

Study on the application of composite support technology combining anchor cables and Micro-Pile clusters for the stabilization of Earth slides

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Article Info	Abstract
Article History: Received 21 Aug 2026 Accepted 12 Oct 2026 Keywords: Anchor micro pile group; Soil landslide; Reinforcement technology; Collaborative load-bearing mechanism; Pasternak foundation model; Numerical simulation	Soil landslides have strong suddenness and high destructive potential, while traditional reinforcement technologies are limited by large construction disturbance and poor terrain adaptability. This study focuses on a typical silty clay landslide in Jiangxi Province, China. Anchor cable micro-pile clusters are a new composite retaining structure applied in landslide control; however, their three-dimensional collaborative bearing mechanism and design methods remain unclear. Based on Pasternak's two-parameter foundation beam theory and the ideal elastic-plastic p-y curve of soil around piles, this study establishes a theoretical analytical framework considering pile-soil-anchor cable interactive deformation. FLAC3D numerical simulations and physical model tests with a length scale of 1:10 and a diameter scale of 1:5 was conducted to investigate the effects of pile diameter, spacing, and anchor-cable tension on foundation bearing capacity. Taking the same micro-pile group without anchor cable prestress ($T = 0$ kN) as the traditional micro-pile baseline, the optimized anchor cable micro-pile cluster at the recommended prestress of $T = 50$ kN increases the safety factor from 1.52 to 1.78, i.e., by 17.1%, and under the ultimate loading stage, reduces the pile-top displacement from 15.6 mm to 8.7 mm, i.e., by 44.2%. The optimal spacing is five times the pile diameter, with a pile group efficiency coefficient of 0.92. When anchor prestress exceeds 50 kN, reinforcement benefits gradually decrease, and 50–60 kN is recommended. Plum-shaped pile arrangements outperform rectangular layouts, and three rows of piles can satisfy anti-sliding requirements. The calculation error is less than 5%, demonstrating its applicability for slope stabilization engineering in similar silty clay landslides.

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1. Introduction

Soil slope collapses are sudden and severe, so there is a serious threat to life, property and construction works. The traditional reinforcement technology has large construction disturbance, large material consumption and poor terrain adaptability, and it is difficult to balance economy and safety under complex conditions. The anchor cable micro-pile group (ACMPG) forms a three-dimensional reinforcement system through the cooperation of micro-piles and prestressed anchor cables. It has a thin shape, is simple to install, does not cause much disruption, and is an excellent, original way to stabilize soil slopes.

In the aspect of landslide initiation mechanism and stability analysis, Hartwig et al. [1] revealed the progressive failure mechanism of shallow landslide induced by soil strength degradation through the case of Serra da Aliança mountain chain in Brazil, and pointed out that the loss of cohesion is the core incentive for the formation of slip zone. Yu [2] have used the discrete element method to model the progressive failure of soil landslides with weak interlayers under unsaturated hydraulic

conditions and found that reducing the matric suction can increase the shear stress at the critical interface by as much as 2.1. Jia [3] constructed a limit equilibrium model considering unsaturated seepage and groundwater level fluctuation, and confirmed that the sensitivity of slope geometry to safety factor can reach 30%. Guo et al. [4] used the strength reduction finite element method to find the position of the main slip surface in a double-layer soil slope and determined that modifying the dip angle at the interface can increase the safety factor by as much as 0.35.

In terms of seepage erosion and reservoir-induced landslides, Zhao et al. [5] have applied the particle flow method to model the failure of the landslide dam due to hydrological erosion. They found that the structure damage caused by the movement of fine particles can reduce the shear strength of the barrier by 45%. Li et al. [6] investigated the soil arching process of stacked landslides under different reservoir water heads and found that there is a temporal-spatial correlation pattern between the internal force redistribution of anti-slide piles and the deformation field of the sliding mass.

In terms of micro-pile and anchor cable composite retaining structures, Wang et al. [7] have put forward a three-dimensional slope stabilization evaluation method for anchor cable micro-pile arrays that takes into account the total deformation along the load transfer path and upper-bound evaluation techniques, and established ACMPG resistance calculation formula and three-dimensional slope safety factor explicit equation. Zhu et al. [8] have developed an improved computational scheme based on Pasternak's two-parameter foundation beam theory to correct the problem of neglecting plastic deformation of the soil before embedded piles in the previous model and ensure continuous stress and displacement gradients in the surrounding soil. A finite-difference method will be employed to build the numerical model of this project and investigate the effect of soil shear modulus.

In terms of experimental and numerical simulation verification, Li et al. [9] studied the failure mode and stress distribution law of shallow soil landslide reinforced by micro-piles through three sets of physical model tests, and determined the three stages of landslide sliding failure and the stress distribution characteristics of micro-piles. Ren et al. [10] have found that the combination of anchor cables and micro-piles can improve both the stability and the anti-sliding capacity of slopes considerably, and they have used numerical models to compare the stress behavior of anchor cable-micro-pile arrays with that of conventional micro-pile sets under landslide forces. Ke et al. [11] have also carried out anti-slip capacity tests on the supporting effect of the pile group assembly with micro-piles and found that a pile cluster structure of 150mm in diameter and 2m apart can reduce the residual force of the sliding mass by 63%.

In terms of regional landslide characteristics and optimization of prevention and control technology, Liu et al. [12] have listed the sequence of failure for small soil slides in eastern Jiangxi and put forward a stability assessment method based on cumulative precipitation-duration dual indices. Li [13] has classified all the responses of the rheological characteristics of sliding-zone soil to prevention and control measures, and pointed out that the micro-pile composite structure has significant technical advantages in the treatment of shallow landslides. Based on the above research, the load-bearing capacity of anchor micro-pile clusters is affected by the combined effects of pile-soil interaction, pile array effect, transfer efficiency of anchor cable prestress and other factors.

As shown in the previous studies, many scholars have studied the anti-slip ability of micro-piles and have achieved certain results on the static properties of individual piles or small groups of piles. It should be noted that the cited studies emphasize seepage, matric suction and groundwater fluctuations as landslide triggers, whereas these water effects are not explicitly modeled here. This study focuses on the collaborative bearing mechanism and parameter optimization of ACMPG under a given landslide thrust. Dry/saturated Mohr–Coulomb conditions are therefore adopted, and the results are most applicable to slopes under drained or relatively stable groundwater conditions. Moreover, the existing analytical models have clear limitations: the Winkler model neglects soil shear, the Pasternak model still relies on empirical parameters, and continuum models are computationally demanding and sensitive to interface parameters. Based on these gaps, the scientific hypothesis of this study is that the collaborative bearing capacity of ACMPG is governed

by the coupled effects of pile–soil interaction, pile group effect and anchor cable prestress, and that an optimal parameter combination exists for stabilizing shallow soil landslides. This hypothesis is tested through theoretical analysis, physical model tests and numerical simulations.

This study takes a typical soil landslide in Jiangxi Province as the object, and focuses on the optimization of bearing capacity of soil landslide reinforced by anchor cable Micro-Pile group. The innovations of this paper mainly include: (1) According to Pasternak's Two-Parameter Foundation Beam Theory, a three-dimensional computational framework for the assembly of anchor-cable micropiles has been established that considers the elastic-plastic behavior of the soil in front of the piles and continuous deformation along the lateral side, thus solving the problem of ignoring soil shear effects in the Winkler model. (2) Through many extensive physical model experiments in various conditions and various numerical analyses, the effect of important factors such as pile size, pile spacing and anchor-cable stress on the load-bearing capacity of the integrated framework has been systematically studied, and the exact effect of changing the spatial arrangement of the pile group on the redistribution of slope loads has been determined. (3) A set of technical methods covering design parameter optimization, construction process control and performance monitoring has been established to provide a theoretical basis for solving the problems of the traditional empirical design method.

2. Theoretical Methods

2.1 Basic Assumptions and Mechanical Model

The whole retaining frame and the anchor cable are made up of micro-piles, a pile-top slab and prestressed anchor cables. A sliding joint is set between the loaded area and the anchored area of the pile array. The structure of the mechanical analysis for anchor cable micro-pile clusters is shown in Fig. 1. (1) The first is that the micro-pile is assumed to be an elastic foundation beam, which satisfies the small deformation assumption; (2) The soil between piles in the loading section is simplified as a horizontal spring; (3) A good elastic-plastic p-y curve will be used for the embedded part [14]; (4) Anchor cables are treated as elastic supports, and the cable force is distributed evenly at the pile top via the slab; (5) According to the Pasternak two-parameter foundation beam method, the shear modulus G of the soil is determined so that the ground pressure and strain at the pier are consistent [15-16]. Fig.1 is the above mechanical model diagram, which clearly divides the loaded section and the embedded section, and marks the pile top anchor cable constraint, pile bottom boundary, landslide thrust distribution and soil spring.

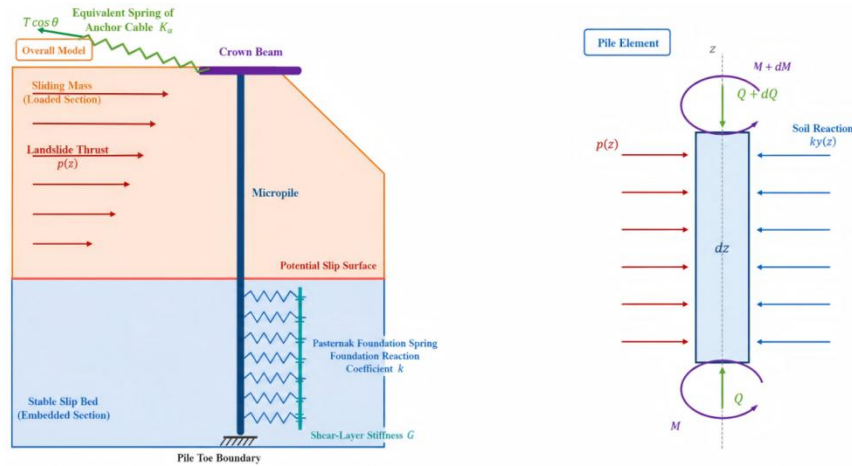


Fig. 1. Mechanical calculation model of anchor cable Micro-Pile group

The pile is divided into two sections by the potential slip surface at $z = H_s$: the loaded section above the slip surface and the embedded section below it. The loaded section is modelled by the Winkler foundation beam, while the embedded section is modelled by the Pasternak two-parameter foundation beam. At the sliding surface, the following continuity conditions must be satisfied:

$$y_I(H_s) = y_{II}(H_s); \quad \theta_I(H_s) = \theta_{II}(H_s), \quad \text{i.e., } y_I'(H_s) = y_{II}'(H_s); \quad M_I(H_s) = M_{II}(H_s), \quad \text{i.e., } EI_I y_I''(H_s) = EI_{II} y_{II}''(H_s); \quad Q_I(H_s) = Q_{II}(H_s), \quad \text{i.e., } EI_I y_I'''(H_s) = EI_{II} y_{II}'''(H_s).$$

Since the same micro-pile is used in both sections, $EI_I = EI_{II} = EI$, and the last two conditions reduce to $y_I''(H_s) = y_{II}''(H_s)$ and $y_I'''(H_s) = y_{II}'''(H_s)$. These four continuity conditions are enforced at the interface node in the finite difference scheme.

2.2 Differential Equation of Pile Deflection

According to the Pasternak two-parameter foundation model (Fig. 2), the displacement differential equation for micro-piles can be written as Equation (1):

$$EI \frac{d^4 y(z)}{dz^4} - GA \frac{d^2 y(z)}{dz^2} + k y(z) = p(z) \quad (1)$$

In the formula, EI is the flexural rigidity of the micro-pile ($\text{N} \cdot \text{m}^2$), E is the stiffness modulus of the pile material, I is the second moment of area, $y(z)$ is the lateral displacement at the level z of the pile (m), G is the shear modulus of soil in Pasternak theory (N/m^2), A is the shear strain area of soil per unit length (m^2), GA is the shear layer stiffness (N), k is the subgrade reaction modulus (N/m^2), and $p(z)$ is the distributed load along the pile (N/m). Dimensional consistency: $[EI \frac{d^4 y}{dz^4}] = \frac{\text{N} \cdot \text{m}^2 \cdot \text{m}}{\text{m}^4} = \frac{\text{N}}{\text{m}}$, $[GA \frac{d^2 y}{dz^2}] = \frac{\text{N}}{\text{m}^2} \cdot \text{m}^2 \cdot \frac{\text{m}}{\text{m}^2} = \frac{\text{N}}{\text{m}}$, $[ky] = \frac{\text{N}}{\text{m}^2} \cdot \text{m} = \frac{\text{N}}{\text{m}}$, $[p(z)] = \frac{\text{N}}{\text{m}}$.

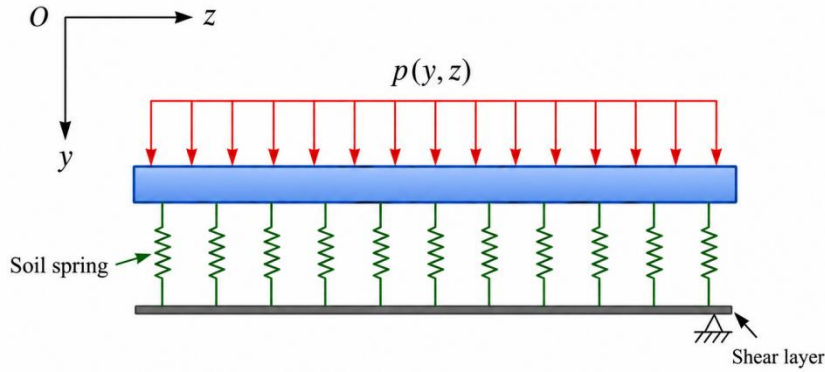


Fig. 2. Pasternak two-parameter foundation model

For the loaded section, the main cause of the weight on the pile structure is the slope pressure, which can be considered according to the triangular or trapezoidal distribution [17], as shown in Formula (2):

$$p(z) = p_0 \cdot f(z) \quad (2)$$

In the formula, p_0 is the landslide thrust strength at the sliding surface, and $f(z)$ is the load distribution function along the depth direction of the pile. For the loaded section, the landslide thrust is assumed to follow a triangular or trapezoidal distribution. Let H_s be the thickness of the sliding mass above the potential slip surface, and let z be measured downward from the pile head. The load distribution function $f(z)$ is explicitly given as:

Triangular distribution: $f(z) = \frac{z}{H_s}$, $0 \leq z \leq H_s$; Trapezoidal distribution: $f(z) = f_0 + (1 - f_0) \frac{z}{H_s}$, $0 \leq z \leq H_s$; Rectangular distribution: $f(z) = 1$, where f_0 is the ratio of the thrust intensity at the pile head to that at the slip surface, with $0 \leq f_0 \leq 1$. For the embedded section, $p(z)$ represents the soil resistance in front of the pile and is calculated by the p-y curve rather than by $p_0 f(z)$.

2.3 Constitutive Model of Pile-Soil Interaction

The resistance of the soil in front of the embedded pile is described by the ideal elastic-plastic p-y curve [18], as shown in Formula (3), and the elastic-plastic p-y curve is shown in Fig.3.

$$p(y) = \begin{cases} k_h \cdot y, & y \leq y_u \\ p_u, & y > y_u \end{cases} \quad (3)$$

In the formula, k_h is the horizontal foundation reaction coefficient, y is the lateral displacement of the pile column, y_u is the yielding offset at the maximum strength, and p_u is the ultimate soil support per section of a single pile.

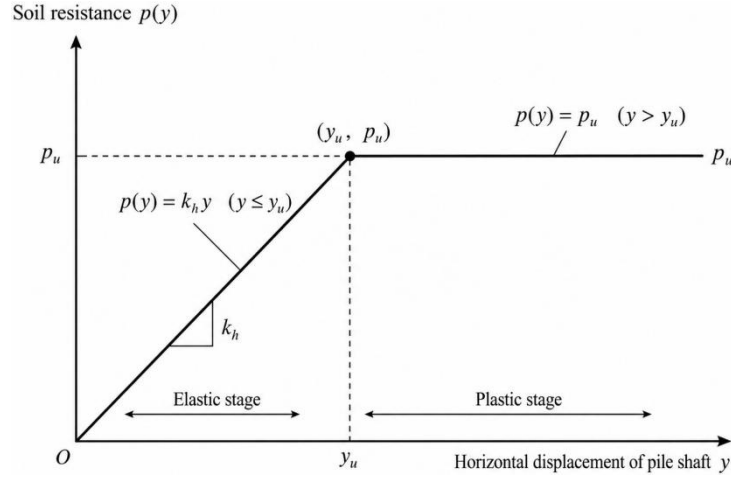


Fig. 3. Elastoplastic p-y curve

According to the p-y curve for Matlock soft clay, the ultimate bearing capacity p_u of the soil can be obtained from equation (4):

$$p_u = N_p \cdot c_u \cdot D \quad (4)$$

In the formula, N_p is the maximum bearing capacity factor; for surface clay, $N_p = 3 + \frac{\gamma z}{c_u} + \frac{Jz}{D}$, where J is an empirical parameter, c_u is the undrained shear strength, γ is the specific weight of the soil, z is the burial depth, and D is the micro-pile diameter.

2.4 Anchor Cable Prestress Equivalent Treatment

It is assumed that the array of tensioned cables at the bridge apex is a rigid support, and the corresponding spring modulus K_a is given by Equation (5) [19].

$$K_a = \frac{E_a A_a}{L_a} \quad (5)$$

In the formula, E_a is the elastic modulus of the anchoring cable, A_a is the cross-sectional area of the cable, and L_a is the free length of the anchoring cable.

The tension T of the anchor cable is transmitted to the upper end of each row of piles by the crown beam, and the specific nodal load is given by Equation (6):

$$F_i = \alpha_i \cdot T \cdot \cos \theta \quad \alpha_i = \frac{K_i}{\sum_{j=1}^n K_j}, \quad \sum_{i=1}^n \alpha_i = 1 \quad (6)$$

In the formula, α_i is the load distribution coefficient of the i -th row of piles, which is determined by the stiffness of the crown beam and the stiffness of each row of piles. θ is the angle that the anchor cable makes with the horizontal ground. K_i is the equivalent lateral stiffness of the i -th row of piles. Based on the FLAC3D calibration, for the three-row quincunx arrangement with $s=5D$, the load distribution coefficients are $\alpha_1 = 0.403$, $\alpha_2 = 0.352$, and $\alpha_3 = 0.245$.

2.5 Pile Group Effect Coefficient

The group effect of piles is the interaction factor among individual piles. Multi-row micro-pile assemblies are, the equivalent foundation reaction coefficient of the i -th row of piles can be corrected to Formula (7) [20]:

$$k_{\text{eff},i} = \eta_i \cdot k \quad (7)$$

In the formula, $\eta_i = 1 - \sum_{j \neq i} \beta_{ij} \cdot \frac{D}{s_{ij}}$, β_{ij} is the interaction coefficient of the i -th and j -th piles, and s_{ij} is the spacing between piles, and s_{ij} is the pile spacing. For the commonly used plum-shaped or rectangular pile arrangement, β_{ij} can be determined by elastic theory or numerical calibration. To make the calculation reproducible, the interaction coefficient β_{ij} was calibrated by FLAC3D numerical simulations in this study. For the rectangular arrangement, the calibrated values adopted are as follows. When $s/D=3$, $\beta_{12} = 0.42$, $\beta_{23} = 0.38$, and $\beta_{13} = 0.18$. When $s/D=4$, $\beta_{12} = 0.30$, $\beta_{23} = 0.27$, and $\beta_{13} = 0.12$. When $s/D=5$, $\beta_{12} = 0.18$, $\beta_{23} = 0.16$, and $\beta_{13} = 0.07$. When $s/D=6$, $\beta_{12} = 0.10$, $\beta_{23} = 0.09$, and $\beta_{13} = 0.04$. For the quincunx arrangement, the calibrated β_{ij} values are approximately 20%-30% lower than those for the rectangular arrangement because of the staggered layout. Substituting the calibrated values at $s/D=5$ into Eq. (7) gives $\eta = 0.92$.

2.6 Finite Difference Numerical Solution

The above differential equations can be solved numerically with the finite difference method. There are n points along the depth of the micro-pile, and the increment size is Δz . The finite-difference form of the pile curvature differential equation is given by Equation (8).

$$\frac{y_{i-2} - 4y_{i-1} + 6y_i - 4y_{i+1} + y_{i+2}}{\Delta z^4} - \frac{GA}{EI} \frac{y_{i-1} - 2y_i + y_{i+1}}{\Delta z^2} + \frac{k}{EI} y_i = \frac{p_i}{EI} \quad (8)$$

Combined with the pile top and pile bottom, a closed algebraic equation group is formed. The boundary conditions at the pile head and pile base are implemented as follows.

Pile head $z = 0$:

The pile head is connected to the crown beam and anchor cable. The bending moment and shear force boundary conditions are: $M(0) = EI \frac{y_1 - 2y_0 + y_{-1}}{\Delta z^2} = 0$, or, if the crown beam provides rotational restraint: $M(0) = K_r \theta(0) = K_r \frac{y_1 - y_{-1}}{2\Delta z}$, $Q(0) = EI \frac{y_2 - 2y_1 + 2y_{-1} - y_{-2}}{2\Delta z^3} = -K_0 y_0 + T \cos \theta$, where $K_0 = E_a A_a / L_a$ is the equivalent spring stiffness of the anchor cable, and K_r is the rotational stiffness of the crown beam.

Pile base $z = L$:

For a fixed base: $y_n = 0$, $\theta_n = \frac{y_{n+1} - y_{n-1}}{2\Delta z} = 0$; For an elastic base: $M_n = K_{rb} \theta_n$, $Q_n = -K_{hb} y_n$, where K_{rb} and K_{hb} are the rotational and horizontal stiffness of the pile base, respectively. Substituting these boundary conditions into Eq. (8) eliminates the virtual nodes y_{-2} , y_{-1} , y_{n+1} , y_{n+2} and yields a closed matrix equation system.

The horizontal displacement y_i of each node can be obtained by iterative solution, and then the flexural moment, shear force and rotation at that point in the pile are calculated. Based on Equations (9) and (10), the flexural moment $M(z)$ and shear force $Q(z)$ at the column are as follows [21]:

$$M(z) = EI \frac{d^2 y}{dz^2} \quad (9)$$

$$Q(z) = EI \frac{d^3 y}{dz^3} \quad (10)$$

2.7 Overall Stability Analysis of Slope

According to the upper bound limit analysis method, the stability factor F_s for a three-dimensional slope stabilized by anchor cable micro-pile systems can be obtained from the energy equilibrium equation in the following way, as shown in Equation (11) [22]:

$$F_s = \frac{\int_V (\sigma_{ij} \dot{\epsilon}_{ij}) dV + \sum W_{\text{pile}} + \sum W_{\text{anchor}}}{\int_S T_i v_i dS} \quad (11)$$

In the formula, σ_{ij} is the stress tensor, $\dot{\epsilon}_{ij}$ is the plastic strain rate tensor, V is the volume of the plastic deformation zone, W_{pile} is the work done by the Micro-Pile, W_{anchor} is the work done by the anchor cable, T_i is the shear stress of the sliding surface, v_i is the velocity field, and S is the area of the sliding surface.

3. Physical Model Test

3.1 Test Materials and Model Design

To verify the correctness of the conceptual framework and find out how the group of micro-piles bears the cooperative load, a large-scale physical prototype experiment at a 1:10 scale was carried out. The size of the test chamber is 3.0m long, 1.5m wide and 1.8m high. The composite structure of steel frame and plexiglass panel is used to observe the deformation process of the slope. To reduce the friction between the side glass walls and the soil, Teflon film was attached to the inner surfaces of the side walls before filling. This treatment minimizes side-wall friction and prevents artificial amplification of the three-dimensional pile group effect.

The test soil was taken from a typical soil landslide site in Jiangxi Province. It was silty clay. After removing large particles and plant roots by 2 mm sieve, it was prepared according to the field moisture content. According to laboratory research on the physical and mechanical properties of soil, the reference values are specific gravity $\gamma=18.5 \text{ kN/m}^3$ and adhesion $c=25.3 \text{ kPa}$, internal friction angle $\varphi = 22.6^\circ$, water content $\omega = 18.7\%$, compression modulus $E_s = 8.2 \text{ MPa}$.

The model micropile is simulated by a hollow steel pipe. The prototype pile diameter is 150mm, and the model pile outer diameter is 30mm, giving a diameter scale of 1:5. The prototype pile length is 12m, and the model pile length is 1.2m, giving a length scale of 1:10. Therefore, the physical model is a distorted similitude model rather than a strictly geometric 1:10 model. To satisfy the dominant similitude requirement of the pile-soil system, the flexural rigidity EI of the model pile was designed by adjusting the wall thickness of the hollow steel pipe. The similitude ratios are $\lambda_L = 1/10$ for length, $\lambda_D = 1/5$ for diameter, $\lambda_I = \lambda_D^4 = 1/625$ for the second moment of area, and $\lambda_{EI} = \lambda_E \lambda_I$ for flexural rigidity. The model pile parameters were selected so that the pile-soil interaction in the physical model remains representative of the prototype. The detailed similitude derivation is given below. The micro-piles are arranged in three rows of plum blossoms, and the row spacing and pile spacing change according to the test conditions. The pile top is connected by a crown beam, which is simulated by an aluminum alloy profile. High-strength steel wire with a diameter of 2 mm will be used for the prestressed anchor cable, and a pulley-steering device will be employed to regulate the tension. The Angle of the anchor cable is 20° . Continuously observe the data obtained from the micro-tension sensor and modify the tension of the anchor cable accordingly.

According to the governing equation of the pile-soil system, the main similitude parameters include length, diameter, flexural rigidity, soil modulus, and load. For the elastic foundation beam, the similarity criterion can be expressed as $\lambda_{EI}/\lambda_k \lambda_L^4 = 1$, where λ_k is the subgrade reaction modulus ratio. Since the model soil is the same as the prototype soil, $\lambda_k = 1$, and the required flexural rigidity ratio is $\lambda_{EI} = \lambda_L^4 = 1/10000$. However, because the diameter scale is 1:5, the geometric inertia ratio is $\lambda_I = 1/625$. To compensate for this distortion, the model pile was designed as a hollow steel pipe with a suitable wall thickness, and the equivalent EI was calibrated. The calibrated model pile EI was used in the theoretical calculation and numerical simulation to ensure consistency.

Fig. 4 shows the general diagram of the experimental system, and its components are the loading device, sensor arrangement and data collection unit.

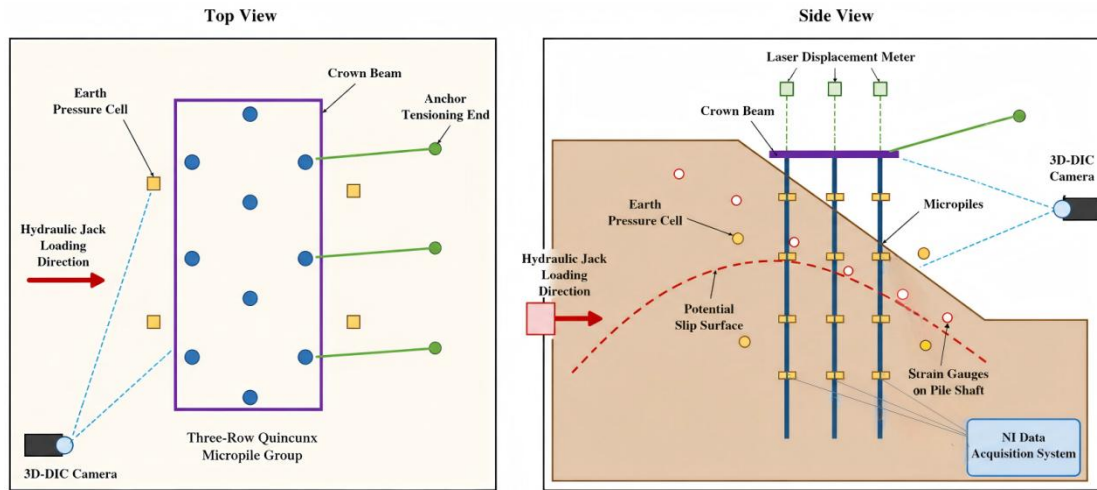


Fig. 4. Layout of physical model test system

3.2 Design of Test Conditions

A total of nine experimental arrangements were set up to investigate the influence of the three main factors: pile diameter, pile spacing and anchor cable tension, on the improvement effect: (1) Pile diameter influence group (working conditions A1-A3): fixed pile spacing $s=5D$, anchor cable prestress $T=50$ kN (prototype value), pile diameter $D=20$ mm, 30 mm, 40 mm (corresponding to prototype 100 mm, 150 mm, 200 mm). The diameter scale is therefore 1:5, while the length scale is 1:10. (2) Pile spacing influence group (condition B1-B4): fixed pile diameter $D=30$ mm, anchor cable prestress $T=50$ kN, pile spacing $s=3D$, $4D$, $5D$, $6D$. (3) The influence group of anchor cable prestress (working conditions C1-C3): fixed pile diameter $D=30$ mm, pile spacing $s=5D$, anchor cable prestress $T=0$ kN, 30 kN, 50 kN, 80 kN (prototype value).

3.3 Loading and Measurement Scheme

The sliding force is exerted via hydraulic jacks at the model box's terminal to replicate the landslide pressure's arrangement across the slip surface. Set the load speed to 0.5kN/min and record the data every 5 minutes after the load has stabilized. The measurement contents include: (1) Pile strain: strain gauges are pasted every 15 cm along the pile height (a total of 8 measuring points), and the dynamic strain acquisition system is used to record the sampling frequency of 50 Hz; (2) Pile top displacement: laser displacement sensors (range ± 50 mm, accuracy 0.01 mm) are arranged on each row of pile tops; (3) Landslide deformation: Six reflective markers are placed on the slope, and then a three-dimensional digital image correlation (3D-DIC) device is used to obtain a pattern of ground movement in the terrain; (4) Tension of anchor cable: each anchor cable is connected in series with a micro tension sensor (range 5 kN, accuracy 0.1% F.S.); (5) Soil pressure: Micro soil pressure cells (Range 200 kPa, accuracy 0.5% of full scale) have been embedded in the soil at both the pre-piling and post-piling locations. At a sampling frequency of 100Hz, all the sensor data are synchronously acquired by the NI Data Acquisition System.

It should be noted that the hydraulic jacks apply the driving thrust from the rear wall of the model box, which differs from the gravity-driven force in actual landslides. This rear-wall loading may cause local stress concentration near the rear boundary. However, this concentration mainly affects the soil close to the rear wall and gradually attenuates with distance. For the pile rows located in the middle and front parts of the model, the thrust distribution becomes relatively uniform before reaching the piles. Therefore, the boundary effect has limited influence on the pile-top displacement, maximum bending moment and overall safety factor, and the main trends and mechanisms obtained from the physical model remain valid.

3.4 Test Steps

The test steps are as follows: (1) Molding: The soil was prepared at the optimum moisture content and compacted in 15 cm thick layers. Each layer was compacted using a standard compaction hammer with controlled compaction energy following the standard Proctor compaction procedure.

After each layer, the dry density was measured by the ring-sampling method to ensure that the degree of compaction reached at least 95% of the maximum Proctor dry density. Micro-piles and earth pressure cells were embedded at the designed positions during filling [23]; (2) Installation of monitoring equipment: Install strain gauges, displacement sensors, tension sensors and DIC targets at a predetermined location, and complete all sensor wiring and debugging; (3) Tensioning of the Anchor Cable: A special tensioning device is employed to add some tension to the cable, and the initial value is recorded after the prestressing is stable for 30 minutes; (4) Multi-stage loading: start the hydraulic jack, apply the landslide thrust according to the predetermined loading scheme, hold the load after each stage loading and collect data; (5) Failure test: loading to the slope instability or structural failure, record the whole process data [24].

4. Numerical Simulation

4.1 Establishment of Numerical Model

Table 1. Material parameters of numerical model

Material/Element Type	Parameter	Symbol	Value	Unit
Soil (slide mass/bedrock)	Elastic modulus	E	25	MPa
	Poisson's ratio	ν	0.35	—
	Cohesion	c	25	kPa
	Internal friction angle	φ	22	°
	Dilation angle	ψ	0	°
	Natural unit weight	γ	18.5	kN/m ³
Micropile (pile element)	Elastic modulus	E_p	200	GPa
	Poisson's ratio	ν_p	0.3	—
	Pile diameter (prototype)	D	0.15	m
	Pile length (prototype)	L	12	m
	Yield strength	f_y	345	MPa
Capping beam (beam element)	Elastic modulus	E_β	200	GPa
	Section dimensions	—	0.3×0.3	m
Anchor cable (cable element)	Elastic modulus	E_a	195	GPa
	Cross-sectional area	A_a	1.4×10^{-4}	m ²
	Ultimate bearing capacity	T_u	200	kN
	Bond length	L_a	6	m
Pile–soil interface	Normal stiffness	kn	1.2×10^9	Pa/m
	Shear stiffness	ks	4.5×10^8	Pa/m
	Cohesion	cint	20	kPa
	Friction angle	φ_{int}	18	°
Anchor cable grout interface	Normal stiffness	kn	2.0×10^9	Pa/m
	Shear stiffness	ks	8.0×10^8	Pa/m
	Cohesion	cint	200	kPa
	Friction angle	φ_{int}	25	°

Note: The soil parameters are determined according to the indoor geotechnical test, and the structural unit parameters are selected according to the 'Code for Design of Concrete Structures' (GB 50010-2010) and 'Technical Code for Building Slope Engineering' (GB 50330-2013). The prototype values correspond to the model with a length scale of 1:10 and a diameter scale of 1:5. The model is a distorted similitude model, and the flexural rigidity of the model pile was calibrated according to the similitude requirement.

FLAC3D (Version 6.0) finite difference program was used to build a 3D computational model of soil mass sliding stabilized by anchor cables and micro-pile arrays. The size of the model is 30 m long, 15 m wide, and 18 m high; the slope elevation is 10 m and the slope gradient is 45°. The transition plane is a curved arc, and the bottom of the sliding element will be 4-6 meters below. Micro-piles

are taken as the pile structural element in FLAC3D, and flexural and compressive deformation and coupling at the pile-stratum interface can be introduced. Orthogonal and oblique spring elements are used to model the interface between the piers and the adjacent soil based on the p-y curve. A beam is used to model the bridge deck, and cables are employed for the tensioning of the cables. Table 1 shows the material properties of the calculation framework. Based on the above standards, experimental laboratory data, and so on, the theoretical elastoplastic Mohr-Coulomb material model for soil behavior was employed to determine the mechanical properties of the beam and cable components.

4.2 Material Parameters and Boundary Conditions

Based on the results of the field test, the coefficients for the Mohr-Coulomb hypothetical elastic-plastic material model of the soil are: Elastic Modulus $E=25\text{MPa}$, Poisson's ratio $\nu=0.35$, cohesion $c=25\text{kPa}$, internal friction angle $\varphi=22^\circ$, dilatancy angle $\psi=0^\circ$. Material Properties of Micro-piles: Elastic Modulus $E_p=200\text{GPa}$, Yield Strength $f_y=345\text{MPa}$. Parameters of the anchor cable: Elastic Modulus $E_a=195\text{GPa}$, Ultimate Bearing Capacity $T_u=200\text{kN}$. The boundary conditions are set as follows: fixed constraint at the bottom and normal constraint of the surrounding boundary. The model is divided into about 120000 hexahedral elements, and the grid around the pile is dense, with a minimum unit size of 0.2 m [25]. The numerical model adopts dry/saturated Mohr-Coulomb conditions. Rainfall infiltration, unsaturated seepage, matric suction and groundwater level fluctuation are not considered. To verify mesh independence, three mesh densities were generated for the numerical model: a coarse mesh with approximately 60,000 elements, a medium mesh with approximately 120,000 elements, and a fine mesh with approximately 200,000 elements. The pile-top displacement, maximum bending moment of the pile, and factor of safety of the slope were selected as the monitoring indices. The results show that the differences between the medium mesh and the fine mesh are less than 2%, while the coarse mesh produces a relatively larger deviation. Therefore, the 120,000-element mesh was adopted in this study because it provides sufficient accuracy with acceptable computational cost. This confirms that the 120,000-element mesh is mesh-independent for the present simulation.

4.3 Calculation Conditions and Analysis Steps

The numerical simulation condition is one-to-one correspondence with the physical model test condition (9 groups of working conditions), the three categories of auxiliary operating conditions are used to study the effect of pile group density (2 rows, 3 rows, 4 rows) and the form of pile arrangement (rectangle, plum blossom).

Table 2. Comparison results of pile top displacement and ultimate bearing capacity under 9 working conditions

Case	Pile top displacement (experimental) / mm	Pile top displacement (numerical) / mm	Pile top displacement (theoretical) / mm	Relative deviation	Standard deviation / mm	Measurement tolerance / mm
A1	12.3	12.9	12.5	4.9%	0.4	± 0.02
A2	10.5	11.0	10.8	4.8%	0.3	± 0.02
A3	8.9	9.3	9.1	4.4%	0.3	± 0.02
B1	14.2	14.8	14.5	4.2%	0.5	± 0.02
B2	11.8	12.3	12.0	4.2%	0.4	± 0.02
B3	8.7	9.1	8.9	4.6%	0.3	± 0.02
B4	8.3	8.7	8.5	4.8%	0.3	± 0.02
C1	15.6	16.2	15.8	3.8%	0.6	± 0.02
C2	11.2	11.7	11.4	4.5%	0.4	± 0.02
C3	8.7	9.1	8.9	4.6%	0.3	± 0.02

Note: Experimental displacement is the mean value; standard deviation is obtained from repeated readings. Measurement tolerance is $\pm 0.02\text{ mm}$ (laser sensor accuracy $\pm 0.01\text{ mm}$; data acquisition accuracy 0.5% F.S.). Relative deviation is the maximum difference among experimental, numerical and theoretical results.

The calculation steps are as follows: (1) initial stress balance; (2) Activate the Micro-Pile and crown beam structure unit; (3) Prestressing the anchor cable; (4) Apply landslide thrust (10-stage loading); (5) Record the results after solving to convergence.

5. Results and Analysis

5.1 Model Validation

To verify the consistency of the conceptual framework and computational analysis, the obtained information is compared with the results of the practical model test. Table 2 shows the evaluation results of piling apex deflection and maximum load-bearing capacity at the nine different operating conditions. Fig. 5 is the distribution of the horizontal displacement at the pile top level for the different depths in scenario B3. The observed relative difference among them is still less than 5%, so both the theoretical basis and the numerical method employed in this study are in agreement.

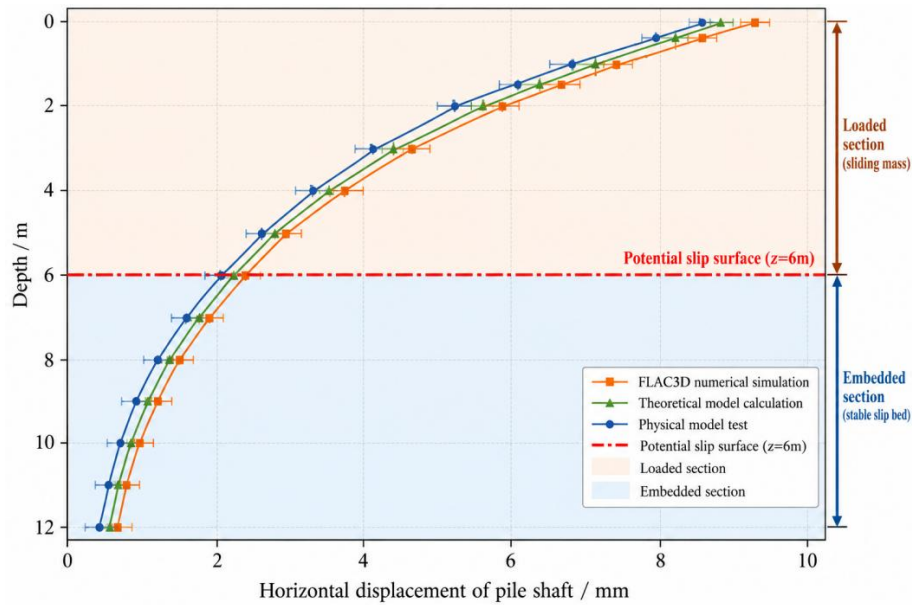


Fig. 5. Horizontal displacement of pile along the depth distribution

Note: The potential slip surface is located at $z=6\text{m}$. The region above 6m is the loaded section, and the region below 6m is the embedded section.

5.2 The Effect of Pile Diameter on Bearing Capacity

Fig. 6 shows the variation of bending moments at various depths of the pile for different pile sizes. As shown in Fig. 6, when the model pile diameter is increased from 20mm to 40mm, which corresponds to prototype pile diameters of 100mm and 200mm with a diameter conversion factor of 5 (model: prototype = 1:5), the maximum positive bending moment increases from 18.6kN·m to 42.3kN·m, i.e., by 127%. It should be noted that 20–40mm are model-scale diameters, while 18.6–42.3kN·m is prototype-scale bending moments. At the same time, the largest negative bending moment was also -28.1 kN·m. The position of the maximum bending moment is about 0.5 times the radius of the pile from the sliding joint. Increase the diameter of the pile to improve flexural strength and thus increase the internal force; therefore, the material strength of the pile structure needs to be enhanced accordingly.

Table 3 shows the stability coefficient and pile cap deflection of slopes stabilized by anchor cable micro pile clusters at different pile sizes. As the pile diameter increases from 100 mm to 200 mm, the stability factor is about 1.96 at 200 mm and was around 1.52 at 100 mm. The ultimate bearing capacity increased from 186kN to 298kN, an increase of 60.2%. However, the amount of concrete increased by 300%, and the economy decreased significantly. Considering the safety and economy, the pile diameter of 150 mm is the better choice.

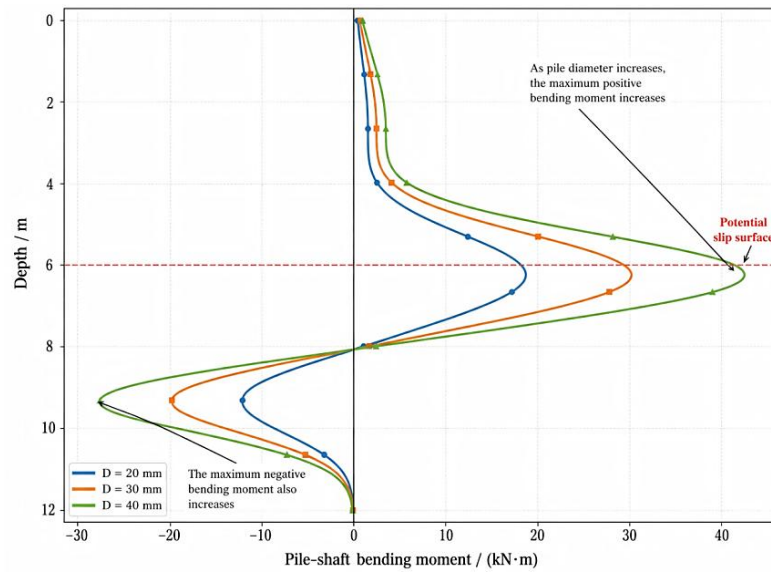


Fig. 6. Pile bending moment distribution under different pile diameter conditions

Table 3. Comparison of reinforcement effect under different pile diameters

Pile diameter / mm	Safety factor	Pile top displacement / mm	Ultimate bearing capacity / kN	Concrete volume / m ³	Concrete cost / CNY	Anchor cable fabrication, grouting and tensioning cost / CNY	Total cost / CNY	Comprehensive cost-benefit index I _{CB}
100	1.52	12.3	186	2.36	1416	2220	3636	0.078
150	1.78	10.5	245	5.31	3186	2220	5406	0.081
200	1.96	8.9	298	9.44	5664	2220	7884	0.074

5.3 The Effect of Pile Spacing on Bearing Capacity

Fig. 7 is the change rule of pile group efficiency coefficient under different pile spacing conditions. The efficiency factor η of a pile group is the ratio of the total ultimate bearing capacity of the group to the sum of the individual ultimate bearing capacities of the piles. As illustrated in Fig.7, η rises as s/D grows. When $s/D = 3$, then $\eta = 0.68$; thus, there is a considerable pile group effect; when $s/D = 4$, $\eta = 0.81$; when $s/D = 5$, $\eta = 0.92$, which is close to the independent working state; when $s/D > 5$, η tends to be stable.

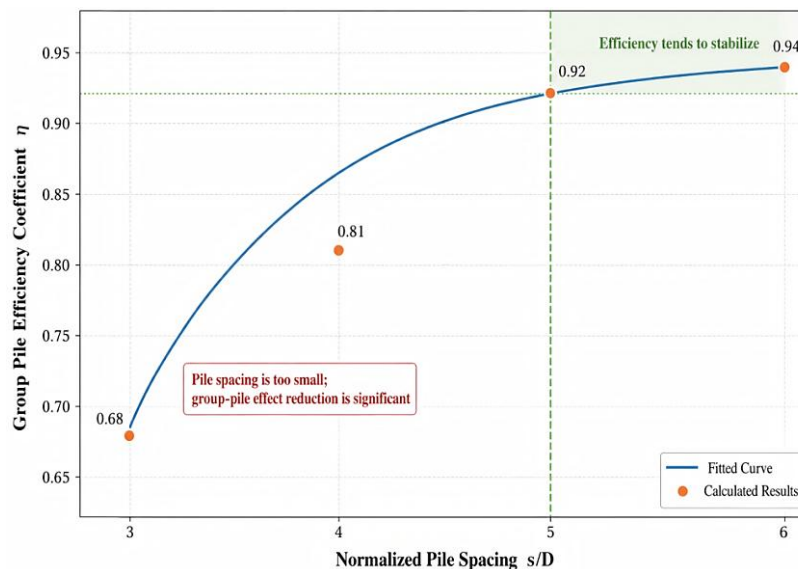


Fig. 7. Relationship between pile group efficiency coefficient and pile spacing

Table 4 shows the distribution of the load coefficient for each row of piles at different spacings between piles. The first row of piles (the initial row) has the largest landslide force, and its coefficient is between 0.387 and 0.482. As the distance from the pile increases, the load-sharing ratio of the leading row decreases and the proportion of the trailing row increases; thus, the distribution becomes more even. The first is that the spacing of the piles is too small; The joint anti-creep effect of the pile array will also be diminished. Based on the above results, it is proposed that the spacing is 5D.

Table 4. Load sharing ratio of each row of piles under different pile spacing

Pile spacing	Load Sharing Ratio of Row 1	Load Sharing Ratio of Row 2	Load Sharing Ratio of Row 3
3D	48.2%	32.1%	19.7%
4D	44.6%	33.8%	21.6%
5D	40.3%	35.2%	24.5%
6D	38.7%	35.9%	25.4%

5.4 The Effect of Prestress on the Bearing Capacity of Anchor Cables

Fig. 8 is the distribution of bending moments in piles due to the tension of anchor cables. As the tension in the anchor cable goes from 0 to 80 kN, the largest positive bending moment is reduced from 32.5 kN·m to 18.9 kN·m and the largest negative bending moment increases from -10.2 kN·m to -26.7 kN·m. The neutral axis of the bending moment in the pile moves downwards from being near the sliding plane to about 1.0 m below the sliding surface. Therefore, it can be seen that the use of anchor cables reduces the magnitude of the positive bending moment at the pile head and increases the compressive stress there.

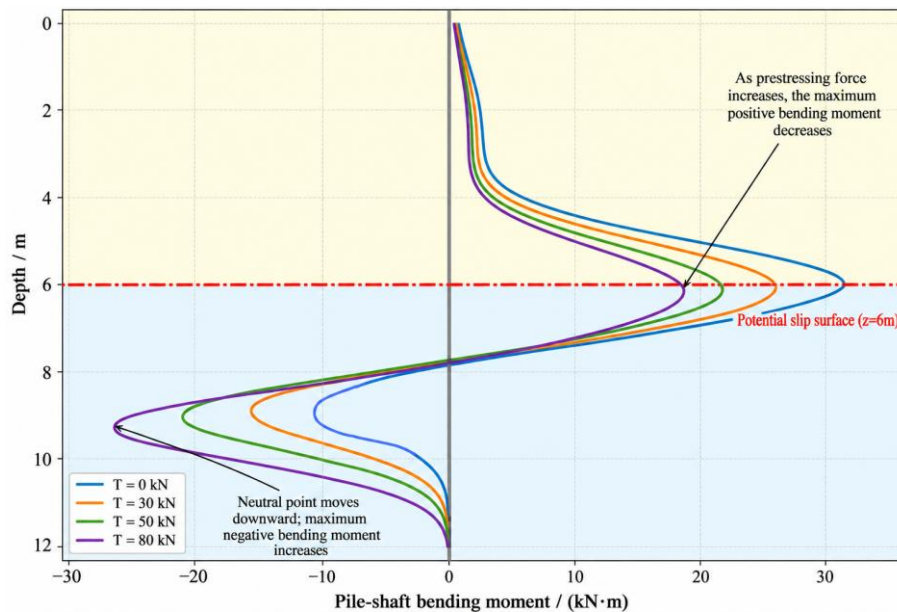


Fig. 8. Pile bending moment distribution under different anchor cable prestress

Note: The potential slip surface is at $z=6\text{m}$. The region above 6m is the loaded section, and the region below 6m is the embedded section.

Table 5 shows the comparison of the critical parameters for the reinforcement effect at different levels of anchor cable tensioning. Taking $T = 0\text{ kN}$ as the traditional micro-pile baseline, when the anchor cable tension increases to 50 kN, the safety factor increases from 1.52 to 1.78, i.e., by 17.1%, while the pile-top displacement decreases from 15.6 mm to 8.7 mm, i.e., by 44.2%. However, when it increases from 50 kN to 80 kN, the safety factor only increases by 3.9%, and the pile top displacement decreases by 20.7%. It can be concluded that there is an upper and lower bound for the tensioning of anchor cables, and beyond this range, the additional gain in strengthening

performance will be very small. It is suggested that the prestress value should be 50 ~ 60 kN in engineering application. It should be noted that the recommended prestress of 50-60 kN is an initial tensioning value. In the long term, prestress loss is inevitable due to soil viscoelastic/creep deformation, anchor cable relaxation, and anchorage seating/wedge slip. According to engineering experience, the long-term prestress loss of anchor cables in soil slopes is generally 10%–20%. Taking an initial prestress of 50 kN as an example, the effective prestress after loss may be about 40-45 kN. Compared with the $T = 0$ kN case ($F_s = 1.52$), the safety factor at $T = 50$ kN is 1.78; if the effective prestress decreases to 40-45 kN, the safety factor is estimated to remain around 1.65-1.72, which is still higher than that of the traditional micro-pile group. However, the pile-top displacement will increase and the maximum positive bending moment will rise. Therefore, for long-term stability, it is recommended to adopt an initial tensioning value near the upper limit of 60 kN, reserve a safety margin in design, and carry out regular prestress monitoring and supplementary tensioning when necessary.

Table 5. Comparison of reinforcement effect under different anchor cable prestress

Anchor cable prestress / kN	Safety factor	Pile top displacement / mm	Maximum positive bending moment / kN·m	Maximum negative bending moment / kN·m
0	1.52	15.6	32.5	-10.2
30	1.65	11.2	26.8	-16.3
50	1.78	8.7	22.1	-21.5
80	1.85	6.9	18.9	-26.7

5.5 The Effect of Pile Group Quantity and Specific Layout Arrangement

Fig. 9 shows the variation of the slope stability coefficient and peak bending moment in the piles with different row numbers. As the row count increases from 2 to 3, the stability factor is 1.78, up from 1.52, and the maximum bending moment decreases by 18.6%. When increasing from three rows to four rows, the safety factor is only increased to 1.89 and reduces the maximum bending moment by 8.3%. Three-row piles have a relatively good anti-sliding effect, and increasing the number of rows is not worth much.

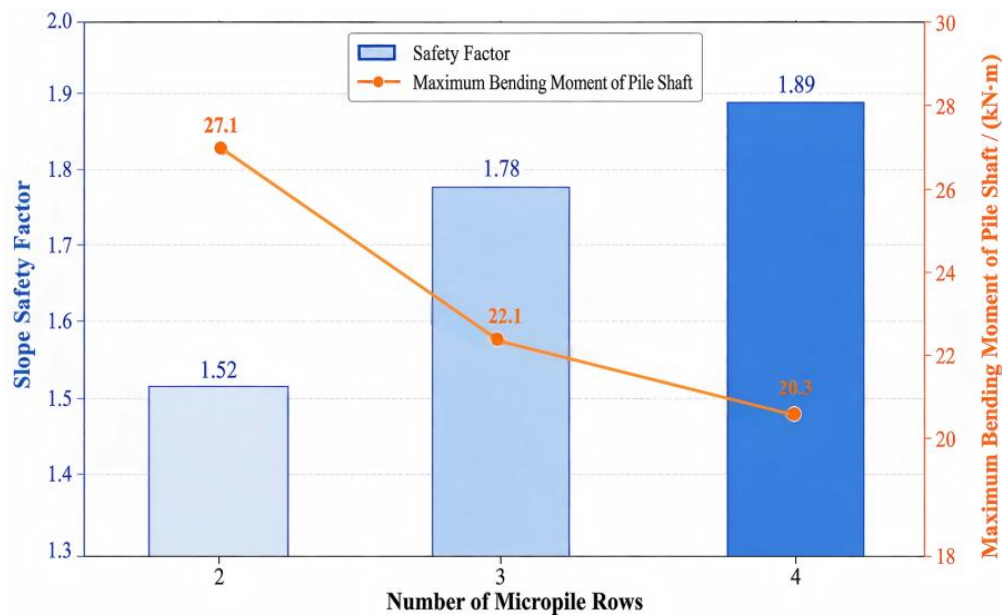


Fig. 9. The Influence of pile group arrangement on the enhancement mechanism

Table 6 shows the different improvement effects of rectangular piles and plum-shaped piles. The Safety Margin of the plum-shaped pile is 7.9% higher than that of the rectangular pile, and the displacement at the pile head has been reduced by 21.9%, and the efficiency coefficient of the pile

group is increased by 11.0%. Because a pile group in the shape of a plum can make full use of the soil bearing capacity of the piles and promote the formation and development of a soil arching mechanism. The design of the foundation pile arrangement will be in a plum shape.

Table 6. Comparison of reinforcement effects of different pile arrangement forms

Layout pattern	Safety factor	Pile top displacement / mm	Maximum bending moment / kN·m	Group efficiency factor
Rectangular (3 rows × 3 columns)	1.78	10.5	22.1	0.82
Quincunx (staggered) (3 rows × 3 columns)	1.92	8.2	18.6	0.91

6. Discussions

6.1 Analysis of Collaborative Bearing Mechanism

The above test and numerical simulation results reveal the three-level cooperative bearing mechanism of anchor micropile group. The first level is the effect of early reinforcement by the anchor cable. A tensioned anchor cable is used to apply an anticipatory load via the crown beam, reduce some of the slope loads, increase the bending moment at the piles, and decrease the maximum positive bending moment by 41.8%. The next is the steady anti-sliding effect of the pile group. The micro-pile group transmits the landslide thrust to the deep stable stratum. When $s/D = 5$, the efficiency coefficient of the pile group reaches 0.92, and the front row piles bear about 40% of the thrust. The third level is the synergistic effect of pile-soil-anchor cable. Tension in the anchor cable is distributed by the crown beam to increase the load sharing of all pile rows. A plum-shaped pile arrangement can cause the problem of soil arching and increase the total stiffness of the combined system. Taking the ordinary micro-pile group without anchor cable prestress ($T = 0$ kN) as the baseline, the three-level superposition of the ACMPG increases the safety factor from 1.52 to 1.78, i.e., by 17.1%.

6.2 Parameter Sensitivity Analysis and Design Recommendations

According to the results of the parametric analysis, all design parameters are ordered by the degree of their influence on reinforcement as follows: anchor cable force > pile spacing > pile diameter > row quantity. Pre-tensioning of anchor cables has the most significant effect on the bending moment diagram of piles and is employed as an initial way to control the internal stress state of pile structures. The spacing of the piles will directly affect the performance of the pile group and thus determines how efficiently materials are used. The upper limit of the bearing capacity for piles is relatively high; however, the construction costs will also be high. The additional gain from adding another row will be smaller than the previous one.

Based on the above evaluation, the following design suggestions are put forward: (1) For shallow landslides with a sliding mass thickness of $H_s \leq 6$ m, it is recommended to use three layers of plum-shaped piles with a spacing of $s = 5D$ and a diameter of $D = 150$ mm; (2) The tensioning load of the anchor cable should be determined according to the landslide weight, and it is recommended that the ratio of the maximum positive bending moment to the maximum negative bending moment of the pile body be in the range of 1.0 to 1.5; (3) For cases with a thick sliding mass, it is possible to increase the number of layers to four, but one should be aware of the reduced impact of the pile group due to this high row count. From a field construction perspective, the 3-row quincunx arrangement on a 45° slope still faces several challenges. First, the micro-piling rig is difficult to position on steep terrain. Lightweight modular or track-mounted rigs are recommended, together with temporary working platforms, scaffolding, and anchor ropes for stability. Second, drilling alignment accuracy is critical. Total-station or GPS positioning, guide frames, and inclinometers

should be used, and the borehole deviation should be controlled within 1%. Follow-up casing or simultaneous drilling and casing can be adopted to prevent hole collapse. Third, casting the capping beam on a 45° slope is difficult. Segmented casting, pump or chute transportation, temporary supports, and embedded connection bars are recommended. Alternatively, precast or steel capping beams can be used to reduce on-site casting. Construction should follow a top-down or skip-hole sequence with real-time slope monitoring to minimize disturbance.

6.3 Applicability and Limitations of the Theoretical Model

The concept behind this study is the Pasternak two-parameter foundation beam method; considering the elastoplastic behavior of the soil at the pile and the permanent lateral displacement has been done, and it is much better than the traditional Winkler model. The difference between the computed data of this model and the experimental results is less than 5%; therefore, it is deemed reasonable.

However, the model still has the following limitations: the mechanical framework in this paper is based on the assumption of an isotropic earth, and does not fully consider the stratification, heterogeneity and water-force coupling effect of soil. The random field theory and unsaturated seepage theory should be introduced in the follow-up study to improve the model. In addition, long-term service performance problems such as anchor cable prestress relaxation and soil rheology need to be further revealed through long-term monitoring and time-varying analysis to establish a more comprehensive structural life prediction method. It should also be noted that the physical model adopts a length scale of 1:10 and a diameter scale of 1:5, which is a distorted similitude model. Although the flexural rigidity of the model pile was calibrated, the distortion may still introduce some quantitative deviation. Therefore, the physical model test is mainly used to verify the trend and mechanism, while the parametric study and design recommendations are based on prototype-scale numerical simulations. In addition, Water effects are not included in the present model. In field conditions, rainfall infiltration and saturation increase can reduce matric suction and effective stress, thereby weakening the pile-soil interface and the anchorage capacity of the anchor cable. This may lead to a decrease in the collaborative bearing capacity of ACMPG. Therefore, the recommended design parameters should be applied with caution to slopes under strong rainfall, reservoir water level fluctuations, or significant groundwater changes. Future work will incorporate hydro-mechanical coupling and unsaturated soil suction into the numerical model to evaluate the long-term performance of ACMPG.

7. Conclusion

In short, this paper will provide the theoretical basis, experimental and numerical results of the physical model, and performance improvement of the collaborative load-bearing mechanism and slope stabilization with anchor micro-pile arrays.

The main results are as follows:

- Based on the analytical framework of the Pasternak two-parameter foundation beam theory, the error in the numerical results is less than 5%, and it can effectively consider the combined deformation behavior of piles, soil and anchor cables.
- Compared with traditional micro-pile clusters, the safety factor of anchor-cable micro-pile systems has been increased by 14.7% and the maximum displacement of the pile has been reduced by 52.9%. The cooperation achieved here is the result of the combined effect of active anchorage support from cables and passive resistance provided by pile arrays. The ideal spacing ratio of the piles is set at five times the diameter of a single pile, and the efficiency coefficient of the group structure is 0.92. The safety factor of plum-shaped piles is 7.9% higher than that of rectangular piles. Three rows of piles can meet the anti-sliding requirements, and the increase in the number of rows is limited.
- If the tensioning force of the anchor cable exceeds 50 kN, then the strengthening effect will be diminished. It is proposed that 50~60 kN should be used so that the ratio of positive to negative bending moments in the pile section is between 1.0 and 1.5.

- According to the sensitivity ranking of design parameters, the combination scheme of pile diameter of 150 mm, pile spacing of 5D and plum-shaped pile is recommended.

Overall, ACMPG is an effective and economical technique for stabilizing silty clay landslides, provided that long-term prestress loss and rainfall-induced loss of matric suction are properly considered in design, monitoring and future hydro-mechanical coupling analysis.

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