The Attenuation Ability of Saturated Joints Filled with Granular Materials under High-Amplitude Stress Wave Loading

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Existing experimental evidence shows that the propagation of explosive waves in the free fields of soils is remarkably affected by the degree of saturation. In the surrounding rocks of underground protective structures, the underground water is normally unavoidable, which is supposed to reduce the isolation efficiency of a passive antiblast barrier. To investigate the effect of water saturation on the stress wave attenuation ability of infilled joints, impact tests were carried out on artificial joints filled with dry and saturated granular materials using a split Hopkinson pressure bar (SHPB). The test results revealed that under the same conditions, the stress and energy transmission coefficients of the waves crossing saturated sand-filled joints were about 3.16–4.13 times and 9.75–11.4 times those of joints filled with dry sand, respectively. The dynamic stress-strain relationship of the filling layer during the impact process and the crushing index of the infill were analyzed. The results showed that the compressibility and the granular crushing index of the dry sand were much greater than that of the saturated sand, and the dynamic stress-strain relationship of the dry sand exhibited three-stage nonlinear characteristics. The experimental results quantitatively uncovered the serious adverse effect of water on the wave absorption properties and markedly diminished the potential of the filled joints as a wave elimination barrier, which should be a matter of great concern in the design of underground protective structures.

1. Introduction

The design for an underground protective structure generally is based on the specified safety requirements according to the importance of the engineering. Considering the fact that the striking ability of modern weapons has been enhanced to destroy underground targets, for cases in which the thickness of the upper shelter rock mass is not sufficiently large, an additional structure must be constructed to attenuate the blast waves.

Since the 1990s, cracks and geological structural faults have been considered to be potential barriers to blast waves [1, 2]. Recently, Wang et al. and Liu et al. reported an innovative technical proposal for the passive explosion protection for underground caverns. Their experimental and numerical studies indicated that the stress wave could be significantly attenuated by constructing artificially filled cracks between the explosion source and the target [3, 4], thus greatly improving the safety of the protected structure. For their tests, however, neither the split Hopkinson pressure bar (SHPB) tests nor the model tests considered the influence of underground water. When groundwater submerged the artificially filled cracks, the wave attenuation capacity of cracks would be greatly reduced. Due to the underground water being almost unavoidable, the wave attenuation ability of filled joints in adverse environments needs to be quantitatively estimated. The propagation and attenuation of stress waves in rock mass is an issue that has attracted significant attention from researchers and engineers in the fields of geophysics, rock mechanics, tunneling, mining engineering, and other areas of research. When a stress wave passes through a joint, the filling materials will be compressed dynamically
and nonlinearly. The compressibility of the filling materials will greatly affect the wave attenuation capacity of the joint [5]. The complexity of this issue is caused by the internal hierarchy structures of rock fractures and joints, the wide range of fillings, and the vagaries of underground environments. The intensive studies conducted in recent years have provided researchers with insight into the dynamic behavior of rock joints. Several key problems, however, remain to be solved, especially those related to the effect of water. Numerous previous studies have indicated that the incident wave parameters, physical properties and thicknesses of the fillings, initial conditions, and boundary conditions of the joint all affect the wave propagation process [6–8].

For dry sand and a mixture of sandy soil, related studies have revealed that the transmission coefficient is generally very low, although it is affected by the lateral confinement effect [9], the initial density [10], the particle size [11], the amplitude of the incident wave [12], the thickness of the filled layer [13–15], and so on. When considering the real working environment of joints, the problem is complicated due to the existence of underground water. Natural soil generally is regarded as a three-phase medium. With the action of a stress wave, the load inside the three-phase medium is borne by the air, water, and grain skeleton. The compressibility of the grain skeleton does not change considerably when the load rises, while the compressibility of the mixture of air and water decreases sharply with the increase of the deformation as the degree of saturation is elevated. The dynamic constitutive relationship of partially saturated soils exhibits incremental hardening characteristics [16]. Among the three phases, gases are the most compressible compared with the others and thus control the stress wave attenuation ability of three-phase saturated granular materials [17]. The air content of highly saturated sand is very low. Consequently, its dynamic compression curve is very steep [18]. Therefore, it almost loses its ability to dissipate and attenuate the stress wave in a free field.

Compared with the cases in a free field, the propagation of a stress wave in saturated sand-filled joints is more complicated, but this effect has rarely been studied. Martin et al. [19] investigated the mechanical properties of fine sand with an HSR impact and found that with an increase in the moisture content (3–20%), the filling materials softened. It was presumed that pore water lowered the friction between the particles, consequently reducing the deformation stiffness of the filled joints. Wu and Zhao [20] investigated the attenuation pattern of P-waves for sandy soils with different saturation levels with a low amplitude impact and found that higher water content led to greater wave attenuation. They argued that a large number of water bridges existed in granular materials with high water content, which dissipated the incident energy during the breaking of the water bridges due to viscous stress. Wang et al. [21] investigated the dynamic response of two materials consisting of Stockton Beach sand and glass beads with saturations ranging from 0% to more than 90% for strain rates between 800 and 2100 s⁻¹. Both Stockton Beach sand and glass beads exhibited their softest stress-strain behavior at a saturation of 25% at an average strain rate of 1000–1300 s⁻¹ due to the lubricating effect of pore water, and glass beads experienced a greater degree of softening at any given saturation above zero. Wang et al. [22] further explored the influence of constraint condition changes on the dynamic response of partially saturated sand. An increase in the confinement rigidity caused a rise in stiffness, with lock-up occurring only in specimens confined to steel chambers. Based on these tests, Wang et al. [23] proposed a simplified empirical approach to quantify the uniaxial compressive stress-strain behavior of partially saturated granular media at a given saturation and initial dry density. Huang et al. [24] studied the deformation and fracture law of filling materials with different moisture content with the action of a high-amplitude stress wave. They found that when the moisture content was high, the transmission coefficient was much higher than that of the dry filling particles and basically independent of the incident wave amplitude. Huang et al. [25] proposed an analytical model to characterize the seismic response of viscoelastic filled joints and declared that the increase in the water content enhanced the viscosity, which could promote equivalent joint stiffness and energy dissipation for stress wave propagation.

Yang et al. [26] conducted laboratory tests of ultrasonic P-wave propagation across water-filled rock joints to investigate the effects of the joint thickness and water content on the compressional wave propagation in the ultrasonic frequency range. Their results showed that the increase of the joint thickness could cause more wave attenuation, while the increase in the water content could lead to less wave attenuation. Their further investigation indicated that the type of filling liquids in rock joints also played an important role in the seismic responses of jointed rock masses [27]. Huang et al. [28] performed a series of dynamic tests investigating stress wave propagation through a liquid-filled rock joint under undrained conditions, and they found a negative correlation between the peak water pressure value and the transmission coefficient.

According to the literature review, although the influence of water has already attracted attention, the propagation regularity and the attenuation mechanism of stress waves through filled joints are still not well understood, especially under the condition of a high degree of saturation. The few results reported in the literature are not consistent and even contradictory. The quantitative change of joint wave attenuation efficiency under saturation conditions remains unclear. Additionally, the distribution of stress between solid particles and pore water and determining artificial interventions of the filling materials to improve wave attenuation efficiency require further study.

In this study, a comparative investigation of the transmission of high-amplitude stress waves through rock joints filled with dry and saturated sand was conducted based on an SHPB test system, and the dynamic compression process and particle breakage characteristics were analyzed in detail. The results indicated that the wave transmission ability of the filling materials was greatly enhanced under saturated conditions. The transmitted energy under saturated conditions could be more than ten times that under dry conditions. For filled joints designed to attenuate blast waves, this degree of efficiency loss is unacceptable. Artificial intervention measures must be taken to improve the wave
attenuation capacity of filled rock joints that work below the groundwater level or are located in a pluvial region.

2. Method and Test Materials

2.1. SHPB Setup. In terms of experimental research on stress wave transmission in filled joints, the modified Kolsky method [29] based on SHPB technology has been widely used. The main characteristic of the modified Kolsky method is that a specimen is placed in a rigid jacket to limit the radial strain, and only one-dimensional strain exists in the longitudinal direction.

In this study, the steel SHPB system mainly consisted of a plenum chamber, a striker bar, an incident bar, a transmitted bar, a buffer unit, and a signal collection system (Figure 1). All of the bars were made of 35CrMnSiA low-alloy ultrahigh-strength steel with a density of 7850 g/cm$^3$, a longitude velocity of 5440 m/s, and Young’s modulus of 233 GPa. The incident bar, the transmitted bar, and the striker bar had the same diameter of 50 mm. The lengths of the incident bar, the transmitted bar, and the striker bar were 3000 mm, 2500 mm, and 400 mm, respectively.

In the sample preparation, the method of Luo et al. [10] was referred and an assembly was used to prepare sand specimens. The assembly consisted of two steel rods and a high-carbon-steel hollow cylinder. Figure 1 shows the schematic view of the experimental setup. The steel hollow cylinder had an inner diameter of 50 mm, an outer diameter of 60 mm, and a length of 30 mm. The steel rods were made of 35CrMnSiA low-alloy ultrahigh-strength steel (identical to the incident bar and transmitted bar to minimize the energy loss when the wave propagated through the interfaces) with a diameter of 49.5 mm and a length of 23 mm. The rods could slide freely in the sleeve. Vaseline was applied in the gap between the bars and the sleeve to stop the seepage of the water.

To record the complete dynamic strain process, the strain gauges were located at points 1400 mm away from the end of the incident bar and 1100 mm away from the start of the transmission bar. Because the distance from the strain gauges to the end of the bars was sufficiently long, the effect of the superposition of reflected waves on the data could be avoided. The data were recorded at a 1 MHz digital resolution. To induce better waveforms, a rubber sheet was used as the pulse shaper. The rubber sheets had an 8.0 mm diameter and 1.0 mm thickness.

2.2. Dynamic Stress Equilibrium. Before the SHPB tests, a trial test was conducted to verify the dynamic stress equilibrium of the equipment. The stress history $\sigma_z(t)$ at the two ends of the specimens could be calculated based on the two-wave method [30].

\[
\sigma_{z\text{-front}}(t) = \frac{A}{A_s} E [\varepsilon_i(t) + \varepsilon_r(t)],
\]

\[
\sigma_{z\text{-back}}(t) = \frac{A}{A_s} E \varepsilon_i(t),
\]

where $\sigma_{z\text{-front}}(t)$ is the stress history at the front end, $\sigma_{z\text{-back}}(t)$ is the stress history at the back end, $t$ is time, $A$ is the cross-sectional area of the steel bar, $A_s$ is the cross-sectional area of the specimen, and $E$ is the elastic modulus of the steel bar; $\varepsilon_i(t)$, $\varepsilon_r(t)$, and $\varepsilon_i(t)$ are the strains corresponding to the incident wave, reflected wave, and transmitted wave, respectively.

Figure 2 shows the stress history during the empty beating. According to Figure 2, the dynamic stress curves of the incident wave and the reflected wave were basically consistent with that for the transmitted wave after superposition, and this synchronicity indicated that the dynamic stress equilibrium was achieved. The positive pressure time of the loading pulse was approximately 475 $\mu s$.

2.3. Specimen Preparation and Test Process. The filling material was quartz sand, and the majority of the particle sizes were between 0.5 mm and 1 mm. The particles with particle sizes of 0.5 mm–0.8 mm accounted for about 70% of the total, and the particles with particle sizes of 0.8 mm–1.0 mm accounted for about 30% of the total. The initial dry density and the void ratio of the quartz sand were 1.567 g/cm$^3$ and 41%, respectively. The tested samples were confined by a 6 mm thick cylindrical shell.

The amplitude of the stress waves was adjusted using the gas pressures, which were set to 0.15 MPa, 0.20 MPa, and 0.25 MPa. Two working conditions, dry and saturated, were adopted to quantify the influence of water. When preparing the dry specimens, the cylindrical shell was filled with 18.45 g of dry quartz sand, and a 6 mm thick layer of fillings was formed. The initial relative compaction of the sand was controlled to be the same by the identical volume and mass. When preparing the saturated specimens, 18.45 g of dry sand was first put into the sleeve. Then, the sleeve was placed into the vacuum saturation device, and water was added until the sample was completely submerged. The vacuum pump was opened to pump the saturation device to a vacuum state, soaking was performed for 10 minutes, and then, the sample was taken out. Vaseline was applied in the gap between the sleeve and the rods to stop the seepage of the water. The saturated moisture content of the saturated quartz sand was 27%. Additionally, during the test, the specimens were undrained. The test equipment and a typical specimen are shown in Figure 3.

There were six working conditions in the test. Two parallel tests were carried out in each group of working conditions, and a total of twelve tests were conducted, with test numbers from D-1 to D-6 and S-1 to S-6 in sequence. Specimens were collected in sealed bags after the test, and the particle size distribution of the sand after impact was analyzed by screening.

3. Test Results

The incident waves induced by different gas pressures and the corresponding transmitted waves recorded by the strain gauges are plotted in Figure 4.

As shown in Figure 4, under the condition of a similar incident wave amplitude, the amplitudes of the transmitted waves of the saturated sand were much higher than those of the dry sand. When the gas pressure was 0.15 MPa, the amplitudes of the transmitted waves of the dry sand and the saturated sand ranged from 13.3 MPa to 14.7 MPa and
from 48.9 MPa to 51.4 MPa. When the gas pressure was 0.20 MPa, the amplitudes of the transmitted waves of the dry sand and the saturated sand ranged from 28.7 MPa to 31.6 MPa and from 102.2 MPa to 118.8 MPa, respectively. When the gas pressure was 0.25 MPa, the amplitudes of the transmitted waves of the dry sand and the saturated sand ranged from 40.3 MPa to 44.8 MPa and from 142.0 MPa to 144.1 MPa, respectively. The amplitudes of the transmitted waves of the dry sand and saturated sand steadily increased when the amplitude of the incident wave increased. The amplitude of the transmitted wave of the saturated sand was 3.16–4.13 times that of the dry sand.

To quantify the wave attenuation ability of the joint, the wave transmission coefficient based on the peak stress was calculated using the test data. The wave transmission coefficient was defined as the ratio of the amplitude of the transmitted wave to that of the incident wave \([31]\), that is,

\[
T = \frac{\sigma_{t, \text{max}}}{\sigma_{i, \text{max}}} = \frac{\varepsilon_{t, \text{max}}}{\varepsilon_{i, \text{max}}},
\]

where \(T\) is wave transmission coefficient, \(\sigma_{i, \text{max}}\) and \(\sigma_{t, \text{max}}\) are the peak stresses of the incident wave and the transmitted wave, and \(\varepsilon_{i, \text{max}}\) and \(\varepsilon_{t, \text{max}}\) are the strains corresponding to the peak stresses of the incident wave and the transmitted wave.

The basic parameters and the main results of the test are given in Table 1.

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**Figure 1: Split Hopkinson pressure bar.**

**Figure 2: Typical response signals of incident, reflected, and transmitted waves.**
According to Table 1, for the same gas pressure, the incident wave amplitude was similar and the test conditions were basically the same. The corresponding test results showed that the two groups of tests with the same working conditions had good repeatability and high reliability.

Using the specific values listed in Table 1, the scatter diagram of the wave transmission coefficient versus the gas pressure is shown in Figure 5. The mean wave transmission coefficient of each group of working conditions was wired to observe the change trend of wave transmission coefficient with the gas pressure.

As shown in Figure 5, the wave transmission coefficient of the saturated sand was much higher than that of dry sand and over three times that of dry sand. With the increase in the amplitude of the incident wave, the average wave transmission coefficient of the saturated sand had little variation, and the average wave transmission coefficient fluctuated slightly between 68.88% and 71.79%. The wave transmission coefficient of the dry sand was proportional to the wave amplitude, while that of the saturated sand did not exhibit a wave amplitude effect.

These results indicated that the saturation condition greatly reduced the wave attenuation ability of the filled materials. Additionally, the wave transmission coefficient of the saturated fillings was consistent with that in a previous study by Huang et al. [24] based on the saturated specimens of a quartz sand and kaolin clay mixture. Note, however, that the dynamic compression characteristics, wave propagation mechanism, pore pressure, effective stress distribution, and grain crushing characteristics of filling materials under saturated conditions have not been reported previously.

4. Dynamic Compressive Deformation Characteristics of Granular Materials

4.1. Dynamic Stress-Strain Relationship. Using the microstrain signals recorded by the strain gauges of the incident bar and the transmission bar, the dynamic stress histories, dynamic strain histories, and strain rate of the specimen under impact were calculated based on one-dimensional wave theory [32]. The calculation formula is given as follows:

\[
\varepsilon(t) = \frac{2C}{L} \int_0^t \varepsilon_i(t) dt, \tag{3}
\]

\[
\dot{\varepsilon}(t) = \frac{2C}{L} \varepsilon_i(t), \tag{4}
\]

\[
\sigma(t) = E \frac{A}{A_s} \varepsilon_i(t), \tag{5}
\]

where \(C\) is the wave velocity within the steel bar and \(L\) is the initial specimen length.

Using formula (4), the peak strain rates for different gas pressure conditions were calculated. When the gas pressures were 0.15, 0.20, and 0.25 MPa, the average peak strain rates of the dry sand were 504.6 s\(^{-1}\), 985.4 s\(^{-1}\), and 1295.0 s\(^{-1}\), respectively, while those for the saturated sand were 420.1 s\(^{-1}\), 551.5 s\(^{-1}\)
1, and 654.5 s⁻¹. These results implied that the peak strain rate of the samples was sensitive to the amplitude of the stress wave, and the strain rate of the dry sand was more sensitive.

By substituting the test data into formulas (3) and (5), the dynamic strain and the dynamic stress of the filled layers under different working conditions were calculated. The
The corresponding dynamic stress-strain curve data for the samples numbered D-1, D-3, D-5, S-1, S-3, and S-5 are drawn in Figure 6.

As shown in Figure 6(a), the dynamic stress-strain curve for the dry sand could be roughly divided into three stages. Taking specimen D-5 as an example, the OA segment before the curve reached the first peak value was approximately regarded as the elastic deformation stage. The slope of the curve was basically unchanged, and the strain increased linearly with the stress. After reaching the first peak value point, the strain-strain curve had an inflection point and decreased (AB segment). The specific performance was that the strain continued to increase and the stress decreased slightly. At that stage, some particles were crushed under dynamic pressure, and some of the original force chains were damaged by the impact, which resulted in the rearrangement of particles and the decrease of the deformation stiffness. With the evolution of the dynamic compression and the decrease of the specimen porosity, the stress again entered the fluctuation rising stage (BC segment) until the end of the loading process. If, however, the stress result from the incident wave was not high enough, after the end of the linear section, there would be no notable stress drop because of less particle crushing. Although the nominal deformation

| Test number | Working condition | Gas pressure (MPa) | Incident wave amplitude (MPa) | Transmitted wave amplitude (MPa) | Transmission coefficient (%) | Average transmission coefficient (%) | Peak strain | Ultimate strain | Peak strain rate |
|-------------|-------------------|--------------------|-------------------------------|-------------------------------|-----------------------------|-------------------------------------|-------------|-----------------|-----------------|
| D-1         | Dry               | 0.15               | 80.4                          | 14.7                          | 18.28                       | 17.87                               | 0.1018      | 0.1233          | 511.7           |
| D-2         | Dry               | 0.20               | 155.2                         | 28.7                          | 18.49                       | 19.83                               | 0.1309      | 0.1797          | 1011.0          |
| D-3         | Dry               | 0.25               | 198.1                         | 44.8                          | 22.61                       | 21.11                               | 0.1485      | 0.2080          | 1224.0          |
| S-1         | Saturated         | 0.15               | 74.4                          | 51.4                          | 69.08                       | 68.88                               | 0.0443      | 0.0760          | 447.4           |
| S-2         | Saturated         | 0.20               | 157.3                         | 118.8                         | 75.52                       | 71.49                               | 0.0706      | 0.0975          | 561.5           |
| S-3         | Saturated         | 0.25               | 199.0                         | 144.1                         | 72.41                       | 71.79                               | 0.0718      | 0.1170          | 614.6           |

**Table 1:** The basic parameters and main results of the test.
EGH = 0.109 GPa
EEF = 0.091 GPa
EDE = –0.023 GPa
EBC = 0.172 GPa
E50(D-1) = 0.328 GPa
E50(D-3) = 1.102 GPa
E50(D-5) = 1.814 GPa
EAF = –0.073 GPa
E_{BC} = 0.023 GPa
E_{DE} = 0.109 GPa

Figure 6: Dynamic stress-strain curves of specimens. (a) D-1, D-3, and D-5. (b) S-1, S-3, and S-5.
modulus decreased after the G point, the stress still fluctuated and increased with the development of the strain (GH segment).

The stress-strain curves of the saturated sand specimens exhibited very different characteristics. Compared with the curves of the dry sand, they were smoother and steeper during the dynamic loading. Because of the rigid passive confinement, the water in the specimen could not be discharged immediately during the impact, and the compressibility of the water was very small. Therefore, the total deformation of the specimen was limited and was only about half of that of the dry sand. In addition, the total stress was much higher than that under dry conditions. The saturated sand specimen had a large initial stiffness and a rapid pressure rise, but the nominal deformation modulus decreased slowly with an increase in the compression deformation. This change trend might have been related to the difficulty of strictly maintaining the undrained conditions during compressions. A small amount of water in the specimen was extruded into the gap of the confinement chamber, which disguised the increase in the nominal compressibility of the specimen.

4.2. Dynamic Deformation Modulus. The deformation modulus is an important index to reflect the stiffness of material deformation. According to the dynamic stress-strain curves shown in Figure 6, the variation of the dynamic deformation modulus of the specimens could be further analyzed. For the saturated sand, the deformation modulus was expressed by the ratio \( E_{50} \) of the stress and strain corresponding to half of the peak stress of the specimen and the secant modulus \( E \), corresponding to the peak stress point. The three stages of deformation were also defined for the dry sand. The initial approximate elastic deformation stage was described by the ratio \( E_{50} \) of the stress and strain corresponding to half of the peak stress of the specimen. The platform stage and the continuous deformation stage were expressed by the slope of the connecting line between the highest point and the lowest point in the interval.

For the dry sand, the deformation modulus in the first deformation stage (approximate elastic deformation stage) increased with the increase of the incident wave amplitude, increasing from 0.328 GPa and 1.102 GPa to 1.814 GPa. The deformation modulus increased faster than the amplitude of the incident wave. In the second deformation stage (stress platform segment), under the condition of a high-amplitude incident wave, the deformation modulus was negative, and the stress dropped while the strain continued increasing. The amplitude of the stress drop increased with the increase of the amplitude of the incident wave. This phenomenon was likely related to the particle crushing. The larger the incident wave amplitude was, the greater the particle crushing degree was, and the more the overall deformation modulus of the specimen decreased. When the incident wave amplitude was low, the amount of particle crushing was lower, and the deformation modulus decreased, but the growth trend of the stress did not reverse.

After entry into the third deformation stage, the stress fluctuation increased and the overall deformation modulus was lower than that of the first stage. However, with the increase of the incident wave amplitude, the deformation modulus of this section also increased. The specific value increased from an \( E_{50} \) of 0.109 GPa to an \( E_{BC} \) of 0.172 GPa. The larger the amplitude of the incident wave was, the faster the compaction speed of the specimen was and the more obvious the dynamic amplification effect was.

Omidvar et al. [33] pointed out that the typical static stress-strain response of dry sand under uniaxial compression and no transverse strain could be divided into four stages: (1) the elastic deformation of individual grains, (2) the yielding of the particle skeleton, (3) the rearrangement of the sand grains and the particles resulting in “lock-up” into a denser arrangement, and (4) the start of individual grains to crush, allowing the skeleton to contract further. In the dynamic impact compression test carried out in this research, in the dry condition, the dynamic stress-strain curve of the specimen followed a similar change trend. This unstable stress-strain relationship reflected the crushing of the granular materials and the adjustment of the stress chains within the specimen. When the load action form and the amplitude were different, the manifestation of this trend was also different. Wang et al. [34] observed particle crushing in different contact forms in coral sand under a static load. Although quartz sand has high strength and good roundness, a similar crushing process would occur under a high impact load, resulting in the fluctuation of the stress-strain curve. Huang et al. [15] also found the fluctuation of the stress-strain curve caused by particle crushing in an impact test on dry sand. Because of the different properties of granular materials and test settings, the test curve in this research was smoother after particle crushing.

As shown in Figure 6(b), the tangent modulus and the secant modulus of the saturated sand were much higher than the dynamic deformation modulus of the dry sand (more than one order of magnitude). Under the same loading condition, the total stress in the saturated sand was much higher and the deformation was smaller, which reflected the fact that the influence of water on the compressibility of the saturated specimens was significant. Water occupied the pores between the particles and expelled the air, which greatly reduced the overall compressibility of the sand. Considering the fact that the applied load was very fast and the water was confined in the sample, the water in the particle pores could be regarded as not moving [30], which contributed to the rapid rise of stress. Therefore, the dynamic compression constitutive relationship of the saturated rock joints showed “harder” behavior.

4.3. Dynamic Compression Deformation. Dynamic compression deformation is an important index for measuring the compressibility of materials under impact loads. It can be reflected by the peak strain (nominal strain corresponding to peak stress) and ultimate strain (nominal strain after impact compression). Using the data in Table 1, a scatter diagram with error bars is shown in Figure 7.

Also, as shown in Figure 7, with the increase of the incident wave amplitude, the peak strain and the ultimate strain of the dry sand and saturated sand followed an upward
trend. The peak strain and the ultimate strain were approximately linear in relation to the amplitude of the incident wave. For the same incident wave amplitude, the ultimate strain of the saturated sand was approximately one-half of that of the dry sand. The slope of the fitting curve of the saturated sand was lower than that of the corresponding dry sand fitting curve.

The peak strain rate of the infill was also approximately linearly related to the amplitude of the incident wave. The greater the incident wave amplitude was, the greater the peak strain rate was. When the gas pressure increased from 0.15 MPa to 0.25 MPa, the peak strain rate of the dry sand increased from 511.7 s\(^{-1}\) to 1224.0 s\(^{-1}\), an increase of 139.1%. The peak strain rate of the saturated sand increased from 447.4 s\(^{-1}\) to 614.6 s\(^{-1}\), an increase of only 37.6%. Apparently, the peak strain rate of the dry sand was more sensitive to the incident wave amplitude.

5. Particle Crushing of the Infill

Observations indicated that during the high-amplitude impact, many of the particles of the infill were crushed. The particle crushing not only was related to the dynamic compression process but also determined the energy dissipation of the stress wave when passing through the filled layer. Particle crushing is affected by many factors, such as the initial void, particle size, particle grading composition, strain rate, pattern of the load, and restriction from the outside [35, 36]. However, under the saturation condition, the whole stress consisted of effective stress and pore water pressure. Because the water was approximately incompressible, while the pore water pressure was spherical stress, the particle crushing of the saturated sand had many differences from that of the dry sand.

To quantify the particle crushing, the relative breakage potential \(B\) proposed by Hardin was adopted in this research, which was defined as [37]

\[
B = \frac{B_i}{B_p},
\]

where \(B_p\) is the breakage potential, which was defined as the area between the lower bound of the 0.074 mm sieve opening and the original size distribution curve, and \(B_i\) is the total breakage, which was defined as the area between the original and the final size distribution curves.

The particle size distributions of D-1-S-6 before and after impact are drawn in Figure 8(a). The relative breakage potential was calculated and drawn as shown in Figure 8(b).

Figure 8 shows that the particle crushing degree of the saturated and dry particles after impact was positively correlated with the gas pressure. The larger the gas pressure was, the greater the particle crushing degree was. When the impact amplitude was 76.1 MPa–80.4 MPa, about 17% of the dry sand had a particle size of less than 0.5 mm. Only a few of the particles were crushed, and compaction played a dominant role in the deformation process. When the impact amplitude was 149.3 MPa–155.2 MPa, about 40% of the dry sand had a particle size of less than 0.5 mm. The effective stress of the particles was much higher than the compressive strength, and numerous particles were crushed. When the impact amplitude was 198.1 MPa–205.5 MPa, about 40% of the dry sand had a particle size of less than 0.25 mm. Most of the particles with a large particle size were crushed and compacted, and the total deformation was the largest.

Figure 8(b) shows the plot of index \(B\). With the increase of the impact amplitude, the average particle crushing degrees of the saturated sand also increased slightly (0.02, 0.05, and 0.08), although the average relative breakage potentials were much lower than those of dry sand (0.12, 0.31, and 0.43). The relative breakage potential of the saturated sand with the action of the maximum amplitude incident wave was still lower than that of the dry sand with the action of the minimum amplitude incident wave.

The proportions of the particles of the saturated samples below 0.5 mm were all less than 10% for the three gas pressure conditions. At the same time, the grading curve showed that the proportion of 0.5–0.8 mm saturated sand particles increased slightly. It was speculated that the sand surfaces were rough and vulnerable to local stress concentration. The edges of some 0.8–1 mm particles were broken with the friction effect.

The total stress in the saturated sand could be divided into two parts: (1) the pore water pressure borne by water and (2) the effective stress borne by the grain skeleton. The pore water pressure was categorized as hydrostatic pressure, which only caused volume compression without distortion, and its contribution to particle crushing could be ignored. However, the effective stress could cause strong stress concentration at the contact point between particles, which was the main reason for particle crushing.

From the particle crushing, it could be inferred that when the saturated sand specimen was subjected to an impact load, most of the stress was borne by the rapidly rising pore water pressure, while the particle skeleton bore only a small part of the load. When the average incident amplitude was 199.25 MPa, the relative breakage potential of the saturated sand was lower than that of dry sand when the average incident amplitude was 78.25 MPa. Therefore, it could be inferred that the effective stress acting on the particle skeleton of the saturated sand with an incident wave with 199.25 MPa amplitude was lower than 14.0 MPa (average transmission amplitude of dry sand), and the rest of the stress (90%) was borne mainly by the water. Considering the fact that the pore water pressure evolution during impact had not been recorded previously, this estimation of the stress distribution relationship was very important. An indirect estimate of the effective stress with the comparative investigation of particle crushing is worthy of further study.

6. Discussion

6.1. The Dynamic Compression of Sand. The effects of the strain rate and loading intensity play a significant role in the load deformation response of granular materials. Iskander et al. [38] presented a comprehensive summary of the different uniaxial compression tests of soil or sand samples performed at an HSR. The deformation behavior of dry sand
Figure 7: Variation diagram under each working condition. (a) Peak strain and ultimate strain. (b) Peak strain rate.
Figure 8: (a) Particle size distribution curves of D-1~S-6 specimens after impact. (b) Variation diagram of average relative breakage potential under different working conditions.
under dynamic uniaxial compression is schematically illustrated in Figure 9 [38, 39].

To explore the stress-strain behavior of dry sand at an HSR, the results of Lin et al. [39] and De Cola et al. [40] are cited, and their data are redrawn in Figure 10(a), as well as D-1, D-3, and D-5 for this study.

As shown in Figure 10(a), the response of the dry sand exhibited two modes: with and without a stress drop during the loading stage. For the dry Ottawa Sand samples tested by Lin et al. [39], the initial elastic response and the yielding of the sand skeleton seemed to be independent of the strain rate for all the initial void ratios. The strain rate effects became more apparent after the particle crushing and particle rearrangement began. No stress drop occurred during the impact loading process. In the work conducted by De Cola et al. [40], the compressive responses of Ottawa Sand with quasispherical grains, Euroquartz Sand with subangular grains, and Q-Rok with polyhedral grain shapes were investigated using long-SHPB experiments. Four phases of the compression response behavior could be identified by observing the stress-strain curve. An apparent stress drop occurred after the initial elastic stage, followed by a denser state produced by the rearrangement of sand grains, and a “lock-up” tendency (characterized by an increase in stiffness) was observed.

Compared with the result reported by Lin et al. [39] and De Cola et al. [40], the curves of D-1, D-3, and D-5 showed some new features. For a relatively low gas pressure condition, the impact stress and strain rates of D-1 were the lowest among the dry samples. Additionally, the stress-strain response displayed a three-phase mode, and no stress drop was observed, which was similar to the curves given by Lin et al. [39]. After the initial elastic response, the yielding of the sand skeleton occurred, which was characterized by lower deformation stiffness. At the last stage, the particle crushing began and led to fluctuations in the stiffness as the stress chains collapsed and void spaces became occupied by progressively crushed sand particles. When the impact stress and strain rate increased (D-3 and D-5), the shapes of the strain-stress curves changed to mode 2, which was characterized by the longer initial elastic response stage with the higher stiffness and apparent stress dropped. The stiffness of the initial elastic response of D-3 and D-5 was roughly equivalent to the results of dry Euroquartz Sand and dry Ottawa Sand reported by De Cola et al. [40], while the stress drops that were observed were relatively lower at approximately the same strain rate. In the phase following the stress drop, the fluctuations of the strain-stress curve of D-3 and D-5 were obvious, which was not observed in the results of the cited studies. The supposed hardening phase was not observed in this study. It was presumed that the impact loading process finished before the sample entered the hardening phase. To improve the insight into the material behavior under more extreme conditions, the SHPB equipment had to be modified to provide longer loading pulses to reach a higher absolute strain (above 20%).

In contrast to dry sand, the majority of the voids in the fully saturated sand were filled by water and thus exhibited much lower compressibility. The stress-strain response of the sand with an increased degree of saturation was complex, and the air content played a key role in the dynamic compression process. Because the water could not be drained during the loading process at the HSR, the deformation of the sand was determined by the deformation of the water (and a very small amount of air).

In Figure 10(b), the results of Wang et al. [21], Lin et al. [39], and Varley et al. [41] are cited for comparative purposes. The degree of saturation of the Ottawa Sand studied by Lin et al. [39] was fully saturated and the other samples had 90–100% degrees of saturation.

The deformation curves of the fully saturated sand were governed by the water (which had a considerably lower compressibility than the air and soil skeleton) occupying the entirety of the void space. The water accommodated the majority of the hydrostatic stress within the sample, resulting in fewer particle breakages and a near-linear strain stress response. The initial modulus of S-3 at a strain rate of 561 s⁻¹ was almost the same as that observed by Lin et al. [39] in the dynamic compression of saturated Ottawa Sand. The deformation modulus only decreased and deviated from the linear relationship in the final section of loading. This deviation in the results could be attributed to the diversity of the geomaterials and the difficulty of maintaining the same boundary conditions (fully saturated condition, rigid lateral confinement, and strict undrained boundary). Comparing the results of S-1, S-3, and S-5, the secant modulus exhibited an apparent strain rate effect, whereas the stress transmission coefficient remained almost unchanged.

When the degree of saturation is less than 100%, because pore air offers little resistance to HSR loading, the dynamic response of dry or unsaturated sand is considerably softer than that of fully saturated sand, even if there is a very small amount of air [21, 41]. The response of partially saturated sand is very different from that of fully saturated sand under HSR loading, which depends on the initial compaction, applied stress level, strain rate, and degree of saturation.

Although many dynamic tests have been performed, the behavior of sand at HSR is still not fully understood for varying degrees of saturation, relative density, particle shape, and grading. To accurately model the response of soil during impact, blasting, and dynamic compaction, rigorous and well-defined testing methodologies must be developed to properly quantify the soil’s material parameters.

6.2 The Stress Transmission Coefficient. The wave transmission coefficient $T$ defined based on the peak stress of the incident wave and the transmitted wave represents the wave attenuation capacity of a specimen but not the infill materials, which are greatly affected by the thickness of the specimen. It is not scientific to directly compare the results from different research studies because of the different sample configurations. To describe the stress attenuation ability of granular material quantitatively, a dimensionless modified attenuation coefficient $k$ was proposed by removing the relative impedance of the bar to the specimen and considering the effects of the size of the specimen [42]. The modified attenuation coefficient could be obtained...
from the SHPB tests using a constant bar, as follows:

\[ k = -\frac{D}{L_s} \ln(T), \]  

(7)

where \( L_s \) is the length of the specimens, \( D \) is the diameter of the specimen, and \( T \) is the wave transmission coefficient.

With the help of the modified attenuation coefficient \( k \), the results of Huang et al. [12, 24] and Lv et al. [42, 43] were redrawn and compared, as shown in Figure 11.

Figure 11(a) shows that although the test materials and configurations vary, the modified attenuation coefficient is not sensitive to the amplitude of the incident wave. The results of this study, under both dry and saturated conditions, were obviously larger than those of Huang et al. [24], although still apparently larger than the results of Lv et al. [42, 43]. It is generally known that the static and dynamic compressibility of calcareous sand is better than that of silica sand [44, 45], and the abnormally low value of \( k \) might be attributed to the initial compaction of the specimens and the discrepancy of the experimental settings.

In formula (7), the modified attenuation coefficient \( k \) is proportional to the specimen diameter. With the assumption that the specimen diameter is large enough that the specimen can be regarded as infinite, the modified attenuation coefficient \( k \) becomes infinitely large, which implies that the planar wave would be fully dissipated by the filled joint. The assumed case is clearly not realistic. A new index is needed to quantitatively compare the results of the attenuation coefficient of different studies with different specimen sizes and experimental settings.

The specimen diameter for this study was 50.0 mm, which was about two times larger than those of Huang et al. [12, 24] and Lv et al. [42, 43] (25.4 mm). If the interference of \( D \) was removed by dividing the ratio of 50/25.4, the results of our study were consistent with those of Huang et al. [24], although still apparently larger than the results of Lv et al. [42, 43]. It is generally known that the static and dynamic compressibility of calcareous sand is better than that of silica sand [44, 45], and the abnormally low value of \( k \) might be attributed to the initial compaction of the specimen and the discrepancy of the experimental settings.

The degree of saturation is very important for the stress wave attenuation ability of sand, especially when the degree of saturation is high. However, compared with the dynamic compression of dry and partially saturated sand, the experimental data for the wave attenuation ability of a joint filled with fully saturated sand rarely has been reported. Although the available data have poor consistency, the fact is clear that the stress wave attenuation ability of a filled joint is seriously weakened by the fully saturated condition. Considering the fact that the blast protective barrier of an underground structure generally is located under the water table, more attention should be paid to investigating the influence factors of infill materials and working conditions. Additionally, the artificial intervention of the wave attenuation ability of a filled joint is necessary to ensure the efficiency of protective structures in an adverse environment.

6.3. Transmission Coefficient Based on Energy. Based on one-dimensional stress wave theory and uniformity assumption, the incident energy \( W_i \), reflected energy \( W_r \), and transmitted energy \( W_t \) are calculated:

\[ W_i = \frac{A}{\rho C_0} \int_0^t \sigma_i^2(t) dt, \]  

(8)
Lin et al. (39), dry C109 ASTM # C778 graded sand, $e = 0.71$
Lin et al. (39), dry ASTM C778 # 20–30 graded sand, $e = 0.68$
Cola et al. (40), dry euroquartz sand, $e = 0.72$
Cola et al. (40), dry Q-Rok sand, $e = 0.67$
Cola et al. (40), dry ottawa sand, $e = 0.67$
D-1, dry fujian standard sand, $e = 0.69$
D-3, dry fujian standard sand, $e = 0.69$
D-5, dry fujian standard sand, $e = 0.69$

Wang et al. (21), stockton beach sand (Saturation = 90% +)
Varley et al. (41), loose wet volcanic sand (Saturation = 95% –100%)
S-5, saturated fujian standard sand
S-3, saturated fujian standard sand
S-1, saturated fujian standard sand

Figure 10: The stress-strain behavior of sand at HSR.
Figure 11: The variation of a modified attenuation coefficient.

(a) vs. amplitude of the incident wave

(b) vs. degree of saturation

- Huang et al. (24), dry quartz sand
- Huang et al. (24), saturated quartz sand
- Huang et al. (12), dry coarse sand
- Dry fujian standard sand
- Saturated fujian standard sand

Modified attenuation coefficient $k$

Amplitude of the incident wave (MPa)

Degree of saturation $S_r$ (%)
Figure 12: Energy distribution histogram of (a) dry sand and (b) saturated sand.
where $\rho$ is the density of the steel bar and $\sigma_i(t)$, $\sigma_r(t)$, and $\sigma_t(t)$ are the stresses corresponding to the incident wave, reflected wave, and transmitted wave, respectively.

Ignoring the energy lost at the interface between the specimen and the bar, the dissipated energy of the specimen can be expressed as

$$W_d = W_i - W_r - W_t, \quad (11)$$

where $W_d$ is the dissipated energy of the specimen during the impact, mainly including the crushing energy, friction energy, acoustic energy, thermal energy, radiation energy, and other forms of dissipated energy.

Based on test data of specimens D-1–S-6, the average energy distribution histogram of the dry sand and saturated sand with different gas pressure is shown in Figure 12.

As shown in Figure 12(a), 62.45%–72.33% of the incident energy of the dry sand was reflected back, and the lower the incident amplitude was, the greater the proportion of the reflected energy was. Only 27.67%–37.55% of the incident energy could be transferred into the sand. About 90% of the energy entering the sand was dissipated by particle breakage and friction deformation. The transmission energy accounted for only 2.78%–4.57% of the incident energy in proportion.

As shown in Figure 12(b), compared with dry sand, the energy distribution in the saturated sand showed different behavior. Between 60.31% and 87.31% of the incident energy entered the filled layer, which was notably higher than that of the dry sand. Additionally, 51.03%–52.52% of the energy entering the filled layer continued to be transmitted. The transmission ratio was stable, implying that the transmission capacity of the saturated sand was an inherent property that was basically independent of the amplitude of the incident wave. The ratio of the transmitted energy to the incident energy increased from 30.87% to 44.56% when the gas pressure was elevated from 0.15 MPa to 0.25 MPa, while the ratio of the dissipated energy to the incident energy increased from 29.44% to 42.75%.

Note that from the point of view of the energy, when the incident wave amplitude was similar, the proportion of transmitted energy in the incident energy of the saturated sand was 9.75–11.4 times that of the dry sand. In other words, the filled fissures that are designed as barriers against blasts will have a sharp efficiency reduction in wave attenuation capacity once they are submerged by groundwater (which is usually unavoidable). If one evaluates the wave absorption efficiency as recommended by the design code (without considering the influence of water), it is highly possible to overestimate the survival probability of protective structures. Determining how to effectively ensure the stress attenuation ability of a filled joint under a water saturation condition is a problem worthy of in-depth study.

The dissipated energy was consumed mainly by particle crushing, the deformation of the soil skeleton, and the viscosity of the water. Based on the test data, the scatter diagram of the average dissipated energy versus the average relative breakage potential is shown in Figure 13.

With linear fitting, it could be found that the relative breakage potential was approximately linearly correlated with the dissipated energy. The average relative breakage
potential $B$ and the ultimate strain of the dry sand for the same gas pressure condition were much smaller than those for the saturated sand. The crushing degree of the saturated sand with the action of the maximum amplitude incident wave was slightly lower than that of dry sand with the action of the minimum amplitude incident wave. This implied that most of the dynamic stress of the saturated sand was borne by the rapidly rising pore water pressure. The effective stress of the particle skeleton bore only a small part of the load. The smaller particle breakage and ultimate strain of the saturated sand also implied that a considerable part of the dissipated energy of the saturated sand was absorbed by water.

7. Conclusions

In this research, the propagation and attenuation characteristics of a high-amplitude stress wave through joints filled with dry and saturated sand were investigated. The conclusions were as follows:

(1) The wave transmission coefficient of the saturated sand was much larger than that of the dry sand. Under the condition of a similar incident wave amplitude, the stress transmission coefficient of the saturated sand was about 3.16–4.13 times that of the dry sand, and the energy transmission coefficient was about 9.75–11.4 times that of the dry sand.

(2) With the increase of the incident wave amplitude, the stress transmission coefficient of the dry sand-filled joints increased from 17.87% to 21.11%, with an increase range of about 18.13%, and the energy transmission coefficient increased from 2.78% to 4.57%, with an increase range of about 64.39%. The stress transmission coefficient of the saturated sand-filled joints fluctuated slightly in the range of 68.88% to 71.79%, which was somewhat independent of the change of the incident wave amplitude, while the energy transmission coefficient increased from 30.87% to 44.56%, an increase of about 44.84%.

(3) The dynamic stress-strain curve of the dry sand showed obvious stage characteristics, whereas the dynamic stress-strain curve of the saturated sand had no periodic characteristics. With different incident wave amplitudes, the dynamic compressive modulus of the saturated sand was much higher than that of the dry sand, while the average ultimate strain was much lower. The dynamic deformation modulus of the fully saturated sand exhibited a notable strain rate effect.

(4) With the action of a high-amplitude stress wave, the crushing degree of the saturated sand was considerably smaller than that of the dry sand. The water bore most of the dynamic loading, reduced the effective stress of the sand skeleton, and lowered the relative breakage potential.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

No potential conflict of interest was reported by the authors.

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