

Article

Experimental Study on Wave Propagation Across Saturated Rock with Different Contact Area Ratios of Joints Under Combined Static–Dynamic Loading

Yunmin Wang ¹, Xin Liu ^{1,2,3,4,*} , Xunjie Hu ^{2,4,*}, Zhenyang Xu ^{2,4} and Hongliang Tang ⁵ 

¹ State Key Laboratory of Safety and Health for Metal Mines, Sinosteel Maanshan General Institute of Mining Research Co., Ltd., Maanshan 243000, China

² School of Mining Engineering, University of Science and Technology Liaoning, Anshan 114051, China

³ Key Laboratory of Ministry of Education on Safe Mining of Deep Metal Mines, Northeastern University, Shenyang 110819, China

⁴ Engineering and Technology Research Center of High-Efficiency Mining-Processing and Utilization of Metal Mineral Resources, Anshan 114051, China

⁵ China Safety Technology Research Academy of Ordnance Industry, Beijing 100053, China

* Correspondence: xinliu0412@ustl.edu.cn (X.L.); hu_xunjie@163.com (X.H.)

Abstract

Underground saturated jointed rock is prone to engineering geohazards under the combined effects of in situ stress and dynamic loading. A modified split Hopkinson pressure bar (SHPB) system was used to conduct dynamic loading tests on artificially fabricated saturated jointed rocks. The effects of joint matching coefficient (JMC) and confining pressure on the dynamic strength, deformation characteristics, energy evolution, and stress wave propagation of the specimens were investigated. The test results show that the dynamic compressive strength and stiffness of saturated jointed rocks increase with the increase in JMC, but the compressive strength is still lower than the typical dynamic strength range. Rock damage mainly occurs at the joint location, and the damage mode is dominated by tensile fracture. In terms of energy, the energy dissipation rate of the rock decreases with decreasing JMC and increasing confining pressure. The propagation of stress waves is mainly affected by the coupling of JMC and three-dimensional static stress, which is manifested as a transition from a rapidly changing phase to an unstable changing phase, a process accompanied by an energy distribution mechanism. These insights fill a gap in the mechanical response of saturated jointed rocks under complex loading conditions underground and help predict the risk of dynamic instability in underground engineering and mining operations.

Keywords: rock mechanics; combined static-dynamic loading; joint matching coefficients; saturated jointed rock; stress wave propagation



Received: 27 October 2025

Revised: 29 January 2026

Accepted: 4 February 2026

Published: 9 February 2026

Copyright: © 2026 by the authors.

Licensee MDPI, Basel, Switzerland.

This article is an open access article distributed under the terms and conditions of the [Creative Commons Attribution \(CC BY\)](https://creativecommons.org/licenses/by/4.0/) license.

1. Introduction

With the expansion of underground engineering projects, such as deep mining and tunnel excavation, into complex geological environments, saturated jointed rock is highly susceptible to engineering geological hazards, including instability and collapse, under the coupled action of in situ static stresses and dynamic loads such as blasting vibrations and mechanical impacts. These hazards pose severe threats to construction safety and engineering stability. The presence of water softens the rock matrix through physico-chemical processes [1–3]. Simultaneously, joints, acting as weak planes within the rock

mass, significantly alter stress transfer paths, energy dissipation patterns, and deformation characteristics, while also serving as critical channels for groundwater storage and flow. Consequently, the mechanical response mechanisms of saturated jointed rock masses under complex loading conditions constitute a pivotal unresolved problem for the safety prevention and control of underground engineering.

Extensive research has been conducted on the dynamic mechanical behavior of saturated rocks, laying a foundation for understanding the water–rock interaction mechanism. Studies have shown that compared with dry rocks, saturated rocks exhibit lower compressive strength, tensile strength, and elastic modulus and exhibit higher friability [4–6]. To simulate the underground three-dimensional stress environment, triaxial compression tests are widely used to investigate the effect of confining pressure on saturated rocks. Zhou et al. [7] demonstrated that the failure mode of rocks under triaxial loading is mainly shear failure, rather than tensile failure under uniaxial impact loading. Wu et al. found that both the strength and deformation of saturated rocks increase with confining pressure, following a linear functional relationship. Wu et al. [8] have found that both the strength and deformation of saturated rock increase with confining pressure, following a linear functional relationship. Wang et al. [9] revealed that confining pressure inhibits the propagation of microcracks inside rocks, and the inhibition degree increases with the rise of confining pressure. Li et al. [10] observed that under triaxial compression, the cohesion and internal friction angle of saturated rocks decrease, while Poisson's ratio increases. Cai et al. [11] noted that under the same prestress and dynamic load, saturated rocks have lower dynamic strength, consume less energy before failure, and dissipate more energy in the post-peak stage. Overall, existing studies on saturated rocks mostly focus on the dry/saturated state or the effects of single confining pressure and single load type, lacking systematic investigation of the multi-factor coupling effect of water saturation and combined loading.

Furthermore, as ubiquitous structural planes in rock, the influence of joint morphological characteristics cannot be neglected. The joint matching coefficient (JMC), which reflects the degree of joint matching by comparing the effective contact area of joints, provides a novel approach for analyzing the morphological features of jointed rock [12]. It is well known that when stress waves encounter a joint plane, the joint interfaces cause wave reflection, propagation, and attenuation [13–15]. The patterns governing these phenomena are closely related to the degree of joint matching. Chen et al. [16] proposed the CIM-JMC model from a theoretical perspective to describe the wave propagation law at joints and quantify reflection and transmission coefficients. However, this model was derived only based on dry joints, without considering the effect of water saturation on the mechanical properties of joint interfaces. Li et al. [17] experimentally analyzed the influence of JMC on stress wave propagation using the SHPB system, but the research object was dry-jointed rock masses, excluding the water-saturated state, thus failing to reflect the coupling mechanism of water–joint–stress wave in underground engineering. In addition, most existing JMC-related studies focus on single dynamic load or static load conditions, failing to cover the combined load scenario of preloading and dynamic impact in underground engineering, making it difficult to reveal the joint regulatory mechanism under actual engineering conditions.

Although scholars have carried out extensive work on the mechanical behaviors of saturated rocks and jointed rock masses separately, significant research gaps remain, mainly reflected in three aspects. First, existing studies mostly focus on single load type or binary factor coupling, lacking systematic analysis on the ternary coupling effect, which cannot clarify the quantitative influence laws of the three factors on rock mass strength and stiffness. Second, most studies on jointed rock masses are based on dry conditions, ignoring the effect of water saturation on the contact characteristics and softening of joint interfaces.

Third, existing studies are mostly limited to macro-mechanical parameter testing, lacking correlation analysis between mesoscopic damage evolution and macroscopic mechanical response, making it difficult to reveal the failure essence of saturated jointed rock masses.

In this study, artificially prepared saturated jointed granite was used as the research object. By cutting symmetric grooves on intact granite, artificial jointed granite specimens with three JMC (0.324, 0.481, 0.712) were fabricated. A modified SHPB system was employed to apply dynamic loads, combined with 9 MPa axial pre-stress and 1–5 MPa confining pressure, to simulate the underground stress environment. Nuclear Magnetic Resonance (NMR) and three-dimensional surface-scanning technologies were used to characterize the mesoscopic damage and surface morphological characteristics of the specimens. The attenuation mechanism of stress wave propagation under the coupling effect of internal and external factors was clarified. A ternary coupling experimental system was established, integrating macro-mesoscopic testing technologies to achieve direct correlation between macro-mechanical parameters and mesoscopic mechanisms. Dynamic instability early warning indicators based on stress wave propagation characteristics were established, enhancing the engineering applicability of the research results. The findings are expected to address critical knowledge gaps in the mechanical response of saturated jointed rock masses under complex loading and provide theoretical support for predicting the risk of dynamic instability in underground engineering.

2. Experimental Specimen and Process

2.1. Specimen Preparation

The granite specimens used in the experiments were collected from a mining area in Rizhao City, Shandong Province (geographical location shown in Figure 1). To minimize the influence of material heterogeneity on test results, all samples were cored from the same rock block. After extraction, the specimens were immediately wrapped in plastic film to prevent weathering and moisture loss. Only intact specimens without visible defects or surface cracks were selected for testing. Polarized light microscopy analysis indicated that the granite was primarily composed of quartz (54%), K-feldspar (23%), plagioclase (15%), biotite (5%), and other minor constituents (Figure 1). Two groups of specimens were prepared: the first group with 50 mm (diameter) × 100 mm (height) for determining basic mechanical parameters and the second group with 50 mm (diameter) × 50 mm (height) for preparing jointed specimens.

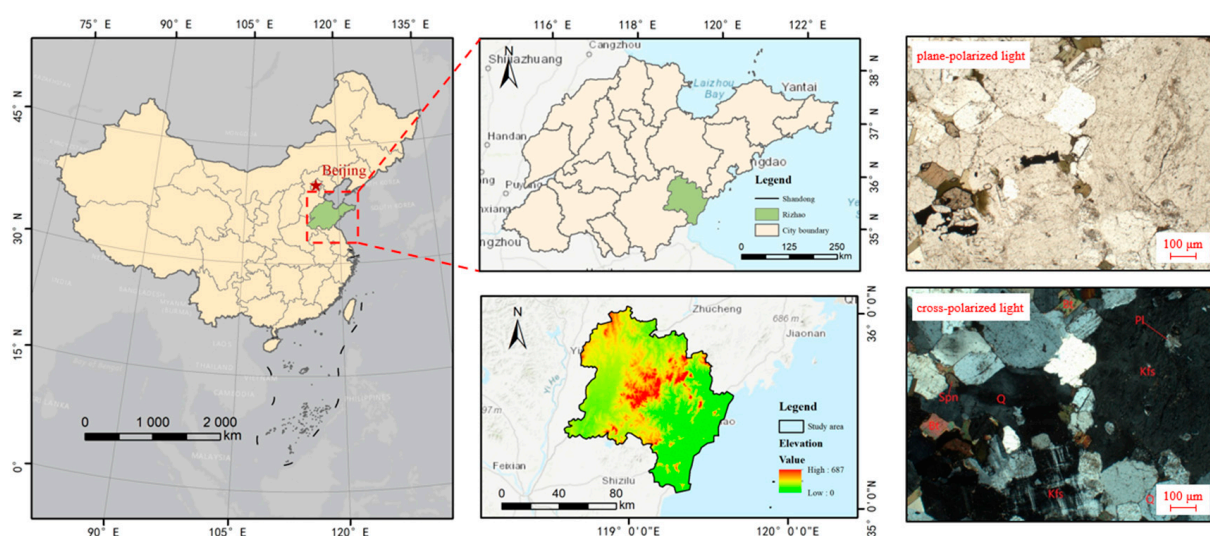


Figure 1. Rock location, thin section authentication, and mineral composition.

The mechanical properties of dry and saturated granite specimens were determined at Henan Polytechnic University by using an electro-hydraulic servo-controlled rock mechanics testing system (RMT-150B, Wuhan Institute of Rock and Soil Mechanics, Wuhan, China) with a precision of 0.002 mm/s. The test results are presented in Table 1 and Figure 2.

Table 1. Mechanical parameters of dried and saturated granite.

Condition	Specimens	Peak Stress (MPa)	Peak Strain	Elastic Modulus (Gpa)	Poisson's Ratio
Dried specimen	USC-D-1	168.455	0.004560	46.246	0.278
	USC-D-2	156.569	0.005474	43.649	0.274
	USC-D-3	159.402	0.005441	41.063	0.276
	Average value	161.475	0.005158	43.653	0.276
Saturated specimen	USC-S-1	136.871	0.008053	19.688	0.141
	USC-S-2	165.603	0.009994	20.272	0.195
	USC-S-3	149.335	0.010564	18.787	0.161
	Average value	150.603	0.009537	19.582	0.166

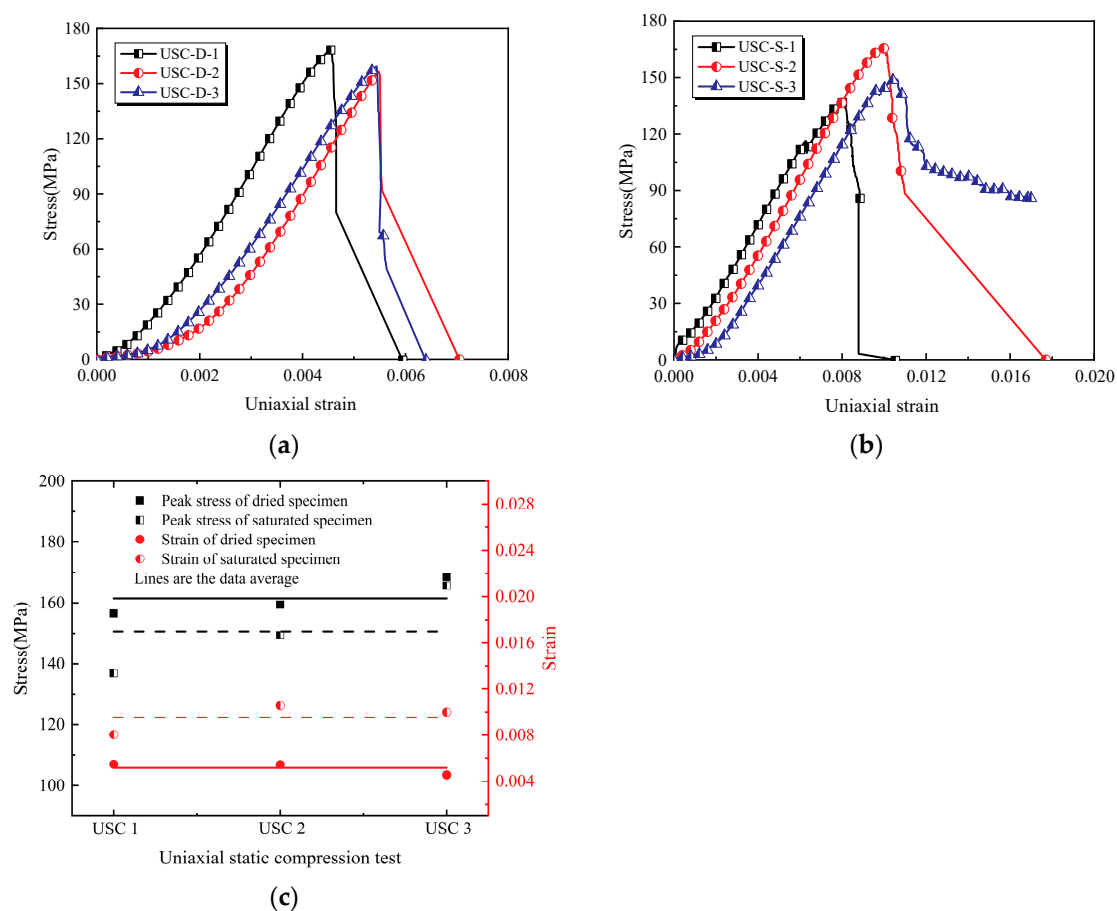


Figure 2. Test results of uniaxial static compression: (a) stress–strain curves of dried granite; (b) stress–strain curves of saturated granite; (c) average peak stress and peak strain of granite.

As shown in Figure 2c, the saturated granite specimens exhibited an average static compressive strength of 150.603 MPa, while the dry specimens showed a higher average strength of 161.475 MPa, representing a 7.2% increase compared to the saturated condition. The saturated granite specimens exhibited an average peak strain of 0.009537, while the dry specimens showed a lower average peak strain of 0.005158, representing a 45.9% decrease compared to the saturated condition. Previous studies have demonstrated that water can

induce softening effects on rock through both physical and chemical interactions, leading to a reduction in compressive strength [18,19]. Granite that is initially dry undergoes a transition from brittle to ductile behavior upon saturation, as the presence of water retards the propagation rate of crack tips, thereby requiring larger strains to reach failure [20,21].

Due to the irregular surface morphology of natural jointed rocks, direct sampling is challenging. To simulate naturally jointed rock, artificial joint specimens were fabricated using a second batch of granite. The deviations in flatness, parallelism, and perpendicularity of the end faces were all less than 0.02 mm, in accordance with the basic requirements of the ISRM. The processed specimens were divided into two groups: one designated as intact rock and the other as jointed rock. Symmetrical grooves were cut into the jointed rock specimens using a cutting machine, with groove numbers of 2, 4, and 6, respectively. Each groove had a depth of 4 mm and a width of 6 mm. One side of the joint rock containing the grooves was combined with the intact rock to form jointed specimens. Figure 3 illustrates the size of joint rocks with different numbers of grooves and jointed specimens. The JMC is defined as the ratio between the contact areas [12], which is calculated by Equation (1).

$$JMC = \frac{A_{joint}}{A_{intact}} \quad (1)$$

where A_{joint} is the surface area of the jointed rock side. A_{intact} is the surface area of the intact rock side. Chen et al. [16] indicated that the JMC of 0.5 was the critical point of the energy dissipation; therefore, the JMC values in the tests are 0.712 (groove numbers of 2), 0.481 (groove numbers of 4), and 0.324 (groove numbers of 6), representing high, medium, and low degrees of geometric matching, respectively.

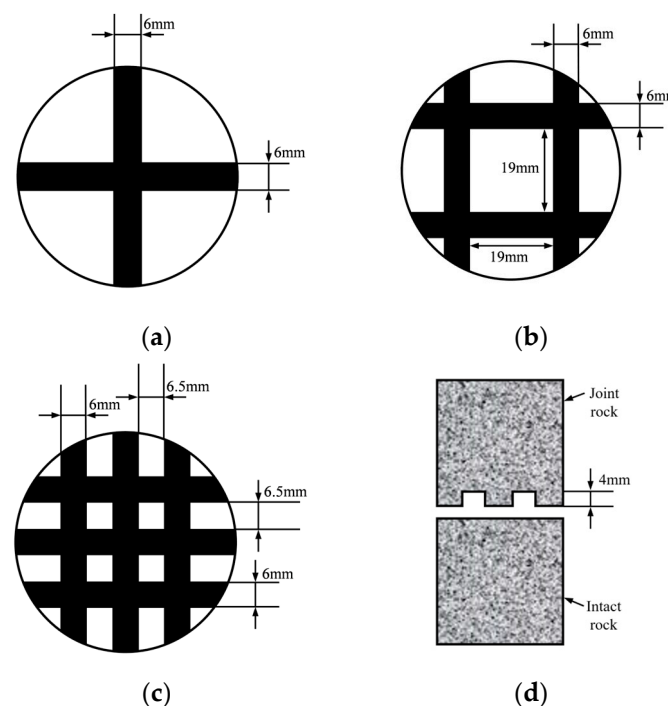


Figure 3. The size of joint rocks with different numbers of grooves and jointed specimens: (a) groove numbers of 2; (b) groove numbers of 4; (c) groove numbers of 6; (d) jointed specimens.

2.2. Experimental Process

A sonic detector (RSM-SY6, Wuhan Zhongyan Technology Co., Ltd., Wuhan, China) was used at University of Science and Technology Liaoning to measure the P-wave velocities of the specimens, and those with significant acoustic discrepancies were excluded. The selected specimens were then placed in a drying box at 105 °C for 24 h to ensure complete

dehydration. After drying, the specimens were transferred to a vacuum saturation device (BHG type), where they were sealed under a negative pressure of 0.1 MPa for 6 h. Distilled water was added until the water level exceeded the highest point of the specimens. The specimens were then kept under ambient temperature and pressure for 24 h to achieve full saturation. Nuclear Magnetic Resonance (NMR; MacroMR12-150H-I, Suzhou Nuomai Analytical Instrument Co., Ltd., Suzhou, China) testing was performed on the specimens before loading at University of Science and Technology Liaoning.

A modified split Hopkinson pressure bar (SHPB; Harbin Institute of Technology, Harbin, China) system was used at University of Science and Technology Liaoning to conduct combined static–dynamic loading tests. The system comprises a launch system, bar system, data acquisition system, axial pressure system, and confining pressure system. The launching system consists of high-pressure liquid nitrogen, an air gun, and a striker (spindle-shaped), with the high-pressure liquid nitrogen providing the driving force for the striker. The bar system includes an incident bar (2100 mm), a transmitted bar (1800 mm), and an absorption bar (500 mm), all made of 50 Cr steel, with an elastic modulus of 210 GPa, a P-wave velocity of 5190 m/s, and a wave impedance of 4.1×10^7 MPa/s. The data acquisition system consists of strain gauges, an ultrahigh dynamic strainometer, and a computer. Strain gauges are symmetrically attached at both ends of the incident and transmitted bars. The axial and confining pressure systems comprise an oil chamber and an electric oil pump, where the magnitude of pre-stress applied to the bars is controlled by adjusting the oil pressure within the chamber.

NMR testing was performed again on the specimens after loading at University of Science and Technology Liaoning. A three-dimensional surface profilometer (NANOVEA PS50, Nanovea, Inc., Irvine, CA, USA) was used to scan the surface of the loaded specimens at University of Science and Technology Liaoning. The entire experimental setup and test procedure are illustrated in Figure 4.

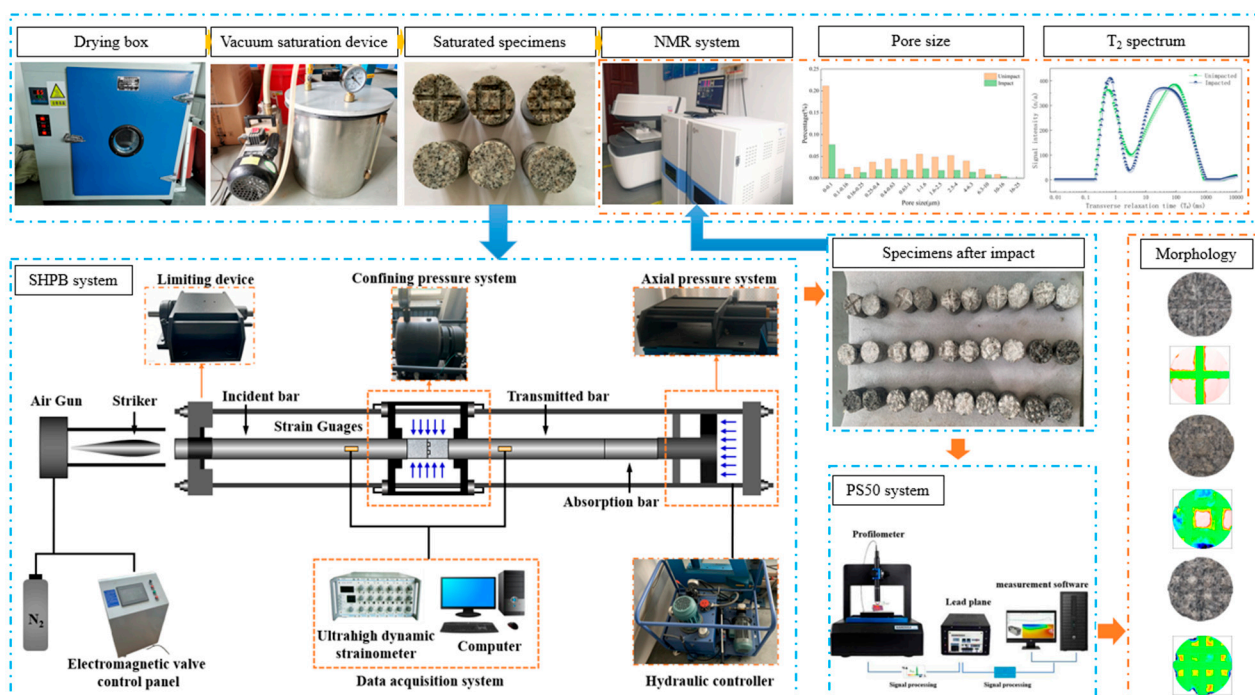


Figure 4. Experimental setup and test procedure.

In the combined static–dynamic loading test, the rock joint contact surfaces were machined into smooth planes and were perpendicular to the loading axis. The specimens were placed in the confining pressure chamber, which restricted the lateral sliding of the

specimens. The specimens and the bar system were aligned on the same horizontal plane; during axial loading, the force lines acting on the specimens coincided with the specimen axes. An appropriate amount of Vaseline was applied to the contact surfaces between the specimens and the bars [22], aiming to minimize shear stress. Thus, it can be assumed that the specimens were only subjected to normal stress during the loading process. NMR technology was employed to investigate the mesoscopic damage characteristics of rocks under combined static–dynamic loading. To maximize the effectiveness of NMR testing, the magnitude of the static load was carefully controlled to ensure that visible cracks appeared upon specimen failure, rather than complete fragmentation [23]. Based on multiple trials, an optimal axial pressure of 9.2 MPa and confining pressures of 1, 2, 3, 4, and 5 MPa were determined. An axial pressure of 9 MPa and a confining pressure of 1 MPa can be regarded as the stress state corresponding to the rock mass at a depth of approximately 156.2 m from the ground surface and at a distance 1.1 times the roadway radius. From the perspective of the triaxial stress state, the rock mass is still under the action of confining pressure. Since a pseudo-triaxial test system was adopted in this study, confining pressure was still applied for the tests. An axial pressure of 9 MPa and a confining pressure of 2 MPa can be considered the stress state matching the rock mass at a depth of about 340 m from the ground surface. The other preset confining pressure values cannot be theoretically correlated with the corresponding burial depths of the rock mass. Therefore, these values can be regarded as artificial confining pressure applied for safety protection measures similar to those used for protective ore pillars in engineering practice. When the applied artificial confining pressure is greater than 2 MPa, the ore pillars can be deemed to be in a safe state.

3. Experimental Results

3.1. Stress–Strain Curves and Strength Characteristics

Loading tests were conducted using a combined static–dynamic SHPB system. The specimens were subjected to a static load P_{s1} and a dynamic load P_d in the axial direction and a static load P_{s2} in the lateral direction, corresponding to three-dimensional coupled loading. The feasibility of SHPB tests is based on the one-dimensional wave theory [24]; in one-dimensional SHPB experiments, the three-wave analysis is generally employed to determine the stress–strain curves of specimens. Li et al. [25] verified through stress analysis of specimens under three-dimensional combined static–dynamic loading that the one-dimensional wave theory remains applicable to such tests. Thus, the three-wave analysis is also suitable for combined static–dynamic loading tests.

When the striker impacts the incident bar, a stress wave is generated and propagates along the incident bar to the specimen, then continues into the transmitted bar, and is eventually absorbed by the absorption bar. The strain gauges mounted on the incident bar and transmission bar recorded the dynamic wave signals generated by the dynamic load P_d . The stress, strain, and strain rate histories of the specimen during loading can be derived from Equation (2) [26]:

$$\begin{cases} \sigma_d(t) = \frac{A_0 E_0}{A_s} \varepsilon_{tr}(t) \\ \varepsilon_d(t) = \frac{C_0}{L_s} \int_0^t [\varepsilon_{in}(t) - \varepsilon_{re}(t) - \varepsilon_{tr}(t)] d\tau \\ \dot{\varepsilon}_d(t) = \frac{C_0}{L_s} [\varepsilon_{in}(t) - \varepsilon_{re}(t) - \varepsilon_{tr}(t)] \end{cases} \quad (2)$$

where $\sigma_d(t)$, $\varepsilon_d(t)$, and $\dot{\varepsilon}_d(t)$ denote the dynamic compressive stress, strain, and strain rate of the specimen, respectively. $\varepsilon_{in}(t)$, $\varepsilon_{re}(t)$, and $\varepsilon_{tr}(t)$ represent the strain signals of the incident

wave, reflected wave, and transmitted wave under dynamic compression, respectively. E_0 , A_0 , and C_0 are the elastic modulus, cross-sectional area, and elastic wave velocity of the bars, respectively. A_s and L_s are the cross-sectional area and height of the joint specimen, respectively. τ is the duration of the elastic wave.

Achieving dynamic stress equilibrium in the specimen is a fundamental prerequisite for the validity of the SHPB test. Figure 5a presents the stress–time histories. Within the time interval of 0–250 μs , the sum of the incident and reflected stresses closely overlaps with the transmitted stress. Defined a stress equilibrium factor σ_{eq} (Equation (3)) to evaluate the uniformity of stress on both ends of the specimen during loading. When σ_{eq} is less than 1%, the specimen is considered to have reached stress equilibrium [27]. As shown in Figure 5b, the specimen maintained good stress equilibrium throughout the impact process, particularly near the peak load. Dynamic stress equilibrium verification was performed on all specimens in this study to ensure the validity of the test results. Since there were no stress gradients in the specimens that could induce inertial forces, the inertial effect can be neglected [28].

$$\sigma_{eq} = \frac{\sigma_{in} + \sigma_{re} - \sigma_{tr}}{(\sigma_{in} + \sigma_{re} - \sigma_{tr})/2} \times 100\% \quad (3)$$

where σ_{in} , σ_{re} , and σ_{tr} are the incident, reflected, and transmitted stresses, respectively.

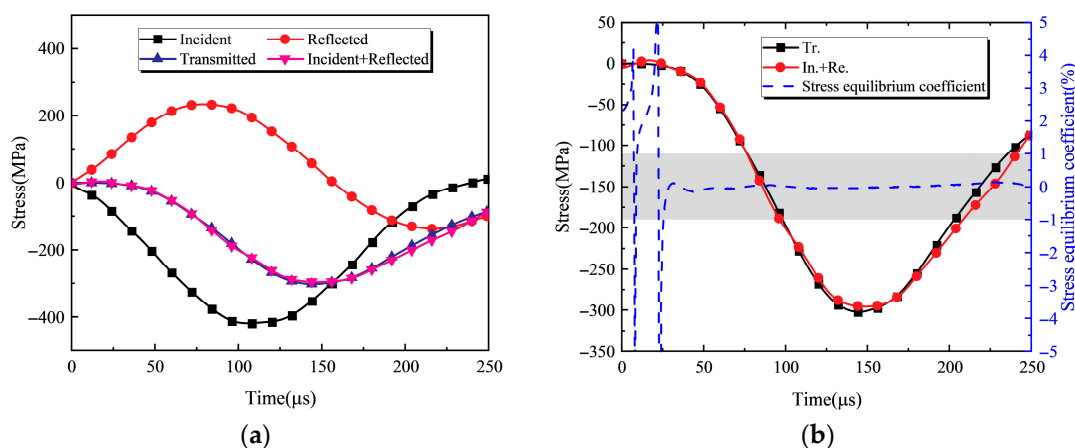


Figure 5. Dynamic stress equilibrium verification: (a) stress wave; (b) stress equilibrium.

Figure 6 presents the dynamic stress–strain curves of saturated jointed rock under combined static–dynamic loading. Although the stress–strain curves of specimens with different levels of joint matching exhibit considerable variation in shape, they all follow the same four-stage evolution process: (1) compaction stage; (2) elasticity stage; (3) plasticity stage; (4) post-peak stage. According to Yin et al. [29], stress–strain curves with residual strain rebound after the peak are classified as Class II, while those without rebound are classified as Class I. When the JMC is 0.324 (i.e., low matching degree), the stress–strain curve is Class I. In the post-peak stage, strain continues to increase while stress decreases, indicating that the specimen releases stored elastic energy slowly or insignificantly during loading, and the failure mode is predominantly plastic. In contrast, when JMC is 0.481 and 0.712 (i.e., medium and high matching degrees), the stress–strain curves are classified as Class II, showing a noticeable rebound phenomenon after the peak. This suggests that recoverable elastic energy is stored within the specimen during loading. A greater rebound indicates a higher amount of stored elastic energy and less plastic deformation. As the degree of joint matching increases, the stress–strain behavior of jointed specimens transitions from Class I to Class II, with increasing rebound magnitude. This indicates a

progressive shift from plastic to elastic deformation and an enhancement in elastic energy storage capacity.

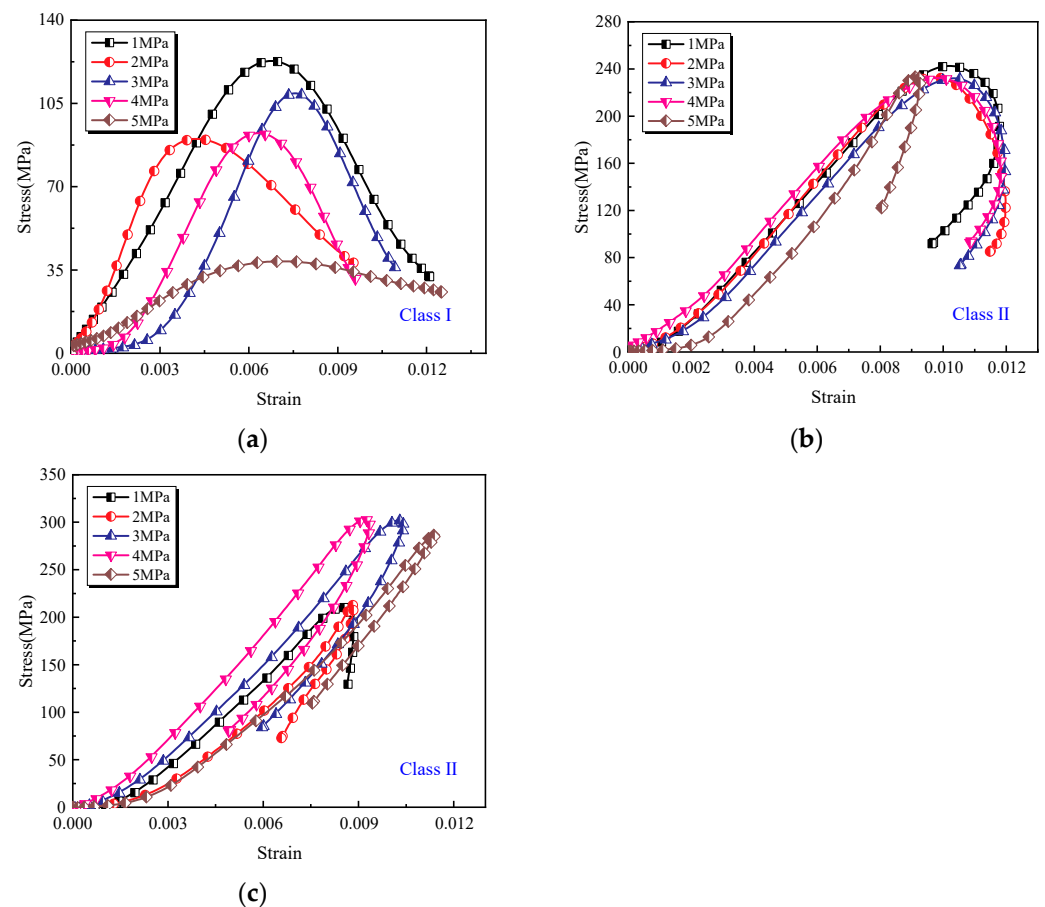


Figure 6. Dynamic stress–strain curves of saturated jointed rock: (a) JMC = 0.324; (b) JMC = 0.481; (c) JMC = 0.712.

The peak stress observed in the dynamic stress–strain curve is defined as the dynamic compressive strength of the specimen. According to the study by Man et al. [30], the dynamic compressive strength of saturated intact granite can be 1–2 times higher than its static strength under moderate strain rates. Based on data from static uniaxial compression tests, the typical dynamic strength range for granite is approximately 300–450 MPa. Figure 7a shows the dynamic compressive strength of saturated jointed granite under various JMC and confining pressures, ranging from 40 to 300 MPa, which is notably lower than the standard range for intact granite. This indicates that both jointing and confining pressure significantly influence the dynamic compressive strength of saturated granite. When the confining pressure is held constant, increasing the JMC from 0.324 to 0.481 results in a considerable increase in dynamic compressive strength. However, when the JMC increases further from 0.481 to 0.712, the dynamic compressive strength decreases under low confining pressures (1–2 Mpa), with reductions of 13.5% and 8.6%, respectively. In contrast, under higher confining pressures (3–5 Mpa), dynamic strength increases by 30.06%, 30.29%, and 22.67%, respectively. Notably, when JMC = 0.324, the contact area between joint surfaces is minimal, resulting in dynamic compressive strength values that are even lower than the static strength of intact saturated granite. Compared to the values at JMC = 0.481, reductions of 49.6%, 61.3%, 52.9%, 60.2%, and 83.4% are observed under confining pressures of 1–5 Mpa, respectively. These results highlight that when the JMC falls below 0.481, the strength of jointed rock declines significantly, a condition that warrants special attention in

engineering applications. The phenomenon is also generally consistent with the conclusion of Chen et al. [16], namely that the JMC of 0.5 was the critical point of the energy dissipation.

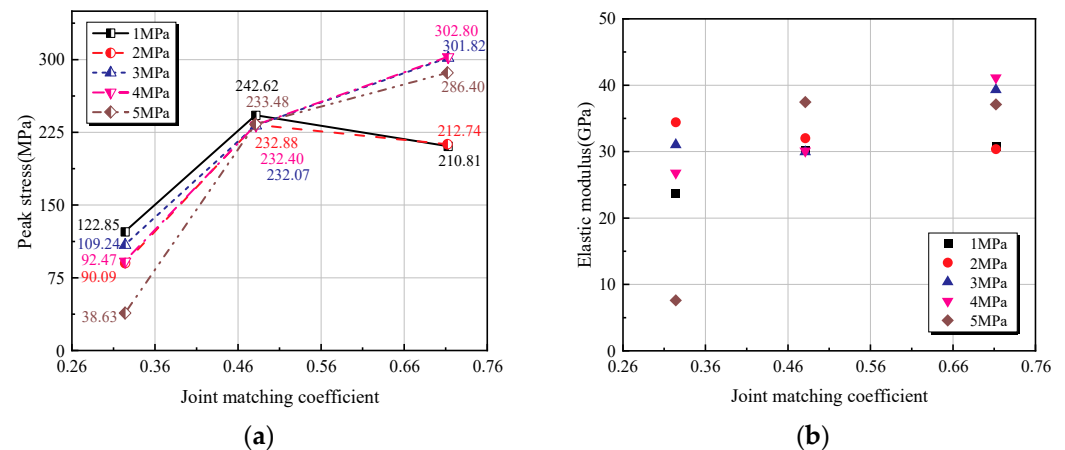


Figure 7. Strength characteristics of saturated jointed rock: (a) dynamic compressive strength; (b) elastic modulus.

Only the linear variation segment in the loading stage of the stress–strain curve is considered, and the elastic modulus is determined by calculating the slope of this segment, which corresponds to the elastic deformation stage. And Figure 7b illustrates the relationship between the elastic modulus of jointed specimens and JMC. Overall, the elastic modulus increases with increasing JMC, accompanied by a reduction in data dispersion. When JMC is relatively high, the specimens exhibit predominantly elastic deformation, which is consistent with the trend observed in the stress–strain curves. A higher contact area indicates more rock-to-rock interaction and fewer microcracks within the contact zone, thereby enhancing the overall stiffness of the specimen [31,32]. Since the elastic modulus is directly related to material stiffness, an increase in contact area leads to a higher elastic modulus.

The secant modulus reflects the deformation resistance of rock, i.e., the stiffness of the specimen [33]. It is typically determined from the slope of the loading stage of the stress–strain curve. However, for saturated jointed rocks, the stress–strain curve exhibits a pronounced compaction stage at the early loading phase, making the use of the slope at the peak stress point highly variable. Therefore, the dynamic secant modulus, E_d (Equation (4)), is adopted to more accurately represent the overall response during the dynamic loading process.

$$\begin{cases} E_1 = \frac{\sigma_{50}}{\varepsilon_{50}} \\ E_p = \frac{\sigma_p}{\varepsilon_p} \\ E_2 = \frac{\sigma_p - \sigma_{50}}{\varepsilon_p - \varepsilon_{50}} \\ E_d = \frac{E_1 + E_p + E_2}{3} \end{cases} \quad (4)$$

where σ_{50} is the stress corresponding to 50% of the peak stress. ε_{50} is the strain corresponding to 50% of the peak stress. σ_p is the peak stress. ε_p is the strain at the peak stress.

The dynamic secant modulus E_d is obtained by taking the average of the calculated values of the three elastic moduli via Equation (4), as shown in Figure 8. As the JMC increases, the dynamic secant modulus shows a rising trend and gradually levels off. Due to the application of axial prestress, the specimens are pre-compressed. For specimens with medium to high JMC, the joint surfaces are fully closed, resulting in an increased effective load-bearing area and a reduced stress per unit area. This makes the specimens more resistant to deformation, which is reflected macroscopically as an increase in the secant modulus.

Confining pressure further compresses the joint apertures by applying lateral constraints, producing a synergistic effect with JMC. When JMC = 0.481, the secant modulus under varying confining pressures is 23.88, 23.99, 22.55, 23.73, and 27.89, respectively, indicating that confining pressure has a limited influence on stiffness and that JMC predominantly governs the variation in specimen stiffness. In contrast, specimens with low JMC exhibit lower compressive strength. Under combined loading, stress concentrations are more likely to form at the edges of contact points, making these specimens more susceptible to deformation and failure.

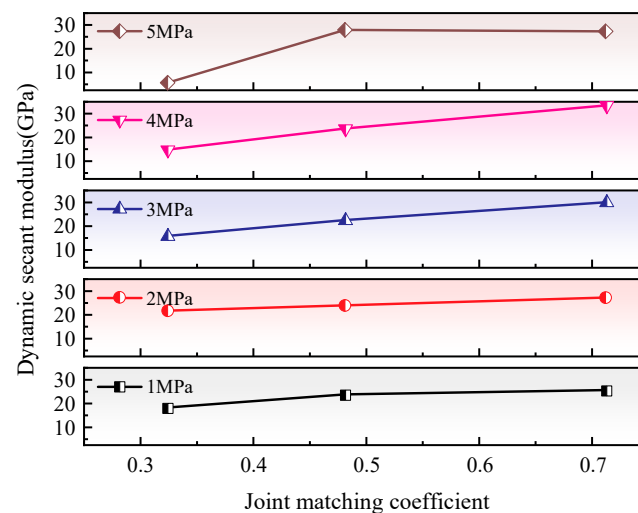


Figure 8. Dynamic sectioning modulus of saturated jointed rock.

3.2. Joint Deformation Characteristics

According to the one-dimensional stress wave propagation theory, the deformation of each specimen is attributed to both the jointed part and the intact rock part [15], expressed as follows [17]:

$$\Delta u = \Delta u_{\text{intact}} + \Delta u_{\text{joint}} \quad (5)$$

where Δu is the total deformation of the jointed specimen. Δu_{intact} is the deformation of intact rock. Δu_{joint} is the deformation of a joint.

The joints usually deform nonlinearly when the stress wave amplitude is large. Since the strain rate during the tests ranged from 30 s^{-1} to 120 s^{-1} , which falls within the medium strain rate regime. Assuming the intact rock behaves in a linearly elastic manner under dynamic loading, its deformation can be described by Hooke's law as [17]:

$$\varepsilon = \frac{\sigma(l_0 - l_{\text{joint}})}{El_0} + \frac{\Delta u_{\text{joint}}}{l_0} \quad (6)$$

where σ and ε are the stress and strain during loading, respectively. l_0 is the total length of the specimen. l_{joint} is the thickness of the joint, which is 4 mm in this experiment. E is the dynamic elastic modulus of intact granite. Accordingly, the deformation of the joint can be expressed as [17]:

$$\Delta u_{\text{joint}} = \varepsilon l_0 - \frac{\sigma(l_0 - l_{\text{joint}})}{E} \quad (7)$$

Figure 9 presents the stress–closure curves of saturated jointed rock under combined static–dynamic loading. During the loading phase, due to the nonlinear deformation behavior of the joints, the relationship between joint closure and stress exhibits a distinct nonlinear trend. At lower stress levels, the deformation is minimal, resulting in a smaller slope of the curve, which appears approximately linear. As the stress increases, the slope

gradually increases, and the curve demonstrates pronounced nonlinearity. During the unloading phase, when the JMC is 0.712 and 0.481, a rebound phenomenon occurs in the post-peak stage, where the closure reaches a maximum and is followed by a reopening behavior. This is attributed to the axial pre-stress applied during the experiment, which induces an initial closure of the joint and leads to significant elastic energy accumulation within the rock. Under dynamic loading, the stored elastic energy is rapidly released, resulting in closure rebound after the peak stress. Given the high loading rate and short duration of dynamic impact, such rebound is not observed when JMC = 0.324. In this case, partial plastic deformation occurs prior to strength failure, thereby preventing the release of elastic energy and eliminating the rebound effect in the post-peak stage.

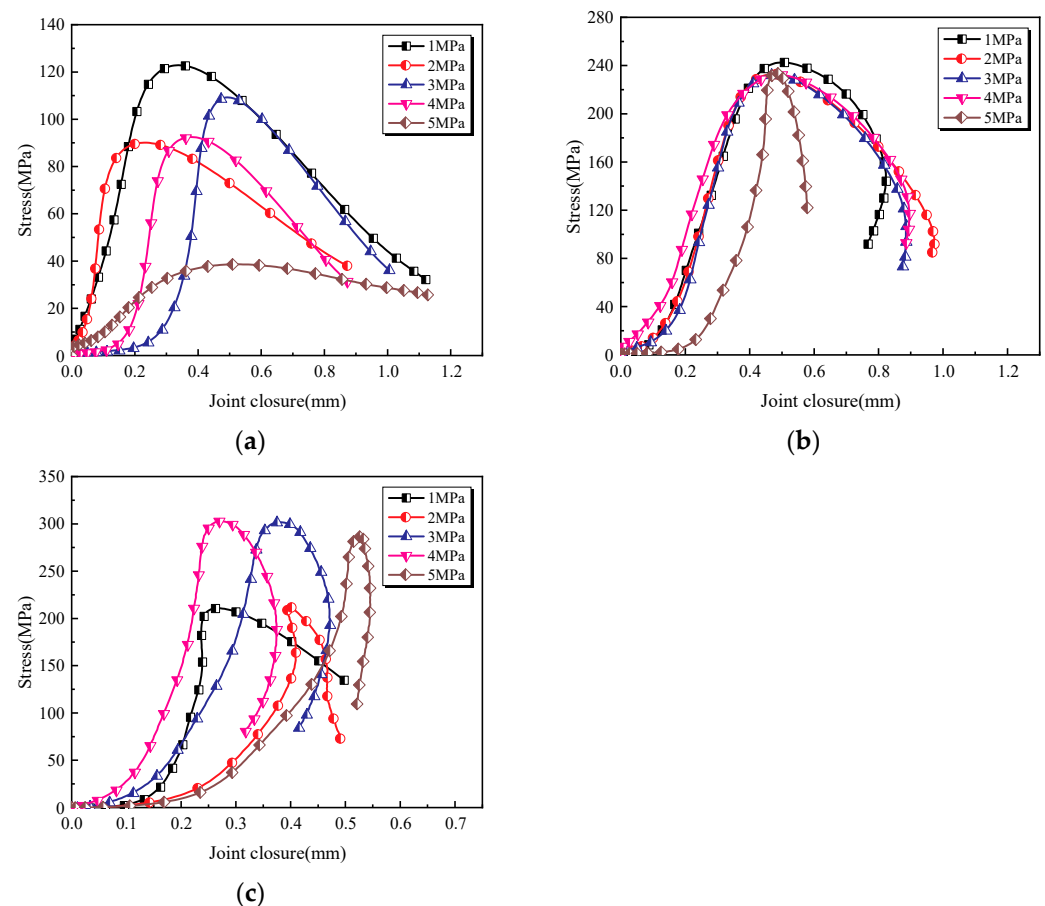


Figure 9. Stress–closure curves of saturated jointed rock: (a) JMC = 0.32; (b) JMC = 0.481; (c) JMC = 0.712.

To further investigate the deformation characteristics, Figure 10 illustrates the total deformation, joint deformation, and intact rock deformation, calculated based on Equations (6)–(8). The total deformation shows a slight increase as JMC decreases. Notably, the proportion of deformation in the intact rock decreases, while that in the joint increases. For JMC = 0.324, the joint deformation dominates across all confining pressures, indicating that the lower the JMC, the greater the contribution of the joint to the overall deformation.

To quantify this trend, the joint deformation rate is defined as the ratio of joint deformation to total deformation. Figure 11 shows the relationship between joint deformation rate and JMC. At constant confining pressure, the joint deformation rate decreases linearly with increasing JMC. When JMC = 0.324, the joint deformation rate reaches 90.13–92.64%, accounting for 90–93% of the total deformation. For JMC = 0.481, the values range from 62.34% to 81.26%, while for JMC = 0.712, they fall between 40.07% and 58.19%. In this

experiment, the specimen length is 100 mm, while the joint accounts for only 4 mm (4% of the total length). Nevertheless, the joint deformation rate exceeds 40% and can reach over 90% at low JMC. This indicated that the deformation is primarily concentrated in the joint. As JMC increases, the joint's contribution to deformation diminishes due to higher contact area, increased dynamic secant modulus, and enhanced deformation resistance. While confining pressure improves lateral restraint and limits lateral deformation, its effect on the normal closure of the joint is limited [34]. Therefore, the joint deformation rate shows no clear dependence on confining pressure.

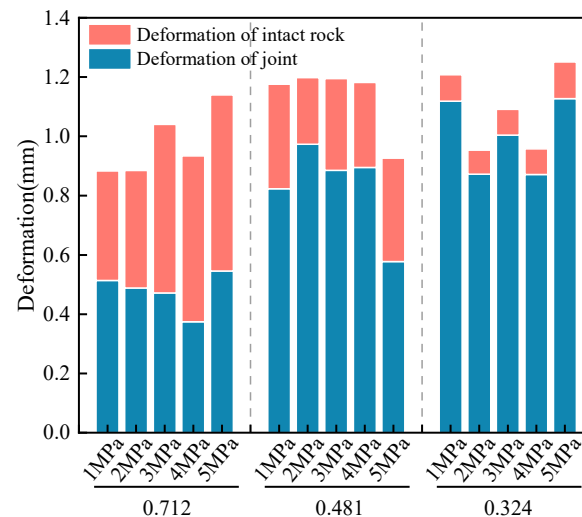


Figure 10. The deformation amount of saturated jointed rock.

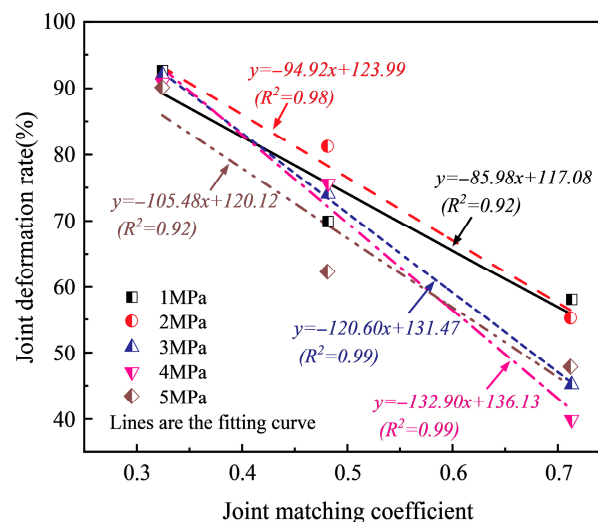


Figure 11. Joint deformation rate of saturated jointed rock.

The magnitude of specimen deformation is not only associated with its strength but also with the resistance of the jointed portion to normal deformation under load. To quantify the joint's resistance to normal deformation under combined static–dynamic loading, the joint-specific stiffness k (Equation (8)) is defined as the slope of the line connecting the peak stress and the origin of the coordinates on the stress–closure curve [35].

$$k = \frac{\sigma_n}{\Delta u_n} \quad (8)$$

where σ_n is the peak stress of the stress–closure curve. Δu_n is the closure at the peak stress of the stress–closure curve.

Figure 12 illustrates the joint-specific stiffness of saturated jointed rock under combined static–dynamic loading. At constant confining pressure, the joint-specific stiffness increases with the JMC, indicating that joints with higher JMC possess a stronger resistance to normal deformation. A positive correlation between JMC and joint-specific stiffness, though the rate of increase varies with confining pressure. When the confining pressure is 5 MPa and JMC = 0.324, the joint-specific stiffness is the lowest at 73.448 GPa. However, under this pressure, the coefficient increases by approximately 640% as JMC increases. At JMC = 0.481, the joint-specific stiffness remains around 485 GPa across different confining pressures, with variations limited to about 1.3%. This indicated that deformation capacity is insensitive to confining pressure at this level of JMC. Under identical confining pressure, specimens with JMC = 0.712 exhibit the highest joint-specific stiffness, indicating that joints with larger contact areas are significantly stiffer and less susceptible to deformation.

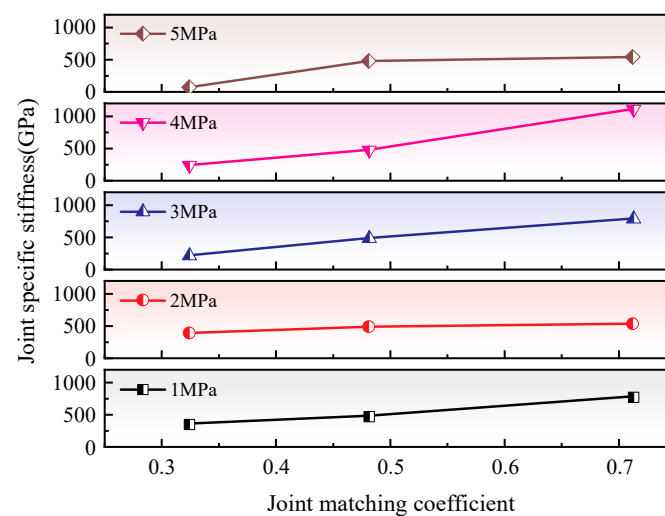


Figure 12. Joint stiffness coefficient of saturated jointed rock.

Since the majority of specimen deformation originates from the joint, the damage morphology of the joint after loading is of particular interest. Figure 13 presents the joint damage morphology and three-dimensional surface topography. With the joint surface set horizontally, the X-axis represents specimen thickness, and the Y-axis aligns with the joint surface. The scanning area was 50×50 mm, with a scan step of 100 μm and a scanning speed of 1 mm/s [36]. The results show that under a constant confining pressure, the extent of joint damage decreases with increasing JMC. Stress concentration is observed primarily at the joint edges or within weak zones of the rock matrix, leading to localized tensile failure. When JMC = 0.712, the joint remains largely intact, with minor rock spalling confined to the edge of the joint's asperities. The variation in joint thickness is minimal, corresponding to 40–59% of the total specimen deformation. When JMC = 0.481, the outer asperities are damaged, and rock spalling appears at the central asperity edges, indicating more extensive joint failure, corresponding to 62–82% of the total deformation. When JMC = 0.324, the joint asperities experience severe failure, with the remaining part forming a “pyramidal” shape, indicating significant stress concentration at asperity edges. In this case, specimen deformation is primarily attributed to joint asperity failure. The failure mode of the jointed specimens was dominated by tensile failure, and no shear slip traces were observed, indicating that the specimens were only subjected to normal stress during loading. Overall, higher joint matching correlates with increased joint stiffness and compressive strength, thereby reducing the likelihood of structural failure under combined static–dynamic loading.

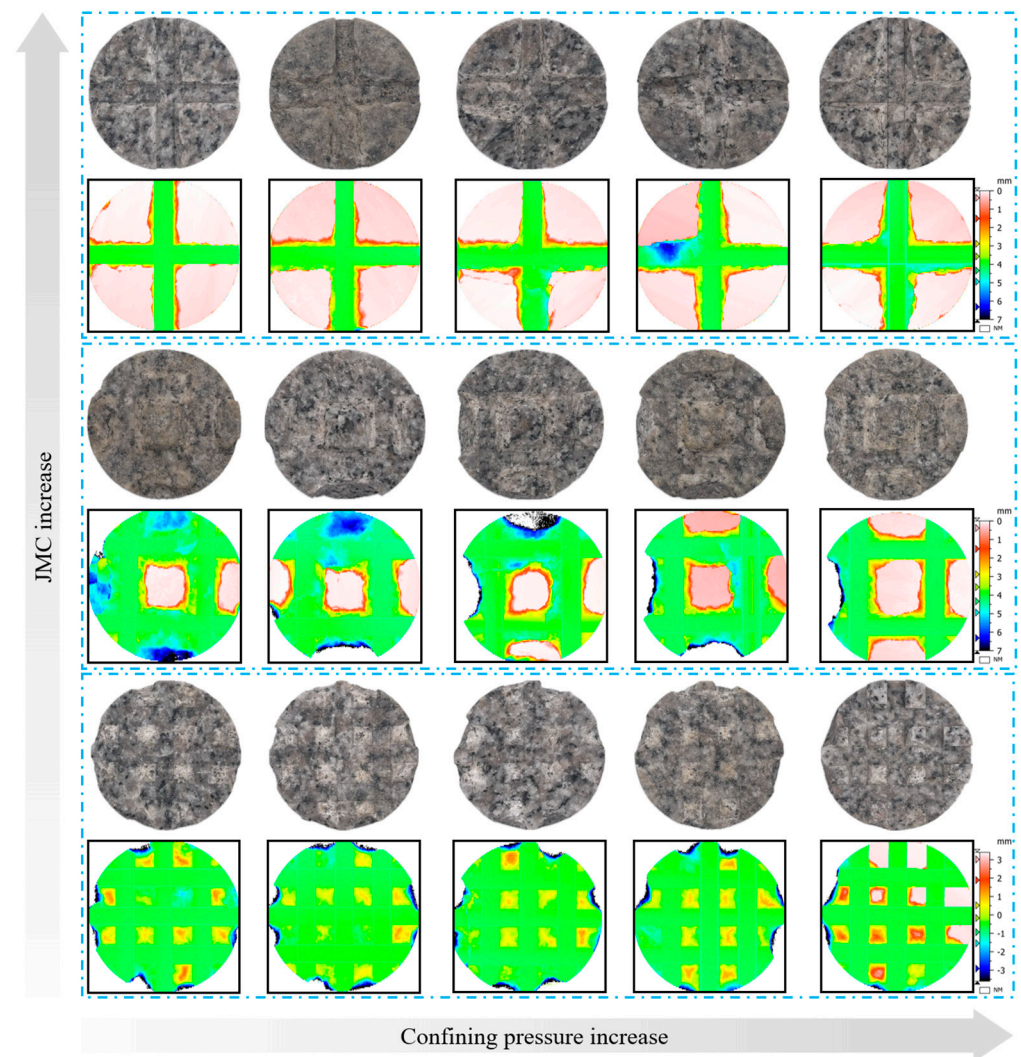


Figure 13. Joint damage morphology and three-dimensional surface topography.

3.3. Energy Characteristics

In SHPB tests, the kinetic energy generated by striker impact serves as the sole energy source for the tests. The energy induced by the corresponding stress waves can be expressed as

$$\begin{cases} W_{in} = A_0 C_0 E_0 \int_0^t \varepsilon_{in}^2(t) dt \\ W_{re} = A_0 C_0 E_0 \int_0^t \varepsilon_{re}^2(t) dt \\ W_{tr} = A_0 C_0 E_0 \int_0^t \varepsilon_{tr}^2(t) dt \end{cases} \quad (9)$$

where W_{in} , W_{re} , and W_{tr} denote the energy of the incident, reflected, and transmitted stress waves, respectively.

All tests were conducted under ambient temperature, with no heat exchange with the external environment. Assuming that the energy dissipation at the interfaces between the specimens and the bars is neglected, the dissipated energy W_{di} of the rock specimens can be calculated as [37]:

$$W_{di} = W_{in} - W_{re} - W_{tr} \quad (10)$$

The reflected and transmitted energies depend on the wave impedance between the specimen and the bars, representing the internal properties of the rock. To quantify the

distribution of energy, the energy reflection rate η_{re} , energy transmission rate η_{tr} , and energy dissipation rate η_{di} are defined as

$$\begin{cases} \eta_{re} = \frac{W_{re}}{W_{in}} \times 100\% \\ \eta_{tr} = \frac{W_{tr}}{W_{in}} \times 100\% \\ \eta_{di} = \frac{W_{di}}{W_{in}} \times 100\% \end{cases} \quad (11)$$

In SHPB tests, the incident energy is delivered to the specimen via stress waves. The greater the incident energy, the larger the amplitude of the stress wave. Strain rate characterizes the deformation rate of the specimen during dynamic loading, and its magnitude is closely related to the amplitude and duration of the incident wave. Figure 14 shows the relationship between strain rate and incident energy of saturated jointed rock under combined static–dynamic loading. As the incident energy increases, the strain rate rises from the order of 10^1 s^{-1} to 10^2 s^{-1} , which belongs to the medium strain rate range. The two variables follow an allometric relationship. In the low energy range, significant data dispersion is observed due to energy consumption in overcoming initial joint closure and pore compression. Strain rate growth is restricted by local plastic deformation. At high incident energy levels, stress wave amplitude increases significantly, and the specimen undergoes predominantly elastic deformation. In this stage, the input energy directly drives elastic deformation of the rock matrix and crack propagation, yielding improved data fitting quality in the high-energy range.

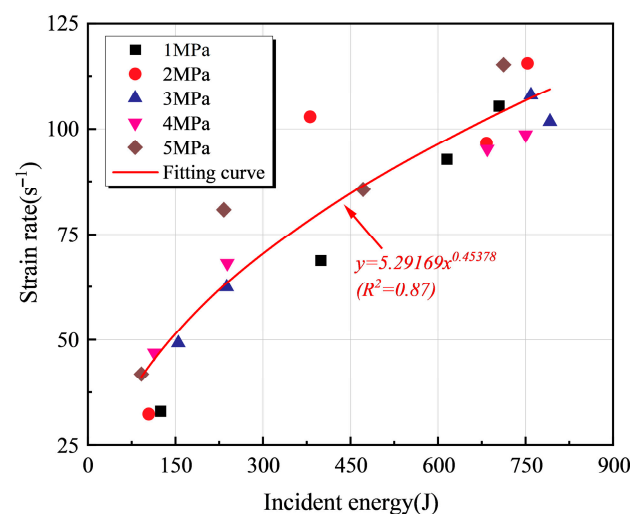


Figure 14. Relationship between strain rate and incident energy of saturated jointed rock.

Figure 15 illustrates the energy variation and energy dissipation rate of saturated jointed rock under combined static–dynamic loading. The total energy increases with JMC. The dissipated energy initially increases and then decreases as JMC rises, while the energy dissipation rate shows a decreasing trend overall. The presence of confining pressure limits the radial expansion of the specimen and constrains the high-velocity movement of rock particles. Furthermore, water reduces the frictional resistance between joint interfaces. As a result, the majority of the dissipated energy is consumed in crack growth and material failure. It can be seen from the three-dimensional surface topography of the specimens (Figure 13) that as JMC decreases, damage becomes more severe and the energy dissipation rate correspondingly increases.

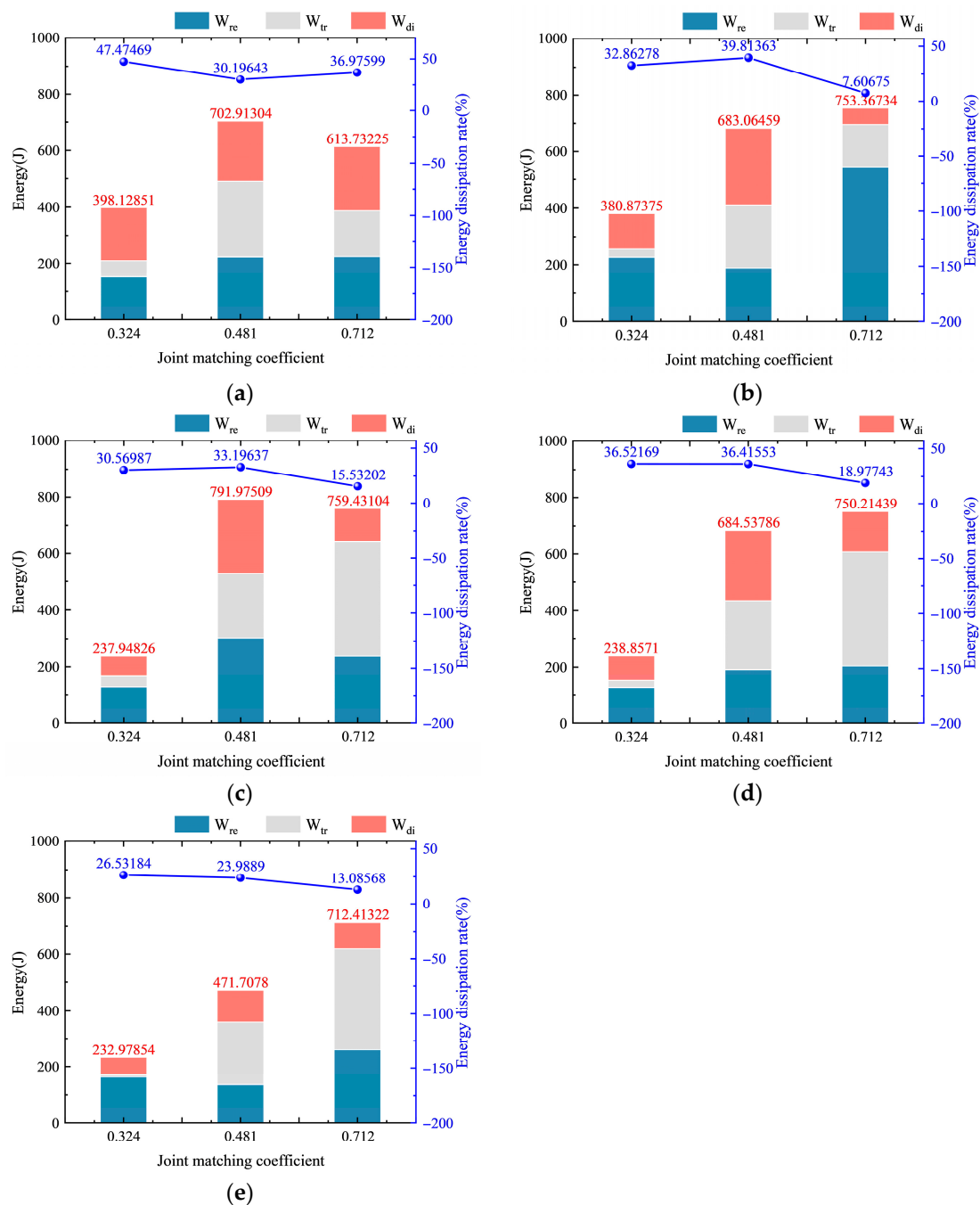


Figure 15. Energy variation and energy dissipation rate of saturated jointed rock: (a) 1 MPa; (b) 2 MPa; (c) 3 MPa; (d) 4 MPa; (e) 5 MPa.

As shown in Figure 15, the energy transmission rate exhibits the following trends: JMC = 0.712 is 20–54%; JMC = 0.481 is 29–47%; JMC = 0.324 is 4–16%. This indicated significantly lower transmission capacity for poorly matched joints. Figure 16 presents the percentage of energy accumulation for JMC = 0.324. The distribution follows the order: energy reflection rate > energy dissipation rate > energy transmission rate. This reflects the energy propagation mechanism and stress wave attenuation characteristics of the joint surface. When JMC = 0.324, the small contact area results in low dynamic compressive strength and joint-specific stiffness. Under axial pre-stress, crack propagation occurs near the joint interface, leading to minor internal damage and increased wave impedance. Consequently, only a small portion of the stress wave energy is transmitted across the joint,

while the majority is reflected back into the incident bar. Thus, reflected energy dominates the energy budget, resulting in a high energy reflection rate and low transmission rate.

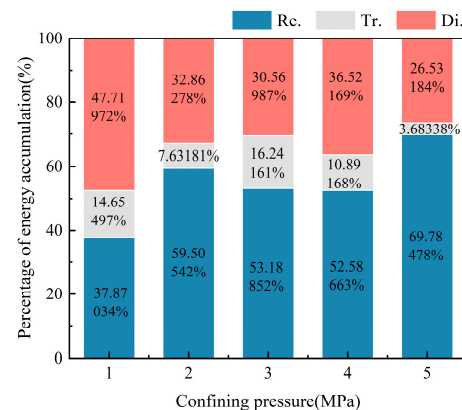


Figure 16. The percentage of energy accumulation of saturated jointed rock for JMC = 0.324.

3.4. Stress Wave Propagation Characteristics

To quantitatively analyze the propagation and attenuation behavior of stress waves, the reflection coefficient R and transmission coefficient T are introduced based on Hertzian contact theory.

$$\begin{cases} R = \frac{\max|\varepsilon_{re}|}{\max|\varepsilon_{in}|} \\ T = \frac{\max|\varepsilon_{tr}|}{\max|\varepsilon_{in}|} \end{cases} \quad (12)$$

Figure 17a shows the relationship between the reflection coefficient and JMC under combined static–dynamic loading. The variation in the reflection coefficient exhibits a transition from a decrease stage to an unstable change stage. Figure 17b shows the relationship between the transmission coefficient and JMC, which transitions from a rapidly increasing stage to an unstable change stage. These variations indicate that the increase in joint contact area promotes stress wave propagation. However, the unstable change is attributed to the combined effects of confining pressure and axial pressure in saturated specimens. In SHPB tests, the JMC plays a dominant role in stress wave propagation. At the initial stage of JMC increase, the synergistic compaction effect of confining and axial stresses is significant. The enlarged joint contact area enhances wave impedance matching, resulting in a rapid variation in both reflection and transmission coefficients. As JMC continues to increase, the joint-specific stiffness also rises, gradually saturating the compaction effect of confining and axial stresses, and the joint closure approaches its limit. Under high confining pressure, the viscous resistance of pore water suppresses slip along the joint interface. However, disturbance of water films in locally unclosed regions disturbs the stress wave transmission path, leading to an unstable stage in both reflection and transmission coefficients.

The specimen is subjected to axial pre-stress, and new pores are created inside the joints. When stress waves encounter these pores, partial reflection and transmission occur, with the reflected wave being a tensile wave [38–40]. These tensile waves repeatedly accumulate between the pores and the joint interfaces, ultimately leading to tensile failure of the specimen, as illustrated in Figure 18. Consequently, localized tensile failure is observed along joint edges or within weak zones of the rock matrix. This accumulation effect is more pronounced under low joint matching conditions. The repeated reflection and accumulation of stress waves result in a substantial portion of the incident wave energy being reflected back into the incident bar. This energy dissipation mechanism explains the higher energy dissipation rate observed in specimens with low JMC.

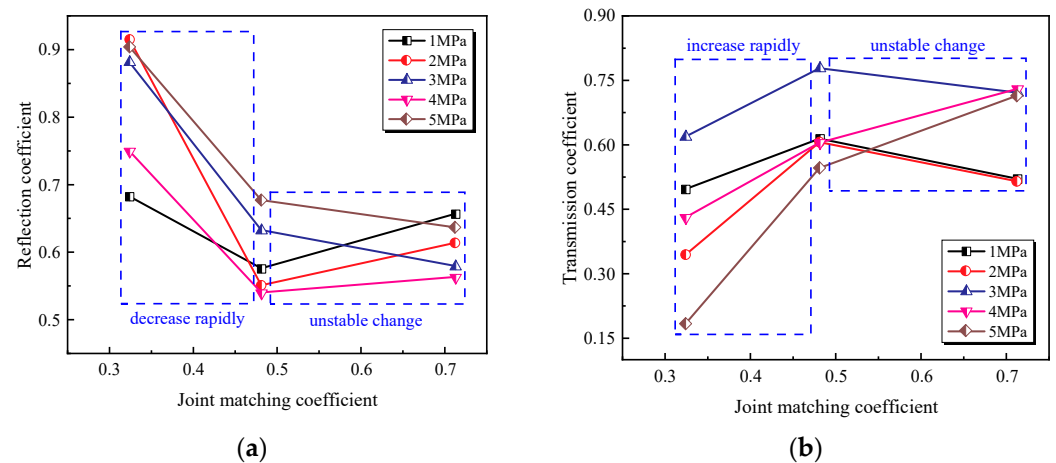


Figure 17. Reflection and transmission coefficient of saturated jointed rock: (a) reflection coefficient; (b) transmission coefficient.

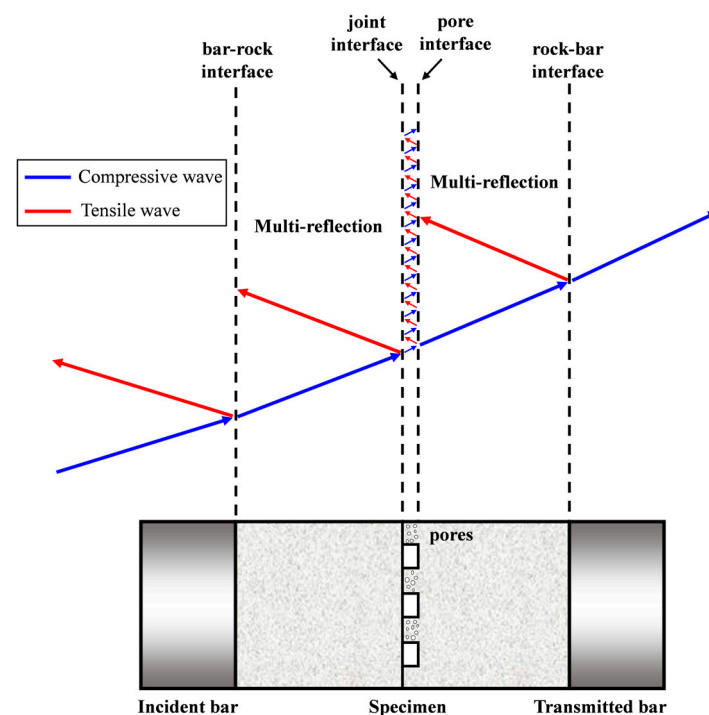


Figure 18. Schematic of stress wave propagation in jointed rock.

The NMR test is employed to measure the transverse relaxation time distribution (T_2) of hydrogen atoms in saturated rocks, providing information related to the size distribution of water-filled pores [41]. According to Anovitz and Cole [42], T_2 can be expressed as follows:

$$\frac{1}{T_2} \approx \rho \left(\frac{S}{V} \right) \quad (13)$$

where ρ is the transverse surface relaxivity. S is the surface area of pores in rock. V is the volume of pores in rock.

The changes in the T_2 spectra distribution before and after loading reveal the transformation between pores and fractures, providing microscopic insight into the mechanical behavior and stress wave propagation in jointed rocks. Figure 19 presents the changes in the T_2 spectra distribution of the jointed rock sections under combined static–dynamic loading. Under different matching conditions, the T_2 spectra generally exhibit two peaks, with smooth transitions between them, indicating a continuous variation in pore size and a

uniform spatial distribution of pores. When $JMC = 0.324$, the total T_2 area decreases after loading, and both peaks shift to the left. In contrast, when JMC is 0.712 and 0.481, the T_2 area increases after loading. These results suggest that JMC influences the evolution of pore structure by altering the joint contact area. At low matching degrees, the increased wave impedance mismatch intensifies wave reflection at the joint interface, resulting in less transmitted stress energy. Under confining pressure, pore water is expelled, and movable water transitions to bound water, manifested in the T_2 spectrum as reduced peak areas and both peaks shift to the left. Conversely, increased matching enhances joint contact area, leading to more uniform stress distribution and greater stress wave transmission, which promotes pore expansion. As a result, the T_2 area increases, and under axial and confining pressure, pore water migrates into newly formed voids, increasing the movable water content and corresponding to the growth of the second spectral peak.

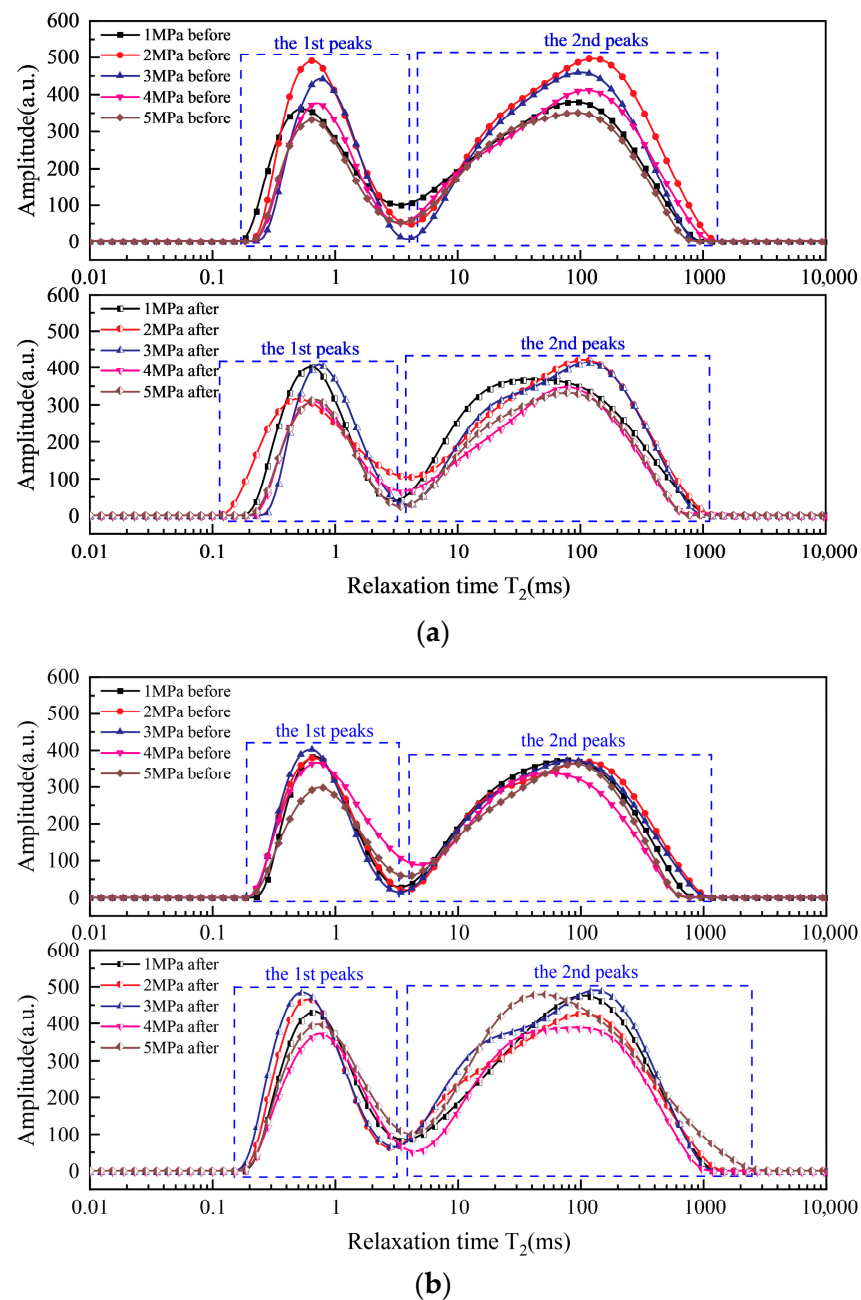


Figure 19. Cont.

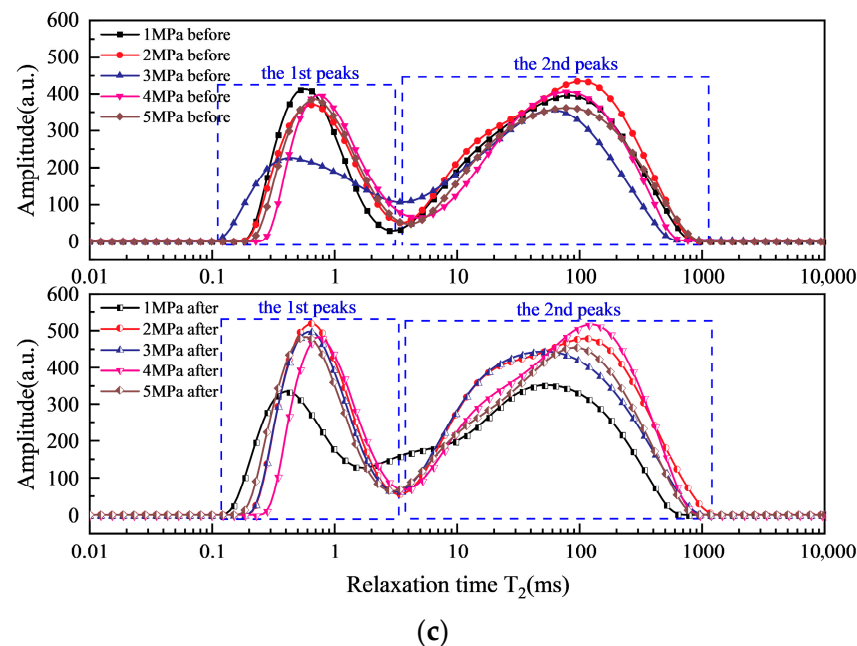


Figure 19. Changes in the T_2 spectra distribution of saturated jointed rock before and after loading: (a) JMC = 0.324; (b) JMC = 0.481; (c) JMC = 0.712.

4. Discussion

This study integrates the SHPB, NMR, and three-dimensional surface-scanning methods to conduct a multi-dimensional investigation on the stress wave characteristics of saturated jointed rocks under combined static–dynamic loading, whose effectiveness is verified through multi-dimensional validation. A core prerequisite for the reliability of SHPB test results is the dynamic stress equilibrium at both ends of the specimen. As shown in Figure 5b, the stress equilibrium factor (σ_{eq}) of all specimens during loading is less than 1%, indicating that the superposition of incident and reflected waves is highly consistent with the transmitted wave, which meets the requirements of the Kolsky method [43]. The rock energy dissipation rate calculated based on stress wave signals exhibits a clear correlation with the rock damage characteristics obtained via three-dimensional surface scanning. The trend of increasing energy dissipation rate with decreasing JMC is associated with the observed aggravation of joint damage and the higher proportion of reflected energy caused by wave impedance mismatch.

However, it should be noted that the joint specimens were fabricated by cutting symmetric grooves on intact granite, which simplifies the irregular morphological characteristics of natural rock joints. This simplification facilitates the establishment of relationships between joint matching degree, rock strength, and deformation. Natural joints typically exhibit asymmetric and discontinuous contact features, which may lead to differences in stress concentration and crack propagation paths compared with artificial regular joints. For instance, the “pyramidal” damage morphology of low JMC specimens observed in Figure 13 is more regular than that of natural joints, which may underestimate the complexity of stress wave transmission during actual engineering failure processes. Therefore, future studies can fabricate joint specimens with different morphological characteristics and quantify the JMC using three-dimensional scanning methods, making the experimental conclusions more consistent with engineering practice.

The experimental results indicate that when the JMC is low, the dynamic compressive strength of saturated jointed rocks is relatively low, and the stress wave transmission coefficient changes rapidly. In engineering practice, priority should be given to detecting and identifying rock areas with low-matching joints, and reinforcement measures such as

grouting or anchoring should be implemented in these high-risk regions. The implementation of a flexible support system is essential to adapt to deformation concentration at joints and resist tensile cracking induced by stress wave reflection. Based on the variation law of stress waves, a monitoring system can be installed to real-time track changes in the stress wave transmission coefficient. An abnormal sudden change in the transmission coefficient indicates deterioration of the joint contact state and cumulative damage, which can serve as an early warning signal for the dynamic instability of rocks.

5. Conclusions

This study combined static–dynamic loading experiments on saturated granite containing artificial joints to investigate the influence of the JMC on the rock’s strength, deformation, energy evolution, and stress wave propagation properties. The attenuation mechanisms of stress wave propagation by the coupling of internal and external factors are clarified. The main conclusions are as follows:

- (1) Under combined static–dynamic loading, the stress–strain behavior of saturated jointed rocks transitions from Class I to Class II with increasing JMC. The dynamic compressive strength of saturated jointed rocks is lower than the typical dynamic strength range, and a significant reduction is observed when JMC falls below 0.481. Both the elastic modulus and the dynamic secant modulus increase as JMC increases.
- (2) The application of pre-stress causes initial closure of the specimens. After loading, the joint deformation rate decreases with increasing JMC, while the joint-specific stiffness increases accordingly. Rock deformation is primarily concentrated in the joint. Under reflected tensile stress waves, local tensile failure is observed within the specimen.
- (3) Energy drives crack propagation and failure in jointed rocks. After loading, the energy dissipation rate increases as JMC decreases. At low matching degrees, stress wave attenuation is the dominant factor leading to a low energy transmission rate and a high energy reflection rate.
- (4) An increase in JMC facilitates stress wave propagation. At low matching degrees, the synergistic compaction effect of confining pressure and axial pressure causes a rapid change in the transmission and reflection coefficients. At high matching degrees, this compaction effect tends to saturate, and the disturbance of pore water films leads to an unstable stage in both reflection and transmission coefficients.

Author Contributions: Conceptualization, Y.W. and X.L.; Methodology, X.L.; Software, X.H.; Validation, Z.X.; Investigation, X.H. and H.T.; Resources, X.H.; Data curation, X.H.; Writing—original draft, X.H.; Writing—review & editing, Y.W. and X.L.; Visualization, Z.X. and H.T.; Supervision, Z.X. and H.T.; Project administration, Z.X.; Funding acquisition, X.L. All authors have read and agreed to the published version of the manuscript.

Funding: This work was supported by the Basic Research Project of the Liaoning Provincial Department of Education (No. LJ212410146019) and the Anhui Province Postdoctoral Research Project (No. 2024C896).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The original contributions presented in this study are included in the article. Further inquiries can be directed to the corresponding authors.

Conflicts of Interest: Authors Yunmin Wang and Xin Liu were employed by the company Sinosteel Maanshan General Institute of Mining Research Co., Ltd. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

References

1. Zhou, K.; Dou, L.; Gong, S.; Chai, Y.; Li, J.; Ma, X.; Song, S. Mechanical behavior of sandstones under water-rock interactions. *Geomech. Eng.* **2022**, *29*, 627–643.
2. Zeng, L.; Dautriat, J.; Monmusson, L.; Xie, Q. Effect of fluid saturation and salinity on sandstone rock weakening: Experimental investigations and interpretations from physicochemical perspective. *Acta Geotech.* **2023**, *18*, 171–186. [\[CrossRef\]](#)
3. Eremenko, A.; Darbinyan, T.; Shaposhnik, Y.; Usol'tseva, O.; Tsoi, P. Physical and mechanical properties of ore and rocks after flooding. *J. Min. Sci.* **2023**, *59*, 723–728. [\[CrossRef\]](#)
4. Liu, G.; Peng, Y.; Zuo, Q.; Su, Y.; Wu, L. Dynamic mechanics and energy dissipation of saturated layered phyllite. *Minerals* **2022**, *12*, 1246. [\[CrossRef\]](#)
5. Zhu, X.; Li, Q.; Wei, G.; Fang, S. Dynamic tensile strength of dry and saturated hard coal under impact loading. *Energies* **2020**, *13*, 1273. [\[CrossRef\]](#)
6. Matejunas, A.; Tawney, J.; Reynolds, E. Effects of water saturation on the dynamic compression and fragmentation response of gabbroic rock. *Rock Mech. Rock Eng.* **2022**, *55*, 4929–4939. [\[CrossRef\]](#)
7. Zhou, Y.; Su, S.; Chen, J. Dynamic response characteristics of dry and water-saturated schist under impact loading. *J. Mt. Sci.* **2020**, *17*, 3123–3136. [\[CrossRef\]](#)
8. Wu, J.; Feng, M.; Yu, B. Experimental investigation on dilatancy behavior of water-saturated sandstone. *Int. J. Min. Sci. Technol.* **2018**, *28*, 323–329. [\[CrossRef\]](#)
9. Wang, J.; Chen, X.; Wei, M. Triaxial fatigue behavior and acoustic emission characteristics of saturated tuff. *Int. J. Geomech.* **2021**, *21*, 04021230. [\[CrossRef\]](#)
10. Li, D.; Sun, Z.; Zhu, Q. Triaxial loading and unloading tests on dry and saturated sandstone specimens. *Appl. Sci.* **2019**, *9*, 1689. [\[CrossRef\]](#)
11. Cai, X.; Zhou, Z.; Du, X. Water-induced variations in dynamic behavior and failure characteristics of sandstone subjected to simulated geo-stress. *Int. J. Rock Mech. Min. Sci.* **2020**, *130*, 104339. [\[CrossRef\]](#)
12. Zhao, J. Rock joint matching coefficients and their effect on joint properties. *Chin. J. Rock Mech. Eng.* **1997**, *16*, 173–178.
13. Li, J.; Zhao, X.; Li, H. Analytical study for stress wave interaction with rock joints having unequally close-open behavior. *Rock Mech. Rock Eng.* **2016**, *49*, 3155–3164. [\[CrossRef\]](#)
14. Fan, L.; Ma, G.; Li, J. Nonlinear viscoelastic medium equivalence for stress wave propagation in a jointed rock mass. *Int. J. Rock Mech. Min. Sci.* **2012**, *50*, 11–18. [\[CrossRef\]](#)
15. Wang, Q.; Song, E.; Xu, P. Dynamic performance and stress wave propagation characteristics of parallel jointed rock mass using the SHPB technique. *KSCE J. Civ. Eng.* **2023**, *27*, 2275–2286. [\[CrossRef\]](#)
16. Chen, X.; Cai, M.; Li, J. Theoretical analysis of JMC effect on stress wave transmission and reflection. *Int. J. Min. Met. Mater.* **2018**, *25*, 1237–1245. [\[CrossRef\]](#)
17. Li, J.; Li, N.; Li, H. An SHPB test study on wave propagation across rock masses with different contact area ratios of joint. *Int. J. Impact Eng.* **2017**, *105*, 109–116. [\[CrossRef\]](#)
18. Ping, S.; Wang, F.; Wang, D. Mechanical damage induced by the water-rock reactions of gypsum-bearing mudstone. *Rock Mech. Rock Eng.* **2024**, *57*, 6377–6394. [\[CrossRef\]](#)
19. Zhu, J.; Deng, J.; Wang, P. Mechanical behavior and failure mechanism of rocks under impact loading: The coupled effects of water saturation and rate dependence. *Int. J. Geomech.* **2023**, *23*, 04023196. [\[CrossRef\]](#)
20. Zhuang, L.; Kim, K.; Diaz, M. Evaluation of water saturation effect on mechanical properties and hydraulic fracturing behavior of granite. *Int. J. Rock Mech. Min.* **2023**, *130*, 104321. [\[CrossRef\]](#)
21. Ma, S.; Xu, Y.; Xu, Z. Dynamic tensile characteristics of an artificial porous granite under various water saturation levels. *Geomech. Geophys. Geo* **2024**, *10*, 164. [\[CrossRef\]](#)
22. Jin, J.; Wang, L.; Yan, Z. Effects of strain rate and bedding on shale fracture mechanisms. *Int. J. Mech. Sci.* **2024**, *277*, 109398. [\[CrossRef\]](#)
23. Weng, L.; Wu, Z.; Li, X. Mesodamage characteristics of rock with a pre-cut opening under combined static-dynamic loads: A nuclear magnetic resonance (NMR) investigation. *Rock Mech. Rock Eng.* **2018**, *51*, 2339–2354. [\[CrossRef\]](#)
24. Boedoni, P. *Stress Waves in Solids*; Clarendon Press: Oxford, UK, 1953; pp. 1210–1211.
25. Li, X.; Zhou, Z.; Lok, T. Innovative testing technique of rock subjected to coupled static and dynamic loads. *Int. J. Rock Mech. Min. Sci.* **2008**, *45*, 739–748. [\[CrossRef\]](#)
26. Li, D.; Xiao, P.; Han, Z. Mechanical and failure properties of rocks with a cavity under coupled static and dynamic loads. *Eng. Fract. Mech.* **2020**, *225*, 106195. [\[CrossRef\]](#)
27. Liu, K.; Liu, H.; Zhou, Y. Experimental study on the dynamic mechanical and progressive fracture behavior of multi-jointed rock mass under repetitive impact loading. *Theor. Appl. Fract. Mech.* **2024**, *131*, 104416. [\[CrossRef\]](#)
28. Dai, F.; Huang, S.; Xia, K. Some fundamental issues in dynamic compression and tension tests of rocks using split Hopkinson pressure bar. *Rock Mech. Rock Eng.* **2010**, *43*, 657–666. [\[CrossRef\]](#)

29. Yin, Z.; Li, X.; Yin, T. Critical failure characteristics of high stress rock induced by impact disturbance under confining pressure unloading. *Chin. J. Rock Mech. Eng.* **2012**, *31*, 1355–1362.
30. Man, K.; Liu, X.; Song, Z. Dynamic compression characteristics and failure mechanism of water-saturated granite. *Water* **2022**, *14*, 216. [[CrossRef](#)]
31. Ban, L.; Du, W.; Zheng, D. Velocity-dependent effect on the peak shear strength of rock joints considering the distribution characteristics of contact joint surface. *Rock Mech. Rock Eng.* **2024**, *57*, 2523–2537. [[CrossRef](#)]
32. Pirzada, M.; Roshan, H.; Sun, H. Effect of contact surface area on frictional behaviour of dry and saturated rock joints. *J. Struct. Geol.* **2020**, *135*, 104044. [[CrossRef](#)]
33. Zhang, J.; Guo, W.; Fu, Y. Study on the size effect of the secant modulus of rocks containing multiple parallel joints. *Front. Mater.* **2022**, *9*, 992133. [[CrossRef](#)]
34. Wang, X.; Liu, Z.; Gao, X. Dynamic characteristics and energy evolution of granite subjected to coupled static-cyclic impact loading. *Geomech. Geophys. Geo* **2023**, *9*, 62. [[CrossRef](#)]
35. Chen, X.; Li, J.; Cai, M. A further study on wave propagation across a single joint with different roughness. *Rock Mech. Rock Eng.* **2016**, *49*, 2701–2709. [[CrossRef](#)]
36. Liu, X.; Xu, Z.; Guo, L. An energy-based model with non-symmetrical branch fractures of different roughness. *Bull. Eng. Geol. Environ.* **2024**, *83*, 320. [[CrossRef](#)]
37. Peng, K.; Liu, Z.; Zou, Q. Mechanical property of granite from different buried depths under uniaxial compression and dynamic impact: An energy-based investigation. *Powder Technol.* **2020**, *362*, 729–744. [[CrossRef](#)]
38. Zhou, Z.; Cai, X.; Li, X. Dynamic response and energy evolution of sandstone under coupled static–dynamic compression: Insights from experimental study into deep rock engineering applications. *Rock Mech. Rock Eng.* **2020**, *53*, 1305–1331. [[CrossRef](#)]
39. Yin, Z.; Li, X.; Jin, J. Failure characteristics of high stress rock induced by impact disturbance under confining pressure unloading. *Trans. Nonferrous Met. Soc.* **2012**, *22*, 175–184. [[CrossRef](#)]
40. Cadoni, E.; Labibes, K.; Albertini, C. Strain-rate effect on the tensile behaviour of concrete at different relative humidity levels. *Mater. Struct.* **2001**, *34*, 21–26. [[CrossRef](#)]
41. Liu, C.; Deng, H.; Chen, X. Impact of rock samples size on the microstructural changes induced by freeze-thaw cycles. *Rock Mech. Rock Eng.* **2020**, *53*, 5293–5300. [[CrossRef](#)]
42. Anovitz, L.; Cole, D. Characterization and analysis of porosity and pore structures. *Rev. Miner. Geochem.* **2015**, *80*, 61–164. [[CrossRef](#)]
43. Kolsky, H. An investigation of the mechanical properties of materials at very high rates of loading. *Proc. Phys. Soc. Sect. B* **1949**, *62*, 676. [[CrossRef](#)]

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.