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Depth-Dependent Mechanical Response and Support-Parameter Sensitivity of a 428 m Ventilation Shaft: Numerical Analysis and Field Evidence

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ABSTRACT: To investigate the stability of surrounding rock in large-scale deep shafts under deep-overburden conditions, this study examines a 428 m deep ventilation shaft in Guizhou, China. A three-dimensional local-segment finite element model was established using Abaqus for selected representative depth intervals extending to 385 m, and a one-factor-at-a-time sensitivity analysis was conducted at 300 m. The analysis considered shotcrete thickness, rock-bolt length, and longitudinal rock-bolt spacing. The numerical results show that radial displacement, support stress and the surrounding-rock deformation influence zone increase nonlinearly with burial depth, with marked acceleration beyond approximately 200 m. Field records showed an overall increase in surrounding-rock pressure magnitude with depth, whereas secondary-lining stress varied non-monotonically and remained high in both the shallowest and deepest monitored sections, providing trend-level evidence rather than point-by-point calibration. Within the deliberately extended numerical sensitivity range, increasing shotcrete thickness from 10 to 40 cm reduced the calculated maximum radial displacement from 0.88 to 0.50 mm, corresponding to a reduction of approximately 43.2%. However, the marginal benefit diminished beyond 30 cm in the selected cases. Longer rock bolts reduced displacement and stress within the investigated range, whereas wider spacing weakened the overall constraint. Within the investigated parameter ranges, the one-factor-at-a-time comparison indicated that shotcrete thickness produced the strongest displacement-control effect, followed by rock-bolt length and spacing. These results indicate that primary-support stiffness plays the dominant role under the considered deep-shaft conditions. Support design should therefore prioritize primary-support stiffness, together with moderate rock-bolt lengthening and appropriate spacing control, while accounting for construction constraints.

KEYWORDS: Deep and large shaft; support structure; lateral pressure coefficient; finite element analysis; parametric sensitivity

1 Introduction

With the advancement of deep-earth resource development and energy storage strategies, large deep vertical shafts have emerged as critical conduits for coal mining, metal ore extraction and pumped-storage hydropower projects. These shafts have gained heightened strategic importance amid the depletion of shallow resources [1]. However, the rapid increase in shaft depth places the rock mass under conditions of high geostress, elevated geothermal temperatures, and high osmotic pressure, posing severe challenges

to surrounding rock stability control [2,3]. Under high geostress, excavation induces significant stress redistribution, readily triggering large-scale plastic zone expansion, sustained rheology, and even instability or collapse. Concurrently, the support structure endures loads far exceeding conventional design limits, with its stress-deformation characteristics directly governing project safety and success [4,5]. Consequently, systematic investigation of the mechanical response of deep large-scale shaft support structures in deep-overburden environments is warranted.

Shaft support technology has evolved from early timber bracing and masonry lining to modern bolt-shotcrete systems, while the associated design basis has shifted from empirical support to rock mechanics, field monitoring and numerical modelling [6,7]. Classical surrounding-rock pressure theories, including the Protodyakonov and Terzaghi approaches, provided early mechanical bases for support-load estimation [8]. Field observations have further shown that rheology, swelling, strength degradation and discontinuity-controlled deformation can govern support failure in deep soft-rock openings [9]. These studies provide the conceptual basis for evaluating the interaction between the rock mass and the support system in deep underground structures.

Numerical modelling has become an important tool for analysing stress redistribution and support response in underground engineering [10,11]. For example, Meguid and Dang [12] employed numerical simulation to examine the impact of erosion voids behind linings, revealing that such voids markedly alter lining stress patterns, inducing bending moment and shear force concentrations and substantially reducing overall bearing capacity and safety factors. Similarly, Zhang et al. [13] applied a post-peak strain-softening model to analyse deep surrounding rock rheology and its long-term loading effects on support. Liu et al. [14] developed a mesoscopic model for bolt pull-out, elucidating the full-process mechanism from interface damage evolution to debonding failure and enhancing simulation accuracy for support mechanical response. Sun et al. [15] proposed a constitutive model for bolts under combined tension-shear loading, clearly delineating the evolutionary path from elastic yielding and plastic development to fracture failure, providing essential tools for assessing ultimate bearing capacity and safety margins. Regarding support parameter optimisation, Zha et al. [16] quantified the effects of varying bolt parameters via numerical simulation, demonstrating that length extension significantly enhances anchored zone bearing capacity, while spacing reduction promotes uniform stress distribution. Nevertheless, most previous studies have focused on horizontal roadways or moderately deep shafts, while the depth-dependent response and support-parameter sensitivity of deep shaft sections remain insufficiently quantified. Moreover, under deep-overburden conditions, the sensitivity and interaction laws of key parameters in the coordinated primary support-bolt system remain unclear, leading to a lack of refined theoretical guidance for design.

Recent highway-tunnel construction in China has progressed towards substantially deeper ventilation shafts. The Tianshan Shengli Tunnel comprises four ventilation-shaft sites with paired bores, resulting in eight shafts with depths of approximately 550–707 m; recent studies have investigated excavation-method selection, pregrouting-curtain design and water-induced middle-partition instability in these shafts [17–19]. However, the depth-dependent response and support-parameter sensitivity of the surrounding rock-bolt-shotcrete system remain insufficiently quantified.

In this study, Abaqus was used to establish a three-dimensional local-segment model of shaft excavation and bolt-shotcrete support in interbedded limestone and mudstone strata. The mechanical parameters of the surrounding rock were obtained from site investigation, field sampling, borehole core observations and laboratory geotechnical tests. To represent different burial conditions without explicitly modelling the entire overlying strata, equivalent vertical overburden pressure was applied at the top of the local model, while the self-weight of the modelled segment was included through body force. A comparable deep-shaft study used equivalent self-weight stress at the model top to represent the overlying strata [20]. The model was

used to analyse selected representative depth intervals along the shaft. Available field monitoring records were then used as independent engineering evidence to examine whether the simulated depth-dependent trends were consistent with the *in-situ* response. Finally, a representative deep case at 300 m was selected for one-factor-at-a-time sensitivity analysis of shotcrete thickness, rock bolt length and rock bolt spacing.

2 Engineering Background and Numerical Methodology

2.1 Project Setting, Geological Conditions and Monitoring

The tunnel shaft project is located in Daozhen Gelao and Miao Autonomous County, Guizhou Province, China. The proposed ventilation shaft system primarily comprises one vertical shaft, one air supply duct, and two exhaust ducts. The shaft has an excavation depth of 428 m and an excavation radius of 3.2 m, with the site ground elevation at 1294.33 m (1985 National Elevation Datum). Fig. 1 illustrates a three-dimensional schematic of the tunnel vertical shaft.

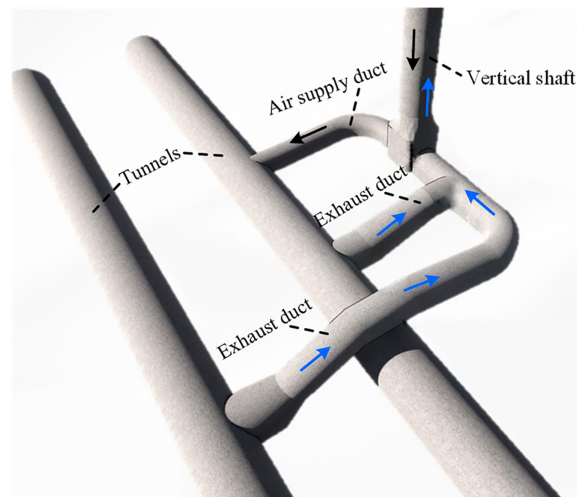


Figure 1: Three-dimensional schematic diagram of the tunnel vertical shaft.

The shaft was constructed using a combined raise-boring and top-down drill-and-blast method. A 230 mm-diameter pilot hole was first drilled downwards and enlarged to 270 mm, after which reverse reaming was performed to form a 1.6 m-diameter mucking raise. The main shaft was subsequently enlarged from the ground surface downwards by full-section drilling and blasting, with the initial support installed after each excavation cycle. Excavated rock was discharged through the mucking raise, and the secondary lining was finally constructed from the bottom upwards using sliding formwork. According to the project construction scheme, the main-shaft enlargement was performed using YT-28 pneumatic drills. The maximum blasthole depth was 2.5 m, and the designed excavation advance was 2.2 m per cycle. Each excavation and mucking cycle was followed by installation of the initial support, establishing a top-down sequence in which one support cycle followed each excavation cycle.

The shaft intersects seven project-specific strata composed mainly of clay, interbedded limestone and mudstone, and limestone. Their basic quality indices (*BQ*) were evaluated in accordance with the Standard for Engineering Classification of Rock Mass (GB/T 50218-2014) [21], which defines five rock-mass quality classes (I–V). Table 1 lists the seven strata and their measured indices; the stratum numbers do not represent seven standard classification levels.

Table 1: Physical and mechanical properties of rock masses.

Strata No.	Thickness of Stratum (m)	Saturation Uniaxial Compressive Strength R_c (MPa)	Integrity Index K_v	Basic Quality Indicators BQ
1	9.5	35	0.30	270
2	50	35	0.60	345
3	55	35	0.74	380
4	85	35	0.60	345
5	165	35	0.73	378
6	20	35	0.53	328
7	40	35	0.75	383

Note: The rock mass classification and physico-mechanical indices were determined from the geological investigation report, field sampling, borehole core observations and laboratory tests.

The site groundwater is predominantly carbonate karst water, primarily distributed within moderately weathered limestone. It is recharged by fissure-cavity water, underground rivers, and atmospheric precipitation, exhibiting strong water-rich characteristics, with a burial depth of 386.5 m. According to the geological investigation report, karst-related adverse geological conditions are present beneath the shaft construction area, rendering the excavation highly susceptible to hazards such as water inrush and mud bursting. Consequently, enhanced monitoring and advanced geological forecasting are essential throughout the construction process. During shaft construction, a field monitoring system was arranged to record the mechanical response of the support system and surrounding rock. The monitoring items included surrounding rock pressure acting on the primary support, stress in the steel arch, shotcrete-related stress or surface strain response, and concrete stress in the secondary lining at representative shaft sections. These monitoring records provide direct engineering information on the load transfer and stress adjustment process during excavation and support installation, and they were used in this study as field evidence for evaluating the reasonableness of the numerical results.

2.2 Basic Assumptions and Models

To investigate the stress and deformation response of the shaft support structure at different burial depths, a three-dimensional local-segment model was established in Abaqus. Owing to the geometric and loading symmetry of the regular shaft segment, a half-shaft model was adopted. The symmetry assumption is a modelling choice for the isolated shaft segment and does not represent the shaft-duct junctions. The model domain was 40 m $\times$ 20 m $\times$ 20 m (length $\times$ width $\times$ height). Each numerical case represents an isolated local shaft segment under the corresponding burial stress condition, rather than a simulation of the entire 428 m shaft or the shaft-duct intersections. Fig. 1 shows the project-wide shaft-and-duct arrangement, whereas the finite-element model isolates a straight shaft segment; the air-supply duct, exhaust ducts and shaft-duct junctions are outside the present modelling scope. Burial depth was represented by an equivalent vertical overburden pressure applied at the model top, and the self-weight of the local model was applied once as body force.

The 20 m dimension represents the height of the local computational domain and does not represent a single excavation or support advance. After equilibrium of the initial stress field, the shaft-core elements were deactivated progressively from top to bottom according to the project-specific excavation advance of 2.2 m. The corresponding shotcrete and rock-bolt elements were activated after each excavation cycle. The response quantities were evaluated in the central part of the model to minimise the influence of the upper and

lower boundaries. This staged procedure reproduces the project sequence by activating the corresponding support after each excavation cycle, while retaining a sufficiently long local domain for three-dimensional stress redistribution.

Normal displacement was constrained on the lateral boundaries, the base was fixed in all translational directions, and the top boundary was free except for the applied vertical pressure. Rock bolts were represented by axially deformable truss elements. The representative bolt had a diameter of 20 mm ($A = 314 \text{ mm}^2$), a length of 3.0 m and a spacing of $1.0 \text{ m} \times 1.0 \text{ m}$; the sensitivity ranges were 2.5–4.0 m and 0.8–1.5 m. The bolts were fully bonded over their embedded length, whereas separate grout behaviour, bond-slip, debonding, pretension and yielding were not explicitly modelled. The initial shotcrete support was represented by shell elements. Because truss elements carry axial force only, bolt bending stiffness and local transverse deformation were not represented. The rock-shotcrete interface was idealised as fully bonded, with no separation or relative tangential slip; therefore, no Coulomb friction coefficient or separate friction-force model was assigned. The model contained 34,885 elements and 66,950 nodes. An additional numerical check showed that refining the near-wall mesh from 0.9 to 0.4 m changed the central radial convergence by approximately 1.3%, whereas extending the lateral domain from $40 \text{ m} \times 20 \text{ m}$ to $100 \text{ m} \times 50 \text{ m}$ changed it by approximately 14%. The results are therefore interpreted as conditional local trends rather than mesh- and boundary-independent absolute predictions. This study focuses on the depth-dependent mechanical response of the shaft support system under the water-controlled, drained condition specified in the project construction scheme. Because site-specific pore-pressure, permeability and water-induced strength-degradation data were unavailable, seepage and fluid-solid coupling were not considered. The three-dimensional numerical model is illustrated in Fig. 2.

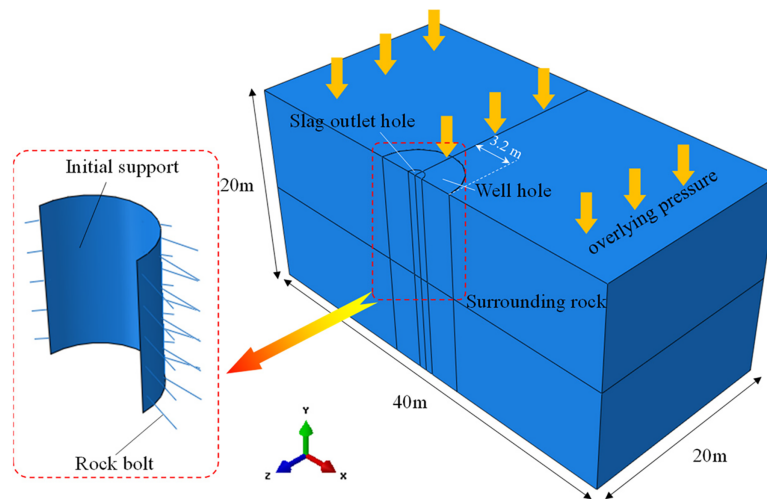


Figure 2: Three-dimensional numerical model.

2.3 Material Modelling and Parameters

Table 1 reports seven project-specific strata, rather than seven standard rock-mass quality levels. They are characterized by stratum thickness, saturated uniaxial compressive strength (UCS), rock-mass integrity index (K_v) and basic quality index (BQ). For numerical modelling, the strata were grouped into two dominant lithological units: the upper 0–165 m interbedded limestone-mudstone unit and the underlying limestone unit, represented by the two material sets in Table 2. The parameters in Table 2 are representative group values derived from the project investigation and laboratory tests rather than row-by-row statistical conversions.

The surrounding rock was modelled using the elastoplastic Mohr-Coulomb criterion [7,11]. A calibrated Hoek-Brown model based on the Geological Strength Index (GSI) was not established because site-specific GSI, the intact-rock constant m_i and intact tensile-strength data were unavailable [6]. Explicit jointed-rock and time-dependent models were outside the present scope because the required site-specific joint-geometry and time-dependent parameters were unavailable [13].

Table 2: Physical and mechanical properties of materials.

Name	Unit Weight γ (kN/m ³)	Cohesion c (MPa)	Friction Angle φ (°)	Elastic Modulus E (GPa)	Poisson's Ratio ν
Interbedded limestone and mudstone	26.37	5	38	3.2	0.29
Limestone	26.46	6	47	3.3	0.28
Initial support	25	–	–	35	0.22
Rock bolt	78	–	–	210	0.20

Note: The parameters of the surrounding rock were obtained from project-specific field sampling, borehole core testing and laboratory geotechnical tests. The parameters of the initial support and rock bolts were determined from the design documents and commonly used engineering material properties for concrete and steel support components.

The stratum distribution and basic mechanical parameters were not assigned arbitrarily. They were determined on the basis of the geological investigation, field sampling, borehole core tests and laboratory geotechnical tests conducted for the project. The key physico-mechanical indices of the rock masses are listed in Table 1, and the material parameters used in the finite element model are summarized in Table 2.

The finite-element model was not entirely linear. The surrounding rock was represented by an elastoplastic Mohr-Coulomb model, whereas the shotcrete and rock bolts were idealised as linear-elastic materials. The Mohr-Coulomb model can reproduce frictional yielding according to the adopted cohesion and friction angle, but does not explicitly represent post-peak strain softening, discontinuity-controlled deformation or time-dependent behaviour. Similarly, the linear-elastic support models do not reproduce shotcrete cracking and tensile softening or rock-bolt yielding. Consequently, the support-parameter analyses are used to compare relative service-stage response trends under consistent assumptions rather than to simulate complete nonlinear failure or provide ultimate-capacity verification.

2.4 Initial Stress and Calculation Procedure

Prior to simulating the excavation process, an initial geostatic stress field was established by applying the self-weight of the model and the equivalent vertical overburden pressure at the model top. No dedicated *in-situ* stress measurements, such as hydraulic-fracturing or overcoring tests, were available in the project investigation documents. Consequently, no site-measured tectonic stress tensor was imposed in the numerical model. The vertical initial stress was represented by the gravitational overburden baseline described below, whereas the absence of measured horizontal tectonic stress is explicitly treated as a limitation of the present analysis.

According to the Technical Specification GB/T 50218-2014 [21], when measured initial rock-stress data are unavailable, the initial stress field may be preliminarily assessed from terrain and geological investigation data. The standard indicates that the vertical initial stress may be approximated by the self-weight of the overlying strata when tectonic and topographic effects are not significant. It also notes that tectonic history, valley topography, surface erosion and local geological conditions may modify the initial stress field. Therefore, the following calculation is used only to establish a gravitational vertical-stress baseline and to

check the consistency of the vertical loading applied to the local numerical model. It is not regarded as a reconstruction of the complete three-dimensional *in-situ* stress field.

Based on vertical static equilibrium, the gravitational vertical stress of the two representative strata adopted in the numerical model was calculated as:

$$\sigma_v^{(g)}(z) = \begin{cases} \gamma_1 z & 0 \leq z \leq z_1 \\ \gamma_1 z_1 + \gamma_2 (z - z_1) & z_1 \leq z \leq z_2 \end{cases} \quad (1)$$

where $\sigma_v^{(g)}(z)$ (kPa) is the gravitational vertical total-stress magnitude at depth z ; z (m) is the vertical depth measured downward from the ground surface; γ_1 and γ_2 (kN/m³) are the bulk unit weight of the upper and lower representative strata, respectively; z_1 (m) is the interface depth between the two strata; and z_2 (m) is the total shaft depth considered in the analysis. The thin residual-clay cover at the ground surface was homogenised into the upper material group for gravitational boundary-load initialization, consistent with the two-material idealisation adopted in the numerical model.

Each numerical case represents an isolated local shaft segment rather than the entire overlying rock mass. The equivalent downward pressure applied at the top of a local model was therefore determined as:

$$q_{\text{top}}(D) = \sigma_v^{(g)}(z_t), z_t = \max\left(0, D - \frac{H_m}{2}\right) \quad (2)$$

where $q_{\text{top}}(D)$ (kPa) is the equivalent pressure applied at the model top; D (m) is the depth of the model centre; H_m (m) is the model height; and z_t (m) is the actual depth of the model top.

The equivalent top pressure represents only the self-weight of the strata above the local numerical domain. The self-weight of the 20 m-high domain is excluded from q_{top} and applied once through the gravity body force. Consequently, the vertical stress increases from $\sigma_v^{(g)}(D - H_m/2)$ at the model top to $\sigma_v^{(g)}(D + H_m/2)$ at the model bottom, thereby avoiding double counting. Using the unit weights listed in Table 2, the geological interface depth obtained from the project investigation and $H_m = 20$ m, the model centred at $D = 300$ m has a top depth of 290 m. The calculated vertical stresses at the model top, centre and bottom are 7.66, 7.92 and 8.19 MPa, respectively, confirming the consistency of the applied pressure and gravity loading.

These results confirm the consistency between the theoretical overburden stress and the vertical-load initialization adopted in the numerical model. All depth cases were referenced to the same ground-surface datum and the same continuous overburden-stress function. Because the local models were independent and not mechanically connected, consistency between depth intervals was maintained through this common depth-dependent initial-stress definition rather than through direct stress transfer between adjacent segments.

Abaqus uses a tension-positive sign convention. The line plots report signed maximum in-plane principal stress, with compression shown as negative. For visual comparison, the contour plots show the corresponding compressive-stress magnitude as a positive value. Thus, the contour and curve values describe the same stress component but use different sign presentations. The shell contours are displayed using multiple section points and the 75% averaging threshold shown in the Abaqus output legend. The preceding calculation checks only the gravitational vertical-stress component and does not determine the site-specific horizontal tectonic stress. To assess the uncertainty associated with the unmeasured horizontal stress, an additional sensitivity analysis was conducted for the representative 300 m case by defining $\sigma_h = K_0 \sigma_v$, where K_0 is the lateral pressure coefficient and σ_h and σ_v are the horizontal and vertical stresses, respectively. Three scenarios, $K_0 = 0.5, 1.0$ and 1.5 , were considered to represent lower, equal and higher horizontal stress relative

to the vertical stress. The corresponding depth-dependent horizontal tractions were applied to the lateral boundaries. These K_0 values are sensitivity scenarios rather than measured site parameters.

3 Results and Analysis

3.1 Model Reasonableness and Stress-Sensitivity Assessment

Before analysing the numerical stress and deformation results, the model was assessed using three complementary approaches. First, the analytical calculation presented in Section 2.4 was used to check the consistency of the gravitational vertical-stress initialization. Second, a K_0 sensitivity analysis was conducted to evaluate the influence of the unmeasured horizontal stress. Third, the simulated depth-dependent response was compared with the available field-monitoring records. These assessments examine the load initialization, horizontal-stress uncertainty and overall response trend, respectively, rather than providing point-by-point calibration of every numerical output. The K_0 sensitivity analysis demonstrates that the predicted radial convergence is strongly controlled by the horizontal stress state. At 300 m, the calculated central radial convergence increased from 3.393 mm at $K_0 = 0.5$ to 6.550 mm at $K_0 = 1.0$ and 9.708 mm at $K_0 = 1.5$. Because the site-specific K_0 was not measured, these results define conditional stress scenarios rather than the actual *in-situ* stress state.

Field monitoring was used to assess whether the model captured the principal temporal and depth-dependent features observed during construction. Fig. 3 shows the five-point layouts for the initial support and secondary lining, together with representative pressure and strain gauges before and after installation. Records were available for five representative intervals: 0–10, 10–60, 60–115, 115–200 and 365–385 m. The pressure gauges measured the load transferred from the surrounding rock to the initial support, whereas the strain gauges characterised the responses of the initial support and secondary lining. These measurements and the numerical outputs represent different structural components and construction stages. They were therefore compared in terms of temporal stabilisation, depth-dependent trends and stress order of magnitude, rather than used for inverse calibration.

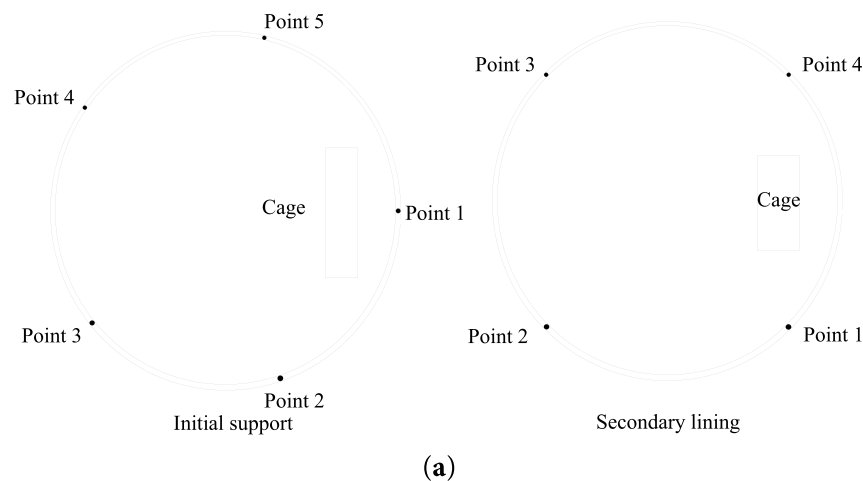


Figure 3: (Continued)

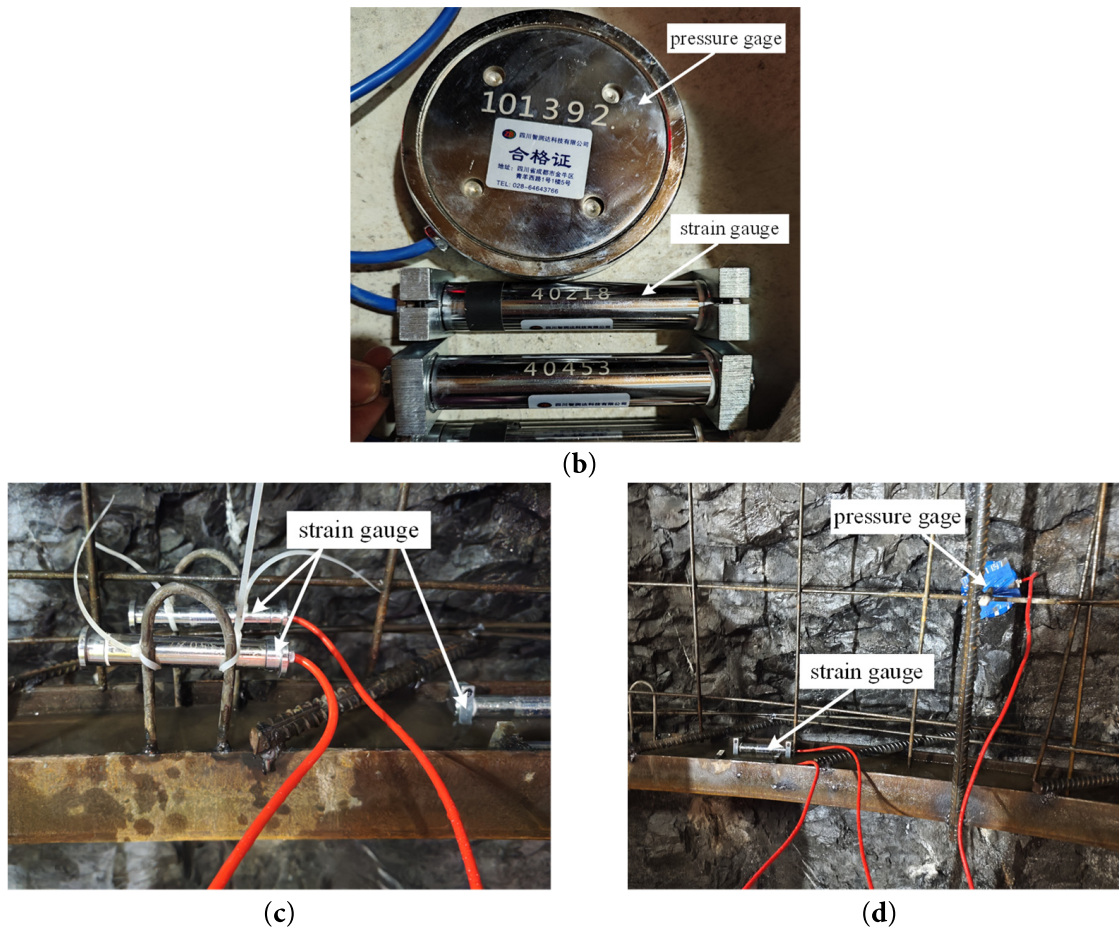


Figure 3: Field monitoring arrangement and representative instrumentation: (a) schematic of the field monitoring layout; (b) representative monitoring instruments before installation; (c) secondary-lining stress/strain sensor installed before concrete placement; (d) surrounding-rock pressure sensor installed behind the primary support.

The surrounding-rock pressure increased rapidly after support installation and approached a stable level after approximately 40–50 days (Fig. 4a). The stable pressure magnitude increased from 55.84 kPa at 5 m to 1153.81 kPa at 375 m (Fig. 4c and Table 3). This depth-dependent increase is consistent with the numerical increase in support stress and deformation under greater overburden. Secondary-lining stress changed markedly during the first week and generally stabilised after approximately 20–25 days (Fig. 4b). Its stable value decreased from 6.92 MPa at 5 m to 2.51 MPa at 87.5 m, before increasing to 6.25 MPa at 375 m. By comparison, the interpolated numerical initial-support stress increased from 0.34 to 9.56 MPa over the same representative depths (Fig. 4d). The two stress quantities are not directly equivalent, but both indicate increased support demand in the deepest monitored interval. Together with the analytical stress check and K_0 sensitivity results, the monitoring evidence supports using the model for comparative depth-dependent assessment. It does not constitute point-by-point calibration of displacement or local stress.

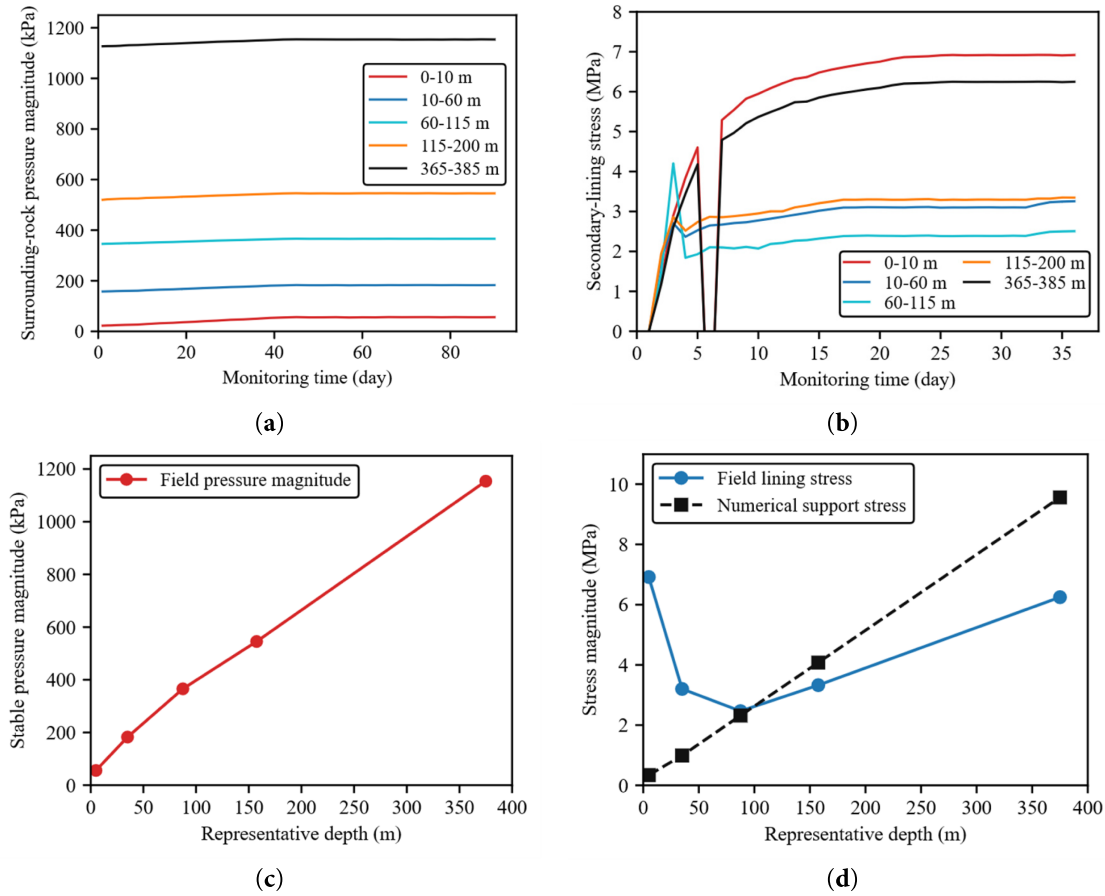


Figure 4: Field monitoring responses and trend-level comparison with the numerical results: (a) time histories of surrounding-rock pressure magnitude acting on the initial support; (b) time histories of secondary-lining stress; (c) stable surrounding-rock pressure magnitude at representative depths; (d) comparison between stable field secondary-lining stress and interpolated numerical initial-support stress magnitude.

Table 3: Processed field monitoring records used for trend validation of the numerical model.

Monitored Depth Interval (m)	Representative Depth (m)	Main Variable	Final Mean Pressure (kPa)	Pressure Magnitude (kPa)	Last-5-Day Mean (kPa)	Secondary Lining (MPa)
0–10 m	5	Surrounding-rock pressure acting on the primary support	–56.0	55.97	–55.84	6.918
10–60 m	35		–182.8	182.76	–182.63	3.256
60–115 m	87.5		–365.8	365.78	–365.72	2.506
115–200 m	157.5		–545.5	545.51	–545.40	3.347
365–385 m	375		–1153.9	1153.89	–1153.81	6.248

3.2 Variation of Radial Displacement of the Support Structure with Depth

Based on the numerical simulation results, this section examines the evolution of radial displacement in the support structure with increasing shaft depth. Fig. 5 presents total displacement-magnitude contours of the initial support and rock bolts at 11 representative depth intervals. At shallow depths of 0–20 m, deformation remains limited and relatively uniform, with a maximum value of 1.019×10^{-4} m. At intermediate

depths of 50–125 m, the maximum displacement increases from 3.399×10^{-4} to 7.031×10^{-4} m, and the deformation zone extends farther into the bolt-reinforced rock. At depths of 190–385 m, shaft-wall convergence and displacement gradients become more pronounced, with the maximum displacement exceeding 2.219×10^{-3} m. These results indicate a progressive transition from small and relatively uniform deformation at shallow depths to greater and more spatially heterogeneous deformation in the deeper sections.

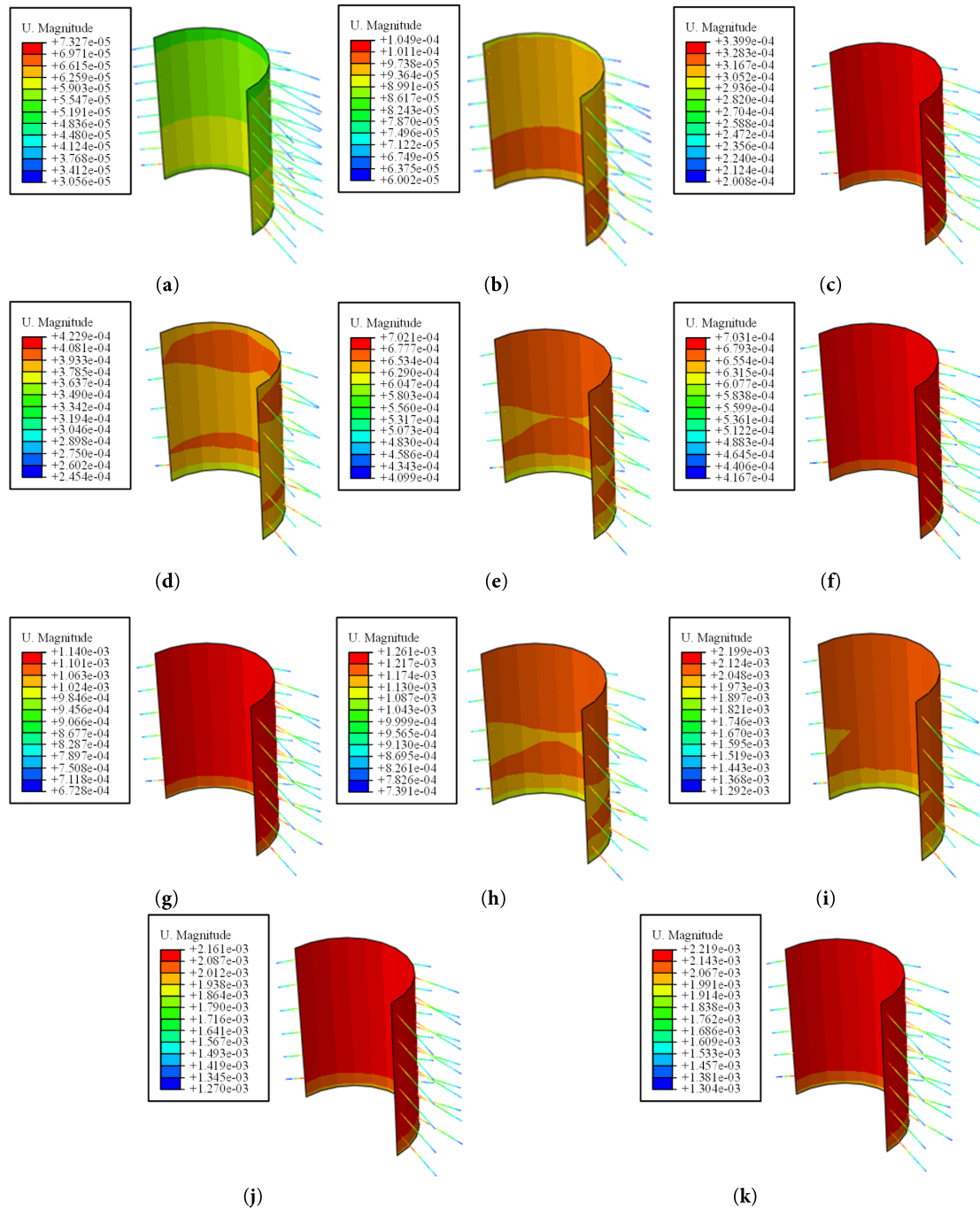


Figure 5: Total displacement magnitude contours of the support structure at different depths (mm): (a) depth = 0–10 m; (b) depth = 10–20 m; (c) depth = 50–60 m; (d) depth = 60–70 m; (e) depth = 105–115 m; (f) depth = 115–125 m; (g) depth = 190–200 m; (h) depth = 200–210 m; (i) depth = 355–365 m; (j) depth = 365–375 m; (k) depth = 375–385 m.

Fig. 6 illustrates radial displacement profiles of the support structure along the depth direction. These profiles confirm a pronounced non-linear increase in radial displacement with depth. For the shallow intervals (Fig. 6a,b, depths 0–20 m), the curves are gentle and nearly linear, with peak displacements around 0.042 mm, reflecting uniform stress distribution and favourable deformation coordination within the support. For the intermediate intervals (Fig. 6c–f, depths 50–125 m), curve slopes steepen, peak displacements rise to approximately 0.28 mm, and profiles transition from linear to slightly curved forms, indicative of progressively stronger stress redistribution and convergence around the shaft. For the deep intervals (Fig. 6g–k, 190–385 m), the curves become clearly nonlinear, with steep near-wall gradients followed by gradual flattening. Peak displacement ranges from 0.45 to 0.87 mm, indicating a broader deformation influence zone and increasing support demand with depth.

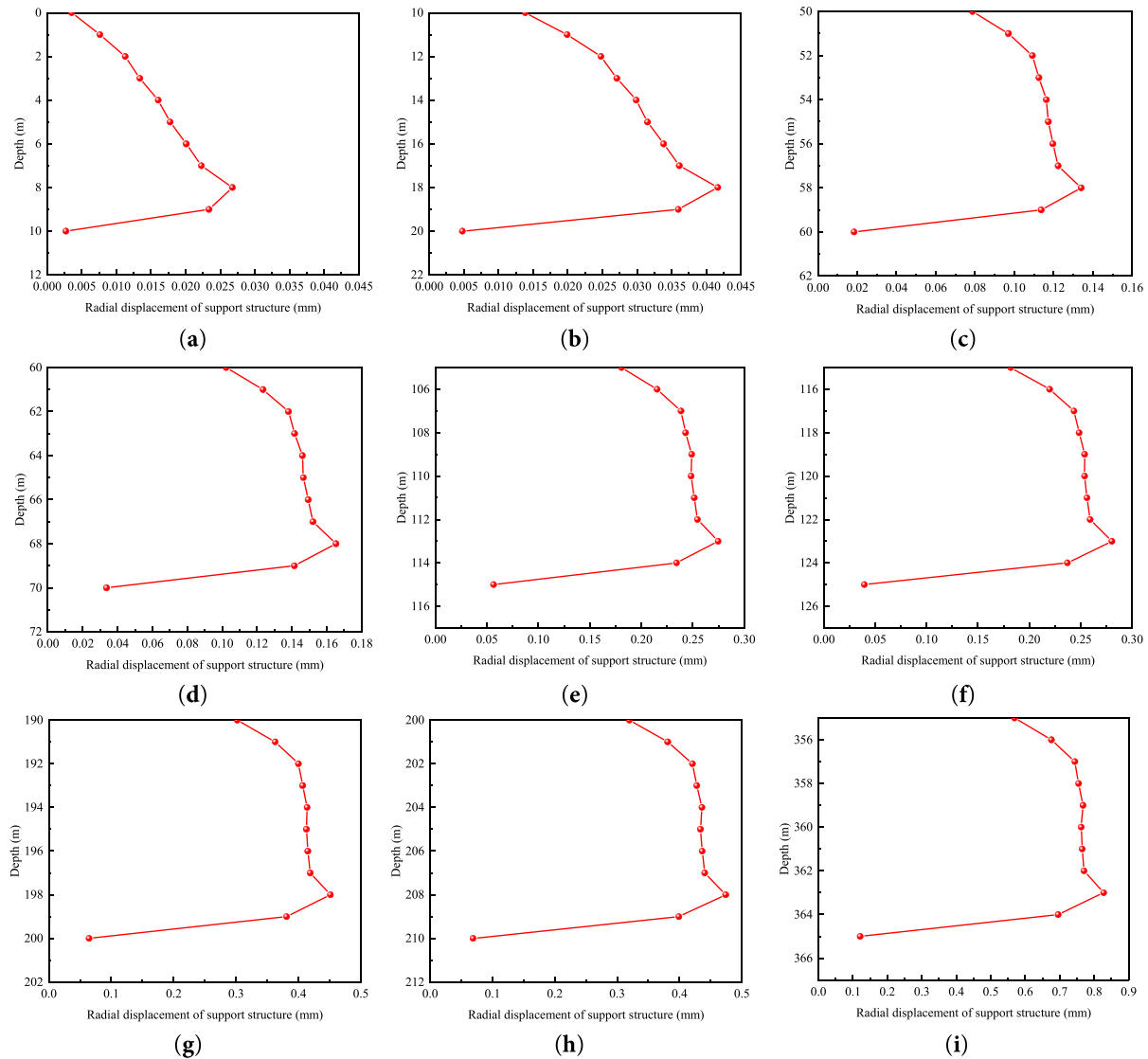


Figure 6: (Continued)

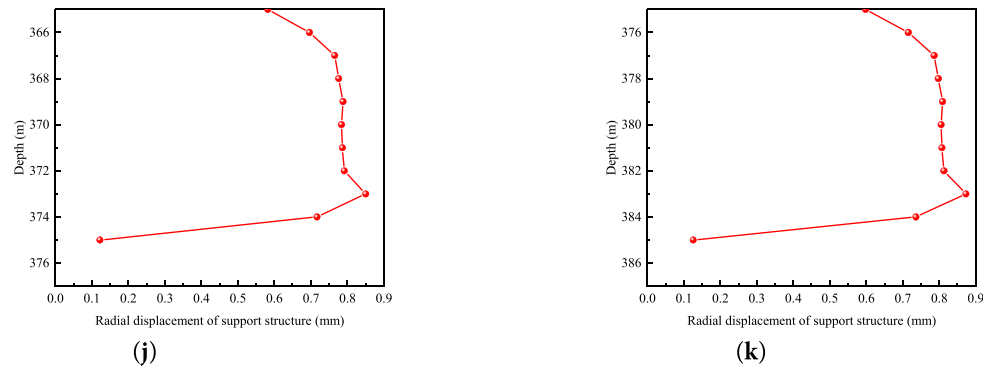


Figure 6: Radial displacement profiles of the support structure at different depth intervals: (a) depth = 0–10 m; (b) depth = 10–20 m; (c) depth = 50–60 m; (d) depth = 60–70 m; (e) depth = 105–115 m; (f) depth = 115–125 m; (g) depth = 190–200 m; (h) depth = 200–210 m; (i) depth = 355–365 m; (j) depth = 365–375 m; (k) depth = 375–385 m.

Fig. 7 summarizes the relationship between maximum radial displacement and depth across the selected intervals. The plot demonstrates a non-linear increase in peak displacement with depth. Below 100 m, growth is gradual (from 0.03 to 0.17 mm). Between 100 and 200 m, values rise to approximately 0.45 mm. Beyond 200 m, a sharp acceleration occurs, reaching 0.87 mm. This pattern underscores the heightened sensitivity of deformation to depth. Overall, the depth-dependent behaviour observed in the displacement contours (Fig. 5) and radial profiles (Fig. 6) of the support structure is primarily governed by the overburden stress γ_z . The linear increase in equivalent vertical pressure γ_z amplifies radial strains in the surrounding rock. At shallow depths under low stress, deformation is dominated by the elastic response of the support structure. As stress escalates, interfacial shear at the rock bolt-surrounding rock boundary intensifies, promoting stronger stress redistribution and convergence in the surrounding rock, with the support system accommodating greater radial displacement to facilitate stress redistribution. At greater depths, elevated γ_z drives stronger deformation and a broader deformation influence zone, diminishing anchoring efficiency and shifting stability control from the support-dominated regime to one governed by the surrounding rock.

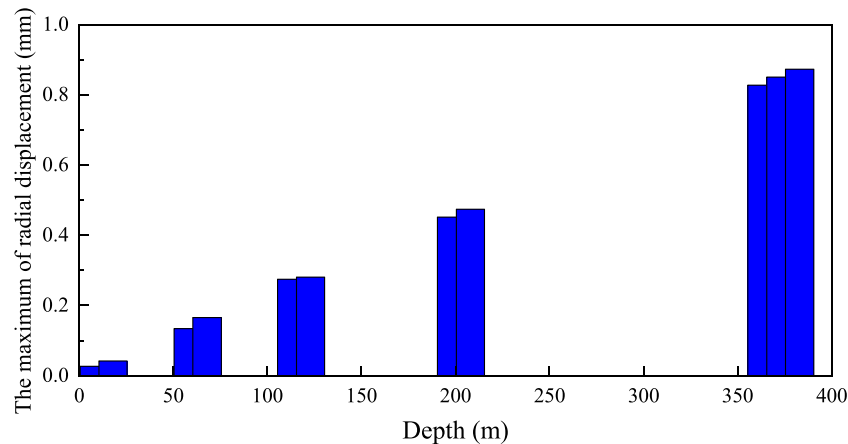


Figure 7: The maximum radial displacement of the support structure varying with depth.

3.3 Variation of Maximum In-Plane Principal Stress of the Support Structure with Depth

Fig. 8 presents maximum in-plane principal stress contours for the initial support and rock bolts across 11 selected depth intervals. Overall, the stress field is predominantly concentrated along the shaft wall and rock bolt zones, with colours shifting towards warmer tones as depth increases, indicating a progressive rise in maximum in-plane principal stress within the initial support and rock bolts. For the shallow intervals (Fig. 8a,b; depths 0–20 m), the stress field exhibits uniform blue-to-green distribution, with maximum stresses of 2.597×10^6 Pa and 3.347×10^6 Pa, respectively, primarily localised near the wall and minimal in the rock bolt zones. For the intermediate intervals (Fig. 8c–f, depths 50–125 m), green-to-yellow regions extend towards the rock bolts, with high-stress zones expanding markedly and concentrating at the wall and rock bolt bodies; maximum stresses range from 1.123×10^7 Pa to 2.383×10^7 Pa. For the deep intervals (Fig. 8g–k, depths 190–385 m), the contours are dominated by yellow-to-red hues, forming distinct high-stress bands, particularly at the shaft wall and shaft bottom of the primary support and around the bolt heads, maximum stresses reach up to 7.526×10^7 Pa.

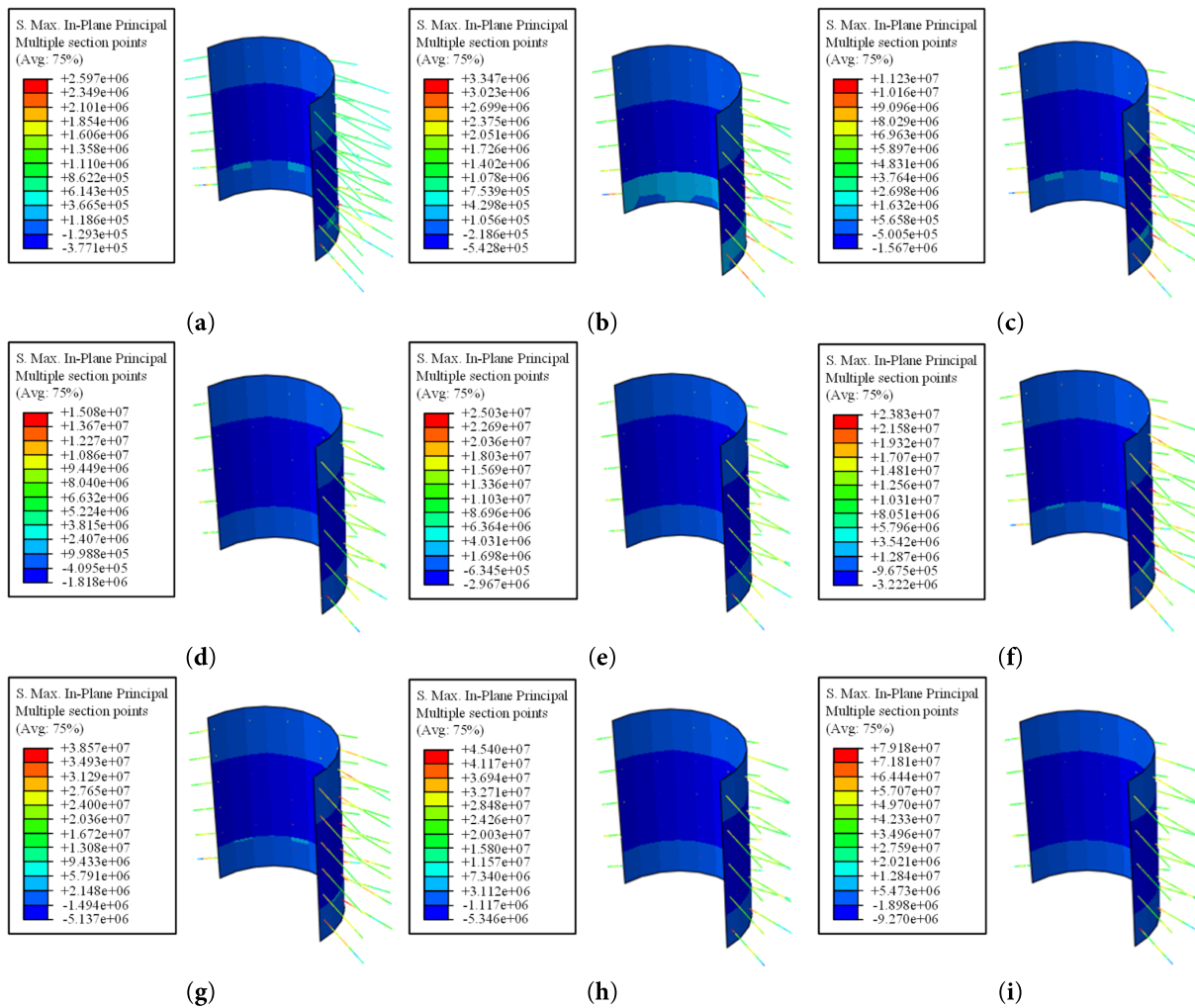


Figure 8: (Continued)

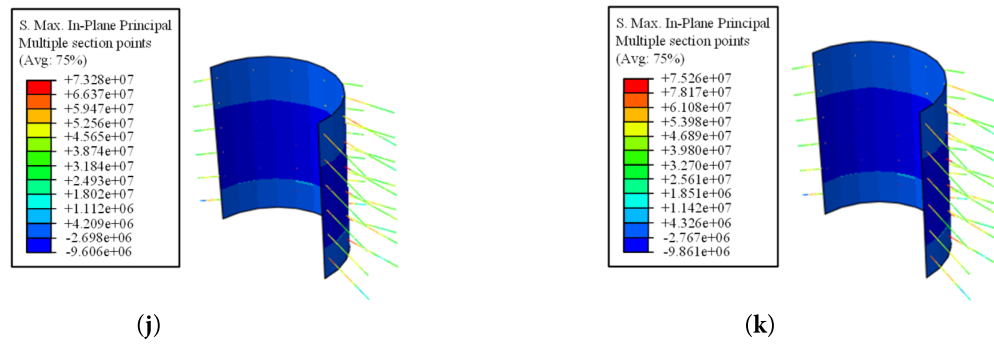


Figure 8: Compressive maximum in-plane principal-stress magnitude contours of the support structure at different depth intervals (MPa): (a) depth = 0–10 m; (b) depth = 10–20 m; (c) depth = 50–60 m; (d) depth = 60–70 m; (e) depth = 105–115 m; (f) depth = 115–125 m; (g) depth = 190–200 m; (h) depth = 200–210 m; (i) depth = 355–365 m; (j) depth = 365–375 m; (k) depth = 375–385 m.

Fig. 9 illustrates maximum in-plane principal stress profiles along radial paths in the support structure. For the shallow intervals (Fig. 9a–c, depths 0–60 m), the curves are nearly linear, with stresses decaying rapidly over short distances and peak values around 1.55 MPa at 57 m depth. For the intermediate intervals (Fig. 9d–f, depths 60–125 m), initial curve slopes steepen, with peaks shifting towards the wall, reflecting increased load transfer from surrounding rock deformation and activation of bolt anchorage, maximum in-plane principal-stress values lie between 1.8 and 3.18 MPa. For the deep intervals (Fig. 9g–k, depths 190–385 m), the profiles are steeply non-linear, featuring pronounced peaks at the wall followed by sharp declines, indicative of substantial radial compressive loading under extreme confinement and marked stress concentration; maximum in-plane principal-stress values attain 9.74 MPa.

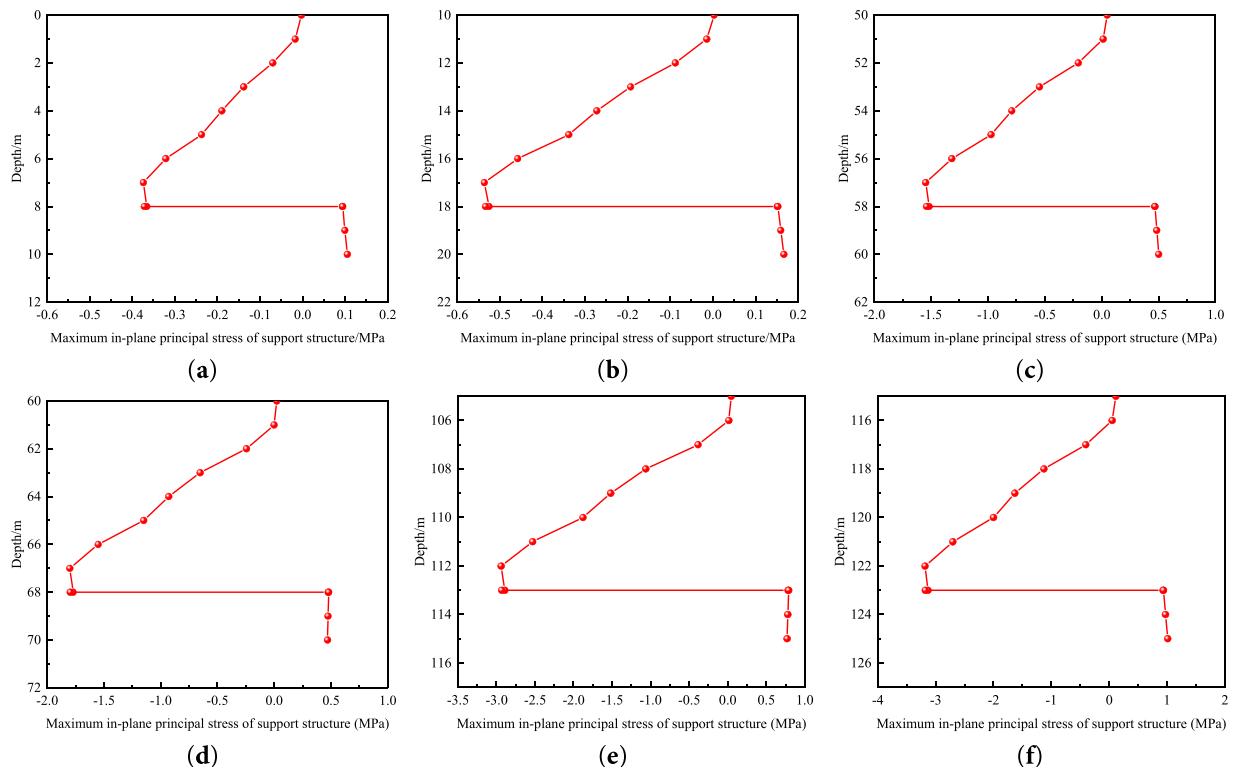


Figure 9: (Continued)

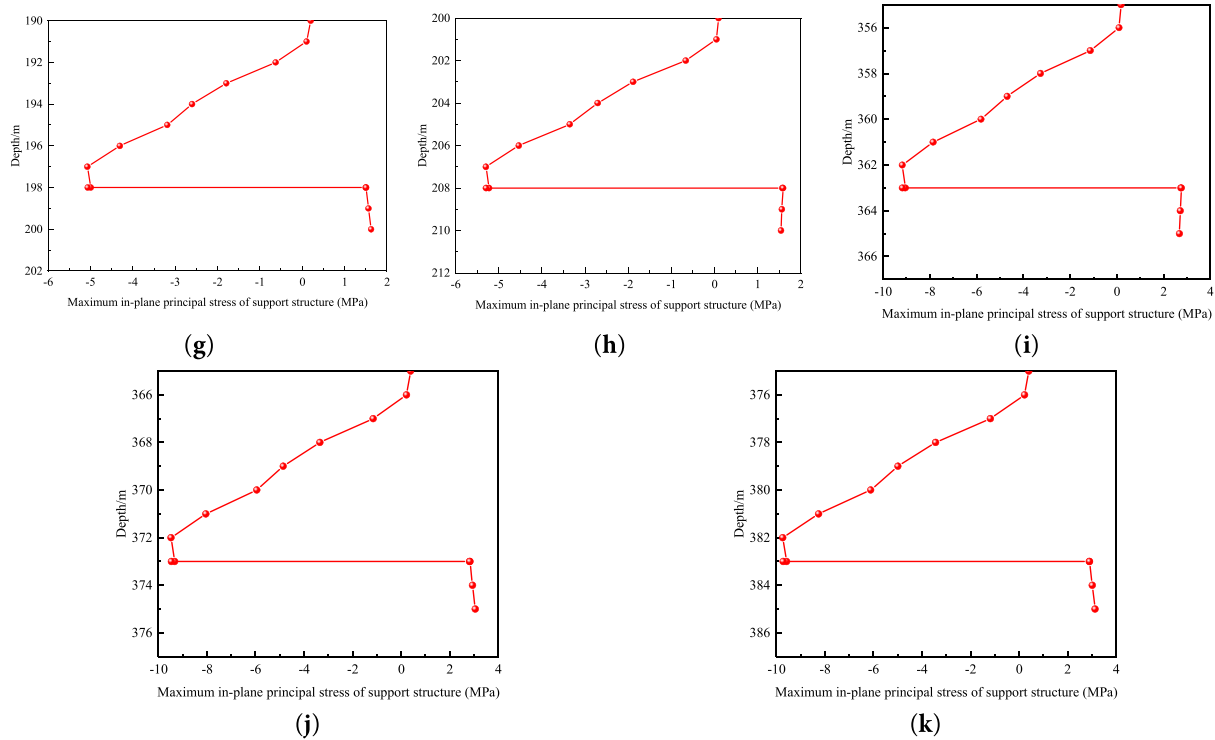


Figure 9: Variation of maximum in-plane principal stress of the support structure along radial paths at different depths: (a) depth = 0–10 m; (b) depth = 10–20 m; (c) depth = 50–60 m; (d) depth = 60–70 m; (e) depth = 105–115 m; (f) depth = 115–125 m; (g) depth = 190–200 m; (h) depth = 200–210 m; (i) depth = 355–365 m; (j) depth = 365–375 m; (k) depth = 375–385 m.

Fig. 10 summarises the relationship between maximum in-plane principal stress and depth across the selected intervals. The plot reveals a non-linear increase in peak stress with depth, directly attributable to the linear augmentation of the equivalent vertical overburden pressure γ_z applied at the model top. For the shallow regions, lower γ_z values enable substantial self-stabilisation of the surrounding rock, resulting in limited load transfer to the support structure and low stress levels. As depth increases, the linear rise in γ_z proportionally amplifies radial compressive forces from surrounding rock deformation. The support structure—particularly the stiffer primary lining—resists this convergence by sustaining sharply escalating support stresses. The rock bolts accommodate shear-induced tensile stresses through their axial resistance, generating localised concentrations at anchorage ends, as evidenced by high-stress zones along the bolt bodies and heads in the contours. The accelerated, non-linear stress escalation at greater depths is attributed to increasing confinement, stress redistribution and support load transfer under elevated confinement, diminishing its load-bearing capacity and compelling the support system to assume a larger share of the burden.

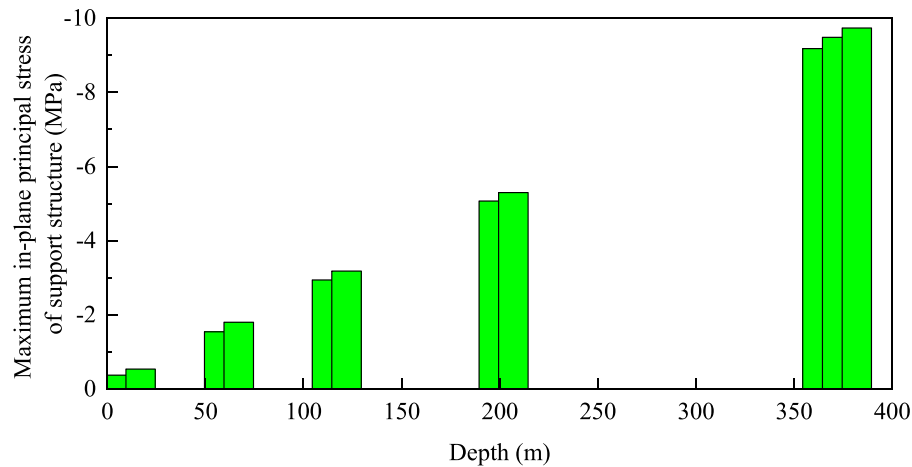


Figure 10: Maximum in-plane principal stress of the support structure with depth.

3.4 Variation of Radial Displacement of the Surrounding Rock with Depth

Fig. 11 presents total displacement magnitude contours for the surrounding rock across 11 depth intervals. The contours illustrate a systematic amplification of radial displacement with increasing shaft depth, with maximum values rising from approximately 9.481×10^{-5} m in shallow intervals (e.g., Fig. 11a, depths 0–10 m) to 2.911×10^{-3} m in deep intervals (e.g., Fig. 11k, depths 375–385 m). Specifically, for the shallow intervals (Fig. 11a,b, depths 0–20 m), the displacement field exhibits uniform blue-to-green distribution, confined to a narrow annular zone adjacent to the shaft wall, with values approaching zero beyond this region and limited spatial extent. As depth progresses to intermediate intervals (Fig. 11c–f; depths 50–125 m), gradients intensify, green-to-yellow zones radiate outward, the high-displacement annulus widens substantially (approximately doubling in breadth), and maximum values increase by roughly an order of magnitude relative to shallow zones. Internal rock displacements become more pronounced, reflecting expanded influence. For the deep intervals (Fig. 11g–k, depths 190–385 m), the field shifts to yellow-to-red dominance, with marked heterogeneity, pronounced annular diffusion, and displacement gradients penetrating deeper into the rock mass. These patterns indicate a transition from small, localised deformation at shallow depths to larger and more spatially distributed stress-relief deformation at greater depths.

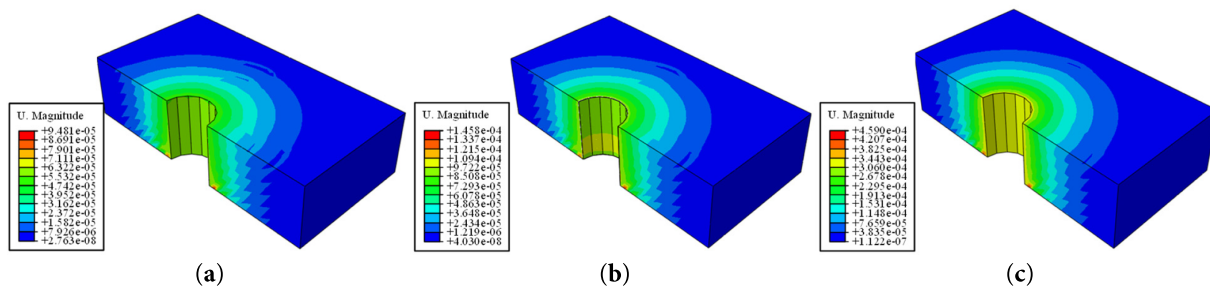


Figure 11: (Continued)

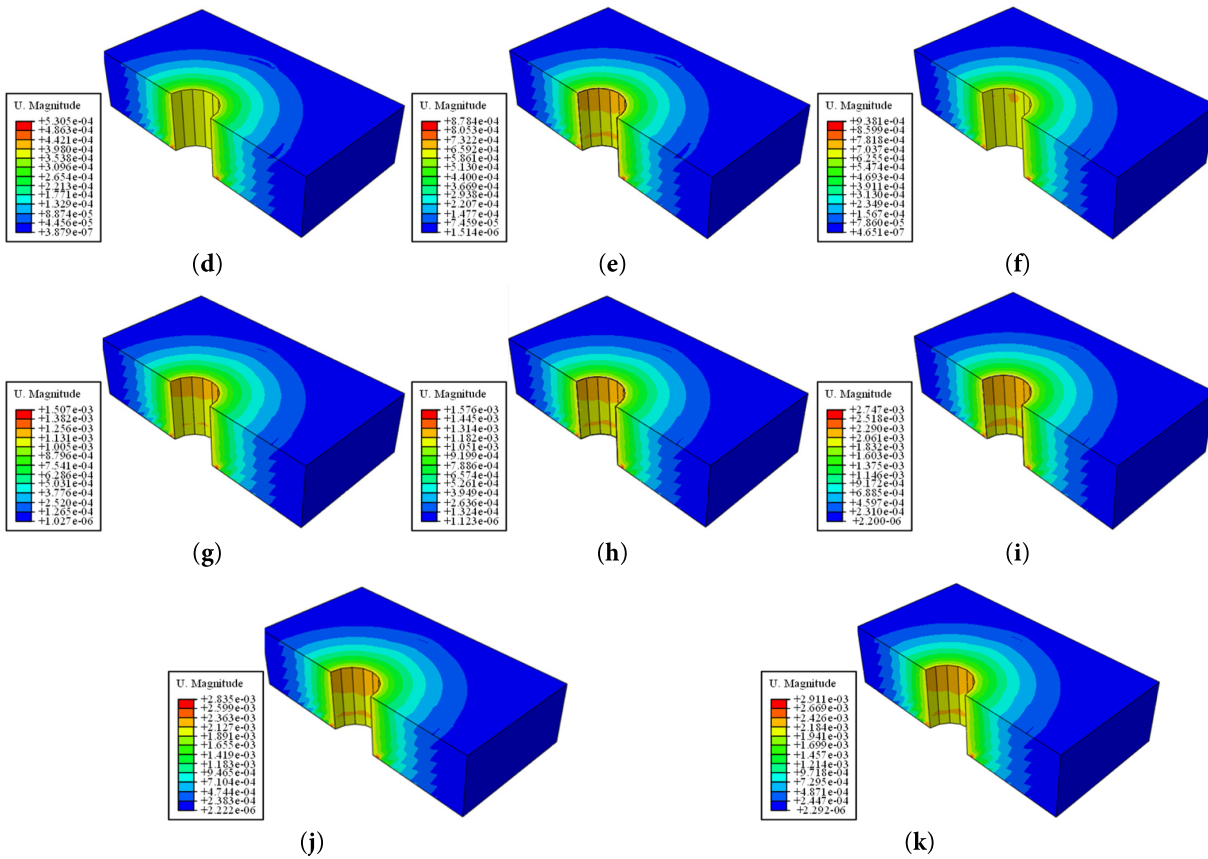


Figure 11: Total displacement magnitude contours of surrounding rock at different depths: (a) depth = 0–10 m; (b) depth = 10–20 m; (c) depth = 50–60 m; (d) depth = 60–70 m; (e) depth = 105–115 m; (f) depth = 115–125 m; (g) depth = 190–200 m; (h) depth = 200–210 m; (i) depth = 355–365 m; (j) depth = 365–375 m; (k) depth = 375–385 m.

4 Parametric Analyses

4.1 Influence of Shotcrete Thickness

According to the project support schedule, the actual primary sprayed-concrete thickness ranges from 0.20 to 0.26 m in the investigated shaft. The thicknesses of 0.10, 0.20, 0.30 and 0.40 m used in the numerical analysis constitute a deliberately extended sensitivity range for examining the influence of support stiffness. In particular, the 0.40 m case is a hypothetical upper-bound numerical case and does not represent the constructed support, a code-specified thickness or a design recommendation. To evaluate the sensitivity of support parameters under high geostress conditions, the 300–312 m interval was selected as a representative deep-level case. This interval lies within the depth range where the numerical results indicate accelerated deformation and greater support demand. Shotcrete thickness, bolt length and bolt spacing were varied one at a time to identify their relative influence on radial displacement and support stress within the selected parameter ranges. The results should therefore be interpreted as mechanical sensitivity evidence rather than as a global optimisation of the support system. Because only one-factor-at-a-time cases were evaluated, the interaction effects among shotcrete thickness, rock-bolt length and rock-bolt spacing were not quantified. A full design optimisation would require combined parameter cases together with cost, constructability, safety-factor and allowable-deformation criteria. Accordingly, the 30 cm shotcrete thickness

discussed below is treated as a diminishing-return threshold in the present numerical cases, not as a universal economic optimum.

Fig. 12 presents the radial displacement and maximum in-plane principal stress of the support structure at depths of 300–312 m for shotcrete thicknesses of 10, 20, 30, and 40 cm. As shown in Fig. 12a, radial displacement increases with depth but decreases as the shotcrete thickness increases. The maximum radial displacement decreases from 0.88 mm at 10 cm to 0.50 mm at 40 cm, corresponding to a reduction of approximately 43.2%. The similar curve shapes indicate that shotcrete thickness mainly affects the deformation magnitude rather than its depth-wise distribution. Similarly, the maximum in-plane principal-stress curves in Fig. 12b show increasing compressive stress with depth. Increasing shotcrete thickness shifts the curves towards zero, and the signed compressive value changes from -14.24 MPa at 10 cm to approximately -4.21 MPa at 40 cm. These results demonstrate that greater shotcrete thickness increases primary-support stiffness and reduces deformation and stress concentration.

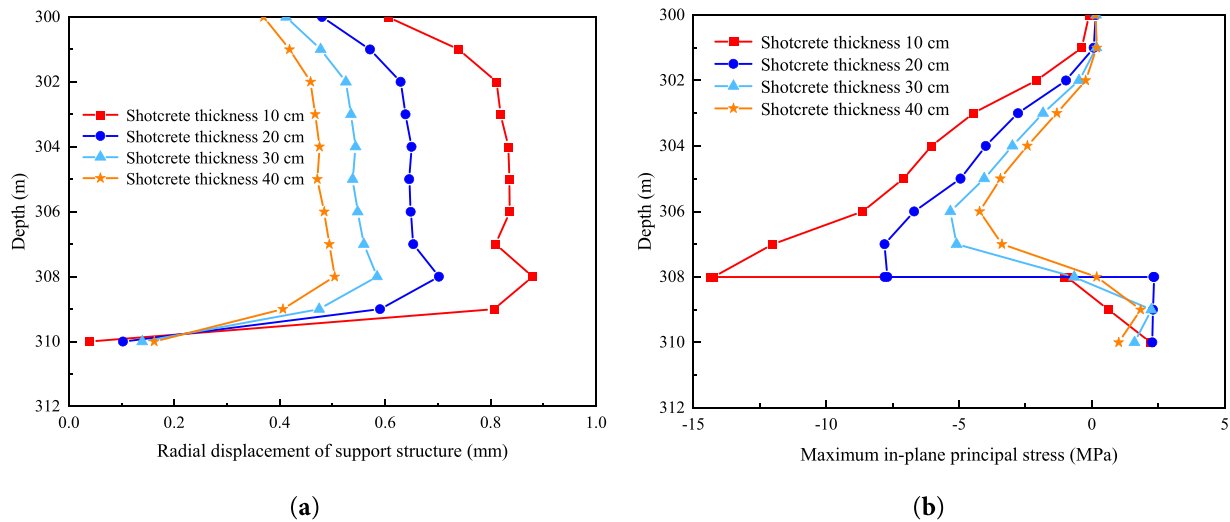


Figure 12: The influence of shotcrete thickness on the mechanical response of the support structure: (a) the curve of radial displacement varying with depth; (b) the curve of principal stress varying with depth.

Fig. 13 quantifies the influence of shotcrete thickness on maximum radial displacement and principal stress of the support structure. Both metrics exhibit monotonic decreasing trends with diminishing marginal returns. Specifically, increasing thickness from 10 to 20 cm yields reductions of approximately 20.45% in displacement and 45.68% in stress, whereas further increases from 30 to 40 cm result in reductions of 14.53% and 20.68%, respectively. This suggests that thickness augmentation within a certain range substantially enhances support stiffness and load-bearing capacity, but benefits saturate beyond a threshold. The results show nonlinear reductions in both radial displacement and support stress. The displacement reduction was approximately 20.45% from 10 to 20 cm and 14.53% from 30 to 40 cm. Increasing thickness raises the axial and bending stiffness of the primary support, while the support material properties remain those listed in Table 2. The stiffer support restrains radial convergence and reduces stress concentration. The smaller incremental improvement at greater thickness indicates diminishing returns within the investigated range.

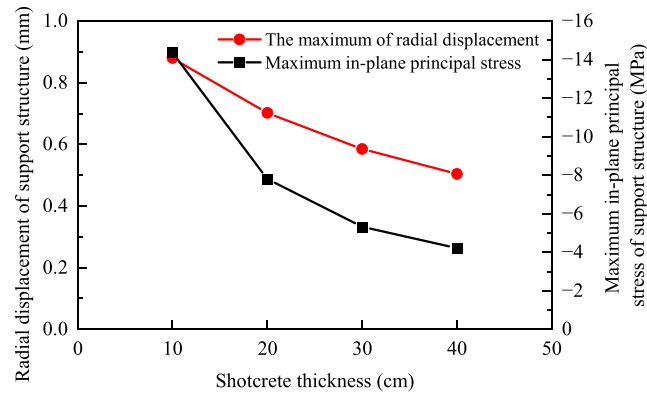


Figure 13: The influence of shotcrete thickness on maximum radial displacement and maximum in-plane principal stress of the support structure.

4.2 Influence of Rock Bolt Length

According to the project support schedule, the rock-bolt lengths adopted in the investigated shaft range from 2.5 to 3.5 m. The 4.0 m case in the numerical model is retained only as a hypothetical upper-bound sensitivity case for examining the marginal mechanical effect of increasing the effective anchorage length. It does not represent the constructed support or a recommendation to install 4.0 m-long one-piece bolts in the 6.4 m-diameter shaft. Moreover, simultaneous activation of the bolt elements in a numerical step represents the completed support state after an excavation cycle and does not imply simultaneous physical installation of all bolts at the construction site.

Fig. 14 illustrates variations in radial displacement and principal stress along depth (300–312 m) for the support structure under different rock bolt lengths (2.5, 3.0, 3.5 and 4.0 m). As shown in Fig. 14a, radial displacement decreases with increasing bolt length, while all curves exhibit an upward trend with depth. At equivalent depths (e.g., 308 m), the highest displacements correspond to the shortest bolts (2.5 m), reducing substantially to approximately 0.51 mm at 4.0 m length from 0.70 mm. The consistent depth-wise progression across curves indicates that bolt length primarily modulates displacement magnitude rather than its distributional pattern along depth. Correspondingly, maximum in-plane principal-stress curves (negative values denoting compression) amplify with depth in Fig. 14b. However, longer bolts shift curves towards zero, mitigating peak stresses from approximately -7.8 MPa at 2.5 m to -5.70 MPa at 4.0 m, demonstrating that extended bolts more effectively redistribute and alleviate concentrated stresses on the initial support.

Fig. 15 depicts the influence of rock bolt length on maximum radial displacement and principal stress of the support structure. Both metrics display a strong linear negative correlation with length. Specifically, peak radial displacement declines from 0.70 mm at 2.5 m to 0.51 mm at 4.0 m, while maximum in-plane principal-stress changes from -7.80 to -5.70 MPa, confirming bolt length as a critical parameter for enhancing support performance.

These observations indicate that increasing rock bolt length induces linear attenuation in both radial displacement and maximum in-plane principal stress within the support structure. Mechanistically, longer bolts extend the anchorage into deeper, more competent rock zones, thereby enlarging the effective anchoring zone. Consequently, appropriate extension of bolt length emerges as an efficacious means to ameliorate the stress state of the rock-support system and enhance overall stability. Although the 4.0 m numerical case produces the lowest calculated displacement and stress, this result reflects only the mechanical tendency associated with increasing the effective anchorage length. Considering the available installation space and

the project construction conditions, the engineering interpretation is restricted to the constructible range of 2.5–3.5 m, and the 4.0 m case is not proposed for field application.

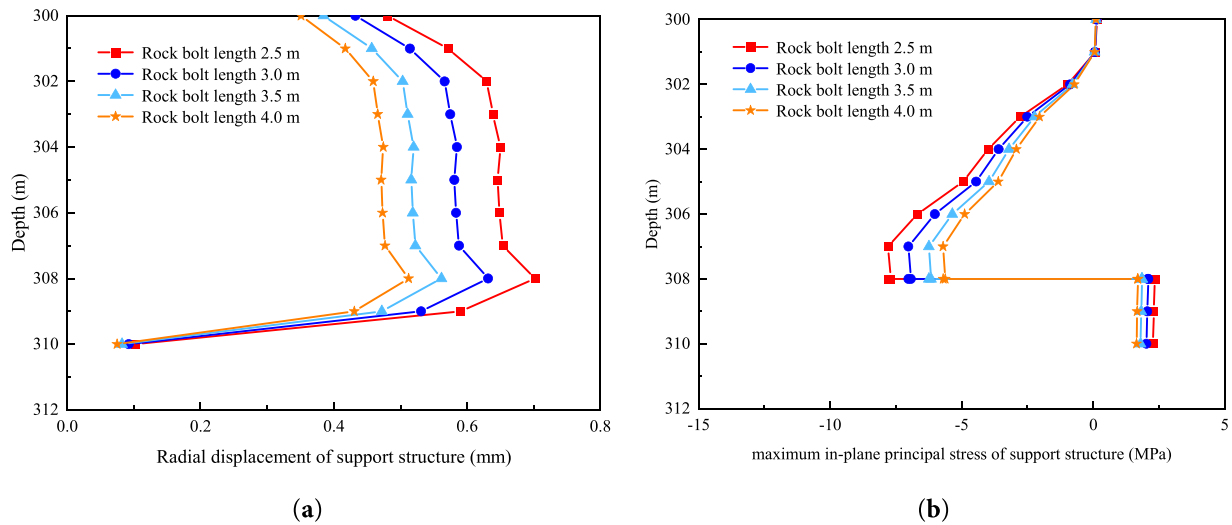


Figure 14: The influence of rock bolt length on the mechanical response of the support structure: (a) the curve of radial displacement varying with depth; (b) the curve of principal stress varying with depth.

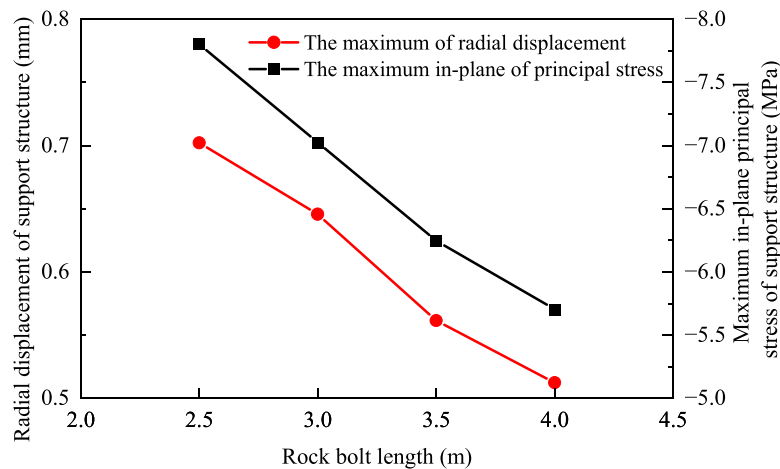


Figure 15: The influence of rock bolt length on maximum radial displacement and principal stress of the support structure.

4.3 Influence of Rock Bolt Spacing

Fig. 16 illustrates variations in radial displacement and principal stress along depth (300–312 m) for the support structure under different rock bolt spacings (80, 100, 120 and 150 cm). As shown in Fig. 16a, radial displacement increases with greater bolt spacing. At equivalent depths, the lowest displacements occur at 80 cm spacing, rising substantially at 150 cm, with a maximum amplification of 33.3%. Although depth-wise progression remains consistent across curves, wider spacing elevates the overall displacement envelope. Correspondingly, in Fig. 16b, maximum in-plane principal-stress curves amplify in magnitude with depth; however, increased spacing shifts curves further from zero, exacerbating peak stresses from approximately -6.84 MPa at 80 cm to -9.62 MPa at 150 cm. This indicates that wider spacing impairs the support system's

collective restraint on surrounding rock deformation, transferring greater loads to the primary lining and intensifying stress concentration.

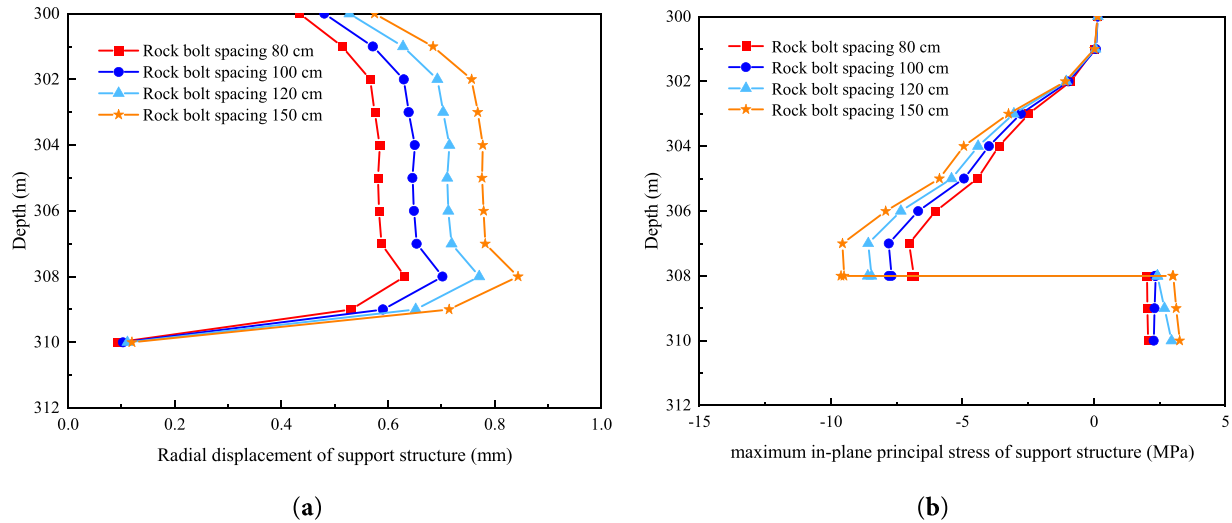


Figure 16: The influence of rock bolt spacing on the mechanical response of the support structure: (a) the curve of radial displacement varying with depth; (b) the curve of principal stress varying with depth.

Fig. 17 depicts the influence of rock bolt spacing on the maximum radial displacement and principal stress of the support structure. Both metrics exhibit significant linear positive correlations with spacing. These patterns demonstrate that increasing rock bolt spacing induces linear amplification in both radial displacement and maximum in-plane principal stress within the support structure, with each 20 cm increment yielding deformation escalations of approximately 20%–30% and heightened overall sensitivity. Narrower spacings (e.g., 80 cm) correspond to higher bolt density, promoting overlap of reinforcement zones between adjacent bolts and enhancing uniform constraint. Conversely, wider spacing diminishes bolt group density, creating inadequately reinforced weak zones between bolts where surrounding rock deformation readily concentrates. This uneven stress distribution compels the primary support to bear pressure from a larger loosened rock volume, manifesting as synchronous linear growth in displacement and stress.

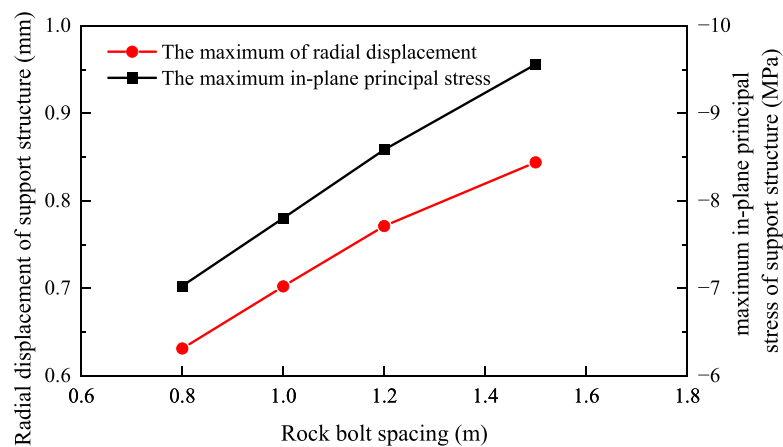


Figure 17: The influence of rock bolt spacing on maximum radial displacement and principal stress of the support structure.

5 Discussion

This study used a three-dimensional Abaqus model to investigate the depth-dependent stress-deformation response of the surrounding rock and support system and to evaluate the sensitivity of shotcrete thickness, rock-bolt length, and longitudinal spacing. The results show nonlinear increases in radial displacement, support stress, and the extent of the surrounding-rock deformation zone with burial depth. Field monitoring also recorded increasing surrounding-rock pressure with depth, while secondary-lining stress varied non-monotonically and remained high at both the shallowest and deepest monitored sections. These trends are broadly consistent with previous deep-shaft studies linking greater overburden stress to intensified stress redistribution and support demand [20,22]. For the representative 300 m case, greater shotcrete thickness and rock-bolt length reduced deformation and stress concentration, whereas wider longitudinal bolt spacing weakened the support response. Because the parameters were varied one at a time over selected ranges, the observed ranking is conditional rather than universal. The monitoring results provide trend-level validation rather than direct displacement calibration because comparable field displacement measurements were unavailable.

Mechanically, increasing burial stress intensifies confinement and excavation-induced stress redistribution, causing greater shaft-wall convergence and more pronounced near-wall stress redistribution. This interpretation is consistent with the load-bearing principles of rock-bolting systems described by Li [5]. Within the investigated ranges, increasing shotcrete thickness from 10 to 30 cm reduced radial displacement by approximately 34.1%, whereas reducing bolt spacing from 150 to 80 cm produced a smaller reduction of approximately 25%. This comparison indicates that shotcrete stiffness made the larger contribution to deformation control, while bolt length and spacing mainly regulated the restraint provided by the anchorage zone. Longer bolts extended the reinforced zone into less-disturbed rock, whereas wider spacing created weak intervals and transferred greater demand to the shotcrete. These observations describe conditional load sharing among the surrounding rock, bolts and shotcrete under the adopted linear-elastic and fully bonded assumptions, rather than ultimate support capacity.

Several limitations should be considered when applying these results. The numerical cases extend only to the 375–385 m interval and do not represent the lowest 385–428 m section of the shaft, where groundwater and karst effects may become more significant. Consequently, the present results should not be extrapolated directly to this unmodelled bottom section. The model represents overburden using equivalent vertical stress and does not include tectonic horizontal stress. The ground was simplified into representative lithologic groups, and the support components were idealised as linear elastic. Groundwater and seepage-induced weakening may further increase support demand in locally connected karst zones. This issue will be investigated in future work using measured hydraulic and water-weakened rock parameters. In addition, the study does not provide a complete code-based support-capacity verification, because such a check would require comparing computed shotcrete stresses, bolt axial forces and connection or bond demands with the corresponding material strengths and design safety factors. The calculated support stresses and bolt forces are therefore treated as demand indicators for the investigated cases, rather than as evidence of code-based strength, safety-factor or allowable-deformation compliance.

6 Conclusions

- (1) The numerical results show a non-linear depth-dependent response of the support structure. Radial displacement and support stress increase slowly in shallow sections and accelerate in deeper sections, especially beyond approximately 200 m under the selected modelling assumptions.
- (2) Within the deliberately extended numerical sensitivity range, increasing shotcrete thickness from 10 to 40 cm reduced the calculated maximum radial displacement from 0.88 to 0.50 mm, corresponding

to a reduction of approximately 43.2%. The 0.40 m case is a hypothetical upper-bound sensitivity case rather than a project design value. Therefore, the result demonstrates the relative influence of support stiffness and should not be interpreted as a recommendation to adopt 0.40 m thick shotcrete.

- (3) Within the investigated parameter ranges, increasing rock-bolt length reduced the calculated support displacement and stress, whereas increasing bolt spacing increased both quantities. These trends describe the tested one-factor cases and should not be interpreted as universal linear laws.
- (4) Within the investigated parameter ranges, the one-factor-at-a-time comparison indicated that shotcrete thickness produced the strongest displacement-control effect, followed by rock-bolt length and spacing. This range-dependent result supports prioritising primary-support stiffness, together with moderate bolt lengthening and appropriate spacing control, but it should not be interpreted as a universal sensitivity ranking or global optimisation rule.
- (5) Field records show rapid adjustment followed by gradual stabilisation. Surrounding-rock pressure magnitude increased with representative depth, whereas secondary-lining stress varied non-monotonically; the monitoring data therefore provide trend-level support rather than point-by-point calibration.

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Availability of Data and Materials: The data that support the findings of this study are available from the corresponding author, Mingfang Wu, upon reasonable request.

Ethics Approval: Not applicable.

Conflicts of Interest: The authors declare no conflict of interest.

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