

Article

Back Analysis of Surrounding Rock Parameters and Stability Assessment of an Underground Powerhouse in Northwest China

Xuan Hu ¹, Naifei Liu ^{2,3,*}, Ning Li ⁴ and Yue Zhong ¹¹ Shaanxi Railway Institute, Weinan 714000, China; 15091543331@sxri.net (X.H.); 201610573@sxri.net (Y.Z.)² School of Civil Engineering, Xi'an University of Architecture and Technology, Xi'an 710055, China³ Shaanxi Key Laboratory of Geotechnical and Underground Space Engineering, Xi'an 710055, China⁴ School of Civil Engineering, Xi'an University of Technology, Xi'an 710048, China; ningli@xaut.edu.cn

* Correspondence: liunaifei@xauat.edu.cn

Abstract

The underground powerhouse of a hydropower station in Northwest China features a large height-to-span ratio and complex geological conditions, posing critical challenges for surrounding rock stability. This study employs a systematic framework of “parameter inversion–stability analysis–support optimization”. The main contributions are: (1) A forward-backward analysis method integrating monitoring data, numerical calculation, and parameter optimization was used to invert the physical and mechanical parameters of the surrounding rock, establishing a practice-consistent numerical model. (2) Considering complex geological conditions from a prior collapse, systematic analyses of deformation characteristics, stress distribution, and overall stability during subsequent bench excavation were conducted, revealing evolutionary patterns of deformation, reproducing the collapse mechanism, and evaluating rock mass safety. (3) An optimized reinforcement scheme was proposed, involving additional anchor cables at arch shoulders and two anchor bolts on each sidewall (Sidewalls 2), with comparative analysis confirming its feasibility. Key findings: (i) Based on the inversion of monitoring data, the obtained physical and mechanical parameters of the surrounding rock can effectively reflect the actual characteristics of the rock mass and align with monitoring trends. (ii) Stability analysis of subsequent layered excavation, accounting for prior collapse impacts, shows that while deformation at key points continued to increase, it remained below critical thresholds, maintaining overall stability—prestressed anchor cables significantly controlled deformation. (iii) The optimized support scheme yields comparatively favorable outcomes with respect to the containment of surrounding rock displacement and the internal forces within the supporting. These outcomes provide valuable references for the design and construction of similar underground powerhouses.

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Keywords: underground powerhouse; surrounding rock; parameter inversion; stability analysis; reinforcement optimization

1. Introduction

During the construction of underground powerhouses, the surrounding rock mass undergoes stress redistribution due to excavation disturbance, typically resulting in unloading deformation toward the cavern interior [1]. Given the complex structure of underground powerhouses and their strong dependence on topographic and geological conditions, intricate interactions exist between the concrete structures and surrounding rock masses.

Influenced by the physical and mechanical properties of different rock layers, underground powerhouses experience more complex stress conditions compared to surface powerhouses. Consequently, regions with well-developed faults and joints, poor rock mass integrity, and inferior mechanical parameters are prone to excessive local deformation, rock spalling, and even instability [2,3].

The mechanical parameters of surrounding rock are the basis for stability analysis of underground caverns, and their accuracy directly determines the reliability of numerical simulation results. However, due to the complexity and uncertainty of geological conditions, parameters determined solely through laboratory testing or engineering analogy often fail to reflect the actual in situ state. Therefore, parameter inversion methods based on on-site monitoring data have become an important way to obtain the true mechanical parameters of surrounding rock. In the context of elastic problems in tunnel engineering, Song et al. [4] applied a differential evolution algorithm to optimize mechanical parameters and support conditions for an inclined shaft tunnel. Ling et al. [5] developed an improved PSO-BP neural network hybrid method for displacement back-analysis of surrounding rock parameters. Zhang et al. [6] proposed an ACSSA-ELM-based stability analysis method integrating deformation prediction with parameter inversion, validated via the Huayang Tunnel case study. Based on field monitoring data from the Baihetan hydropower station, Yan et al. [7] performed a back-analysis of the mechanical parameters of surrounding rock masses in large underground caverns using an orthogonal design-BP neural network approach and further investigated the stress and deformation evolution characteristics during construction to assess cavern stability.

In recent years, intelligent optimization algorithms and machine learning methods have achieved remarkable progress in parameter inversion. Li et al. [8] used the PSO-BP algorithm to invert the surrounding rock parameters of the underground powerhouse of Shuangjiangkou Hydropower Station. Based on monitoring data from multi-point displacement meters and anchor stress meters, intelligent identification of elastic modulus, Poisson's ratio, friction angle, and cohesion was achieved. The inversion results were consistent with the on-site monitoring values. Yan et al. [9] proposed an in situ stress field inversion method for underground cavern surrounding rock based on particle swarm optimization-backpropagation neural networks, improving inversion accuracy through optimization of neural network weights and thresholds. Shi et al. [10] established a GA-PSO-BP neural network model, which integrates the genetic algorithm and particle swarm optimization algorithm to optimize the weights and thresholds of the BP neural network. It achieved good inversion results in the Zhaotong Tunnel project, with an average relative error of only 4.34%. Ma et al. [11] used the discrete element mechanics parameter inversion method combined with the PSO algorithm to verify the effectiveness of the method through typical landslide cases. Li et al. [12] combined the optimized PSO with support vector machine for rock mechanics parameter inversion, with an error of merely 0.91% compared to the assumed actual parameters. The above research indicates that parameter inversion technology has become an important bridge connecting laboratory testing with field conditions, providing a reliable parameter basis for subsequent stability analysis.

After obtaining accurate rock mechanics parameters, conducting systematic stability analysis is the key to ensuring the safety of underground factory construction. Scholars have conducted in-depth research on the stability of underground caverns under different geological conditions using numerical simulation methods. Tian et al. [13] investigated an underground powerhouse in Southwest China, inverting the initial in situ stress field through multiple linear regression based on measured stress and engineering geological data, and subsequently studying surrounding rock stability during cavern group excavation. Liu et al. [14] conducted research on the Shuangjiangkou Hydropower Station underground

powerhouse, revising the regional stress field through geological exploration, field monitoring, and numerical simulation, and investigating deformation and failure mechanisms of high-stress hard rock surrounding rock and corresponding reinforcement measures. Han et al. [15] examined the Wujiang Pengshui Hydropower Station underground powerhouse, investigating abnormal monitoring values during the operation period through back analysis, statistical analysis of monitoring data, and numerical stability calculations based on geological conditions and over ten years of safety monitoring data. Wang et al. [16] studied the Baihetan Hydropower Station left bank underground powerhouse, which is subjected to high in situ stress and complex geological conditions, combining field monitoring, field investigation, and numerical simulation to investigate surrounding rock deformation and failure mechanisms. Li et al. [17] addressed the excavation of high-stress underground powerhouse cavern groups at the Shuangjiangkou Hydropower Station, constructing a high-precision microseismic monitoring system and employing discrete element numerical simulation to investigate the evolution of microseismic source parameters during collapse formation under combined high-stress and structural plane control. Li et al. [18] investigated deformation characteristics of sidewalls and anchorage mechanisms of prestressed cables in steeply dipping layered rock strata, revealing the stress transfer patterns of cables in layered surrounding rock. Huang et al. [19] conducted damage assessment of high-stress brittle surrounding rock in deep-buried, large-span, high-sidewall underground caverns, systematically evaluating damage evolution characteristics through integrated in situ measurement and numerical simulation. Imashev et al. [20] systematically investigated the influence of bedding angle on the stress state of surrounding rock, revealing that significant changes in stress concentration zones occur when the layer inclination exceeds 30°. Petlovanyi and Sai [21] proposed a method for identifying near-failure states based on the H/Lc ratio. These studies collectively embody the comprehensive research framework of “parameter inversion—stability assessment—reinforcement optimization,” providing important references for this research.

Support optimization represents the extension of stability analysis and the practical application endpoint. Upon obtaining true rock mass parameters and completing stability evaluation, optimizing the support design becomes crucial for ensuring long-term stability. Li et al. [22] employed FLAC3D numerical simulation to optimize excavation elevation, sequence, and support schemes for underground cavern groups at a pumped storage power station. Wang et al. [23] utilized finite difference programs to establish three-dimensional numerical models based on engineering geological data, inverting initial in situ stress fields and optimizing rock bolt spacing and row spacing. In recent years, digital twin technology has provided novel technical pathways for underground cavern support optimization. Sharafat et al. [24] proposed a digital twin-driven stability optimization framework for large underground caverns, achieving iterative optimization and dynamic adjustment of support parameters through closed-loop feedback between real-time monitoring data and numerical models, significantly improving the scientific rigor and economic efficiency of support design. Ma et al. [25] conducted multi-objective optimization research on excavation schemes for underground cavern groups, comparing the effects of different excavation sequences on surrounding rock stability through optimized DDARF methods to determine optimal excavation schemes. Tao et al. [26] analyzed the stability of deeply buried underground cavern surrounding rock at the Jinping II Hydropower Station based on improved block theory and the Unwedge program, providing theoretical foundations for support design. These studies indicate that numerical simulation-based support optimization requires accurate mechanical parameters and reliable stability analysis as prerequisites, with the three components forming a complete technical chain for underground powerhouse safety assessment.

In summary, existing research has achieved substantial results in parameter inversion, stability analysis, and support optimization; however, several deficiencies remain: First, for underground powerhouses that have experienced collapse, the mechanical parameters of surrounding rock change after disturbance, and how to obtain post-disturbance parameters through monitoring data inversion requires further investigation. Second, post-collapse stability analysis should treat the collapse as a natural stage in the excavation process rather than an independent event, to better reflect the true response of the surrounding rock. Third, research on rational evaluation and targeted optimization of support systems based on stability analysis remains insufficient.

In view of these considerations, this study investigates an underground powerhouse of a hydropower station in Gansu Province, China, employing discrete finite element numerical simulation methods combined with engineering geological data and field monitoring data to conduct surrounding rock stability analysis and support optimization research. First, surrounding rock physical and mechanical parameters are inverted based on monitoring data to establish a numerical analysis model consistent with engineering practice; second, considering the complex geological conditions resulting from a previous collapse event, systematic analyses of deformation characteristics, stress distribution, and overall stability during subsequent bench excavation are conducted to evaluate the safety status of the surrounding rock; finally, the initial support scheme is optimized, and targeted support improvement measures are proposed. The research findings provide valuable references for the design and construction of underground powerhouses under similar geological conditions.

2. Project Overview

2.1. Engineering Description

The hydropower station project in Gansu mainly consists of three parts: water diversion hub, water diversion system, and plant buildings. The main powerhouse has a length of 48.5 m and a width of 17.6 m. The powerhouse axis is located 9.80 m from the upstream wall and 7.80 m from the downstream wall. The auxiliary powerhouse is situated on the left side of the main powerhouse, arranged in parallel, with dimensions of 22.8 m (length) $\times$ 17.6 m (width), at an elevation of 2037.00 m. The auxiliary powerhouse arch springing is at elevation 2058.00 m, with a radius coefficient of 0.7 (i.e., $R = 0.7B$, where B is the cavern span), crown elevation of 2061.70 m, and central angle of 102.94° .

2.2. Geological Conditions

The underground powerhouse area is characterized by alternating soft and hard rock lithologies. The argillaceous slate exhibits relatively weak properties, with susceptibility to softening upon water exposure. Various structural planes are well-developed, resulting in poor rock mass integrity. The surrounding rock classification of the powerhouse cavern is generally poor, with Class III and Class IV rock masses at the sidewalls and Class IV at the crown. Consequently, the self-stability and overall stability are compromised due to the poor rock mass quality. Fractures and faults present in the powerhouse crown create unstable blocks within the cavern dome. Therefore, the adverse combination of structural planes and the weak properties of argillaceous slate provide geological conditions conducive to block instability resulting from unfavorable structural plane combinations, potentially evolving into large-scale deformation and failure.

The powerhouse area has experienced multiple tectonic episodes, resulting in extremely complex geological conditions with well-developed faults and fractures. The crown and sidewall surrounding rock are affected by six faults and intersecting fractures, dividing the rock mass into blocks of varying sizes and creating potentially unstable rock

masses. During excavation, frequent rock falls and collapses occurred at the crown and sidewalls. The downstream sidewall contains faults F1, F2, F3, F4, and F6 (reverse faults), while the upstream sidewall contains fault F5.

2.3. Collapse Events

27 May 2011: A major collapse occurred at the powerhouse. At 16:00 on May 27, a triangular wedge located at chainage 0+004 to 0+010 m on the downstream side of fault F3 in the crown suddenly collapsed, triggering sudden stress release within the rock mass and causing loosening of the surrounding rock. Subsequently, collapses occurred successively at the downstream sidewall and left end wall. The primary cause of this collapse was the cutting effect of fault F3 and fractures, which created an unstable wedge body. The rock at this location was highly fractured with dense fractures, indicating extremely poor stability.

12 July 2016: During collapse treatment construction, rock falls and debris flow suddenly occurred at the crown of the powerhouse at chainage 0+030 (between rings 16–17). From July 12 to July 17, small rock fragments continuously fell from the debris flow opening. On July 17, the opening was blocked by three rock fragments, and by July 20, sounds of falling rocks could be heard during monitoring. Due to continuous rock falls within the debris flow tunnel, the collapse remained unstable. Three-dimensional laser scanning investigation of the internal cavity revealed dimensions of 5.9 m (height) $\times$ 7.9 m (east–west length) $\times$ 6.0 m (north–south width). Preliminary analysis indicated that the weak zone at the cavity location trended east–west, with a dip angle of the potential sliding surface of 22°.

3. Back Analysis of Surrounding Rock Mechanical Parameters

3.1. Instrumentation Layout

According to the “Instrument Monitoring Data” and “Installation Chamber Monitoring Instrument Description,” instruments near the calculation section of the installation chamber include rock bolt stress meters (K1, K2, K3, K4, K5, K6) and multi-point extensometers (M1, M2, M3), among which K1, K2, and M1 provided relatively reliable data. Following collapse reinforcement, downstream expansion excavation of the powerhouse induced stress redistribution in the surrounding rock, resulting in elastic-plastic deformation and rapid increases in monitoring instrument readings. The instrument layout is shown in Figure 1.

3.2. Back-Analysis Methodology and Framework

The inversion of surrounding rock mechanical parameters employed a “monitoring data-numerical computation-parameter optimization” coupled analysis approach. The specific methodology is as follows: First, based on engineering geological investigation data and similar project experience, initial value ranges for surrounding rock physical and mechanical parameters were determined; second, a discrete finite element numerical model was established to simulate the sidewall expansion excavation process, calculating theoretical response values at monitoring points (rock bolt stress meters K1, K2 and multi-point extensometer M1); then, calculated values were compared with measured monitoring data, with minimization of the discrepancy between the two serving as the objective function. Through continuous adjustment of surrounding rock parameters (deformation modulus, cohesion, internal friction angle, etc.), the calculated curves were progressively converged toward the measured curves; finally, when the calculated and measured values at each monitoring point demonstrated similar trends and magnitudes, the inverted parameters were determined. The core concept of this method utilizes field monitoring data to “calibrate” the numerical model, enabling model parameters to reflect the true mechanical

Stage I (Data Preparation): Collect engineering geological investigation data and similar project experience to determine initial value ranges for surrounding rock physical and mechanical parameters (deformation modulus E , cohesion c , internal friction angle φ , Poisson's ratio ν .); simultaneously, obtain field monitoring data, including measured displacement and stress data from rock bolt stress meters K1, K2 and multi-point extensometer M1.

Stage II (Numerical Modeling and Computation): Establish a discrete finite element numerical analysis model, simulate the sidewall expansion excavation process, and calculate theoretical response values (displacement, stress) at each monitoring point.

Stage III (Comparison Analysis and Optimization): Extract measured monitoring data and perform error comparison analysis between calculated and measured values to determine whether the error has been minimized. If the error has not reached a minimum, proceed to Stage IV for parameter adjustment; if the error has reached a minimum, proceed to Stage V to determine optimal parameters.

Stage IV (Parameter Adjustment and Iteration): Adjust surrounding rock parameters (E , c , φ , ν , etc.), employ optimization algorithms for iterative computation, and feed back to Stage II for re-computation, forming a closed-loop iterative process until the error satisfies convergence criteria.

Stage V (Results Output): Determine the optimal combination of surrounding rock mechanical parameters, verify whether the calculated curves approximate the measured curves (consistent trends, similar magnitudes), and output the inverted surrounding rock physical and mechanical parameters.

Stage VI (Application Verification): Input the inverted parameters into the numerical model for forward analysis verification, comparing the verification numerical computation results with field monitoring data. If verification is successful, the inverted parameters can be applied to subsequent stability analysis and support design; if verification fails, return to Stage I to re-adjust initial parameter ranges and re-conduct the back analysis.

3.3. Inversion Results

The initial search ranges for the parameters subjected to back-analysis are presented in Table 1. The density, which is fixed at 2650 kg/m^3 according to laboratory test data, is not included as an inversion variable. The initial ranges for the remaining parameters are established on a comprehensive basis, with reference to the geological report, the Standard for Engineering Classification of Rock Mass (GB/T 50218-2014 [27]), and empirical data derived from similar engineering practices.

Table 1. The initial value ranges for the back-analysis parameters.

Rock Type	Source	Density (g/cm^3)	Deformation Modulus ($\times 10^3 \text{ MPa}$)	Poisson's Ratio	Cohesion (MPa)	Internal Friction Angle ($^\circ$)
Silty argillaceous slate intercalated with sandy slate	geological report	2.74	2.89~6.75	0.21~0.29	-	-
	Rock mass class IV	2.45~2.25	6.0~1.3	0.3~0.35	0.7~0.2	39~27
Fault fracture zone and weathered rock mass	Rock mass class IV	<2.25	<1.3	>0.35	<0.2	<27

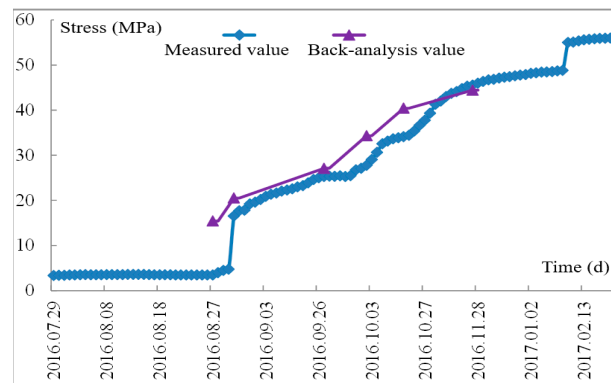
Through simulation of multiple sidewall expansion excavation processes, surrounding rock parameter inversion was performed, yielding the physical parameters presented in Table 2.

Table 2. Physical and mechanical parameters of surrounding rock.

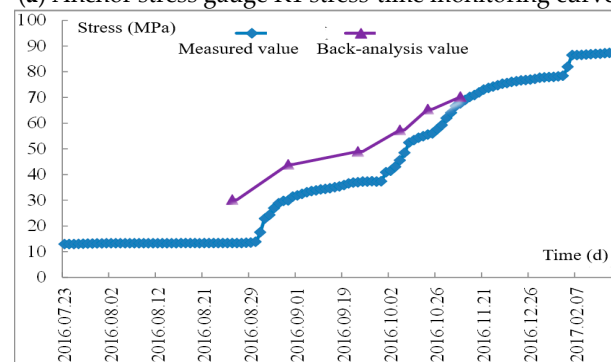
Rock Type	Density (g/cm ³)	Deformation Modulus ($\times 10^3$ MPa)	Poisson's Ratio	Cohesion (MPa)	Internal Friction Angle (°)
Silty argillaceous slate intercalated with sandy slate	2.74	2.9	0.35	0.35	33
Fault fracture zone and weathered rock mass	2.25	1.0	0.4	0.10	20

The monitoring instruments deployed in the installation hall section included anchor stress meters (K1–K5) and multi-point extensometers (M1–M3). Among these, K1 was installed on 9 January 2016; K2, K3 and K5 were installed on 10 January 2016; M1 was installed on 11 January 2016; and M2 was installed on 12 January 2016. Baseline readings for all instruments were taken on 21 January 2016. Following August 2016, three additional excavation stages were carried out on the downstream sidewall of the powerhouse installation hall, each causing a notable change in the monitored responses. According to the monitoring data provided by the client, K3 showed unstable readings shortly after installation, K4 exhibited significant fluctuations throughout the monitoring period, and M2 recorded negative displacement values. Therefore, only instruments K1, K2 and M1, which provided relatively complete and reliable data, were selected for the back-analysis and model validation described in this study. Under this set of inverted parameters, the monitored and inversion values for K1, K2, and M1 during the sidewall expansion excavation process are presented in Figure 3.

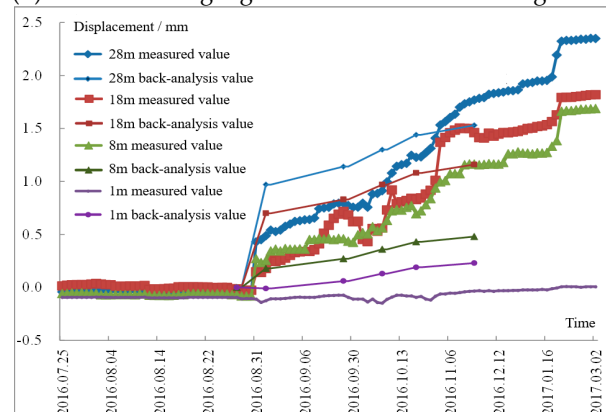
As shown in Figure 3, when employing the back-analyzed parameters, the average relative fitting errors for the anchor stress meters K1 and K2, and the multi-point extensometer M, are 3.1%, 9.3%, and 7.8%, respectively, with the maximum fitting errors across all instruments being confined within 13.3%. Moreover, the variation trends of the back-calculated curves are generally consistent with those of the field-measured data. The initial back-analyzed values for K1 and K2 are slightly higher than the measured values, possibly due to relatively delayed instrument installation and consequently smaller collapse loads. This is primarily attributed to the alternating soft and hard rock lithologies in the powerhouse area, the weak properties of argillaceous slate with susceptibility to softening upon water exposure, well-developed structural planes, poor rock mass integrity, and long-term exposure of the collapse-free surface resulting in dense joint and fracture networks in the shallow cavern surface. The simulation only considered the influence of faults F1, F3, and F5, treating the remaining rock mass as homogeneous; therefore, deviations between back-analysis and actual monitoring results were inevitable. Furthermore, the installation chamber sidewall expansion excavation represents only a preliminary back analysis; new geological conditions may be continuously revealed during subsequent excavation, necessitating enhanced monitoring and progressive calibration and correction.



(a) Anchor stress gauge K1 stress-time monitoring curve



(b) Anchor stress gauge K2 stress-time monitoring curve



(c) Multipoint displacement meter M1 displacement-time monitoring curve

Figure 3. Displacement-time monitoring curves of monitoring and inversion values for K1, K2, and M1.

4. Stability Analysis of the Underground Powerhouse

4.1. Section Selection

Based on the longitudinal and transverse geological structural sections provided by the client, the calculation model is shown in Figure 4.

The initial support for this section employs combined shotcrete and rock bolt support, specifically: the crown employs 25 m long, 1000 KN capacity prestressed cables at 4 m spacing in both directions; 9 m long, $\Phi 32$ mm, 120 KN capacity fully bonded mortar rock bolts at 1.34 m spacing in both directions; $\Phi 6.5$ mm wire mesh; and 23 cm thick C20 shotcrete (with an initial 5 cm thick polypropylene fiber concrete layer). The upstream and downstream walls employ alternating 9 m long, $\Phi 32$ mm, 120 KN capacity and 6 m long, $\Phi 28$ mm, 100 KN capacity fully bonded mortar rock bolts at 150 cm spacing in both directions, with $\Phi 6.5$ mm wire mesh and 20–23 cm thick C20 shotcrete (including an initial

5 cm thick polypropylene fiber concrete layer). The permanent support employs reinforced concrete full-section lining, with lining thicknesses from top to bottom as follows: 0.6 m for the crown and sidewalls above the crane beam corbel; 0.8 m for sidewalls between the generator floor and crane beam corbel; and 1.0 m for sidewalls below the generator floor and the invert.

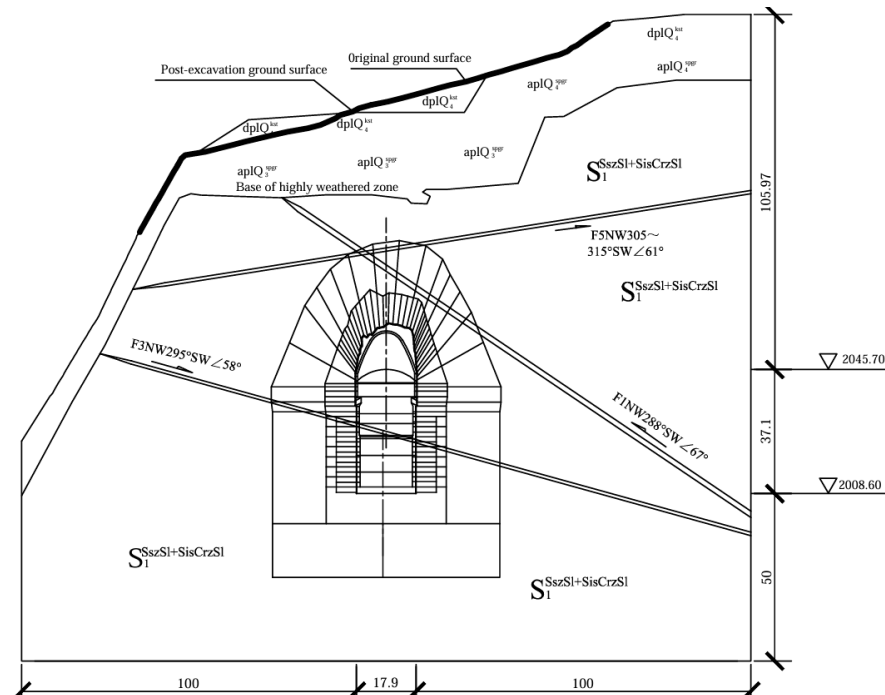
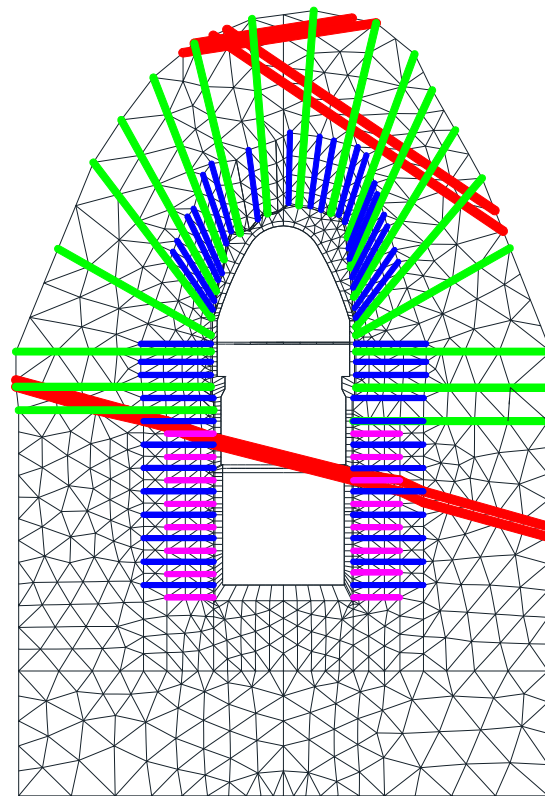


Figure 4. Calculate the generalization model of K0+004 section.

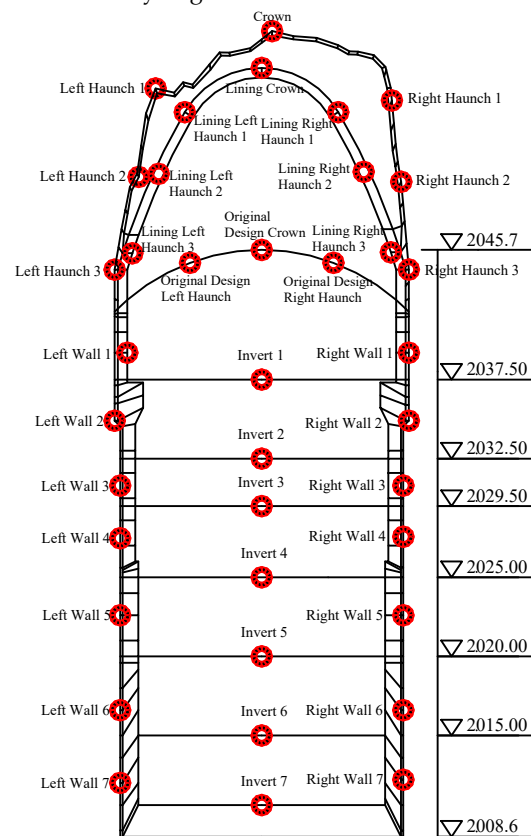
4.2. Numerical Model Establishment

The disasters of typical sections were simulated by using the software FINAL. The software has a unique function in geotechnical engineering. It has 32 types of units, which can simulate a variety of engineering problems, such as the dynamic and static problems of underground engineering, dam construction, slope engineering and foundation engineering. It has been used in many practical projects, such as the underground powerhouse of the Xiangjiaba Hydropower Station on the Jinsha River in China, the Heihe earth-rock dam in China, and the left bank slope of Delsi Hydropower Station in the Republic of Ecuador [28].

The detailed model of the finite element model and detailed details are shown in Figure 5a. In this series of analyses, the finite element model includes wall rock units (LST units), excavation units (LST units), shotcrete unit units (BEAM6 units), and crack units (COJO units) [29]. The rock masses (wall rock and the excavation section) adopt an elastoplastic constitutive model and the Mohr-Columb strength criterion, while a linear elastic constitutive model was used for the shotcrete layer. The grouting effect was simulated by activating the prearranged curved beam element to simulate the shotcrete layer, and the grouted rock bolt was simulated by the bar element. The displacement boundary conditions permitted were that normal constraint at the left and right boundaries, full constraints down the bottom boundary, and freedom to displace along the top boundary.



(a) Numerical analysis generalization model of K0+004 section



(b) Key point schematic diagram of K0+004 section

Figure 5. Numerical analysis model and schematic diagram of key points.

4.3. Subsequent Excavation Sequence

The installation chamber and main powerhouse were excavated by bench according to elevation. The excavation sequence for each bench is presented in Table 3.

Table 3. Later stage layered excavation sequence.

Excavation Bench	Excavation Elevation Range	Bench Height (m)
Bench 1	Above EL. 2037.50 m	/
Bench 2	Above EL. 2032.50 m	5.0
Bench 3	Above EL. 2029.50 m	3.0
Bench 4	Above EL. 2025.00 m	4.5
Bench 5	Above EL. 2020.00 m	5.0
Bench 6	Above EL. 2015.00 m	5.0
Bench 7	Above EL. 2008.60 m	6.4

4.4. Analysis of Surrounding Rock Deformation Characteristics

Following the aforementioned bench excavation sequence, numerical simulation of each bench excavation and support installation process was conducted. Figure 6 presents the displacement fields, stress fields, and plastic zone distributions of the surrounding rock upon completion of each bench excavation.

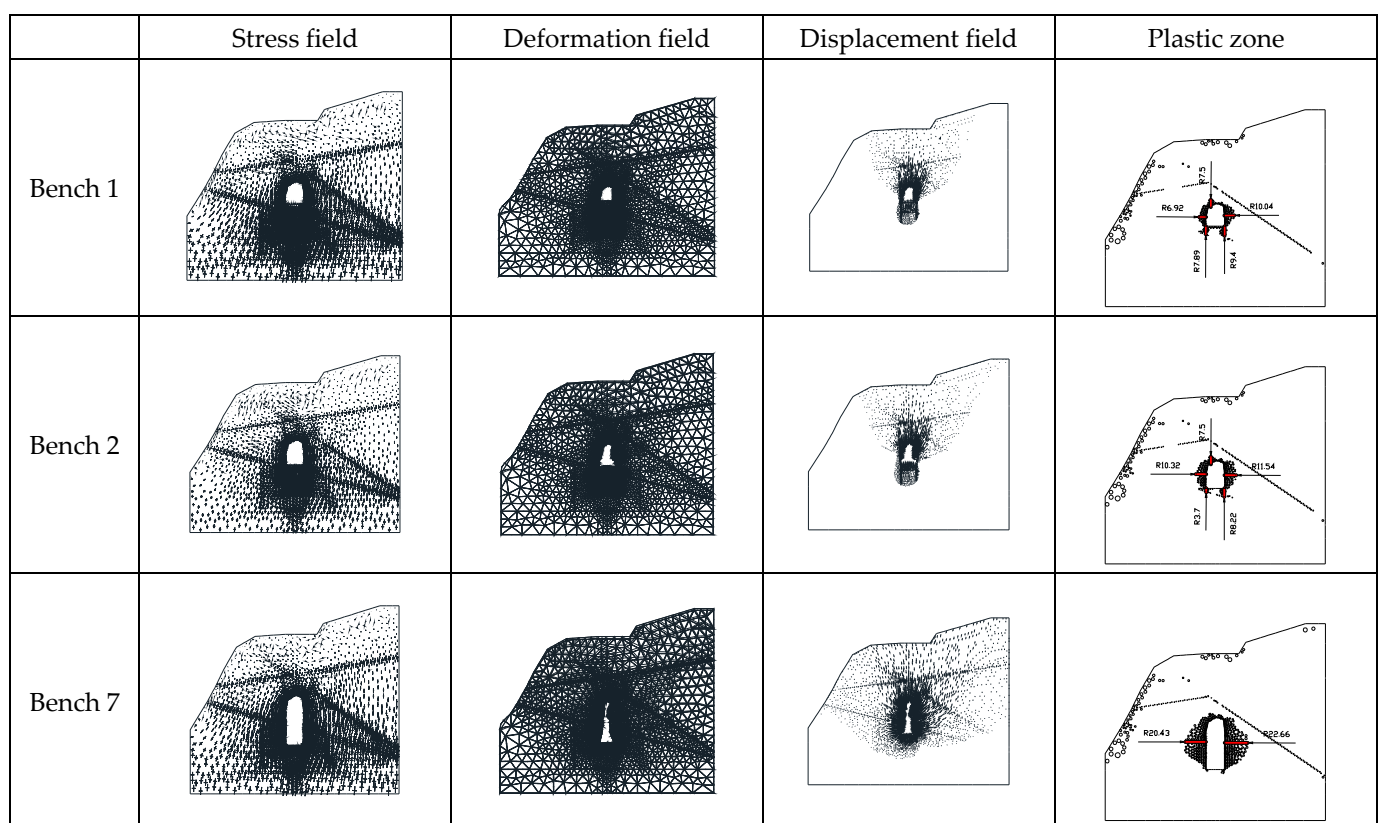


Figure 6. Displacement, stress, and plastic zone of surrounding rock after each excavation stage.

Based on Figure 6, the following observations are made:

- (1) Bench 1 excavation stage (including collapse): Due to the unstable wedge body formed by fault F3 and fracture cutting, surrounding rock collapse occurred. Following collapse, the surrounding rock deformed toward the free surface, with tensile stress zones of $L = 2.77$ m and $L = 1.45$ m appearing at the left and right crown, respectively. The maximum tensile stress was 0.08 MPa, located at the arch shoulder. Plastic zones

with maximum plastic-zone depth of $r = 7.5$ m, $r = 6.92$ m, $r = 10.04$ m, $r = 7.89$ m, and $r = 9.4$ m appeared at the left arch shoulder, left sidewall, right sidewall, left arch springing, and right arch springing, respectively. The simulated collapse location, extent, and failure mode are generally consistent with actual field conditions, demonstrating that the established numerical model and selected mechanical parameters reasonably reflect engineering practice and validating the model correctness and rationality.

- (2) Bench 2 through Bench 7 excavation stages: Following collapse reinforcement treatment, continued bench excavation downward proceeded with gradual development of surrounding rock deformation. However, the plastic zone extent did not significantly expand, displacement field distribution patterns remained generally consistent, and stress field distribution tended toward uniformity. Post-excavation deformation of the surrounding rock was primarily concentrated at the crown, arch shoulders, and sidewalls, with deformation directions generally pointing toward the cavern interior, consistent with the general pattern of excavation unloading in underground caverns, indicating improved overall stability of the surrounding rock.
- (3) Deformation evolutionary patterns: Comparison of results from each bench excavation indicates that with increasing excavation depth, surrounding rock displacement magnitudes gradually increase, but the growth rate progressively decreases, demonstrating that timely support measures effectively control the development of surrounding rock deformation.

4.5. Quantitative Stability Analysis of Surrounding Rock

To further evaluate the stability status of the surrounding rock, displacement and stress data at key points following each bench excavation were extracted for quantitative analysis.

4.5.1. Key Point Response During Collapse Stage

Table 4 presents the deformation and stress values at key points around the cavern following Bench 1 excavation (including collapse).

Table 4. Key point deformation and stress of the surrounding rock.

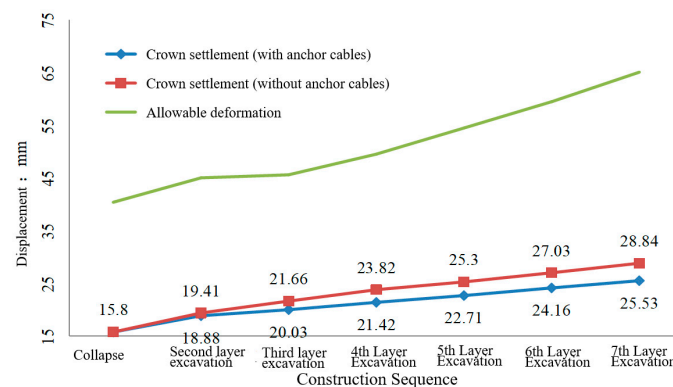
Location		Deformation/mm		Stress/MPa	
		x-Direction	y-Direction	σ_1	σ_2
Crown		−1.64	−15.80	0.04	−1.04
Arch shoulder 1	Left	10.30	−9.16	−0.01	−1.54
	Right	−14.22	−10.22	−0.04	−1.55
Arch shoulder 2	Left	11.86	−9.43	0.03	−1.37
	Right	−13.59	−10.62	−0.31	−1.84
Arch shoulder 3	Left	11.47	−4.23	−0.14	−2.22
	Right	−14.97	−7.60	−0.40	−2.72
Sidewall 1	Left	9.09	0.86	−0.14	−2.27
	Right	−10.87	−2.05	−0.15	−2.34
Invert		1.79	12.72	−0.36	−0.47

As shown in Table 4, following collapse, all key point deformations around the cavern are convergent deformations. Crown settlement is approximately 15.80 mm, invert heave is 12.72 mm, relative convergence at arch shoulder 1 is 24.52 mm, at arch shoulder 2 is 25.45 mm, at arch shoulder 3 is 26.44 mm, and at sidewall 1 is 19.96 mm. All deformations are below the critical threshold (72 mm). Furthermore, all key points around the cavern are

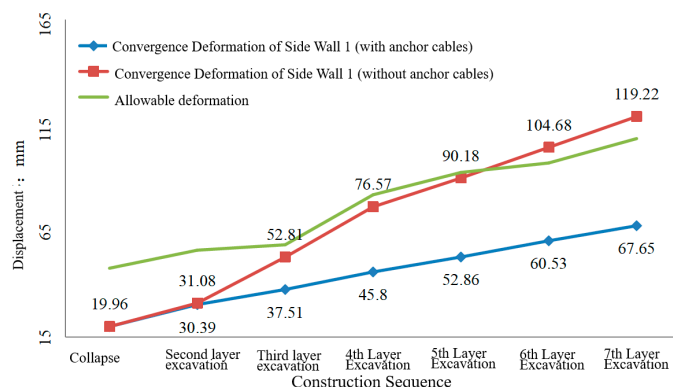
primarily under compressive stress, with a maximum value of approximately 2.72 MPa at arch shoulder 3.

As shown in Figure 6, following collapse stress release, the surrounding rock deformed toward the free surface. Tensile stress zones of $L = 2.77$ m and $L = 1.45$ m appeared at the left and right crown, respectively, with a maximum tensile stress of 0.08 MPa at the arch shoulder. Plastic zones with radii of $r = 7.5$ m, $r = 6.92$ m, $r = 10.04$ m, $r = 7.89$ m, and $r = 9.4$ m appeared at the left arch shoulder, left sidewall, right sidewall, left arch springing, and right arch springing, respectively.

Following collapse, shotcrete and rock bolt support was implemented. The deformation variation curves for the crown and sidewall 1 under conditions with and without prestressed cable reinforcement after each bench excavation are presented in Figure 7.



(a) Deformation variation curve of the crown at K0+004 section



(b) Deformation variation curve of the sidewall 1 at K0+004 section

Figure 7. Deformation curve of key points.

As shown in Figure 7, deformations at all key points with cable reinforcement are smaller than those without cable reinforcement and generally remain below the critical deformation threshold. Without cable reinforcement, deformations during subsequent excavation exceed the critical deformation threshold, and shotcrete and rock bolt stresses exceed design values, indicating potential rock mass failure. This demonstrates that prestressed cable reinforcement is necessary and relatively effective.

4.5.2. Surrounding Rock Stability upon Completion of Excavation

Upon completion of Bench 7 support construction, the stresses on the shotcrete layer, rock bolts, and cables at key points are summarized in Table 5.

Table 5. Stress on the supporting structure of key points.

Support Type		Shotcrete		Rock Bolt	Cable
Location	$\sigma_{\max}/\text{MPa}$	$\sigma_{\min}/\text{MPa}$	Axial Force/kN	Axial Force/kN	Location
Crown	0.04	−3.14	92.20	252.22	Crown
Arch shoulder 1	Left	0.21	−4.57	121.45	284.89
	Right	0.01	−0.91	81.72	278.67
Arch shoulder 2	Left	−0.06	−2.79	75.26	276.65
	Right	−0.14	−1.79	73.55	278.81
Arch shoulder 3	Left	−0.15	−2.87	84.35	284.89
	Right	0.75	−0.16	77.19	249.97
Sidewall 1	Left	0.69	0.01	126.42	305.90
	Right	0.83	0.03	148.42	319.48
Sidewall 2	Left	1.04	0.15	122.43	301.48
	Right	1.08	0.29	120.10	310.27
Sidewall 3	Left	1.03	0.03	181.55	299.16
	Right	0.26	−0.64	167.45	311.90
Sidewall 4	Left	0.97	0.01	121.19	-
	Right	0.44	−0.02	211.62	-
Sidewall 5	Left	0.69	0.00	111.09	-
	Right	0.94	0.00	180.31	-
Sidewall 6	Left	0.68	0.00	117.40	-
	Right	0.67	0.01	188.38	-
Sidewall 7	Left	0.01	−2.06	34.85	-
	Right	0.01	−1.94	67.42	-

As shown in Table 5, the maximum compressive stress in the shotcrete occurs at arch shoulder 1, approximately 4.57 MPa, which does not exceed the design compressive strength of C20 concrete (10.0 MPa). The maximum tensile stress occurs at sidewall 2, approximately 1.08 MPa, which does not exceed the design tensile strength of C20 concrete (1.1 MPa). The maximum axial force in rock bolts occurs at sidewall 4, 211.62 kN, which does not exceed the design value (246 kN). The maximum axial force in cables occurs at sidewall 1, 319.48 kN, which does not exceed the design value (520 kN).

4.6. Optimized Reinforcement Design Scheme

Based on the above computational analysis, the powerhouse experienced collapse during initial excavation and has achieved basic stability following reinforcement treatment; continued bench excavation downward with timely support implementation has maintained surrounding rock deformation within allowable limits, and rock bolt and cable forces remain within allowable ranges. However, local shotcrete stresses are excessive, potentially leading to failure. Therefore, this section proposes reinforcement optimization of the initial support scheme based on the original design.

The specific schemes are as follows:

Scheme I: Addition of one prestressed cable of the same type at each left and right arch shoulder;

Scheme II: Addition of one prestressed cable of the same type at each left and right arch shoulder, plus two 9 m rock bolts on each side of sidewall 2;

Scheme III: Addition of two 9 m rock bolts on each side of sidewall 2.

The optimized reinforcement locations for each scheme are illustrated in Figure 8.

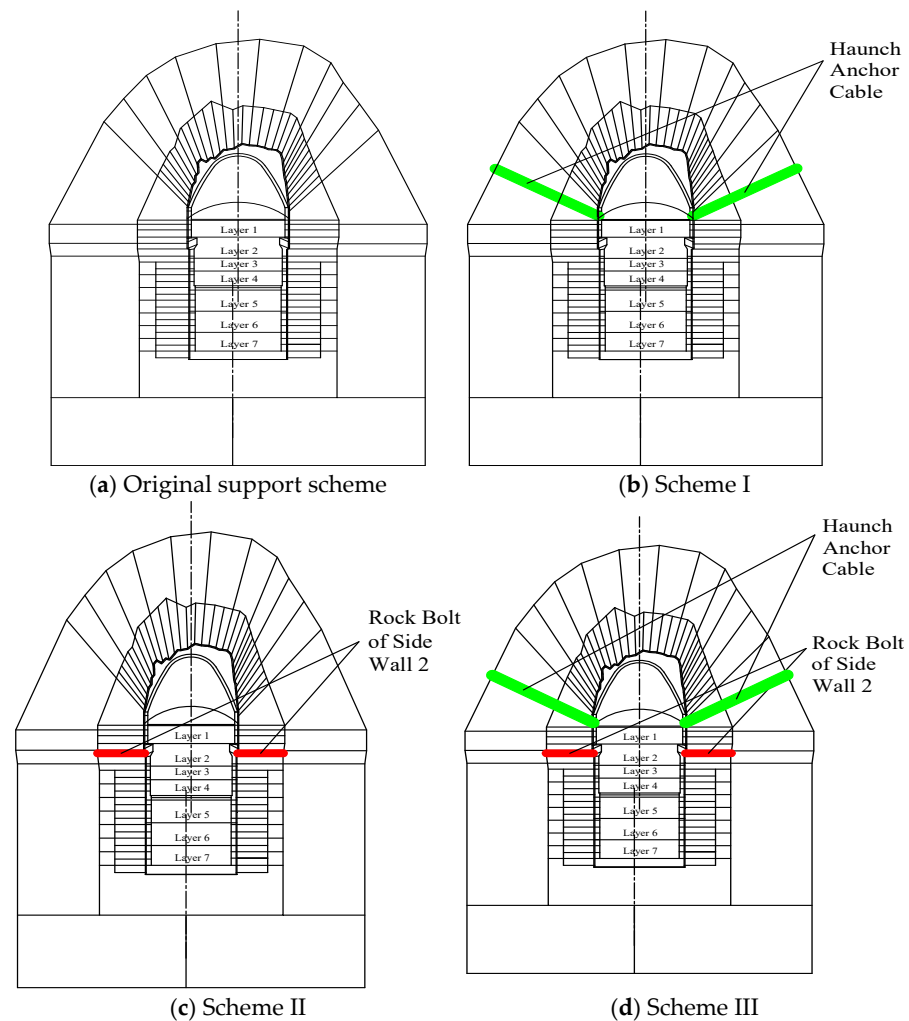


Figure 8. Reinforcement schematic diagram of the programs.

The shotcrete stresses after Bench 7 support for the three schemes are shown in Table 6.

As shown in Table 6, under the original support scheme, the maximum tensile stress in the shotcrete at sidewall 2 is 1.08 MPa. With the arch shoulder cable addition scheme, the maximum tensile stress at sidewall 2 is 1.11 MPa, representing an increase relative to the original design. With the arch shoulder cables plus sidewall 2 rock bolts scheme, the maximum tensile stress at sidewall 2 is 1.11 MPa, also representing an increase relative to the original design. With the sidewall 2 rock bolts addition scheme, the maximum tensile stress at sidewall 2 remains 1.08 MPa, unchanged from the original design.

In summary, although the scheme with arch shoulder cables plus sidewall 2 rock bolts shows no change in shotcrete tensile stress following Bench 7 support, the tensile stress during Bench 3 through Bench 7 excavation is reduced to within reasonable limits, i.e., not exceeding the design tensile strength of C20 concrete (1.1 MPa). Therefore, the scheme with two rock bolts added on each side of sidewall 2 demonstrates relatively better comprehensive performance in controlling surrounding rock deformation and support loading.

Based on the above computational analysis, the following recommendations are provided: implement reinforcement with arch shoulder cables plus two rock bolts on each side of sidewall 2; promptly apply shotcrete and rock bolt support (wire mesh with polypropylene fiber shotcrete, rock bolts welded to wire mesh) following each bench excavation; promptly install random rock bolts and grout reinforcement upon exposure of faults and fractures; conduct surrounding rock deformation convergence monitoring to promptly obtain deformation conditions (particularly before and after excavation).

Table 6. The spray layer stress of the programs after the seventh support.

Reinforcement Scheme		Original Design		Arch Shoulder + Cables		Arch Shoulder + Cables, Sidewall 2 + Bolts		Sidewall 2 + Bolts	
Support Type		Shotcrete		Shotcrete		Shotcrete		Shotcrete	
Location		$\sigma_{\max}/\text{MPa}$	$\sigma_{\min}/\text{MPa}$	$\sigma_{\max}/\text{MPa}$	$\sigma_{\min}/\text{MPa}$	$\sigma_{\max}/\text{MPa}$	$\sigma_{\min}/\text{MPa}$	$\sigma_{\max}/\text{MPa}$	$\sigma_{\min}/\text{MPa}$
Arch shoulder 1	Left	0.21	−4.57	0.20	−4.45	0.19	−4.38	0.19	−4.50
	Right	0.01	−0.91	0.01	−0.74	0.01	−0.74	0.00	−0.91
Arch shoulder 2	Left	−0.06	−2.79	−0.06	−2.70	−0.06	−2.63	−0.06	−2.73
	Right	−0.14	−1.79	−0.13	−1.61	−0.13	−1.62	−0.14	−1.79
Arch shoulder 3	Left	−0.15	−2.87	−0.17	−3.17	−0.16	−3.00	−0.13	−2.70
	Right	0.75	−0.16	0.70	−0.14	0.70	−0.14	0.76	−0.16
Sidewall 1	Left	0.69	0.01	0.83	0.01	0.90	0.01	0.79	0.02
	Right	0.83	0.03	0.94	0.03	0.94	0.03	0.83	0.03
Sidewall 2	Left	1.04	0.15	1.05	0.15	1.03	0.23	1.02	0.22
	Right	1.08	0.29	1.11	0.31	1.11	0.31	1.08	0.29
Sidewall 3	Left	1.03	0.03	1.04	0.03	1.03	0.03	1.03	0.03
	Right	0.26	−0.64	0.26	−0.56	0.26	−0.56	0.26	−0.63
Sidewall 4	Left	0.97	0.01	0.98	0.01	0.99	0.01	0.98	0.01
	Right	0.44	−0.02	0.47	−0.02	0.48	−0.02	0.45	−0.02

5. Discussion

Taking the underground powerhouse of a hydropower station in Northwest China as the research object and following a systematic research framework of “parameter back-analysis—stability assessment—support optimization,” this study has obtained a series of engineering-significant conclusions through the cross-validation of numerical simulations and field monitoring data.

Although the parameter inversion results have passed the reasonableness checks, the following sources of uncertainty should be noted: (1) the equivalent continuum assumption neglects the influence of faults and fractures on localized deformation, which may result in underestimated equivalent parameters; (2) the inversion results are sensitive to the quality of monitoring data, and anomalous data may introduce bias into the parameter estimates; and (3) the in situ stress estimation involves a certain degree of empiricism, which may affect the determination of absolute parameter values. When applying these parameters to other sections, appropriate adjustments should be made based on the site-specific geological conditions of each section.

6. Conclusions

- (1) The physical and mechanical parameters of surrounding rock obtained through monitoring data back analysis reasonably reflected actual rock mass properties, with back-analysis results showing consistent trends with monitored values. The obtained parameters can be applied to subsequent stability analysis calculations.
- (2) Numerical simulation of the bench excavation process revealed the evolutionary patterns of surrounding rock displacement, stress, and plastic zones. The Bench 1 results align with actual collapse conditions, validating the model. During subsequent excavation, key monitoring point deformations increased but remained below critical thresholds, maintaining overall stability. Prestressed cable reinforcement effectively controlled surrounding rock deformation and support loading.
- (3) Building upon the original support design, the optimized reinforcement scheme involving arch shoulder cables plus two rock bolts on each side of sidewall 2 demon-

strated relatively better comprehensive performance in controlling surrounding rock deformation and support loading.

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