

Article

In Situ Model Test and Numerical Simulation Study of Suspension Bridge Tunnel-Type Anchorage Adjacent to Bifurcated Tunnels

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Abstract

The construction of suspension bridges in mountainous expressways often involves tunnel-type anchorages in close proximity to shallow-buried bifurcated tunnels, particularly in soft rock strata with dense overlying structures. This proximity poses significant challenges to construction safety and stability. This study aims to investigate the influence of tunnel-type anchorage construction on the ground surface, surrounding rock, and adjacent bifurcated tunnels under such complex conditions. It was hypothesized that the anchorage load transfer and deformation mechanisms would significantly affect the adjacent tunnel, with potential cumulative effects due to the twin-anchor configuration. To address this, a combined approach of in situ scaled model testing (1:10 scale) and three-dimensional numerical simulation was employed. The model test incorporated monitoring of deformation and stress at key locations (anchor plug, rock mass, and anchor–rock interface) under incremental cable loads. Quantitative results from the model test indicate that at the design load (1P, equivalent to 2016.84 kN per anchor), deformations were minimal (e.g., maximum anchor displacement 0.35 mm). The anchor–rock interface exhibited limited slip (max 0.06 mm at 1P), and contact stresses were highest in the rear part of the anchor plug, indicating a non-uniform load transfer. Under overload conditions, the system reached yield at 7P and peak strength at 10.5P, with measured ground surface cracks up to 5 mm. Numerical simulations, calibrated against the experimental data, revealed that under increasing load (up to 10P), the plastic zones around the two anchors progressively expanded and eventually coalesced, leading to a characteristic “inverted trapezoid” failure pattern propagating to the surface, accompanied by shear failure along the 14° bedding plane. The combined results quantify the progressive interaction between the twin anchorages and the surrounding rock, highlighting the critical role of the anchor–rock interface and the cumulative effect of twin anchors on ground deformation and potential failure mechanisms. This research provides a scientific basis for the design and construction of tunnel-type anchorages in similar challenging geological and spatial settings.



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1. Introduction

The hexagonal segment is a unique lining form for TBM water conveyance tunnels. Compared to traditional rectangular segments, it offers advantages in construction simplicity, structural stability, installation speed, and operational efficiency. This form also provides superior compressive resistance and sealing performance, making it particularly suitable for water diversion projects in regions with uneven water resource distribution and giving it significant potential for broader engineering application.

Scholars domestically and internationally have primarily studied bridge tunnel-type anchorages through several approaches: in situ model tests, laboratory model tests, theoretical derivation, and numerical simulation. Tang Hua et al. [1] conducted systematic research on the pullout resistance mechanism and load-bearing capacity of bridge tunnel anchors through indoor model tests, revealing the stress process and failure mechanisms of tunnel anchors under tensile loads, while evaluating their load-bearing capacity and design conservatism. Liu et al. [2] obtained real load response data through field-scale model tests, combined with FLAC3D numerical simulation for parameter extension and mechanism verification, establishing a systematic research framework of “experimental observation—parameter inversion—sensitivity analysis—failure mode induction” to comprehensively reveal the influence of weak interlayers on the pullout performance of tunnel anchors. Han et al. [3] analyzed the load-bearing behavior of tunnel-type anchors in soft rock with weak interlayers through field model testing. Liu et al. [4] studied the stability and potential failure mechanisms of Wujiang Suspension Bridge tunnel anchors based on field geological data and numerical simulation methods. Zhang et al. [5] systematically investigated the rock foundation stability of Aizhai Suspension Bridge tunnel anchors under complex canyon terrain conditions through geological surveys, numerical simulations, 3D block cutting analysis, and construction monitoring. Wang Dongying et al. [6] developed pullout load-bearing capacity and safety assessment methods for tunnel anchors through theoretical analysis, engineering case validation, and model test comparisons. Zhang Qihua et al. [7] based on the Pulite Bridge tunnel anchorage project, constructed a 1:25 in situ scaled model in Carboniferous limestone. Through 1P–8P load tests, 6P and 12P rheological tests, and a 50P destructive test, they investigated the load transfer mechanism, surrounding rock deformation characteristics, and rheological behavior of the anchorage–rock system. The results indicated that the overload stability factor of the anchorage was greater than 8, the long-term stability factor was greater than 6, and the surrounding rock did not fail under 50P. Yu Meiwan et al. [8] conducted comparative tests on cylindrical and truncated conical anchor plug bodies. The results showed significant differences in their failure modes: for a cylindrical plug, brittle failure occurred along the anchor–rock contact surface; for a truncated conical plug, the surrounding rock failed along unfavorable structural plane combinations, experiencing a relatively long plastic deformation stage before failure. The pullout resistance generated by the clamping effect, as the main component of the tunnel anchorage’s pullout capacity, was far greater than the shear strength of the anchor–rock contact surface. Wen Lina et al. [9], based on the Luding Dadu River Bridge tunnel anchorage project, conducted a 1:10 in situ scaled model test. Through monitoring and analysis of key data, they studied the deformation characteristics of the tunnel anchorage under 1P, 3.5P, and 7P loads. The results showed that under ultimate load, the relative displacement at the anchor–rock interface increased significantly with accompanying squeezing, the plastic zone continuously expanded into the surrounding rock above the anchorage, ultimately manifesting as a compression-shear failure mode. Wang Zhonghao et al. [10], based on the Jinsha River Bridge project, conducted a 1:10 in situ scaled model test, obtaining displacement, stress, and strain data under various loading levels. The results indicated that under a 9.5P load, the model anchorage and surrounding

rock bore the load together in a linear elastic state, with very small deformations of the anchor plug, surrounding rock, and relative displacement at the anchor–rock interface. Due to the clamping effect, the rock mass in the rear one-third of the anchor plug bore the main load, representing the critical stressed area. Yu Meiwan et al. [11], based on the Jindong Bridge tunnel-type anchorage project in complex rock mass strata, conducted a 1:11 in situ scaled model test to study the deformation characteristics and bearing capacity of the anchor plug and surrounding rock. The results showed that the model anchorage was essentially in an elastic state within 8P load, and no creep occurred under long-term 6P load, suggesting that the overload stability factor of the actual bridge anchorage was greater than 8, and the long-term stability safety factor was greater than 6. Mao Dawei et al. [12] conducted in situ model tests at 1:12 and 1:20 scales to systematically investigate the effects of different scales on tunnel anchor bearing characteristics. The results demonstrated that in terms of rheological properties, the 1:12 model exhibited primarily crack compression in the first-stage load (1P), while the 1:20 model showed predominant compression during the third-stage load (3P), with the latter's rheological displacement under 3P being approximately 6.5 to 9 times that of the former. Regarding deformation patterns, both scale models exhibited consistent trends in anchor plug and surrounding rock deformation, though the 1:20 model showed significantly greater overall deformation due to more fractured surrounding rock. The deformation distribution patterns differed: the 1:12 model displayed a saddle-shaped profile, whereas the 1:20 model exhibited a bimodal shape. In terms of overload performance, neither model showed significant yield bifurcation when loaded to 11 times or 12 times the design stress, indicating excellent overload stability across scales. Post-unloading, both models exhibited substantial permanent deformation with high residual strain rates in both anchor plugs and surrounding rock. Han et al. [13] utilized a rectangular test chamber equipped with digital image correlation (DIC) technology to conduct physical model tests on shallow tunnel anchors in soft rock strata, systematically studying the progressive failure behavior of tunnel anchors under stepwise loading. The results demonstrate that the load-bearing process of tunnel anchors can be divided into four stages: linear elastic deformation, slow nonlinear deformation, plastic deformation, and failure. The failure mode evolves from single rock mass failure to a dual failure involving both rock mass failure and interfacial slip detachment. Han et al. [14] systematically investigated the mechanical behavior and progressive failure characteristics of tunnel anchors under tensile loads. The horizontal displacement field of the surrounding rock exhibited a transition from a trumpet-shaped to a shuttle-shaped pattern, while the vertical displacement field displayed a layered vertical strip-like distribution at the upper anchor body and an approximately arc-shaped distribution at the lower part. The surface vertical displacement exhibited phased, regional, and lagged characteristics. The interfacial normal stress distribution evolved with the failure process: a bimodal shape with larger stresses at both ends and smaller stress at the middle during the elastic stage; the peak shifted forward during the plastic slip stage; and a nearly trapezoidal distribution during the detachment stage.

Yu Zhengfu [15] studied the failure mechanism of the anchorage–rock system under pullout load using finite element analysis. The results indicated that the loading of a tunnel-type anchorage can be divided into elastic and plastic stages: deformation increases linearly with load in the elastic stage; when the load reaches a certain value, the anchorage deformation increases dramatically, later showing flow deformation until final failure, with a failure shape characterized as “narrow at the bottom and wide at the top”. Yang Zhongping et al. [16] used FLAC3D 6.00 finite difference software to study the influence of anchorage burial depth, cross-sectional size, expansion angle, and anchor plug length on the bearing performance of a mudstone tunnel anchorage. The results showed that the

maximum deformation of the anchor plug was closely related to the size of the rear anchor face; larger rear anchor face dimensions led to better bearing characteristics of the tunnel anchorage; the failure mode of the actual bridge anchorage manifested as fracturing of the surrounding rock due to lateral extrusion, eventually leading to the overall pullout of the anchorage. Liu et al. [17] studied the bearing characteristics, influencing factors, and patterns of tunnel anchorages using finite element analysis. A prediction model for the safety factor of tunnel anchorages was proposed based on an artificial neural network method; reliability probability calculations for the tunnel anchorage yielded probabilities that met the target reliability requirements. Lim et al. [18], based on the geometric characteristics of the tunnel anchorage and rock joint properties, used PLAXIS 3D 2023.0 finite element software to study the pullout performance of the tunnel anchorage, analyzing the influence of anchor plug expansion angle, anchorage spacing, rock joint orientation, joint spacing, and joint shear strength on pullout performance. The results showed that as the expansion angle of the tunnel anchorage gradually increased, the failure mode transitioned from pullout failure to wedge failure; pullout resistance decreased with decreasing anchorage spacing, similar to the group pile effect; pullout resistance was minimum when joints were perpendicular to the main cable load direction; ultimate load increased with increasing joint spacing; the friction angle had a greater influence on pullout performance than the cohesion between rock joints. Ma Guojie et al. [19] conducted FLAC3D 6.00 numerical simulations to compare tunnel anchor models in homogeneous and weakly layered surrounding rock, revealing that weak interlayers significantly degrade the structural performance of tunnel anchors. During construction, surrounding rock displacement increases approximately 10-fold, posing safety risks. In operation, the integrity of surrounding rock is compromised, reducing anchor load-bearing capacity by about 30% and increasing displacement by 30–50% under equivalent loads. Additionally, weak interlayers alter the failure mode of anchors, accelerating the development of plastic zones and causing premature structural failure. Liu Mingchao et al. [20] proposed a simplified calculation formula for tunnel anchor ultimate bearing capacity considering the “clamping effect” of surrounding rock through theoretical derivation and ABAQUS numerical simulation. This formula decomposes bearing capacity into two components: the pullout force from anchor self-weight and the additional stress-induced pullout force from clamping effects. Studies indicate that the ultimate bearing capacity safety factor of the tunnel anchor reaches 28.85, far exceeding code requirements, with most pullout force provided by the clamping effect. The shear stress criterion based on the inverted wedge failure mode shows that the tunnel anchor reaches ultimate state at 19 times the design cable force. Li Mei et al. [21] systematically investigated the influence of dip angles (20–70°) in inclined layered rock formations on the bearing stability of tunnel anchors using FLAC3D numerical simulations. The results indicate that the most unfavorable conditions occur at a rock layer dip angle of 30°, where the anchor plug experiences maximum axial displacement, the surrounding rock exhibits the widest plastic zone, and the failure mode primarily involves sliding along the mudstone layer. At a dip angle of 70°, the tunnel anchor demonstrates a counterclockwise rotation trend under design load, resulting in a non-compressed zone at the rear anchor face with zero normal stress at the contact surface. As the dip angle increases, the principal stress contour lines of the surrounding rock fluctuate more significantly due to resistance from rock displacement, manifesting as upward convexity in sandstone layers and downward concavity in mudstone layers. When the dip angle exceeds that of the anchor plug (35°), the anchor plug no longer forms a concentrated compression ring. Yang Guojun et al. [22] conducted theoretical derivations and FLAC3D numerical simulations, using sine functions to model dynamic loads on the main cable, to systematically investigate the load-bearing performance of tunnel anchors under dynamic loads and their sensitivity to influencing

factors. The results show that under dynamic loads, the stress and displacement distribution patterns at the anchor–rock interface resemble those under static loads, but with significantly higher amplitudes and growth rates. Displacement increases reach 36% in the arch top direction and 112% in the arch base direction, with a “threshold value” at seven times the static load amplitude where response differences emerge. Dynamic loads cause premature appearance and accelerated expansion of the surrounding rock’s plastic zone, reducing the tunnel anchor’s ultimate bearing capacity by approximately 21%. The sensitivity ranking of geometric parameters to dynamic load-bearing capacity is: anchor plug length > expansion angle > spacing. While dynamic load frequency has minimal impact on bearing capacity, it induces “resonance phenomena” when approaching the anchor’s natural frequency, leading to significant response amplification.

From the current state of research, there are no reported in situ model tests for tunnel anchorages adjacent to structures. Most studies focus on analyzing the force mechanism of the tunnel anchorage structure itself, neglecting the influence of adjacent tunnels. Even when some studies consider adjacent tunnels in modeling, significant simplifications are made, and the clear distance between the anchorage and tunnel is relatively large, with insufficient consideration given to tunnel excavation and support, leading to numerous problems exposed during the construction of adjacent tunnels.

2. Materials and Methods

To systematically investigate the influence of suspension bridge tunnel-type anchorage construction and operation on the ground surface, surrounding rock, and adjacent bifurcated tunnels in soft rock strata, this study adopts a hybrid research approach combining in situ scaled model testing with three-dimensional numerical simulation. Through mutual validation and supplementation between experimental and numerical methods, a research framework is established that extends from local response observation to global mechanical behavior analysis.

(1) In situ Scaled Model Test

To capture the mechanical response and deformation behavior of the tunnel-type anchorage under authentic geological conditions, a 1:10 large-scale in situ model test was conducted at a location adjacent to the project area with analogous geological characteristics. The test model was rigorously designed following the geometric similarity ratio $C_L = 10$ and stress similarity ratio $C_\sigma = 1$. A rear-push loading system was employed to simulate the design main cable load (1P) and progressive overload conditions up to failure. By embedding multi-point displacement meters, dislocation meters, and strain gauges at critical locations including the plug body, surrounding rock, ground surface, and anchor–rock interface, real-time monitoring of deformation and stress data under various load levels was achieved. The analysis focused on:

- Displacement distribution patterns of the plug body and surrounding rock;
 - Dislocation characteristics at the anchor–rock interface;
 - Stress transfer mechanisms within the plug body and across the interface;
 - Ground surface deformation and progressive failure development of surrounding rock.
- ### (2) Three-dimensional Numerical Simulation Analysis

To address the limitations of in situ tests regarding restricted monitoring points and difficulty in observing macroscopic mechanical fields, this study employed FLAC3D 6.00 finite difference software to conduct true three-dimensional numerical simulations with dimensions identical to the test model. A refined mesh model was established based on geological data from the test site. The ubiquitous-joint model was adopted as the constitutive model for the rock mass to reflect the mechanical anisotropy of bedding

planes, while Interface elements were used to simulate slip and separation behavior at the anchor–rock contact surface.

The key steps of the numerical simulation included:

Parameter back-analysis and validation: Through iterative trial calculations, mechanical parameters of the rock mass and contact surfaces were calibrated. Typical monitoring points (e.g., front and rear anchor surface displacements) were selected for comparison and fitting with in situ test data to ensure the numerical model authentically represented the load-response behavior of the tunnel-type anchorage.

Full-process simulation analysis: The entire construction and loading sequence—comprising anchor tunnel excavation, plug body backfilling, and main cable load application (1P, 4P, 7P, and overload conditions)—was simulated to systematically analyze:

- Three-dimensional deformation fields of surrounding rock and ground surface;
- Principal stress distribution characteristics of the plug body and surrounding rock;
- Evolution patterns of plastic zones and failure modes;
- Normal contact and tangential slip behavior at the anchor–rock interface.

(3) Synergistic Analysis of Test and Simulation

This study integrates the discrete “point” data obtained from in situ tests with the continuous “field” analysis derived from numerical simulations. The experimental results provide realistic boundary conditions and a basis for parameter calibration in the numerical model, while the numerical simulations extend the observational dimensions of the physical tests, revealing macroscopic failure mechanisms such as plastic zone penetration in surrounding rock and the evolution of interface slip. The two approaches mutually validate each other, collectively elucidating the bearing characteristics of tunnel-type anchorages in soft rock strata and their influence on adjacent bifurcated tunnels, thereby providing a scientific foundation for the design and construction of similar engineering projects.

3. In Situ Scaled Model Test of Tunnel-Type Anchorage

3.1. Overview of the Project

The project under study is the BDC Yangtze River Suspension Bridge on a Chinese expressway in Chongqing and the adjacent BTP bifurcated tunnel. The main span of the bridge is 916 m, with a tunnel-type anchorage adopted on the north bank. The surrounding rock consists of Triassic Middle Series Badong Formation (T₂b) marl and argillaceous limestone. The total length of the tunnel-type anchorage cavern is 155.79 m, comprising a 32 m long catwalk, a 60 m long front anchor chamber, a 60 m long anchor plug body, and a 3.9 m long rear anchor chamber. The axis inclination of the catwalk is 26°, and the axis inclination of the anchor plug body is 41°. The maximum burial depth is approximately 113 m, and the center-to-center spacing of the left and right anchor bodies ranges from about 48 to 66 m. The anchor plug body is designed as a wedge shape, smaller at the front and larger at the rear. The cross-section has an arched top and straight sidewalls and bottom. The front anchor face dimensions are 14 m (width) × 15 m (height) with a top arc radius of 7 m. The rear anchor face dimensions are 17 m (width) × 18 m (height) with a top arc radius of 8.5 m. The design main cable load per anchorage is 201,684 kN. Influenced by the route alignment, the minimum distance between the tunnel anchorage and the adjacent tunnel is only 4.5 m (Figure 1), and it is located in soft rock strata with dense overlying structures.

The BTP tunnel is a double-track tunnel with a design clearance of 13.0 m (width) × 8.0 m (height). The left line is 2152 m long and the right line is 2127 m long, classifying it as a long tunnel. The tunnel exit connects to the BDC Bridge, with the connection section being a bifurcated tunnel.

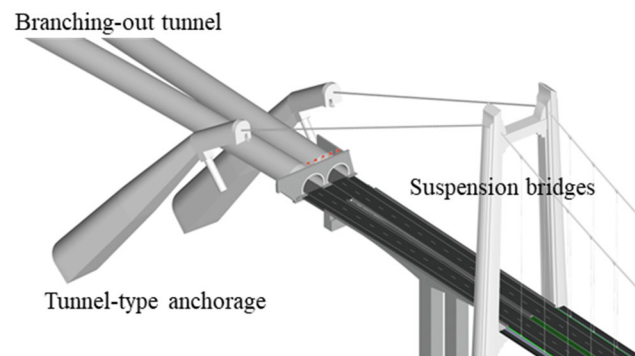


Figure 1. Spatial position relationship between tunnel-type anchorage and approaching tunnel.

In terms of construction sequence, the tunnel anchorage is built before the tunnel. The analysis did not consider the impact of the tunnel anchorage on the construction of the bifurcated tunnel, but the design should consider the influence of bifurcated tunnel excavation on the pre-existing tunnel anchorage.

3.2. Model Anchorage Site Selection and Construction

The model anchorage is located 30 m vertically below the entrance portal along the axis of the actual bridge anchorage. The upper layer of the surrounding rock is marl, and the lower layer is argillaceous limestone (Figure 2), belonging to the same bedrock stratum as the actual bridge anchorage, providing certain geological representativeness. However, due to the shallow burial depth of the model anchorage, its surrounding rock is more weathered and unloaded, with weaker rock mass properties compared to the actual bridge anchorage. Applying the test results to the actual bridge anchorage is therefore more conservative and reliable.

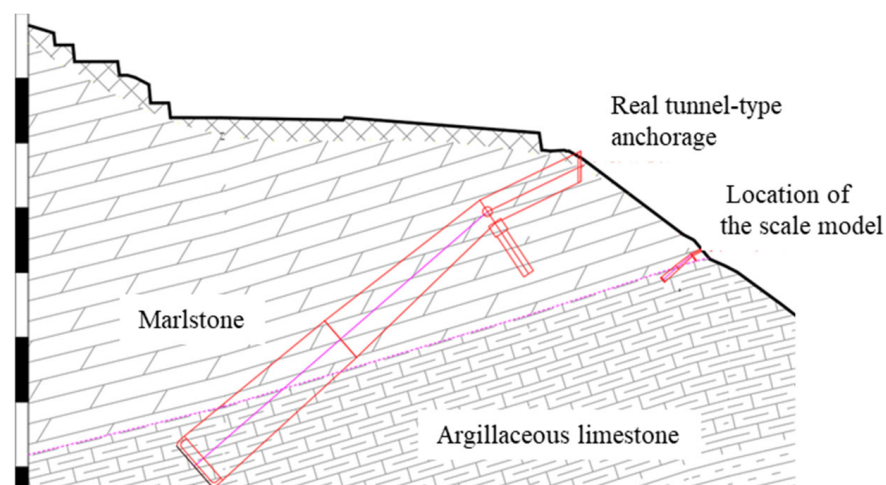


Figure 2. Profile of the location of the scale model and real tunnel-type anchorage.

The geometric similarity constant for the model is $C_L = 10$, the elastic modulus similarity constant is $C_E = 1$, and the Poisson's ratio similarity constant is $C_\mu = 1$. The model test maintained a strain similarity constant $C_\epsilon = 1$; thus, the displacement similarity constant $C_\delta = 10$ and the stress similarity constant is $C_\sigma = 1$. Theoretically, the unit weight similarity constant should be $C_\gamma = 1/10$, but for in situ testing, the density of the geomaterials and gravitational acceleration remain unchanged, so the actual unit weight similarity constant is $C_\gamma = 1$, resulting in some imperfections in the similarity between the model and prototype. The required stress for the model corresponds to the external load applied by the test jacks.

To satisfy the stress similarity constant $C_\sigma = 1$, the area similarity constant for the rear anchor face is $C_S = 100$, meaning the model load is set to 1/100 of the prototype load.

Based on similarity principles, the designed schematic of the model anchorage is shown in Figure 3. The anchor plug body is a wedge shape, smaller at the front and larger at the rear. The front anchor face dimensions are 1.4 m (width) $\times$ 1.5 m (height) with a top arc radius of 0.7 m. The rear anchor face dimensions are 1.7 m (width) $\times$ 1.8 m (height) with a top arc radius of 0.85 m. The anchor plug body length is 6.0 m, and the center-to-center spacing of the two anchor plugs is 5.2 m. The front anchor chamber cross-section is consistent with the front anchor face, with a length of 6.0 m. The rear anchor chamber cross-section is consistent with the rear anchor face, with a length of 1.5 m. The direction of the central axis between the two anchorages is consistent with the actual bridge anchorage axis orientation of 169° . The axis inclination of the anchorage cavern is 41° . The axis of the model anchorage cavern is kept consistent with that of the actual bridge anchorage cavern, i.e., the orientation of the left anchorage cavern axis is 165° , and the right anchorage cavern axis is 173° .

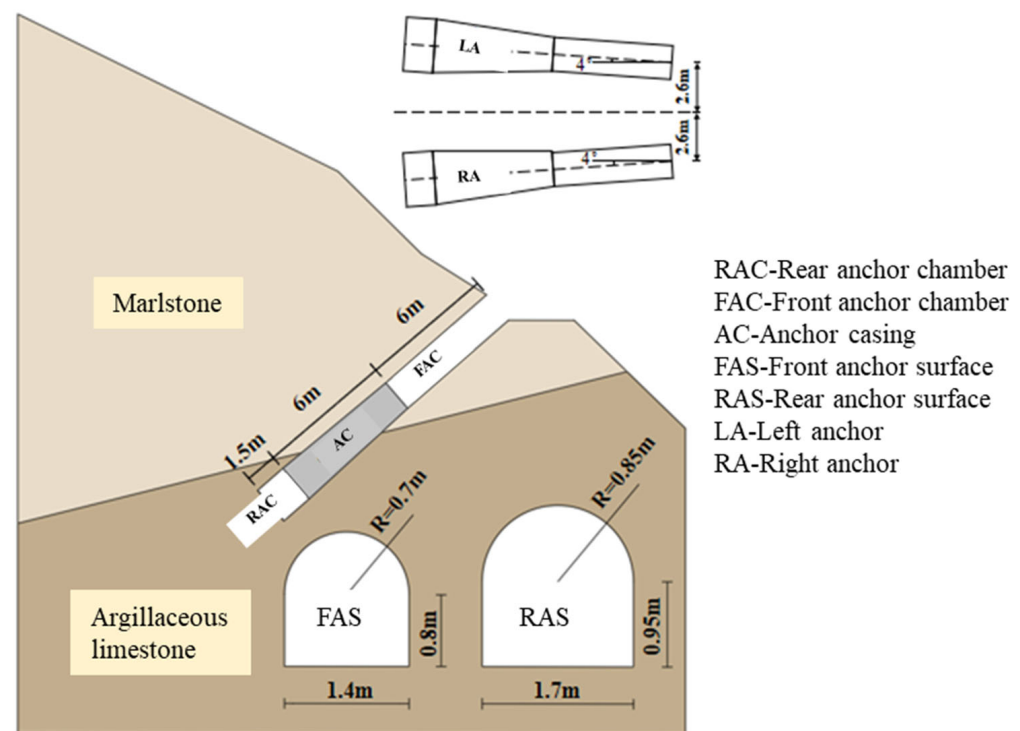


Figure 3. Sketch map of geological section and plug body size of 1:10 tunnel-type anchorage model.

The test employed the rear-push loading method [23–25]. According to the design requirements, the single-anchorage main cable design load under standard combination for the actual bridge anchorage is 201,684 kN. Therefore, the single-anchorage main cable design load (1P) for the model anchorage is taken as: $201,684/100 = 2016.84$ kN. Each anchorage uses ten 3000 kN capacity jacks for combined loading, capable of reaching a maximum load of 14.8P. The specific arrangement is shown in Figure 4.

3.3. Layout of Monitoring Points for In Situ Model Test

(1) Deformation Monitoring Points

Multiple grating displacement sensors were installed on the front and rear anchor faces and the ground surface surrounding rock of the model anchorage to measure deformation along the pulling force direction at various locations. The specific layout is shown in Figure 5. Three boreholes were drilled along the pulling force direction, each equipped

with a 4-point multi-point extensometer. The hole bottom, exceeding 3 times the cavern diameter from the anchorage loading point, was considered the reference point (Figure 6).

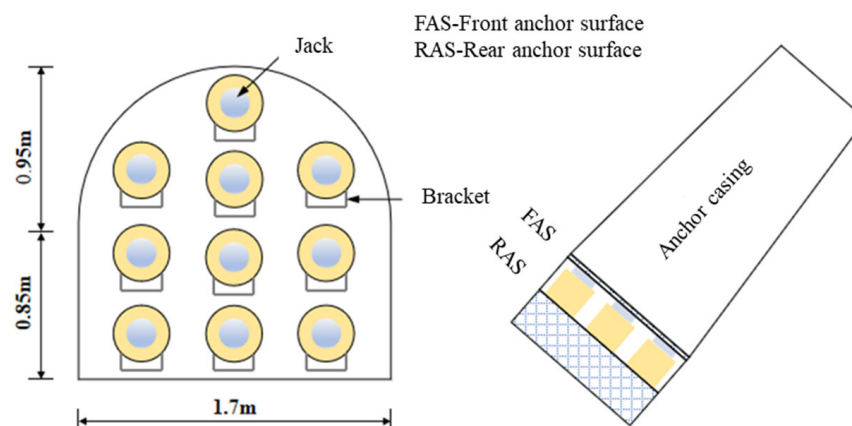


Figure 4. Installation diagram of rear anchor chamber jack and reaction rear seat.

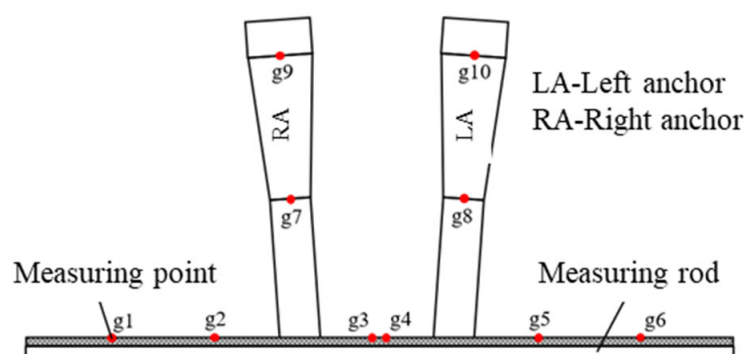


Figure 5. Layout diagram of grating displacement sensors.

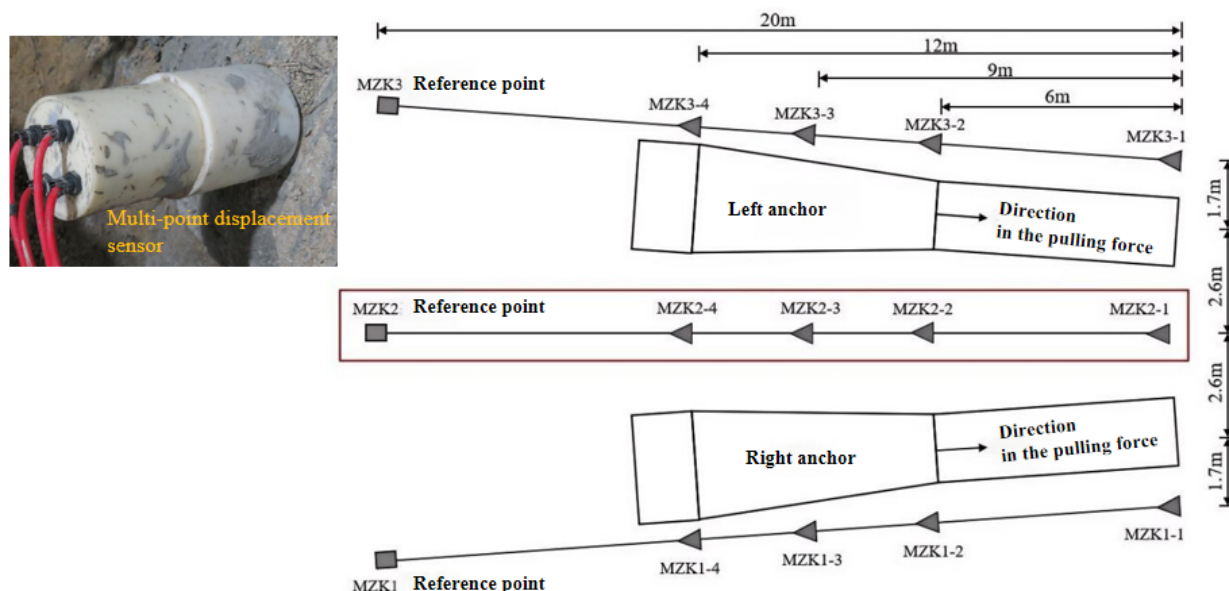


Figure 6. Schematic diagram of multi-point displacement transducer arrangement.

To obtain the relative deformation between the anchor plug body and the surrounding rock, two sections were set on the left anchor. Four joint meters were installed on the anchor–rock contact surface at each section (Figure 7).

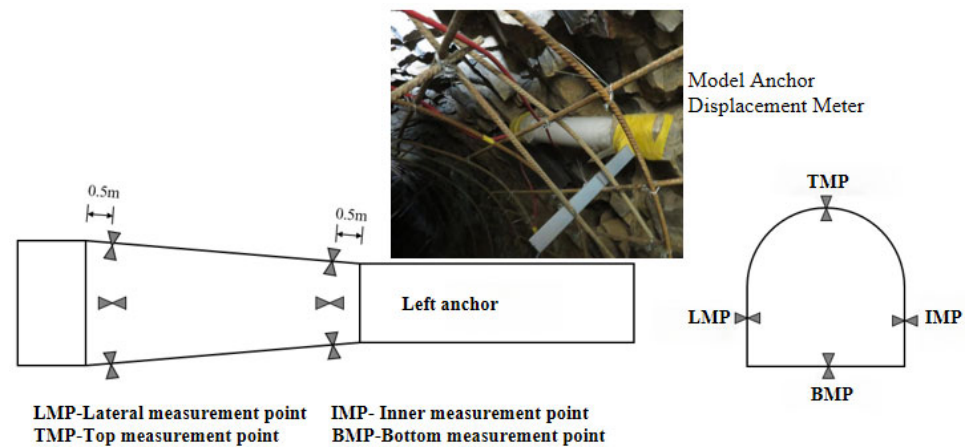


Figure 7. Schematic diagram of dislocation counter arrangement.

(2) Stress Monitoring Points

To obtain the strain distribution along the pulling force direction inside the anchor plug body under different load levels, one measurement line with five axial strain gauges was installed on the right anchor (Figure 8). To determine the interaction force on the anchor–rock contact surface after loading, strain gauges were installed on the contact surface. Contact stress was obtained by multiplying the strain by the elastic modulus of the anchor plug concrete. Four sections were selected on the right anchor, with four strain gauges installed on each section (Figure 9).

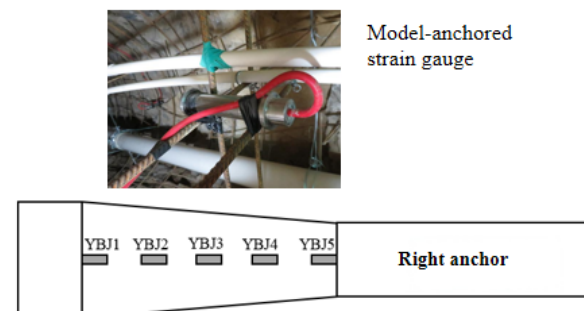


Figure 8. Schematic diagram of strain gauge arrangement on anchor plug body.

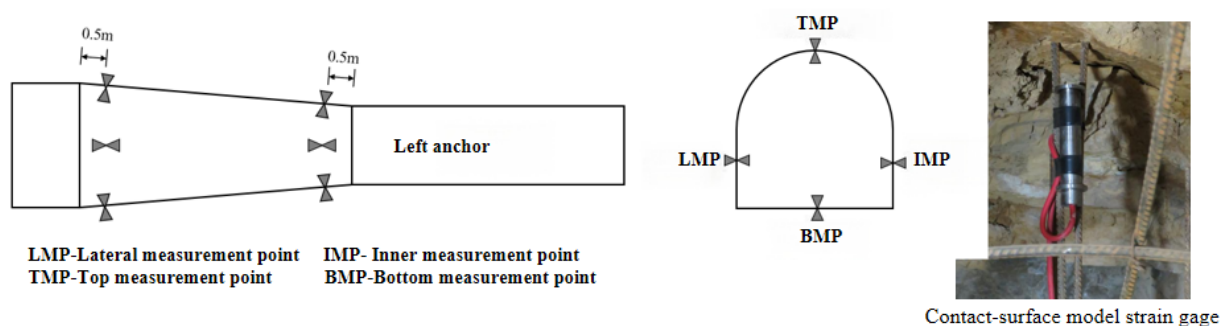


Figure 9. Schematic diagram of strain gauge arrangement at anchor–rock interface.

3.4. Analysis of In Situ Model Test Results

(1) Deformation Analysis

Under main cable load, the anchor plug body drives the deformation of the surrounding rock. Therefore, the deformation of the anchorage system is a comprehensive reflection

of the compression deformation of the anchor plug concrete, the relative displacement at the anchor–rock interface, and the deformation of the surrounding rock.

① Deformation of Anchor Plug Body

The relationship curve between the average deformation along the pulling direction at four measurement points (g7–g10) on the anchor body and the load grade is plotted as shown in Figure 10.

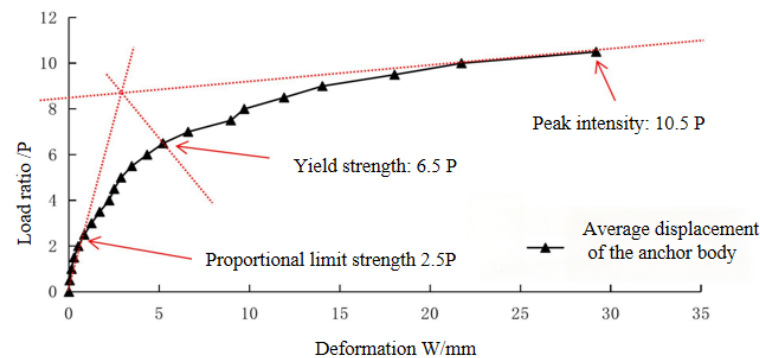


Figure 10. Load grade-average deformation relation curve of plug body.

From Figure 10, it can be seen that after ultimate loading to 10.5P, the load could not be increased further. At this time, field observation revealed a maximum surface crack width of 5 mm, indicating that deep surrounding rock had already failed. Thus, 7P is taken as the yield strength of the model anchorage, and 10.5P as the peak strength.

② Surface Deformation

Curves showing the deformation along the pulling direction of the ground surface (g1–g6) versus position under various load grades are plotted in Figure 11.

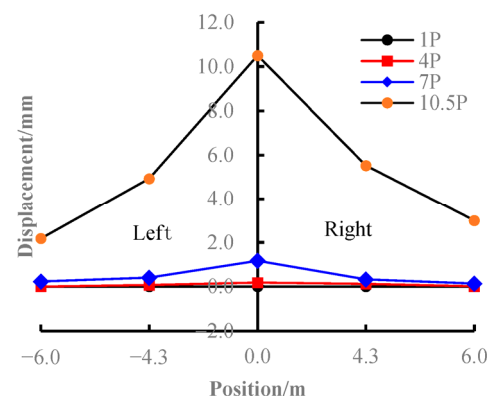


Figure 11. Curve of deformation of surface with position change.

From Figure 11, it can be seen that under various load levels, with the central pier as the axis, the deformation law of the surrounding rock on the left and right sides along the pulling direction is basically consistent. The maximum surface deformation occurs at the central pier (average of g3, g4), followed by the ground surface at 1 times the cavern diameter on the left and right sides of the anchor plug body (g2, g5), and finally at 2 times the cavern diameter on the left and right sides (g1, g6). This indicates that after the anchor plug body is subjected to the main cable pulling force, it drives the deformation of the surrounding rock. The farther from the anchor plug body, the smaller the force and deformation of the surrounding rock. Due to the superposition effect of the double anchorages, the deformation at the central pier is greater than that of the rock mass on both sides.

③ Deformation of Surrounding Rock Around Anchor Plug Body

Taking the measurement points of the borehole between the two anchorages (MZK2) as an example, the curve of surrounding rock deformation (along the pulling direction) versus position is plotted in Figure 12. It can be seen that the deformation of the surrounding rock along the pulling direction increases with increasing distance from the anchor plug body. The deformation shows a trend of decreasing from the rear anchor face to the front anchor face under various load levels, being smallest at the ground surface.

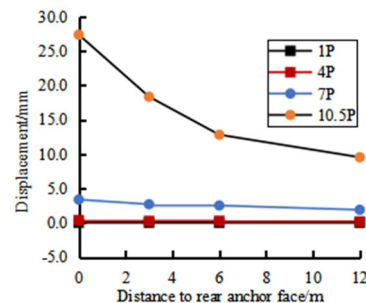


Figure 12. Curve of deformation of surrounding rock of borehole with position change.

④ Deformation of Anchor–Rock Contact Surface

The relative displacement (slip) deformation at the anchor–rock contact surface increases with increasing load grade, but the overall slip amount is small. Under 1P, 4P, and 7P loads, the maximum slip deformations between the anchor plug body and surrounding rock are 0.06 mm, 0.56 mm, and 1.74 mm, respectively. Throughout the loading process to failure of the model anchorage, the slip deformation at the anchor–rock interface did not show obvious nonlinear growth. That is, the failure mode of the anchorage–rock system is not sliding failure along the anchor–rock contact surface, but pullout failure of the anchor plug body together with the surrounding rock mass.

(2) Stress Analysis

① Stress Transfer in Anchor Plug Body

Multiplying the strain gauge test results by the deformation modulus of the reinforced concrete anchor body gives the stress borne by the anchor body. The anchor plug body uses C40 commercial concrete. According to the “Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts” (JTG 3362-2018) [26], the deformation modulus for C40 concrete is taken as 32.5 GPa. Since strain gauge YBJ1 failed, based on data from strain gauges YBJ2~YBJ5, the curve of stress at various measurement points in the anchor plug body versus position under different loads is plotted in Figure 13.

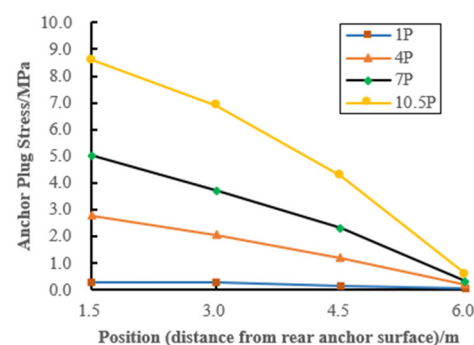


Figure 13. Curve of internal stress of plug body with position change.

From Figure 13, it can be seen that as the distance from the rear anchor face increases, the stress in the anchor plug body gradually attenuates. The stress at the front anchor face

is very small, indicating that the vast majority of the stress has been transferred to the surrounding rock.

② Stress Transfer at Anchor–Rock Contact Surface

The curve of stress at the anchor–rock contact surface versus position is plotted in Figure 14.

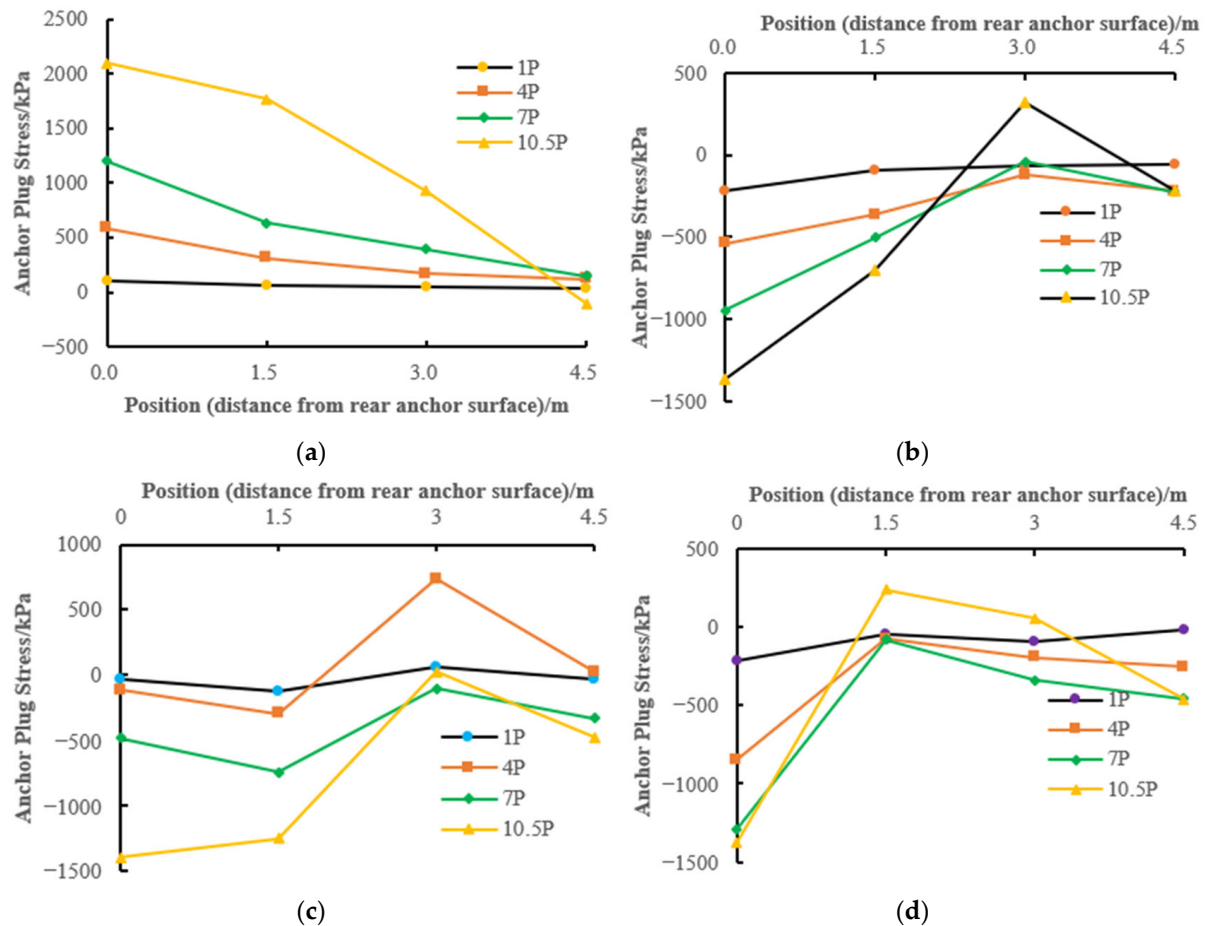


Figure 14. Curve of stress of plug body-rock contact surface with position change. (a) Anchor plug/top rock contact surface; (b) anchor plug/bottom rock contact surface; (c) anchor plug/external rock contact surface; (d) anchor plug/inner rock contact surface.

The force on the anchor plug body–top surrounding rock contact surface appears as compressive stress under various load levels. This is because when the rear part of the anchor body is loaded, there is a tendency to squeeze the top rock mass upward along the pulling direction. The stress value decreases along the pulling direction from the rear to the front of the anchor body, and this trend becomes more pronounced at higher load levels.

The force on the anchor plug body–bottom surrounding rock contact surface mainly appears as tensile stress under various load levels. This is because the vertical component of the load counteracts the gravity of the anchor body, unloading the initially compressed contact surface under the action of the main cable load.

The force on the anchor plug body–left/right side surrounding rock contact surfaces appears as tensile stress under various load levels, with individual measurement points showing compressive stress. This is because the upward movement tendency of the anchor plug body under load is relatively obvious, far exceeding the squeezing effect of the surrounding rock on the left and right sides. This causes the strain gauges embedded in the anchor plug concrete on the left and right sides to be stretched, manifesting as tensile stress.

Under the same load level, the contact stress borne by the rear part of the anchor plug body is greater than that at the front. The surrounding rock near the rear part of the anchor plug body reaches its ultimate strength before the front part. The rock fracture surface should initially develop from the rear part of the anchor plug body and gradually propagate. Furthermore, from Figure 14, it can also be seen that even when the load exceeds 10P, only the measurement points at the rear part of the anchor plug body bear large forces, which rapidly decay along the length direction, while the forces at the front measurement points are very small. This indicates that the length of the anchor plug body is excessive, meaning the design of the anchor plug body size is overly conservative.

4. Numerical Simulation Analysis of Model Anchorage

Since the in situ model test can only analyze specific measurement points, making it difficult to observe from a macroscopic perspective, full-scale numerical simulation of the scaled model was conducted to supplement the model test results and determine reasonable numerical simulation parameters.

4.1. Establishment of Numerical Model

To facilitate numerical analysis, the following basic assumptions and related settings were made for the numerical model:

- (1) The rock mass in the model is assumed to be a homogeneous continuous medium, ignoring existing voids, fractures, etc., within the rock mass; the anchor plug body is a homogeneous, isotropic material.
- (2) Due to the shallow burial depth of the model anchorage, the influence of the tectonic stress field is not significant. Therefore, the calculation analysis only considers the self-weight stress field [27].
- (3) The influence of temperature, seepage, etc., is not considered.
- (4) The anchor–rock contact interface element is assumed to have zero thickness.

The scaled model of the tunnel anchorage was modeled in true 3D using Rhino 6, and the griddle 2.0 plug-in was used for mesh generation before importing into FLAC3D 6.0 for numerical calculation and analysis.

According to Saint-Venant's principle, the influence of cavern excavation on surrounding rock decreases with increasing distance from the excavation until it disappears. If the numerical model scope is too small, boundary effects can lead to inaccurate analysis results; conversely, if the scope is too large, it wastes computation time. Based on extensive case verification, the influence range of cavern excavation on surrounding rock mass is typically within 3–5 times the cavern diameter. The relative stress error is usually less than 10% at 3 times the side length and less than 3% at 5 times the side length. The strain of the rock mass at boundaries within this range can be considered zero. It is best to limit the model dimensions within this range. Based on geological data and the geological conditions of the tunnel anchorage test model excavation, a 1:10 in situ scaled test numerical simulation model was established. The model dimensions were set to length (X) 30 m × width (Y) 26 m × height (Z) 32 m, with mesh refinement around the tunnel anchorage. The overall model is shown in Figure 15.

Boundary conditions were set as fixed constraints on the bottom surface, normal constraints on the side surfaces, and free on the ground surface. The excavation method used full-face excavation. The main cable tensile load was applied as an equivalent surface force on the rear anchor face.

The constitutive model for the rock mass adopted the FLAC3D elastic–plastic ubiquitous-joint model (ubiquitous-joint plasticity model) to simulate the anisotropy of rock mass mechanical properties in different directions [28]. When rock mass failure occurs

along the bedding plane, the bedding plane strength parameters are used; when failure occurs perpendicular to the bedding, the comprehensive rock mass strength parameters are used. This model, also known as the ubiquitous-joint model, is based on the Mohr–Coulomb model with added consideration for joint planes. The joint planes also obey the Mohr–Coulomb yield criterion. This model considers both the physical and mechanical properties of the rock matrix and the joints. Rock mass failure may occur in the rock matrix, on the joint planes, or both simultaneously, depending on factors such as rock mass stress state, joint orientation, and mechanical properties of the rock mass and joints [29]. The constitutive relationship for the concrete structure (anchor plug body) used the elastic linear elastic model.

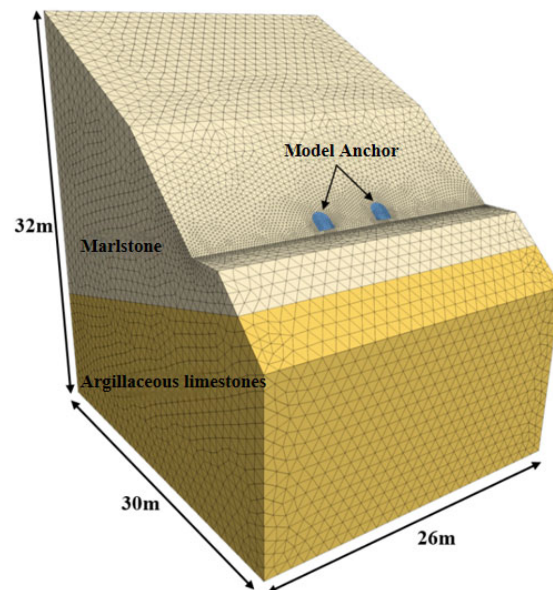


Figure 15. Numerical analysis model of scaled tunnel-type anchorage.

For simulating the contact interface, FLAC3D’s built-in interface element was used, which can analyze slip, sliding, and separation that occur on the contact surface between two different objects under given loading conditions [30]. Interfaces have Coulomb slip characteristics, characterized by parameters such as friction angle, cohesion, dilation angle, normal stiffness, shear stiffness, and tensile strength. The interface element has zero thickness and uses a Mohr–Coulomb shear model.

4.2. Selection of Calculation Parameters

C40 concrete of the same grade as the actual bridge anchorage was used for casting the anchor plug body. Its deformation modulus and Poisson’s ratio were taken according to the values for C40 concrete in the “Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts” (JTG 3362-2018) [26], and density was assigned based on engineering experience.

The mechanical parameters of the surrounding rock (rock mass) and the concrete–rock interface were determined through repeated trial calculations of the numerical model for the tunnel anchor, based on the results of rock mass deformation tests using the bearing plate method under natural conditions, natural-state shear tests, and concrete–rock interface shear tests (Figures 16–18), with consideration of marl and argillaceous limestone. These parameters were further validated by comparing the results from characteristic measurement points with those from field scaled model tests. As shown in Figure 19, the fitting results are relatively good when the load ratio ranges from 1 to 9.5P. The final numerical

simulation parameters effectively reflect the load response patterns of the tunnel anchor, with detailed parameters provided in Tables 1–3.

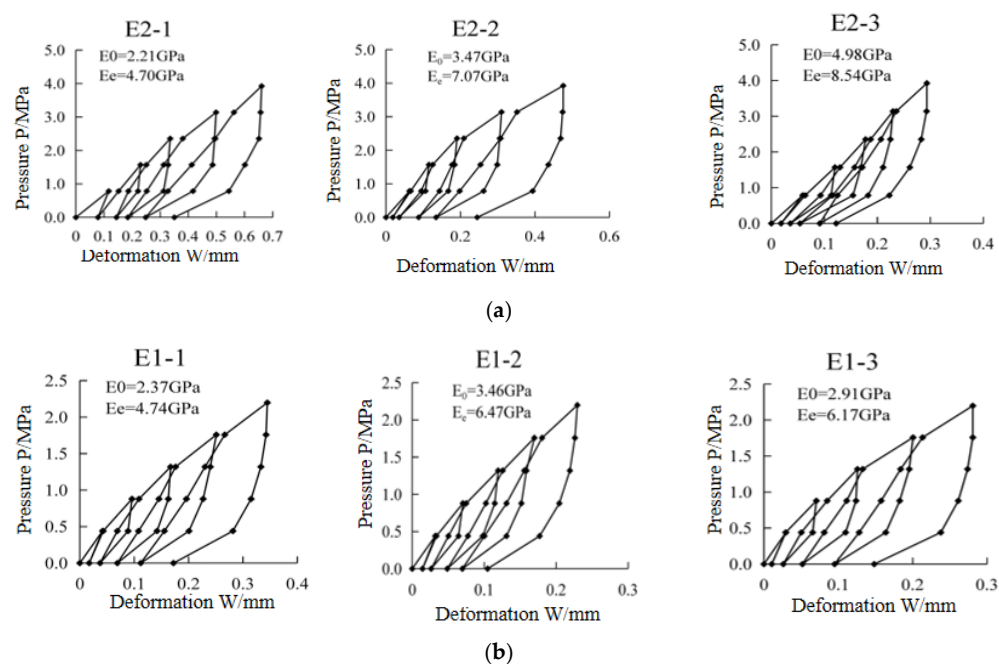


Figure 16. Typical P~W curve of rock mass deformation test using bearing plate method. (a) Marlstone; (b) argillaceous limestone.

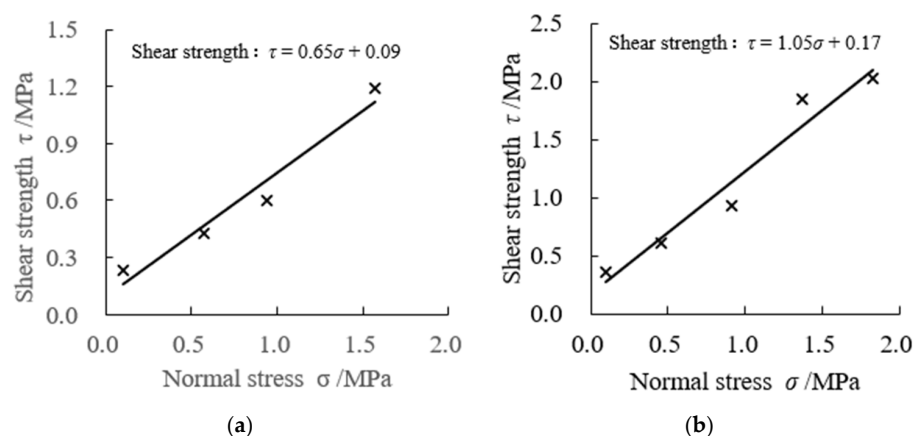


Figure 17. τ-σ relationship curve of natural-state rock mass shear test. (a) Marlstone; (b) argillaceous limestone.

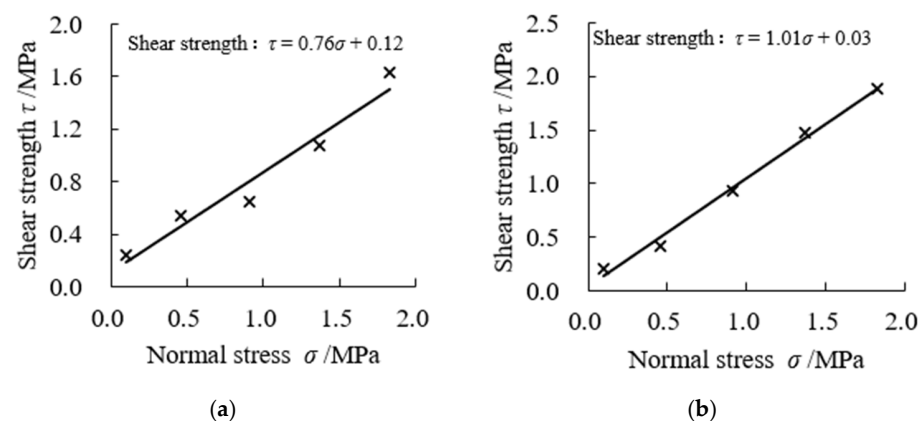


Figure 18. τ-σ relationship curve of concrete-rock interface shear test. (a) Concrete-marlstone; (b) concrete-mudstone.

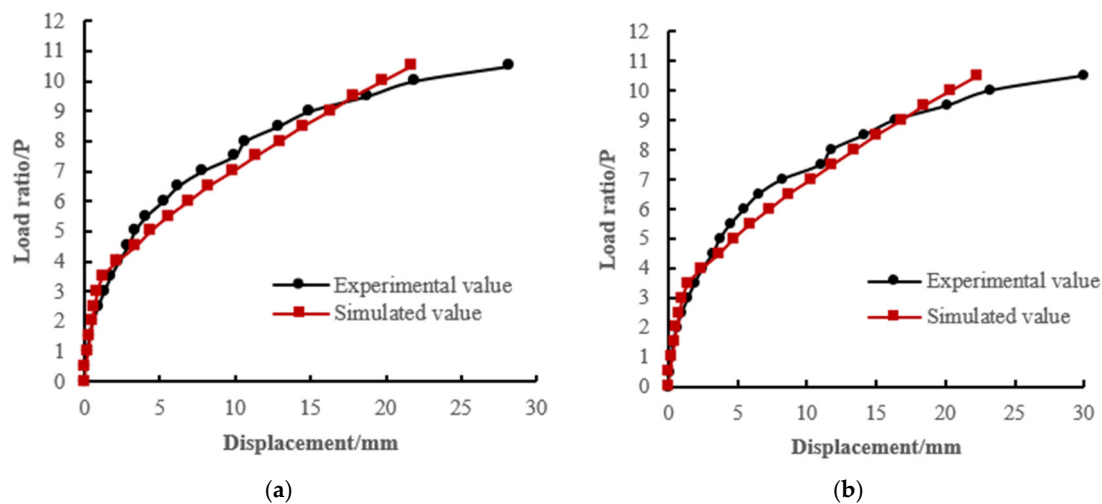


Figure 19. Schematic diagram of constitutive model of contact surface. (a) Right anchor front anchor face measurement point; (b) right anchor rear anchor face measurement point.

Table 1. Basic mechanical parameters of rock mass and concrete used for simulation.

Model Material	Density ρ (kg/m ³)	Deformation Modulus E (GPa)	Poisson's Ratio μ	Friction Angle φ (°)	Cohesion c (MPa)	Tensile Strength R_t (MPa)
Marl	2300	1.0	0.45	33 ($f = 0.65$)	0.15	0.05
Argillaceous Limestone	2400	2.0	0.35	37 ($f = 0.75$)	0.35	0.10
(Reinforced) Concrete	2450	32.5	0.2	/	/	/

Table 2. Basic mechanical parameters of stratification used for simulation.

Rock Stratum Property	Bedding Dip Direction (°)	Bedding Dip Angle (°)	Bedding Cohesion (kPa)	Bedding Friction Angle (°)
Marl	270	14	50	24.5
Argillaceous Limestone			80	27

Note: The joint dip direction is kept consistent with the bedding dip direction. Since the bedding planes face due east in the model, the dip direction is taken as 270°.

Table 3. Basic mechanical parameters of contact surfaces used for simulation.

Contact Surface Type	Normal Stiffness K_n (GPa)	Shear Stiffness K_s (GPa)	φ (°)	c (°)
Concrete-Marl Contact Surface	2.0	1.0	35	0.18
Concrete-Argillaceous Limestone Contact Surface	3.0	2.0	37	0.20

4.3. Analysis of Model Anchorage Simulation Results

4.3.1. Deformation Characteristics

(1) Excavation of Tunnel Anchorage Cavern

The deformation of surrounding rock after tunnel anchorage cavern excavation is shown in Figure 20. After completion of cavern excavation, the maximum crown settlement occurs at the rear part of the anchor chamber, measuring 0.30 mm. The crown settlements of the front and rear anchor chambers are relatively smaller, ranging from 0.10 to 0.20 mm. The heave at the bottom of the anchor chambers induced by excavation is less than the settlement, with a maximum heave deformation of 0.22 mm. The sidewalls of the anchor chambers converge inward overall, with a maximum deformation of 0.21 mm. Due to the

distance between the left and right anchorages, the deformation of the rock mass between them is not significant. Simultaneously, cavern excavation causes settlement in the upper part of the slope and heave in the lower part, but the deformation magnitude is very small and negligible.

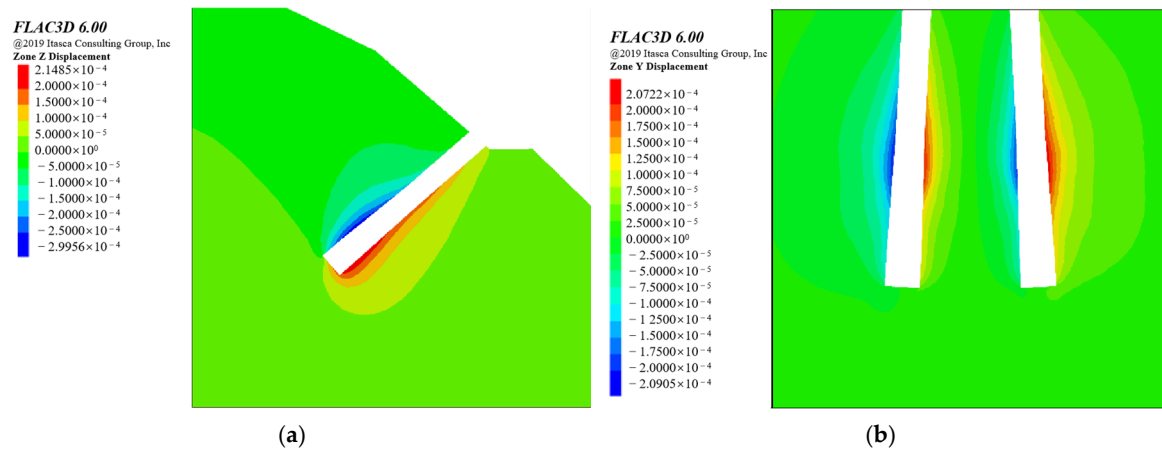


Figure 20. Displacement contour of surrounding rock and anchor plug body after cavern excavation. (a) Vertical displacement contour after cavern excavation (left cavern); (b) horizontal displacement contour after cavern excavation.

(2) Backfilling of Anchor Plug Body

The simulation did not consider the installation of initial support, only excavation. During anchor plug backfilling, the surrounding rock continues to deform, and the anchor plug body has self-weight. This causes the maximum deformation to appear at the crown of the rear part of the anchor plug body, measuring 0.36 mm. Furthermore, backfilling reduces the maximum heave slightly to 0.19 mm. The horizontal convergence deformation is similar to that after excavation.

(3) Application of Main Cable Load

After completion of tunnel anchorage construction, the displacement field and plastic zones were reset to zero. The main cable load was applied to the anchor plug body to simulate normal operation (1P) and overload operation (4P, 7P) states of the tunnel anchorage. The deformation of surrounding rock and anchor plug body under 1P, 4P, and 7P loads is shown in Figure 21.

From Figure 21, it can be seen that under the main cable design load (1P), the maximum displacement of the anchor plug body along the pulling direction occurs at the rear anchor face, with a value of about 0.35 mm. The displacement of the anchor plug body decreases from the rear anchor face to the front anchor face to about 0.25 mm. The displacement of the rock mass between the two anchorages is greater than that on the outer sides. The displacement zone of the surrounding rock exhibits an “ellipsoidal” distribution extending from the area around the anchor plug body to the slope surface, with values ranging between 0 and 0.22 mm, representing slip deformation along the main cable load direction.

Under the main cable design load, the displacement deformations of both the anchor plug body and surrounding rock are very small, all less than 1 mm. Under overload conditions, the displacements of both the anchor plug body and surrounding rock increase with increasing load. Under 4P load, the maximum displacement of the anchor plug body reaches about 2.58 mm, and under 7P load, it reaches 10.54 mm. The deformation of the anchor plug body is significantly greater than that of the surrounding rock, indicating that under main cable load, the anchor plug body drives the surrounding rock to deform

jointly. The deformation of the anchor plug body is directly caused by the load, while the deformation of the surrounding rock is passive.

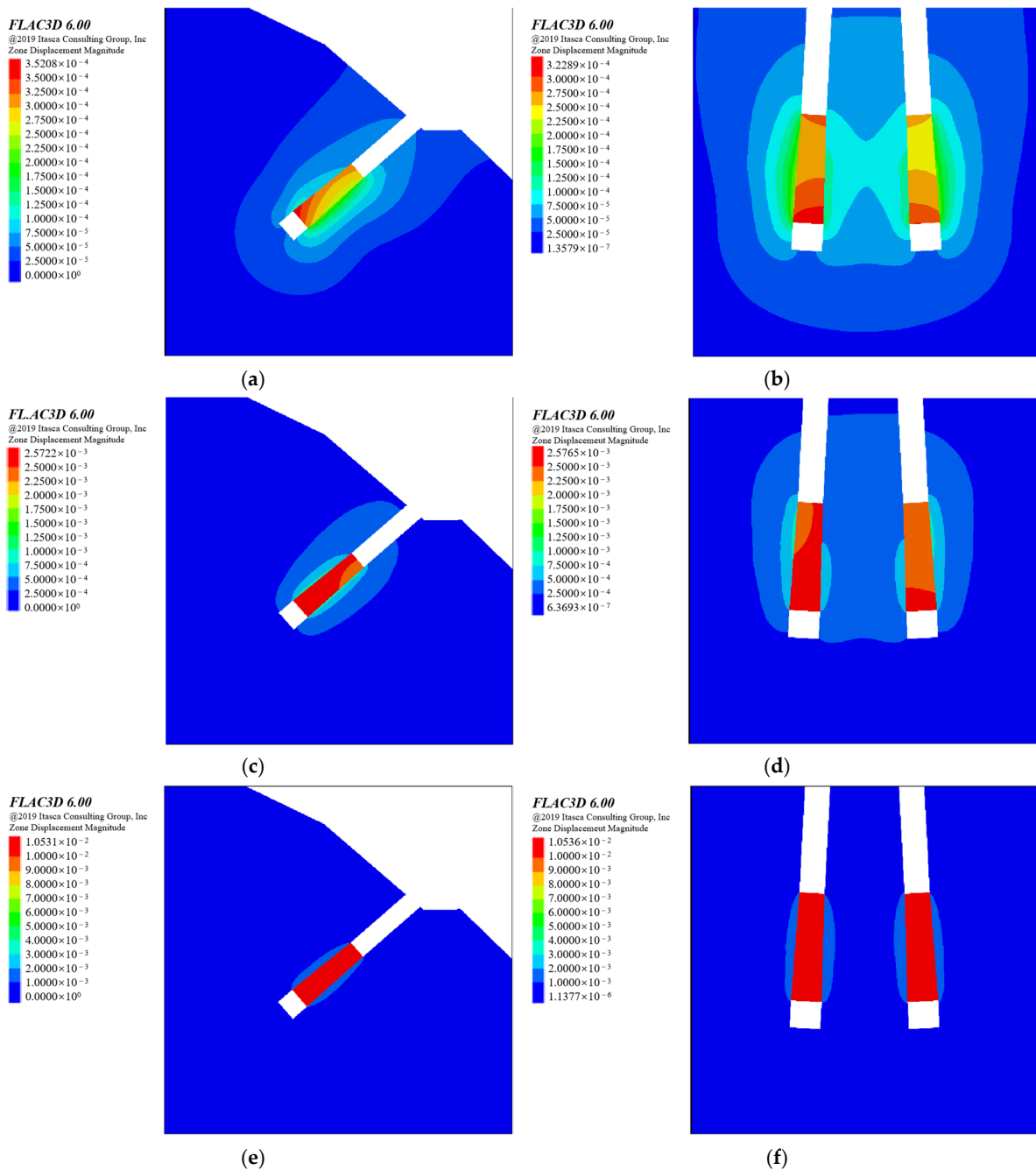


Figure 21. Displacement nephogram under main cable load. (a) 1P load (left anchor); (b) 1P load (oblique section); (c) 4P load (left anchor); (d) 4P load (oblique section); (e) 7P load (left anchor); (f) 7P load (oblique section).

4.3.2. Stress Characteristics

(1) Excavation of Tunnel Anchorage Cavern

After tunnel anchorage excavation, the stress in the surrounding rock is released. The stress distribution in the two side caverns is basically the same, mainly manifesting as compressive stress. Tensile stress zones are mainly distributed on the upper slope surface, with a small distribution range and a maximum value of only about 0.06 MPa. Compressive

stress concentration zones are distributed near the face of the rear anchor chamber, with a maximum value of about -0.92 MPa.

(2) Backfilling of Anchor Plug Body

After anchor plug backfilling, the tensile stress on the upper slope surface increases slightly compared to before backfilling, but the value remains very small. The maximum tensile stress after backfilling appears at the wall toe of the front anchor face of the anchor plug body, about 0.37 MPa. Compressive stress concentration zones are distributed from the rear part of the anchor plug body to the vicinity of the rear anchor chamber face, with a maximum compressive stress of about -0.80 MPa. Furthermore, the surrounding rock around the tunnel anchorage does not experience significantly increased compressive stress due to the gravity of the anchor plug body. This is mainly because the unit weight of the anchor plug body is similar to that of the original rock, and part of the in situ stress was released during cavern excavation. Therefore, backfilling of the anchor plug body does not cause a substantial increase in compressive stress on the surrounding rock around the tunnel anchorage.

(3) Application of Main Cable Load

Figures 22 and 23 show the maximum and minimum principal stress contours of the surrounding rock and anchor plug body after applying the main cable load.

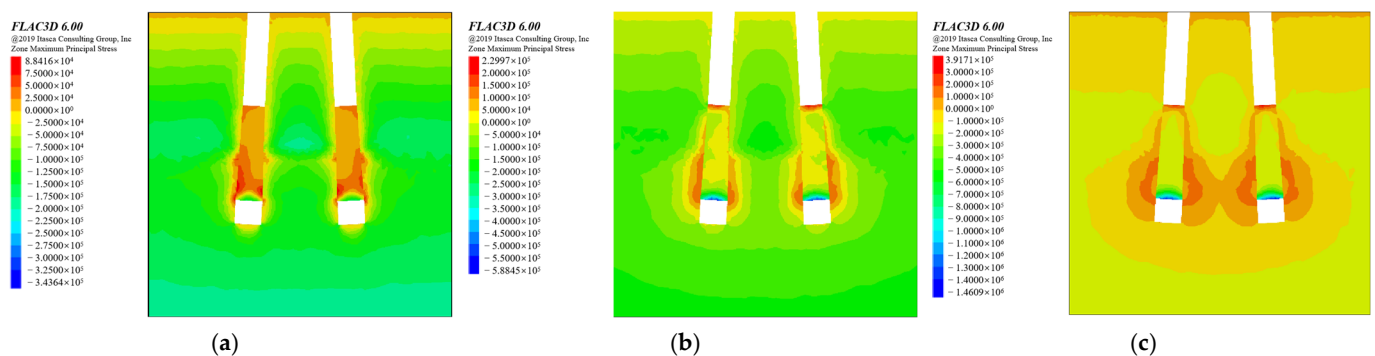


Figure 22. Maximum principal stress nephogram under main cable load. (a) 1P load; (b) 4P load; (c) 7P load.

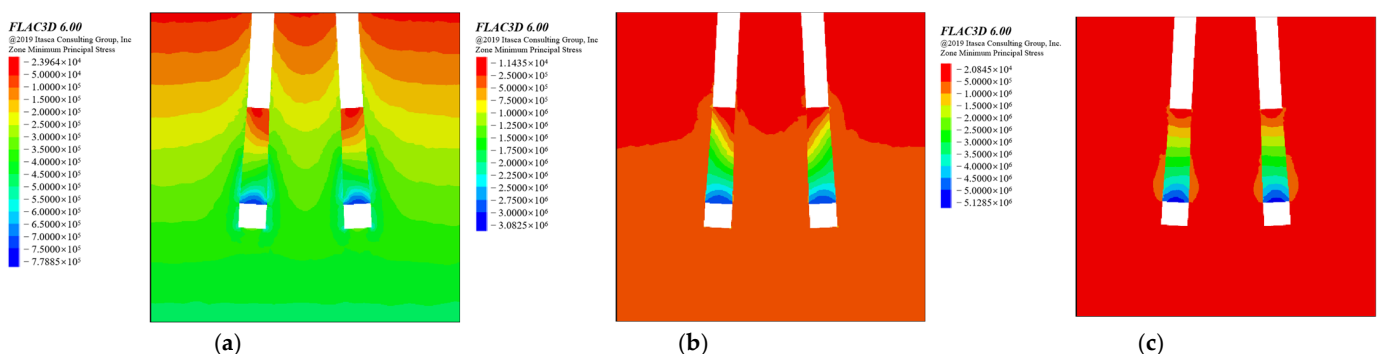


Figure 23. Minimum principal stress nephogram under main cable load. (a) 1P load; (b) 4P load; (c) 7P load.

After applying the main cable load, the surrounding rock overall maintains a low level of compressive stress. The upper slope surface still maintains a low level of tensile stress. The anchor plug body and the tightly adhering part of the surrounding rock are subjected to tension. Under 1P, 4P, and 7P loads, the maximum tensile stress values are 0.35 MPa, 0.50 MPa, and 0.86 MPa, respectively. Most of the surrounding rock around the anchor

plug body simultaneously bears the shear and compression forces from the anchor plug body, mainly in a compression-shear stress state. Under 1P, 4P, and 7P loads, the maximum compressive stress values are -1.11 MPa, -5.11 MPa, and -10.49 MPa, respectively.

4.3.3. Plastic Zone Characteristics

The distribution of plastic zones in the tunnel anchorage under different main cable loads is shown in Figure 24. Since the ubiquitous-joint plasticity model was used, the plastic zones are overall quite complex. The analysis primarily focuses on the extent of the plastic zones.

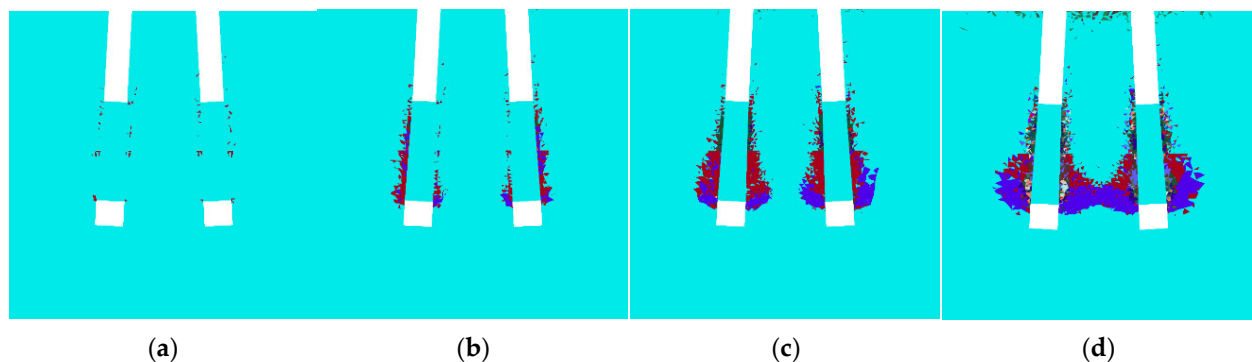


Figure 24. Distribution of plastic zone under main cable load. (a) 1P load; (b) 4P load; (c) 7P load; (d) 10P load.

From Figure 24, it can be seen that under the 1P design load, the plastic zones of the left and right anchor plug bodies are distributed within a certain area around themselves, independent and not influencing each other. Very small plastic zones also appear near the front and rear anchor chambers. When the main cable load on the model anchorage is small, both the tunnel anchorage and its surrounding rock are in an elastic working state. Only a small portion of the surrounding rock undergoes plastic yielding, occurring first near the rear anchor face. As the load gradually increases, the yielding range of the surrounding rock near the rear anchor face increases and shows a trend of upward propagation. Part of the upper slope surface also experiences tensile yielding. As the main cable load continues to increase, the mutual interaction effect between the left and right anchorages becomes increasingly significant. The plastic zones of the surrounding rock gradually expand, and the plastic zones in the rock between the anchorages gradually become interconnected (Figure 24d).

As shown in Figure 25, the plastic zone of the entire model ultimately penetrates to the ground surface in an inverted trapezoid shape. The rock mass below fails in shear along the 14° bedding plane direction. The term “j-shear” in the figure refers to joint shear yielding.

4.3.4. Bearing Characteristics of Contact Interface

The anchor–rock contact interface is established during anchor plug backfilling. Therefore, this section mainly analyzes the characteristics of the contact interface after anchor plug backfilling and during the main cable loading stage.

(1) Normal Contact

During anchor plug backfilling, due to gravity, the top of the contact interface may not be tightly bonded. However, as the main cable load gradually increases, the anchor plug body experiences upward displacement, and the contact area at the top gradually increases, enabling better mobilization of the surrounding rock to bear the load together, as shown in Figure 26 (taking the left anchor as an example).

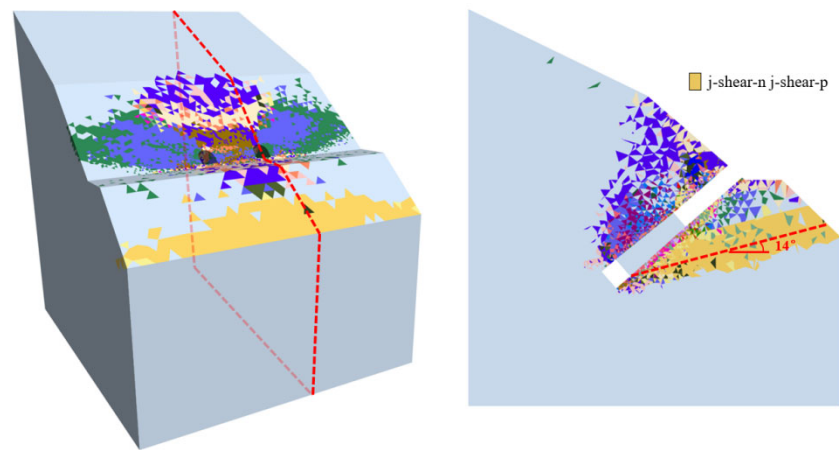


Figure 25. Penetration of plastic zone of tunnel-type anchorage model.

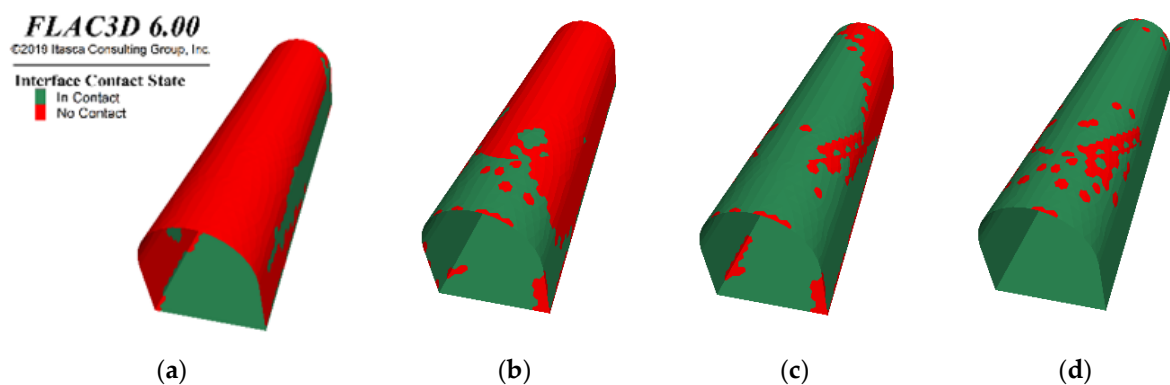


Figure 26. Normal contact condition of plug body contact surface. (a) Anchor plug backfilling; (b) 1P load; (c) 4P load; (d) 7P load.

(2) Tangential Slip

Under 1P load, most of the main cable load is balanced by the self-weight component of the anchor plug body. Tangential slip on the anchor–rock contact interface occurs only in a very small area at the rear. As the main cable load gradually increases, the tangential slip range gradually expands. In the later stages, almost the entire contact surface exhibits tangential slip, as shown in Figure 27 (taking the left anchor as an example).

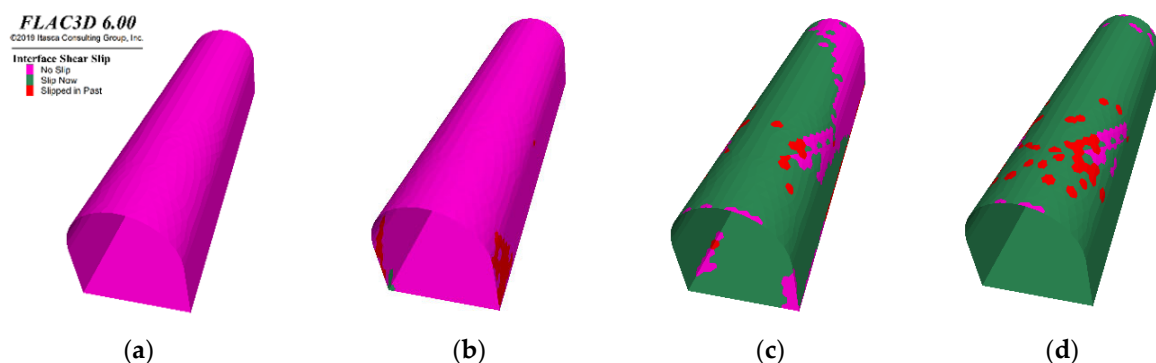


Figure 27. Tangential slip condition of plug body contact surface. (a) Anchor plug backfilling; (b) 1P load; (c) 4P load; (d) 7P load.

(3) Deformation Analysis

The maximum tangential deformations of the anchor–rock contact interface under 1P, 4P, and 7P loads are 0.20 mm, 2.16 mm, and 9.65 mm, respectively. The maximum

normal deformation of the anchor–rock contact interface does not change significantly with increasing load level.

5. Discussion

This study presents a comprehensive investigation into the mechanical behavior and failure mechanisms of twin tunnel-type anchorages adjacent to bifurcated tunnels in soft rock strata, using a combined approach of 1:10 in situ scaled model testing and three-dimensional numerical simulation. The integrated methodology enabled both localized deformation monitoring and macroscopic field analysis, offering a more complete understanding of the anchorage–rock interaction under design and overload conditions.

One of the key breakthroughs of this research is the identification of the cumulative interaction effect between twin anchorages, which has not been sufficiently addressed in previous studies. Under increasing cable loads, the plastic zones around the two anchorages progressively expanded and eventually coalesced, forming a characteristic “inverted trapezoid” failure pattern that propagated to the ground surface. This phenomenon, validated by both experimental and numerical results, highlights the importance of considering twin-anchor configurations in stability assessments, particularly in spatially constrained environments with weak rock masses.

Another significant finding is the dominant role of geological structure—specifically the persistent 14° bedding plane—in governing the ultimate failure mode. The system did not fail along the anchor–rock interface, but rather through shear failure along this plane, indicating that the interface itself remained intact while the surrounding rock mass underwent progressive yielding. This underscores the necessity of incorporating major structural discontinuities into both design and numerical modeling.

Despite the robustness of the findings, several limitations should be acknowledged. First, the in situ model test, while large-scale and geologically representative, was conducted at a shallow depth with more weathered rock compared to the actual bridge anchorage. As a result, the observed deformation and failure thresholds may be conservative when extrapolated to deeper, less weathered conditions. Second, the numerical model assumed a homogeneous and continuous rock mass with simplified bedding plane behavior, which may not fully capture the complexity of natural discontinuities, such as fractures, joints, or varying saturation conditions. Third, the influence of dynamic loads (e.g., traffic or seismic activity) and long-term creep behavior were not considered in this study, limiting the applicability of the results to long-term operational safety assessments.

Future research should aim to address these limitations through several avenues. First, more refined constitutive models that account for the stochastic nature of fractures and the anisotropic behavior of layered rock masses should be developed and validated against field data. Second, the dynamic response of tunnel-type anchorages under seismic or traffic-induced vibrations warrants systematic investigation, particularly in terms of cumulative damage and fatigue effects. Third, long-term monitoring of actual bridge anchorages in similar geological settings could provide valuable data for calibrating time-dependent deformation models. Finally, parametric studies on anchor spacing, burial depth, and bedding orientation could help establish design guidelines that mitigate the cumulative interaction effects observed in twin-anchor systems.

In summary, this study contributes both experimentally and numerically to the understanding of tunnel-type anchorage behavior in complex geological settings. The findings provide a scientific basis for safer and more economical design of suspension bridge anchorages in mountainous regions, especially when adjacent underground structures are present.

6. Conclusions

This study integrated 1:10 in situ scaled model tests with three-dimensional numerical simulations to investigate the mechanical behavior and failure mechanisms of a suspension bridge's twin tunnel-type anchorages constructed adjacent to bifurcated tunnels in soft rock strata. The key findings are summarized as follows:

- (1) **Bearing Performance and Failure Thresholds:** The in situ model test demonstrated that the anchorage system exhibits excellent serviceability under the design load (1P), with a maximum anchor displacement of only 0.35 mm and minimal interface slip (0.06 mm). The system reached its yield point at 7P and achieved peak strength at 10.5P, at which point significant surface cracks (up to 5 mm) developed. The load transfer along the anchor plug was highly non-uniform, with the rear section bearing the majority of the load, suggesting that the current design length may be conservative.
- (2) **Cumulative Effect of Twin Anchorages:** A significant finding is the progressive interaction between the two adjacent anchorages. As the cable load increased beyond the design level, the plastic zones in the rock mass between the two anchors expanded and eventually coalesced. This cumulative effect, which cannot be observed in single-anchor studies, resulted in an “inverted trapezoid” failure pattern that propagated to the ground surface. This highlights that twin-anchor configurations in close proximity can have a mutually detrimental impact under extreme loading conditions.
- (3) **Failure Mechanism and Geological Control:** The ultimate failure of the system was not a sliding failure along the anchor–rock interface, but rather a pullout failure of the anchor plug together with a substantial mass of surrounding rock. The failure mode was governed by the geological structure, with shear failure ultimately occurring along the persistent 14° bedding plane. This underscores the critical importance of characterizing and incorporating major structural planes (e.g., bedding, joints) into stability analyses for tunnel-type anchorages.
- (4) **Anchor–Rock Interface Behavior:** The interface played a crucial role in load transfer. Initially, the top of the interface was not fully bonded due to the self-weight of the concrete. However, under increasing cable load, the upward displacement of the anchor plug engaged the contact, mobilizing the surrounding rock. While normal contact improved with load, tangential slip progressively increased, reaching 9.65 mm at 7P, indicating a gradual loss of shear stiffness along the interface.

In conclusion, while the anchorage system is stable under design loads, its ultimate capacity and failure mode are controlled by the interplay between the twin anchors and the inherent weakness of the rock mass structure (bedding planes). The findings provide a scientific basis for the design and construction of tunnel-type anchorages in similar challenging geological and spatial settings, emphasizing the need to account for cumulative twin-anchor effects and the presence of weak structural planes.

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