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Mechanical Properties and Constitutive Model Optimization of Rockfill Materials for CFRD with Deep Overburden Layer

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Received: 13 July 2026; Accepted: 21 September 2026; Published: 21 October 2026

ABSTRACT: Computer modeling and numerical calculation of concrete face rockfill dams (CFRDs) with deep overburden layers remains challenging because the mechanical response of rockfill materials is highly nonlinear, pressure-dependent, and influenced by particle breakage. To improve the predictive capability of such simulations, this paper develops a nonlinear constitutive model that incorporates a dynamic failure ratio, the spatially mobilized plane criterion, and particle breakage, and establishes a calibration procedure for its numerical implementation. Large-scale consolidated-drained triaxial tests were conducted on three gradations under five confining pressures ranging from 200 to 1000 kPa. The peak deviatoric stress increased with confining pressure, whereas the peak friction angle decreased, and the specimens exhibited contraction followed by dilatancy, particularly at 200–600 kPa. The calibrated model was implemented in a finite-difference computational framework. The framework was applied to the Jinchuan CFRD using an engineering-scale numerical model. The computed maximum settlement was 84.8 cm at approximately 0.25 times the dam height, the maximum horizontal displacement was 15.874 cm (0.13% of the dam height), and the maximum and minimum principal stresses were 1.76 and 0.63 MPa, respectively. The simulated settlement trends generally agreed with field monitoring data. These results demonstrate that the proposed constitutive model can capture the nonlinear mechanical response of rockfill materials and provide a practical numerical tool for the analysis and design of CFRDs with deep overburden layers.

KEYWORDS: Rockfill material; large-scale triaxial shear experiment; constitutive model; numerical simulation

1 Introduction

With the increasing demand for high dam construction in China, rockfill materials are the core filling materials for earth-rock dams and other projects, and their mechanical properties have a direct bearing on the project stability and safety. In particular, an increasing number of concrete face rockfill dams (CFRDs) have been built on deep overburden foundations in southwest China, where the significant foundation deformation and elevated stress levels further complicate the mechanical response of rockfill materials. Therefore, investigations into the mechanical properties of rockfill materials have long been a key area of focus in geotechnical engineering. The current research on rockfill materials for earth-rock dams primarily involves laboratory tests, optimization of constitutive models, and numerical simulations.

Laboratory tests are the basis for studying the mechanical properties of rockfill materials. Through uniaxial compression, triaxial shear, and other test methods, the basic mechanical parameters of rockfill material can be obtained, revealing the stress-strain mechanism of rockfill material in different stress environments. Altuhafi and Coop [1] explored the effect of initial gradation and density on the mechanical properties of different sand samples using a one-dimensional compression test, where different particle gradations lead to different damage modes, and there exists a final or critical gradation curve, when the particle gradation stabilizes and crushing stops. Salami et al. [2] designed a brand new one-dimensional compression device to study the crushing behavior of particles under different constraints. The experimental results found that the particle crushing pattern is closely related to the constraints, and the crushing strength of the particles increases with the increase of the number of constraints. Zhang et al. [3] carried out one-dimensional tests of sandy mudstone and sandstone with four different initial gradations under different normal stresses, and established a fractal model of particle gradation based on particle gradation curves and fractal theory. One-dimensional tests provide a favorable way to study the basic mechanical behavior of rockfill materials. However, in practical engineering applications, the stress state of rockfill materials is much more complex than one-dimensional conditions. To more realistically simulate the mechanical response of rockfill material under actual working conditions, three-dimensional tests have become an essential research tool. Numerous scholars have conducted in-depth studies on the mechanical properties of rockfill materials in various aspects, including confining pressure [4–8], porosity [9–11], particle gradation [12–16], particle shape [17] and breakage [18–21], and stress path [22–26]. Guo and Chen [27] proposed an energy-based model for particle breakage and gradation evolution during triaxial shearing, demonstrating that the fractal dimension and breakage indices are proportional to breakage energy under different confining pressures and increase with shear strain at a decreasing rate toward stable values. Shahnazari and Rezvani [28] conducted an experimental study on particle breakage in carbonate sands to investigate the effect of factors such as confining pressure, relative density, axial strain, drainage conditions, and particle gradation. Suwal and Kuwano [29] evaluated the Poisson's ratio and small strain stiffness of three grain sizes of materials by applying small strain cyclic loading at multiple stress states through triaxial tests.

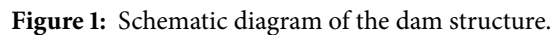
Rockfill material constitutive models are of great significance in describing the mechanical behavior of rockfill material. Up to now, rockfill material constitutive models can be broadly classified into incrementally nonlinear models [30], nonlinear hypoelastic models [9], elasto-plastic models [31] and hypoplastic models [32]. These constitutive models are widely used due to their effectiveness in numerical analysis. However, with the continuous improvement of engineering requirements and the more in-depth exploration of the performance of rockfill materials, their limitations have gradually emerged. Therefore, to capture the mechanical behavior of rockfill material under different stress and strain conditions, many scholars worldwide have conducted in-depth discussions and proposed a series of improved constitutive models [33,34]. Hu et al. [35] proposed a double yield surface elasto-plastic constitutive model considering particle breakage, providing an important reference for simulating the mechanical behavior of granular materials under complex stress paths. Ling and Liu [36] proposed a generalized plasticity model, incorporating the power term of the average effective stress into the loaded plasticity modulus in order to obtain the volume change related to the stress level by combining the stress-expansion relationship. The Duncan-Chang model, as a nonlinear elastic model, has been extensively applied in the numerical analysis of rockfill dams owing to its simplicity and practicality. Despite extensive research on rockfill constitutive models, several critical

limitations remain unresolved. First, the classical Duncan–Chang E-B model, which is most widely used in rockfill dam engineering, adopts the Mohr–Coulomb failure criterion and therefore neglects the influence of the intermediate principal stress. This simplification leads to an underestimation of the strength of rockfill materials under true three-dimensional stress states. Second, the Duncan–Chang model employs a constant failure ratio independent of confining pressure, which cannot capture the transition from particle-sliding-dominated deformation at low confining pressures to particle-crushing-dominated deformation at high confining pressures. Third, existing models rarely account for the coupled evolution of particle breakage, stress dilatancy, and modulus degradation in a unified framework, despite experimental evidence showing that these mechanisms are intrinsically linked. In addition, the Duncan–Chang model does not consider the dilatancy phenomenon under shear stress, so there will be a deviation from the actual value when the numerical analysis of rockfill dam settlement is carried out.

The main theoretical contribution of this study differs from previous work in three aspects. First, the proposed model introduces a confining-pressure-dependent dynamic failure ratio function that captures the transition of failure mechanisms across a wide range of confining pressures. Second, while some previous studies have incorporated either the SMP criterion or particle breakage into rockfill models separately, the present work couples both mechanisms simultaneously with the stress–dilatancy relationship, allowing the model to capture the interactive effects of intermediate principal stress, particle crushing, and volumetric deformation. To this end, based on the face rockfill dam project of Jinchuan hydropower station, southwest China, this paper conducts large-scale triaxial shear tests on rockfill materials under various confining pressures, proposes a nonlinear constitutive model considering dynamic failure ratio, intermediate principal stress and particle breakage, and suggests the calibration method of model parameters. Subsequently, numerical simulations were carried out using the proposed constitutive model, which proved the validity of the constitutive model.

2 Engineering Background

Situated in Jinchuan County, Aba Tibetan and Qiang Autonomous Prefecture, China, Jinchuan hydropower station holds the position as the sixth cascade hydropower project along the main stream of the Dadu River. Jinchuan Hydropower Station is primarily developed for power generation, boasting a total installed capacity of 900 MW. The main structures of the project include a CFRD, a left-bank diversion power generation system, a right-bank spillway, an ecological spillway, and a discharge tunnel. The dam features a crest elevation of 2258 m, a top width of 10 m, and a maximum height of 112 m, as shown in [Fig. 1](#). The toe plate at the base of the dam sits on the overburden layer, with the overburden layer reaching a maximum thickness of approximately 65 m. The deep overburden at the dam site consists primarily of alluvial and glaciofluvial deposits. As CFRDs with the deepest overburden layer in hydropower station currently under construction, the stability and safety of CFRDs are facing serious challenges, so the mechanical properties of rockfill material are important for accurately evaluating and predicting the mechanical behavior of the dams on foundations with deep overburden layer.



3.1 Experimental Plan

The sample material of the experiment is the main rockfill material of Jinchuan concrete face rockfill dam. The sample size is 300 mm × 600 mm, and the maximum particle size is 40 mm. The standard particle size sieve with different apertures is used to divide the test particle size into 10 groups according to 20~40 mm, 10~20 mm, 5~10 mm, 2.5~5 mm, 1.25~2.5 mm, 0.6~1.25 mm, 0.3~0.6 mm, 0.15~0.3 mm, 0.075~0.15 mm and <0.075 mm, as shown in Fig. 3a. According to the similar gradation method, three sets of test gradations that meet the requirements of the particle size of the field rockfill are obtained, as shown in Fig. 3b.

The rockfill material was air-dried and sieved into the designated particle size fractions. Each specimen was prepared in six layers by compaction to a target dry density of 2.18 g/cm^3 , corresponding to an initial void ratio of approximately 0.25. After placement in the triaxial pressure cell, water was pumped into the cell; when the water level reached the specified height, a confining pressure of 30 kPa was applied and the water pump was activated. Saturation was considered complete once the degree of saturation exceeded 95%. The consolidated-drained shear tests were subsequently conducted at a constant axial strain rate of 1.5 mm/min and terminated at an axial strain of 16%, which is sufficient to capture both the peak strength and the critical-state behavior of the rockfill material. Duplicate tests were performed for selected conditions to verify repeatability, and the peak deviatoric stress values from duplicate tests agreed within 10%, confirming the reliability of the experimental results.

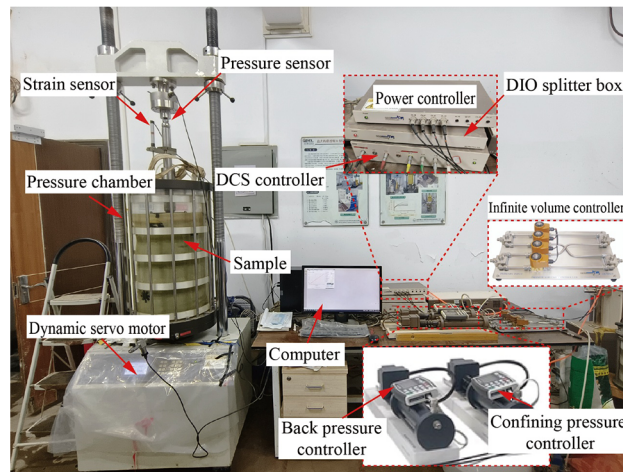


Figure 2: Large-scale triaxial shear test system.

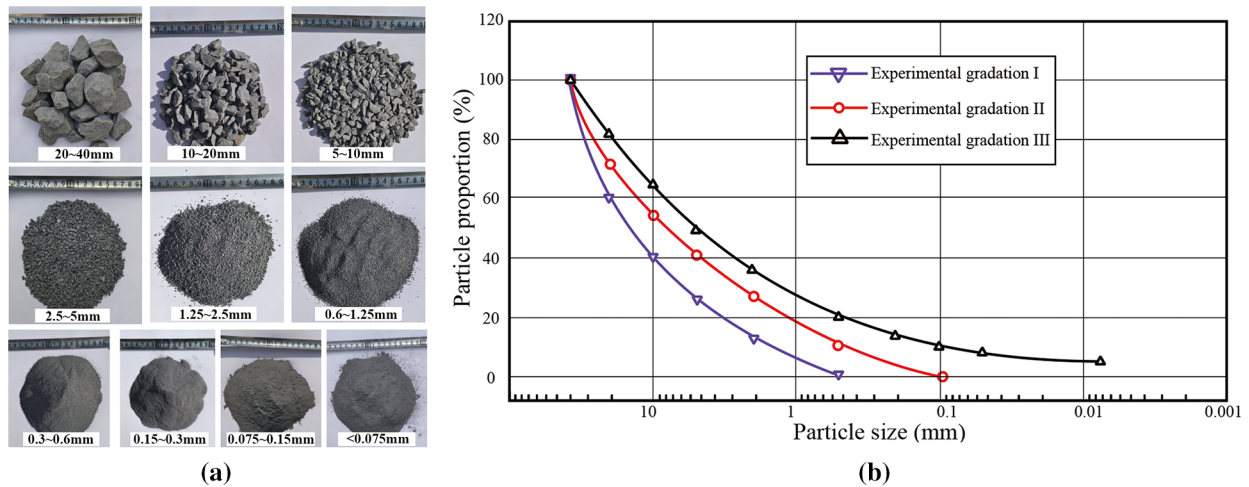


Figure 3: Experimental sample. (a) Different particle size; (b) Experimental gradation.

This experiment aims to reveal the mechanical properties of rockfill materials under different confining pressures. Consolidation and drainage tests were conducted for each gradation shown in Fig. 3b under confining pressures of 200, 400, 600, 800, and 1000 kPa. Table 1 lists the specific experimental plan.

3.2 Mechanical Properties of Rockfill Materials

3.2.1 Stress-Strain Characteristics

The relationship between deviatoric stress and volumetric strain of rockfill with axial strain under different confining pressures is shown in Fig. 4. The deviatoric stress q shows a gradual increase as the axial strain rises, showing typical nonlinear characteristics. In the low strain stage, the deviatoric stress q rises more rapidly. As the axial strain increases to 10%, the growth rate of the deviatoric stress decelerates and gradually approaches a certain asymptotic value, referred to as the progressive deviatoric stress. With the increase of confining pressure, the peak deviatoric stress rises notably. This indicates that a higher confining pressure improves the material's compressive strength and its ability to resist deformation. The confining pressure

increases the contact strength between the particles by limiting the lateral displacement of the particles, thus delaying the occurrence of particle breakage and localized deformation. However, under low confining pressure conditions (200, 400 kPa), strain softening occurs.

Table 1: The specific experimental plan.

No.	Gradation	Confining Pressures/kPa
JC-1-200	Experimental gradation I	200
JC-1-400		400
JC-1-600		600
JC-1-800		800
JC-1-1000		1000
JC-2-200	Experimental gradation II	200
JC-2-400		400
JC-2-600		600
JC-2-800		800
JC-2-1000		1000
JC-3-200	Experimental gradation III	200
JC-3-400		400
JC-3-600		600
JC-3-800		800
JC-3-1000		1000

The curve of volumetric strain vs. axial strain reflects the volumetric change characteristics of rockfill material during the loading process, with positive values in the curve indicating contraction and negative values indicating dilatation. From Fig. 4, it is observable that the volume strain increases rapidly in the low strain stage, and then the slope gradually decreases and even the volume dilatation characteristics appear. Therefore, during the experiment, the rockfill shows the characteristics of shear contraction first and then dilatation. This dilatation characteristic is particularly significant under confining pressures ranging from 200 to 600 kPa.

Fig. 5 shows the comparison of peak deviatoric stress under different gradations and confining pressures. The peak deviatoric stress rises as confining pressure increases, and the curves of different grades are nearly parallel, which indicates that the peak deviatoric stress increases with confining pressure by approximately the same magnitude. For example, when the confining pressure rises from 200 to 1000 kPa, the peak strength of the three graded samples increases by 3.6, 3.4, and 3.2 times, respectively.

3.2.2 Friction Angle

By studying the critical state of the soil, Schofield and Wroth [37] suggested that the variation of the friction angle in triaxial shear experiments can be expressed as:

$$\sin \varphi = \frac{3\eta}{6 + \eta} \quad (1)$$

where φ is the friction angle, and η is the stress ratio.

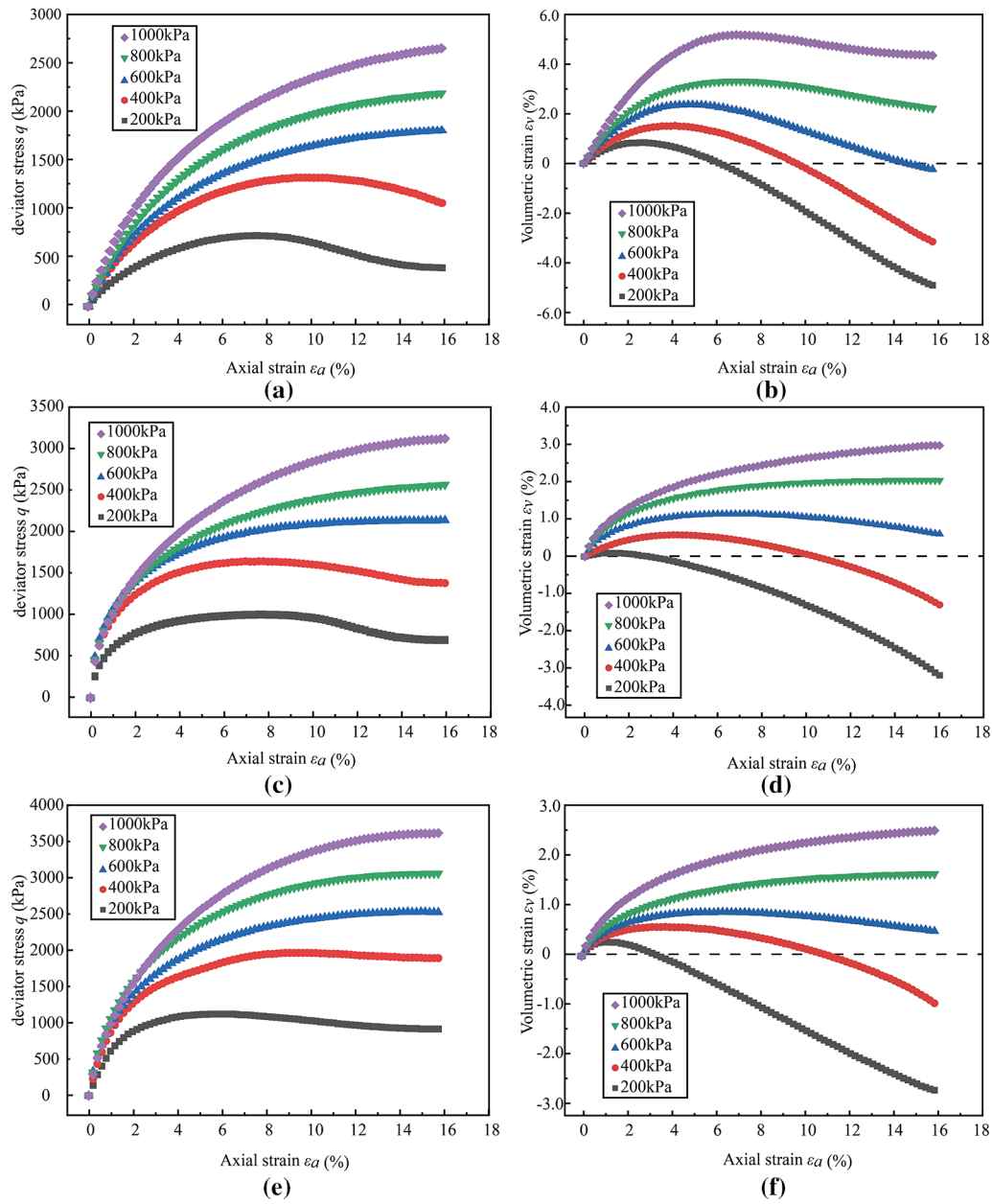


Figure 4: Stress-strain characteristics of rockfill material under different confining pressures. (a, b) Gradation I; (c, d) Gradation II; (e, f) Gradation III.

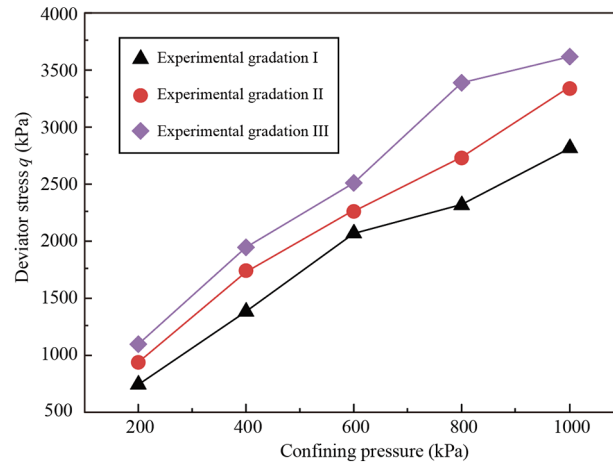


Figure 5: Peak deviatoric stress under different confining pressures and gradations.

Fig. 6 shows the relationship between the friction angle and the strain of the same gradation under various confining pressures. In the stage of small strain, the friction angle increases rapidly, which indicates that in the initial loading stage, the contact points between particles gradually increase, and the friction and occlusion gradually increase. With the gradual increase of strain, the change of friction angle tends to be gentle, and finally reaches a stable value, indicating that the shear strength of the material tends to be stable. This may be due to the fact that the contact points between particles have been fully developed, and the friction and occlusion have reached the maximum value. The friction angle at this time is called the critical state friction angle, corresponding to the steady-state residual strength at large strains with no further volume change. Under the same axial strain, a higher confining pressure corresponds to a lower friction angle. The friction angle increases rapidly as axial strain increases under low confining pressure; while the friction angle increases at a slower rate under high confining pressure. This indicates that under low confining pressure, the friction and interlocking between particles are more likely to increase with deformation. In contrast, under high confining pressure, the contact stress distribution between particles is more uniform, leading to a gentle change in the friction angle. The high and low confining pressures show a differentiation: the friction angle always increases at high confining pressures until it reaches the critical state friction angle, whereas at low confining pressures (200, 400 kPa), the friction angle reaches a maximum—the peak friction angle reflecting the maximum particle interlocking—and then gradually decreases. This is consistent with the law obtained by many scholars after numerous triaxial tests. Fig. 6d presents how the peak friction angle changes with confining pressure. The peak friction angle decreases gradually as confining pressure rises, so confining pressure has a significant effect on the peak friction angle.

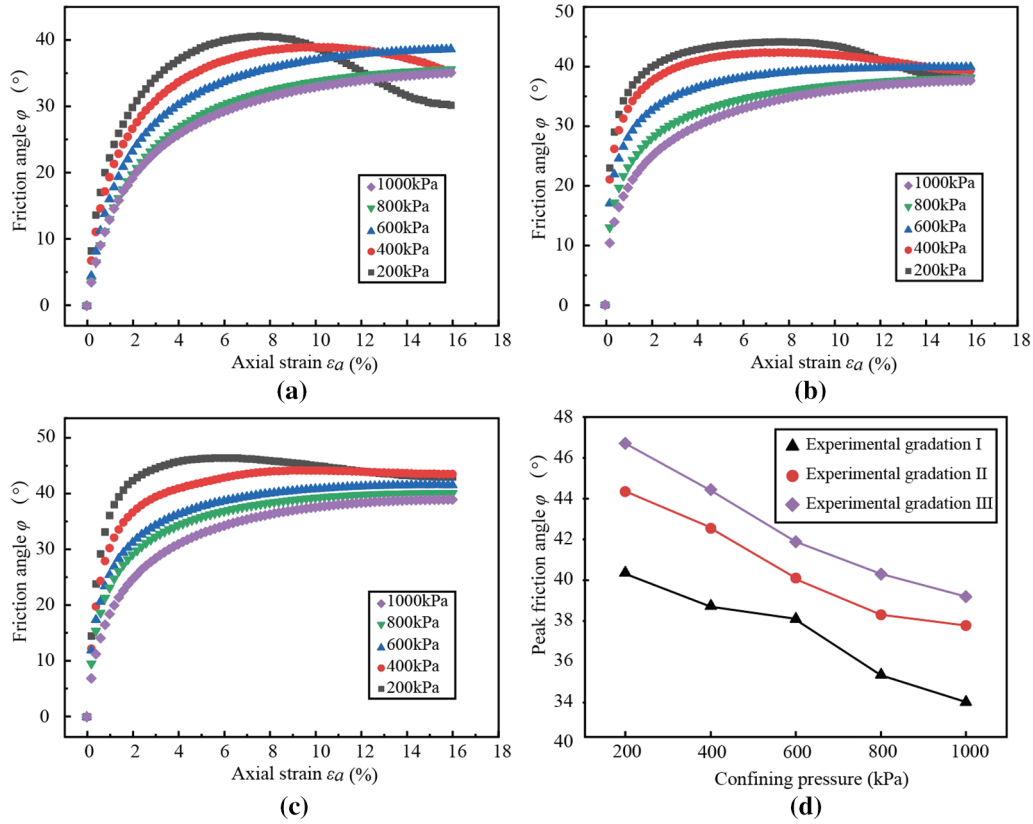


Figure 6: Variation characteristics of friction angle. (a) Experimental gradation I; (b) Experimental gradation II; (c) Experimental gradation III; (d) Peak friction angle.

4 Nonlinear Constitutive Model for Rockfill Materials

4.1 Nonlinear Elasticity Theory

The hypoelastic theory assumes that the general relationship between stress, stress rate, strain, and strain rate is:

$$F_{ij}(\sigma_{kl}, \sigma_{mn}, \varepsilon_{kj}, \varepsilon_{mn}) = 0 \quad (2)$$

Coon and Evans [38] assumed that the relationship between incremental strain and incremental stress is linear, and simplified Eq. (2):

$$d\varepsilon_{kl} = C_{ijkl}(\sigma_{mn}) d\sigma_{kl} \quad (3)$$

C_{ijkl} is a complementary constitutive tensor associated with the stress level.

The expressions of mean effective stress p , deviatoric stress q , volumetric strain ε_v , and deviatoric strain ε_s are:

$$p = \frac{\sigma_{kk}}{3} = \frac{(\sigma_1 + \sigma_2 + \sigma_3)}{3} \quad (4)$$

$$q = \frac{1}{\sqrt{2}} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \quad (5)$$

$$\varepsilon_v = (1 + \varepsilon_{11})(1 + \varepsilon_{22})(1 + \varepsilon_{33}) - 1 \approx \varepsilon_{11} + \varepsilon_{22} + \varepsilon_{33} \quad (6)$$

$$\varepsilon_s = \sqrt{\frac{2}{3} \varepsilon_{ij}^d \varepsilon_{ij}^d}, \varepsilon_{ij}^d = \varepsilon_{ij} - \frac{1}{3} \varepsilon_v \delta_{ij} \quad (7)$$

On the q - p plane, Eq. (2) can be expressed as [39]:

$$d\varepsilon_v = \frac{dp}{K} + \frac{dq}{J_1} \quad (8)$$

$$d\varepsilon_s = \frac{dp}{J_2} + \frac{dq}{G} \quad (9)$$

where K is the bulk modulus, which reflects the volumetric stiffness of the material under dp . J_1 is the shear dilatancy modulus, which is used to explain the volume change caused by dq . G is the shear modulus, which determines the shear deformation of the material under dq . J_2 is the coupling modulus, which describes the shear effect caused by dp . Yin et al. [40] proposed that $J_1 = J_2 = J$, in order to simplify the model. This assumption is also used in this paper, where shear dilatation corresponds to $J < 0$, whereas shear contraction corresponds to $J > 0$.

4.2 Dynamic Failure Ratio Function

In the nonlinear elastic theory, the failure ratio is defined as the ratio of the actual failure deviatoric stress to the theoretical maximum deviatoric stress, that is, the ratio of the deviatoric stress at failure to the progressive deviatoric stress:

$$R_f = \frac{q_f}{q_{ult}} \quad (10)$$

However, in the classical Duncan-Chang model, R_f is a constant independent of confining pressure. This assumption implies that the failure mode of granular materials under various confining pressures is the same as the energy dissipation mechanism. However, the experimental data show that the failure ratios differ under various confining pressures, as shown in Fig. 7.

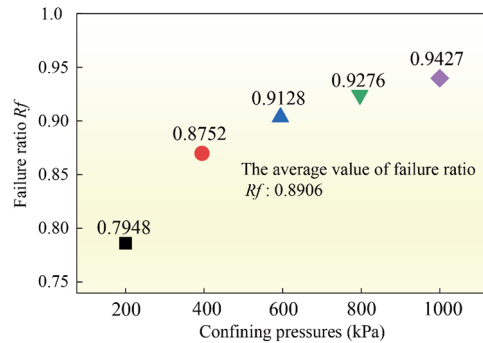


Figure 7: The failure ratio of gradation II with different confining pressures.

Confining pressure significantly affects the failure ratio R_f value. Under low confining pressure, sliding rearrangement is easy to occur between particles, and particle sliding dominates deformation, resulting in a smaller R_f , which is significantly lower than the theoretical value ($R_f < 1$). The strength of the material increases under high confining pressure, approaching the theoretical value, and R_f approaches 1, but the

original model still uses a fixed R_f , resulting in the modulus attenuation term $(1-R_{fs})^2$ being underestimated. Therefore, it is necessary to modify R_f as a continuous function of confining pressure to reflect the regulation of confining pressure on the failure mechanism. According to the experimental results, the mathematical form of R_f and σ_3 should satisfy the following boundary conditions:

- (1) $\sigma_3 \rightarrow 0$: $R_f \rightarrow R_{f0}$. R_{f0} is the failure ratio without confining pressure.
- (2) $\sigma_3 \rightarrow \infty$: $R_f \rightarrow 1$. The material is close to the theoretical strength and inhibits softening.

In order to meet the above boundary requirements, the following function is used to correct the confining pressure dependence of the failure ratio.

$$R_f = R_{f0} + (1 - R_{f0}) \cdot \left[1 - \exp\left(-\gamma \frac{\sigma_3}{P_a}\right) \right] \quad (11)$$

where R_{f0} is the failure ratio reference value when $\sigma_3 \rightarrow 0$, γ is the confining pressure sensitivity coefficient, and P_a is atmospheric pressure ($P_a = 100$ kPa). By calculation, $R_{f0} = 0.761$, $\gamma = 0.1824$, so the dynamic failure ratio function is expressed as:

$$R_f = 0.761 + 0.2387 \cdot \left[1 - \exp\left(-0.1824 \frac{\sigma_3}{P_a}\right) \right] \quad (12)$$

4.3 K, G and J Modulus Functions

The bulk modulus K reflects the relationship between the volumetric strain ε_v and the average effective stress p . Sun et al. [41] argue that the relationship between the volumetric strain ε_v and the average effective stress p is more appropriately expressed by an exponential function:

$$\varepsilon_v = C_t \left[\left(\frac{p}{p_a} \right)^n - \left(\frac{p_0}{p_a} \right)^n \right] \quad (13)$$

where C_t and n are parameters for fitting the experimentally obtained stress-strain data; p_0 is the initial isotropic stress. Differentiating Eq. (13) yields:

$$d\varepsilon_v = C_t n \left(\frac{p}{p_a} \right)^{n-1} \frac{dp}{p_a} \quad (14)$$

According to the definition of the bulk modulus K :

$$K = \frac{dp}{d\varepsilon_v} = \frac{p_a}{C_t n} \left(\frac{p}{p_a} \right)^{1-n} \quad (15)$$

Assuming $K_b = 1/(C_t * n)$ and $n_1 = 1 - n$, the Eq. (15) can be rewritten as:

$$K = K_b P_a \left(\frac{p}{p_a} \right)^{n_1} \quad (16)$$

In shear tests, the deviatoric stress q and shear strain ε_s exhibit a relationship that can be described using the following hyperbolic function:

$$q = \frac{\varepsilon_s}{a + b\varepsilon_s} \quad (17)$$

where a and b are constants determined by fitting the experimental data.

Differentiation of Eq. (17) gives the shear modulus G_{TC} for triaxial conditions:

$$G_{TC} = \frac{dq}{d\varepsilon_s} = \frac{1}{a} [1 - bq]^2 \quad (18)$$

By associating Eq. (18) with Eqs. (8) and (9), the expressions for the shear modulus G and the coupling modulus J can be solved as:

$$G = \frac{KG_{TC}dq^2d\varepsilon_s}{Kdq^2d\varepsilon_s - Kd\varepsilon_v dqdp + G_{TC}dp^2d\varepsilon_s} \quad (19)$$

$$J = \frac{KG_{TC}dq d\varepsilon_s}{Kdq d\varepsilon_v - G_{TC}dp d\varepsilon_s} \quad (20)$$

Let $\Lambda = dq/dp$ and $D = d\varepsilon_v/d\varepsilon_s$, then G and J can be simplified:

$$G = \frac{KG_{TC}\Lambda^2}{K\Lambda^2 - DK\Lambda + G_{TC}} \quad (21)$$

$$J = \frac{K\Lambda G_{TC}}{DK\Lambda - G_{TC}} \quad (22)$$

The initial tangent modulus G_{TC} varies with the effective stress p and is expressed as:

$$G_{TCi} = K_G P_a \left(\frac{p}{P_a} \right)^{n_2} \quad (23)$$

K_G is the material modulus, and n_2 is the power index that determines the variation of G_{TCi} with effective stress p . Therefore, G can be expressed as:

$$G_{TC} = K_G P_a \left(\frac{p}{P_a} \right)^{n_2} \left(1 - R_f \frac{q}{q_f} \right)^2 \quad (24)$$

4.4 Spatially Mobilized Plane Criterion

Matsuoka [42] argues that the SMP (spatially mobilized plane) criterion can correctly explain the strength and damage characteristics of soil:

$$\sigma_{SMP} = \sigma_1 \cdot a_1^2 + \sigma_2 \cdot a_2^2 + \sigma_3 \cdot a_3^2 = \frac{3I_3}{I_2} \quad (25)$$

$$\tau_{SMP} = \sqrt{(\sigma_1 - \sigma_2)^2 \cdot a_1^2 \cdot a_2^2 + (\sigma_2 - \sigma_3)^2 \cdot a_2^2 \cdot a_3^2 + (\sigma_3 - \sigma_1)^2 \cdot a_3^2 \cdot a_1^2} = \frac{\sqrt{I_1 I_2 I_3 - 9I_3^2}}{I_2} \quad (26)$$

$$\frac{\tau_{SMP}}{\sigma_{SMP}} = \sqrt{\frac{I_1 I_2 - 9I_3}{9I_3}} \quad (27)$$

where I_1 , I_2 , and I_3 denote the first, second, and third stress invariants, respectively, and σ_{SMP} and τ_{SMP} represent the shear and normal stresses acting on the SMP.

Yao et al. [43,44] proposed that the SMP criterion can be converted to the Extended Mises criterion by transforming the stress tensor $\tilde{\sigma}_{ij}$.

$$\tilde{\sigma}_{ij} = \frac{pq\delta_{ij} + q_c(\sigma_{ij} - p\delta_{ij})}{q} \quad (28)$$

$$q_c = \frac{2I_1}{3\sqrt{\frac{(I_1I_2 - I_3)}{(I_1I_2 - 9I_3)} - 1}} \quad (29)$$

where δ_{ij} is the Kronecker function, and q_c is the transformed parameter. Therefore, the SMP criterion can be formulated as:

$$\tilde{q}_f = M_f \tilde{p} \quad (30)$$

Embedding the SMP criterion into the shear modulus of Eq. (24), we obtain:

$$G_{TC} = K_G P_a \left(\frac{\tilde{p}}{P_a} \right)^{n_2} \left(1 - R_f \frac{\tilde{q}}{M_f \tilde{p}} \right)^2 \quad (31)$$

Based on the phenomenon of shearing dilatation Liu et al. [45] established the stress dilatancy equation:

$$D = \frac{d\varepsilon_v}{d\varepsilon_s} = \frac{mM^{m+1} - m\eta^{m+1}}{(m+1)\eta^m} \quad (32)$$

$$M = \frac{6\sin\psi}{3 - \sin\psi} \quad (33)$$

where ψ is the phase transformation friction angle, m is the dilatancy coefficient. The dilatancy equation represents the change rate of volume strain with shear strain. Compared with the classical Duncan-Chang EB model, it can describe the volume change characteristics of rockfill material more accurately. In Eq. (32), the phase transformation stress ratio M serves as the critical threshold separating the compressive and dilative regimes. Specifically:

When $0 < \eta < M$, the volumetric strain increment $d\varepsilon_v > 0$, indicating that the rockfill is in the compression stage at low strains, as particles are rearranged into a denser packing.

When $\eta = M$, $d\varepsilon_v = 0$, marking the phase transformation point at which the material transitions from compression to dilatancy.

When $\eta > M$, $d\varepsilon_v < 0$, indicating that after the initial compression at low strains, the rockfill enters the dilatancy stage at higher strains as particles must ride over one another to continue shearing.

This formulation can be regarded as a general form for describing the dilatancy characteristics of granular materials, capable of capturing the complete contraction-to-dilatancy transition process.

4.5 Particle Breakage Evolution Model

Chen et al. [46] summarized a reasonable breakage evolution model through extensive experiments.

$$B = \begin{cases} 0, & p < p_B \\ \frac{p - p_B}{p + (\delta_0 + \mu\eta)p_B}, & p \geq p_B \end{cases} \quad (34)$$

where p_B is the breaking threshold, which is the p corresponding to the beginning of breaking.

The breakage index is defined based on the ultimate GSD (Grain Size Distribution) defined by Einav [47] using fractal distribution, as shown in Fig. 8.

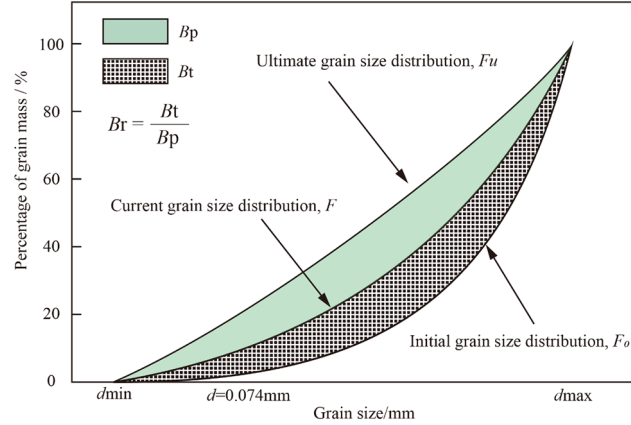


Figure 8: Definition of breakage index.

Fig. 9a–c is the particle gradation curves before and after the experiment. Fig. 9d is the correlation between the breakage index and the deviatoric stress of three particle gradation materials under different stress levels during unloading. It can be seen that the breakage index of Gradation I is relatively high, and the difference between the particle gradation after the experiment and the initial gradation is also relatively large. The breaking threshold p_B can be obtained from Fig. 9d, and breakage-related parameters can also be calibrated by fitting the breakage index with the deviatoric stress and stress ratio.

Zeng and Liu's [48] research shows that particle breakage will lead to a decrease in material strength. Therefore, Eqs. (16) and (31) can be improved to take breakage into account.

$$K_B = K_b P_a \left(\frac{p}{p_a} \right)^{n_1} (1 - \alpha B) \quad (35)$$

$$G_{TC} = K_G P_a \left(\frac{\tilde{p}}{P_a} \right)^{n_2} \left(1 - R_f \frac{\tilde{q}}{M_f \tilde{p}} \right)^2 (1 - \beta B) \quad (36)$$

Yu [49] carried out a set of triaxial shear tests on granular materials, revealing that particle breakage results in a reduction in the friction angle. Therefore, Thus, the shear strength associated with particle breakage can be expressed as:

$$\varphi_B = \varphi - \Delta\varphi_2 B = \varphi_0 - \Delta\varphi_1 \lg \left(\frac{p}{p_a} \right) - \Delta\varphi_2 Br \quad (37)$$

$$\Psi_B = \Psi - \Delta\Psi_2 B = \Psi_0 - \Delta\Psi_1 \lg \left(\frac{p}{p_a} \right) - \Delta\Psi_2 Br \quad (38)$$

The physical mechanism underlying the above breakage-coupled formulation can be interpreted as follows. During triaxial shearing, particle breakage occurs when the contact stress between particles exceeds the particle crushing strength. As breakage progresses, the original coarse particle skeleton is gradually replaced by a finer, more densely packed granular assembly, which leads to three macroscopic consequences. First, particle breakage reduces the interlocking and frictional resistance between particles, resulting in a

decrease in the friction angle, as described by Eqs. (37) and (38). Second, the generation of finer particles fills the inter-particle voids, increasing the material's volumetric compressibility and thereby reducing the bulk modulus K , as reflected in Eq. (35). Third, breakage-induced gradation evolution alters the shear stiffness; the modulus attenuation parameters β in Eq. (36) quantify the rate at which the shear modulus G degrades with increasing breakage index B .

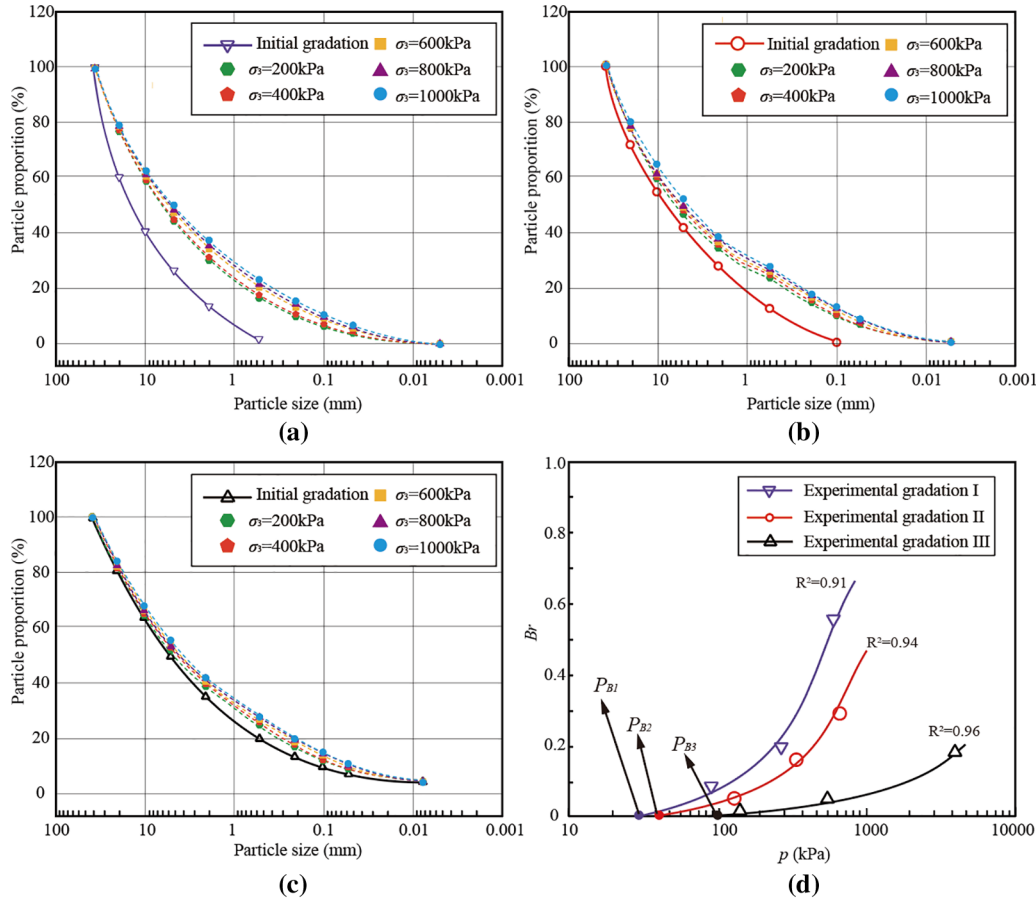


Figure 9: Gradation before and after experiment. (a) Experimental Gradation I; (b) Experimental Gradation II; (c) Experimental Gradation III; (d) Breakage evolution.

4.6 Parameter Calibration

The nonlinear constitutive model contains a total of 18 parameters, among which parameter m is the core parameter describing stress dilatancy behavior. By calculating the stress ratio and stress dilatancy rate under different confining pressures, m is obtained by using the stress dilatancy equation D for fitting. K_b , n_1 , K_G , and n_2 are basic modulus parameters obtained through triaxial shear experiments. R_{f0} , γ is the dynamic failure ratio parameter, which has been calibrated in Section 4.2. P_B , ∂_0 , and μ are breakage-related parameters. The calibration method can be found in the research of Chen et al. [46]. φ_0 , $\Delta\varphi_1$, $\Delta\varphi_2$, Ψ_0 , $\Delta\Psi_1$, $\Delta\Psi_2$ are friction angle-related parameters. The fracture index B and $\lg\sigma_3$ are used as independent variables, and the friction angle is used as the dependent variable for two-variable fitting calibration. α and β are modulus attenuation parameters obtained by fitting the theoretical modulus and breakage index. Parameter calibration results are presented in Table 2.

Table 2: The nonlinear model parameters of Jinchuan CFRD.

Parameter Category	Gradation I	Gradation II	Gradation III
Dynamic failure ratio	$R_{f0} = 0.7095, \gamma = 0.162$	$R_{f0} = 0.78, \gamma = 0.12$	$R_{f0} = 0.82, \gamma = 0.09$
Modulus parameters	$K_b = 196, n_1 = 0.23$ $K_G = 1216, n_2 = 0.45$	$K_b = 353, n_1 = 0.19$ $K_G = 1873, n_2 = 0.35$	$K_b = 324, n_1 = 0.19$ $K_G = 2105, n_2 = 0.34$
Dilatancy equation	$m = 0.84$	$m = 0.77$	$m = 0.81$
Breakage-related	$p_B = 20, \partial\theta = 0.10,$ $\mu = 0.05$	$p_B = 35, \partial\theta = 0.06,$ $\mu = 0.10$	$p_B = 100, \partial\theta = 0.03,$ $\mu = 0.15$
Friction angle-related	$\varphi_0 = 46.8, \Delta\varphi_1 = 3.5, \Delta\varphi_2 = 2.6$ $\Psi_0 = 37.5, \Delta\Psi_1 = 1.8, \Delta\Psi_2 = 1.5$	$\varphi_0 = 44.5, \Delta\varphi_1 = 2.8, \Delta\varphi_2 = 1.8$ $\Psi_0 = 40.2, \Delta\Psi_1 = 1.5, \Delta\Psi_2 = 1.0$	$\varphi_0 = 43.2, \Delta\varphi_1 = 2.0, \Delta\varphi_2 = 0.9$ $\Psi_0 = 42.0, \Delta\Psi_1 = 1.0, \Delta\Psi_2 = 0.5$
Modulus attenuation	$\alpha = 0.25, \beta = 0.28$	$\alpha = 0.18, \beta = 0.22$	$\alpha = 0.12, \beta = 0.15$

4.7 Experimental Verification

Fig. 10 presents a comparison between the predicted results based on the optimized constitutive model and the experimental data for rockfill material under three different gradations. The prediction curve of the improved constitutive model aligns well with the experimental data curve in terms of overall trend, which can better capture the law of deviatoric stress changing with axial strain. The constitutive model can also accurately describe the shearing dilatation phenomenon of volume strain under low confining pressure, indicating that the stress dilatancy equation D is reasonable.

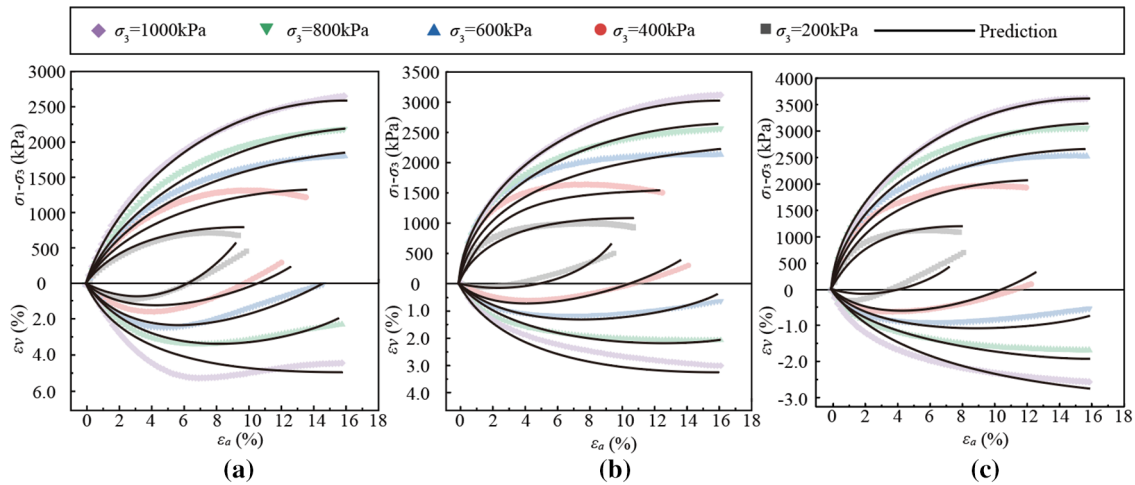


Figure 10: Comparison between experimental and predicted results of triaxial tests on Jinchuan CFRD rockfill materials. (a) Gradation I; (b) Gradation II; (c) Gradation III.

5 Engineering Application

5.1 Models and Parameters

To validate the effectiveness of the optimized constitutive model in engineering applications, numerical simulation was performed using the finite difference method based on the Jinchuan CFRD. Fig. 11 shows the numerical model. The model size is $900 \text{ m} \times 900 \text{ m} \times 700 \text{ m}$, with a total of 1.36 million units and 890,000 nodes. The model considers the layered filling of the dam, the overburden layer, the cutoff wall, and other structures.

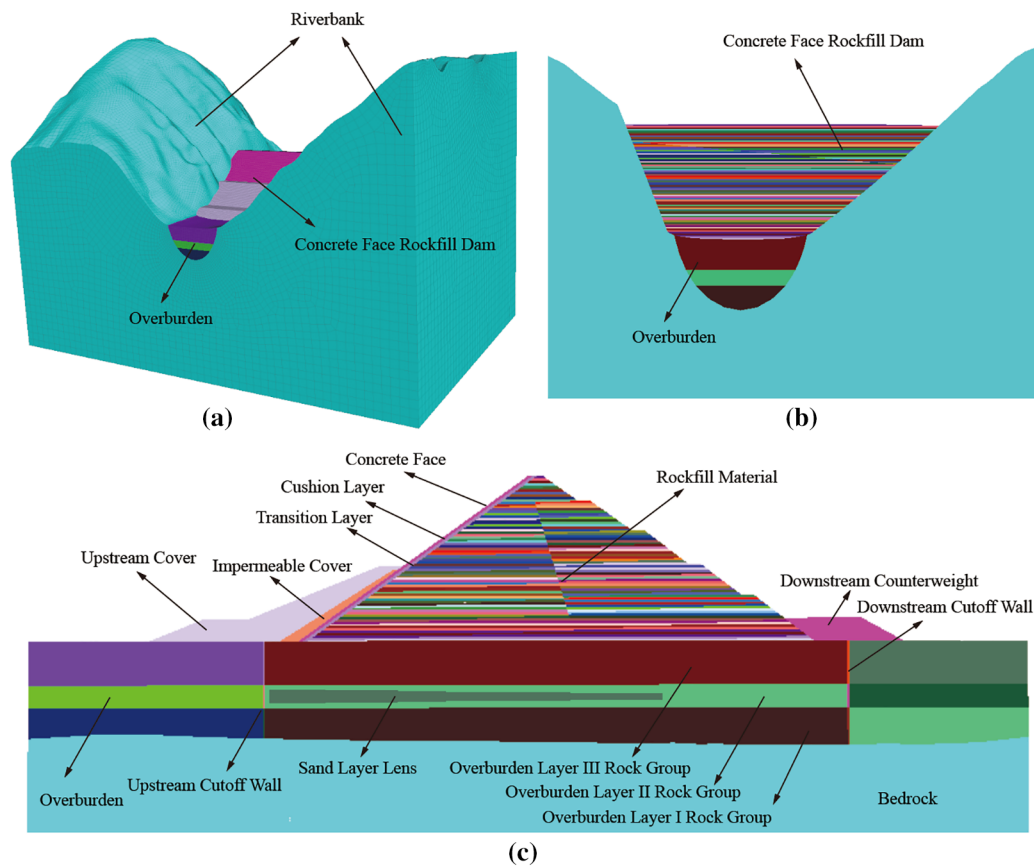


Figure 11: Numerical simulation model. (a) 3D diagram; (b) Front view; (c) Side view.

The constitutive model of the dam rockfill material adopts the optimized constitutive model in the previous section, and the parameters are selected as the parameters based on the gradation II calibration in the triaxial test, as shown in Table 1. The concrete face is modeled using isotropic materials in shell elements, with a thickness of 0.4 m, Young's modulus of 28 GPa, and Poisson's ratio of 0.167. The cutoff wall is modeled using a linear elastic model, with Young's modulus of 25 GPa and Poisson's ratio of 0.17. It should be clarified that the proposed constitutive model is developed specifically for the rockfill material and is applied only to the main rockfill zone, while the remaining material zones (including the cushion layer and the transition layer) and the deep overburden foundation adopt the Duncan–Chang E-B model, with parameters determined from field exploration and laboratory tests and listed in Table 3.

Table 3: Part of the material calculation parameters.

Material	P (g/m ³)	C (kPa)	Φ_0 (°)	$\Delta\Phi$ (°)	K	n	R_f	Kb	m
Cushion Layer	2.26	198	54.1	9.7	776	0.26	0.7	370	0.18
Transition Layer	2.26	237	54.9	8.0	910	0.15	0.73	350	0.01
Group I of Overburden Layer	2.24	153	53.9	9.8	891	0.25	0.65	711	0.12
Group II of Overburden Layer	2.20	174	52.8	8.9	785	0.38	0.61	580	0.34
Group III of Overburden Layer	2.24	199	54.9	9.8	960	0.40	0.66	560	0.29
Sand Layer Lens	1.9	0	38.0	5.9	450	0.28	0.75	250	0.08

The numerical model adopts the following boundary conditions: (1) the bottom boundary is fixed in all three directions; (2) the lateral boundaries are constrained in the normal direction; (3) the top surface and the dam slopes are free boundaries. The construction process of the dam is simulated using the staged construction module, following the actual construction sequence of the Jinchuan CFRD. As indicated by the dam zoning diagram (shown in Fig. 1), the filling sequence is I1 → I2 → III1 → III1 → II2 → III2 → IV, followed by the concrete face slab.

5.2 Stress-Strain Characteristics of Dam

Fig. 12a shows the settlement of the dam when it is filled to the dam crest. The maximum settlement position of the dam is located near 2179 m, and the maximum settlement is 84.8 cm, as indicated in Fig. 12. This position is roughly located at 0.25H of the dam, while the maximum settlement of the dam on the bedrock mostly occurs in the middle (0.4H~0.6H). Due to the compressibility of the overburden layer, the settlement of the overburden layer during the dam filling process is large, which makes the maximum settlement position closer to the dam foundation.

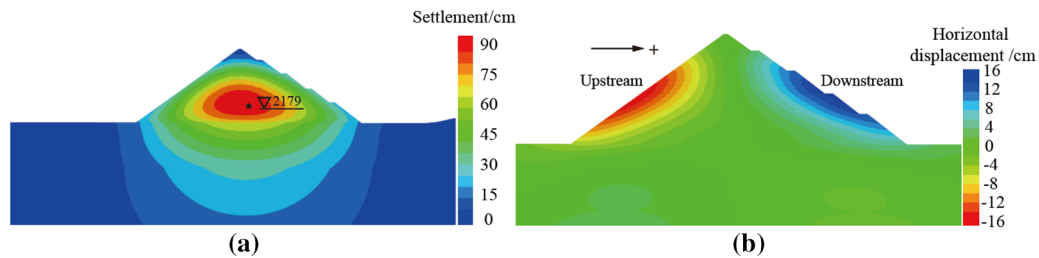


Figure 12: Deformation of the dam body at the maximum section. (a) Settlement; (b) Horizontal displacement.

Fig. 12b shows the horizontal displacement diagram along the river direction when the dam was filled to the dam crest. The horizontal displacement of the dam body is greater near the upstream and downstream dam slopes, while the displacement of the dam body central axis is low, almost 0. The maximum horizontal displacement is 15.874 cm (0.13%H), which is significantly smaller than the dam body settlement.

Fig. 13 shows the dam body settlement at different embankment elevations. As the dam height rises, the settlement at each elevation gradually increases, and the elevation at which the maximum settlement occurs also gradually increases. When the dam is filled to below 30 m, the maximum settlement occurs at the dam foundation, meaning that the deformation of the overburden layer accounts for a significant proportion of the total settlement. As filling continues to 2200 m, the location of the maximum settlement also rises to 2157 m. When the dam is filled to crest, the location of the maximum settlement is at 2179 m (0.25H).

The calculated maximum and minimum principal stresses within the dam body are presented in Fig. 14. The maximum principal stress is 1.76 MPa, and the maximum value of the minimum principal stress is 0.63 MPa.

5.3 Comparison of Numerical Simulation and Monitoring Data

Fig. 15 displays the comparison of settlements between the numerical simulation results and the monitored data during the dam filling process. The general tendency of simulated settlement aligns with the monitored data, and the monitored overburden layer settlement is greater than the calculated value. This may be because the Duncan-Chang EB model parameters used in the overburden layer were obtained from the previous exploration, which underestimates the compressibility of the overburden layer. This suggests that future research should also focus on improving the constitutive modeling of the overburden foundation

material. Nevertheless, the overall settlement pattern and the dam body deformation are well captured, confirming the effectiveness of the proposed rockfill model.

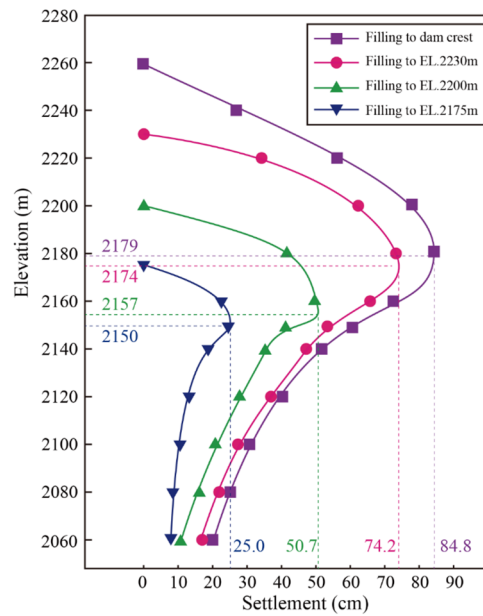


Figure 13: Dam settlement at different elevations.

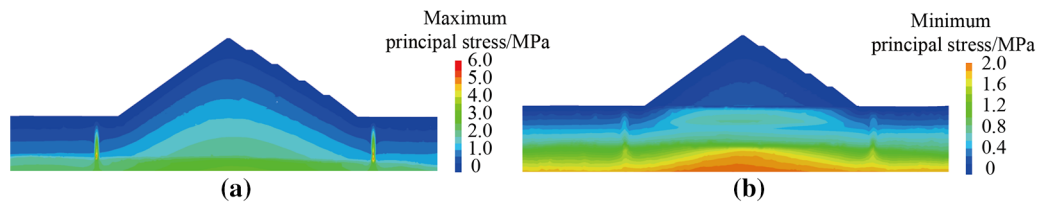


Figure 14: Stress distribution characteristics. (a) Maximum principal stress; (b) Minimum principal stress.

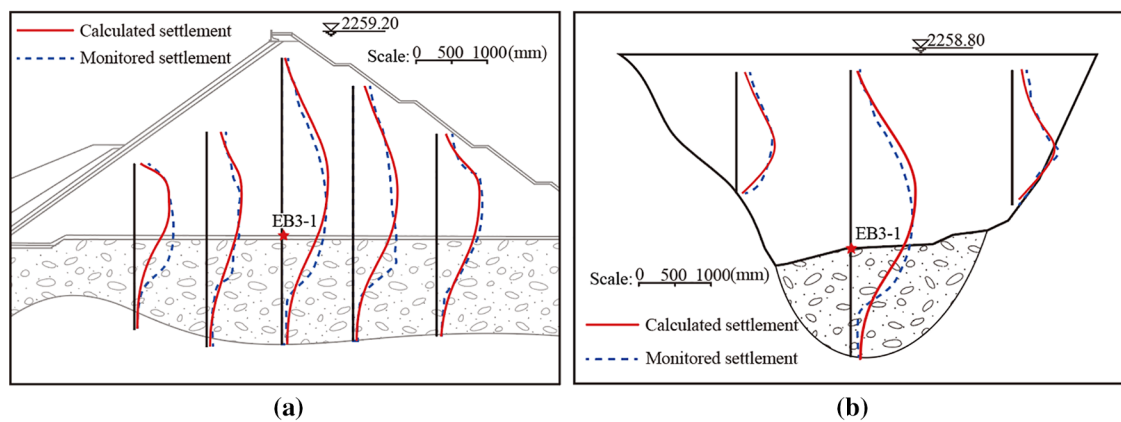


Figure 15: Comparison of numerically calculated dam settlement with measurements. (a) Section of the vertical dam axis; (b) Section along the dam axis.

Fig. 16 compares the simulation results with monitoring results of the measuring point EB3-1. The figure shows that the simulation results are basically consistent with the monitoring results. The settlement speed of the dam is low at the initial stage of filling. With the increase of the filling weight of the dam, the settlement gradually increases, and the settlement decreases again at the end stage of filling. The calculated curve is slightly lower than the monitoring curve, which may be because the fact that the creep of rockfill is not considered in the calculation.

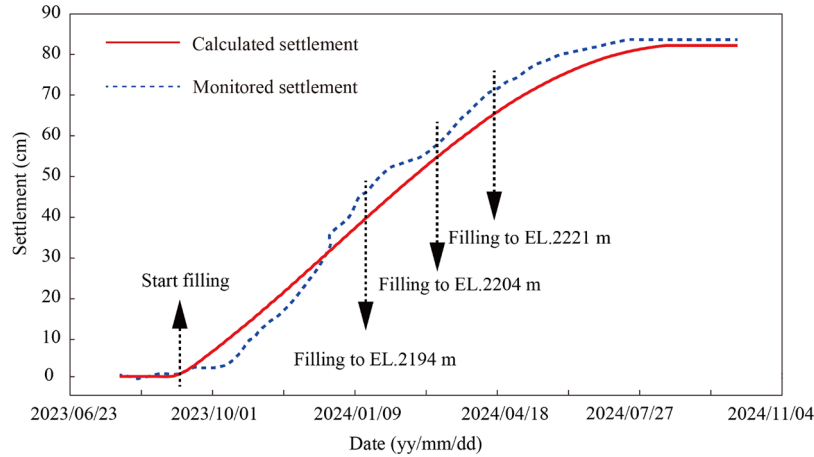


Figure 16: Settlement–time curves at the monitoring point EB3-1.

In summary, the numerical simulation results are basically consistent with the monitoring data in terms of trends, verifying the effectiveness of the optimized constitutive model in engineering applications.

6 Conclusions

To address the limitations of existing constitutive models, this study systematically investigated the mechanical properties and constitutive models of rockfill materials for CFRD with deep overburden layer. The following main conclusions can be drawn.

- (i) Through large-scale triaxial shear tests under different confining pressures, the stress-strain characteristics of rockfill material under different stress conditions were revealed. Under the same particle gradation, the deviatoric stress and peak strength increase as the confining pressure increases. However, the peak friction angle shows a decreasing trend as the confining pressure increases. With the increase of axial strain, it shows the characteristics of shear contraction first and then dilatancy, especially under the confining pressure of 200~600 kPa.
- (ii) A nonlinear constitutive model considering dynamic failure ratio, spatially mobilized plane criterion, and particle breakage was proposed. The constitutive model contains 18 parameters, all of which are obtained through conventional triaxial shear tests. The validity of the optimized nonlinear constitutive model was verified by simulating the triaxial test of rockfill in Jinchuan hydropower station. The model can describe the stress-strain characteristics of rockfill more accurately, such as the dilatancy characteristics that cannot be described by the traditional Duncan-Chang model.
- (iii) This nonlinear constitutive model was applied to the Jinchuan CFRD using the finite difference method. Through numerical simulation, the characteristics of settlement deformation in the process of dam filling were reproduced. The numerical simulation results showed good agreement with the field monitoring results, indicating that the proposed nonlinear constitutive model could correctly

capture the mechanical response during dam filling, verifying the effectiveness of the research results in engineering applications.

Despite the promising results, several limitations of the present study should be acknowledged. First, the model is validated against conventional triaxial shear test data only; its performance under complex stress paths that occur during reservoir impoundment and operation remains to be verified. Second, the model parameters are calibrated for the specific rockfill material of the Jinchuan CFRD, and the transferability of the parameter values to other rockfill materials requires further investigation. Third, the engineering validation is based on a single project; additional independent case studies are needed to confirm the model's generalizability. Future research will focus on: (i) validating the model under complex stress paths; (ii) applying the model to additional deep overburden layer CFRD projects to establish a more comprehensive parameter database; and (iii) improving the constitutive modeling of the overburden foundation material.

Acknowledgement: The authors sincerely thank all non-author contributors who assisted with this study, including those involved in experimental preparation, field data collection, technical discussions, data processing, and manuscript preparation, for their valuable support and contributions.

Funding Statement: This research was funded by the National Natural Science Foundation of China, grant numbers U23A2060 and 42525208.

Author Contributions: Conceptualization: Yuan Tian, Haoyu Mao and Nuwen Xu; methodology: Haoyu Mao and Jianjun Chen; software: Wei Zhao; validation: Yuan Tian, Haoyu Mao and Nuwen Xu; formal analysis: Wei Zhao; investigation: Biao Li; resources: Biao Li; data curation: Haoyu Mao; writing—original draft preparation: Haoyu Mao; writing—review and editing: Nuwen Xu; visualization: Biao Li; supervision: Nuwen Xu; project administration: Yuan Tian and Jianjun Chen; funding acquisition, Nuwen Xu. All authors reviewed and approved the final version of the manuscript.

Availability of Data and Materials: The data that support the findings of this study are available from the corresponding author upon reasonable request.

Ethics Approval: Not applicable.

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

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