



Research paper

Disturbance effect on interface shear creep behavior of soil anchors

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ARTICLE INFO

Keywords:

Grouted soil anchor
Anchor-soil interface
Disturbance-creep model
Disturbance energy ratio
Disturbed state concept

ABSTRACT

Shear creep tests on the anchor-soil interface of grouted soil anchors were carried out under disturbance conditions. A disturbance energy ratio index was proposed to quantify the degree of shear disturbance on the interface, and the combined effect of disturbance and shear stress on the creep behavior was investigated. The disturbance-creep response exhibits four stages: primary creep, secondary creep, disturbance-induced jump, and post-disturbance creep. The disturbance-induced jump is irreversible, and its magnitude increases with shear stress level. An equivalent creep curve method was developed by minimizing the area deviation before and after the jump. A disturbance-creep model incorporating disturbance energy ratio and shear stress level was established using the Disturbed State Concept (DSC). The model showed strong training and prediction performance, with all parameters conforming to a unified correlation. The initial shear modulus decays sharply under low-stress and low-disturbance conditions, while the viscosity coefficient decreases significantly with increasing disturbance energy ratio, highlighting the need for field disturbance monitoring to assess residual anchor service life. This study provides a new tool for quantifying the disturbance-creep behavior of anchor-soil interfaces and offers valuable insights for the resilience design of coastal highway slope anchorage systems under environmental disturbances.

1. Introduction

Grouted anchorage systems have been employed as a core technical means in geotechnical engineering applications such as tunnel support, slope reinforcement, and deep excavation stabilization due to their excellent load transfer capacity and stability (Ma et al., 2013; Chen et al., 2020; Dai et al., 2004). In major infrastructure projects such as high-speed railway tunnels and high-steep slopes in mountainous areas, the service life of anchorage structures is often required to last for several decades or even longer. As a result, strict requirements are placed on the long-term performance of anchors (Zhao et al., 2020; Yang et al., 2025a, b). The interface that is formed between the grout body and the surrounding geomaterial is known to be a critical component governing the stress transfer of anchors. The creep behavior of this interface is directly linked to the long-term performance of the anchorage system (Chen et al., 2021).

Coastal highway slopes are exposed to complex service conditions, posing a dual challenge to the long-term stability of the anchor-soil

interface. Geologically, these slopes are often composed of weathered rock masses or poorly consolidated sediments, which are characterized by low strength and high sensitivity to environmental changes (Rahardjo et al., 2013). In terms of loading conditions, coastal highway slopes are inevitably subjected to multiple environmental disturbances during their service life, including traffic-induced vibrations, impacts from rockfalls, excavation unloading, and rainfall infiltration. As illustrated in Fig. 1. Among these, traffic-induced vibrations, particularly those generated by high-speed railway systems, have been extensively studied in the context of vehicle-track-bridge coupling under seismic conditions (Lai et al., 2025; Peng et al., 2025). The combined effects of the above disturbances and unfavorable geological conditions can disrupt the existing long-term mechanical equilibrium of the anchor-soil interface, inducing accelerated creep and strength degradation, and ultimately leading to anchorage failure. For example, the Xiaomojiu landslide, located in the upper reaches of the Jinsha River on the Qinghai-Tibet Plateau, experienced long-term creep deformation of a large deep-seated deposit landslide due to the frequent

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Received 29 July 2026; Received in revised form 23 September 2026; Accepted 2 October 2026

Available online 10 October 2026

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disturbances of long-term river erosion, short-term extreme flood events, and rainfall-induced surface erosion (Yang et al., 2025a, b). The significant influence of environmental disturbances on the long-term performance of anchor-soil interfaces is clearly shown by this case.

To date, the short-term mechanical behavior of soil anchor interfaces has been investigated by many studies. The effects of different anchor configuration factors—such as grout strength, anchor surface morphology, soil/rock type, and tendon material—on the interface shear strength characteristics have been mainly focused on in these investigations (Xu et al., 2018; Yu et al., 2022, 2025; Teymen and Kılıç, 2018; Zhang et al., 2022, 2026). However, the long-term performance of anchor-soil interfaces has been studied by only a limited number of investigations. In these few studies, the influence of soil physical state parameters, such as moisture content and dry density, on the long-term mechanical response has been examined (Zhang et al., 2020; Zhu et al., 2023). The effects of environmental disturbances encountered in field conditions have been largely overlooked, even though such disturbances are common.

In the studies that have addressed the long-term bearing characteristics of materials under environmental disturbances, the focus has mainly been placed on single geomaterial media. Physical model tests conducted under disturbance conditions (such as excavation and vibration) have been used to reveal the influence mechanisms of disturbance parameters (e.g., intensity and frequency) on the long-term deformation of rock and soil masses (Chen et al., 2025a, 2025b; Zhao et al., 2024, 2025; Zha et al., 2021; Niu et al., 2024). However, the anchor-soil interface has not been addressed in these investigations. This interface constitutes the critical load-transfer component of an anchor. Therefore, it is still not understood how environmental disturbances trigger creep at the anchor-soil interface or how these disturbances are linked to the loss of the interface's long-term bearing capacity. As a result, the evolution of the long-term mechanical performance of anchor-soil interfaces under disturbance conditions remains largely unknown.

Fairly mature theoretical models have been established to describe the creep behavior of single geomaterials (Zhao et al., 2017; Liao et al., 2017). However, the anchor-soil interface is a composite system. Under environmental disturbances, significant nonlinear time-dependent characteristics are often exhibited by this interface. Sudden changes in creep rate, premature onset of tertiary creep, and early interface failure are included in this complex response. The theoretical modeling of such complex creep behavior often requires the application of the Disturbed State Concept (DSC) theory (Desai and Ma, 1992; 2001; Frantziskonis and Desai, 1987). A single system that can include the complex mechanical behaviors of various materials is provided by DSC theory. The appropriate limit state models and disturbance function forms can be flexibly selected based on specific material characteristics. The capability of DSC to characterize nonlinear, discontinuous, time-dependent, and disturbance-sensitive responses has already been confirmed in applications such as long-term tunnel settlement analysis and rock creep modeling (Zhu et al., 2014; Huang et al., 2020a,

2020b). However, the application of DSC theory to characterize the creep behavior of soil anchor interfaces subjected to external disturbances has not been reported in the literature.

In this study, a series of tests was carried out to investigate the creep characteristics of anchor-soil interfaces under varying levels of disturbance energy. From these tests, creep curves corresponding to different combinations of disturbance energy and shear stress were obtained. Then, DSC theory was introduced to establish a disturbance-creep model for the anchor-soil interface. Through model parameter identification, the evolution of interface creep behavior under disturbance conditions was revealed. The findings of this research are expected to provide an important theoretical reference for the prediction of long-term service performance and the prevention of hazards related to soil-anchored coastal highway slopes.

2. Disturbance-creep tests on anchor-soil interface

2.1. Experiment materials

The preparation of grouted anchor specimens primarily involved three types of materials: soil, cement mortar, and reinforcement bar. The soil samples were collected from a reserved land parcel of Hunan City University in Yiyang, Hunan, China. The collected soils were oven-dried, crushed, and passed through a 5 mm sieve for later use. Fig. 2 shows the soil particle-size distribution curve. The cement mortar was prepared using ordinary portland cement of grade 42.5, with a water-cement ratio of 0.45. The reinforcement bar was a hot-rolled ribbed bar (HRB400) with a length of 23 cm and a nominal diameter of 10 mm. One end of the rebar was equipped with a 3 cm long screw thread to facilitate the application of pullout load on the anchor. The material parameters of the soil, cement mortar, and rebar are described in Table 1.

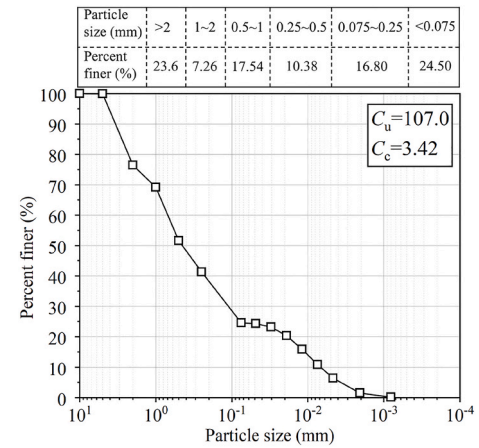


Fig. 2. Particle-size distribution of the test soil.

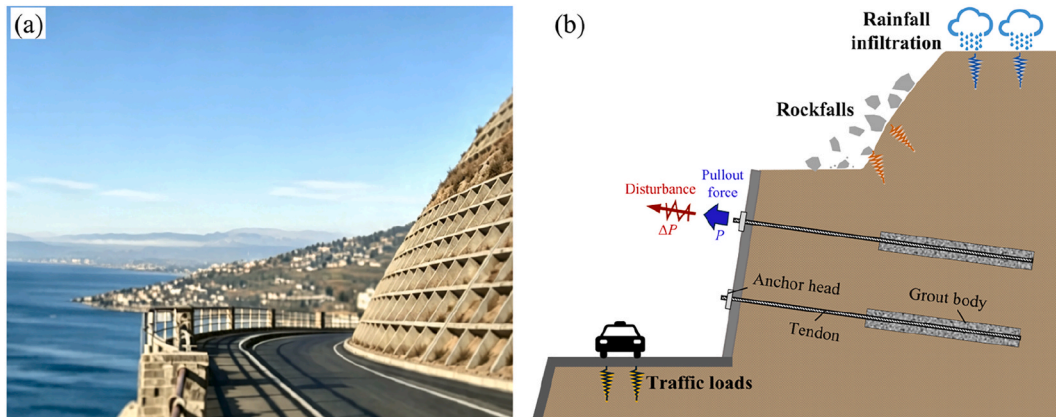


Fig. 1. Anchored coastal highway slope under environmental disturbances: (a) site photograph; (b) schematic diagram.

2.2. Anchor specimens and test setup

Drawing upon the “element method” concept employed in the load transfer analysis of anchors (Chen et al., 2015, 2019; Zhang et al., 2024), the grouted segment of an anchor was discretized into a number of elemental sections, and one such elemental section was selected for the preparation of the element-scale anchor specimen. This testing methodology has been applied and validated in previous studies conducted by the authors (Zhang et al., 2020; Zhu et al., 2023; Cai et al., 2024). The dimensions of the element-scale anchor specimen are illustrated in Fig. 3.

It should be noted that, in this test, the ratio of the bond length to the diameter of the grout hole ($L/d = 1.67 < 4$) satisfies the condition required for the uniform distribution of anchor-soil interface shear stress along the element anchor (Benmokrane et al., 1995; Ma et al., 2013). Furthermore, the ratio of the specimen diameter to the grout hole diameter ($L/d = 6.25 > 5$) ensures that the influence of boundary effects can be neglected (Chen and Zhu, 2024).

The tests were carried out using a specially developed creep pullout testing setup for element-scale anchors under disturbance conditions, which was designed by the authors' research group (Zhang et al., 2020; Gao et al., 2018). This setup is primarily composed of a loading system and a measurement system, as shown in Fig. 4. In the loading system, a combination of dead weights and a pulley assembly was adopted to achieve the stepwise application of pullout force. A disturbance in the pullout force was induced at a given load level by allowing a disturbing weight of a specified mass to fall freely from a predetermined height, impacting the dead weight corresponding to the current load level. It should be clarified that, in practical engineering, anchor systems are subjected to various types of environmental disturbances. The impact method using dead weights employed in this test was primarily designed to simulate the disturbance of the locked-in prestress of anchors induced by events such as rockfalls and vehicle collisions. During the test, the creep displacement and the pullout load were monitored in real time by a displacement gauge and a load cell, respectively.

2.3. Test design

2.3.1. Definition of disturbance energy ratio

To quantitatively characterize the extent to which environmental disturbances affect the creep behavior of the anchor-soil interface, an index termed the “Disturbance Energy Ratio” (R_{de}) was introduced in this study. This index was defined as the ratio of the cumulative disturbance energy (E_d) acting on the anchor-soil interface to the failure energy (E_f) required for the failure of the anchor specimen under ultimate conditions, which can be expressed as:

Table 1
Properties of materials used in element anchor specimens.

Properties/Description	Values/Description
<i>Soil</i>	
Specific gravity, G_s	2.68
Liquid limit, w_l (%)	44.3
Plastic limit, w_p (%)	25.2
Maximum dry density, ρ_{dmax} (g/cm ³)	1.68
Optimum water content, w_{opt} (%)	17.0
Uniformity coefficient, C_u	107.0
Coefficient of curvature, C_c	3.42
Cohesion, c (kPa)	8.34
Internal friction angle, ϕ (°)	9.29
<i>Cement mortar</i>	
Cement type	P.O 42.5
Water/cement ratio (%)	45
Grouting method	Gravity
Diameter of grouting hole, d (mm)	48
<i>Reinforcement bar</i>	
Tendon type	HRB 400
Diameter, d_b (mm)	10
Tensile strength, f_t (MPa)	460
Elastic modulus, E_b (GPa)	200
Yield strength, f_y (MPa)	380

$$R_{de} = \frac{E_d}{E_f} \quad (1)$$

For the present test, the specific definition of the disturbance energy ratio is illustrated in Fig. 5. The disturbance energy E_d was determined jointly by the times of impacts of the disturbing body (n), the mass of the disturbing body (m), and the drop height (Δh), such that $E_d = mg \cdot n \cdot \Delta h$. This represents the theoretical input energy estimated from the gravitational potential energy. Since the loading apparatus has high rigidity and the impact duration is short, the energy losses caused by weight rebound and local contact deformation are relatively small and were therefore not considered in this study. Failure energy E_f is defined as the product of the ultimate pullout force F_u of the anchor specimen and its corresponding displacement S_u , i.e., $E_f = F_u \cdot S_u$, rather than the area under the complete pullout force–displacement curve. This simplification is intended to provide a concise reference energy scale, but may slightly underestimate the absolute value of R_{de} , because the actual area under the softening curve is smaller than $F_u \cdot S_u$. However, since all specimens share the same geometry and loading configuration, and the impact mass, drop height, and time interval were kept constant with only the number of impacts varied, R_{de} remains a consistent relative comparison index among different conditions.

Although the definition of the disturbance energy ratio (R_{de}) was proposed on the basis of the disturbance-creep tests conducted on the anchor-soil interface, it can be extended to quantify the disturbance effects experienced by general geomaterials during long-term loading processes. It should be clarified that R_{de} is not intended to establish an equivalent mechanical work relationship between E_d and E_f . Instead, it is a dimensionless normalized index that characterizes the relative magnitude of the external disturbance energy with respect to the ultimate energy capacity of the material. This is similar to commonly used normalized indices in engineering, such as stress ratio or safety factor. To verify the applicability of this definition, a partial statistical survey of existing data from disturbance uniaxial compression creep tests and direct shear creep tests reported in the literature (Liu et al., 2021; Wang et al., 2020; Hu et al., 2025; Fan et al., 2024; Sun et al., 2018; Zhu et al., 2019) was carried out by the authors, and the corresponding disturbance energy ratios were calculated, as presented in Fig. 6. It should be clarified that, in the disturbance uniaxial compression creep tests on rock, the disturbance energy E_d was calculated from the cumulative input of impact loading, and the failure energy E_f was taken as the product of the ultimate compressive strength and the corresponding strain. In the disturbance direct shear creep tests on rock, E_d can be characterized by the shear stress–displacement curve, and E_f was expressed as the product of the peak shear stress and the corresponding displacement. From the perspective of statistical distribution characteristics, the disturbance energy ratios of geomaterials in disturbance creep tests generally follow a log-normal distribution, which is commonly observed in natural phenomena. Although E_f depends on the anchorage area, the ratio form of R_{de} weakens the specimen geometry effect, ensuring good comparability across different test setups, and to some extent reflecting the reasonableness of using this index to quantitatively characterize the disturbance-creep behavior of geomaterials. It should be noted that the available data are all from impact loads and the sample size is limited; caution is required when extending this index to non-impact disturbances or non-geomaterials.

2.3.2. Design of disturbance conditions

Based on the statistical distribution characteristics of the disturbance energy ratio reported in the literature, combined with the pullout creep characteristics of element-scale anchor specimens embedded in clayey soils in the absence of disturbance, the disturbance loading conditions for the disturbance-creep tests on the anchor-soil interface were designed, as presented in Table 2. Due to the natural variability of soil materials, which is much greater than that of homogeneous rock materials, parallel specimens often exhibit considerable scatter. Therefore, only one independent specimen was tested for each condition. Such

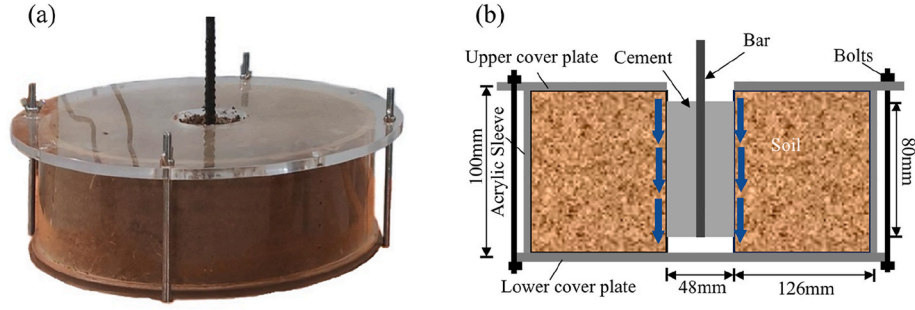


Fig. 3. Element-scale anchor specimen: (a) photograph; (b) composition and dimensions.

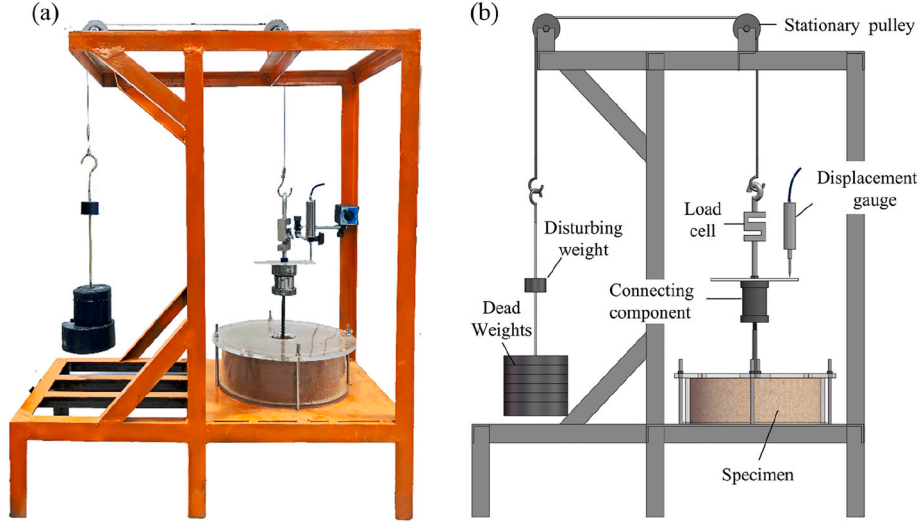


Fig. 4. Creep pullout testing setup for element-scale anchor specimen: (a) photograph of actual setup; (b) schematic diagram.

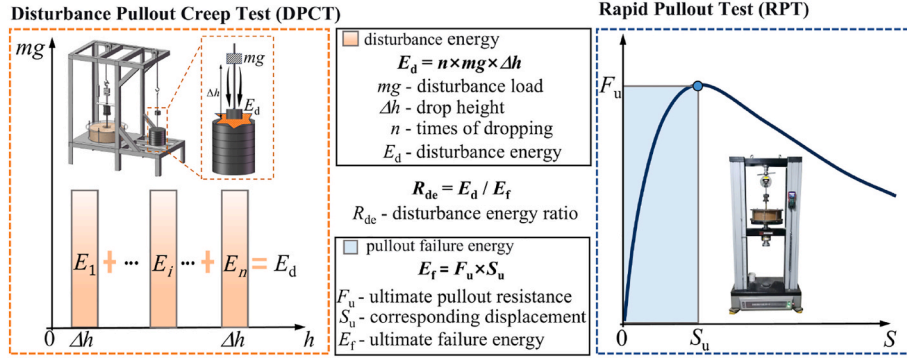


Fig. 5. Schematic diagram illustrating the definition of disturbance energy ratio in the present test.

specimen variability mainly affects the instantaneous pullout stage, i.e., the displacement at the moment of loading may show some deviation; It is noteworthy that, when the disturbance loading conditions adopted in this test were incorporated into the statistical distribution shown in Fig. 6, the corresponding disturbance energy ratios were found to be located near the mean value of the log-normal distribution. This observation provides additional support for the reasonableness of the disturbance condition design employed in the present test.

2.3.3. Design of loading strategy

To conduct the pullout creep tests on the element-scale anchor specimens embedded in clayey soils, a stepwise loading strategy for the pullout force was required to be determined. Short-term (rapid) pullout tests were first carried out on element-scale specimens prepared from

the same batch. From these tests, the pullout force-displacement curves of the specimens were obtained, and the peak pullout force was further determined (as illustrated in Fig. 5). Subsequently, load levels corresponding to 48%, 57%, 66%, 75%, 84%, and 91% of the ultimate pullout force were selected as the load levels for the pullout creep tests. The holding period for each load level was designed as 48 h. After each load level was applied and held for 24 h, a disturbance load was applied, with a disturbance time interval set to 5 s. Once the disturbance application was completed, the specimen continued to sustain the load until the holding period was fulfilled. The next load level was then applied, and the aforementioned disturbance application procedure was repeated. The specific loading strategy is illustrated in Fig. 7.

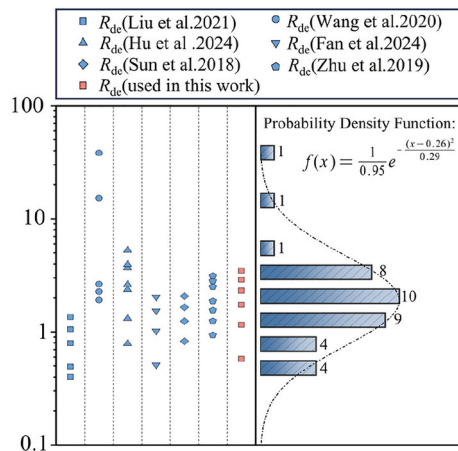


Fig. 6. Statistical distribution of disturbance energy ratio for geomaterials.

Table 2
Test design for disturbance loading conditions.

Specimen ID	Number of impacts N	Disturbance energy $E(J)$	Disturbance energy ratio R_{de}
A0	0	0	0
A1	3	0.6	0.58
A2	6	1.2	1.16
A3	9	1.8	1.74
A4	12	2.4	2.32
A5	15	3.0	2.90
A6	18	3.6	3.48

2.4. Test procedures

2.4.1. Preparation of element-scale specimens

To ensure the consistency of the element-scale anchor specimens to the greatest extent possible, the specimen preparation process in this test followed a standardized five-step procedure. The specimen morphology corresponding to each step is illustrated in Fig. 8.

- (1) **Moisture content conditioning:** The soil samples collected from the field were crushed and passed through a 5 mm sieve. The sieved soil was then spread evenly on trays and placed in an oven at 105 °C for continuous drying over a period of 8 h to ensure complete desiccation. After the soil material was cooled to room temperature in a desiccator, a predetermined amount of distilled water was uniformly sprayed onto the soil using the atomization method to achieve the natural moisture content of 24%.
- (2) **Moisture equilibration:** The moisture-conditioned soil material was sealed in plastic bags and allowed to equilibrate for 24 h to ensure adequate migration and uniform distribution of moisture within the soil mass.

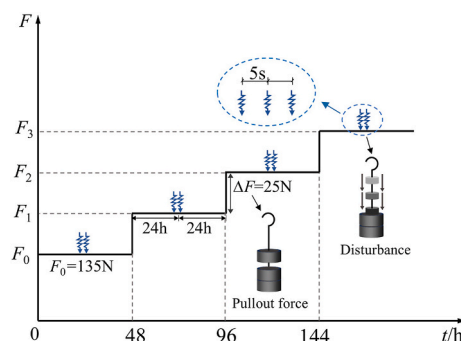


Fig. 7. Design of loading strategy for disturbance-creep tests.

- (3) Soil blank preparation: The layered compaction technique was employed to control the dry density of the soil. The required amount of soil material was weighed according to the designed dry density and placed into the mold in multiple layers. Each layer was compacted individually, and the surface of each compacted layer was scarified to ensure proper contact between adjacent layers, thereby producing a homogeneous soil blank.
- (4) Grout injection into anchor hole: Cement mortar was injected into the preformed anchor hole within the soil blank. The pressure grouting method was adopted to form the grout body, ensuring that the grout was densely compacted and that a well-bonded interface was formed with the surrounding soil.
- (5) Curing: The prepared specimens were placed in a constant-temperature curing room and allowed to cure undisturbed for 72 h. This curing period was intended to allow the grout body to attain initial strength while maintaining a stable moisture content within the soil mass.

2.4.2. Loading and measurement

The disturbance-creep characteristics of the anchor-soil interface in clayey soils were investigated through pullout creep tests conducted on element-scale specimens. The tests specifically involved a two-stage loading procedure, consisting of the application of a constant pullout force corresponding to the interface shear stress and a subsequent impact load corresponding to the interface disturbance shear stress. The overall loading procedure was carried out as follows.

- (1) Specimen installation: The element-scale anchor specimen was first positioned centrally within the testing frame and fixed to the frame using an acrylic cover plate and bolt assemblies. Subsequently, the centering device, displacement gauge, and load cell were installed, as illustrated in Fig. 4. It should be noted that, in order to minimize the influence of ambient temperature and humidity fluctuations on the moisture content of the soil during the test, a layer of petroleum jelly was uniformly applied along the interface between the sidewall of the soil specimen and the surrounding air.
- (2) Data acquisition: The displacement gauge and the load cell were connected to a data acquisition system to enable real-time monitoring of displacement and load variations during the creep process. During the creep loading stage, displacement data were automatically recorded by the system at 1-min intervals, with an acquisition accuracy of 0.01 mm, which was deemed sufficient to meet the observational requirements for interface creep deformation.
- (3) Application of stepwise loads: In accordance with the design of disturbance conditions shown in Fig. 6 and the design of the loading strategy illustrated in Fig. 7, the stepwise loads and the corresponding disturbance loads were applied.
- (4) Termination of loading: Step (3) was repeated until creep failure of the specimen occurred under the final load level.

3. Test results and analysis

3.1. Complete creep curves under disturbance

The complete disturbance-creep curves obtained from typical specimens A2 and A4 are presented in Fig. 9, corresponding to disturbance energy ratios R_{de} of 1.16 and 2.32, respectively. The complete creep curves for the remaining specimens are provided in Appendix A. It should be clarified that the pullout load applied to the element-scale anchor specimen has been converted into the average shear stress level along the anchor-soil interface through the assumption of uniform shear stress distribution over the surface of the element-scale specimen.

An examination of the common characteristics of the disturbance-load-creep curves under a single load level reveals that the disturbance load induces a small abrupt increment in creep displacement, resulting in a four-stage feature as illustrated in Fig. 10. These four stages are identified as primary

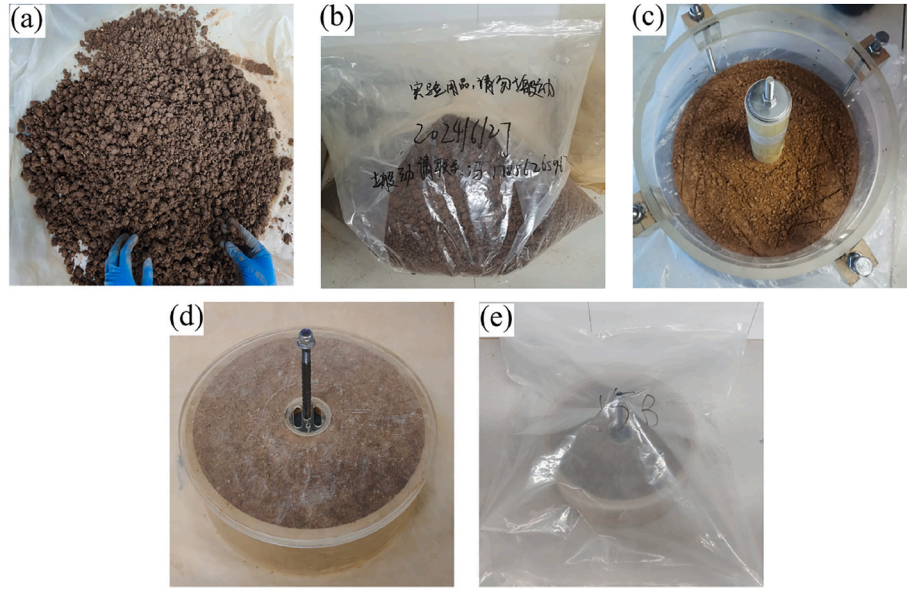


Fig. 8. Preparation procedure for element-scale anchor specimen: (a) moisture content conditioning; (b) moisture equilibration; (c) soil blank preparation; (d) grout injection into anchor hole; (e) curing.

creep, secondary creep, disturbance-induced jump, and post-disturbance creep. The specific behavior observed in each stage is described as follows.

- (1) Primary creep stage: At the instant of load application, an instantaneous elastic deformation is produced in the specimen, and the creep deformation rate reaches a local maximum. Subsequently, under the action of a constant load, the deformation rate gradually decreases with elapsed time, and the interface progressively enters a steady-state development phase.
- (2) Secondary creep stage: Continuous deformation of the interface is generated at an approximately constant creep rate. The creep rate is governed by the level of interface shear stress. As the shear stress level increases, the creep rate correspondingly increases, exhibiting a clear stress-dependent behavior.
- (3) Disturbance-induced jump stage: At the instant the disturbance load is applied, an irreversible small abrupt change occurs in the interface creep. With an increasing number of disturbances, these abrupt changes accumulate to form a distinct creep jump. The magnitude of the abrupt change is positively correlated with both the disturbance energy and the shear stress level. Specifically, the greater the disturbance energy or the higher the shear stress, the more pronounced the displacement increment induced by a single disturbance becomes. Additionally, the force fluctuation induced by each impact stabilizes within approximately 1 s,

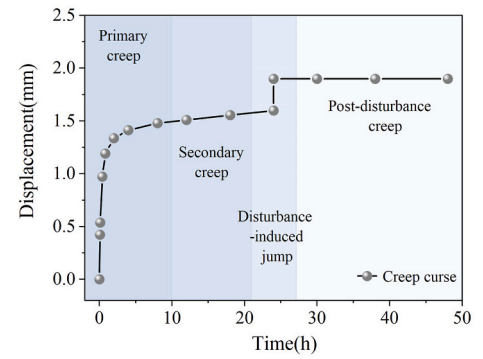


Fig. 10. Four-stage characteristics of disturbance-creep curve under a single load level.

indicating that the interface quickly re-establishes equilibrium after disturbance without sustained stress concentration.

- (4) Post-disturbance creep stage: During this stage, the creep rate is approximately zero, and the influence of the shear stress level and disturbance energy on the creep rate is no longer significant. The displacement increment during this stage is nearly zero, which further confirms that the interface shear stress remains uniform after impact.

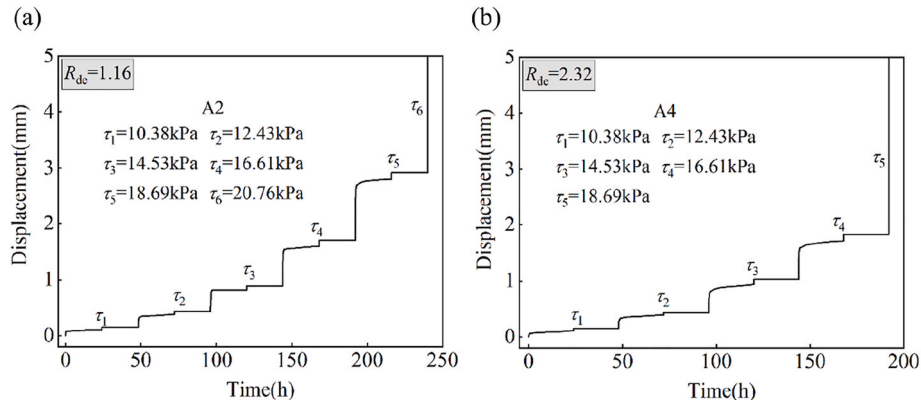


Fig. 9. Complete disturbance-creep curves of anchor-soil interface: (a) Specimen A2; (b) Specimen A4.

3.2. Equivalent creep curves

In order to quantify the influence of the disturbance-induced jump on the creep characteristics under each load level, a power function was adopted to perform an equivalent fitting of the three stages—namely, secondary creep, disturbance-induced jump, and post-disturbance creep—on the basis of the overall trend analysis of the disturbance-creep curves. Through this fitting procedure, the original curves were transformed into a single equivalent creep curve, as illustrated in Fig. 11.

The power function employed for the equivalent creep curve is expressed as follows:

$$f(x) = ax^{\frac{1}{2}} + bx + c \quad (2)$$

Where a , b , and c are equivalent parameters. This power function was selected because its $x^{1/2}$ term captures the decelerating secondary creep, the linear term bx reflects the steady-state trend, and the constant term c represents the initial displacement, consistent with the observed creep development. With only three parameters, it simplifies the constrained optimization and avoids over-parameterization. Compared with exponential and hyperbolic functions, it is simpler in form and does not introduce extra inflection points or oscillations, thereby preserving the original variation trend.

The parameters a , b , and c were determined by a constrained nonlinear least-squares method under four constraint conditions: (1) the equivalent curve must pass through the starting point of the secondary creep stage; (2) the equivalent curve must pass through the end point of the post-disturbance creep stage; (3) the equivalent curve must pass through the midpoint of the disturbance-induced jump stage; and (4) the relative difference between the areas S_1 and S_2 enclosed by the equivalent curve and the secondary creep stage and the post-disturbance creep stage, respectively, shall not exceed 1%. It should be noted that the use of an area-difference criterion for parameter identification is not new; similar approaches have been adopted in previous studies for curve fitting and creep parameter extraction (Qamar and Wilayat Husain, 1993; Campbell et al., 2017). Initial values of a , b , and c were obtained by linearizing the function with respect to the three constraint points. The iteration was considered converged when the relative change in the objective function between two consecutive iterations was less than 0.1 and the area-difference constraint was satisfied.

The four-stage creep curves corresponding to different load levels were first processed using Chen's loading method (Chen and Kang, 1991). Subsequently, the processed curves were transformed using the aforementioned equivalent method, thereby yielding the separate loading creep curves for the anchor-soil interface under different disturbance conditions, as shown in Fig. 12.

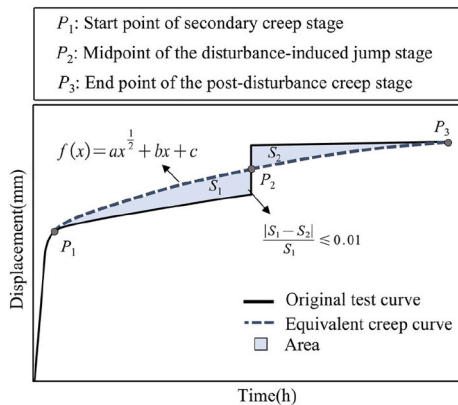


Fig. 11. Schematic diagram illustrating the transformation into equivalent creep curve.

3.3. Total creep displacement under different disturbance conditions

To investigate the influence of disturbance on the creep behavior of the anchor-soil interface, the corresponding total creep displacement (i.e., the maximum creep displacement on the curve) was extracted from the separate loading creep curves presented in Fig. 12. From these data, the variation of the total creep displacement with the disturbance energy ratio under different shear stress levels was obtained, as illustrated in Fig. 13. It should be clarified that, when the disturbance energy ratio was relatively large (exceeding 1.16), shear failure of the element-scale specimen occurred under the fifth load level (shear stress of 18.69 kPa). Consequently, the total creep displacement corresponding to the fifth load level could not be monitored during the test, and only three data points under lower disturbance energy ratios are presented for this load level.

An examination of Fig. 13 reveals the following observations. At relatively low shear stress levels (e.g., 10.38 kPa and 12.43 kPa), the total creep displacement is not significantly sensitive to variations in the disturbance energy ratio, and the overall variation amplitude is relatively small. This behavior reflects that, under low shear stress conditions, the disturbance toughness of the anchor-soil interface is relatively high, and no appreciable damage occurs at the interface. With an increase in the shear stress level, a trend of increasing total creep displacement with increasing disturbance energy ratio becomes progressively apparent, and the rate of increase becomes more pronounced. Specifically, the increments corresponding to the respective shear stress levels are 0.06 mm, 0.19 mm, 0.32 mm, 0.89 mm, and 0.47 mm, respectively. This indicates that, under higher shear stress conditions, the disturbance toughness of the anchor-soil interface gradually decreases, and irreversible damage accumulation progressively develops at the interface.

4. DSC-based disturbance-creep model for anchor-soil interface

According to the Disturbed State Concept (DSC) theory, the macroscopic response of a material is regarded as a weighted combination of two limit responses: the relatively intact (RI) state and the fully adjusted (FA) state, with a continuous transition between them achieved through a disturbance function. Given that the disturbance damage accumulation at the anchor-soil interface during the creep process is fully consistent with this fundamental principle, and that this theory has been effectively applied to the characterization of the long-term bearing behavior of the anchor-soil interface (Zhu et al., 2023; Cai et al., 2024), the DSC theory is further extended in this study to the disturbance-creep behavior of the anchor-soil interface. Unlike existing studies that mainly address short-term or static loading, this work focuses on the progressive degradation of the interface from the RI state to the FA state under external disturbances, based on which the corresponding disturbance-creep model is established.

4.1. Model establishment

Based on the DSC theory, the anchor-soil interface is considered to be composed of two types of randomly distributed units: those in the RI state and those in the FA state. The RI units represent the initial undisturbed state, in which the structural integrity is well preserved. The FA units correspond to the limit state in which the structure has been fully damaged and has reached a stable condition after sufficient disturbance. At the initial moment, all units are in the RI state. As the load is applied or the disturbance is intensified, the RI units are gradually transformed into FA units. These two types of units work together, and the macroscopic mechanical response is governed jointly by the dynamic adjustment of their proportions and the corresponding sharing of stress.

For the disturbance-creep conditions of the anchor-soil interface examined in the present test, the RI and FA units are required to satisfy a displacement compatibility condition, i.e., the two types of units sustain the load simultaneously. Under this condition, the interface shear creep displacement can be taken as the sum of the contributions from the two states. Consequently, a DSC-based interface creep model was established

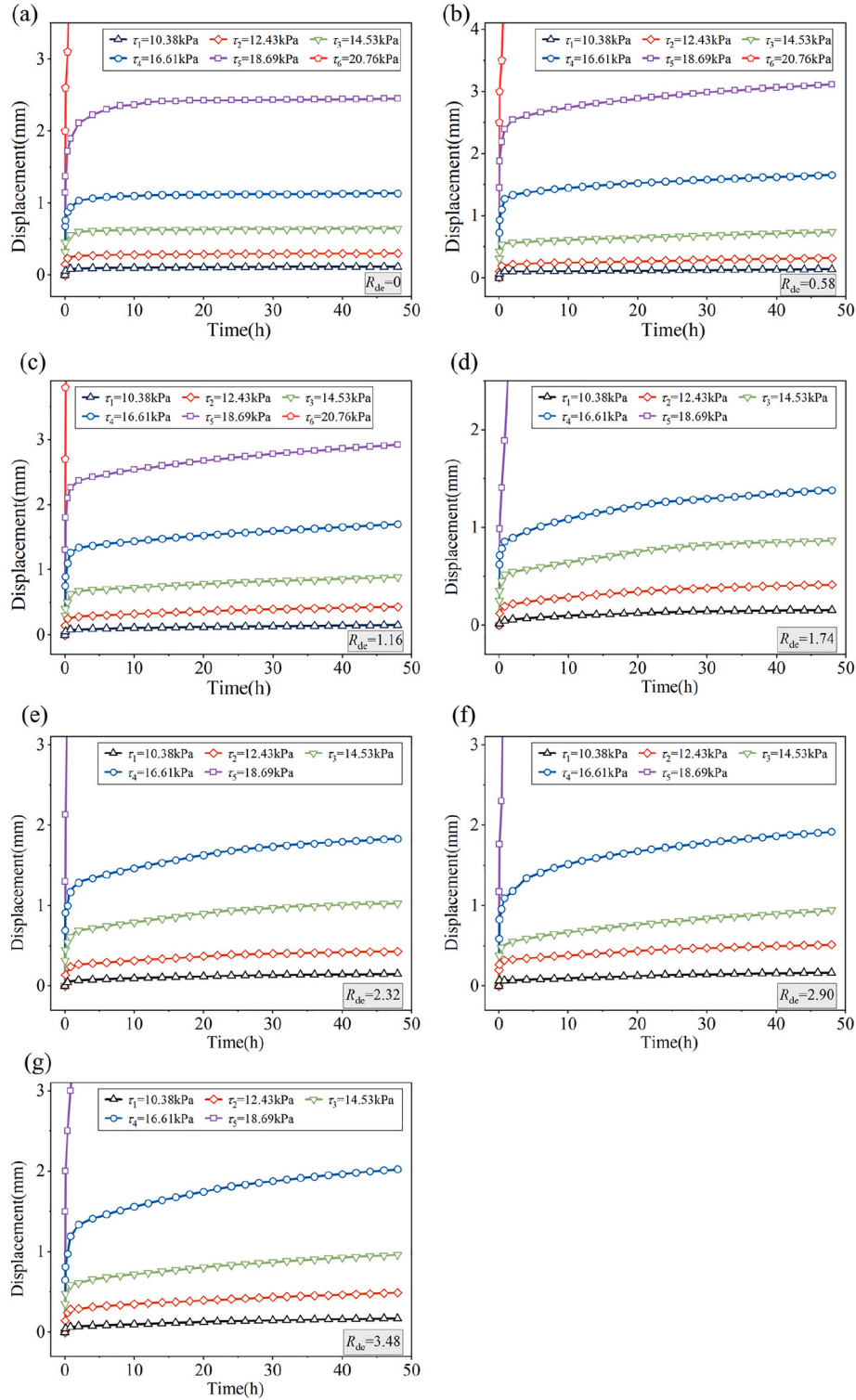


Fig. 12. Separate loading creep curves of anchor-soil interface under different disturbance conditions (a) Specimen A0; (b) Specimen A1; (c) Specimen A2; (d) Specimen A3; (e) Specimen A4; (f) Specimen A5; (g) Specimen A6.

by connecting the rheological models corresponding to the two states in series, as illustrated in Fig. 14. This series configuration can not only capture the elastic-viscoelastic creep behavior of the RI state but also characterize the plastic-viscoplastic creep response of the FA units. Accordingly, the model is capable of effectively describing the development process of the anchor-soil interface from instantaneous deformation to steady-state creep under the action of shear stress. The mathematical expression of this model is given in Eq. (3).

$$\begin{cases} \tau = \tau^{\text{RI}} = \tau^{\text{FA}} \\ s = (1 - D_t)s^{\text{RI}} + D_t s^{\text{FA}} \end{cases} \quad (3)$$

where D_t is the disturbance function; s^{RI} is the shear displacement of the anchor-soil interface in the RI state; s^{FA} is the shear displacement of the anchor-soil interface in the FA state; τ^{RI} and τ^{FA} are the shear stresses at the anchor-soil interface in the RI state and the FA state, respectively;

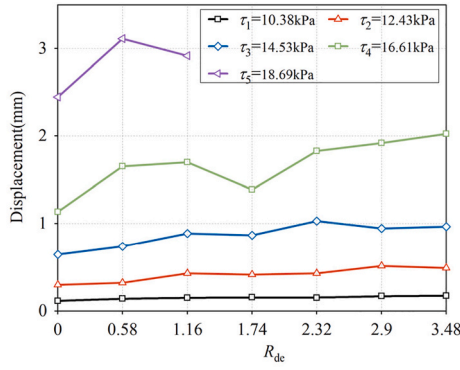


Fig. 13. Relationship between total creep displacement and disturbance energy ratio under different shear stress levels.

and s and τ are the shear displacement and the shear stress at the anchor-soil interface measured in the disturbance-creep tests, respectively.

4.1.1. RI state

At the early stage of service, the mechanical behavior of the anchor-soil interface is governed primarily by the intact cementation and interlocking effects. Its creep response exhibits a typical primary creep characteristic: the creep rate gradually decreases with elapsed time and eventually approaches a stable value. The physical mechanism underlying this characteristic lies in the fact that the shear band at the anchor-soil interface undergoes self-stabilizing time-dependent structural adjustment, rather than irreversible macroscopic slip. As a constitutive model capable of simultaneously characterizing time-dependency and deformation self-stabilization, the Kelvin model yields a mechanical response of strain rate decay and stabilization that is highly consistent with the physical mechanism described above. Accordingly, the Kelvin model can be adopted to describe the relatively intact (RI) state of the anchor-soil interface. The corresponding creep equation is expressed as:

$$s^{\text{RI}}(t) = \frac{\tau}{E_1} \left[1 - \exp\left(-\frac{E_1 t}{\eta_1}\right) \right] \quad (4)$$

Where E_1 is the interface shear modulus; and η_1 is the interface shear viscosity coefficient.

4.1.2. FA state

When the anchor-soil interface reaches the fully adjusted (FA) state as a result of accumulated damage, its cementation effect is essentially lost. Under this condition, the applied load is transferred primarily through a friction-dominated macroscopic slip mechanism. The creep behavior enters a stage in which deformation develops at an approximately constant rate (i.e., secondary creep), while a certain elastic response capability is still retained. To describe the creep behavior in this state, the constitutive creep model is required to meet two conditions: first, it must be capable of representing long-term steady viscous flow to simulate the continuous slip; and second, it must incorporate an instantaneous elastic response to characterize the elastic adjustment of the slip surface contacts. The constitutive equation of the Maxwell model naturally decouples instantaneous elastic deformation from linear viscous flow. Both its mathematical form and the underlying physical mechanism can accurately correspond to the macroscopic slip state described above. Therefore, the Maxwell model can be adopted to describe the FA state of the anchor-soil interface. The corresponding creep equation is expressed as:



Fig. 14. RI-FA series model for disturbance-creep of anchor-soil interface.

$$s^{\text{FA}}(t) = \tau \left(\frac{1}{E_2} + \frac{t}{\eta_2} \right) \quad (5)$$

Where E_2 is the interface shear modulus; and η_2 is the interface shear viscosity coefficient.

4.1.3. Disturbance function

The disturbance function D reflects the degree of disturbance experienced by the anchor-soil interface under external loading through the load-sharing proportion between the RI-state and FA-state units. Its value ranges between 0 and 1. A larger value of D indicates a higher degree of disturbance. When $D = 0$, no disturbance exists at the anchor-soil interface, and all units remain in the RI state. When $D = 1$, the anchor-soil interface is fully disturbed, and all units have been transformed into the FA state. The disturbance function D of the anchor-soil interface is defined as the ratio of the number of damaged interface units n_f to the total number of interface units n , which is expressed as:

$$D = \frac{n_f}{n} \quad (6)$$

It is assumed that the strength of the individual interface units follows a Weibull distribution (Baghini et al., 2018; Huang et al., 2020a), with the probability density function $f(x)$ given by:

$$f(x) = \frac{\eta}{\xi} \left(\frac{x}{\xi} \right)^{\eta-1} \exp \left[-\left(\frac{x}{\xi} \right)^{\eta} \right] \quad (7)$$

Where η and ξ are probability parameters.

When the shear displacement is, the number of damaged interface units n_f can be determined as:

$$n_f = \int_0^s n \cdot f(x) dx = n \cdot \left\{ 1 - \exp \left[-\left(\frac{s}{\xi} \right)^{\eta} \right] \right\} \quad (8)$$

By substituting Eq. (8) into Eq. (6), the expression for the disturbance function D can be obtained as:

$$D = 1 - \exp \left[-\left(\frac{s}{\xi} \right)^{\eta} \right] \quad (9)$$

The fundamental nature of creep tests lies in the continuous accumulation of interface damage with elapsed time. To characterize this time effect, a time variable must be introduced within the DSC framework to describe the progressive evolution of damage from the initially intact state to the fully adjusted state. In creep tests, the shear stress is constant and the interface shear displacement increases monotonically with time; therefore, the displacement s can be regarded as a monotonic function of time t . Based on this relationship, the displacement-based disturbance function can be converted into a time-dependent form, thereby establishing a quantitative relationship between the degree of disturbance and creep time.

Accordingly, the time-dependent disturbance function D_t is defined as the ratio of the number of FA-state anchor-soil interface units to the total number of units, and its expression is given by:

$$D_t = 1 - \exp \left[-\left(\frac{t}{\alpha} \right)^{\beta} \right] \quad (10)$$

It should be noted that micro-units in the interface have random strengths; under disturbance, weaker units fail first and transform into the FA state, with cumulative failure following a Weibull distribution. Since displacement and time are monotonically related, replacing displacement with time preserves this statistical law, so the Weibull form remains applicable.

In accordance with the RI-FA series modeling logic of the DSC theory (Fig. 14), the DSC-based disturbance-creep model for the anchor-soil interface can be obtained by substituting Eqs. (4), (5) and (10) into Eq. (3), as illustrated in Fig. 15. The corresponding mathematical expression is given by:

$$s(t) = \exp\left[-\left(\frac{t}{\alpha}\right)^\beta\right] \frac{\tau}{E_1} \left(1 - \exp\left(-\frac{E_1 t}{\eta_1}\right)\right) + \left(1 - \exp\left[-\left(\frac{t}{\alpha}\right)^\beta\right]\right) \tau \left(\frac{1}{E_2} + \frac{t}{\eta_2}\right) \quad (11)$$

Where E_1 , E_2 , η_1 , η_2 , α and β are the model parameters to be determined.

A further analysis of the displacement response corresponding to the DSC-based disturbance-creep model for the anchor-soil interface is presented in Fig. 16. It can be observed that, at the initial stage of creep, the RI-state components dominate the anchor-soil interface, and the viscoelastic behavior described by the Kelvin model plays a predominant role. Under this condition, the interface shear displacement increases slowly with elapsed time and tends to stabilize within this stage, indicating that the shear band at the anchor-soil interface is undergoing elastic adjustment and delayed deformation. As the disturbance function D_t gradually increases, the proportion of FA-state components at the anchor-soil interface progressively rises, and the viscoplastic flow behavior described by the Maxwell model becomes increasingly apparent. Correspondingly, the creep rate tends to stabilize, indicating that the anchor-soil interface has entered the secondary creep stage. As the disturbance function D_t approaches its limit value, the anchor-soil interface is essentially fully transformed into FA-state components, and the creep rate remains constant, suggesting that the anchor-soil interface has entered a continuous slip stage. It should be noted that the model does not include an explicit criterion for accelerated creep or failure. The above analysis focuses on the primary and secondary creep stages.

5. Discussion

5.1. Model parameter identification

To identify the parameters of the DSC-based disturbance-creep model for the anchor-soil interface, the proposed model was adopted to characterize the disturbance-creep response of Specimen A1 (corresponding to a disturbance energy ratio of 0.58 and a shear stress level of 18.69 kPa). The characterization results are presented in Fig. 16. The model parameters identified through this process are as follows: $E_1 = 35.0 \text{ kPa} \cdot \text{mm}^{-1}$, $E_2 = 6.4 \text{ kPa} \cdot \text{mm}^{-1}$, $\eta_1 = 8.4 \text{ kPa} \cdot \text{mm}^{-1}$, $\eta_2 = 6006.0 \text{ kPa} \cdot \text{mm}^{-1}$, $\alpha = 0.36$, and $\beta = 0.33$. By employing the identified parameter values, the creep responses of the anchor-soil interface in the fully RI state and the fully FA state were further obtained using Eqs. (4) and (5), respectively (see Fig. 17).

5.2. Effect of disturbance energy ratio

To further investigate the influence of disturbance on the creep characteristics of the anchor-soil interface, a sensitivity analysis of the model parameters under varying disturbance energy ratios was conducted using the DSC-based disturbance-creep model for the anchor-soil interface. Furthermore, a disturbance-creep model incorporating the effect of the disturbance energy ratio was established. The specific procedure is outlined below, taking the specimens under a shear stress level of 16.61 kPa as an example.

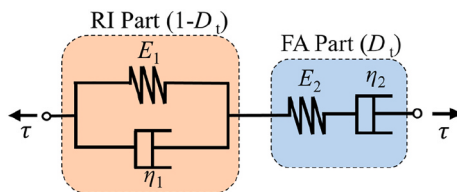


Fig. 15. Structure of the DSC-based disturbance-creep model for anchor-soil interface.

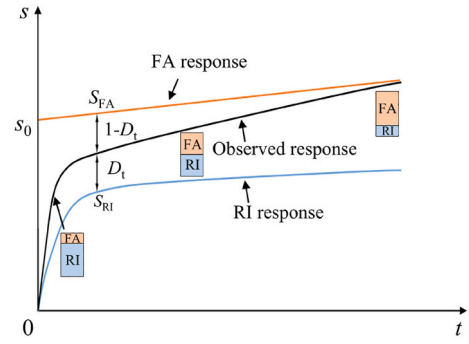


Fig. 16. Displacement response of the DSC-based disturbance-creep model for anchor-soil interface.

- (1) The disturbance-creep curves of the anchor-soil interface for seven groups of specimens under a shear stress level of 16.61 kPa, corresponding to different disturbance energy ratios, were first divided into a training set (corresponding to R_{de} values of 0, 0.58, 1.74, 2.32, and 3.48) and a validation set (corresponding to R_{de} values of 1.16 and 2.90). The training set covers the full range from no disturbance to relatively high disturbance and provides comprehensive information for parameter calibration. The validation set lies within this range and was not involved in calibration, and is used to examine the interpolation prediction capability rather than extrapolation. It should be noted that the disturbance energy ratio corresponding to the validation case of 2.90 exceeds approximately 79% of the data compiled in this study, representing a relatively high disturbance level, which may to some extent reflect the generalization capability of the model.
- (2) The DSC-based disturbance-creep model for the anchor-soil interface was employed to fit the five curves in the training set. The fitting results are presented in Fig. 18, and the corresponding values of the six model parameters (E_1 , E_2 , η_1 , η_2 , α , β) were obtained, as summarized in Table 3.

As shown in Table 3, the fitting accuracies of the DSC-based disturbance-creep model for the training data under a shear stress level of 16.61 kPa all exceeded 0.9, and the prediction accuracy for the validation data also exceeded 0.85. These results validate the capability of the proposed model to characterize the creep response of the anchor-soil interface under different disturbance conditions. However, it should be noted that the correlation matrix of the fitted parameters indicates that some parameter pairs are strongly correlated, and the confidence intervals for these parameters are relatively wide. This is a common phenomenon in nonlinear creep models. It can be attributed to inherent

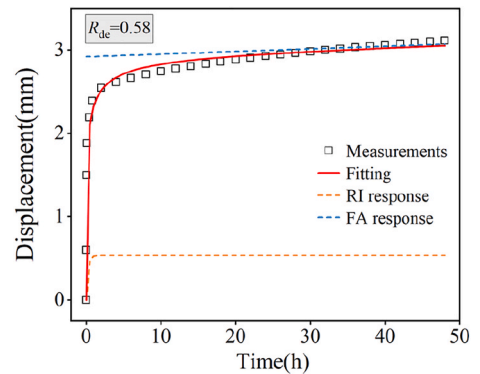


Fig. 17. Characterization results of the DSC-based disturbance-creep model for the creep response of Specimen A1.

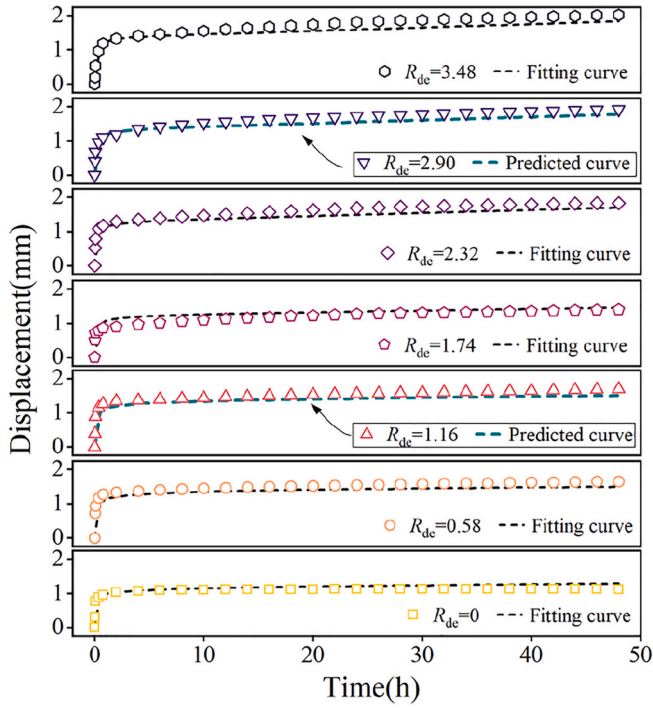


Fig. 18. Training and prediction of the DSC-based disturbance-creep model for anchor-soil interface under different disturbance energy ratios.

coupling and compensation effects between parameters, as well as overlapping functions of some parameters in controlling the creep response. Furthermore, the limited number of creep tests restricts the independent information available for uniquely identifying all parameters.

- (3) Based on Table 3, the evolution of each model parameter with the variation of the disturbance energy ratio was further analyzed, and the corresponding correlations were established, as illustrated in Fig. 19.

With increasing disturbance energy ratio R_{de} , the model parameters exhibited distinct trends. The parameter α decreased continuously, reflecting a transition from strain hardening to strain softening that becomes progressively dominant under high disturbance energy. The parameter β increased continuously, with a relatively rapid rate at the initial stage, indicating accelerated damage accumulation. The interface shear modulus E_1 decreased sharply at first and then approached a residual value, suggesting that even a small disturbance can transform near-surface units into the fully adjusted state and cause an abrupt stiffness reduction. In contrast, E_2 remained stable and was insensitive to disturbance. The interface shear viscosity coefficient η_1 showed an

initially slow decrease followed by an accelerated decline, revealing a threshold in damage accumulation. The viscosity coefficient η_2 decreased continuously and was highly sensitive to disturbance, indicating that long-term creep behavior strongly depends on the state transformation of the interface, once it begins, the long-term creep resistance is significantly weakened.

- (4) By substituting the established correlations between the model parameters and the disturbance energy ratio into the model function, the disturbance-creep model for the anchor-soil interface incorporating the effect of the disturbance energy ratio under a shear stress level of 16.61 kPa was obtained. The corresponding function is expressed as Eq. (12):

$$\left\{ \begin{aligned} s(t) &= \exp \left[-\left(\frac{t}{\alpha} \right)^\beta \right] \frac{\tau}{E_1} \left(1 - \exp \left(-\frac{E_1 t}{\eta_1} \right) \right) + \left(1 - \exp \left[-\left(\frac{t}{\alpha} \right)^\beta \right] \right) \tau \left(\frac{1}{E_2} + \frac{t}{\eta_2} \right) \\ \alpha &= 0.54 \exp(-0.14 R_{de}) + 0.01 \\ \beta &= 0.20 R_{de}^{0.58} + 0.14 \\ E_1 &= 98.33 \exp(-4.01 R_{de}) + 30.66 \\ E_2 &= 12.94 \exp(-(R_{de} - 2.10)/20.78) \\ \eta_1 &= -0.29 R_{de}^{2.64} + 9.23 \\ \eta_2 &= 9799.77 \exp(-0.82 R_{de}) + 872.73 \end{aligned} \right. \quad (12)$$

5.3. Effect of shear stress level

To further investigate the influence of shear stress level on the disturbance-creep characteristics of the anchor-soil interface, a sensitivity analysis of the parameters of the DSC-based disturbance-creep model for the anchor-soil interface was conducted under varying shear stress levels. Furthermore, a disturbance-creep model incorporating the effect of shear stress level was established.

Following a modeling procedure similar to that presented in Sect. 5.2, the disturbance-creep curves corresponding to five different shear stress levels were divided into a training set (shear stress levels of 10.38 kPa, 12.43 kPa, 16.61 kPa, and 18.69 kPa) and a validation set (shear stress level of 14.53 kPa), taking the specimens under a disturbance energy ratio of 1.16 as an example. The values of the model parameters are summarized in Table 4, and the training and prediction performance of the model is illustrated in Fig. 20. It can be observed that the training accuracy of the model reached 0.85, and the prediction accuracy reached 0.89, thereby validating the capability of the proposed model to characterize the creep response of the anchor-soil interface under different shear stress levels.

Based on Table 4, the correlations between the model parameters and the shear stress level were established, as illustrated in Fig. 21.

With increasing shear stress level τ , the model parameters exhibited distinct evolution trends. The parameter α decreased gradually and continuously, reflecting a steady transformation of more units into the

Table 3

Parameter values of the DSC-based disturbance-creep model for anchor-soil interface (corresponding to a shear stress level of 16.61 kPa).

R_{de}	Model parameters						R^2	RMSE
	α	β	E_1 (kPa.mm ⁻¹)	E_2 (kPa.mm ⁻¹)	η_1 (kPa.h)	η_2 (kPa.h)		
<i>Fitting</i>								
0	0.53	0.14	132.1	11.7	9.1	10500	0.90	0.093
0.58	0.51	0.31	38.4	11.8	8.9	7534	0.93	0.042
1.74	0.44	0.40	30.2	13.5	8.3	3541	0.92	0.082
2.32	0.43	0.48	30.1	13.0	8.1	1896	0.91	0.036
3.48	0.31	0.60	40.4	12.3	2.0	1645	0.92	0.077
<i>Prediction</i>								
1.16	0.47	0.36	31.6	12.4	8.8	4655	0.85	0.059
2.90	0.37	0.49	30.6	12.5	3.5	1768	0.89	0.094

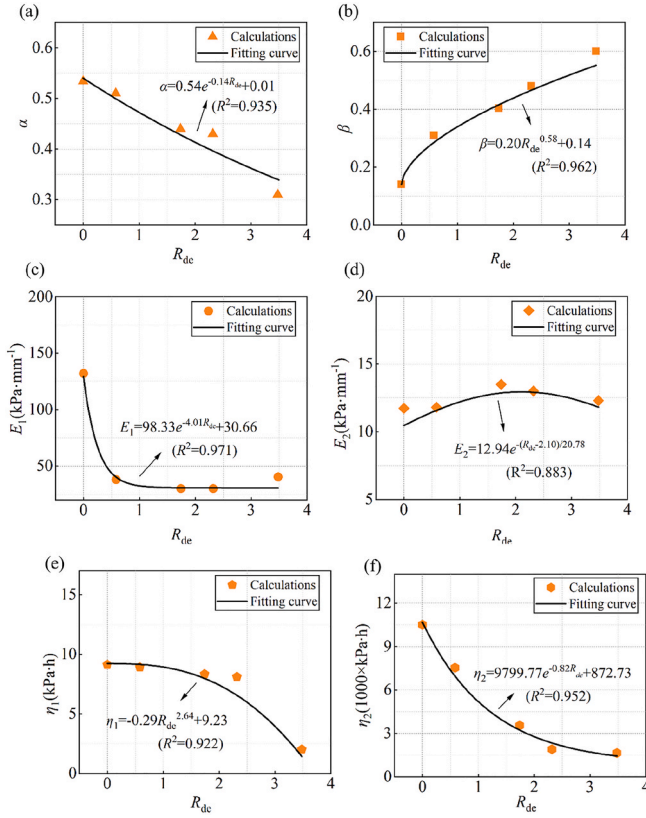


Fig. 19. Correlations between model parameters and disturbance energy ratio: (a) α ; (b) β ; (c) E_1 ; (d) E_2 ; (e) η_1 ; (f) η_2 .

fully adjusted state without abrupt changes. The parameter β increased gently, with no distinct acceleration stage, far less pronounced than the exponential acceleration induced by disturbance energy. The interface shear modulus E_1 showed an initial sharp decay followed by a residual value; compared with the disturbance-induced decay, this decay was gentler in the low-stress range and more continuous. The interface shear modulus E_2 decayed exponentially and sharply, a dominant effect unique to τ . Its plastic hardening capacity depended strongly on stress level and was almost entirely lost at high stress levels, although a residual friction modulus remained. The interface viscosity coefficient η_1 decreased uniformly without a threshold, suggesting that the regulation of short-term viscosity by τ is closer to a linear relationship, unlike the disturbance-induced decrease. The parameter η_2 also decreased continuously and was sensitive to disturbance, though the magnitude of variation under τ was smaller than that induced by disturbance energy.

By substituting the established correlations between the model parameters and the shear stress level into the model function, the disturbance-creep model for the anchor-soil interface incorporating the effect of shear stress level under a disturbance energy ratio of 1.16 was obtained. The corresponding function is expressed as Eq. (13):

$$s(t) = \exp \left[-\left(\frac{t}{\alpha} \right)^{\beta} \right] \frac{\tau}{E_1} \left(1 - \exp \left(-\frac{E_1 t}{\eta_1} \right) \right) + \left(1 - \exp \left[-\left(\frac{t}{\alpha} \right)^{\beta} \right] \right) \tau \left(\frac{1}{E_2} + \frac{t}{\eta_2} \right)$$

$$\alpha = 47.77\tau^{-1.7} + 0.02$$

$$\beta = 0.14\tau^{0.38} - 0.065$$

$$E_1 = 56.96/(\tau - 10.13) + 25.77$$

$$E_2 = 3513.00\exp(-0.29\tau) - 30.00$$

$$\eta_1 = 845.18\tau^{-1.64} - 0.16$$

$$\eta_2 = 1131.00\exp[-0.58(\tau - 10.38)] + 3909.00$$

(13)

5.4. Combined effect of disturbance energy ratio and shear stress level

The DSC-based disturbance-creep model for the anchor-soil interface was adopted to characterize all the creep curves obtained from the tests, and the corresponding values of the model parameters for each curve were determined, as presented in Appendix B.

To analyze the combined effect of the disturbance energy ratio and the shear stress level on the creep response of the anchor-soil interface, a three-dimensional response surface was employed to describe the evolution of each model parameter with variations in the disturbance energy ratio and the shear stress level, as illustrated in Fig. 22.

A further analysis of Fig. 22 reveals that the variation of each model parameter with the disturbance energy ratio and the shear stress level conforms to a unified correlation, i.e., it can be described by a single function. This observation further indicates that the DSC-based disturbance-creep model for the anchor-soil interface proposed in this study possesses a favorable generalization capability. The specific function form is expressed as:

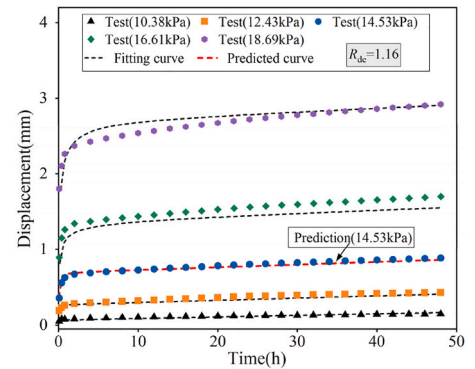


Fig. 20. Training and prediction of the DSC-based disturbance-creep model for anchor-soil interface under different shear stress levels.

Table 4

Parameter values of the DSC-based disturbance-creep model for anchor-soil interface (corresponding to a disturbance energy ratio of 1.16).

τ (kPa)	Model parameters						R^2	RMSE
	α	β	E_1 (kPa·mm ⁻¹)	E_2 (kPa·mm ⁻¹)	η_1 (kPa·h)	η_2 (kPa·h)		
<i>Fitting</i>								
10.38	0.90	0.30	254.6	150.2	19.2	5124	0.85	0.015
12.43	0.71	0.31	50.2	45.1	11.3	4312	0.91	0.030
16.61	0.47	0.37	35.4	10.9	8.6	4112	0.93	0.047
18.69	0.31	0.38	31.2	7.0	8.1	3914	0.90	0.120
<i>Prediction</i>								
14.53	0.51	0.35	38.7	21.0	9.9	4010	0.89	0.028

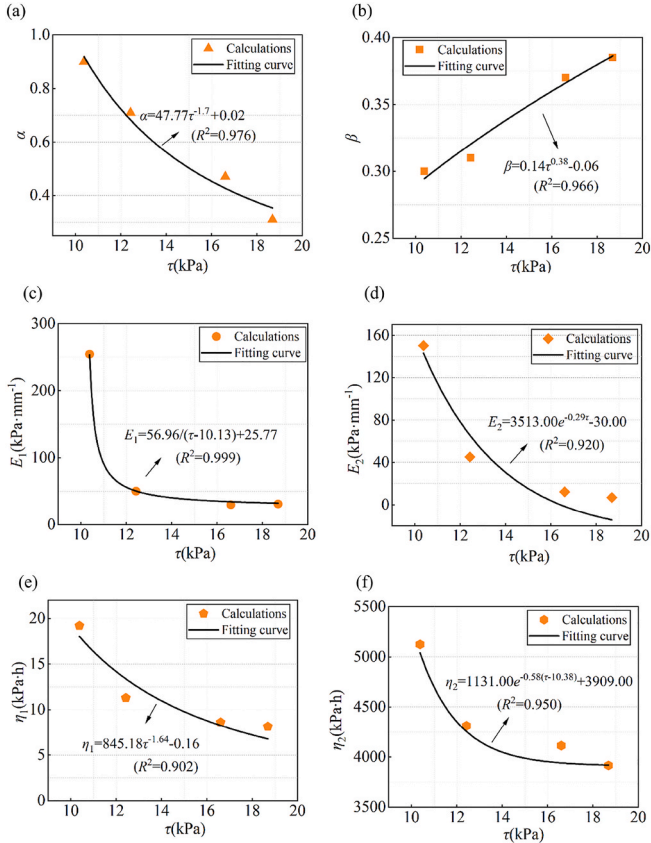


Fig. 21. Correlations between model parameters and shear stress level: (a) α ; (b) β ; (c) E_1 ; (d) E_2 ; (e) η_1 ; (f) η_2 .

$$y = a + b \exp\left(-\frac{R_{de}}{c}\right) \exp\left(-\frac{\tau}{d}\right) \quad (14)$$

Where y represents a specific model parameter; and a , b , c and d are the undetermined coefficients of the function. The specific values of these coefficients corresponding to each model parameter are summarized in Table 5

By combining Fig. 22 with the unified correlation of the model parameters, the mechanism by which the disturbance energy ratio and the shear stress level jointly influence the disturbance-creep response of the anchor-soil interface can be further analyzed.

Parameter α decreases continuously, indicating a transition from strain hardening to strain softening, loss of plastic deformation capacity, and increased brittle failure risk; its value can serve as a reference for evaluating interface embrittlement. Parameter β increases exponentially with shear stress level and R_{de} , reflecting accelerated damage; it can serve as a reference index for assessing whether the anchorage system is approaching instability, and when β rises rapidly beyond a threshold, timely reinforcement or unloading measures should be taken.

Shear modulus E_1 decreases sharply under low stress and low disturbance and then approaches a residual value, reflecting degradation of initial elastic stiffness; in slope reinforcement, attention should be paid to early-stage stiffness reduction, with a higher safety factor or strengthened near-surface structure. Shear modulus E_2 decays exponentially mainly with shear stress level and is insensitive to disturbance energy, indicating that plastic hardening capacity is controlled almost entirely by shear stress level; in permanent anchor design, E_2 should be reasonably reduced according to the anticipated shear stress level to avoid overestimating residual strength.

Viscosity coefficient η_1 decreases with increasing shear stress level and R_{de} , and the decrease accelerates after the disturbance energy

exceeds a threshold; for coastal highway slope anchors subjected to frequent disturbances, high-viscosity grouting materials or improved interface roughness should be adopted to delay deformation. Viscosity coefficient η_2 is most sensitive to disturbance, decreasing sharply with increasing R_{de} and closely related to the creep rate. A substantial decrease in η_2 accelerates deformation even in the secondary creep stage, which may lead to prestress loss and excessive deformation. Using the proposed model with field-monitored disturbance energy, the trend of η_2 can be preliminarily estimated to provide a reference for long-term performance assessment of the anchorage system.

6. Conclusions

In this study, shear creep tests on the anchor-soil interface of grouted anchors embedded in clayey soils were carried out under different disturbance conditions, and the evolution of the interface creep behavior under the combined action of disturbance and shear stress was systematically investigated. Based on the Disturbed State Concept (DSC), a disturbance-creep model capable of comprehensively incorporating the effects of both the disturbance energy ratio and the shear stress level was established. The main conclusions drawn from this study are as follows.

- (1) From an energy perspective, an index termed the disturbance energy ratio was proposed. This index effectively characterizes the proportional relationship between the shear disturbance acting on the anchor-soil interface and the ultimate bearing capacity of the interface, thereby providing a unified quantitative index for evaluating the influence of shear disturbance on the long-term bearing performance of the anchor-soil interface.
- (2) The disturbance-creep process of the anchor-soil interface can be divided into four stages: primary creep, secondary creep, disturbance-induced jump, and post-disturbance creep. The application of a disturbance load induces an irreversible creep jump, and the magnitude of this jump increases with increasing shear stress level.
- (3) An equivalent creep curve method was proposed by minimizing the area deviation between the creep curves before and after the disturbance-induced jump. This method transforms the disturbance-creep curve into a creep curve free of disturbance-induced jumps, thereby providing benchmark data for the theoretical modeling of disturbance-creep behavior.
- (4) A DSC-based disturbance-creep model for the anchor-soil interface was established, which can comprehensively incorporate the effects of both the disturbance energy ratio and the shear stress level. The model demonstrated a significant performance in both training and prediction with respect to the experimental data, and the model parameters were found to conform to a unified correlation with the disturbance energy ratio and the shear stress level.
- (5) Among the parameters of the disturbance-creep model for the anchor-soil interface, the initial shear modulus E_1 undergoes a sharp decay under low-stress and low-disturbance conditions, and particular attention should therefore be paid to the stiffness degradation of the anchor-soil interface at the early stage. The viscosity coefficient η_2 decreases sharply with increasing disturbance energy ratio, and field disturbance monitoring could be beneficial for assessing the long-term performance of anchors.

The above research provides a new tool for quantitatively characterizing the shear creep behavior of the anchor-soil interface under disturbance, and the findings can serve as a reference for the resilience design of anchorage systems. However, several limitations should be noted. The generalizability of the proposed disturbance energy ratio to other disturbance patterns or non-geomaterials requires further verification. In addition, the DSC-based disturbance-creep model does not include an explicit accelerated-creep or failure criterion, and some model parameters are strongly correlated with relatively wide confidence intervals. The

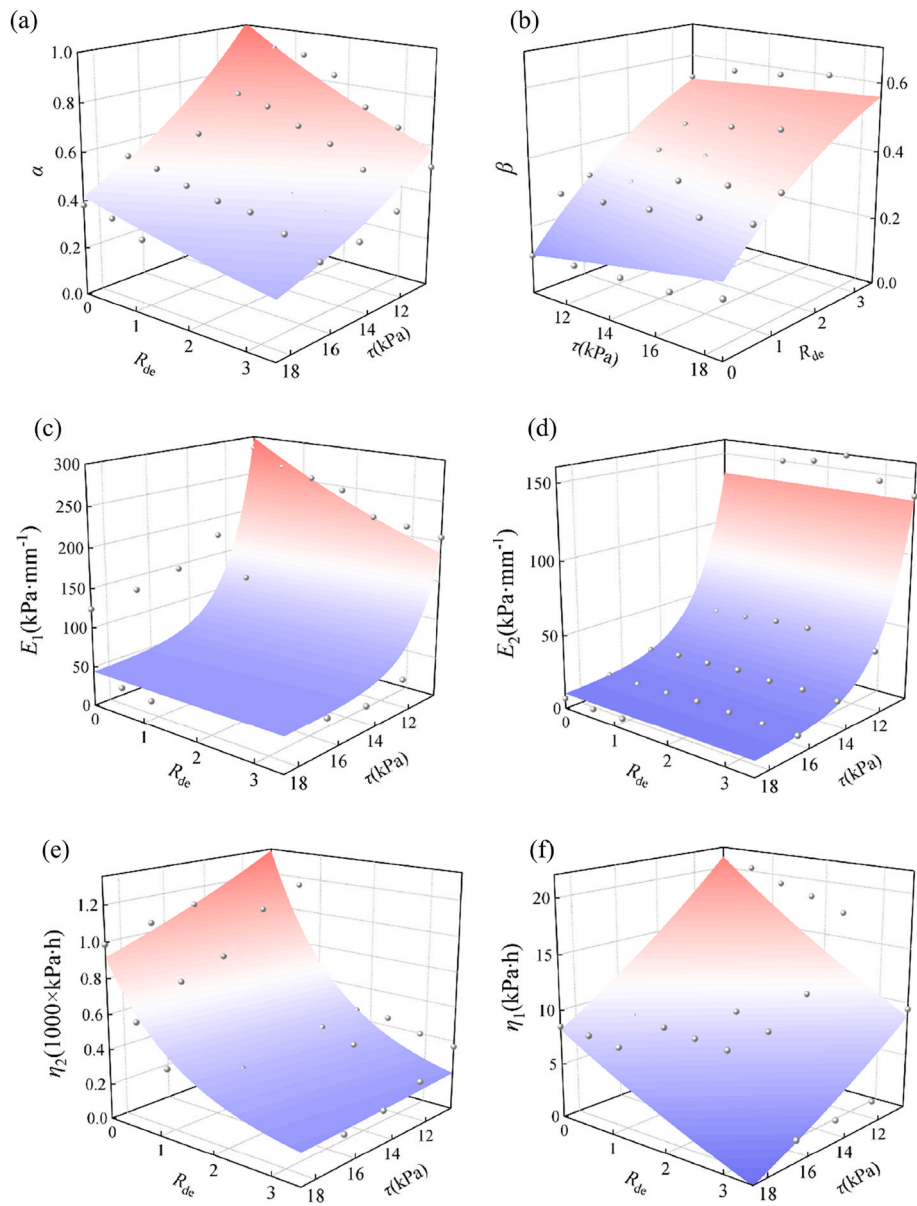


Fig. 22. Combined variation of model parameters with disturbance energy ratio and shear stress level: (a) α ; (b) β ; (c) E_1 ; (d) E_2 ; (e) η_1 ; (f) η_2 .

disturbance energy was estimated from gravitational potential energy rather than measured directly, and the assumption of uniform interface shear stress was not verified by local stress measurement. Moreover, Due to the inherent discreteness of soil materials, parallel replicate tests were not conducted. Future work will extend the tests to other soils and impact patterns, perform direct energy calibration and local stress measurement, supplement replicate and independent validation, and incorporate a failure criterion and anchor configuration effects into the model.

Table 5
Values of undetermined coefficients for the unified correlation of model parameters.

Model parameters	Correlation coefficients				R^2
	a	b	c	d	
α	-0.02	3.04	9.43	6.40	0.95
β	0.76	-0.80	51.11	3.53	0.94
E_1	42.93	284404.73	1.48	5.98	0.84
E_2	9.55	73202.48	1.63	2.17	0.95
η_1	-17.91	63.37	20.71	9.12	0.80
η_2	1148.13	20899.69	19.50	1.37	0.92

CRediT authorship contribution statement

Genbao Zhang: Resources, Methodology, Funding acquisition, Conceptualization. **Tianchong Feng:** Writing – original draft, Investigation, Formal analysis, Data curation. **Yibing Liu:** Visualization, Validation. **Haibin Ding:** Writing – review & editing, Supervision. **Changjie Xu:** Writing – review & editing, Supervision.

Data availability

All of the data that support the findings of this study are available from the corresponding author upon request.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgments

This work was sponsored by National Natural Science Foundation (Grant number: 52578400), National Key R & D Program of China (Grant number: 2023YFC3009400), Natural Science Foundation of

Hunan Province (Grant number: 2025JJ50294), Hunan Furong Talent Program (Grant number: 2026RC3239), and Young Talent Project of Yiyang City (Grant number: 202401YC07). The authors appreciate their supports.

Appendix A

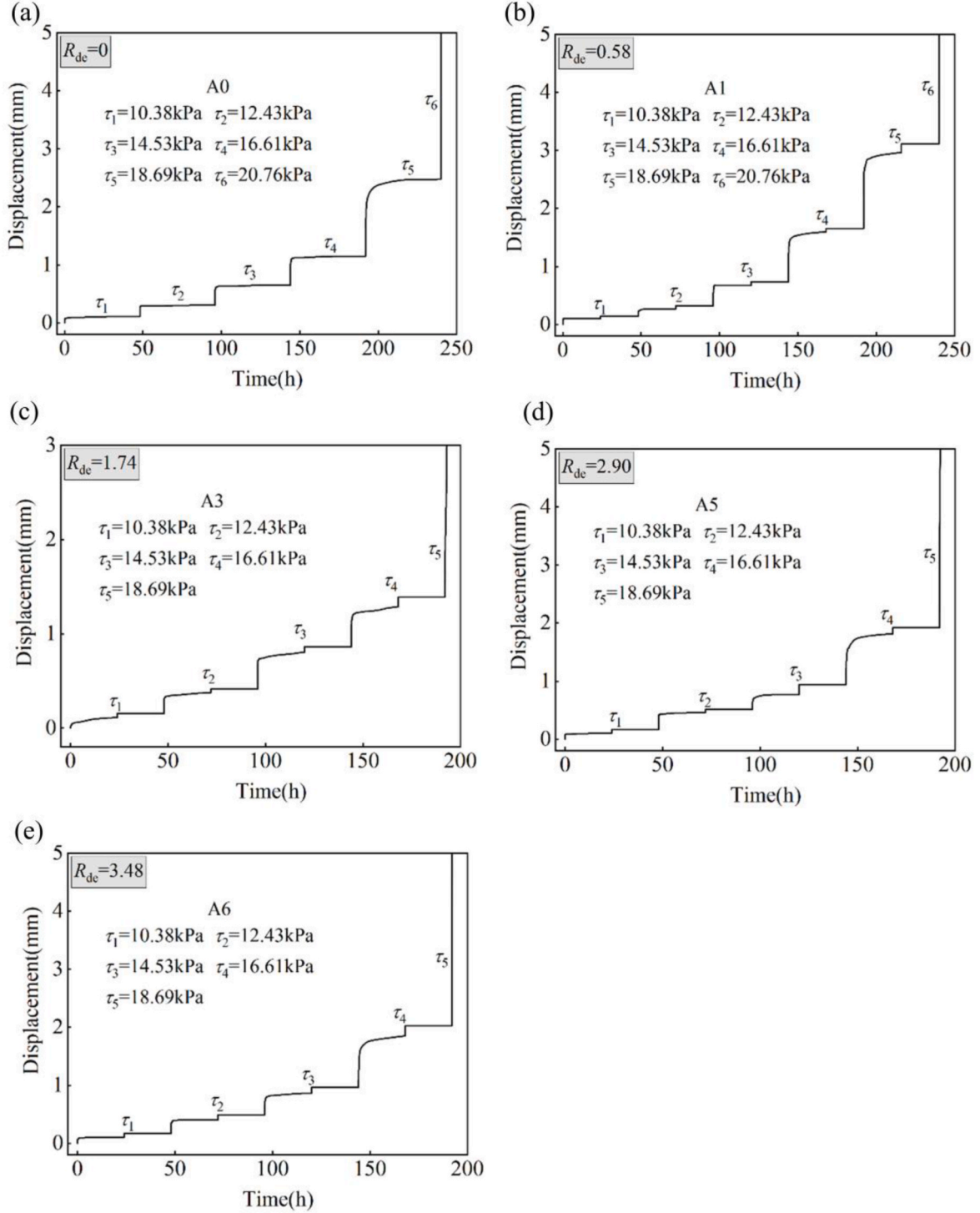


Fig. 23. Complete disturbance-creep curves of anchor-soil interface: (a) Specimen A0; (b) Specimen A1; (c) Specimen A3; (d) Specimen A5; (e) Specimen A6

Appendix B

Table 6
Parameter values of the DSC-based disturbance-creep model for anchor-soil interface under all test conditions

Specimen	τ (kPa)	Model parameters						R^2
		α	β	E_1 (kPa.mm ⁻¹)	E_2 (kPa.mm ⁻¹)	η_1 (kPa·h)	η_2 (kPa·h)	
A0 ($R_{de} = 0$)	10.38	0.98	0.11	284.3	103.9	20.4	12120	0.90
	12.43	0.75	0.12	175.6	38.0	15.4	11420	0.93
	14.53	0.62	0.13	144.2	19.5	12.4	11031	0.94
	16.61	0.53	0.14	132.0	11.7	9.1	10500	0.97
	18.69	0.38	0.17	124.2	6.9	8.5	9861	0.97
A1 ($R_{de} = 0.58$)	10.38	0.91	0.27	264.1	101.0	20.3	11540	0.90
	12.43	0.74	0.28	125.4	44.2	11.6	10540	0.90
	14.53	0.61	0.29	45.4	21.5	11.1	8346	0.91
	16.61	0.51	0.31	38.0	11.8	8.9	7534	0.93
	18.69	0.36	0.33	35.0	6.4	8.4	6006	0.90
A2 ($R_{de} = 1.16$)	10.38	0.90	0.30	254.6	150.2	19.2	5124	0.85
	12.43	0.71	0.31	50.2	45.1	11.3	4312	0.92
	14.53	0.52	0.35	39.5	22.0	9.7	4100	0.87
	16.61	0.47	0.37	30.0	12.2	8.6	4112	0.94
	18.69	0.31	0.38	31.2	6.9	8.1	3914	0.90
A3 ($R_{de} = 1.74$)	10.38	0.83	0.34	244.5	153.0	18.4	4507	0.87
	12.43	0.65	0.38	84.4	47.5	10.8	4241	0.91
	14.53	0.46	0.39	35.4	23.5	9.5	3712	0.85
	16.61	0.44	0.40	30.0	13.5	8.3	3541	0.92
	10.38	0.71	0.40	216.5	159.4	17.3	4473	0.91
A4 ($R_{de} = 2.32$)	12.43	0.60	0.44	69.7	48.1	10.6	3675	0.92
	14.53	0.44	0.46	30.5	22.4	8.4	1946	0.91
	16.61	0.43	0.48	30.1	13	8.1	1896	0.91
	10.38	0.65	0.41	211.7	145.4	9.8	3975	0.92
	12.43	0.52	0.45	33.5	42.3	2.1	2312	0.85
A5 ($R_{de} = 2.90$)	14.53	0.40	0.47	32.3	23.1	2.1	2104	0.88
	16.61	0.38	0.47	29.4	12.5	2.1	1842	0.93
	10.38	0.51	0.53	205.4	138.6	9.4	3691	0.91
	12.43	0.38	0.57	42.1	43.7	2.1	2535	0.91
	14.53	0.32	0.58	31.4	22.1	2.0	1895	0.93
A6 ($R_{de} = 3.48$)	16.61	0.31	0.6	40.4	12.3	2.0	1645	0.92

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