

Article

Effect of Joint Morphology on Fracture Behavior for Rock Mass: DEM Investigation on a Single Joint

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Abstract

This study proposes a novel discrete element modelling (DEM) framework for investigating the fracture behavior of jointed rock masses, with particular emphasis on rock specimens containing a single pre-existing joint. The proposed method explicitly represents joint geometry through constrained mesh generation, enabling accurate simulation of joint-controlled fracture processes without introducing additional joint constitutive models. The reliability of the numerical approach is validated by comparison with laboratory uniaxial compression tests on rock specimens containing a single closed joint. On this basis, the influences of joint dip angle, length and width on the mechanical response and fracture evolution of single-jointed rock specimens are systematically examined. The results demonstrate that the proposed method can effectively capture stress–strain behavior, crack initiation, propagation, and coalescence governed by joint geometry. Owing to its high computational efficiency, straightforward implementation, and accurate geometric representation, the proposed framework shows strong potential for simulating brittle fracture processes in jointed rock masses.

Keywords: joint morphology; fracture behavior; discrete element method (DEM); single joint

1. Introduction

Natural rock masses commonly contain geological discontinuities such as joints and fractures [1,2]. The presence of these discontinuities causes rock masses to exhibit discontinuous, heterogeneous, inelastic, and anisotropic characteristics, thereby posing significant safety risks to engineering structures [3]. In many engineering applications, however, the mechanical behavior and failure mechanisms of rock masses are often governed by a limited number of dominant discontinuities [4,5]. Among them, rock masses containing a single joint or flaw represent the most basic structural form and constitute a fundamental unit for understanding the mechanical response of more complex fractured rock systems [6–9].

In natural rock slopes, the geometric and morphological characteristics of joints—such as length, inclination, and persistence—together with their mechanical properties, play a critical role in controlling slope stability, strength, deformation behavior, and failure modes [10,11]. These characteristics form the theoretical basis for the prediction and assessment of rock slope stability. Under complex stress conditions, pre-existing single joints may undergo tensile opening, sliding, or closure, leading to significant stress redistribution and strongly influencing the initiation, propagation, and coalescence of new cracks [12–14].



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Fracture structures are ubiquitous in natural rock masses and represent one of the most important factors affecting rock mass stability, with profound implications for the long-term safety and reliability of rock engineering structures. Numerous catastrophic engineering failures have been attributed to an insufficient understanding of the mechanical behavior and fracture mechanisms of jointed rock masses, such as the Malpasset Dam failure in France and the Vajont landslide associated with the Vajont Hydropower Station in Italy [15,16]. These events demonstrate that even a limited number of unfavorably oriented or sized joints may dominate the failure process of a rock mass, highlighting the necessity of systematically investigating joint-controlled fracture behavior starting from simplified configurations.

Experimental investigations on rock masses containing complex joint networks are generally time-consuming and costly, making systematic parametric studies difficult to conduct [17,18]. Consequently, many researchers have focused on laboratory-scale model tests and numerical simulations of rock specimens with simplified joint configurations, among which single-joint specimens have been extensively adopted as a representative research model. Studies on the fracture mechanisms of single-joint rock masses date back to the 1960s. Hoek and Bieniawski [19] were among the earliest researchers to investigate the mechanical properties and compressive failure characteristics of brittle materials containing pre-existing joints. Brace et al. [7] conducted uniaxial compression tests on plate specimens with a single flaw and demonstrated that crack initiation typically occurs at the joint tips and propagates along the direction of the maximum principal stress. Wong et al. [11] further identified three primary crack types—tensile cracks, shear cracks, and mixed tensile–shear cracks—based on uniaxial compression tests on marble and gypsum specimens containing a single flaw. Lee et al. [20] investigated crack initiation and propagation stresses in specimens containing two non-parallel flaws and compared the results with those of single-flaw specimens, further highlighting the fundamental role of single-joint models in fractured rock research.

Although previous experimental and numerical studies have significantly advanced the understanding of crack initiation and propagation mechanisms in single-joint rock specimens, most existing works have primarily focused on crack patterns and failure modes, while the systematic influence of joint morphology on fracture behavior has not yet been fully clarified. In particular, the effects of key morphological parameters, such as joint length and inclination, on stress evolution, deformation localization, and strength degradation remain insufficiently quantified.

Therefore, in this study, a two-dimensional discrete element model is established to investigate the fracture behavior of rock specimens containing a single joint under uniaxial compression. The study demonstrated that the parameters of the angle of inclination and the length of the joint significantly affect the strength characteristics and resistance to cracking in rocks during uniaxial compression. The results aim to provide deeper insights into the joint-controlled fracture mechanisms of rock masses and to serve as a fundamental reference for understanding more complex jointed rock systems.

The remainder of this paper is organized as follows. Section 2 describes the rigid-block discrete element modelling framework, including mesh generation, contact laws, and numerical implementation. Section 3 presents the numerical model setup and parameter calibration. Section 4 discusses the effects of joint inclination, length, and width on strength characteristics and fracture behavior. Finally, Section 5 summarizes the main findings and conclusions of the study.

2. Method

2.1. Flowchart of the Study

A rigid block discrete element model of a rock mass can be created by several methods, such as from vertices or geometry. For the experiments in this paper, the choice was made to create a rigid fast discrete element model by means of a finite element mesh. The model is based on a finite element mesh and separates the object into multiple discrete block elements in order to better describe the contact behavior between the blocks. In this model, the individual block elements can be rotated and translated, and the contact between them generates stresses and deformations and is analyzed in terms of forces according to the chosen material model. The numerical methodology adopted in this study consists of three main components: (1) constrained mesh generation for explicit joint representation, (2) rigid-block discrete element formulation with contact constitutive laws, and (3) numerical simulation of fracture evolution under uniaxial compression. These components are sequentially coupled to ensure that joint geometry, mechanical interaction, and crack propagation are consistently represented within a unified framework.

Model building requires that the geometry is first meshed so that the solution domain can be characterized. In addition, the mesh needs to be progressively refined in order to achieve the required accuracy of the results, while avoiding over-refinement that leads to an increase in the solution time and memory footprint. Afterwards, the mesh information in the grid model of the joint rock mass containing joints can be obtained, and the numbering of each element, node, and edge of the cell is unique and only unique. For the convenience of subsequent simulation experiments, the grid containing joint boundaries needs to be labelled and recorded one by one. If they are on the internal joint boundaries, the edges of the joint elements and the grid elements connected to the joint edges are labelled and grouped to obtain the final closed joint rock mass numerical simulation model. Once the above steps are completed, a mesh of rigid blocks based on the finite element method that have different shapes, sizes, locations and contacts with each other is obtained. The next important step is to assign parametric properties to the mesh blocks and contact points in order to transform them from independent rigid mesh blocks to interacting 2D rock specimen simulation interfaces.

The meshes in the model that contain the boundaries of the joints have been recorded during mesh dissection, so it is easy to find all meshes at the original joint. When assigning contact parameters, individual parameters were weakened somewhat at the joints due to discontinuities between the joints. The flowchart of the whole process is shown in Figure 1.

2.2. DEM Using Rigid Blocks

The discrete element method is a numerical simulation method first proposed by Cundall in 1971 [21]. The method decomposes the discontinuity into a series of rigid elements, and then solves the equations of motion and dynamics of these rigid elements to simulate the discontinuous mechanical behavior of the target model. Usually, the DEM solves the numerical solutions of these equations by time-step iterations to obtain the motion state of the whole discontinuity. In previous studies, scholars have mostly used spherical (3D) or circular (2D) particles as fillers to establish the geometric model of the rock mass. However, they are inherently limited in their ability to accurately represent complex geometries. Such simplified shapes cannot capture angularity, surface roughness, and interlocking effects that are critical to the mechanical behavior of many geomaterials and granular assemblies. Consequently, the block type DEM simulation is a better alternative for the simulation as shown in Figure 2. An increasing number of recent studies have adopted block-based or polyhedral elements in DEM simulations to achieve a more realistic representation of particle morphology and to improve the fidelity of numerical predictions.

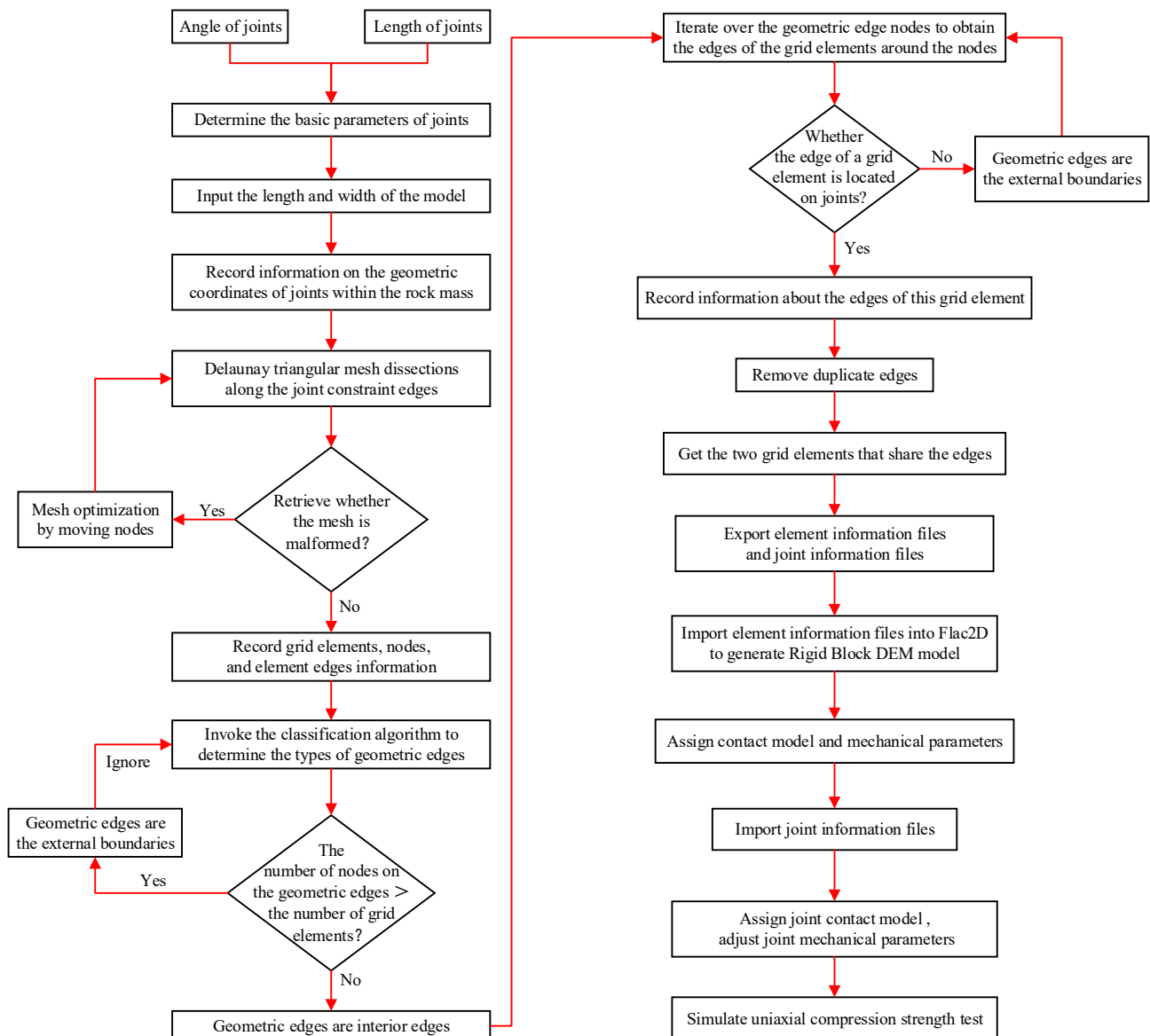


Figure 1. Flow chart for uniaxial compression strength test of rock mass.

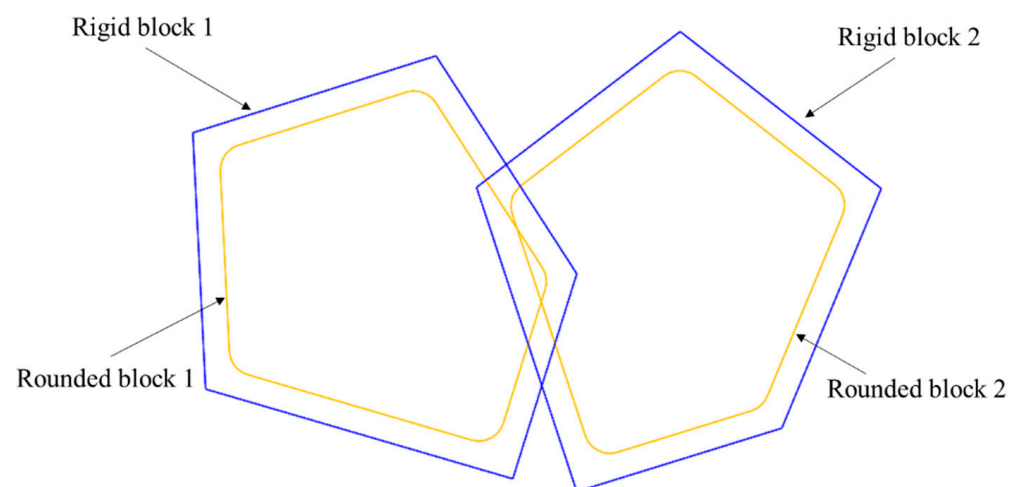


Figure 2. Illustration of the rigid blocks in DEM.

The rock mass modelled by the discrete element method based on rigid blocks consists of a certain number of externally convex rigid blocks [22]. Due to the suitability of

geometrical features for describing polyhedral (three-dimensional) and polygons (two-dimensional), the rigid block discrete element method allows for more accurate rock mass modelling. Rock mass modelling using rigid block discrete elements is simpler and more straightforward. The contact interface between rigid blocks is flat and smooth, which is more suitable for modelling the extensive fracture network in the rock mass.

2.3. Model Generation of Jointed Rock

2.3.1. Generation of Rigid Block

During the DEM simulation using rigid blocks, only the convex shape is supported. In this work, the mesh generation technique for the FEM mesh generation is employed. Afterwards, the element of mesh is translated to the rigid block. For simplicity, the triangle mesh using Delaunay algorithm is used. This approach enables a direct transfer of complex geometries from continuum models into discontinuum frameworks, preserving the original structural features with high fidelity. The resulting rigid blocks can effectively represent angularity, heterogeneity, and discontinuities, thereby improving the realism of contact interactions and failure mechanisms. This work uses the Delaunay Bowyer–Watson algorithm [23] to dissect the grid and tries to optimize the intermediate nodes through a force smoothing process, which usually generates very homogeneous and well-shaped grid elements of high quality. Constrained Delaunay triangulation [24] is an algorithm that extends the Delaunay triangulation algorithm to arbitrary polygonal regions and satisfies additional restrictions. Such algorithms generally need to satisfy specific constraints, e.g., convex polygon boundaries, line segment constraints, point constraints, etc., and in this paper, constrained Delaunay triangulation is performed with a preconfigured cross joint as the boundary.

To ensure numerical stability near the joint interface, local mesh density was controlled during constrained Delaunay triangulation. A minimum allowable edge length was prescribed to prevent the formation of excessively small triangular elements along joint boundaries, which could otherwise lead to numerical singularities or unstable contact forces. After triangulation, a mesh optimization and smoothing procedure was applied to improve element shape quality and eliminate sharp angles or short edges. Preliminary mesh-sensitivity checks indicated that, once the joint geometry was adequately resolved, further mesh refinement had a negligible influence on the macroscopic stress–strain response and crack evolution patterns, while significantly increasing computational cost. Therefore, the adopted mesh density was considered sufficient for the purposes of this study. The steps for its realization are roughly as follows:

- (1) An initial Delaunay triangle is generated based on the four vertices of the pre-specified rectangular region.
- (2) Insert the scattered points in the point set in turn. Find the triangle whose outer circle contains the insertion point (called the triangle of influence of the point) in the triangle chain table. Delete the common edge of the triangle, and connect the insertion point with all the vertices of the triangle of influence. Complete the insertion of a point in the chain table of Delaunay triangles, and loop until all the points are inserted (See Figure 3).
- (3) The intersecting joints in the specimen are used as constraint edges, and for each constraint edge, a Delaunay triangle adjacent to it is found. All the edges in this triangle that do not belong to the constraint edges are deleted and a new edge is inserted between the endpoints of the constraint edges to obtain a new triangle.
- (4) When there are constraint edges that intersect, special treatment is required to divide the constraint edges into two parts with the point of intersection and carry out the process of generating the new triangles separately.

- (5) At this point the constrained Delaunay triangulation for jointed rock is completed.
- (6) For each element, the vertices are adjusted into the format of rigid blocks and the information is written to the input file of the DEM simulation.

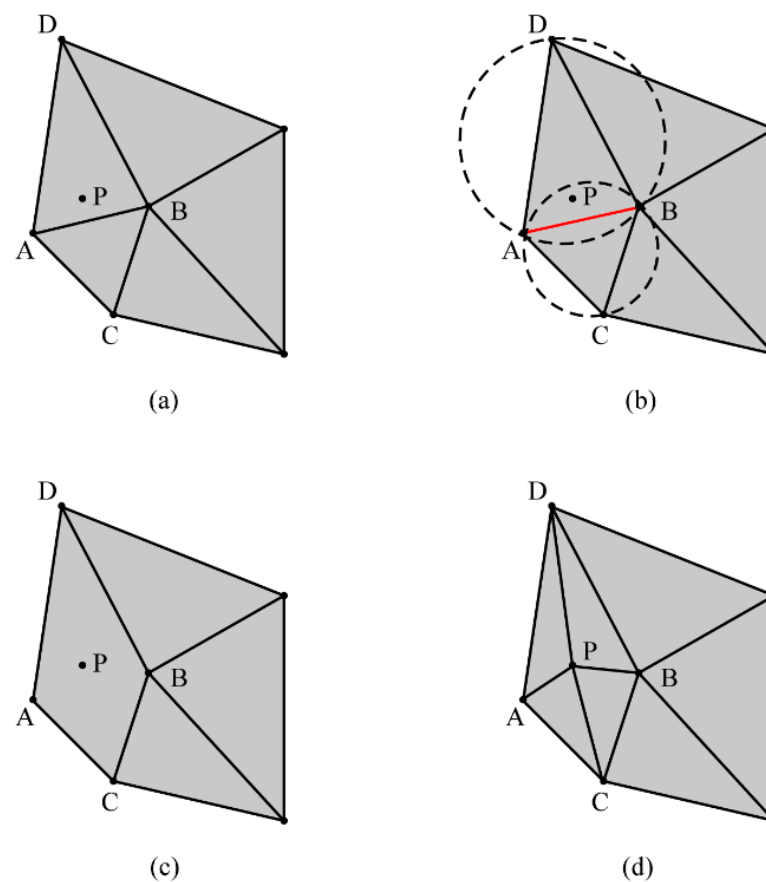


Figure 3. Schematic of point-by-point insertion in the Bowyer–Watson algorithm: (a) Insert new node P; (b) Decide how to connect P to other vertices; (c) Delete edge AB; (d) Triangle formation.

2.3.2. DEM Simulation for Single Joint

Unlike the simulation method of SJM, the simulation of jointed rock mass in this study is realized by grid control. For structurally weak interfaces such as joints, the simulation of realistic joints is approached by transforming them into mesh edges and assigning weaker contact parameters; therefore, the model generation process strictly requires mesh dissections with joint interfaces as constraints. In order to avoid “small edges” and “small corners” affecting the calculation, we also need to smooth the mesh part. In this paper, we refer to Meng et al.’s [25] method of joint mesh generation and refinement, and use self-programming scripts to realize fracture intersection operation for cross joints.

A simple illustration of DEM simulation using rigid blocks is shown in Figure 4. The joint is meshed as a constrained geometry. In the DEM simulation, the mechanical behavior is affected using the contact mechanical parameters. For the edge in the fracture, it is possible to get the rigid blocks at two sides. By setting a weak parameter in for these contacts, the propagation of the cracks can be simulated easily with using other special constitution like SJM.

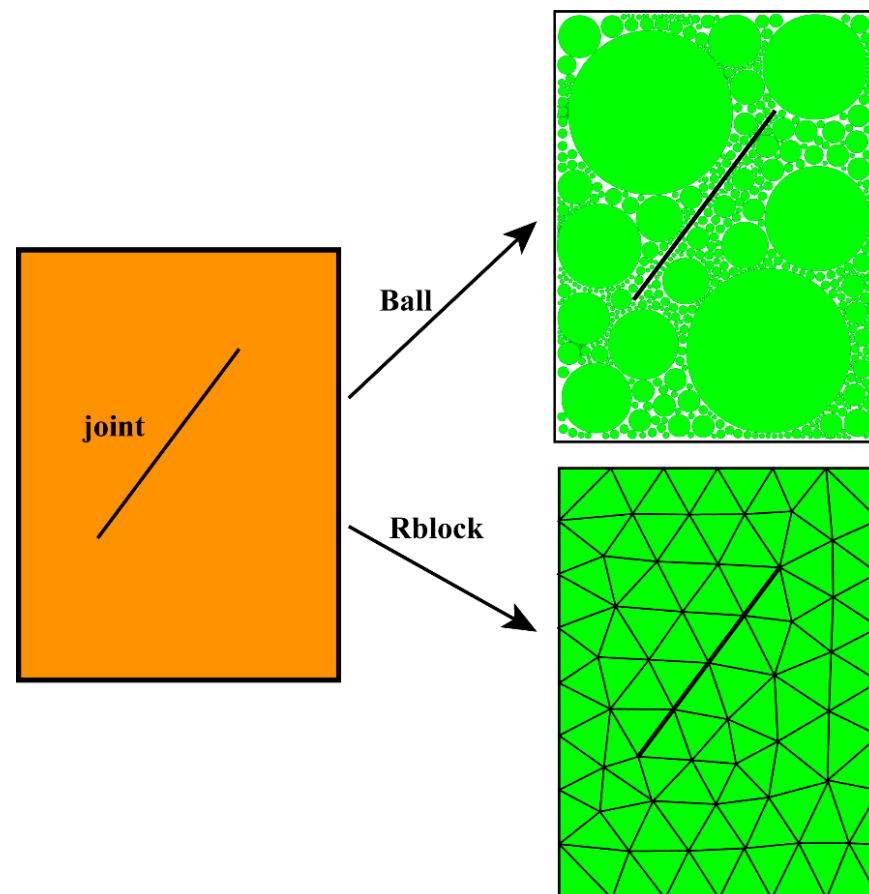


Figure 4. Comparison of Ball and Rigid Block simulated jointed rock mass.

3. Results

3.1. Numerical Simulation Model Generation

To validate the rationality and applicability of the proposed numerical modeling approach, a series of discrete element models for rock masses containing single joints with different orientations and lengths were established. These models were used for parameter calibration and subsequent numerical simulations. For validation purposes, the numerical results were compared with the uniaxial compression experiments on rock specimens containing a single closed flaw conducted by Hui et al. [26]. In the following, the numerical modeling procedure is illustrated by taking a single-joint specimen as a representative example. The numerical model generation process consists of three main steps, as described below.

- (1) In the experimental study conducted by Hui et al. [26], the influence of a single closed flaw on the failure behavior of rock specimens under uniaxial compression was systematically investigated. The specimens were rectangular blocks with dimensions of 100 mm × 50 mm. Two groups of tests were designed. In the first group, the joint inclination angle was fixed at $\alpha = 60^\circ$, while the joint length was varied as 0, 5, 10, 15, 20, and 25 mm. In the second group, the joint length was fixed at $l = 15$ mm, and the joint inclination angle was varied as 15° , 30° , 45° , 60° , and 75° . All specimens were subjected to uniaxial compression loading.
- (2) The polygon coordinate data describing the geometry of the jointed rock specimens were imported into finite element software to generate an initial Delaunay triangulation. A joint restoration algorithm was then applied to ensure that the predefined joint boundaries coincided with mesh edges. Subsequently, a mesh optimization procedure

was performed to improve the overall mesh quality and the geometric accuracy of individual elements. The final output included the triangulated mesh information and the corresponding joint contact pair data.

- (3) The optimized finite element mesh was imported into the discrete element program. Using the built-in rigid block generation command, the finite element mesh was converted into a set of rigid triangular blocks, thereby forming the discrete element model required for numerical simulation.

3.2. Numerical Simulation Results of Uniaxial Compression Tests

In this study, a rigid-block discrete element method (DEM) was employed to simulate the uniaxial compression behavior of jointed rock specimens. The mechanical behavior of the model was governed by a linear parallel bonding model, which requires the specification of refined (microscopic) mechanical parameters to reproduce the macroscopic mechanical response of the rock specimen. Notably, no explicit analytical relationship exists between the refined mechanical parameters and the macroscopic mechanical properties, making direct parameter conversion infeasible.

To ensure that the refined mechanical parameters accurately reproduce the macroscopic mechanical behavior of the specimen, an iterative calibration procedure was adopted prior to the formal numerical simulations. Due to the absence of advanced parameter inversion or optimization algorithms in the present study, a conventional trial-and-error calibration strategy was employed. In this procedure, the refined mechanical parameters were sequentially adjusted until the numerical results exhibited satisfactory agreement with the macroscopic stress–strain response and failure characteristics reported in experimental observations. Once such consistency was achieved, the corresponding parameter set was deemed suitable for subsequent numerical analyses.

It should be noted that the calibration procedure does not aim to identify a unique set of microscopic parameters. Due to the absence of a direct analytical relationship between micro- and macro-scale properties in rigid-block DEM, equifinality may exist, whereby different parameter combinations can produce similar macroscopic responses. This limitation is inherent to discrete element modelling. In this study, parameter calibration focused on achieving a physically reasonable and numerically stable parameter set capable of reproducing multiple macroscopic indicators simultaneously. The subsequent parametric analyses were conducted using the same calibrated parameters to investigate relative trends in strength and fracture behavior, rather than absolute strength prediction.

Owing to practical limitations, including time constraints, testing equipment, and specimen availability, physical model tests were not conducted in this study. Instead, the calibration of refined mechanical parameters was performed by referencing the uniaxial compression experimental results reported by Hui et al. [26]. The final calibrated parameters of the linear parallel bonding model, obtained through iterative numerical adjustment under uniaxial compressive loading conditions, are summarized in Table 1.

Regarding the mechanical parameters of the joint, the rigid-block DEM framework represents joints implicitly as block–block contacts. In principle, the joint contacts share the same constitutive formulation as the intact block contacts, differing primarily in their mechanical strength. Therefore, in the subsequent numerical simulations, the microscopic mechanical parameters of the joints were obtained by applying a strength reduction scheme to the calibrated intact rock parameters. The corresponding joint parameters are also listed in Table 1.

To verify the validity of the calibrated refined mechanical parameters, the numerical simulation results were compared with experimental data. In general, numerical results exhibiting errors within 20% of experimental values are considered acceptable for subsequent

parametric studies [27]. Uniaxial compression simulations were therefore conducted on a numerical specimen containing a single closed joint, and the results were evaluated in terms of crack evolution patterns and stress–strain responses.

Table 1. Calculation of fine-scale mechanical parameters.

Lin_pb Model Parameters	Values	Joint Parameters	Values
E _{mod} (N/m)	4×10^{10}	Reduction factor-a	0.01
Pb _{ten} (N/m)	7×10^8	Pb _{ten} * (N/m)	7×10^6
Pb _{coh} (N/m)	9.1×10^8	Pb _{coh} * (N/m)	9.1×10^6
K _{ratio}	1.2	Reduction factor-b	0.05
Fric	0.2	Fric *	0.005
Dp _n ratio	0.01	Reduction factor-c	0.005
Dp _s ratio	0.01	Pb _{fa} * (°)	0.1
Pb _{fa} (°)	20		

* : coefficient after reduction.

The stress–strain response obtained from the calibrated numerical simulation is shown in Figure 5. The specimen with a joint length of $l = 20$ mm and an inclination angle of $\alpha = 60^\circ$. A high degree of agreement is observed between the numerical and experimental curves. During Stage I, the stress–strain relationship is approximately linear, indicating elastic deformation. With increasing axial strain, the specimen enters Stage II, characterized by nonlinear deformation and rapid crack development. After reaching the peak stress, crack propagation continues in Stage III, eventually leading to the formation of through-going fractures.

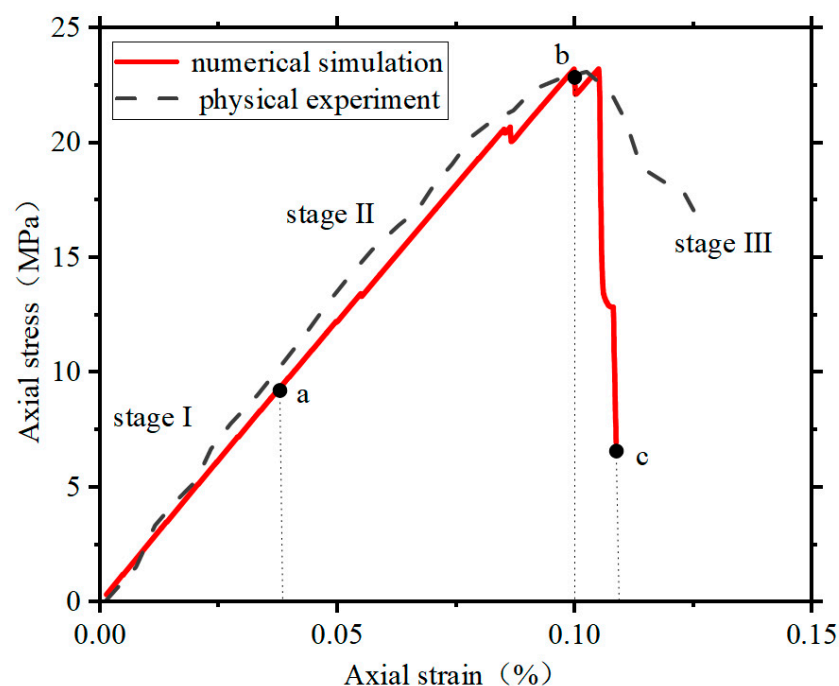


Figure 5. Comparison of stress–strain curves: (a) Microcrack initiation; (b) Stress peak point; (c) Macroscopic failure stability .

As shown in Figure 5, both the numerical and experimental stress–strain curves reach their peak stress at approximately 20 MPa, with a corresponding axial strain of about 1.0%. Moreover, the pre-peak stress–strain evolution exhibits nearly identical trends in both curves. The numerical simulation was terminated when the axial stress decreased to 30% of the peak strength, consistent with the adopted failure criterion.

Based on the close agreement between the numerical simulations and experimental observations in terms of crack evolution and stress–strain behavior, it can be concluded that the calibrated refined mechanical parameters are reasonable. Furthermore, the rigid-block DEM framework adopted in this study is demonstrated to be effective for simulating the uniaxial compression behavior of rock specimens containing a single joint.

4. Discussions

4.1. Effect of Joint Inclination

Previous studies have demonstrated that the mechanical behavior of jointed rock masses is governed by multiple joint-related factors, including joint length, inclination, number, spacing, and connectivity. Among these factors, joint inclination plays a particularly critical role in controlling strength and failure behavior [28]. However, many existing numerical studies rely on particle-based models composed of spherical elements, which may limit the accurate representation of joint geometry. In the present study, the Rigid Block discrete element approach combined with the algorithm was employed to generate jointed rock models with explicitly represented joint geometry.

A total of 11 numerical models were constructed with joint inclination angles ranging from 0° to 165° , using an increment of 15° . The case of $\alpha = 180^\circ$ was not considered, as it yields identical results to $\alpha = 0^\circ$ due to geometric symmetry. The specimen dimensions were fixed at $150 \text{ mm} \times 200 \text{ mm}$, and the length of the single joint was maintained at 40 mm for all simulations. Representative numerical models are shown in Figure 6, where six typical joint inclinations are presented for clarity.

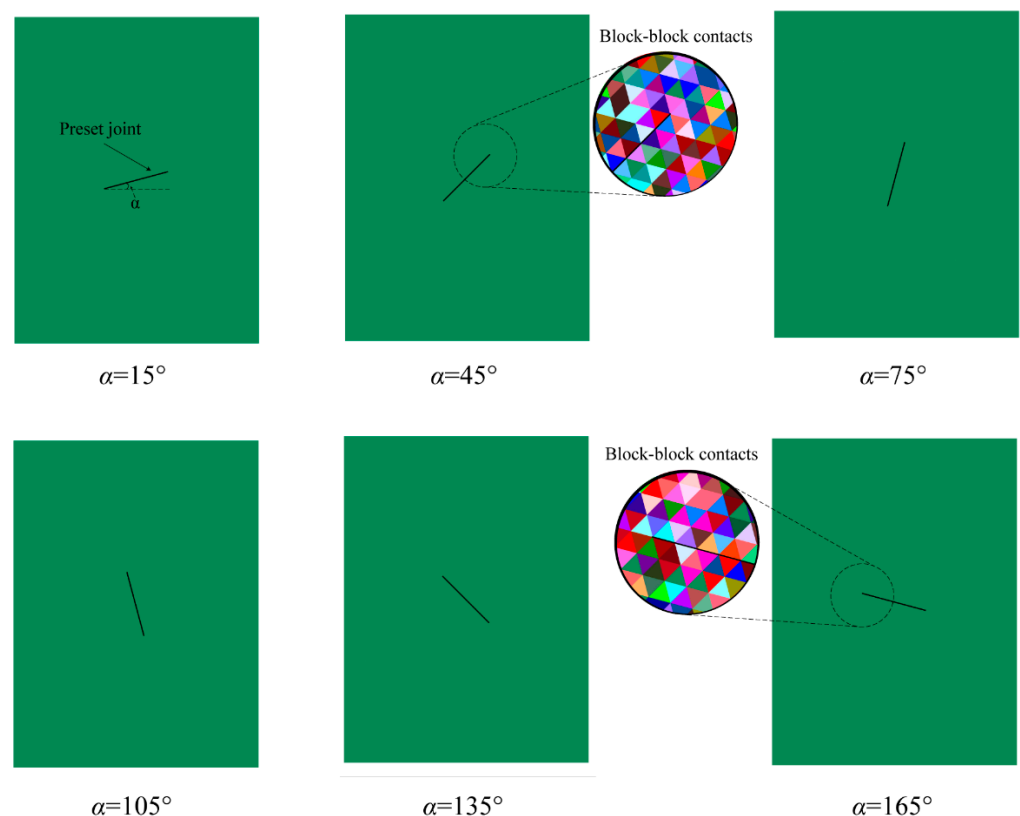


Figure 6. Joint rock mass models with different inclinations.

Figure 7 presents the axial stress–strain responses of specimens with different joint inclinations under uniaxial compression. The compressive strength reaches a minimum at a joint inclination of approximately $\alpha = 45^\circ$, which can be identified as the critical inclination

angle under the present modeling conditions. The corresponding crack evolution process is illustrated in Figure 8, with the case of $\alpha = 15^\circ$ selected as a representative example. Despite variations in joint inclination, the overall fracture process exhibits a consistent three-stage failure pattern.

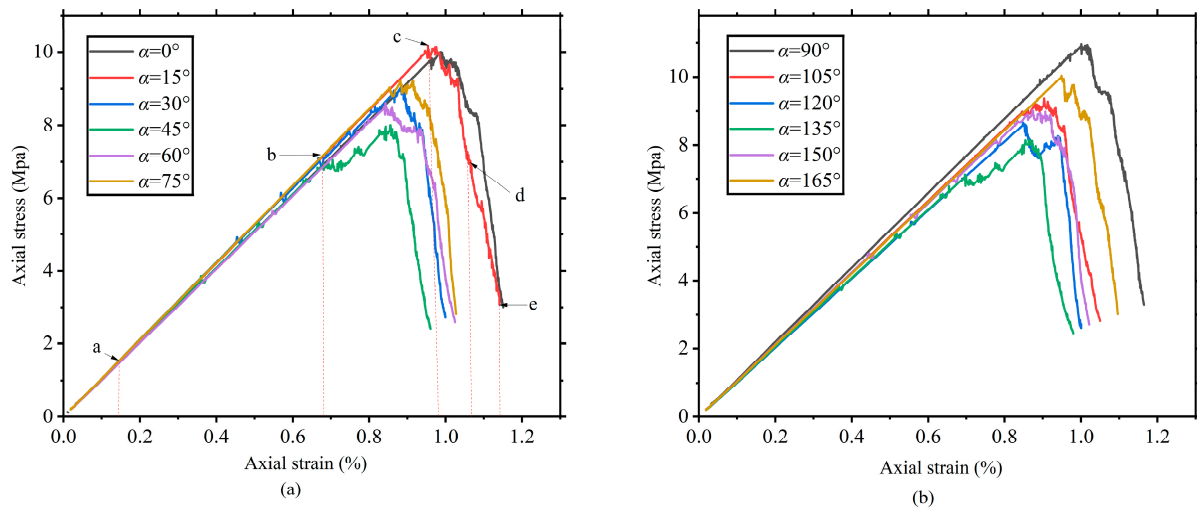


Figure 7. Comparison of stress–strain curves of rock specimens under uniaxial compression with different fracture inclinations: (a) 0° – 75° stress–strain curve; (b) 90° – 165° stress–strain curve. (a: Microcrack appear; b: Crack extension, polymerization; c: Tensile crack appears; d: Macro destruction; e: Stable rupture).

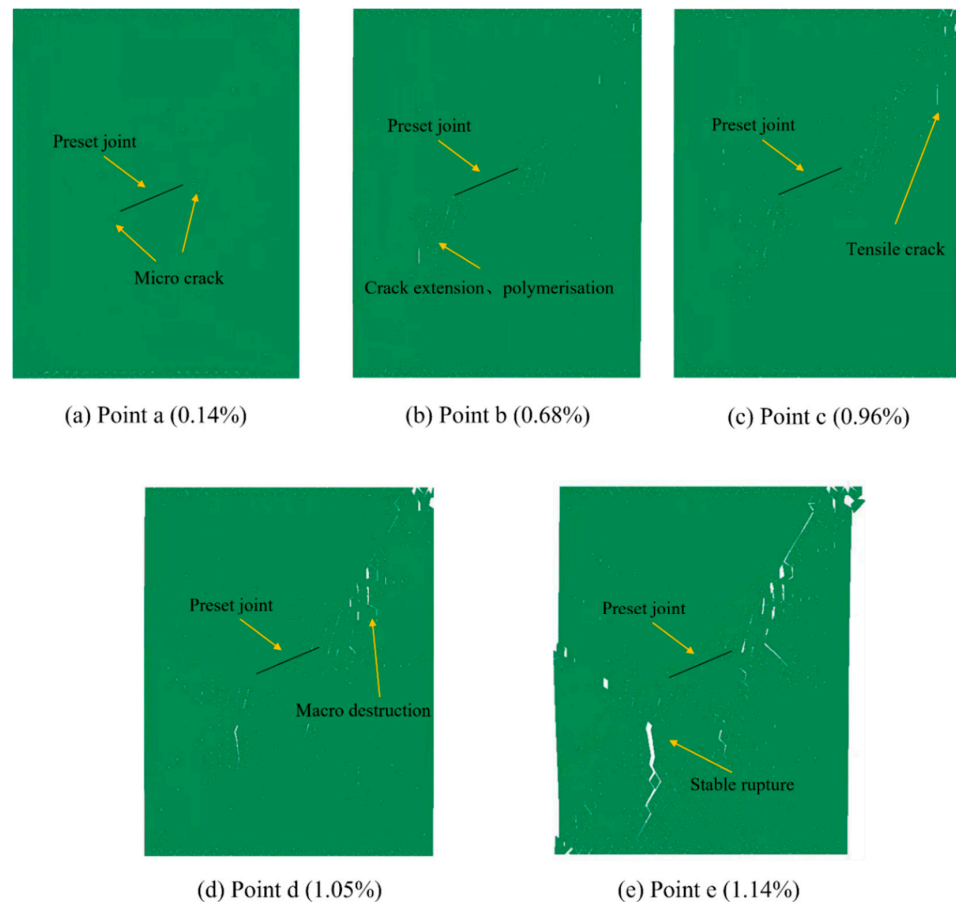


Figure 8. Crack development in joint rock mass under UCS: (a) Microcrack appear; (b) Crack extension, polymerization; (c) Tensile crack appears; (d) Macro destruction; (e) Stable rupture ($\alpha = 15^\circ$).

In Stage I (crack initiation stage), microcracks first appear near the tips of the pre-existing joint under increasing axial stress, indicating strong stress concentration effects at the joint endpoints. In Stage II (stable crack propagation stage), microcracks progressively propagate, coalesce, and extend along preferred paths governed by the joint orientation and the local stress field. In Stage III (post-peak failure stage), crack growth becomes unstable, leading to the formation of through-going fractures and macroscopic failure of the specimen. The transition from tensile-dominated wing cracking to shear-dominated sliding with increasing joint inclination is progressive rather than abrupt. This evolution reflects the gradual transformation of axial compressive stress into shear stress along the joint plane. Although contact parameters such as stiffness ratio and friction coefficient influence the detailed failure mode, these parameters were kept constant to isolate the effect of joint geometry.

Although the failure stages remain qualitatively similar for different joint inclinations, joint inclination significantly influences the stress–strain response and peak strength of the specimens. As shown in Figure 7, the axial stress increases monotonically with axial strain prior to reaching the peak stress, while the strain corresponding to peak stress varies with joint inclination. This behavior suggests that joint inclination alters the stress redistribution pattern and crack propagation path, thereby affecting both the deformation capacity and the strength of the jointed rock specimen.

4.2. Effect of Joint Length

To investigate the influence of single-joint length on the mechanical behavior of rock specimens under uniaxial compression, numerical simulations were performed for joint lengths of $l = 30, 40, 50$, and 60 mm. All specimens had identical dimensions of $150 \text{ mm} \times 200 \text{ mm}$ and a fixed joint inclination angle of $\alpha = 45^\circ$, ensuring that the effect of joint length could be examined independently. The numerical model configuration and joint layout are illustrated in Figure 9.

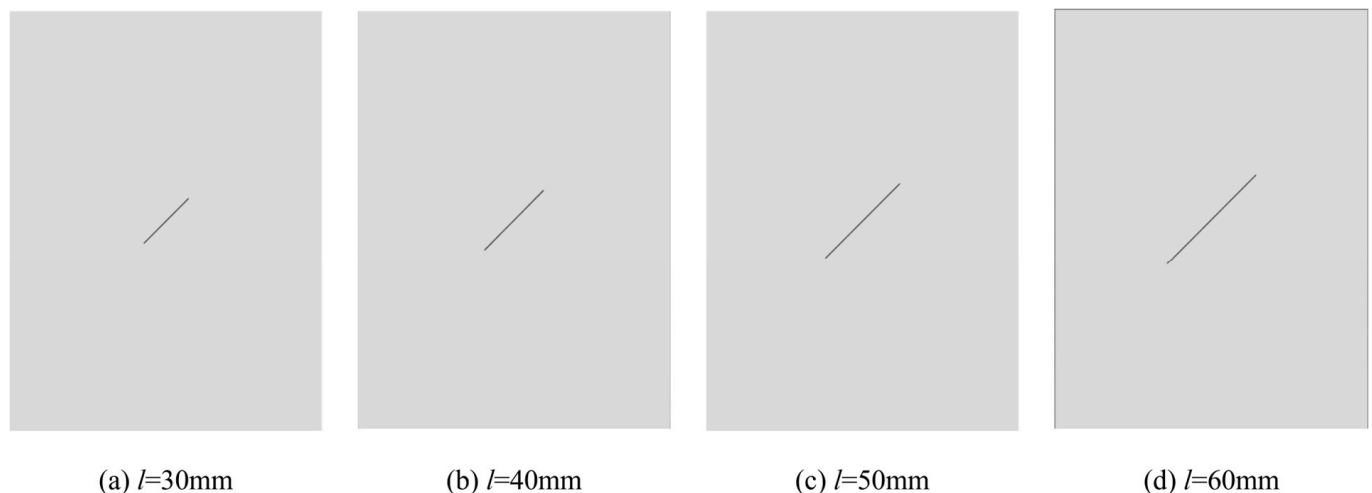


Figure 9. Model joint setting diagram.

To reduce the influence of mesh dependency inherent in rigid-block DEM simulations, a statistical simulation strategy was adopted. For each joint length, 12 independent numerical models were generated by randomly varying the mesh seed size within a prescribed range of $0.5\text{--}5.0$ mm. All models were subjected to uniaxial compression under identical loading conditions, and the peak uniaxial compressive strength (UCS) was recorded.

The simulation results indicate (Figure 10) that joint length plays a significant role in controlling stress redistribution, crack initiation, and failure development. For all

joint lengths, the failure process follows a consistent three-stage pattern: (1) crack initiation at joint tips due to stress concentration, (2) stable crack propagation accompanied by progressive stiffness degradation, and (3) post-peak coalescence of cracks leading to macroscopic failure.

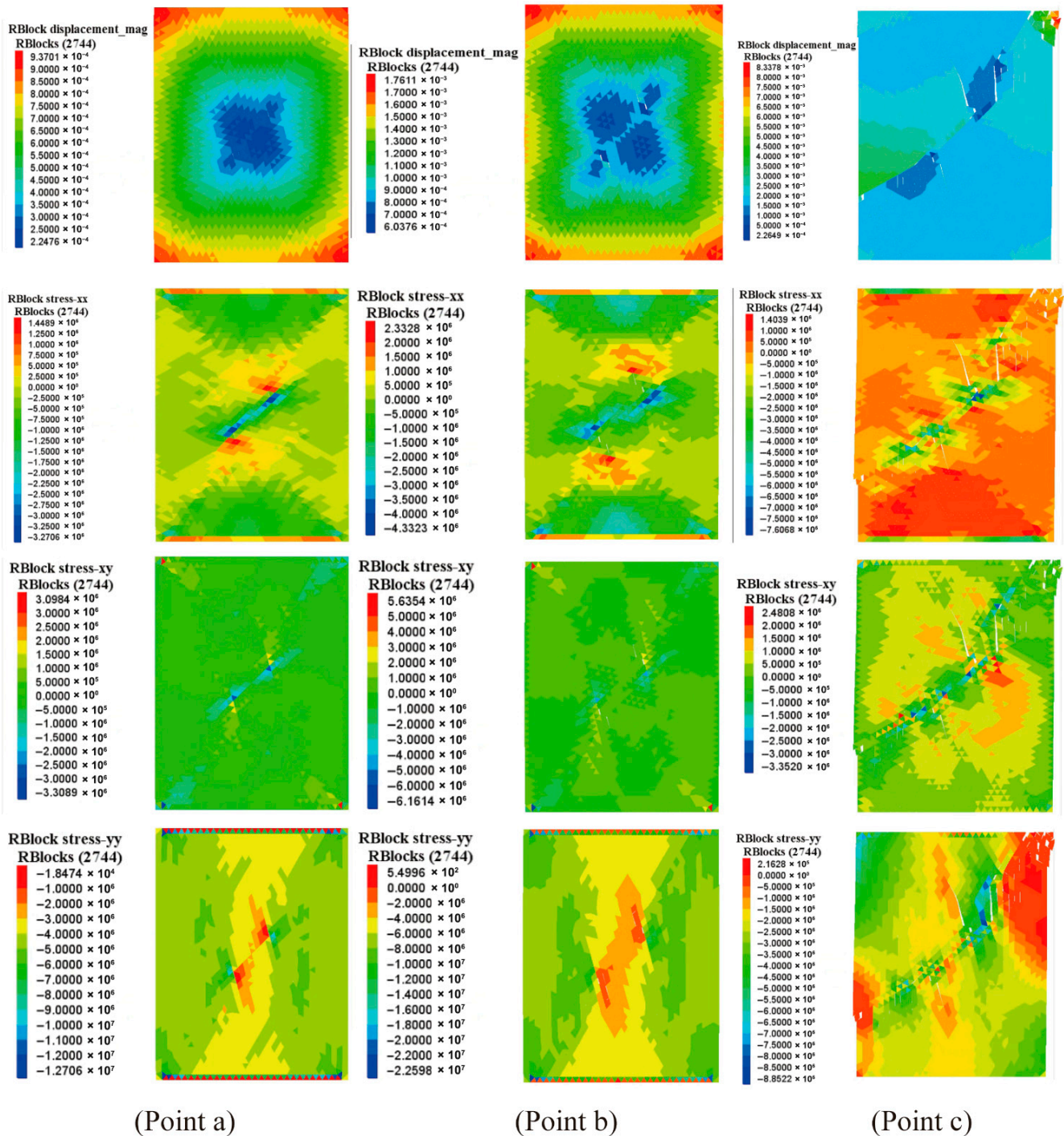


Figure 10. Displacement and stress evolution of the rock specimen containing a single flaw with a length of $l = 40$ mm.

Figure 11 presents the statistical distribution of UCS values for different joint lengths using box plots. Although individual realizations exhibit scatter due to mesh randomness, a clear monotonic decreasing trend of UCS with increasing joint length is observed in terms of the median and overall distribution. As the joint length increases, the effective load-

bearing area of the specimen is progressively reduced, facilitating earlier crack coalescence and resulting in lower compressive strength.

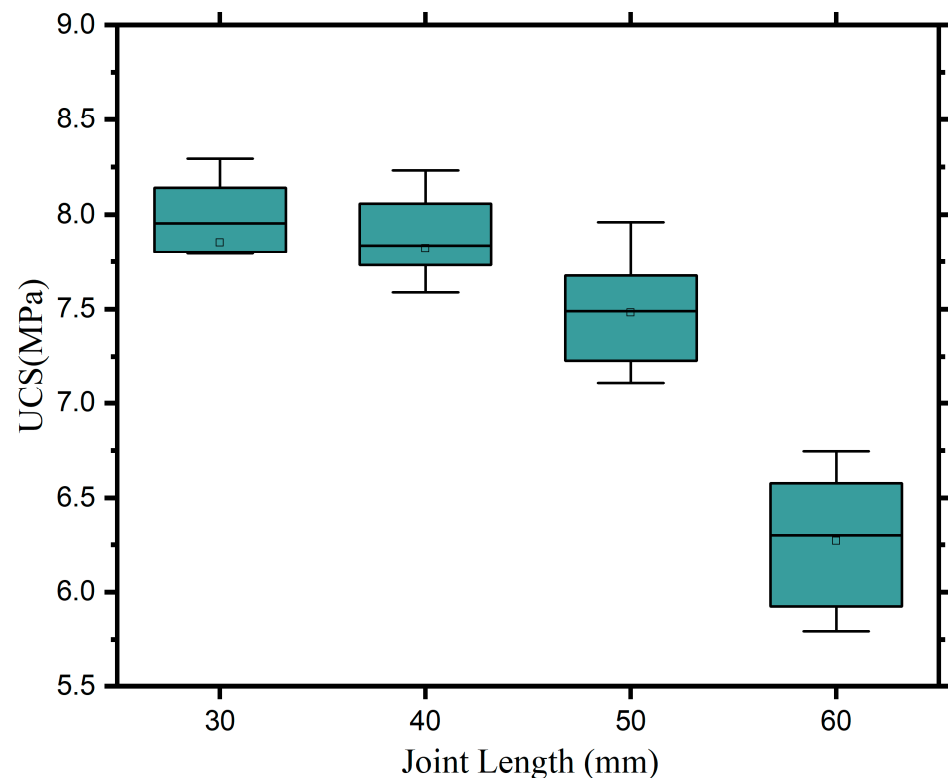


Figure 11. Statistical distribution of UCS for models with different joint lengths.

It should be noted that a slightly higher UCS value may appear in individual realizations at $l = 40$ mm compared to $l = 30$ mm. However, this non-monotonic behavior does not represent a physical strengthening mechanism associated with joint length. Instead, it is attributed primarily to local mesh density variations and the discrete nature of block contacts in DEM simulations, which can temporarily alter stress transfer paths at the microscale. When considering multiple realizations, this apparent anomaly disappears, and no critical joint length corresponding to a strength maximum can be identified.

Overall, the statistical results confirm that increasing joint length leads to a systematic reduction in UCS, which is consistent with fundamental rock mechanics principles and fracture mechanics expectations. The present findings highlight the importance of ensemble-based analysis when interpreting DEM simulation results involving geometric discontinuities.

4.3. Effect of the Joint Width

To investigate the effects of single-joint width on the failure patterns and mechanical strength of the specimens, three groups of numerical simulations were conducted. In the rigid-block DEM framework, joint width is explicitly modeled as a geometric void between adjacent blocks, and no load is transmitted directly across the joint gap. The joint widths are set to $d = 1$ mm, 3 mm, and 5 mm (Figure 12), respectively. For all specimens, the model dimensions were maintained at 150 mm $\times$ 200 mm, and the inclination angle of the single joint was fixed at $\alpha = 45^\circ$.

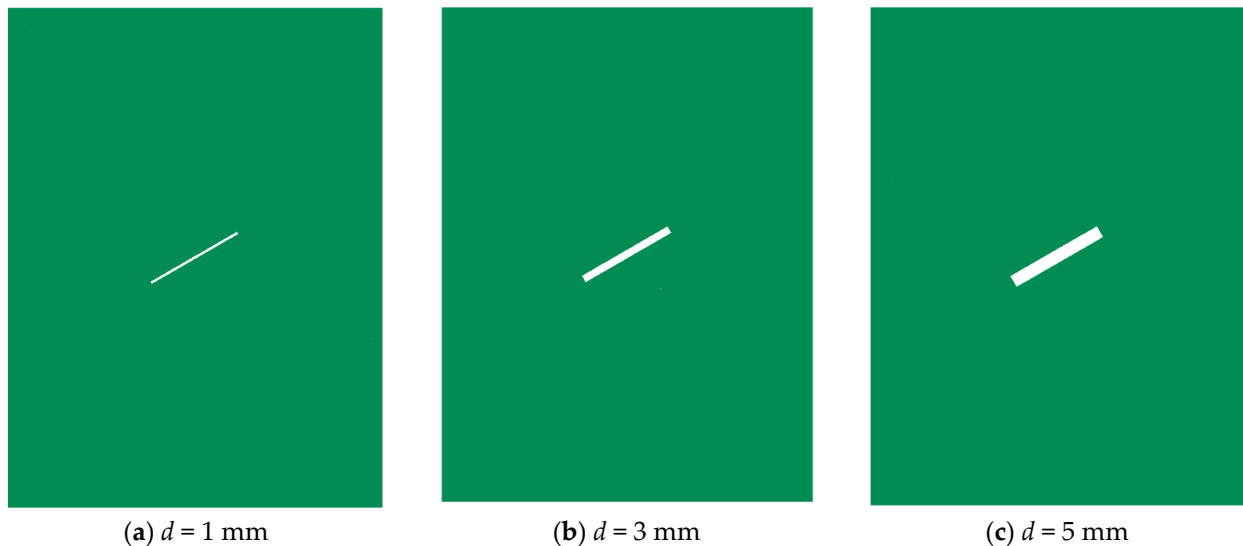


Figure 12. Rock mass with different fracture width.

Figure 13 illustrates the stress–strain curves of rock specimens containing a single joint with different widths under uniaxial compression. The horizontal axis represents the axial strain ϵ of the specimen, while the vertical axis denotes the corresponding axial stress σ . The joint widths considered are $d = 1$ mm, 3 mm, and 5 mm.

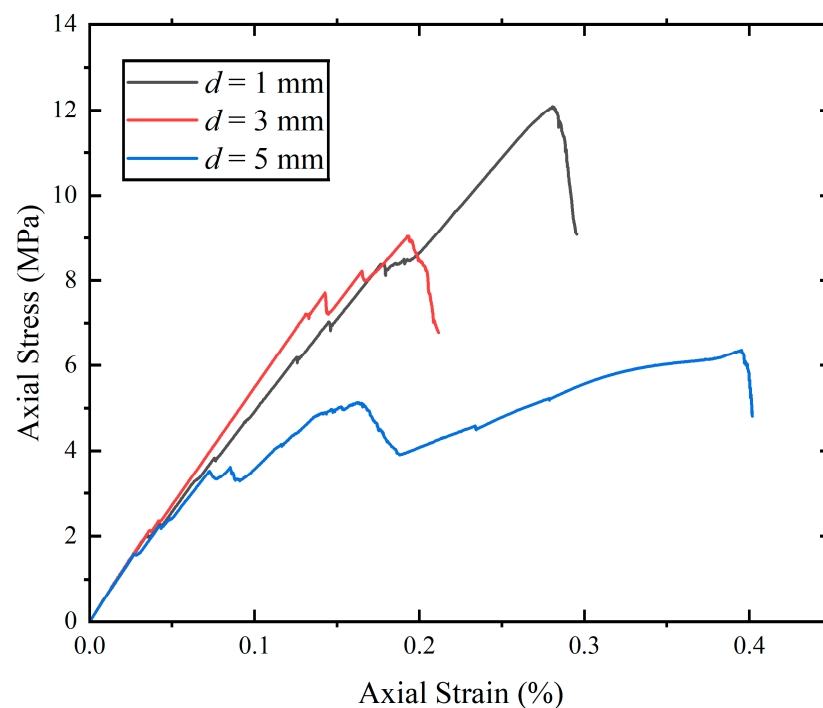


Figure 13. Stress–strain curves of rock specimens with different joint widths under uniaxial compression.

As discussed previously, the failure process of the specimens can be divided into three distinct stages. In Stage I, microcracks initiate at both tips of the pre-existing joint under uniaxial compression, accompanied by the development of tensile and shear crack branches. In Stage II, with continuous loading, these microcracks propagate in a stable manner, gradually extending and coalescing. During this stage, tensile and shear cracks become clearly distinguishable, and macroscopic failure occurs when the axial stress reaches its peak value. In Stage III, after the peak stress is exceeded, the axial stress decreases

with increasing strain, and crack propagation tends to stabilize, with macroscopic fractures mainly developing along the previously formed failure paths.

In addition, the stress–strain responses indicate a clear dependence of mechanical strength on joint width. It should be emphasized that the strength discussed here represents the numerically obtained peak stress under uniaxial compression, reflecting the combined effects of contact weakening and reduced interlocking induced by increased joint width, rather than an analytical strength derived from theoretical fracture criteria. The specimen with a joint width of $d = 1$ mm exhibits the highest peak stress and stiffness, indicating stronger load-bearing capacity. When the joint width increases to $d = 3$ mm, the peak stress decreases moderately, while the post-peak behavior becomes more abrupt, suggesting a reduction in structural stability. For $d = 5$ mm, the peak stress is significantly lower, and the specimen shows pronounced strain softening and a more ductile post-peak response. Overall, the peak axial stress decreases monotonically with increasing joint width, demonstrating that wider flaws weaken the mechanical strength of the rock specimen and promote earlier failure under uniaxial compression.

Figure 14 shows the displacement distributions of rock specimens containing a single joint with different joint widths at the peak stress under uniaxial compression. As the joint width increases, both the magnitude and the spatial extent of displacement around the joint become more pronounced. For $d = 1$ mm, displacement is mainly concentrated near the joint tips, and the deformation remains relatively localized, indicating a higher load-bearing capacity. When the joint width increases to $d = 3$ mm, the displacement concentration zone expands along the loading direction, accompanied by enhanced crack propagation and interaction. For $d = 5$ mm, large displacements are widely distributed throughout the specimen, suggesting severe damage and extensive crack coalescence prior to failure. Correspondingly, the peak axial stress decreases from 1.21×10^7 Pa ($d = 1$ mm) to 9.04×10^6 Pa ($d = 3$ mm) and further to 6.37×10^6 Pa ($d = 5$ mm). These results indicate that increasing joint width significantly weakens the mechanical strength of the rock specimen and promotes larger deformation and earlier macroscopic failure under uniaxial compression.

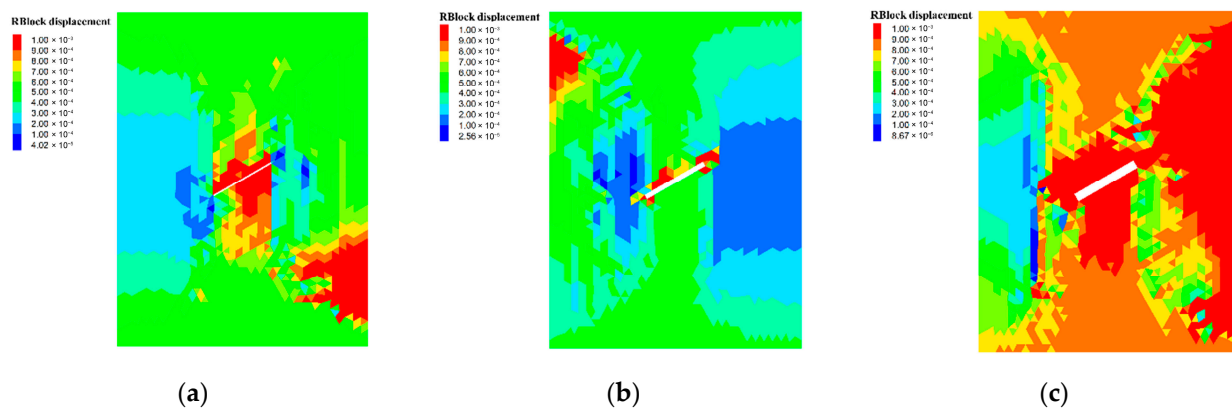


Figure 14. Distribution of displacement at the at the peak stress. (a) $d = 1$ mm; (b) $d = 3$ mm; (c) $d = 5$ mm.

5. Conclusions

Based on a rigid-block discrete element modeling framework, this study systematically investigated the mechanical behavior, crack evolution, and failure mechanisms of rock specimens containing a single pre-existing joint under uniaxial compression. The effects of joint inclination and joint length on strength and fracture behavior were analyzed in detail. It should be noted that the strength discussed in this study represents the peak

macroscopic stress obtained from discrete element simulations, serving as a numerical indicator of load-bearing capacity rather than an analytical strength derived from fracture mechanics theory. In this study, crack initiation and propagation are not controlled by predefined analytical fracture criteria or characteristic fracture-cell parameters; instead, they arise naturally from stress concentration and progressive contact failure within the discrete element framework. The main conclusions can be summarized as follows:

Single-joint rock specimens exhibit a consistent three-stage failure process regardless of joint geometry. The failure process can be divided into crack initiation, stable crack propagation, and post-peak failure stages. Microcracks preferentially initiate at joint tips due to stress concentration, followed by crack coalescence and the formation of through-going fractures that govern macroscopic failure.

Joint inclination significantly influences stress redistribution and deformation behavior but does not alter the overall failure mode. Although specimens with different joint inclinations display similar qualitative fracture patterns, joint inclination affects the stress–strain response and peak strength by modifying crack propagation paths and local stress concentration characteristics.

Joint length has a pronounced effect on the peak strength of single-joint rock specimens. Statistical analysis of multiple numerical realizations indicates that the uniaxial compressive strength generally decreases with increasing joint length. Minor non-monotonic fluctuations observed in individual simulations are attributed to mesh-induced numerical variability rather than a physical strengthening mechanism. Overall, increasing joint length reduces the effective load-bearing area and promotes crack coalescence, resulting in lower macroscopic strength.

Joint width exerts a pronounced control on the mechanical strength and deformation characteristics of single-jointed rock specimens. With increasing joint width, peak strength and stiffness decrease monotonically, while displacement localization and crack coalescence become more extensive, leading to earlier failure and a more pronounced post-peak softening behavior. The conclusions regarding strength variation are based on numerical peak-stress responses obtained from DEM simulations and are intended to provide comparative and mechanistic insights, rather than theoretical strength calculations within classical fracture mechanics frameworks.

Single-joint specimens serve as a fundamental mechanical unit for understanding more complex jointed rock masses. The insights gained from the simplified single-joint configuration provide a mechanistic basis for interpreting fracture behavior in rock masses with multiple joints and complex joint networks.

From an engineering perspective, the results of this study provide useful insights into the role of joint geometry in controlling strength degradation and failure mechanisms of jointed rock masses. The identified effects of joint inclination, length, and width can assist engineers in recognizing unfavorable discontinuity configurations that may lead to premature failure under compressive loading. In practical applications such as tunnel excavation, slope stability assessment, and foundation design, the qualitative trends revealed by this study may support risk evaluation and the selection of appropriate reinforcement or excavation strategies. Although the present numerical results are model-dependent, they contribute to a better mechanistic understanding of joint-controlled fracture behavior in rock engineering.

It should be noted that the present study is limited to two-dimensional numerical simulations under uniaxial compression conditions. Future research will extend the proposed approach to three-dimensional models and more complex joint networks, while also considering different loading paths. In addition, coupled parametric analyses will be conducted to investigate the combined influence of joint geometry parameters on macroscopic

strength and failure modes, thereby further improving the applicability of the method to practical rock engineering problems.

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