

Article

Study on the Mechanical Properties of Frozen Clay in Coastal Formations and Damage Constitutive Models

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Abstract

To address the complex mechanical properties of coastal clay encountered during artificial ground freezing construction in coastal regions, this study investigates coastal clay by conducting mechanical tests under various freezing temperatures and confining pressures. The variations in peak strength, elastic modulus, and shear strength parameters of the frozen coastal clay were systematically analyzed. The results indicate that the peak strength and elastic modulus of the coastal clay exhibit a clear upward trend with approximately linear increases in confining pressure and temperature. A damage constitutive model for frozen soil was established, accounting for the coupled effects of temperature and confining pressure. Furthermore, a Physics-Informed Neural Network (PINN), incorporating the constitutive equations as physical constraints, was introduced to achieve the inversion and prediction of model parameters. The results demonstrate that the PINN model exhibits excellent predictive accuracy, with the coefficient of determination (R^2) reaching 0.8513 under uniaxial compression and 0.9689 under triaxial compression. Compared with traditional empirical models, the PINN model reduces the prediction error of the elastic modulus to 8.96% (at 0 MPa) and 4.79% (at 0.8 MPa), demonstrating superior generalization capability. This study provides a reliable theoretical basis and parameter guidance for frozen soil construction in coastal regions.

Keywords: artificial freezing method; coastal clay; mechanical properties; damage constitutive model; PINN model

1. Introduction

As surface transportation in China's coastal cities approaches saturation, the development of underground space has become an inevitable trend [1,2]. However, these projects often require traversing coastal clay strata, which exhibit characteristics such as high water content and low strength, and are typically in a soft plastic or flow plastic state. Under construction loads, they are highly susceptible to failure modes including instability deformation, settlement, and landslides [3,4]. For underground engineering in coastal regions, artificial freezing not only enhances soil strength but also forms a water-stopping curtain. Compared to methods like grouting and diaphragm walls, it offers greater applicability in complex geological conditions while minimizing surrounding impacts [5,6]. However, the mechanical behavior of frozen coastal clay is influenced by the coupled effects of multiple factors, including temperature and confining pressure. Therefore, in-depth investigation of the mechanical behavior of frozen clay in coastal areas, revealing its mechanical properties



Academic Editor: Tiago Filipe da Silva Miranda

Received: 4 June 2026

Revised: 21 June 2026

Accepted: 24 June 2026

Published: 3 July 2026

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and constitutive relationships, is of great significance for stability analysis in underground engineering construction.

The application of artificial freezing technology in underground engineering construction has a history of over a century [7–9]. Physical experiments have revealed the mechanical properties of soils under complex environmental conditions. Shastri et al. [10] conducted triaxial shear tests on in situ natural permafrost in Alaska, investigating the influence of temperature, confining pressure, and strain rate on the stress–strain characteristics, strength parameters, and volumetric deformation of frozen soil. Li et al. [11] conducted triaxial unloading creep tests, discovering that reduced confining pressure significantly weakens lateral confinement, accelerates internal damage evolution, and causes premature plastic yielding and failure in frozen soil at lower stress levels. Fan, GW et al. [12] conducted triaxial shear tests on frozen soils with loose aquifers to investigate the effects of initial moisture content, freezing temperature, and confining pressure on peak strength, residual strength, and elastic modulus of frozen soils, clarifying the failure evolution characteristics of frozen soils under mining stresses. To further investigate the microscopic mechanisms underlying macroscopic failure of frozen soil, Liang, YW et al. [13] conducted triaxial shear tests on salt-containing frozen sand with varying densities. By measuring internal pore water pressure, they revealed the effects of pore ice and unfrozen water on the effective cohesion and internal friction angle of sand.

Regarding constitutive model research, accurately and efficiently describing the mechanical behavior and damage mechanisms of frozen soil under complex external conditions remains a challenge. Yao, XL et al. [14] proposed a Burgers model incorporating cohesion decay, which reasonably describes the continuous evolution of creep strain in frozen soil from the decaying to non-decaying stages. Sun, K et al. [15] established a poly-elastic–plastic constitutive model for frozen soil, incorporating temperature effects, confining pressure effects, and ice cementation. This model was developed through triaxial tests under varying temperatures and confining pressures, enabling simultaneous description of soil hardening/softening and shear contraction/shear expansion characteristics. An elastic–plastic constitutive model for frozen soil incorporating freeze-induced anisotropy was developed by Yin, Q et al. [16] based on residual stress theory. A unified constitutive relationship between thawed and frozen soil was achieved through a two-step extension. A constitutive model incorporating a two-variable damage evolution equation was developed by K Chen et al. [17] based on Lemaitre’s principle and Weibull distribution to enable quantitative assessment of low-temperature damage and precise prediction of stress–strain behavior. Addressing more complex freeze–thaw cycling conditions, Dan Wang et al. [18] accurately reproduced the nonlinear deformation characteristics under the coupled mechanism of ice-gel bonding fracture and friction slip based on a dual-medium model, providing a novel perspective on how microstructure governs macroscopic mechanical behavior. Although the above models are effective in explaining physical mechanisms, they have notable limitations in engineering applications, particularly in predictive accuracy and computational efficiency. Burgers-type and dual-medium models involve numerous empirical parameters calibrated by predefined fitting functions and multi-step regressions, which are costly, prone to local optima, and weak in generalization. Similarly, Lemaitre-based damage models rely heavily on subjective empirical fitting, making efficient parameter inversion across different conditions difficult.

To overcome the bottlenecks of computational inefficiency and limited accuracy inherent in traditional constitutive modeling, the Physics-Informed Neural Network (PINN) framework has emerged as a superior alternative. Recent studies have validated this potential: Huiwei Zhao et al. [19] have successfully applied RBF neural networks integrating multi-level macro–micro parameters for precise prediction of uniaxial compressive

strength (UCS). Addressing the limitation of traditional data-driven models relying on large amounts of labeled data, KQ Li et al. [20] embedded heat conduction equations into PINN to achieve finite-element-level accuracy in temperature field predictions; Ayensa-Jiménez et al. [21] proposed PGNNIV to output physically meaningful internal variables for macroscopic mechanical computation; and Roy et al. [22] utilized transfer learning-based PINNs to achieve rapid convergence with minimal training data. Despite remarkable advances in frozen soil mechanics and data-driven modeling techniques, integrating the physical damage mechanisms of frozen clay into neural network frameworks to address the drawbacks of traditional parameter calibration and purely data-driven models remains a prominent and urgent research challenge.

This study conducted a series of mechanical tests on coastal clay to characterize its mechanical performance. Based on experimental observations, a damage constitutive model incorporating temperature and confining pressure effects was established and embedded into the Physics-Informed Neural Network (PINN) framework. The innovation lies in embedding the physical damage evolution laws of frozen coastal clay directly into the forward propagation of the neural network. This integration ensures physically consistent predictions, reduces dependency on extensive training data, and guarantees generalization capabilities comparable to traditional models, effectively overcoming the poor extrapolation performance of purely data-driven approaches. Consequently, the mechanical parameters and stress–strain behaviors of coastal clay are predicted efficiently and accurately, providing reliable theoretical support for artificial freezing construction in complex coastal regions.

2. Mineral Composition and Physical Properties of Coastal Clay

The coastal clay was obtained from a shallow-layer silty clay in a coastal region, where thick marine alluvial–fluvial cohesive soils underlie the area, presenting complex engineering geological conditions. The test specimen for this study was silty clay from the Quaternary Holocene marine deposits (Q_4^m). This soil exhibits a gray to gray-black color, predominantly in a fluid plastic state with localized soft plastic sections, and emits a putrid odor. Mineral composition of the coastal clay sample was analyzed via an X'Pert PRO X-ray diffractometer, and the mineral diffraction patterns and component contents are presented in Figures 1 and 2. The results show that muscovite, illite, sodium feldspar and quartz dominate the clay, with mass fractions of 57.1%, 22.6%, 11.5% and 8.8%, respectively.

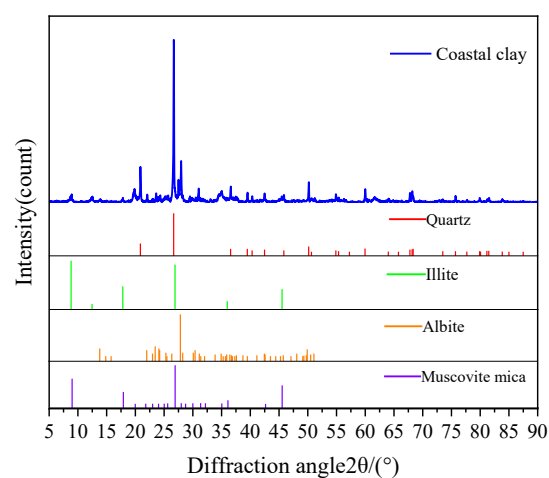


Figure 1. Mineral diffraction spectrum of coastal clay.

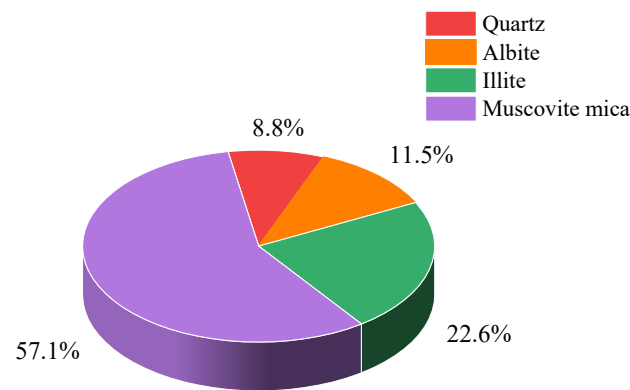


Figure 2. Mineral composition content diagram of coastal clay.

In addition, the particle size distribution of the sample was determined through field sampling and laboratory testing, as presented in Figure 3.

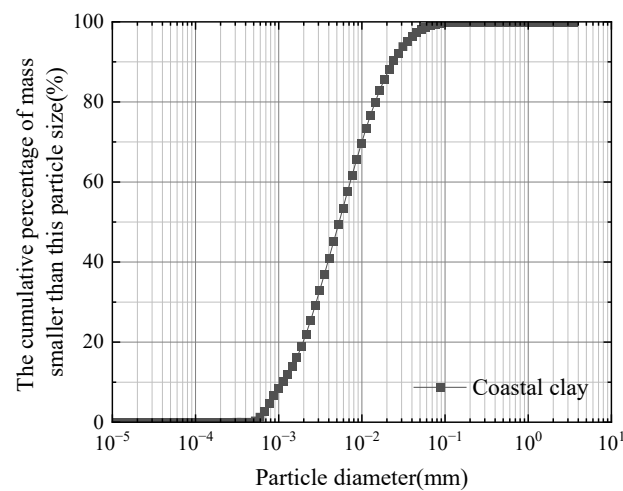


Figure 3. Particle size distribution curve of coastal clay.

Data indicate that the coastal clay in this area contains the highest proportion of silt particles (0.005–0.075 mm), reaching 50.06%, fine sand (0.075–0.25 mm) accounted for 0.64% and silt (<0.005 mm) constituted 45.14%. The particle size distribution reflects the uniformity of particle grading and determines the pore distribution of the soil, thereby influencing the water migration pathways during the freezing process of frozen soil. The basic physical parameters of the coastal clay are presented in Table 1.

Table 1. Basic physical parameters of soil sample.

Soil Sample	Natural Moisture Content/%	Natural Specific Gravity/ $\text{kN} \cdot \text{m}^{-3}$	Wet Density/ $\text{g} \cdot \text{cm}^{-3}$	Liquid Limit/%	Plastic Limit/%
Silty clay	48.2	17.6	1.78	53.63	25.50

3. Test Plan

To eliminate interference from other factors in the undisturbed soil and enhance test comparability, this experiment employs remolded soil specimens uniformly prepared into cylinders measuring 61.8 mm in diameter and 125 mm in height. The preparation process for remolded soil is as follows: (1) Crush the dried soil samples using a soil pulverizer. (2) Re-mix with water. (3) Sieve using a 5 mm soil sieve to ensure the maximum particle size of the mixed soil meets the standard for soil testing methods GB/T 50123-2019 [23], where

the maximum allowable particle size of the soil sample must be less than $d_{\max} \leq 1/10$ of the specimen diameter D . (4) Seal the screened soil in airtight bags and let it rest for 24 h. (5) Re-test the moisture content of the rested soil sample. (6) Coat the inner walls of the mold with petroleum jelly and compact the remolded soil sample using the compression method. The specimen preparation process is shown in Figure 4. Place the prepared specimen into a film-forming cylinder, cover it with a latex film, and place it in a programmable constant temperature and humidity test chamber. Set the test temperature and maintain it for at least 12 h.

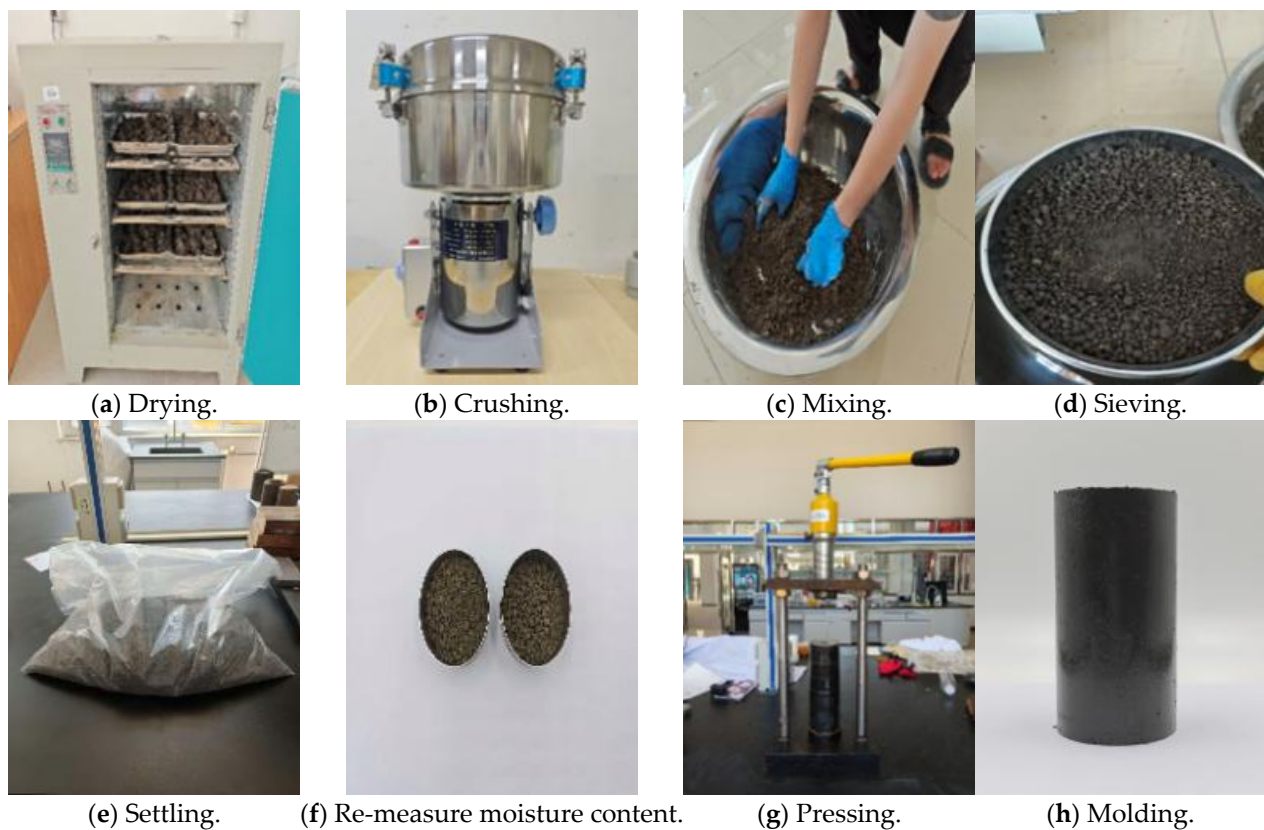


Figure 4. The process of preparing coastal clay samples.

In this study, to systematically investigate the influence of freezing temperature on the mechanical properties of coastal clay, four temperature levels, namely $-5\text{ }^{\circ}\text{C}$, $-10\text{ }^{\circ}\text{C}$, $-15\text{ }^{\circ}\text{C}$, and $-20\text{ }^{\circ}\text{C}$, were adopted in the triaxial compression tests, covering the temperature gradient along the thickness direction of the actual freezing curtain [24]. Meanwhile, with comprehensive consideration of the burial depth, frost heave pressure, and the extrusion effect induced by construction disturbances, the confining pressure was set at four levels of 0 MPa, 0.5 MPa, 1.0 MPa, and 1.5 MPa [25], resulting in a total of 16 test combinations. According to the standard for physical and mechanical property tests of artificially frozen soils, the axial strain rate was set at 1%/min (1.25 mm/min). The detailed test design parameters are presented in Table 2. To ensure the reliability and repeatability of the results, three replicate specimens were prepared and tested under each condition. The average values were adopted for the analysis of compressive strength and elastic modulus, and the standard deviations were calculated to evaluate data dispersion. For other parameters, representative data were selected for analysis.

Table 2. Triaxial compression test protocol.

Test ID	Confining Pressure/MPa	Freezing Temperature/°C
DW-1	0	−5
DW-2		−10
DW-3		−15
DW-4		−20
DW-5	0.5	−5
DW-6		−10
DW-7		−15
DW-8		−20
DW-9	1.0	−5
DW-10		−10
DW-11		−15
DW-12		−20
DW-13	1.5	−5
DW-14		−10
DW-15		−15
DW-16		−20

4. Analysis of Test Results

4.1. Stress–Strain Curves at Different Freezing Temperatures and Confining Pressures

Figure 5 shows the stress–strain curves of frozen soil at different freezing temperatures under varying confining pressures. Results indicate that during uniaxial compression tests, stress initially increases with strain. Upon reaching a certain strain level, stress gradually decreases with further strain increase before stabilizing at a relatively constant state. In triaxial compression tests, the deviatoric stress initially increases with strain, then exhibits decay, and finally increases slowly or remains constant as strain increases. No peak stress is observed throughout the test. Depending on loading conditions, the relationship between deviatoric stress and strain under loading can be categorized into two types: strain softening with a peak under uniaxial compression, and strain hardening without a peak under conventional triaxial compression.

4.2. Shear Failure Characteristics

During triaxial compression testing, frozen soil undergoes deformation failure, exhibiting distinct failure characteristics. To more intuitively reveal the damage evolution patterns of frozen coastal clay under loading, the macroscopic failure morphology of specimens after triaxial compression testing was analyzed, as shown in Figure 6. The figure displays the final failure patterns of specimens under different freezing temperatures (−5 °C, −10 °C, −15 °C, −20 °C) and confining pressures (0 MPa, 0.5 MPa, 1.0 MPa, 1.5 MPa). Image binarization was employed to extract and process the failure characteristics of post-test specimens. Through comparative analysis, the following patterns were identified:

Confining pressure is the key factor determining the failure mode of frozen coastal clay. Under uniaxial compression, specimens exhibited shear failure at all freezing temperatures. Macroscopic cracks penetrated the specimen surfaces, with specimens at −20 °C even showing spalling. This failure pattern corresponds to the strain softening stage of stress–strain curves with peaks. This indicates that under unconfined conditions, fracture of the ice-cemented structure within the frozen soil dominates the failure process. Under triaxial compression conditions, the failure mode of the specimens fundamentally changed from shear failure to swelling failure. Although the specimen surfaces were rough and wrinkled, no obvious through-cracks were observed. The specimens exhibited a “barrel-shaped”

profile, with a thicker middle section and thinner ends. This aligns with the stress–strain curve exhibiting no peak stress and strain hardening.

4.3. Effects of Freezing Temperature and Confining Pressure on Specimen Strength and Elastic Modulus

Based on the stress–strain curves of specimens under different temperatures and confining pressures, the variation patterns of compressive strength and elastic modulus with temperature and confining pressure can be obtained. For uniaxial compression tests, the stress–strain curve exhibits a distinct peak. The stress corresponding to the peak strain is taken as the compressive strength. For triaxial compression tests, no peak strength appears in the stress–strain curve; 15% of the strain is taken as the compressive strength. Additionally, based on geotechnical testing standards, the peak stress–strain and residual stress–strain of the coastal clay specimens were calculated.

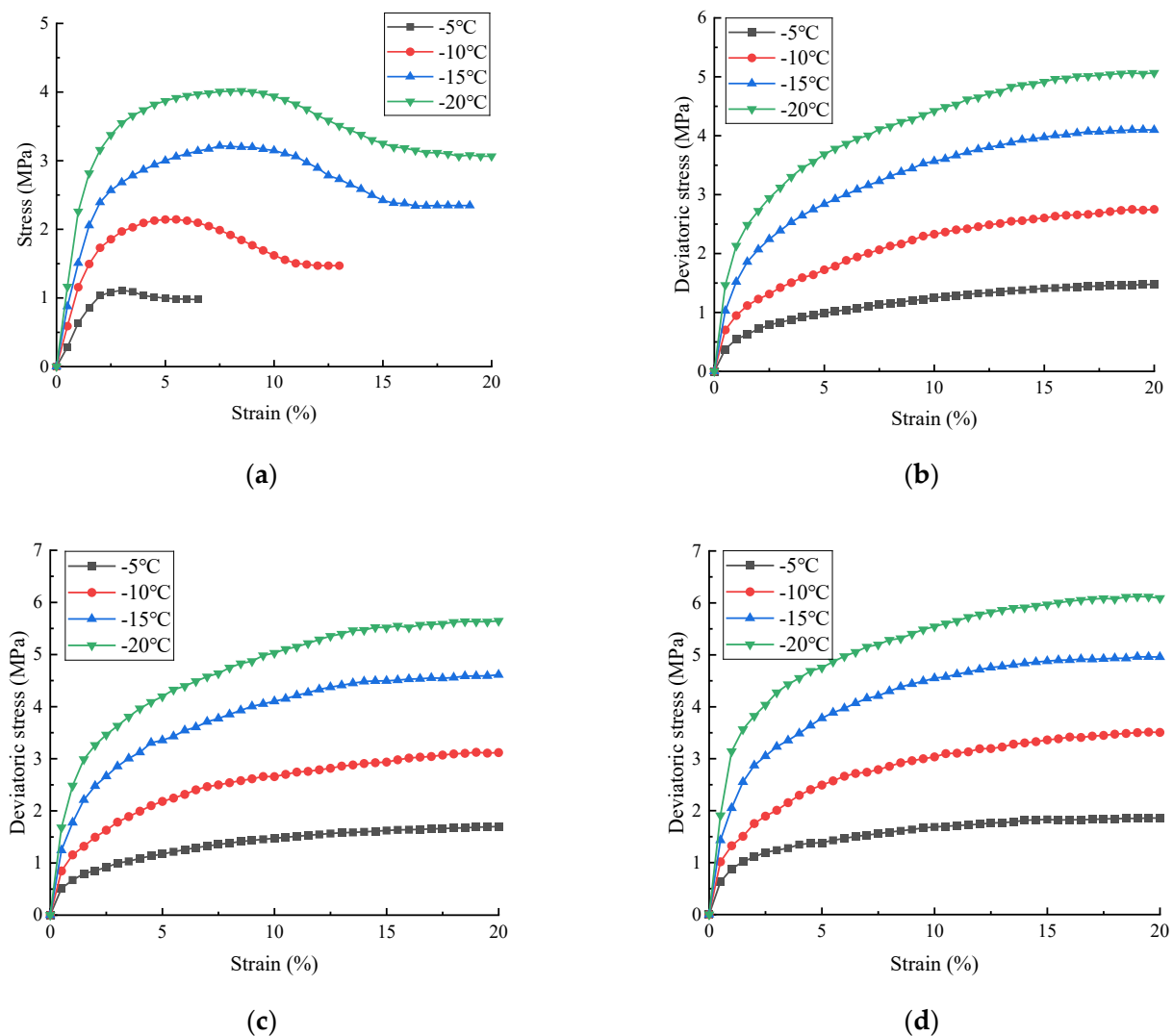


Figure 5. Stress–strain curves of frozen soil under different confining pressures. (a) Stress–strain curves under uniaxial compress. (b) Stress–strain curves under confining pressure of 0.5 MPa. (c) Stress–strain curves under confining pressure of 1 MPa. (d) Stress–strain curves under confining pressure of 1.5 MPa.

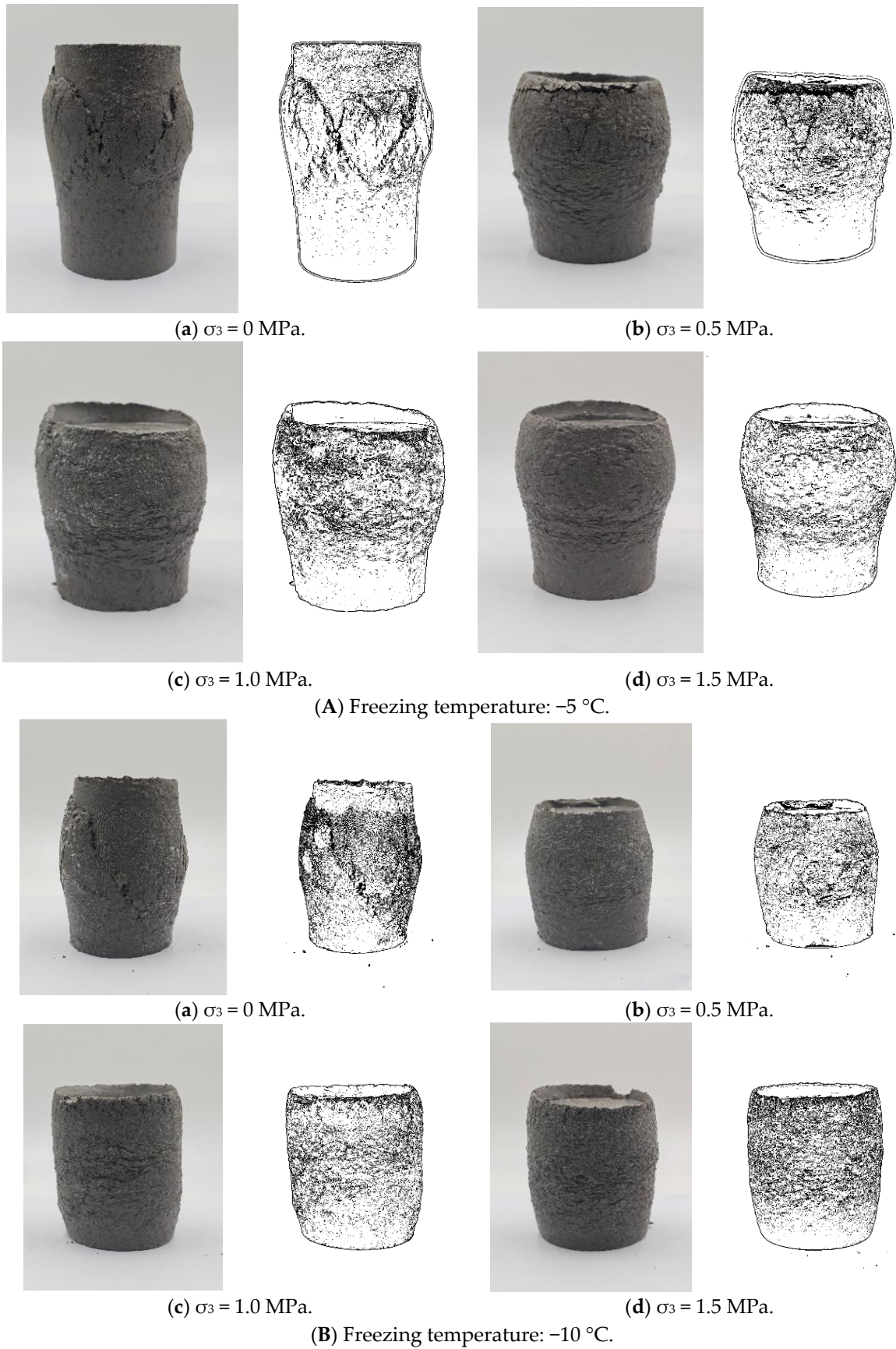


Figure 6. Cont.

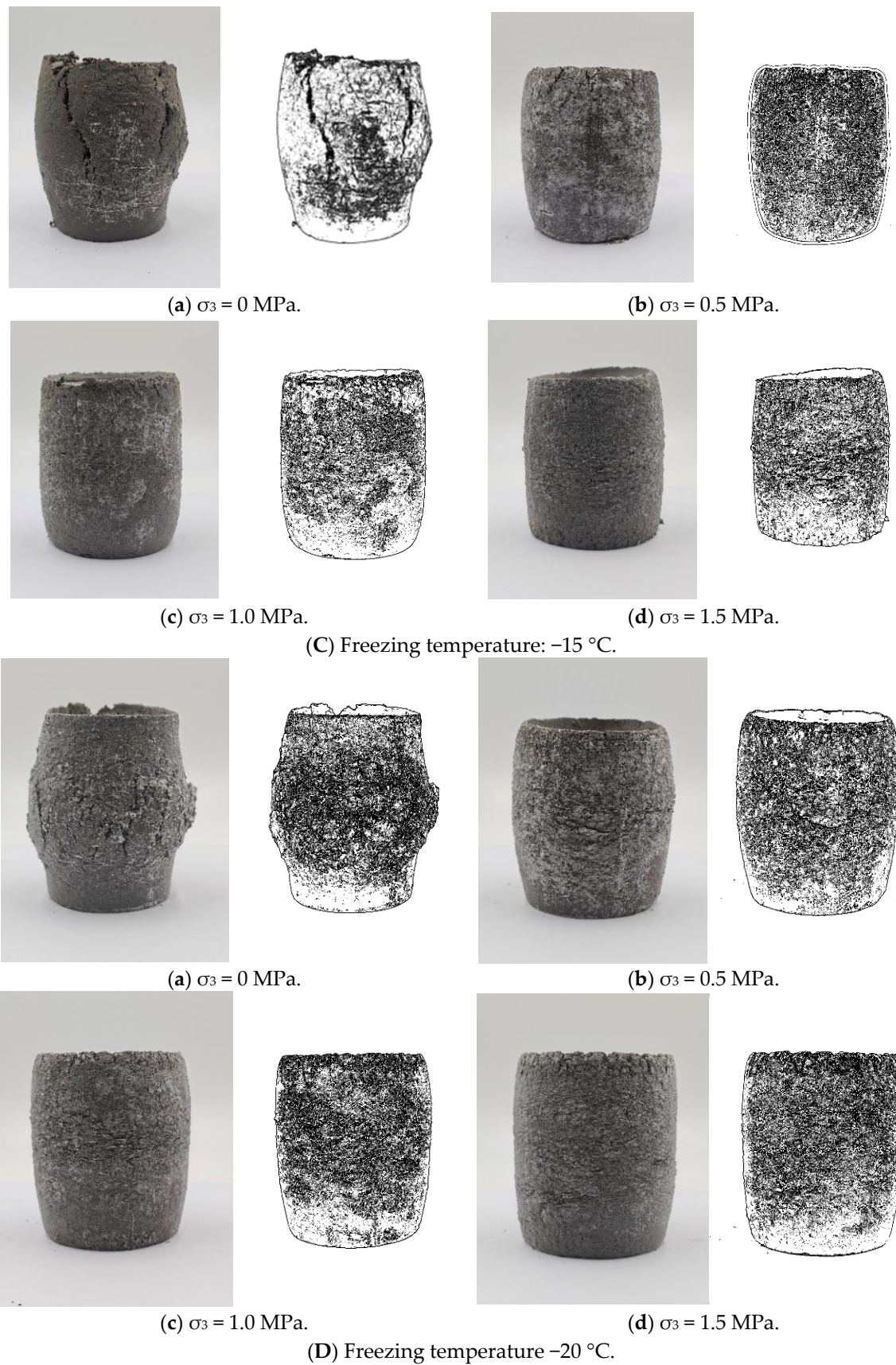


Figure 6. Failure patterns of frozen soil under different freezing temperatures and confining pressures.

The deformation process of frozen soil is a temperature-dependent mechanical process during which a phase transition from liquid to solid water occurs within the soil matrix.

However, not all water is converted to ice; even at $-60\text{ }^{\circ}\text{C}$, unfrozen water persists within the sample. At this stage, the soil matrix comprises four phases: soil skeleton, ice crystals, unfrozen water, and air [26]. Figure 7 illustrates the relationship between freezing temperature and compressive strength. Under constant confining pressure (0–1.5 MPa), the compressive strength of specimens increases with decreasing freezing temperature, exhibiting near-linear growth.

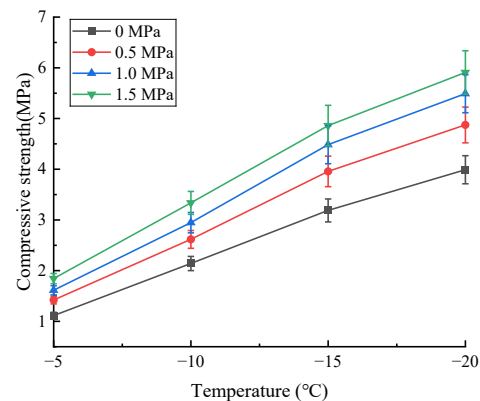


Figure 7. Temperature-dependent variation in compressive strength.

At $-5\text{ }^{\circ}\text{C}$, strength differences across varying confining pressures are minor; however, at $-20\text{ }^{\circ}\text{C}$, these differences significantly widen. This indicates that at lower temperatures, the effect of confining pressure on strength enhancement becomes more pronounced. Figure 8 illustrates the relationship between confining pressure and compressive strength. At a constant freezing temperature, the compressive strength of the specimens increases linearly with rising confining pressure. In frozen soil testing, the elastic modulus E is a crucial parameter reflecting the soil's resistance to elastic deformation. Following the relevant textbooks and literature, the slope of the secant line drawn through the initial linear segment of the stress–strain curve is used to represent the soil's elastic modulus. Figure 9 illustrates the variation in elastic modulus with freezing temperature. At $-5\text{ }^{\circ}\text{C}$, the elastic modulus ranges from 55 MPa to 130 MPa across different confining pressures. At $-20\text{ }^{\circ}\text{C}$, the elastic modulus ranges from 230 MPa to 385 MPa under varying confining pressures. Similar to compressive strength, the elastic modulus curves at $-20\text{ }^{\circ}\text{C}$ for different confining pressures are more dispersed compared to those at $-5\text{ }^{\circ}\text{C}$. This indicates that lower temperatures enhance the influence of confining pressure on soil stiffness. Figure 10 illustrates the relationship between confining pressure and elastic modulus. The curve increases with rising confining pressure, as lateral confinement progressively enhances the stiffness of the soil skeleton.

Figure 11 better illustrates the variation in specimen elastic modulus with freezing temperature under different confining pressure conditions. Freezing temperature exerts a decisive influence on the specimens. Elastic modulus significantly increases as freezing temperature decreases. The effect of confining pressure on elastic modulus is correlated with freezing temperature, exhibiting a clear dependency.

The relationship between freezing temperature and peak stress–strain as well as residual stress–strain under uniaxial compression is shown in Figure 12. Under uniaxial compression conditions, both the peak stress and residual stress of the specimens increase with decreasing freezing temperature, exhibiting a negative correlation. As the freezing temperature decreased from $-5\text{ }^{\circ}\text{C}$ to $-20\text{ }^{\circ}\text{C}$, the peak stress of the specimens increased from 1.11 MPa to 4.02 MPa, representing a 362% increase. Similarly, the residual stress rose from 0.98 MPa to 3.06 MPa, showing a 312% increase and exhibiting a comparable growth trend. As temperature decreases, the difference between peak stress and residual

stress gradually increases. This indicates that at lower temperatures, the specimens exhibit more pronounced failure characteristics, with a greater magnitude of strength loss after failure. The peak strain of the specimens increased approximately linearly with decreasing temperature. At -5°C , the peak strain was 3%. When the temperature dropped to -20°C , the peak strain reached 8.5%, representing a 5.5% increase. This indicates that under low-temperature uniaxial compression, the specimen can withstand greater deformation before failure due to the significant enhancement in soil strength. The residual strain exhibits a different trend compared to peak strain, increasing with decreasing temperature but at a gradually slowing rate. Overall, the peak stress of the specimens generally exceeds their residual stress, while the peak strain is generally lower than the residual strain.

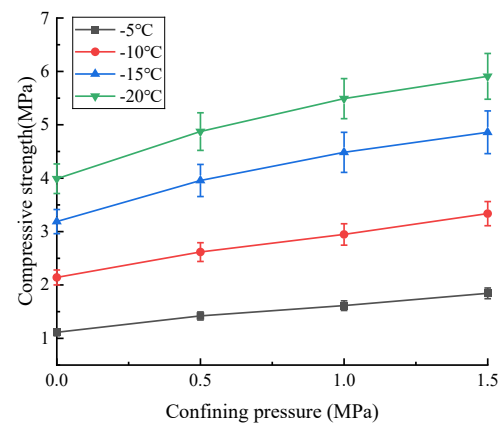


Figure 8. Variation in compressive strength with confining pressure.

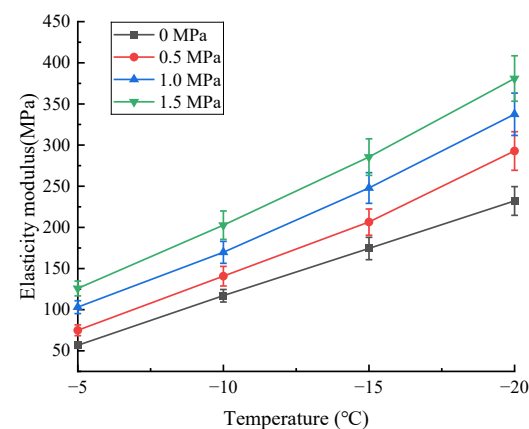


Figure 9. Temperature-dependent variation in elastic modulus.

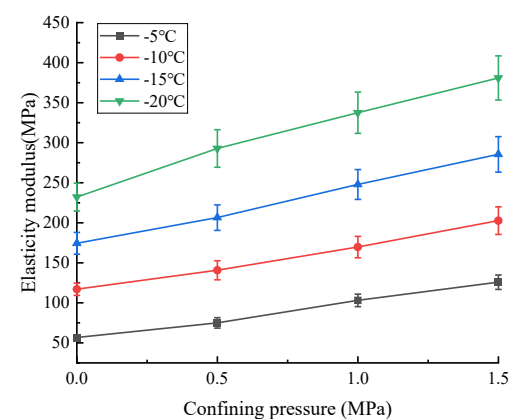


Figure 10. Variation in elastic modulus with confining pressure.

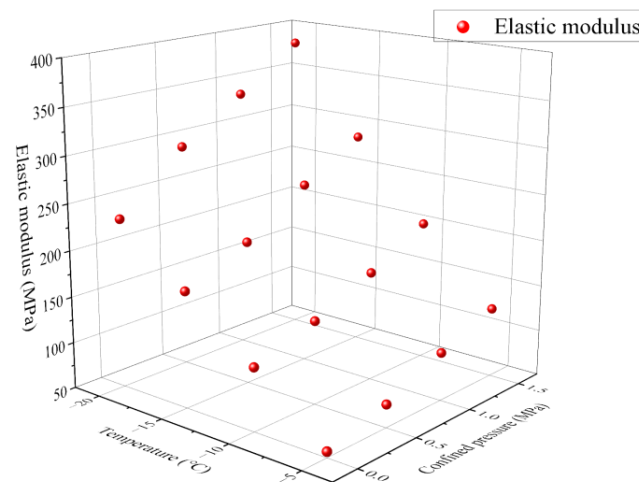


Figure 11. The relationship between temperature, confining pressure and elastic modulus.

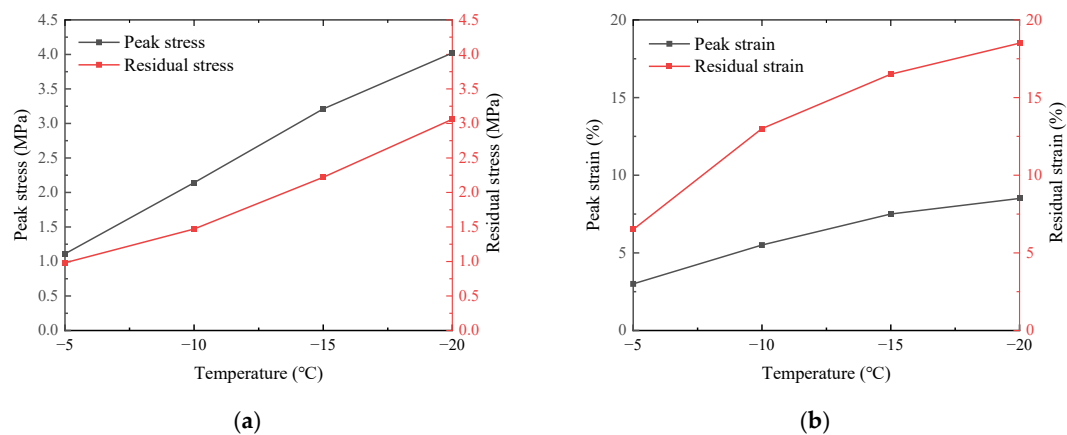


Figure 12. The relationship between freezing temperature and peak stress–strain as well as residual stress–strain under uniaxial compression. (a) The relationship between peak stress, residual stress and temperature. (b) The relationship between peak strain, residual strain and temperature.

4.4. Shear Strength Parameters of Specimens at Different Freezing Temperatures

Based on triaxial compression test results at different freezing temperatures, the shear strength parameters of frozen clay cohesion (c) and internal friction angle (φ) were determined by plotting the Mohr stress circle. The relationship curves between freezing temperature and internal friction angle/cohesion are shown in Figure 13.

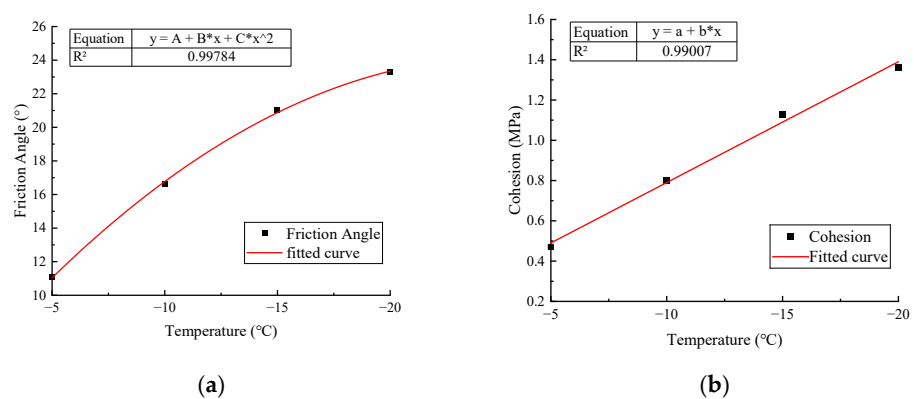


Figure 13. Stress–time–damage variable of cavern specimens with different joint angles. (a) The relationship between the friction angle and temperature. (b) The relationship between cohesion and temperature.

The results indicate that both the angle of internal friction and cohesion exhibit a positive correlation with decreasing freezing temperature. Notably, the angle of internal friction increases sharply within the range of -5°C to -15°C , rising from 11.09° to 21.04° . Specifically, a mere 5°C decrement between -5°C and -10°C yields a substantial increase of 5.51° in the friction angle. Conversely, this growth rate decelerates significantly beyond -15°C ; as the temperature drops from -15°C to -20°C , the friction angle increases by only 2.25° (from 21.04° to 23.29°), demonstrating a clear trend of diminishing sensitivity at lower temperatures. The relationship between the internal friction angle and freezing temperature follows an exponential function, which can be described by a polynomial parabolic function:

$$\varphi = A + Bx + Cx^2 \quad (1)$$

where A , B , and C are correlation coefficient. The curves for the internal friction angle at different freezing temperatures show good agreement, with a correlation coefficient of 0.997. The specific formula is $\varphi = 3.67 - 1.6358x - 0.0326x^2$.

The relationship curve between sample freezing temperature and cohesion shows that as the freezing temperature decreases from -5°C to -20°C , the sample's cohesion increases from 0.47 MPa to 1.36 MPa, representing an approximate 189% increase. Cohesion exhibits a pronounced linear growth trend with freezing temperature, which can be described by the linear function:

$$c = a + b * x \quad (2)$$

where a and b are the correlation coefficient. The cohesion curves at different freezing temperatures show good agreement, with a correlation coefficient of 0.990. The specific formula is: $c = 0.19 - 0.06 * x$.

5. Constitutive Model

The stress–strain curve of frozen coastal clay exhibits significant nonlinear characteristics under the coupled effects of freezing temperature and confining pressure. Extensive research indicates that the micro-element failure of frozen soil follows the Weibull random distribution [27–29].

$$P(F) = \frac{\beta}{F_0} \left(\frac{F}{F_0}\right)^{\beta-1} \exp\left[-\left(\frac{F}{F_0}\right)^{\beta}\right], (F > 0) \quad (3)$$

Under unconfined conditions, soil exhibits strain hardening followed by softening; under confined conditions, soil shows strain hardening without softening or peak strain. To uniformly describe this complex mechanical behavior, based on the Lemaitre strain equivalence hypothesis, the frozen soil specimen is assumed to consist of countless infinitesimal elements.

From a probabilistic perspective, this can be expressed as:

$$D = \frac{N_t}{N} \quad (4)$$

When axial stress reaches a certain level, the number of damaged elements is:

$$N_t(F) = \int_0^F NF(x)dx = N \left\{ 1 - \exp\left[-\left(\frac{F}{F_0}\right)^{\beta}\right] \right\} \quad (5)$$

Defining the damage variable D as the ratio of damaged elements to the total number, integrating the probability density function yields the cumulative damage distribution function:

$$D = 1 - \exp\left[-\left(\frac{\varepsilon_1}{F_0}\right)^\beta\right] \quad (6)$$

The undamaged portion can be regarded as the force borne by the soil skeleton, while the damaged portion is considered as particles or fragments that have already failed but still retain some bearing capacity under confining pressure. According to the parallel superposition principle, macroscopically, the deviatoric stress σ_{dev} equals the superposition of these two stresses:

$$\sigma_{dev} = \sigma_{dev}^{ud}(1 - D) + \sigma_{dev}^d D \quad (7)$$

For the undamaged portion, it is assumed to obey Hooke's law of linear elasticity. Under triaxial compression, the deviatoric stress exhibits a linear relationship with strain. Therefore, the undamaged portion can be expressed as:

$$\sigma_{dev}^{ud} = E\varepsilon_1 \quad (8)$$

For the damaged portion, once the soil is completely damaged, the soil skeleton loses all bearing capacity and transforms into loose particles or fragments. Passive constraint forces within the specimen arise from the coupling of Poisson's ratio effects and confining pressure. Therefore, the deviatoric stress in the damaged portion is:

$$\sigma_{dev}^d = \sigma_1^d - \sigma_3 = 2\nu\sigma_3 \quad (9)$$

Substituting Equations (8) and (9) into Equation (7) yields the damage constitutive equation:

$$\sigma_{dev} = E\varepsilon_1(1 - D) + 2\nu\sigma_3 D \quad (10)$$

Experimental data indicate that under varying freezing temperatures and confining pressures, specimen damage induces systematic variations in stress–strain curves. To enable the model to more accurately describe the stress–strain relationship, it is expressed as functions of parameters β , F_0 , E , temperature T , and confining pressure σ_3 :

$$\begin{aligned} E &= f(T, \sigma_3) \\ \beta &= g(T, \sigma_3) \\ F_0 &= h(T, \sigma_3) \end{aligned} \quad (11)$$

To determine the parameters β and F_0 , the code employs the Levenberg–Marquardt algorithm to minimize the sum of squares of differences between experimental stress and stress predicted by the constitutive model, thereby solving for optimal parameters.

$$\min_{\beta, F_0} \left[\sigma_{exp,i} - \left(E \cdot \varepsilon_i \cdot e^{-(\varepsilon_i/F_0)^\beta} + 2\nu\sigma_3(1 - e^{-(\varepsilon_i/F_0)^\beta}) \right) \right]^2 \quad (12)$$

Using the elastic modulus as a known quantity, the calculated parameters β and F_0 are shown in Table 3.

As seen in the table, the values of R^2 are all above 0.9, indicating overall good fitting performance. Substituting Equations (6) and (13) into Equation (10) yields the final damage constitutive equation relating temperature and confining pressure:

$$\sigma_p = E_{f(T, \sigma_3)} \varepsilon \exp\left[-\left(\frac{\varepsilon}{F_{0h(T, \sigma_3)}}\right)^{\beta_{g(T, \sigma_3)}}\right] + 2\nu\sigma_3[1 - \exp\left(-\left(\frac{\varepsilon_1}{F_{0h(T, \sigma_3)}}\right)^{\beta_{g(T, \sigma_3)}}\right)] \quad (13)$$

The parameters obtained earlier were used to fit the freezing temperature (T) and confining pressure (σ_3) to the parameters (E , β , and F_0) via rational and polynomial functions, respectively, as shown in Figure 14.

Table 3. Statistical parameters of coastal clay.

Sample ID	Parameter β	Parameter F_0	R^2
DW-1 (−5 °C, 0 MPa)	1.5254	0.0509	0.9308
DW-2 (−10 °C, 0 MPa)	0.9593	0.0501	0.9634
DW-3 (−15 °C, 0 MPa)	0.734	0.0474	0.96
DW-4 (−20 °C, 0 MPa)	0.6962	0.0443	0.913
DW-5 (−5 °C, 0.5 MPa)	0.326	0.0117	0.9954
DW-6 (−10 °C, 0.5 MPa)	0.3395	0.0147	0.9922
DW-7 (−15 °C, 0.5 MPa)	0.4061	0.0235	0.9826
DW-8 (−20 °C, 0.5 MPa)	0.4035	0.0201	0.9686
DW-9 (−5 °C, 1.0 MPa)	0.2494	0.0029	0.9972
DW-10 (−10 °C, 1.0 MPa)	0.3409	0.0123	0.9967
DW-11 (−15 °C, 1.0 MPa)	0.4004	0.0198	0.9873
DW-12 (−20 °C, 1.0 MPa)	0.4015	0.0184	0.9687
DW-13 (−5 °C, 1.5 MPa)	0.226	0.0012	0.9885
DW-14 (−10 °C, 1.5 MPa)	0.3075	0.0077	0.9992
DW-15 (−15 °C, 1.5 MPa)	0.395	0.017	0.9896
DW-16 (−20 °C, 1.5 MPa)	0.4101	0.0176	0.9512

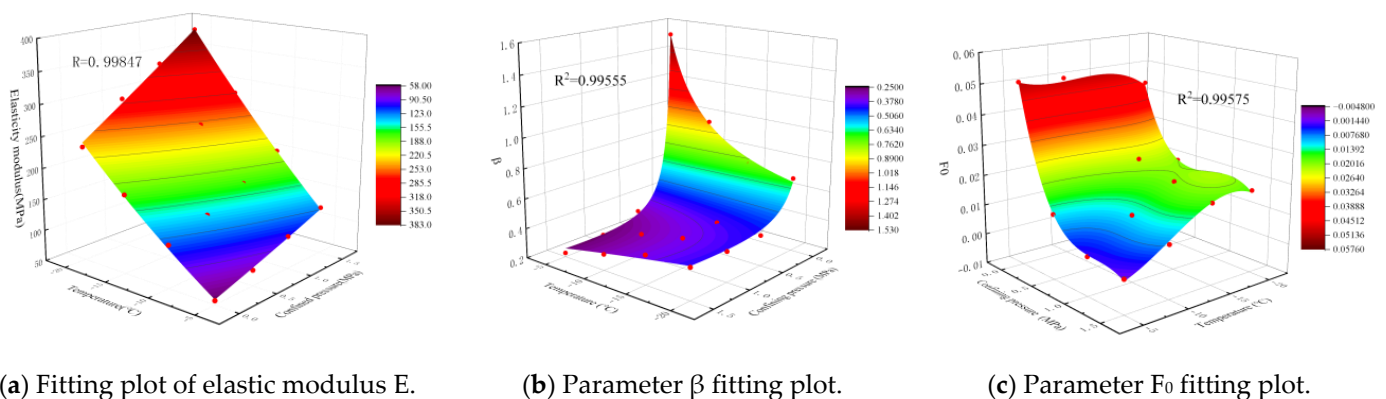


Figure 14. Relationship between freezing temperature and confining pressure with parameters E , β , and F_0 .

Using Origin software to perform nonlinear surface fitting on the points corresponding to the parameters in the three-dimensional coordinate system yields the general expressions for the surfaces of:

$$E = p_{00} + p_{10}T + p_{01}\sigma_3 + p_{20}T^2 + p_{02}\sigma_3^2 + p_{11}T\sigma_3 \quad (14)$$

$$\beta = \frac{p_{00} + p_{10}T + p_{01}\sigma_3 + p_{20}T^2 + p_{02}\sigma_3^2 + p_{11}T\sigma_3}{1 + q_{10}T + q_{01}\sigma_3} \quad (15)$$

$$F_0 = p_{00} + p_{10}T + p_{01}\sigma_3 + p_{20}T^2 + p_{11}T\sigma_3 + p_{02}\sigma_3^2 + p_{30}T^3 + p_{21}T^2\sigma_3 + p_{12}T\sigma_3^2 + p_{03}\sigma_3^3 \quad (16)$$

The detailed values and R^2 are shown in Table 4:

Table 4. Surface fitting function coefficients.

E		β		F0	
$R^2 = 0.99847$		$R^2 = 0.99555$		$R^2 = 0.99575$	
p00	11.7695	p00	-1.36×10^{44}	p00	0.06376
p10	-8.53396	p10	7.68×10^{42}	p10	0.00428
p01	23.06775	p01	-8.36×10^{43}	p01	-0.14089
p20	0.14372	p20	-4.92×10^{40}	p20	3.97×10^{-4}
p02	2.2475	p02	-1.01×10^{44}	p11	-0.00337
p11	-3.41676	p11	1.90×10^{43}	p02	0.11575
-	-	q10	2.30×10^{43}	p30	1.17×10^{-5}
-	-	q01	-1.66×10^{45}	p21	-3.04×10^{-5}
-	-	-	-	p12	0.00104
-	-	-	-	p03	-0.03313

6. PINN-Based Constitutive Model Parameter Inversion and Prediction Validation

Although the damage constitutive model established earlier can adequately describe the stress–strain relationship of frozen marine clay, the model parameters E , β , and F_0 exhibit complex nonlinear coupling with freezing temperature and confining pressure. Traditional fitting methods rely on selected functional forms, involve complex computations with numerous parameters, and are difficult to calibrate. Therefore, based on the established damage constitutive theory for frozen coastal clay, combined with the Physically Informed Neural Network (PINN) framework, the model is trained using experimental data to achieve inversion of constitutive model parameters and prediction for untrained experimental data. The accuracy differences between the PINN model and traditional empirical models in parameter inversion and stress–strain curve prediction are compared and analyzed.

6.1. Model Training and Parameter Inversion

The model uses the freezing temperature (T), confining pressure (σ_3), and strain (ε) from the aforementioned 16 test sets as inputs, incorporating stress and elastic modulus into the loss function. The hidden layer consists of three layers, each containing 40 neurons. The output layer generates axial deviatoric stress. The hyperbolic tangent (Tanh) activation function was selected for its suitability to continuously varying stress–strain curves, with its smooth nonlinear characteristics aligning with the damage evolution patterns of frozen soil.

$$\tanh(x) = \frac{e^x - e^{-x}}{e^x + e^{-x}} \quad (17)$$

To ensure the model accurately predicts soil mechanical behavior with limited data, Equation (10) is embedded into the forward propagation. To prevent overfitting, 10% dropout is incorporated alongside L2 regularization, gradient clipping, and learning rate decay to enhance model generalization. Finally, a physically guided neural network was constructed using a purely data-driven neural network to fit the constitutive model parameters, followed by stress calculations via the constitutive model. Training parameters are shown in Table 5.

The loss function is defined as the sum of stress loss and elastic modulus loss with assigned weight relationships:

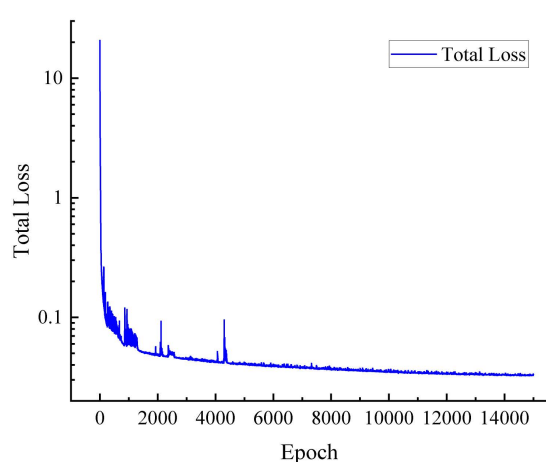
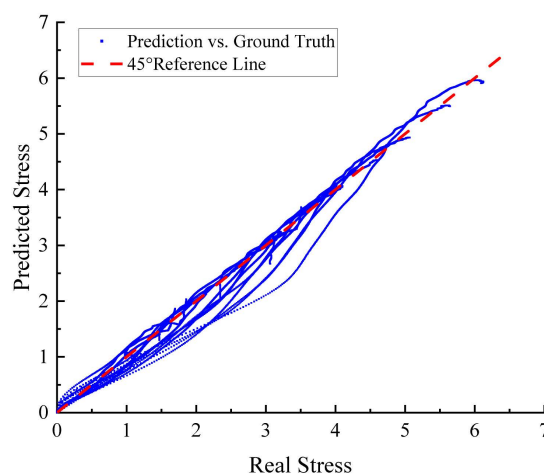
$$L = L_{stress} + \lambda L_E = \frac{1}{N_s} \sum_{i=1}^{N_s} (\sigma_{PINN}^i - \sigma_{exp}^i)^2 + \lambda \frac{1}{M} \sum_{j=1}^M (E_{PINN}^j - E_{exp}^j)^2 \quad (18)$$

Table 5. PINN model training parameters and configuration.

Parameter	Optimal Setting Value
Activation Function	tanh
Loss Function	MSE
Optimizer	Adam
Regularization	L2 Regularization, Dropout
Number of Hidden Layers	3
Number of Neurons per Layer	40
Initial Learning Rate	2×10^{-3}
Number of Iterations	14,000
Number of Training Samples	17,910
Number of Test Samples	1800

6.2. Training Process and Convergence Analysis

Figure 15 illustrates the model's performance during training. At the initial stage, the loss value reached as high as 20.5888, indicating the network had not yet learned soil features. As iterations progressed, the loss rapidly decreased to 0.0603 by the 1000th epoch, subsequently entering a stable fine-tuning phase. By the end of 14,000 iterations, the loss converged to 0.0322 with a model accuracy of 0.9900, demonstrating overall good convergence. This indicates that the PINN model successfully captured the nonlinear mapping relationship of frozen coastal clay under varying temperatures and confining pressures, confirming that the damage constitutive model effectively describes the mechanical properties of frozen coastal clay.

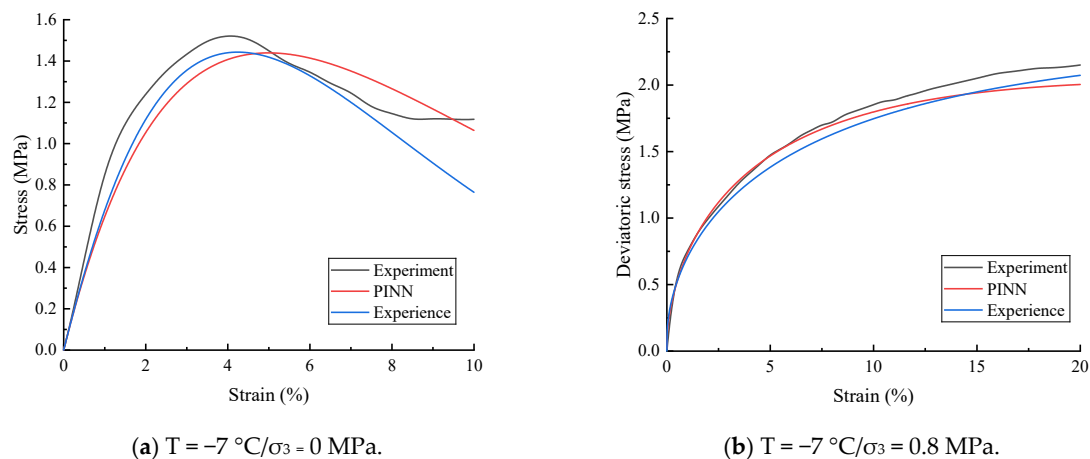
**(a)** Training loss.**(b)** Regression analysis.**Figure 15.** Model performance evaluation.

6.3. Verification of Generalization Capability

To further validate the generalization capability of the PINN model under other test conditions, two untrained test conditions ($T = -7\text{ }^{\circ}\text{C}/0\text{ MPa}$ confining pressure and $T = -7\text{ }^{\circ}\text{C}/0.8\text{ MPa}$ confining pressure) were selected for evaluation. The results are compared with those from a traditional empirical model, as shown in Table 6. The stress–strain curves are depicted in Figure 16.

Table 6. Comparison of prediction performance between PINN Model and theoretical models under test conditions.

Test Condition	Model Type	E (MPa)	(%)	Curve R ²
T = −7 °C, $\sigma_3 = 0$ MPa	Test Measurement Value	87.32	-	-
	Empirical Model	78.5	10.10	0.8260
	PINN Model	79.5	8.96	0.8513
T = −7 °C, $\sigma_3 = 0.8$ MPa	Test Measurement Value	109.17	-	-
	Empirical Model	117.6	7.72	0.9607
	PINN Model	114.4	4.79	0.9689

**Figure 16.** Comparison of model and measured stress–strain curves under $-7\text{ }^{\circ}\text{C}$ test conditions.

From the table, we can observe that at $-7\text{ }^{\circ}\text{C}$, the elastic modulus predicted by the PINN model is very close to that of the empirical model. However, when compared with experimental values, the empirical model exhibits an elastic modulus error of 10.10% under 0 MPa confining pressure, while the PINN model shows an error of 8.96%. At 0.8 MPa confining pressure, the empirical model's elastic modulus error is 7.72%, while the PINN model's error is 4.79%. Regardless of confining pressure presence, the PINN model demonstrates higher prediction accuracy than the empirical model, with a particularly pronounced advantage under confining pressure conditions. Regarding curve fitting, the curves of the empirical and PINN models showed differences under uniaxial compression but remained within acceptable error ranges. Under triaxial compression, they are very close, fully validating the damage constitutive equation. The measured curve results show that under uniaxial compression, the PINN curve has an R^2 of 85.13%, while the empirical model curve has an R^2 of 82.60%, which is 2.53% higher than the theoretical model. Under triaxial compression, the PINN curve achieved an R^2 of 96.89%, while the empirical model reached 96.07%, indicating the PINN model's generalization capability is comparable to or even superior to the empirical model.

7. Discussion

7.1. Microstructural Interpretation of Macroscopic Mechanical Behavior

Zhang M et al. [30] and Jiang N et al. [31] stated in their study that the macroscopic hardening response and transformation of failure modes in frozen clay are fundamentally controlled by pore water phase transition as well as the evolutionary characteristics of ice–soil interfacial interactions. Such microstructural variations further dominate the ultimate failure patterns of frozen soil specimens. However, no microscale testing and microstructural characterization were implemented for the tested specimens in the present

work. Relevant microscopic experimental investigations will be conducted in follow-up research.

7.2. Comparison Between the PINN Model and the Traditional Model

Compared with Burgers, Lemaitre and dual-medium models, the PINN framework presents prominent improvements in computational efficiency and prediction accuracy.

For computational efficiency, PINN avoids tedious manual empirical function screening and multi-stage regression. Physical constitutive relations are embedded into its loss function, enabling one-step gradient optimization to invert multiple coupled parameters (scale, shape, elastic modulus, etc.) simultaneously and greatly lowering computation costs.

In terms of prediction performance, physical constraints serve as effective regularizers to mitigate overfitting against sparse training datasets and unphysical predictions. Consequently, the PINN model well complies with fundamental mechanical laws, delivers stronger generalization and extrapolation performance on unseen samples, and balances data-driven flexibility and physical interpretability.

7.3. Practical Implications for Artificial Ground Freezing in Coastal Tunneling

While the laboratory findings and the PINN model provide valuable insights, their application to real-world artificial ground freezing (AGF) in coastal tunneling requires careful consideration of energy, stability, and scale effects. When extrapolating these research findings to full-scale field projects, the limitations including sample disturbance, scale effects, and temperature gradient discrepancies must be fully recognized.

8. Conclusions

This study systematically investigated the mechanical properties of frozen coastal clay under different freezing temperatures ($-5\text{ }^{\circ}\text{C}$ to $-20\text{ }^{\circ}\text{C}$) and confining pressures (0–1.5 MPa) through triaxial compression tests. A temperature- and confining-pressure-dependent damage constitutive model was established based on the Weibull distribution and Lemaitre's strain equivalence hypothesis. Furthermore, a Physics-Informed Neural Network (PINN) framework was introduced to invert model parameters and predict untrained conditions. The main conclusions are as follows:

1. Confining pressure fundamentally alters the stress–strain behavior. Under uniaxial compression, frozen coastal clay exhibits strain softening with a clear peak stress. Under triaxial compression (0.5–1.5 MPa), the stress–strain curve transforms into a strain-hardening type without a distinct peak, indicating pronounced plastic flow characteristics.
2. Freezing temperature is the dominant factor controlling strength and deformation. Compressive strength, elastic modulus, cohesion, and internal friction angle all increase as temperature decreases. Cohesion increases linearly with decreasing temperature, while the internal friction angle follows a parabolic relationship.
3. Failure modes depend on the combined effect of confining pressure and temperature. Under uniaxial loading, shear failure dominates, with macroscopic through-cracks. Under triaxial loading, failure changes to “barrel-shaped” shear-bulging without through-cracks. Lower temperatures enhance ice cementation and restrain lateral deformation.
4. The proposed damage constitutive model uniformly describes both strain hardening and strain softening. By incorporating the Weibull random distribution and considering the parallel contribution of damaged and undamaged soil elements, the model captures the full stress–strain process. Key model parameters (scale parameter F_0 , shape parameter m , and damaged-portion coefficient c) are expressed as func-

tions of freezing temperature and confining pressure, yielding good agreement with experimental data ($R^2 > 0.9$).

5. The PINN-based approach outperforms traditional empirical models in parameter inversion and prediction. With the constitutive equation embedded as a physical constraint, the PINN model achieves high prediction accuracy under untrained conditions (e.g., $-7\text{ }^{\circ}\text{C}$ under 0 MPa and 0.8 MPa confining pressures). The coefficient of determination (R^2) exceeds 0.85, and in triaxial compression reaches 0.97, demonstrating strong generalization capability and practical value for engineering applications in coastal frozen ground construction.

Author Contributions: Conceptualization, Q.S.; methodology, R.C.; software, R.H.; formal analysis, N.W.; investigation, J.Z. (Jialong Zeng); resources, J.Z. (Jiajia Zeng); data curation, N.W.; writing—original draft preparation, R.C.; writing—review and editing, N.W.; supervision, D.L.; project administration, Q.S.; funding acquisition D.L. All authors have read and agreed to the published version of the manuscript.

Funding: The research was supported by the National Natural Science Foundation of China (Grant No. 42577544 and 52209123), Xingliao Talent Program of Liaoning Province (Grant No. XLYC2402027), Key Interdisciplinary Project of Dalian University (Grant No. DLUXK-2024-ZD-003 and DLUXK-2025-FX-005), Liaoning Provincial Science and Technology Plan Joint Project (Natural Science Foundation-General Program) (Grant No. 2024-MSLH-013), Research Initiation Project for Excellent Talents of Dalian University Project (Grant No. 2025-XRCZK-03) and Research Center for Geotechnical and Structural Engineering Technology of Liaoning Province and Dalian University, Dalian, 116622 (Grant No. DLSZD2025[010]).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The datasets generated during and/or analyzed during the current study are available from the corresponding author upon reasonable request.

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

Abbreviations

$P(F)$	Probability density
F	Random variable
β	Shape parameter
F_0	Scale parameter
D	Damage variable
N_t	Damaged elements
N	Total number of elements
σ_{dev}	Deviatoric stress
σ_{dev}^{ud}	Deviatoric stress of the undamaged portion
σ_{dev}^d	Deviatoric stress of the damaged portion
E	Elastic modulus
ε_1	Strain
T	Temperature
σ_3	Confined pressure
L	Total loss function
L_{stress}	Loss for the stress component
λ	Weighting coefficient
L_E	Loss for the elastic modulus component
σ_{PINN}	Predicted shear stress
σ_{exp}	Experimentally measured shear stress

E_{PINN}^j	Predicted elastic modulus
E_{exp}^j	Measured elastic modulus
N_s	Total number of shear stress data samples
M	Total number of elastic modulus data points

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