Load Transfer Behavior and Failure Mechanism of Bird’s Nest Anchor Cable Anchoring Structure

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Abstract: To research the internal load transfer behavior and failure mechanism of a bird’s nest anchor cable anchoring structure based on a pull-out test, a bond-slip failure model is established on the basis of statistical damage theory, and the distribution formula of shear stress at anchorage agent–rock interface is deduced. Combined with theoretical analysis, bird’s nest anchor cable pulling out test and particle flow code (PFC) numerical simulation test, as well as axial force distribution of the cable and shear stress distribution of its interface, help reveal its load transfer behavior and failure mechanism. Results show that: (1) The established bond-slip model can reflect the internal load transfer behavior and failure process of bird’s nest anchor cable anchorage structure. (2) The shear stress of the anchorage agent interface increases exponentially to the peak value and then decreases exponentially to the residual strength. The process is repeated at every location of the anchorage agent interface. The curve of the axial force and shear stress of the bird’s nest anchor cable is a negative exponential distribution with anchorage depth, and the maximum value occurs at the load end. (3) The crack of the anchorage agent interface extends from the load end to the other end and finally cuts through the whole interface. Rock mass generates radial cracks by the split effects of the bird’s nest. The failure mode is a combination of the debonding slip of the interface and the shear failure of the rock mass. The shear stress distribution and failure mode of the anchor structure are basically consistent according to laboratory tests and simulation tests, and PFC2D better reflects the internal load transfer behavior, failure mechanism, and failure process of the bird’s nest anchor cable under tensile loads.

Keywords: bird’s nest anchor cable; pulling-out test; failure mechanism; PFC; fracture process

1. Introduction

Anchor cables are a common technical means for the reinforcement of deep roadways that has been widely used in many fields, such as underground mine roadways, open-pit slopes, and foundation pits [1–3]. With the increase in mining depth, complex and variable geological structures and stress environment result in a separated roof, roof collapse accidents, and a high repair rate of anchor cables. The ejection phenomenon of broken anchor cables is common [4,5]. In addition to the influence of load and sudden external factors, the anchoring structure of anchor cables is also subject to damage accumulation, which results in bond failure, pull or shear failure of anchor parts, and other forms of anchorage system failure [6–9]. Traditional anchor cable design methods have limitations in deep roadway support [10,11]. Usually, only the axial bearing capacity of the anchor cable is considered, and the interaction of cable-surrounding rock is ignored. Therefore, a reasonable anchor cable design is of considerable significance in controlling the deformation of surrounding rock and improve safety [12,13].

At present, scholars at home and abroad mainly focus on the mechanical effect of anchor cables but ignore the influence of surrounding rock and the relative relationship between the anchor cable and surrounding rock [14]. A large number of studies show that
the deformation and failure of surrounding rocks in deep roadways are gradual [15–19]. Previous studies do not reflect the influence of the progressive failure behavior of surrounding rock and mechanical characteristics of cable wells, and the internal load transfer behavior and mechanism of anchor cable still cannot be explained quantitatively and evaluated correctly. Ordinary anchor cables are not adopted to high-deformation environments of surrounding rock. In view of the limitation of traditional anchor cables with respect to mechanical properties and severe engineering environments, new anchor cables with extraordinary mechanical properties should be developed. Existing NPR anchor cables have solved the deformation problem of deep, soft rock roadways [20,21]. The deformation capacity increases to 3000–4000 mm, and the maximum constant resistance is 567.7–800 kN [22]. Some scholars have studied the failure mechanism of anchor cable systems using numerical software, achieving remarkable results. Through theoretical analysis and simulation verification, the mechanical properties of anchor cables under tension–torsion coupling action have been explored, and the tension–torsion coupling coefficient has been proposed and determined [23]. The surrounding rock characteristics of steel arch supports and NPR anchor cable supports are compared with 3DEC software [24].

Bird’s nest anchor cables are a new type of anchor cable. Compared with ordinary anchor cables, they have higher anchor force and elongation. The synergistic principle and evaluation method of rock surrounding bird’s nest anchor cables are still in the research stage, and the failure process and the mechanisms of anchor structures are not clearly understood. The influence of different anchor cable supporting parameters on the stress diffusion of surrounding rock is studied by FLAC3D, and optimal parameters are proposed [25]. An anchor cable and C tube withstand transverse shear force are developed, and the new scheme can effectively control surface convergence and plastic zone expansion by FLAC3D [26]. Lagging theoretical research results in some accidents during construction or after completion [27,28]. Designers do not have a clear understanding of its action mechanism to deal with the design and application of bird’s nest anchor cables. The failure process of rock surrounding bird’s nest anchor cables is analyzed and compared with theoretical analysis as well as pull-out tests and PFC simulation tests. Furthermore, the crack evolution process and failure mechanism of bird’s nest anchor cables and rock mass, which have certain theoretical significance and practical value in terms of guiding engineering practice, are revealed.

2. Pull-out Test of Bird’s Nest Anchor Cables
2.1. Production of Anchorage Structure

(1) Material selection

To study the deformation and failure characteristics of bird’s nest anchor cable anchoring structures, a bird’s nest with 1 × 7 strands 17.8 mm, 1720 steel strands, 1200 mm length, 300 mm spacing, and 17.8 mm diameter anchor cable is selected. The specifications and mechanical property parameters of the bird’s nest anchor cable are shown in Table 1. The anchorage agent is a MSZ2835 medium-speed resin with a diameter of 28 mm and a single-reel length of 350 mm. To eliminate the size effect, polyvinyl chloride (PVC) pipe is used to anchor samples of different sizes, including 1250 mm height, with 110, 200, and 250 mm outer diameters. The center hole is 1500 mm high, and the outer diameter is 32 mm.

![Table 1. Specifications and mechanical property parameters of the bird’s nest anchor cable.](image)

(2) Preparation of anchor structure

The ratio and strength of similar materials are determined by multiple ratio tests (Table 2). The test material includes 32.5 Portland cement and 0–15 mm well-graded river
sand, and the rock mass is made with M15 mortar with a mixture ratio of cement, sand, and water of 1:4.7:1. Strain gauges are pasted inside the anchor cable and rock to monitor the stress–strain of the anchor cable and rock. The rock mass is poured three times, and the strain gauges (BE120-03AA type) are pasted and marked in the designated positions.

Table 2. Ratio and strength of similar materials.

| Amount/mm³ | Density | Mean Fraction of Loss Intensity |
|------------|---------|-------------------------------|
| Portland cement/kg | River sand/kg | Water/kg | 2000 kg/m³ | 18 MPa |
| 330 | 1550 | 330 |

The self-made measuring force bolt is prepared before performing the pull-out test. Six pairs of strain gauges are slotted symmetrically on both sides of the bolt cable with 150 mm equal spacing. To avoid the destruction of strain gauges during the process of rotating installation and tension, the strain gauges and wire joints are sealed with 704 silicone rubber and then covered with EP30 Osborne AB glue. The well-tested measuring force bolt is shown in Figure 1.

Figure 1. Measuring point figure of bird’s nest anchor cable.

The specific preparation is shown in Figure 2, as follows: ① Initial pouring. After placing the 32 mm PVC pipe in the center of the model, the allocated mortar is poured into a PVC pipe with a 110 mm outer diameter and vibrated and poured by plug-in vibrator. The cast similar material model is left for 7 days and then demolded and cured for approximately 7 days. The strain gauge is pasted and marked on the sample surface. ② Second and third castings. After the surface is roughened, a tube with an outer diameter of 200 mm PVC is set on the outside of the poured model, and the above casting process is repeated. The second and third casting processes are then repeated. ③ Cover the outer tube. ④ Installing the anchor agent. The anchor agents are sent into the reserved hole, and the model is finished.

2.2. Test Device

The load equipment is an RRH-6010 double-acting hollow jack with 0.75 kW electric pump power and a 257 mm span. It is composed of a monitoring device, as well as data acquisition and processing system (Figure 3), which can realize real-time data acquisition, dynamic display, real-time chart drawing, and other functions.

A 22-channel analysis system of DH3818N-2 static signal test is used to collect the strain gauge data of the anchor cable and rock sample. Other monitoring devices are composed of a BLR-1/50T pressure sensor and BL100-V-1000 displacement sensor.

2.3. Analysis of Pull-Out Test Results

(1) Shear stress and displacement curve

When the anchorage agent length is short, the shear stress on the anchorage interface can be considered uniform; that is, the shear stress at any point of the anchorage interface is

\[ \tau = \frac{Pdl}{\pi} \]
Figure 4 shows research results [29–31] based on shear stress (τ) and relative shear displacement (ω).

![Diagram](image-url)

**Figure 2.** Depiction of the finished anchorage structure. (a) Sketch of the anchorage structure; (b) 1–1 cross-section program of the anchorage structure.

![Diagram](image-url)

**Figure 3.** Pulling-out cable test system.

When the ascending stress value reaches peak stress, a linear softening curve occurs until debonding is initiated, followed by a horizontal line that represents the residual strength after debonding is completed (Figure 5).

According to [32,33], the statistical damage constitutive model of the anchorage structure based on Weibull distribution and the Weibull distribution parameter are expressed as follows:

\[ \tau = k_1 \omega \exp \left( -k_2 \left( \frac{\omega}{\omega_1} \right)^{k_3} \right) + \beta \tau_p, \]

where \( \omega_1 \) is displacement, corresponding to peak strain; \( \tau_p \) is the peak stress; and \( \beta \) is the influence coefficient of residual strength (Figure 5a). \( \omega < \omega_1 \) is referred to as the nonlinear...
elastic stage. When \( \omega_1 \leq \omega < \omega_2 \), the anchorage agent separates from the rib of the bolt, which is referred to as the debonding slip stage. The interfacial adhesion and interlock force disappear simultaneously in the debonding segment, whereas the interfacial friction force gradually approximates the residual strength. When \( \omega > \omega_2 \), the frictional force depends on the friction surface of the anchorage agent. The period when the shear stress is equal to the residual shear strength of the cable and debonding appears is referred to as the debonding stage. Before the debonding of the model interface, the bond–slip curve is linear, and no damage occurs, so \( \beta = 0 \) \((k_1, k_2, k_3, \text{and } \beta) \) are some parameters of the formula derivation that can be obtained by fitting the experimental data (Figure 5b). The parameters of the ascending stages are 120.9908, 0.2680, 1.25, and 0; the parameters of the descending stages are 0.2213, −4.184, −0.2465, and 0; and the parameters of the horizontal stages are 0, 0, 0, and 0.195.

Figure 4. Bond-slip models. (a) Model by Cai et al. (b) Model by Ren et al. (c) Model by Ma et al. \( \tau_u \) and \( \tau_r \) are the peak shear strength and residual shear strength, respectively, of a rock bolt.

Figure 5. Shear stress–displacement curve of the proposed model. (a) Bond–slip models of the proposed model. (b) Fitting curve, as well as experimental and theoretical curves.

(2) Distribution law of anchor cable axial force

The obtained axial force distribution curve of the anchor cable under ultimate load (148 kN) is shown in Figure 6. The axial force of the anchor cable gradually decreases along the load end. The peak value is 148 kN, and the minimum value is 28 kN (Figure 6).

(3) Distribution law of shear stress at the anchoring–rock interface

The obtained shear stress distribution curves of the anchor cable under three loads are shown in Figure 7. The shear stress of the anchor cable gradually decreases along the anchorage depth. The ultimate shear stress value is 2.37 MPa, and the minimum value tends toward 0. When the axial force is 60 kN, shear stress decreases gradually along the anchorage depth. When the axial force is 100 kN, the shear stress increases first and then
decreases. When the axial force is 148 kN, every point of the anchorage agent interface goes through an elastic deformation stage, a debonding slip deformation stage, and a residual deformation stage.

![Figure 6. Axial force distribution of the cable.](image)

![Figure 7. Shear stress distribution law of measuring point at the anchorage agent interface.](image)

(4) Distribution law of shear stress at the rock mass

The distribution law of shear stress in the rock mass when the peak value is 148 kN is shown in Figure 8 with a comparison of the two sets of data. The shear stress near the cable are higher than those far away in the rock mass, and the shear stress of the load end is higher than that of the other end, and the shear stress values of the two interfaces decrease simultaneously.

(5) Fracture mode

Figure 9 shows the final failure mode of the anchoring structure. The cable does not break under tensile load, but the anchorage agent and cable slip at the interface of the rock mass and the anchorage agent. Due to the strong adhesion and shear strength between cable and anchor agent and the mechanical interlocking between bird’s nest and anchor agent, it is difficult for relative slip to occur. When the shear stress at the load end is greater than the shear strength at the rock mass anchorage agent interface, internal surface of the rock mass exhibits a radial extension crack caused by the bird’s nest dilatation effect and anchorage agent extrusion, and the brittleness of similar material influences the spread range of the crack. The rock mass and anchorage agent interface is the most sensitive and vulnerable part of the whole anchorage system because of the similar material properties.
115,245 particles, including 103,567 rock mass particles and 11,540 anchor agent particles.

Figure 8. Shear stress distribution of the rock sample.

Figure 9. Failure of the anchoring structure. (a) Cross section; (b) Longitudinal section.

3. PFC Numerical Simulation Test

3.1. PFC Model

The model of the bird’s nest anchor cable is established by PFC2D. The model is a cylinder with a base diameter of 250 mm and length of 1200 mm. It is composed of 115,245 particles, including 103,567 rock mass particles and 11,540 anchor agent particles.
Moreover, the rock mass and anchorage agent are represented as gray and blue particles, respectively. The anchorage agent and rock mass are composed of particles with a radius of 0.5–0.85 mm. The cable is composed of 138 particles with a diameter of 17.8 mm and 3 particles with a diameter of 20 mm. According to the measuring point locations, three measuring circles are arranged in the anchorage agent to record the shear stress distribution, and four measuring circles are arranged on the anchorage agent interface to record the shear stress distribution of the anchorage agent interface (Figure 10).

Figure 10. Model and measuring point of PFC simulation test.

To highlight crack, tension and shear cracks are presented as green and red, respectively. The tensile strength and bond force of the rock mass anchored by the PFC are obtained by a uniaxial compression test and Brazilian splitting test and have a value of 3.5 MPa and 3.0 MPa, respectively. The physical and mechanical parameters of the resin anchorage agent and anchor cable are provided by the manufacturer and are equal to 3.0 MPa and 2.5 MPa, respectively. The parameters are constantly adjusted to calibrate the mechanical parameters of similar sample materials. Table 3 shows the parameters of similar materials used in PFC2D.

Table 3. Main parameters of PFC2D in the pull-out test.

| Particle Parameter            | Numerical Value | Parallel Bond Model Parameters        | Numerical Value |
|------------------------------|-----------------|---------------------------------------|-----------------|
| Particle radius (mm)         | 0.5–0.85        | Parallel bond modulus                 | 1.4 × 10^9      |
| Density (kg/m^3)             | 1.4–1.9 × 10^3  | Normal and tangential stiffness ratio  | 2.0             |
| Contact modulus between particles | 1.2 × 10^9     | Normal critical damping ratio          | 0.5             |
| Normal and tangential stiffness ratios | 1.0            | Normal tensile strength               | 5.1 × 10^6      |
| Coefficient of friction      | 0.58            | Cohesive force                        | 3.1 × 10^6      |

3.2. Failure Process and Failure Mode

Figures 11 and 12 show the axial force distribution law and shear stress evolution process of measuring point by PFC simulation. The results of PFC simulation are consistent with the pull-out test result, as shown in Figures 5–7, 11 and 12. Steps 2600 and 10,000 are the boundary lines of the elastic deformation stage and bird’s nest split rock failure stage, respectively.

Combined with the pull-out test result and PFC simulation result, the crack evolution processes are divided into three stages, namely the elastic deformation stage, the debonding slip stage, and the split failure of the bird’s nest, as shown in Figures 12–14. The failure process is explained as follows:

Cracks can be divided into three stages, as shown in Figure 14. Stage I (elastic deformation stage): before step 1350, shear crack dominates the evolution process of cracks. In steps 1350–2600, tension cracks exceed shear cracks. The curve of this stage rises, and the shear stress–slip curve is in an elastic state. The bond between the anchor cable and anchor agent mainly consists of chemical bond force, friction force, and mechanical interlocking force. Due to the relatively small tensile load, the anchorage agent is the elastic coordinated deformation stage. When the shear stress of interface reaches shear stress strength, the anchor cable moves the small slip at the load end, which results in the gradual loss of chemical bond force of the anchorage agent, and the anchor agent interface exhibits a few cracks. Before cracks develop near the bird’s nest, cracks extend inwards with anchorage...
agent depth, but the rock mass does not fail. After cracks develop near the bird’s nest, they continue to extend inwards, and rock mass failure occurs.

![Figure 11. Axial force distribution law of measuring point.](image1)

![Figure 12. Shear stress evolution process of measuring point.](image2)

Stage II (debonding slip stage): loading occurs at steps 2600–10,000, the tension crack effect dominates, and shear crack number has a constant value. With additional debonding length, the tensile load of the anchor cable decreases. The relative slip of anchoring interface is no longer coordinated with the increase in load. The shear strength of the bird’s nest anchor cable is composed of adhesion, mechanical interlocking, and friction in the axial direction at the coupling interface between the anchor cable and the anchorage agent. As the load increases, interfacial slip and shear dilation appear, and in turn, the interfacial radial force and friction force, as well as the peak shear strength, are elevated. A debonding region occurs at the load end and gradually expands to the other end.

Stage III (bird’s nest split rock failure stage): After step 10,000, the shear crack and tension crack number are basically a constant value, and a few cracks extend along the rock mass depth. The tensile load of the anchor cable declines significantly up to the residual tensile stress. The friction force and the extruding force of the bird’s nest hinder the anchor cable from being pulled out. At this stage, the interface of the bird’s nest generates radial cracks, and cracks rapidly expand the rock mass. The radial stress produced by mechanical biting force balances the annular tensile stress of the anchor agent, with only friction resistance remaining. The anchorage agent is sheared, and rock crumbs are drawn unceasingly with the pull-out of the anchor cable.
Figure 13. Failure process of the anchoring structure during PFC simulation test. (a) Step 250 (crack number 4000, shear crack 2500, tension crack 1500); (b) step 1250 (crack number 28000, shear crack 16000, tension crack 12000); (c) step 2400 (crack number 58000, shear crack 19000, tension crack 39000); (d) step 2600 (crack number 62000, shear crack 20000, tension crack 42000); (e) step 10000 (crack number 87000, shear crack 21000, tension crack 66000).

The failure mode is a mixed failure between debonding slip of the interface and shear failure of the rock mass (Figure 9). PFC2D can better reflect the internal load transfer behavior, failure mechanism, and failure process of the bird’s nest anchor cable under tensile loads.
4. Conclusions

(1) A bond-slip model of bird’s nest anchor cable is established on the basis of a statistical damage constitutive model. The shear stress and slip curve of the anchorage structure are divided into a nonlinear elastic deformation stage, a debonding slip stage, and a residual stage. With an increase in the pull-out load, the shear stress of the anchorage agent interface increases exponentially to the peak value from the load end, then decreases, and finally stabilizes to the residual strength. The process transmits every point with the anchorage depth. The model parameters are acquired by fitting data, and the theory curve and test curves are similar, thereby verifying the reasonability of the proposed mode.

(2) The axial force of the anchor cable declines negatively along the load direction; the maximum value (148 kN) is near the load end, and the minimum value (28 kN) is at the other end. The shear stress of the rock mass decreases negatively along the load direction and is transferred from the load end to the other end, but the shear stress value near anchorage cable value is higher than that far away from the anchorage structure.

(3) The failure of the anchorage structure is divided into three stages according to PFC simulation: an elastic deformation stage, a debonding slip stage, and a splitting rock failure. The anchorage structure of the load end first exhibits a small number of cracks (crack number: 4000), then extends along the interface of the anchorage agent (crack number: 62000), and then penetrates into the anchorage agent–rock mass interface (crack number: 87000). The failure mode of the anchoring structure is mixed failure, namely debonding slip of the interface and shear failure of rock mass. PFC can simulate the crack evolution process of the bird’s nest anchor cable well.

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