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Freeze–Thaw Damage Characteristics of Soil–Rock Mixtures in Open-Pit Coal Mines and Stability Analysis of Slopes in Discharge Sites Based on Partial Flow Code

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Abstract: More than 80% of open-pit coal mines in China are located in northern regions, and the mechanical properties and stability of loose soil–rock mixtures in waste disposal sites are significantly affected by freeze–thaw effects. This article takes the external dumping site of the Baorixile open-pit coal mine in the northern high-altitude region of the Inner Mongolia Autonomous Region as the research object. Through on-site investigation and sampling, indoor triaxial tests (confining pressures of 100 kPa, 200 kPa, and 300 kPa; moisture contents of 18%, 21%, and 24%), numerical simulation, and other methods, the mechanical properties of soil–rock mixtures in the dumping site under different freeze–thaw cycle conditions were tested to reveal the specific influence of the number of freeze–thaw cycles on the mechanical properties of soil–rock mixtures. Using the discrete element software PFC, the microstructural changes in soil–rock mixtures formed by freeze–thaw cycles were studied, and the deformation mechanism and slip mode of loose slopes in waste disposal sites under different freeze–thaw cycle conditions were explored. The relationship between the number of freeze–thaw cycles and slope stability was clarified. The following conclusions can be drawn: the compressive strength of soil–rock mixtures decreases as a quadratic function with increasing freeze–thaw cycles; as the number of freeze–thaw cycles increases, the internal cracks of the soil–rock mixture model increase exponentially; and as the number of freeze–thaw cycles increases, the stability of the slope in the dumping site decreases significantly, and this stability also decreases with an increase in dumping height.

Keywords: open-pit coal mine; dump site; soil–rock mixture; freeze–thaw cycle



Citation: Li, R.; Guo, Y.; Zhou, W.; Lu, X.; Zhang, Z.; Tian, Y.; Qi, X. Freeze–Thaw Damage Characteristics of Soil–Rock Mixtures in Open-Pit Coal Mines and Stability Analysis of Slopes in Discharge Sites Based on Partial Flow Code. *Appl. Sci.* **2024**, *14*, 11585. <https://doi.org/10.3390/app142411585>

Academic Editor: Tiago Miranda

Received: 21 October 2024

Revised: 26 November 2024

Accepted: 3 December 2024

Published: 11 December 2024



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1. Introduction

As a special geological material mixed with soil and rocks, the soil–rock mixture is widely found in mountainous areas, roads, and open-pit mines, and its unique physical and mechanical properties are different from those of soil and rock, with many influencing factors. In order to investigate the physical and mechanical properties of the soil–rock mixture, many scholars have conducted a large number of in situ field tests, indoor tests, and numerical simulations for different properties of the soil–rock mixture that have achieved good results, and the theory of the mechanical properties of soil–rock mixtures has been mastered to a certain extent. For in situ tests, Coli [1] found, through six in situ shear tests, that the strength of a geotechnical body is related to its internal rock content and that the angle of internal friction increases and cohesion decreases as the rock content increases. Xu Wenjie [2], on the right bank of Longbian, investigated the soil–rock mixture by conducting a large-scale horizontal push–shear test and obtained similar results to those obtained from Coli’s study; both found that the stone content has a strong effect on the strength of the soil–rock mixture. They put forward the idea that the study of the strength of soil–rock mixtures should fully consider the effect of particle size composition and the water

content of the soil–rock mixture. Hamidi et al. [3]. investigated the effects of stone content and relative density on the mechanical properties of soil–stone mixtures by means of indoor large-scale straight shear tests and found that the higher the stone content, the stronger the brittleness and softness of the specimens. Salimi et al. [4]. conducted indoor straight shear tests on soil–rock mixtures of different shapes and sizes with different densities and found that the different shapes and sizes of blocks have different effects on the strength of soil–rock mixtures, and the specimens with angular blocks show higher shear strength and shear expansion than the rounded block specimens. Dine [5] carried out a large-scale triaxial test and found that large grain size can improve the strength of soil–rock mixtures and pointed out that factors such as moisture content and perimeter pressure also have an effect on the strength and mechanical properties of soil–rock mixtures. For numerical simulation, Shan et al. [6] used PFC2D to model the soil–rock mixture and performed an experimental simulation using MATLAB, and the results showed that when the rock content reaches 80%, the strength is reduced due to the high content of rock, resulting in larger internal pores, and the structure will be reorganized during the compaction process.

At present, there are many studies on rocks under the action of freeze–thaw cycles, and many scholars have taken granite, mudstone, sandstone, concrete, and other materials under freeze–thaw cycles as the object of research to study the changes in physical and mechanical properties. Tan et al. [7] carried out freeze–thaw cycle tests on granite and found that its compressive strength decreases with the increase in the number of freeze–thaw cycles and increases with the increase in peripheral pressure. Yamabe et al. [8] conducted indoor uniaxial and triaxial tests on frozen sandstone, and the results showed that its compressive and tensile strengths increase with the decrease in freezing temperature. Argandoña et al. [9] used CT scanning technology to study the change process of the pore structure in the freeze–thaw cycle of Spanish dolomite at $-15\sim-10\text{ }^{\circ}\text{C}$ and found that the rock samples began to present with fragmentation after seven cycles of freezing and thawing; furthermore, the formation of penetrating cracks occurred after twelve cycles. Numerous scholars have conducted indoor tests and numerical simulations on the internal structure parameters of ambient soil–stone mixtures, such as stone content, grain size, water content, and block distribution, and in addition to the internal factors, the mechanical properties of soil–stone mixtures are also affected by environmental factors, such as rainfall, groundwater, freezing–thawing cycles, etc. Although research on soil–stone mixtures is constantly expanding in regard to depth, there are not yet many studies on the physical and mechanical properties and deformation and damage characteristics of soil–stone mixtures under the action of freezing and thawing cycles. Zhou et al. [10] modeled room-temperature soil–rock mixtures and frozen soil–rock mixtures, and their comparative tests showed that the shear modulus of soil–rock mixtures gradually increased with the increase in rock content, and the shear modulus of frozen soil–rock mixtures was nearly 3.7 times higher than that of room-temperature soil–rock mixtures. Zhou et al. [11] conducted indoor triaxial tests on soil–rock mixtures with different numbers of freeze–thaw cycles and simulated them using PFC3D, and the results showed that the freeze–thaw cycles weakened the strength of soil–rock mixtures, and their modulus of elasticity and shear strength showed a decreasing trend. Kai et al. [12]. investigated the damage mechanism of soil–rock mixtures with different stone contents under freeze–thaw cycles, observing that the elastic modulus and shear strength of soil–rock mixtures decreased after undergoing freeze–thaw cycles. This was determined through indoor triaxial tests and numerical simulation using particle flow software (PFC5.0), and the extent of the decrease in the elastic modulus and shear strength varied according to the different stone contents; ultimately, the damage of soil–rock mixtures with 55% stone content was the most serious. Harris et al. [13] simulated the deformation process of slopes during the freeze–thaw process using a slope model simulation test and found that the factors affecting the stability of slopes are the number of freeze–thaw cycles, thaw depth, freeze–thaw time, slope gradient, and water content. Griffiths [14] analyzed the stability of slopes under freezing and thawing using the finite element method; Blijenberg et al. [15] used the Monte Carlo calculation method; and

Ugai et al. [16] used the finite element strength discounting method. Park [17] investigated the variation in thermophysical parameters with temperature in granite and sandstone and found that when the temperature is in the range of 40–160 °C, it has little effect on the thermal conductivity of the rock, but the heat capacity and thermal expansion coefficient increase with the increase in temperature.

The slope stability problem is one of the important branches in the field of rock engineering. Scholars both domestic and abroad mainly focus on the slope stability of open-pit mines in terms of the landslide mode and destabilization criterion [18–20], quantification of slope stability, and stability analysis method [21–24], slope stability, and parameter selection under different influencing factors [25,26], slope support and engineering reinforcement measures [27–29], etc. At present, there are extensive technical experience and scientific research results on the stability of conventional rocky slopes, but there is still a lack of research on the stability of slopes in open-pit mines in cold regions. In cold regions, the surface failure and collapse of steps are the common forms of damage to rock slopes in open-pit mines after repeated freezing and thawing processes over a long period of time. In Hokkaido, Japan, shallow damage to rock slopes occurs frequently, with an average depth of 2–3 m [30]. The depth of the frozen layer in open-pit mines in cold regions is affected by factors such as latitude, altitude, and climatic conditions. Natural disasters caused by freezing and thawing often occur in cold-zone mines both domestically and abroad, which seriously threaten the safe production of mines. For example, the slope instability of open-pit mines in the Wardha Valley Coalfield in central India [31], the permafrost ablation landslides in the Caucasus region of Russia [30], the landslides in the British Columbia region of Canada [32], the freezing and thawing landslides in the Punta Thurwieser region of Italy [33], and the deformation and instability of permafrost slopes in the Muli Coal Mine in China [34] are all examples of such natural disasters. Wang et al. [35] conducted an in-depth study of the damage patterns of frozen rock slopes in the Muli Juhuqian mine through on-site investigations and found that the damage patterns of frozen rock slopes are mainly characterized by progressive damage in the upper part of the slopes. Li Guofeng et al. [36] used a simplified algorithm of temperature, seepage, and stress field coupling (THM) with phase change to investigate the local damage evolution of mine slopes under the action of freezing and thawing and discussed the operability of local stability analysis based on yield proximity and based on this, they discussed the influence of different factors on the stability of slopes.

A percentage of 80% of China's open-pit coal mines are located in Inner Mongolia, Xinjiang, and other northern regions [37], where the seasonal temperature difference is large, and the slopes of the water-bearing drainage field will show obvious characteristics of freeze–thaw cycle [38,39]. The freeze–thaw cycle will have a huge impact on the slope stability of the discharge site, and the risk of landslides and slides will increase, seriously threatening the safety and stability of the open-pit mine discharge site [40]. The Baorixile outer soil discharge field is located in the north, composed of a long low temperature and long existence time, which will be damaged and deteriorate to a certain extent after undergoing a freeze–thaw cycle. At present, as a kind of complex bulk material, there have been numerous studies on the differences in its internal structure, but the research on the damage characteristics of soil–stone mixtures after the freeze–thaw cycle and the stability of soil–stone mixture slopes in the external soil discharge site after the freeze–thaw cycle has just begun. Therefore, this paper takes the Baorixile open-pit coal mine as the study background, carries out research on the damage characteristics and slope stability of soil–rock mixture slopes in the external discharge field, and analyzes the stability of soil–rock mixture slopes after undergoing freeze–thaw cycles. This is conducted using indoor experiments on soil–rock mixtures in different freeze–thaw cycles and numerical simulation software (PFC5.0) to explore the influence of freeze–thaw cycles on soil–rock mixture slopes and to provide the best solution for the slope safety of the external discharge field in open-pit coal mines. The slope safety of the external discharge site provides the basis.

2. Preparation

2.1. Specimen Preparation

The soil and rock mixture (SRM) material for this lab test was taken from the external dump site of the Baorixile open-pit coal mine. The Baorixile open-pit coal mine is located in Chenbalhu Banner, Hulunbeier City, Inner Mongolia, close to the northernmost part of China; with its cold climate, the open-pit mine has existed in a sub-zero environment for more than 6 months, and the temperatures in the last three years are shown in Figure 1b, which is composed of a long period of freezing and thawing erosion and has an unfavorable effect on the stability of the slopes. The dataset is provided by the National Cryosphere Desert Data Center (<http://www.ncdc.ac.cn>) (accessed on 1 May 2024) [41]. The main components were sand, clay, and gravel. Based on the material retrieved from the dump site, particle size screening was carried out in the laboratory, and the mass of each particle size and the percentage of mass smaller than a specific size were weighed to obtain the particle size grading diagram of the sampled material.

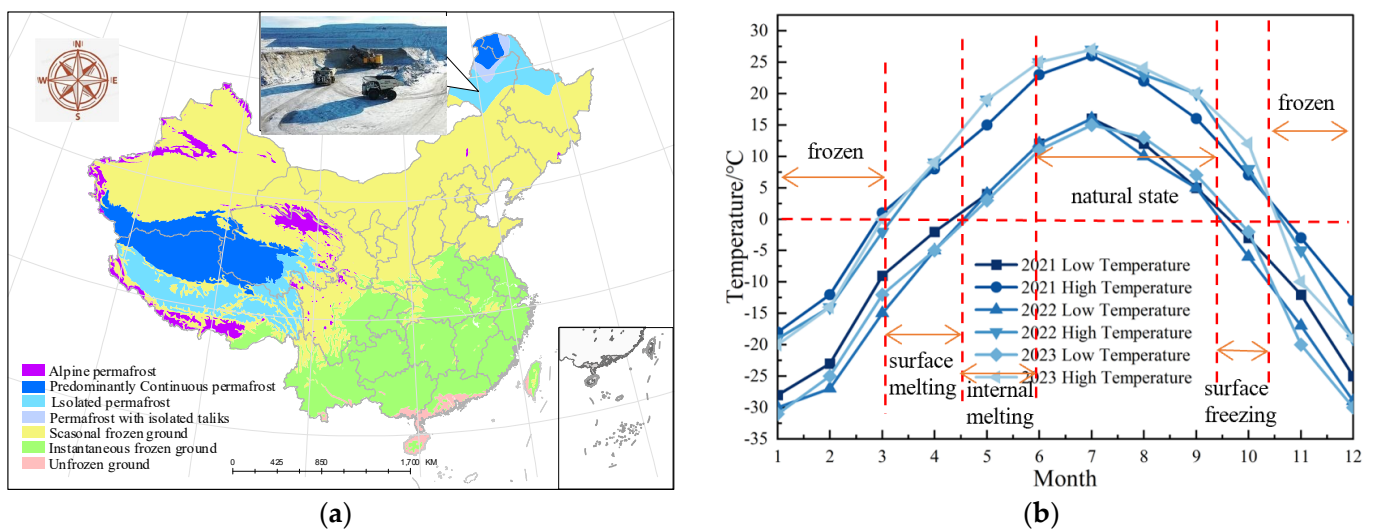


Figure 1. Baorixile open-pit coal mine. (a) Location of the mine site and its zones. (b) Average temperature of the last five years in the mining area.

A similar grading method was used to reduce the particle size of the material, and the test-permitted particle size gradation was obtained for the lab triaxial test. A comparison of the original gradation, lab test, and numerical simulation of the content of particles smaller than a specific size is shown in Figure 2.

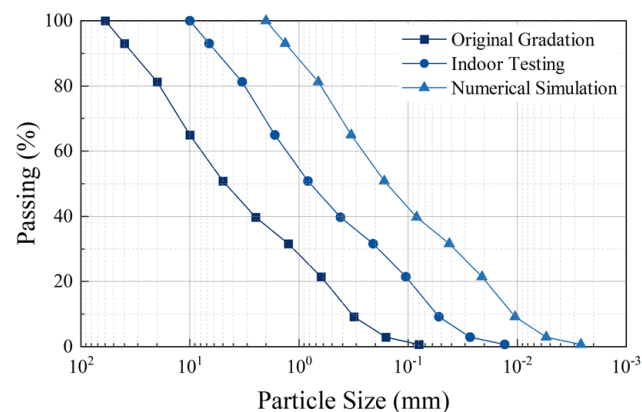


Figure 2. Gradation curves of the SRM materials.

The moisture content of the control samples is the same. According to the preparation conditions of the moisture content group samples, when the moisture content is 18%, the

mixed particles of the material are more discrete. The material mixture is relatively viscous when the moisture content is 24%. Therefore, the moisture content is 21% based on the two conditions. The number of freeze–thaw cycles was 0, 1, 3, 5, and 10, respectively. Triaxial tests were conducted to explore the mechanical properties of the soil–rock mixture. The sample gradation refers to the scaled gradation diagram. For a single sample, 400 g of material is taken, 53.3 g of pure water is taken for the sample with a moisture content of 18%, 73.3 g of pure water is taken for the sample with a moisture content of 21%, and 93.3 g is taken for the sample with a moisture content of 24%. After thoroughly mixing the earth–rock mixture material and purified water, it was put into a sealed bag and left to stand for 24 h so that the material and water were thoroughly mixed. After 24 h, the soaked permeable stone and filter paper were put into the consolidation instrument mold, the mixed material was loaded into the consolidation instrument mold four times, and the mold was placed in the consolidation instrument for graded loading, with each level of load at 100 kPa [42].

After solidifying for 24 h, the sample was removed, wrapped in plastic wrap, and placed in a freeze–thaw box for several freeze–thaw cycles (freeze–thaw cycles of 0, 1, 3, 5, and 10, respectively). The freezing temperature of the freeze–thaw box was set to $-30\text{ }^{\circ}\text{C}$, frozen for 12 h, and then thawed at room temperature for 12 h.

2.2. Test Process

In this experiment, the stress–strain curve of the SRM under triaxial compression after the freeze–thaw cycles was obtained using indoor experiments to study the mechanical properties of the SRM and its damage characteristics under freeze–thaw cycles. The SRM samples prepared above were subjected to groups of triaxial tests. The confining pressures of each group of tests were set to 100 kPa, 200 kPa, and 300 kPa, respectively. The test method employed included the non-consolidation and non-drainage test, and the loading rate was 1.0 mm/min. The end condition was that the strain reached 10%; the data and photos were recorded [42].

2.3. Determination of Macro Parameters

After the SRM sample's consolidation, freezing, and thawing, the unconsolidated, undrained triaxial test sample is shown in Figure 3a. In comparing the samples with different freezing and thawing times when not pressed, the damage caused by the number of freezing and thawing cycles did not appear on the surface of the sample. As shown in the Figure 3a, no major difference was observed in the surface of the samples after 0, 3, and 5 freeze–thaw cycles.

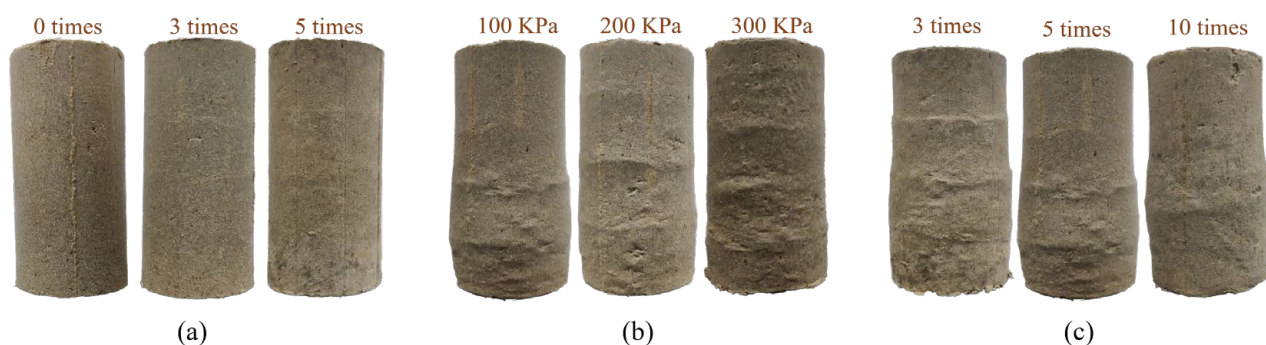


Figure 3. Comparison of samples before and after the lab test. (a) Specimens frozen and thawed at different times. (b) Specimens damaged under different peripheral pressures. (c) Specimens damaged by different numbers of freezing and thawing.

Figure 3b shows the shape of the sample that was frozen and thawed 3 times, 5 times, and 10 times after being destroyed under a confining pressure of 100 kPa. Figure 3c

shows the damage of the specimen under the same circumferential pressure for 3, 5 and 10 freeze-thaw times.

Comparing the three situations, it can be found that the SRM sample shows a bulging failure form under the action of axial pressure. The increase in freeze-thaw cycles does not have a major impact on the failure form of the soil-rock mixture. The increase in lateral confining pressure restrains the specimen's lateral deformation, so the specimen's lateral deformation under 200 kPa and 300 kPa confining pressure is smaller than that of the specimen under 100 kPa confining pressure.

Stress–Strain Curve

- Stress–strain curves of SRM after freeze–thaw cycles under different confining pressures:

The SRM sample with a moisture content of 21% was subjected to 0, 1, 3, 5, and 10 freeze–thaw cycles. A total of 15 samples were subjected to unconsolidated and undrained triaxial tests. The confining pressures were 100 kPa, 200 kPa, and 300 kPa, respectively, and the obtained stress–strain curves are shown in Figure 4.

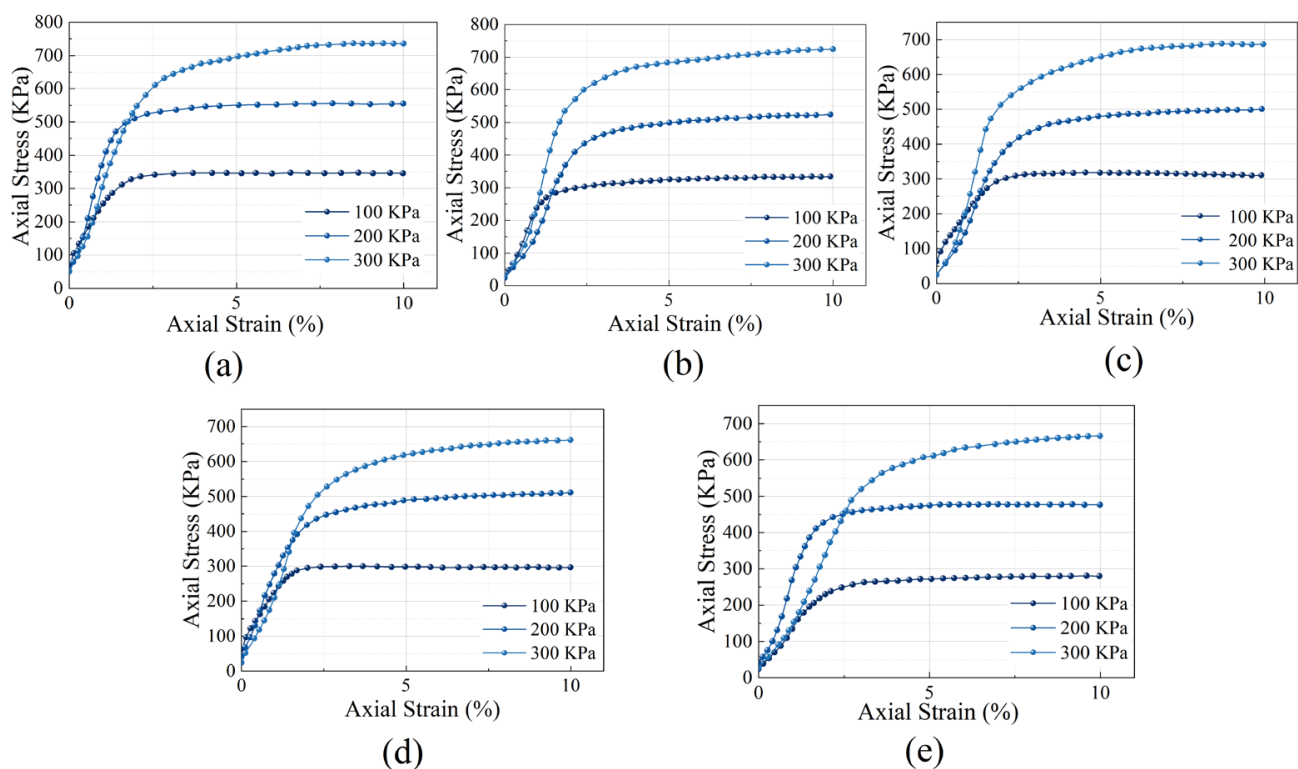


Figure 4. Stress–strain curve of SRM under different confining pressures. (a) At 21% moisture content and 0 freeze–thaw cycles. (b) At 21% moisture content and 1 freeze–thaw cycle. (c) At 21% moisture content and 3 freeze–thaw cycles. (d) At 21% moisture content and 5 freeze–thaw cycles. (e) At 21% moisture content and 10 freeze–thaw cycles.

Figure 4, respectively, lists the stress–strain conditions of the SRM under different confining pressures under the condition of constant moisture content. It can be seen from the above figures, Figure 4a–e, that under confining pressure and different freeze–thaw cycles, the change trends of the stress–strain curves of the SRM are similar, showing the phenomenon of “rapid growth—slow growth—unchanged stress”, which corresponds to the SRM sample test—the process of elastic deformation–hardening yield–sample failure.

At the same time, by comparing the sample curves with different confining pressures, it can be found that the higher the confining pressure, the higher the axial stress the sample can withstand, the higher the strength, and the longer the strain hardening process of the

sample. Under the confining pressure of 300 kPa, when the strain reaches 10%, the axial stress of the sample continues to grow slowly.

- Stress–strain curves of SRM under different freeze–thaw cycles:

It can be seen from Figure 5a–c above that under a confining pressure of 100 kPa, as the freeze–thaw cycle increases, the axial stress that the SRM sample can withstand gradually decreases, while under a confining pressure of 200 kPa, the axial stress difference between the number of freeze–thaw cycles is reduced from 1 to 10. At 5 freeze–thaw cycles, the axial stress exceeds that of the sample with 3 freeze–thaw cycles. The reason may be that it is impossible to accurately control the distribution of soil and rocks in each sample during the sample preparation process, and this reason also exists in studies by other researchers. Under a confining pressure of 300 kPa, the difference in peak stress is more minor and even occurs when the peak stress of 10 freeze–thaw cycles exceeds 5 freeze–thaw cycles. The reason is that in addition to the slight gap between the samples, the freezing and thawing process and the effect of confining pressure will change the internal porosity and cementation degree of the soil–rock mixture, which will have a particular impact on the strength of the SRM sample. Under the action of high confining pressure, the effect of transverse confining pressure compensates to a certain extent for the damage experienced by the specimen during the freeze–thaw cycle. As a result, the peak stress gap under 300 kPa confining pressure is smaller than that under 100 kPa.

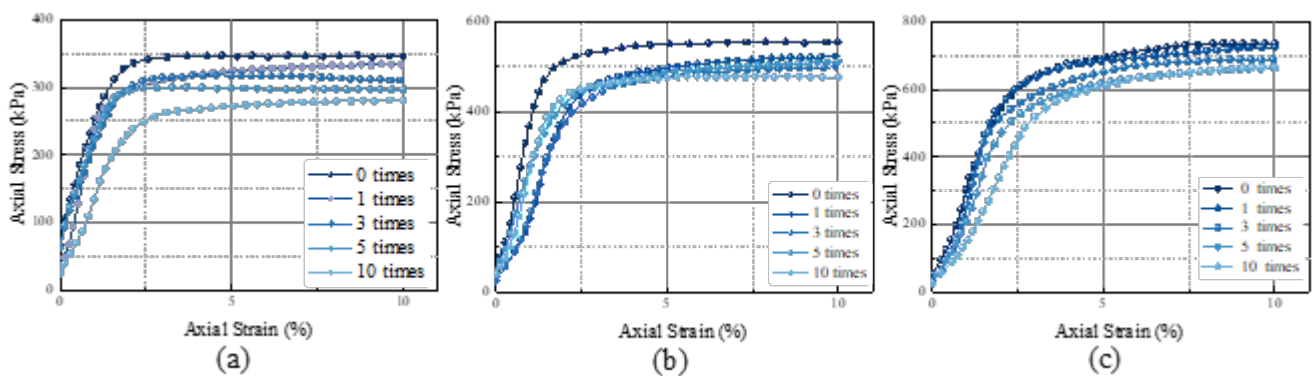


Figure 5. Stress–strain curve of SRM under different freeze–thaw cycles: (a) 100 kPa perimeter pressure; (b) 200 kPa perimeter pressure; (c) 300 kPa perimeter pressure.

- The effect of the number of freeze–thaw cycles on the peak stress of soil–rock mixture:

Based on the data obtained from the triaxial test of the soil–stone mixture specimen subjected to freeze–thaw cycles as well as the stress–strain curve, the triaxial compressive strength of the specimen at 10% strain can be determined, and the data corresponding to each level of peripheral pressure and the number of freeze–thaw cycles are shown in Table 1.

Table 1. Compressive strength of soil–rock mixtures under different numbers of freeze–thaw cycles.

Number of Freeze–Thaw Cycles (Times)	Compressive Strength (kPa)		
	100 kPa Perimeter Pressure	200 kPa Perimeter Pressure	300 kPa Perimeter Pressure
0	345.65	554.63	735.60
1	325.59	523.50	724.18
3	310.24	500.49	687.45
5	296.75	511.27	661.69
10	280.19	476.62	666.16

It can be seen from the curve that when the number of freeze–thaw cycles increases from 0 to 10, the peak stress decreases by 65 kPa at a confining pressure of 100 kPa. The peak

stress of the sample decreases as the number of freeze–thaw cycles increases, indicating that the freeze–thaw cycle has a specific damage effect on the soil–rock mixture. The more cycles, the higher the degree of damage. This is because the pore water of the SRM sample condenses into ice during the freezing process, and the volume expands, generating internal stress inside the sample. As a result, the compactness of the sample is reduced, and the pores are enlarged. After melting, the damage caused cannot be recovered, resulting in a reduction in strength; under a confining pressure of 200 kPa, the peak stress is reduced by 78 kPa, and the peak stress appears reversed after freezing and thawing 5 times. This condition continues to decrease after 10 freeze–thaw cycle times; under a confining pressure of 300 kPa, the peak stress decreases by 44 kPa, and the peak stress appears reversed after 10 freeze–thaw cycletimes On one hand, the soil and rock content and distribution during the preparation of the SRM affect its mechanical properties. On the other hand, in the case of high confining pressure, the effect of confining pressure will compensate for the damage caused by freezing and thawing to a certain extent. It can be seen from Figure 6b that under different freeze–thaw cycles, the peak stress of the SRM increases linearly with the increase in confining pressure. The slopes of the curves for different freeze–thaw cycles are almost the same, indicating that the freeze–thaw cycle does not affect the sensitivity of the SRM samples to confining pressure.

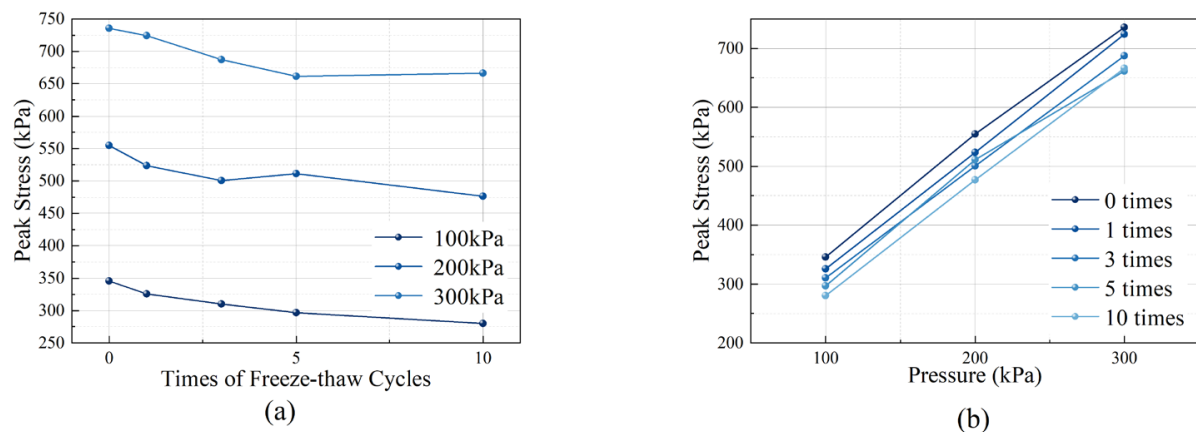


Figure 6. Relationship between compressive strength, number of freeze–thaw cycles, and confining pressure.

3. Numerical Simulation

3.1. PFC

Particle Flow Code, referred to as PFC, integrates two-dimensional (PFC2D) and three-dimensional (PFC3D) characteristics. This software is based on discrete unit codes used to simulate the movement and interaction between particles to study the mechanical properties of materials. Among them, particles are rigid bodies with mass. They interact through paired contact forces and internal inertial forces and moments. The contact forces interact by updating the internal forces and moments. The PFC model consists of a body, apiece, and a contact. The basic building blocks of particles include balls and pebbles. The bonding of multiple balls can simulate breakage when the bond breaks. One or more clusters can simulate an indestructible rigid cluster of any shape. In addition, the wall representing the boundary is also defined, as well as the contact between particles and between particles and walls. The motion of balls and clusters obeys Newton’s laws of motion [43].

3.2. Simulation Methods

3.2.1. Rock Damage Mechanism

The theory of volume expansion is that during the freezing process of rock and soil, the internal water will freeze into ice in a low-temperature environment, and the volume

will increase significantly. The volume expansion caused by the water and ice phase change in the rock and soil will significantly impact the surrounding rock and soil. Large stress will cause the cracks in the rock and soil to expand further, causing damage to the rock and soil structure. As the freeze–thaw cycle proceeds, this damage will continue to accumulate, causing the structure of the rock and soil to gradually deteriorate. However, not all freeze–thaw damage can be attributed to volume expansion. Freeze–thaw damage is a complex process involving the interaction of multiple factors, including water migration.

During the freezing process, water migrates from high-potential energy areas to low-potential energy areas. Especially when the soil freezes, unfrozen water will transfer to the frozen surface. This phenomenon is called normal frozen soil moisture migration. This kind of moisture migration has an important impact on freeze–thaw damage. The migrated moisture forms ice crystals during the freezing process, causing volume expansion and exerting pressure on the rock and soil structure, leading to internal cracking and fragmentation. During the melting process, melting ice crystals may lead to instability and the settlement of geotechnical structures.

The SRM results from the re-consolidation and settlement of bulk rock and soil. Its internal structure is complex and changeable. In the water-containing soil–rock mixture, the volume expansion and migration of water particles during the freezing and thawing process is one of the main causes of freeze–thaw damage.

3.2.2. Parameter Calibration

For different numbers of freeze–thaw cycles, the parameters of the contact model for the soil–stone mixture specimen are calibrated when it has not undergone freezing and thawing. This is because the contact model between the soil and stone is based on a parallel adhesion model, so the parameters requiring calibration include the effective modulus (emod), effective modulus of adhesion (pb_emod), tensile strength (pb_ten), cohesive force (pb_coh), and coefficient of friction (fric). For the stress–strain curve obtained from the indoor test, there are three main stages: firstly the linear elasticity stage, where the stress grows rapidly; then, the yield stage, where the strain hardens; and finally, the destructive stage, where the stress is basically unchanged. This corresponds to parameter calibration of the numerical simulation divided into the elasticity modulus and the compressive strength stages.

The modulus of elasticity reflects the elastic properties and deformation capacity of a material when subjected to a force, which is expressed as the slope of the curve in the stress–strain curve. In the parallel bonding model, the slope of the curve can be adjusted by changing the emod and the pb_emod during the operation; in order to facilitate the calibration, the values of the emod and the pb_emod are controlled to be the same, and the value of the emod is adjusted continuously. It can be observed that there is a linear relationship with the elasticity modulus, and the elasticity modulus of the material is the same as that of the pb_emod during the operation, as shown in Figure 7a.

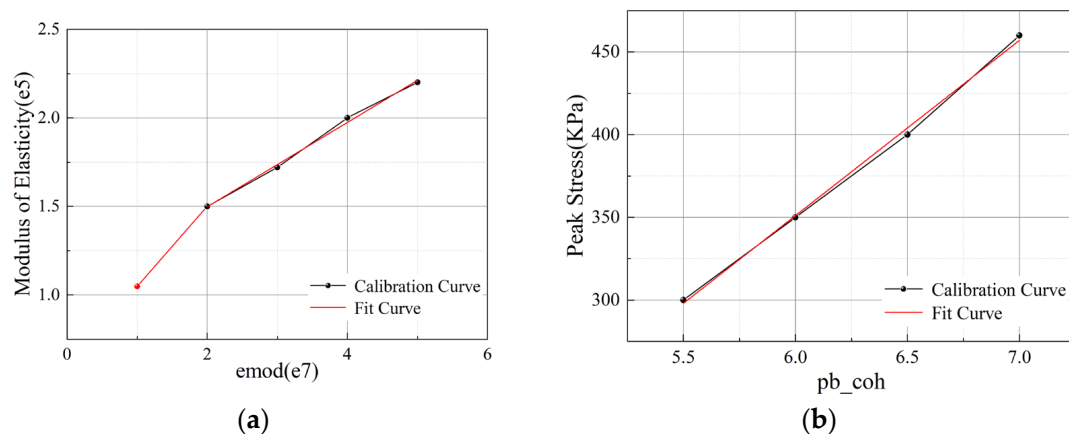


Figure 7. (a) Modulus of elasticity fitting curve; (b) compressive strength fitting curve.

Once the slope of the curve is determined and the compressive strength is calibrated, the ratio of pb_ten/pb_coh is determined to be constant, which is shown in Table 2. The values of pb_coh and pb_ten are adjusted so that the numerical simulation is as close as possible to the compressive strength of the indoor test, and this process is shown in Figure 7b.

Table 2. Calibration results of the parallel bonding model parameters.

Parametric	Value	Parametric	Value
emod	40 MPa	pb_ten	300 kPa
kratio	5.0	pb_coh	60 kPa
pb_emod	40 MPa	fric	0.8

3.2.3. Thermal Coupling Algorithm

Thermal coupling refers to a series of changes caused by the interaction of two physical fields: the temperature field and the stress field. In the numerical simulation, the damage caused by the freeze–thaw cycle is simulated using the discrete element thermal coupling method for subsequent research on the slope stability of dump sites.

In thermodynamic calculations, the heat conduction equation for a continuum medium is as follows (assuming that strain changes are negligible in thermal calculations) [43]:

$$-\frac{\partial q_i}{\partial x_i} + q_v = \rho C_v \frac{\partial \kappa}{\partial t} \quad (1)$$

$$q_i = -k_{ij} \frac{\partial \kappa}{\partial x_j} \quad (2)$$

In this formula, q_i is the heat flux; q_v is the volume heat intensity; ρ is the density; C_v is the specific heat; T is the temperature; and k_{ij} is thermal conductivity.

In the particle flow program, the model is composed of many particles, and the continuum heat conduction equation changes to the heat conduction between two particles, and its equation is as follows:

$$-\sum_{p=1}^N Q_p + Q_v = m C_v \frac{\partial \kappa}{\partial t} \quad (3)$$

$$Q = -\frac{\Delta \kappa}{\eta L} \quad (4)$$

N is the number of contacts between two particles; Q_p is the heat energy transferred in the p -th contact; Q_v is the heat source intensity; m is the mass; C_v is the specific heat; ΔT is the temperature difference between the two particles; retains the heat value per unit length of contact; and L is the contact length.

The indoor test results and the parameter calibration process lead to an exploration of how damage changes with the number of freeze–thaw cycles T . The changes in macroscopic strength and elastic modulus with the number of freeze–thaw cycles is shown in Figure 8.

Based on the pre-pressed and cemented model, the thermodynamic calculation module is opened. The initial default temperature is 0 °C to simulate the freezing and thawing process of the lab test. For the SRM model, the freezing temperature of the surrounding particles is −30 °C, and the melting temperature is 25 °C. The process of heat conduction in the freeze–thaw cycle of the SRM model is shown in Figure 9.

Based on the lab test results and parameter calibration process, the changing pattern of damage with the number of freeze–thaw cycles T is explored. The changes in the compressive strength and elastic modulus with the number of freeze–thaw cycles are shown in Figure 4.

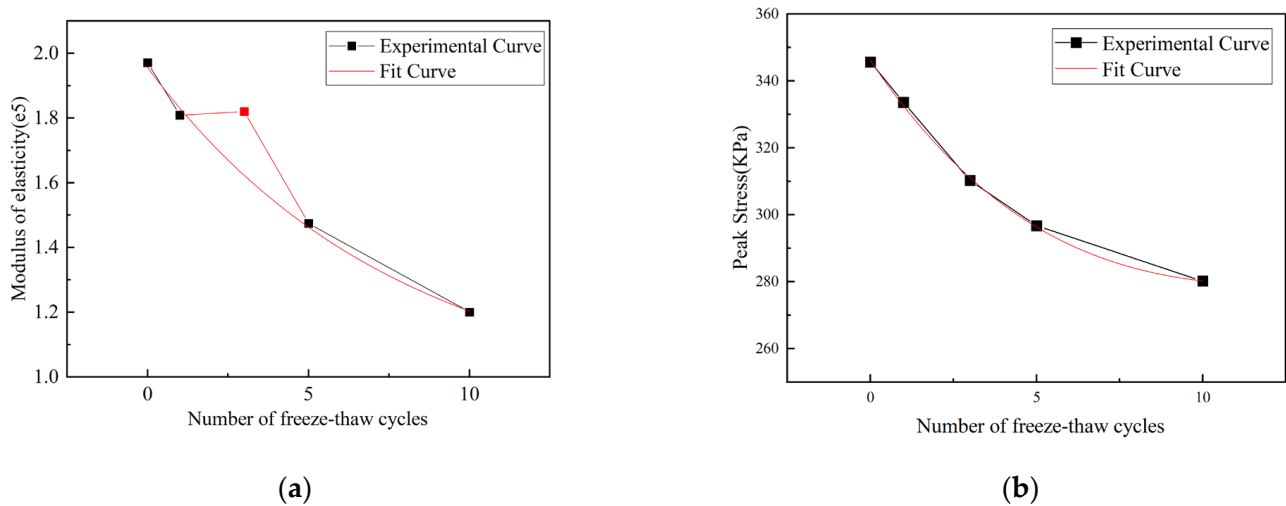


Figure 8. (a) Relationship between modulus of elasticity and number of freeze–thaw cycles. (b) Relationship between compressive strength and number of freeze–thaw cycles.

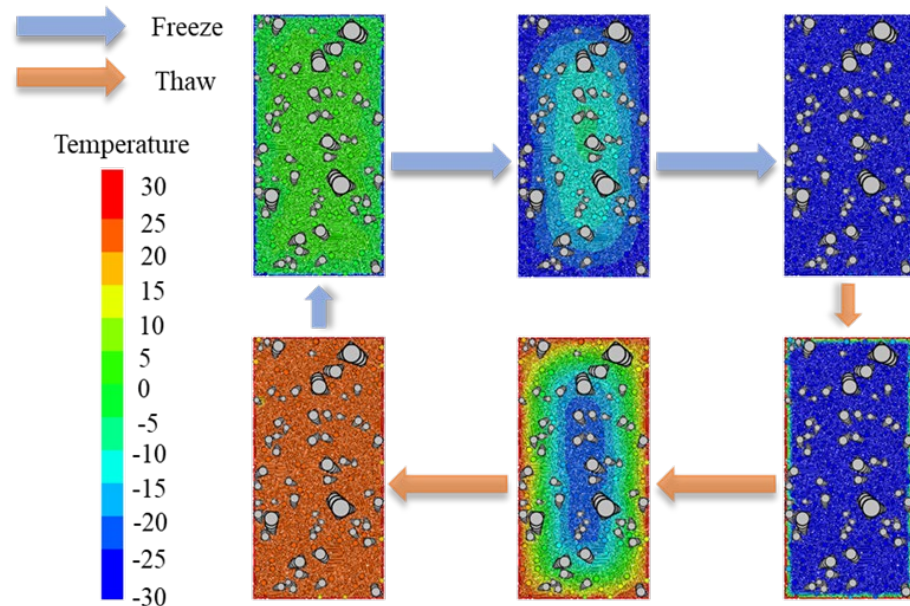


Figure 9. Freeze–thaw cycle process of SRM model.

According to the parameter calibration results, the peak stress has a linear relationship with the cohesion force pb_coh , and the elastic modulus has a linear relationship with the effective modulus mod . Therefore, as a result of the integrated trial calculation analysis, the effective modulus damage $D_e(T)$ and cohesion damage $D_c(T)$ can be obtained. The evolution function with the number of freeze–thaw cycles T is as follows:

$$D_e(T) = 1 - \frac{4.37^{-\frac{T}{7.76}} - 0.441}{3.928} \quad (5)$$

$$D_c(T) = 1 - \frac{2.95 - 0.14T + 0.01T^2 - 0.0002T^3}{2.95} \quad (6)$$

By introducing the above formula into the model, we can obtain the changes in the physical and mechanical properties of the SRM with the number of freeze–thaw cycles during the entire thermodynamic calculation process and examine its microscopic changes.

3.3. Obtaining Mesoscopic Parameters

3.3.1. Distribution of Damage Forms

When simulating the specimen loading process, the contact bonds between the SRM specimens will break. The damage evolution process during the specimen loading process can be presented from the contact perspective.

Figure 10 shows the contact state of the SRM sample during the loading process, with strain ranging from 0% to 10% over three freeze–thaw cycles. The development process of damage cracks and shapes can be seen. In the particle flow program, the contact state `pb_state` of the parallel bonding model has four values. When the value is 3, it is bonded, and the other values are not bonded or broken or sheared (can be represented in the diagram). Before the simulation starts, the contact state of the sample is red and cemented. When the axial strain reaches 2%, the inter-particle cementation begins to be destroyed, and blue particles begin to appear in the sample, indicating that some of the inter-particle cementation is broken. At this time, it corresponds to the linear elastic growth of stress. When the axial strain reaches 5%, the contact damage between particles further increases, cracks gradually develop and form, and “X”-type shear failure begins. At the same time, the specimen begins to fail laterally, but due to the constraints of lateral stress, the middle and lower parts of the specimen begin to bulge outward. After the axial strain reaches 10%, the simulation stops, and the shear cracks are completely formed, extending from the upper left corner to the lower right corner of the sample and from the middle right corner to the lower left corner; the two cracks stagger to form an X-shaped fissure that covers the entire sample, and large lateral deformation occurs on both sides of the model, as shown in Figure 11 compared with the lab test.

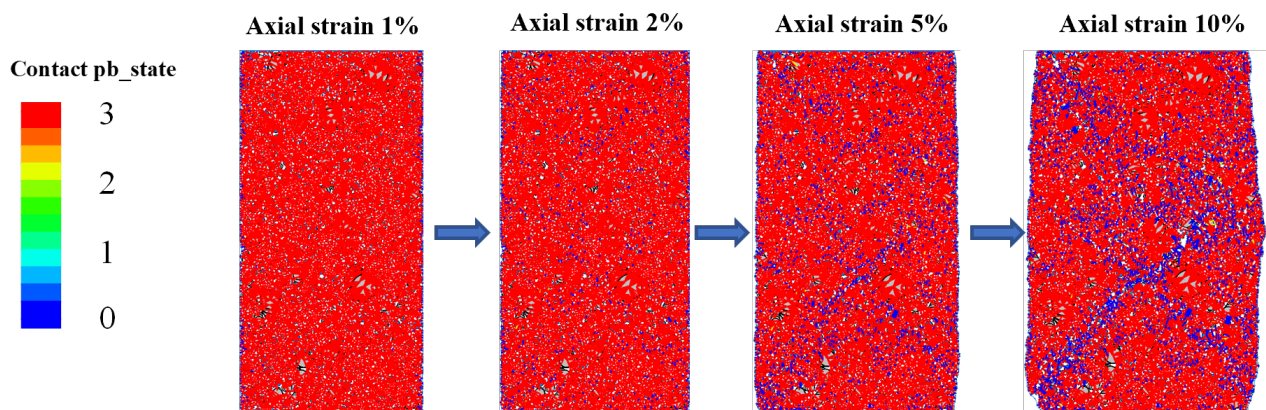


Figure 10. Sample contact damage process.

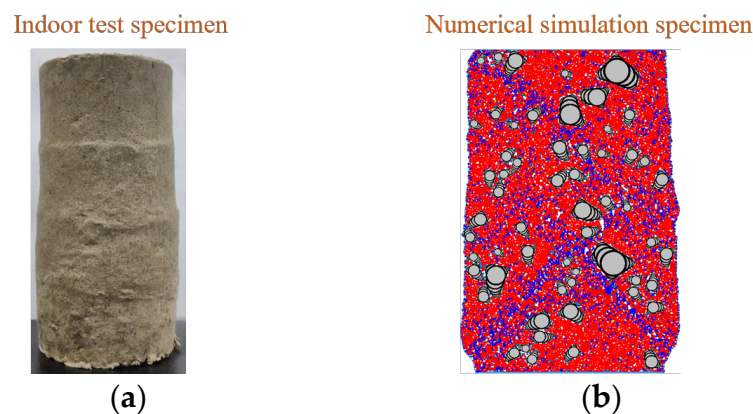


Figure 11. Comparison of sample damage between lab tests and numerical simulations. (a) Indoor test specimen. (b) Numerical simulation specimen.

Comparing Figure 11a with Figure 11b, it can be seen that the results of the numerical simulation are similar to the lab test. After loading, the specimen state is a shear failure and exhibits a certain lateral bulging deformation. As can be seen from Figure 11b, the failure cracks will bypass the rocks during the development process, create cracks between relatively weak soil bodies, and squeeze out the rocks. At the same time, areas with denser rocks will form a certain skeleton structure, making this area more fragile. Firms are concentrated so that fissures develop between regions.

3.3.2. Damage Analysis of Different Freezing and Thawing times

- Crack damage during the freezing and thawing process:

During the freezing and thawing process of SRM samples, the volume change caused by the water ice phase change is the main cause of irreversible freeze–thaw damage. Only its macroscopic strength and failure mode can be studied in lab tests. Further microscopic analysis of freeze–thaw fissure damage was performed via the particle flow program.

Figure 12a demonstrates the number of fissure developments under different freeze–thaw cycles. As the number of freeze–thaw cycles increases, the fissures generated by contact fractures between particles in the SRM also increase. At the same time, the damaging impact generated by the freeze–thaw cycle and the increase in the number of fissures will be reduced. This indicates that while the damage caused by the freeze–thaw cycle on the SRM will reach its peak value in a certain number of cycles, the subsequent increase in the number of freeze–thaw cycles will gradually decrease the impact on the soil–rock mixture. This indicates that the damage of the freeze–thaw cycle on the SRM will reach a peak at a certain number of cycles, and the subsequent increase in the time of freeze–thaw cycles on the SRM will gradually reduce the impact.

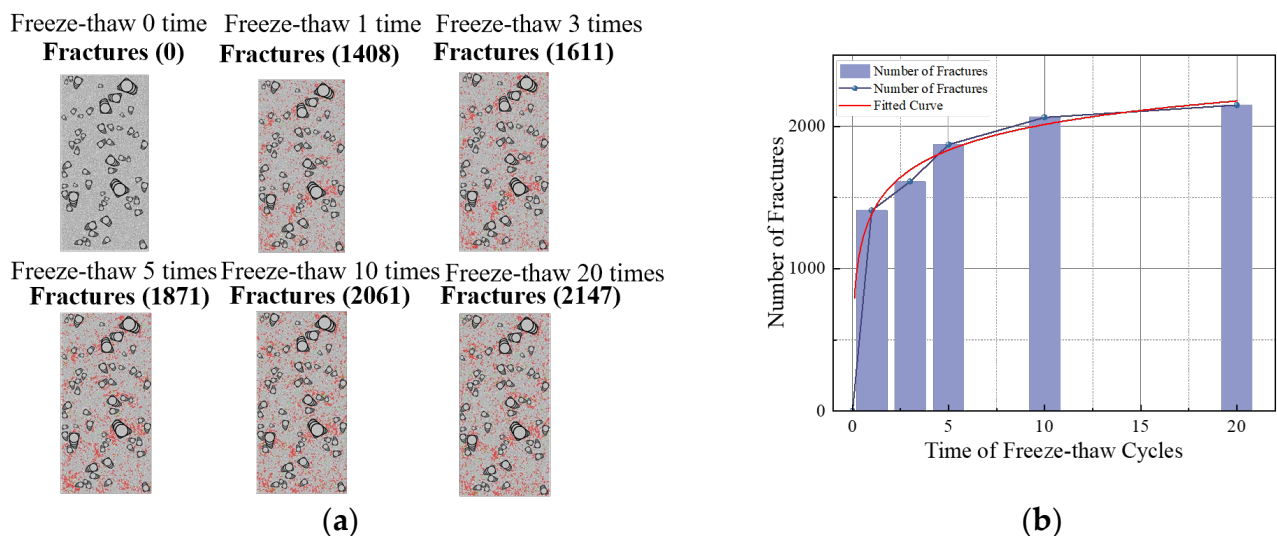


Figure 12. Number of cracks in SRM under different numbers of freezing and thawing cycles. (a) Number of fractures in the soil–rock mixtures at different numbers of freeze–thaw cycles. (b) Relationship of modeled cracks with the number of freeze–thaw cycles.

- Stress–strain curve:

Figure 13 shows the stress–strain curve of the SRM sample after different freeze–thaw cycles under a confining pressure of 100 kPa. From the figure, it can be seen that under different numbers of freeze–thaw cycles, the peak stress and elastic modulus of the curve will be different and decrease as the number of freeze–thaw cycles increases. On the other hand, after the stress reaches the peak value, the curve of cycle 0 almost fluctuates near the peak value. After undergoing freeze–thaw cycles, the curve will show a certain post-peak softening trend.

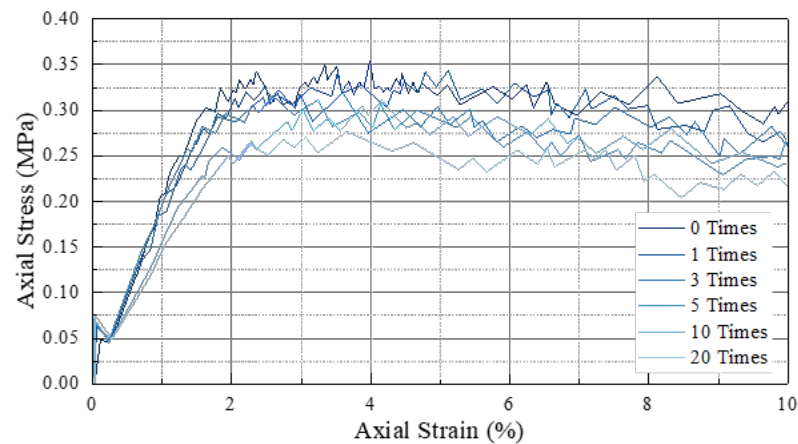


Figure 13. Stress–strain curves for different numbers of freeze–thaw cycles.

In the process of increasing the freeze–thaw cycles, it can be seen that the first freeze–thaw cycle causes the greatest damage to the soil–rock mixture, ranging from never experiencing frost heaving to contact fracture between particles to produce tensile cracks, the peak strength is reduced. As the number of freeze–thaw cycles increases, further damage will be caused, but the degree of damage will gradually decrease. Finally, it will almost remain unchanged when it reaches a certain number. For example, when the number of freeze–thaw cycles increases from 10 to 20, less than 100 tensile cracks are added, and the peak intensity is reduced by about 10 kPa.

3.4. Numerical Calculation Process

Figure 14 shows that the initial model is established as 130 m in length and 60 m in width. After balancing, the boundary servo is carried out according to the endowment conditions. The gravity is given to the soil and rock particles to make them settle. Finally, the slope-cutting treatment is carried out according to the height of a single step of 20 m, a slope surface angle of 33°, and a flat disk width of 50 m. This establishes a slope model for a soil dump site.

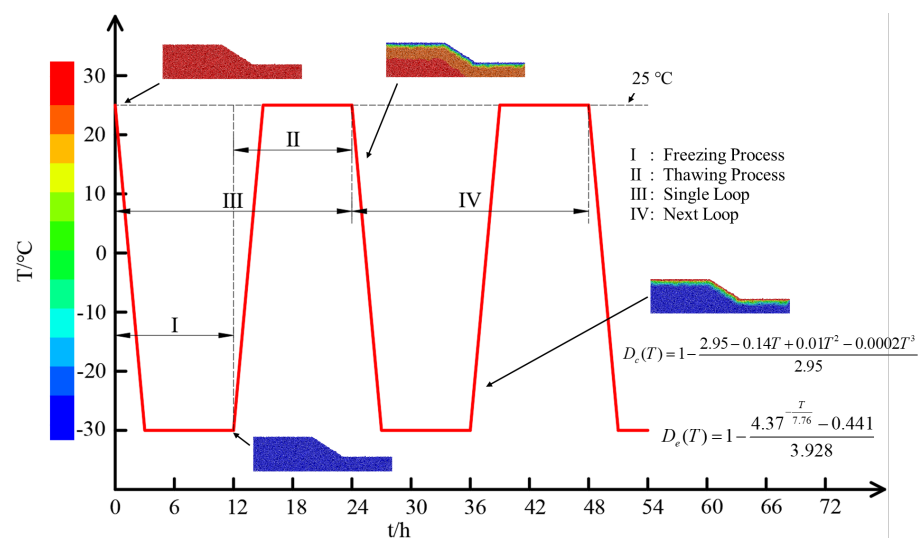


Figure 14. The freezing and thawing process of the dump site slope model.

The tensile and shear strength of the model are mainly determined by the parameters pb_ten and pb_coh . The parameters are continuously reduced until critical failure of the slope occurs. At this time, the ratio of the front and rear parameters is determined as the strength reduction coefficient. The most intuitive displacement mutation basis for judging

critical damage in PFC(5.0) software is used to judge the critical damage of the slope. As the reduction coefficient increases, the slope deformation changes greatly. After reaching a certain reduction coefficient, the slope displacement will suddenly increase, and the slope will slide as a whole. This reduction coefficient is the safety factor of the slope model.

4. Results

4.1. Simulation Results of Slopes with Different Freeze–Thaw Cycles

According to the freeze–thaw temperature curve and damage evolution function in Figure 9, a coupled simulation of freeze–thaw cycles was performed on the dump site slope model, and the slope stability of 1, 3, 5, 10, and 20 freeze–thaw cycles was analyzed. Figure 15 and Table 3 show the slope landslide conditions under different freeze–thaw cycle conditions and the strength reduction coefficient. Under the condition of one freeze–thaw cycle, when the reduction coefficient gradually increased to 1.60, a large-scale landslide occurred on the slope, affecting the entire slope, and the maximum particle displacement reached 18.82 m. Under 3, 5, 10, and 20 freeze–thaw cycles, the dumpsite slope model produced large displacement landslides exceeding 10 m when the reduction coefficient reached 1.50, 1.50, 1.30, and 1.20, respectively, and the maximum particle displacement was 11.30 m, 16.58 m, 11.93 m, and 12.53 m. Comparing the landslide conditions of each slope under different freeze–thaw cycles, it can be observed that the landslide area and slip surface are the same, indicating that freeze–thaw cycles will affect the stability coefficient of the slope but will not have a significant impact on the shape of the landslide.

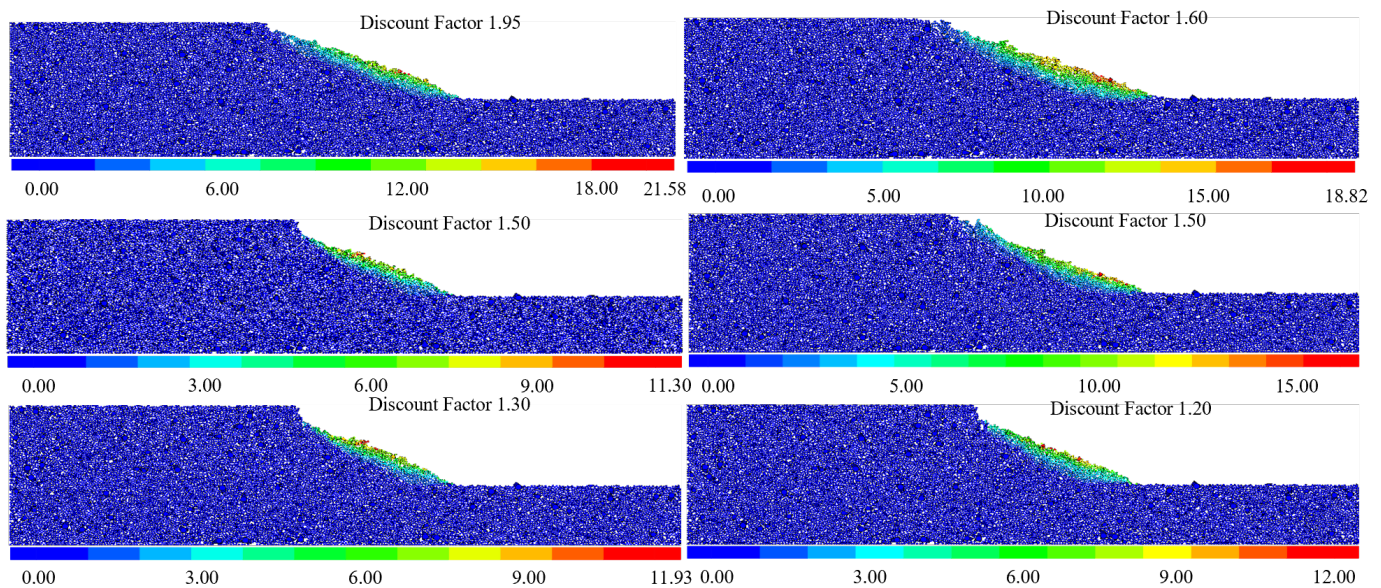


Figure 15. Slope displacement under different freeze–thaw cycle conditions.

Table 3. Slope damage under different freeze–thaw cycle conditions.

Number of Freeze–Thaw Cycles	0	1	3	5	10	20
Discount Factor	1.95	1.60	1.50	1.50	1.30	1.20
Maximum particle displacement (m)	21.58	18.82	11.30	16.58	11.93	12.53

Under the influence of freeze–thaw cycles, the slope of the dump site shows an increase in cracks and a decrease in contact bonds on a microscopic scale. Numerical simulations were performed for this angle, and the results are shown in Figures 11 and 12. From the slope model, after freezing and thawing, it can be found that the area with the most dense cracks is on the left side of the upper working flat plate. There are fewer cracks on the right side of the flat plate, the lower working flat plate, and the slope surface. For each position of the slope model, measuring points are set up as shown in Figure 16 ranging

from No. 1 to 8; the stress at each measuring point is monitored; and the stress in the x and y directions is obtained as shown in Table 4. In the slope model, the stress in the shallow soil area (measurement points 1–6) is small, in the range of 10 kPa to 70 kPa, especially at measurement points 1 and 2; this stress is very small compared to the deep soil at Nos. 7 and 8. During the freeze–thaw cycle, the frozen volume of water particles becomes larger, exerting greater stress on the surrounding rock and soil. Because the shallow soil has a smaller original stress and is loosely consolidated, the influence of the freeze–thaw cycle will generate more cracks in this area. The deep soil has greater stress and tighter consolidation. The freeze–thaw cycle under the same conditions has less impact, and the number of cracks in this area is smaller.

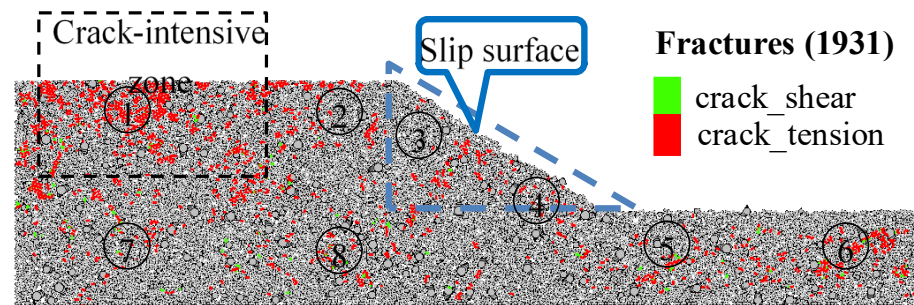


Figure 16. Crack distribution of the slope model (①–⑧ are the monitoring point codes).

Table 4. Measurement point stress data.

Measuring Point	1	2	3	4	5	6	7	8
Stress in the X-direction/kPa	20	14	27	71	32	140	160	100
Stress in the Y-direction/kPa	49	25	49	62	27	64	420	320

According to Figure 17, as the number of freeze–thaw cycles gradually increases, the cracks generated in the slope model also increase, and the number of model contacts decreases. When the freeze–thaw cycle increases from 0 to 1, the number of cracks increases by 1931, and the number of contacts decreases by 1102, representing the largest changes. From 1 freeze–thaw cycle to 10, the number of cracks increases by 941, and the number of contacts decreases by 416. When the number of freeze–thaw cycles increases from 10 to 20, the number of cracks increases by 648, and the number of contacts decreases by 345. It can be seen that the number of cracks increases, and the number of contacts decreases by 345. The greater the number of freeze–thaw cycles, the smaller the damage to the slope. As the number of subsequent freeze–thaw cycles increases, the damage will gradually decrease to approximately zero.

Freezing and thawing cycles produce two types of cracks in geotechnical mixtures: one caused by stretching and one caused by shearing [44]. Among them, the most important damage is caused by volume expansion and stretching; volume expansion occurs in geotechnical materials during the freezing process when the internal water in the low-temperature environment freezes into ice. This transition significantly increases volume and produces greater stress on the surrounding geotechnical soil. This leads to further expansion of geotechnical fissures; the geotechnical structure of the damaging effects of the freezing and thawing cycle will continue to accumulate, resulting in the gradual deterioration of the geotechnical structure. Although the volume expansion theory can explain part of the freeze–thaw damage phenomenon, not all freeze–thaw damage can be attributed to volume expansion. Freeze–thaw damage is a complex process that involves the interaction of several factors, including water migration, chemical action, and the formation and growth of ice crystals. During freezing, water migrates from areas of high potential energy to areas of low-potential energy, especially when the soil freezes, and unfrozen water is transferred to the frozen surface, a phenomenon known as positive permafrost

water migration. In the freeze–thaw cycle, moisture is continuously transformed and migrated between solid, liquid, and gaseous states as the temperature rises and falls. This moisture migration has an important effect on freeze–thaw damage. Migrated moisture forms ice crystals during freezing, which produces volume expansion and pressure on the geotechnical structure, leading to cracking and fragmentation within the geotechnical structure. In turn, the melting of ice crystals during thawing may lead to destabilization and the settlement of the geotechnical structure. The volume expansion and migration of water particles during the freeze–thaw process is one of the main causes of freeze–thaw damage, as shown in Figure 18.

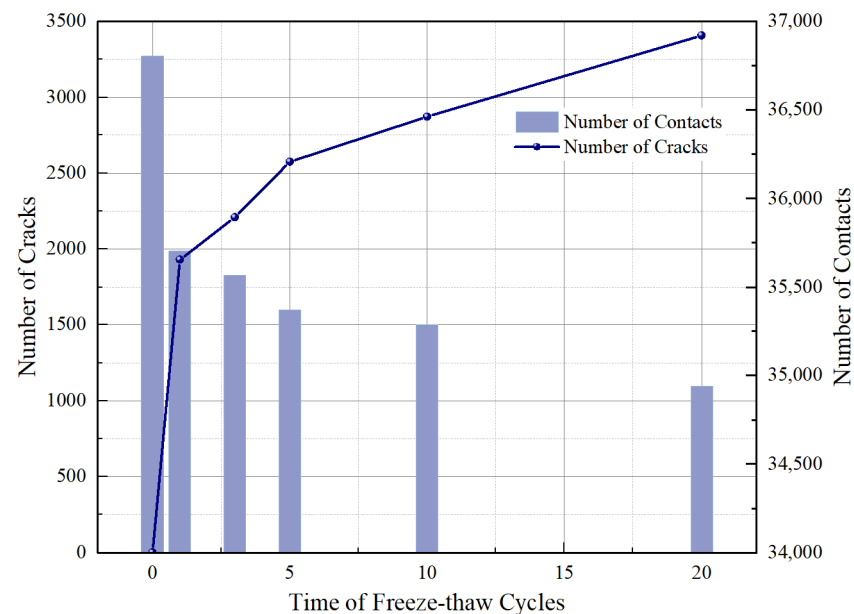


Figure 17. Changes in the number of contacts and cracks during the freeze–thaw cycle.

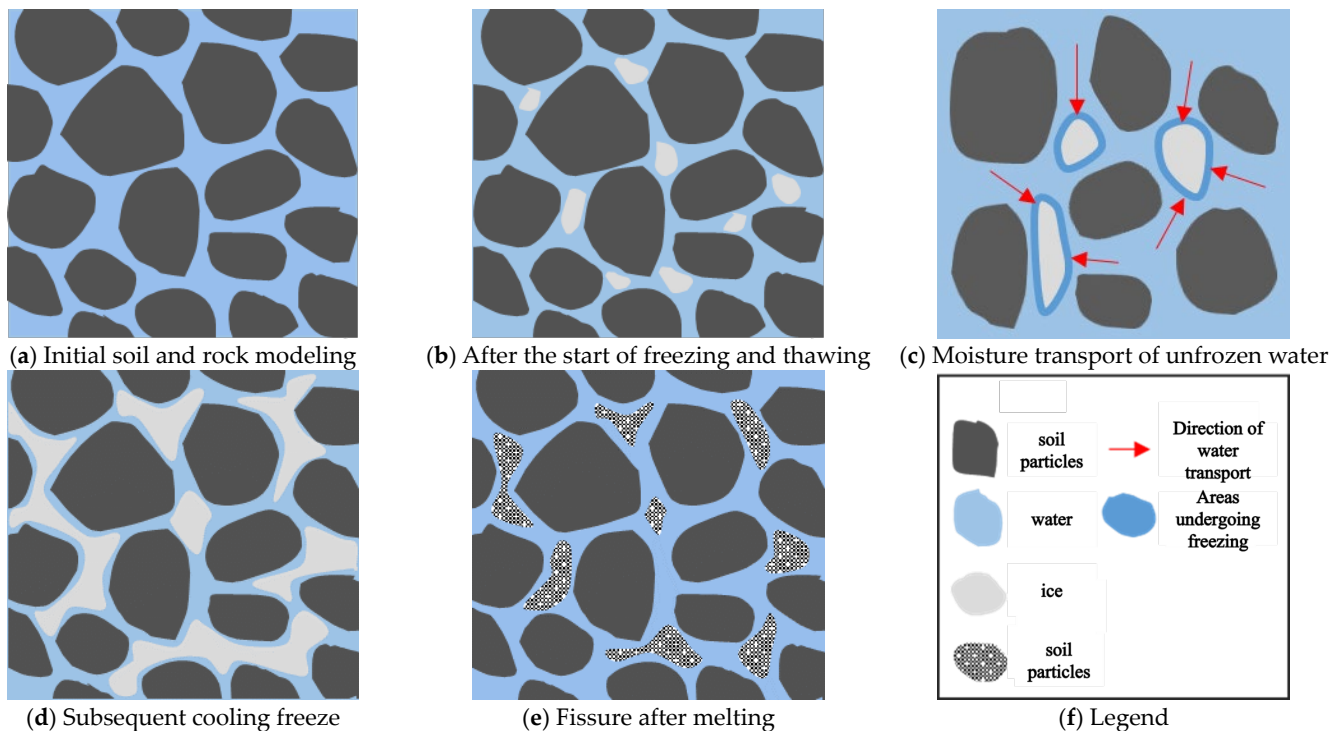


Figure 18. Freeze–thaw damage process.

Figure 19a shows the relationship between the maximum particle displacement and the reduction coefficient obtained by reducing the strength of the dump site slope under different freeze–thaw cycle conditions to induce landslides. At 0, 1, 3, 5, 10, and 20 freeze–thaw cycles, the reduction coefficients when particle displacement suddenly occurs are 1.92, 1.58, 1.47, 1.43, 1.27, and 1.18, respectively. According to each freeze–thaw cycle, the slope safety factor under cyclic conditions can be obtained by obtaining the relationship between the slope stability coefficient of the dump site and the freeze–thaw cycle period, as shown in Figure 19b.

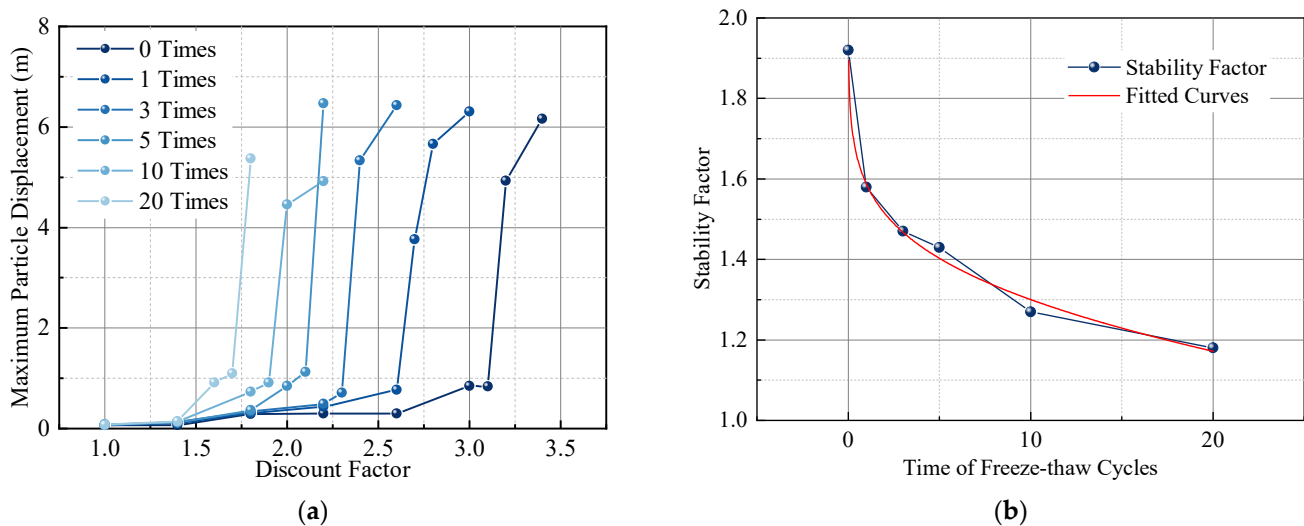


Figure 19. Relationship between the stability coefficient of the dump site slope and the number of freeze–thaw cycles. (a) Relationship between maximum particle displacement and folding coefficient. (b) Stability coefficient versus the number of freeze–thaw cycles.

Analysis of Figure 19b shows that the stability coefficient of the dump site slope decreases with the increase in the number of freezing and thawing cycles. According to the fitting results, the two have a power function relationship:

$$F_s = (-175.06 + 180.32 \times T^{-0.0065})^{-\frac{1}{3.59}} \quad (7)$$

Among them is the stability coefficient; T is the number of freeze–thaw cycles, and the correlation coefficient is 0.93.

Under the influence of freeze–thaw cycle deterioration, the macroscopic strength, cohesion, and internal friction angle of the SRM will be damaged, thus affecting the stability coefficient of the dump site slope. After undergoing the first freeze–thaw cycle, the slope stability coefficient dropped sharply by 0.34, with an attenuation amplitude of about 17.71%. Then, it steadily decreased with the increase in the number of freeze–thaw cycles before finally becoming stable. The slope stability coefficient attenuated the entire process by about 38.54%.

4.2. Simulation Results of the Freeze–Thaw Slope of the Dump Site Under Different Dumping Heights

After conducting stability analysis on the 20 m high dump site slope, we established 10 m high and 30 m high slope models, respectively, under 0, 1, 3, 5, 10, and 20 freeze–thaw cycles. After strength reduction, the shapes of 10 m, 20 m, and 30 m slopes after landslides are shown in Figure 20.

From the figure, we can see that the particle displacement after landslides is different for slopes with different dump heights; when the dump height is 10 m, the maximum displacement of particles after landslides is 4.94 m, while when the dump height reaches 20 m and 30 m, the maximum displacement of particles after landslides reach 21.58 m and 30.71 m. The volume of particles that have been slid varies significantly when the

dump height is 10 m and the landslides occur in a small area. The influence of slides on the upper and lower working flat disks is small, while the influence of slides on the upper and lower working flat disks is far more than that of 10 m and 20 m platforms. When the dump height is 10 m, the landslide occurs in a small area, the particles on the slope surface slide downward, and the influence on the upper and lower working flat plates is small. In comparison, when the dump height reaches 30 m, the influence of the sliding body on the upper and lower working flat plates is far more than that in the case of 10 m and 20 m steps; meanwhile, when the height of the steps is high, in the process of the landslide occurring, not only do the particles along the sliding surface slide downward but also there is a certain amount of particles collapsing at the top of the slope.

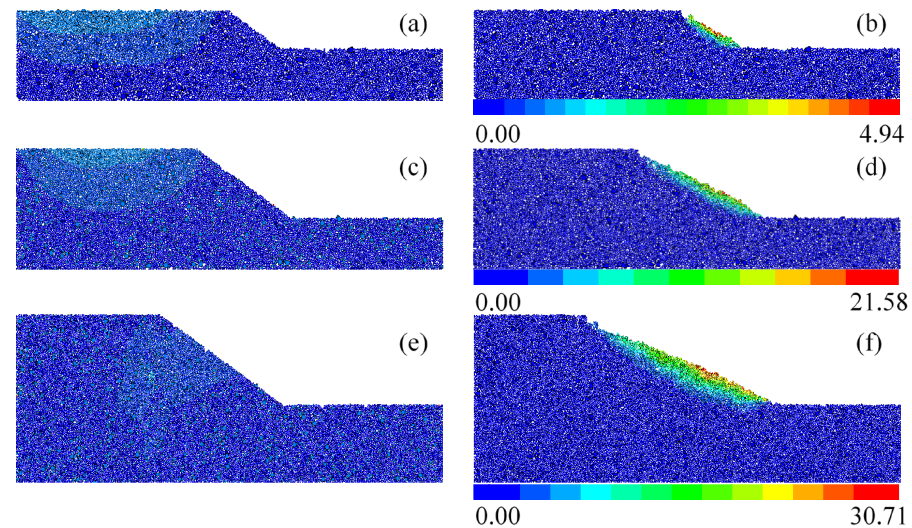


Figure 20. Comparison of before and after landslides on slopes with different drainage heights. (a) Undamaged slopes of a 10 m high earth disposal site. (b) Damaged slopes of a 10 m high earth disposal site. (c) Undamaged slopes of a 20 m high earth disposal site. (d) Damaged slopes of a 20 m high earth disposal site. (e) Undamaged slopes of a 30 m high earth disposal site. (f) Damaged slopes of a 30 m high earth disposal site.

The slope model of the dump site with different dump heights was monitored for the stress condition at the top and bottom of the slope during landslides. The locations of the monitoring points are shown in Figure 21, and the change in stress in the horizontal and vertical directions at each point is shown in Figure 22.

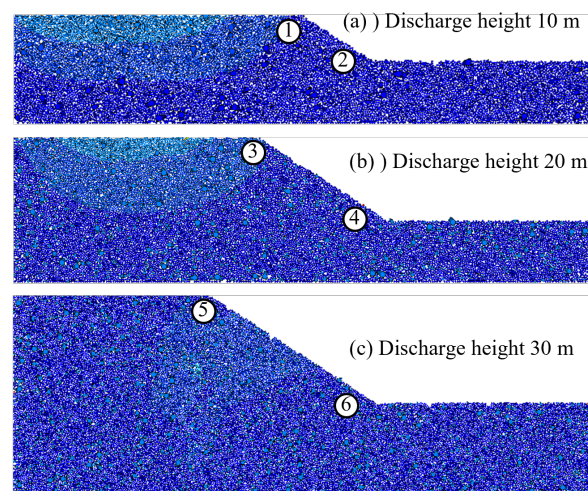


Figure 21. The location of the monitoring points of the slope model with different dump heights.

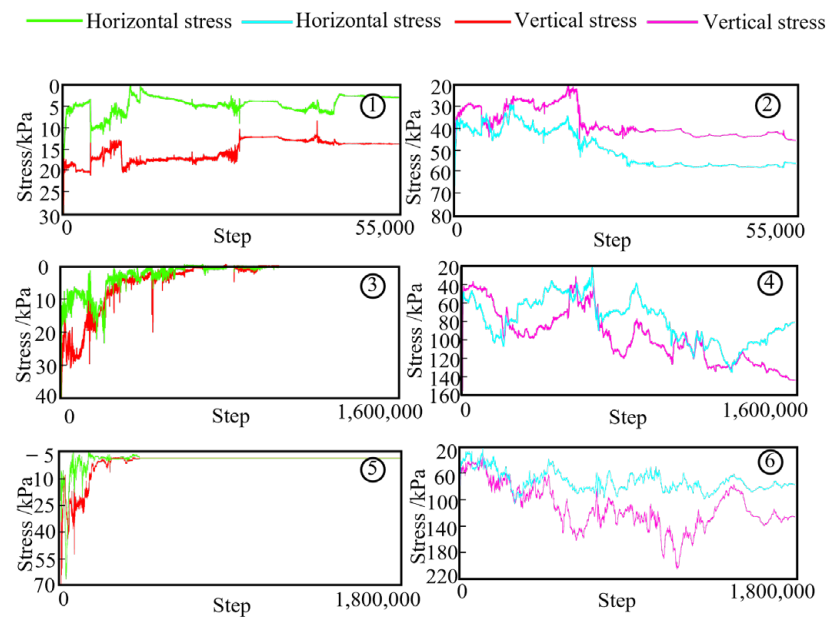


Figure 22. Stress changes at each monitoring point (①–⑥ are the data of Figure 21).

The overall trend observed is that the horizontal and vertical stresses at the top of slopes with different abandonment heights are decreasing, while the horizontal and vertical stresses at the bottom of slopes are increasing. During landslides, the particles at the top of the slopes slide downward along the sliding surface and finally pile up at the bottom of the slopes, so the stress at the top of the slopes decreases, and the stress at the bottom of the slopes increases. After the 10 m height landslide, the particle displacement is small, the particles at the monitoring point of the top of the slope do not all slide down, the particles at the monitoring point of the top of the slope after the landslide of 20 m and 30 m height all slide down the slip surface, and the final stress in the horizontal and vertical direction is reduced to zero. In addition, at the bottom of the slope, the particles are observed to slide out to the lower working flat disk. The stress at the monitoring point at the bottom of the slope increases to a certain fixed value; the 10 m step is kept at a horizontal stress of 40 kPa and a vertical stress of 60 kPa; and the 20 m and 30 m steps are kept at a horizontal stress of 60 kPa and a vertical stress of 140 kPa or so. At this point, as the slope begins to slide, the particles above the monitoring point at the bottom of the slope accumulate to their maximum value. Following this, the stress increases step by step, resulting in a trend where the stress first increases locally and then decreases. Additionally, at this point, the slope slides completely. The particles accumulate above the monitoring point at the bottom of the slope to their maximum value and then increase. The particles above slide to the monitoring point above and then roll down to the work plate, resulting in a trend where the stress increases locally and then decreases.

Figures 23 and 24 show the slope stability of the dumpsite slope under freeze–thaw cycles at 10 m and 30 m disposal heights, respectively. For a dump site slope with a disposal height of 10 m, its stability coefficient has a power function relationship with the number of freeze–thaw cycles.

Combined with the above 20 m step height of the slope dump site, it can be seen that the stability of the slope at different heights exhibits a similar trend in relation to the number of freeze–thaw cycles. This relationship can be described as a power function of the trend of change. As the number of freeze–thaw cycles gradually increases, the stability coefficient decreases. However, the rate of this decrease will become smaller and smaller. With the increase in dump height, the stability coefficient of the slope of the dump site will become lower and lower, and the slope will become more unstable. With a dump height of 10 m, after 20 freeze–thaw cycles, the slope stability coefficient will change from 3.31 to

1.72, where the slope is still in a stable condition, there is a dump height of 30 m after 20 freeze–thaw cycles, and the slope stability is reduced to 0.88. In this case, the slope is easy to destabilize the landslide. Therefore, for the slope of the open-pit mine dump, increasing the height of stacking steps can, to a certain extent, increase the volume of dumping in a fixed area. Still, the slope stability will be reduced, and the impact of landslides will be more considerable after undergoing freeze–thaw cycles; the stability of the slope of the dump will be further reduced, which is a more significant potential danger to the safety of open-pit mine production.

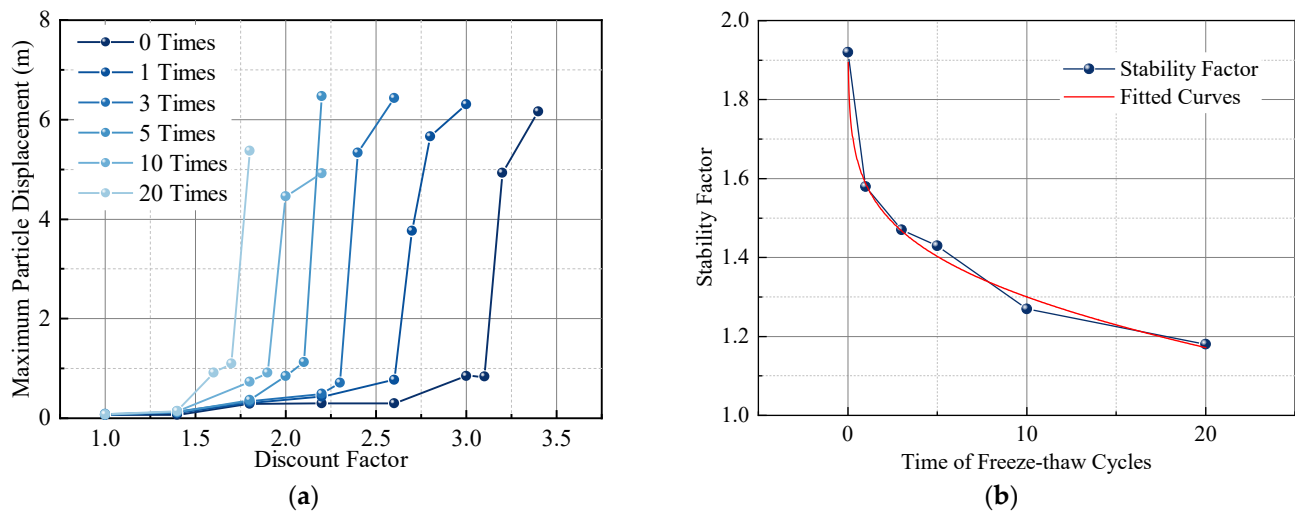


Figure 23. The relationship between the stability coefficient of the 10 m step slope and the number of freeze–thaw cycles. (a) Relationship between maximum particle displacement and folding coefficient. (b) Stability coefficient versus the number of freeze–thaw cycles.

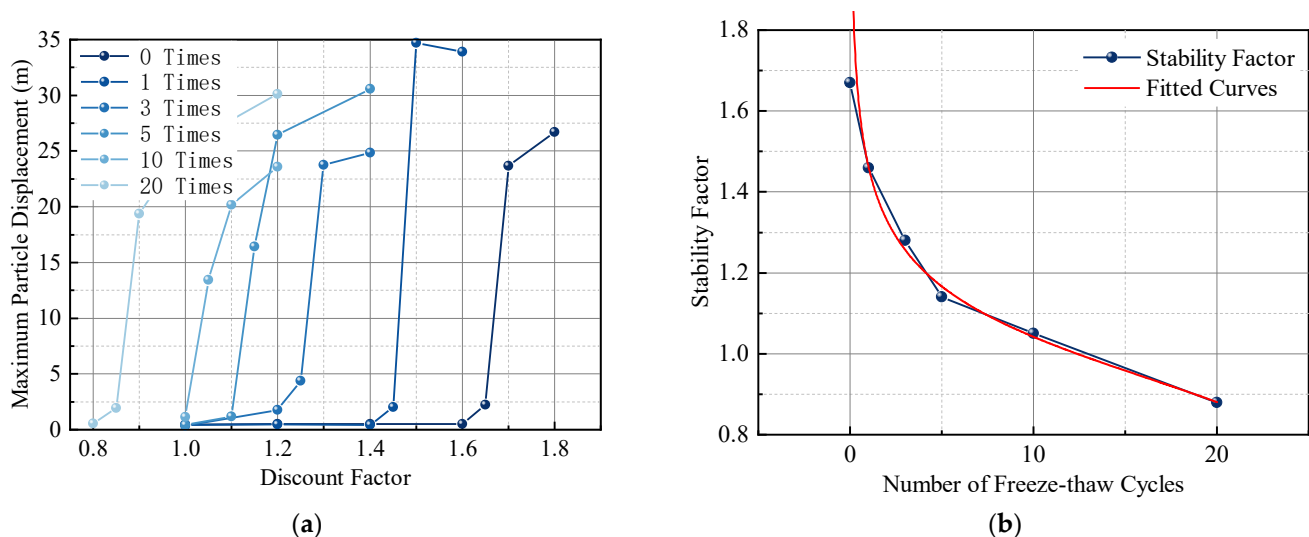


Figure 24. The relationship between the stability coefficient of the 10 m step slope and the number of freeze–thaw cycles. (a) Relationship between maximum particle displacement and folding coefficient. (b) Stability coefficient versus the number of freeze–thaw cycles.

5. Conclusions

In this paper, for the stability of an open-pit mine dump field in a cold area, the mechanical properties of a soil–stone mixture under different numbers of freeze–thaw cycles were investigated using a lab triaxial test and a numerical simulation of the particle flow program. The destructive properties of the soil–stone mixture were investigated from the macroscale and fine viewpoints, and the model of the open-pit mine dump field was

set up to study the stability law of the slope of the dump field under the conditions of freeze–thaw cycles in terms of the actual dimensions. Thus, the following conclusions were obtained:

- The stress–strain curves of soil–rock mixtures, tested under triaxial tests of different numbers of freezing and thawing cycles and different water contents, demonstrate similar trends. These curves show three stages: linear growth, stress yielding, and a stage that was unchanged. Additionally, at higher peripheral pressures, the strain hardening phenomenon is more obvious. Comparing the strength changes in a soil–stone mixture under different numbers of freeze–thaw cycles, its compressive strength under 100 kPa peripheral pressure decreases by 65 kPa as a quadratic function of the increase in the number of freeze–thaw cycles, cohesion decreases by 12 kPa, and the angle of internal friction decreases by 2.7° as a power function of the increase in the number of freeze–thaw cycles. Comparing the stress–strain curves of soil–stone mixture samples with different water content after freeze–thaw cycles, the lower the water content, the more obvious the strain hardening; the higher the water content, the greater the effect of freeze–thaw cycles on soil–stone mixtures, and the greater the decrease in compressive strength, cohesion, and internal friction angle.
- As the number of freezing and thawing cycle simulations increases, the number of internal fine cracks in the soil and rock mixture model also increases. When the number of cycles changes from 0 to 1, the number of cracks increases to 1408 due to damage caused by freezing and thawing. However, when the number of freezing and thawing cycles changes from 10 to 20, the change in the number of cracks is small, the compressive strength reduction is also small, the mechanical properties of soil and rock mixture tend to be stabilized, and the effect of the freezing and thawing cycle becomes smaller.
- For different numbers of freezing and thawing cycles under the row of slopes, the stability coefficient and model fissure change rule are similar; with the increase in the number of freezing and thawing cycles, the slope stability coefficient gradually decreased from 0 to 20; the stability of the slope decreased by about 0.74, resulting in a reduction in the magnitude of about 38.54%, of which the stability coefficient occurred between 0 and 1 cycle, after which the stability began to stabilize and remained unchanged.
- Under different discharging heights, the slope stability coefficient decreases with the increase in the number of freeze–thaw cycles in a power function trend; the higher the discharging height, the lower the slope stability coefficient. The slope becomes more unstable and more prone to landslides after undergoing more freeze–thaw cycles. It also has a larger impact after landslides with higher risks.

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

Funding: This research was supported by the National Key R&D Program of China (2023YFF1306001); the National Natural Science Foundation of China (52204159); and the Natural Science Foundation of Jiangsu Province, China (BK20221125).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

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

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

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