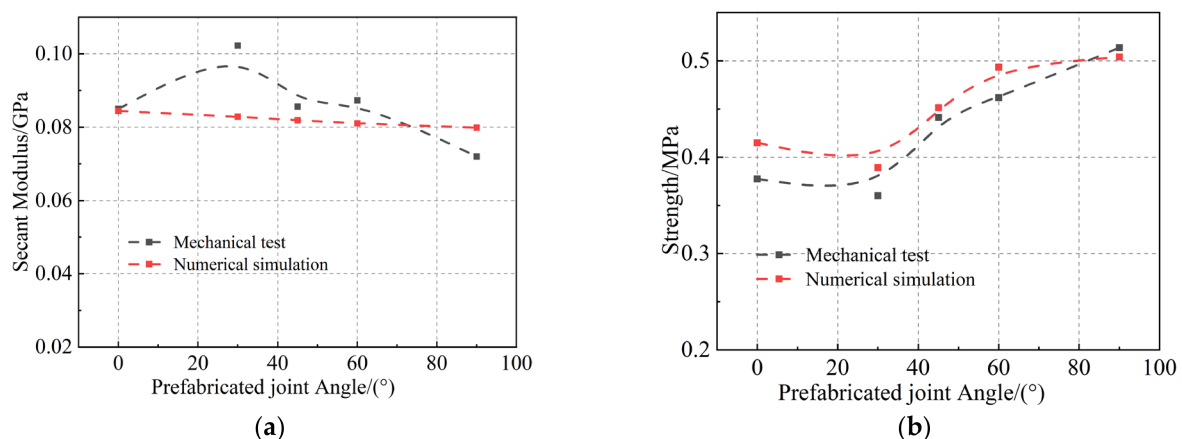


**Figure 16.** Comparison between numerical simulation and mechanical test results.

#### 4.2. Comparison of Mechanical Responses Between Mechanical Tests and Numerical Simulations of Single-Joint Rock Specimens

##### (1) Comparison of Mechanical Responses for Single-Joint Specimens

To further analyze whether the mechanical response of jointed rock-like materials follows nonlinear behavior, the scaled results ( $C_\sigma = 1:150$ ) of numerical simulations based on actual engineering prototypes were compared with those from mechanical tests. Under uniaxial compression, rocks typically exhibit nonlinear behavior, especially for rocks with multiple fractures or weak layers. The stress–strain curve can be divided into compaction, elastic, plastic, and failure stages. The secant modulus reflects the average stiffness within a certain strain range and is suitable for analyzing nonlinear materials. As shown in Figure 17, the secant moduli from both the mechanical tests and numerical simulations are close and exhibit an overall decreasing trend, reaching a minimum at a joint angle of  $90^\circ$ . Comparison of peak stresses indicates that the compressive strength ranking is for joint orientation of  $90^\circ > 60^\circ > 45^\circ > 0^\circ > 30^\circ$  for both, with nearly identical peak strengths. This suggests that the mechanical response characteristics of the single-joint rock-like materials are generally consistent with those of the engineering prototype.

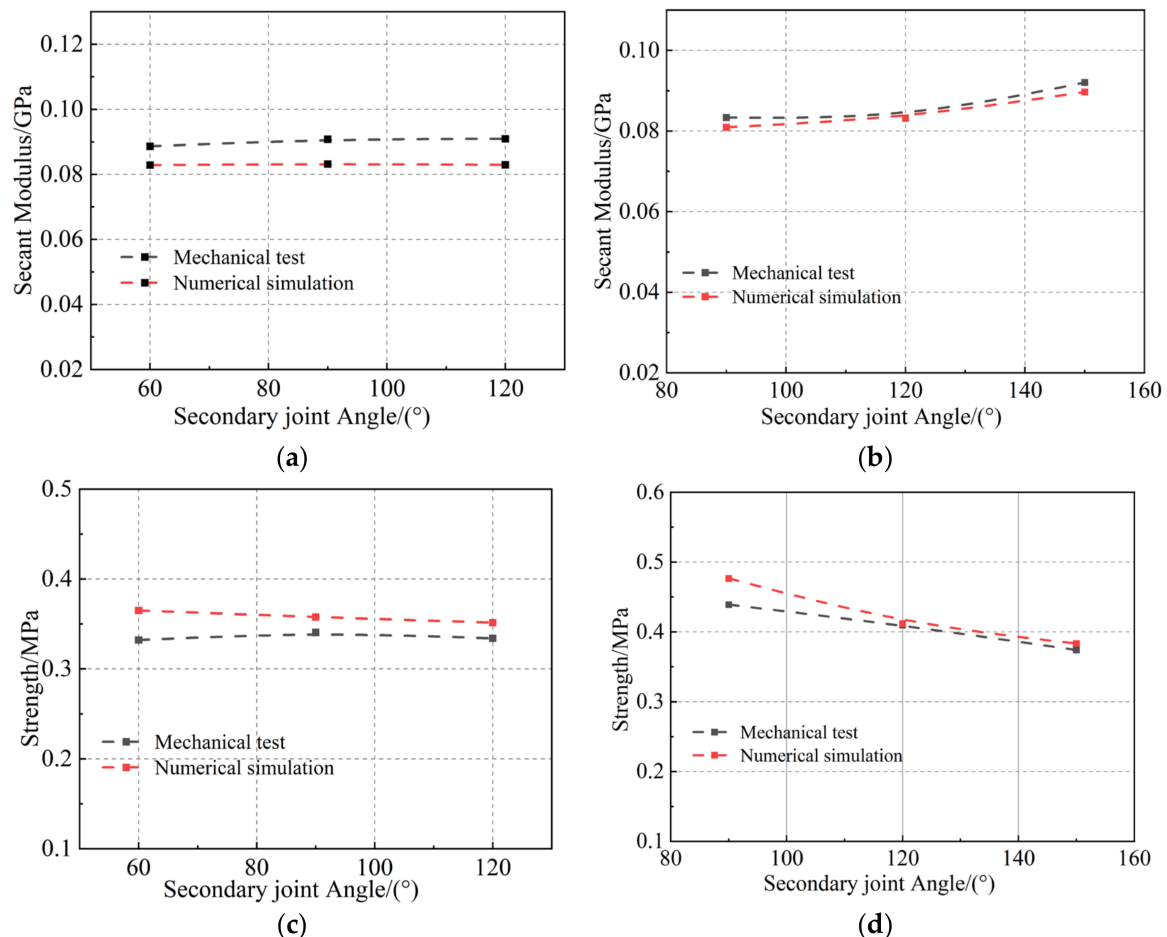


**Figure 17.** Comparison of mechanical responses of single-joint specimens. (a) Variation of secant modulus for single-joint specimens. (b) Variation of peak strength for single-joint specimens.

##### (2) Comparative Analysis of Mechanical Responses in Cross-Jointed Rock Specimens

Figure 18 compares the mechanical response characteristics of cross-jointed rock specimens obtained from numerical simulations and mechanical tests. When the primary

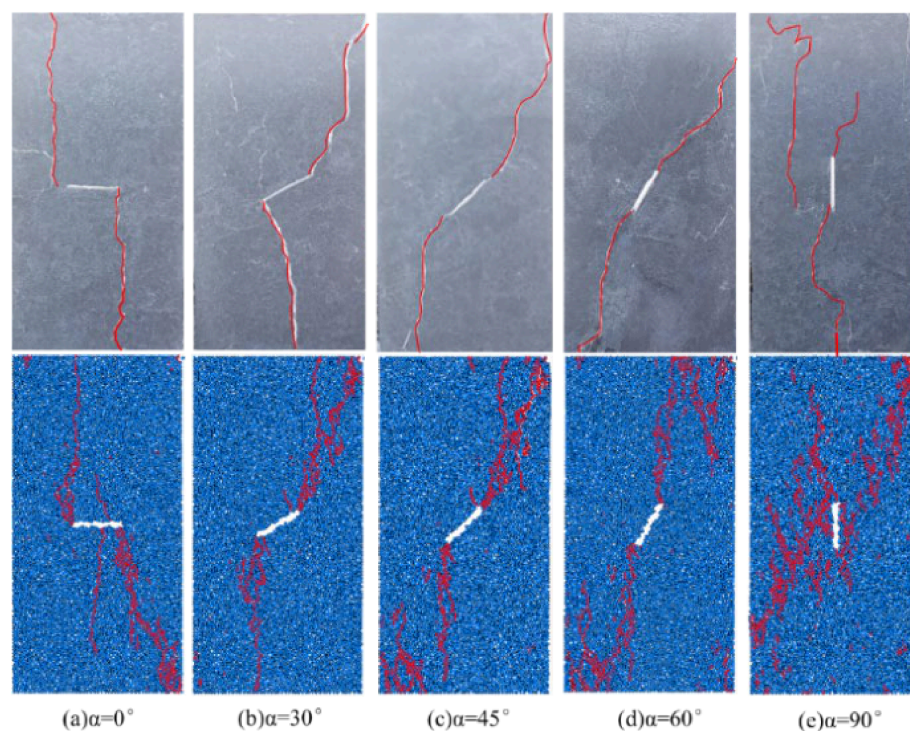
joint angle is  $30^\circ$ , the trends in secant modulus from both approaches are largely consistent, with the secondary joint angle exerting minimal influence on both secant modulus and peak strength, which remain nearly unchanged. For a primary joint angle of  $60^\circ$ , the numerical simulation and mechanical test results also closely agree, showing a slight increase in secant modulus accompanied by a decrease in peak strength. Building on the previously discussed trends in secant modulus and peak strength for single-joint specimens, it is further confirmed that the strength of jointed rock is predominantly controlled by the primary joint, while the secondary joint plays a subordinate role in further weakening the specimen's strength.



**Figure 18.** Comparison of mechanical responses in cross-jointed specimens. (a) Secant modulus with secondary joint at  $30^\circ$ . (b) Secant modulus with secondary joint at  $60^\circ$ . (c) Peak strength with secondary joint at  $30^\circ$ . (d) Peak strength with secondary joint at  $60^\circ$ .

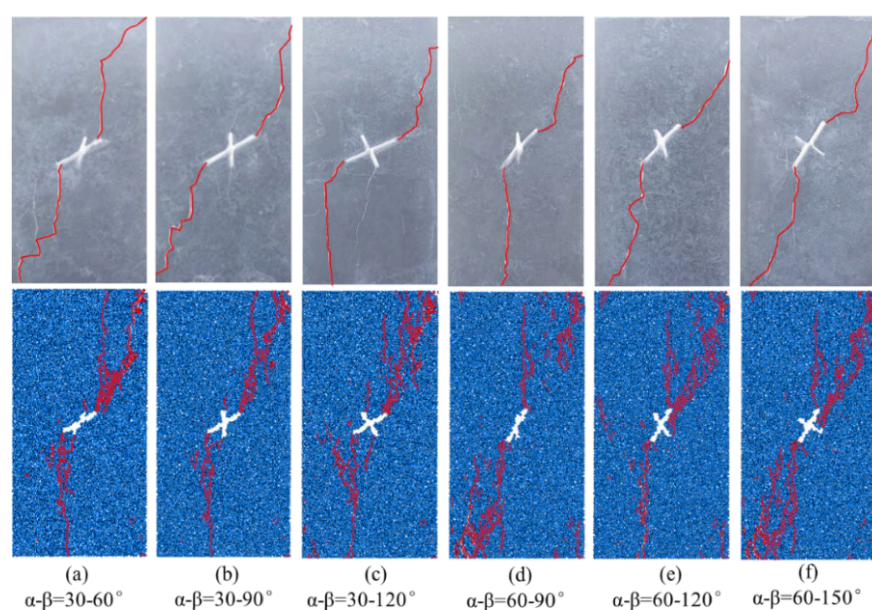
#### 4.3. Comparison of Failure Modes Between Mechanical Tests and Numerical Simulations

The crack propagation patterns in single-joint rock specimens exhibit a high degree of consistency between mechanical tests and numerical simulations. As shown in the failure mode comparison in Figure 19, cracks in the mechanical tests primarily initiate and propagate along the joint planes. Under certain stress conditions, these cracks extend laterally to adjacent regions, forming stepped or arcuate through-cracks, which are characteristic of the shear-tensile composite failure mode typical of jointed and fractured rock masses. The simulation results similarly demonstrate crack initiation along the joint planes, followed by gradual deviation toward the direction of the maximum principal stress, ultimately producing a through-failure pattern comparable to that observed in the experiments.



**Figure 19.** Failure modes of single-joint specimens.

Figure 20 illustrates that the failure patterns of the jointed rock mass with intersecting joints are also generally consistent. Crack propagation is primarily controlled by the main joint set, with cracks initiating and extending along the main joint before spreading to adjacent regions. This behavior occurs because, in the presence of intersecting joints, cracks preferentially propagate along the joint set with the lowest shear resistance. Since the main joint set has a longer trace length, the failure is predominantly concentrated along these main joint planes. Comparative analysis further confirms that the failure behavior of the jointed rock mass analog material aligns well with the nonlinear mechanical behavior observed in natural jointed and fractured rock masses.



**Figure 20.** Failure mode of cross-jointed rock mass.

## 5. Conclusions

In this study, a jointed rock mass analogue material was developed. Its nonlinear mechanical behavior was verified through mechanical testing, numerical simulation, and distortion energy theory analysis. The following conclusions may be drawn:

- (1) The optimal mix proportion for the intact rock mass analogue was determined as river sand–barite powder–quartz sand–gypsum–water = 1:2.1:0.6:1.11:0.56.
- (2) The peak uniaxial compressive strength of single-joint specimens follows the order  $90^\circ > 60^\circ > 45^\circ > 0^\circ > 30^\circ$ . In cross-jointed specimens, the secondary joint has a limited weakening effect; the primary joint dominates the overall mechanical behavior. These trends are consistent with previous studies.
- (3) As joint inclination increases, the crack initiation angle decreases significantly. The measured crack angles agree well with predictions from distortion energy theory, and the lower crack angle is consistently larger than the upper one.
- (4) Numerical simulations (PFC2D) and mechanical tests show close agreement in both mechanical responses and failure patterns. The secant moduli exhibit an overall decreasing trend, and the peak strength order is identical to that in the experiments. The observed failure patterns correspond to the typical shear-tensile composite mode of jointed and fractured rock masses.

As a final concluding remark, the developed analog material reliably reproduces the nonlinear mechanical characteristics of jointed and fractured rock masses, is therefore suitable for physical model tests of underground caverns in such geological environments, and is specifically intended for model test applications; its direct applicability to general studies on natural rock masses remains limited.

**Author Contributions:** J.Z.: Conceptualization, Methodology, Software, Writing—Original Draft. M.Z.: Investigation, Data Curation, Software, Validation. M.H.E.N.: Writing—Review and Editing. J.H.: Software, Methodology. X.Z.: Conceptualization, Investigation, Data Curation. X.D.: Supervision. All authors have read and agreed to the published version of the manuscript.

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**Conflicts of Interest:** The authors declare no conflicts of interest.

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