

Article

Response Characteristics and Adaptability Analysis of the Benching Method in Grade IV and V Surrounding Rocks in Sandstone–Mudstone Strata

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Abstract

To improve construction efficiency for large-section tunnels in sandstone–mudstone strata, this study investigates the applicability of the two-bench method and the three-bench method for grade IV and grade V surrounding rock, respectively. Based on FLAC3D, numerical simulations of excavation and support for the two benching methods were conducted to analyze deformation responses, including ground settlement, crown settlement, haunch convergence, floor uplift, and face extrusion. The simulation results were then compared and validated against field monitoring data to evaluate the applicability and feasibility of the construction methods. The results show that, for grade IV surrounding rock excavated using the two-bench method, crown settlement, floor uplift, and horizontal convergence converge and stabilize on days 17, 15, and 25, with stable values of 14.0 mm, 10.3 mm, and 13.2 mm, respectively. For grade V surrounding rock excavated using the three-bench method, these indices stabilize on days 24, 22, and 32, with stable values of 26.3 mm, 20.3 mm, and 20.8 mm, respectively. The surrounding rock pressures at the crown and spandrel gradually attenuate after excavation and stabilize at 1–4 MPa after approximately 20–26 days, whereas stress release at the haunch lasts longer and the stabilized stress level remains higher. Meanwhile, the anchor bolt axial force at the haunch is significantly greater than that at the spandrel, indicating that the haunch is a critical zone for support load-bearing and deformation control. The benching method can effectively control surrounding rock deformation under grade IV and V surrounding rock conditions in sandstone–mudstone strata; however, in engineering practice, the haunch should be treated as a key monitoring target, and targeted support and reinforcement measures should be implemented.

Keywords: sandy-mudstone strata; the two-bench method; the three-bench method; simulations; spandrel; haunch



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1. Introduction

With tunnel engineering developing toward large cross-sections and long spans, sandstone–mudstone strata, characterized by pronounced water sensitivity and creep behavior, pose greater challenges to tunnel construction and surrounding rock stability under such geological conditions [1–6]. This type of surrounding rock features well-developed bedding, weak interlayers, and fractures, showing pronounced anisotropy and heterogeneity [7]. Owing to its low cementation, it readily softens and disintegrates

upon water exposure, and its strength and stiffness are sensitive to stress unloading and variations in water content [8]. Under excavation-induced disturbance and stress release, the plastic zone readily propagates along bedding planes, typically manifesting as combined deformation such as crown settlement, haunch convergence [9], and invert heave [10], and may even trigger instability of the initial support. At present, methods such as the double sidewall drift, single sidewall drift, and central pilot drift are commonly employed; however, their applicability is still constrained by surrounding rock conditions, construction efficiency, and deformation control capability. Therefore, numerical simulations should be combined with field monitoring to elucidate the mechanical responses and deformation patterns of different construction methods, thereby providing a basis for method selection and support-parameter design.

For large-section tunnel construction in weak surrounding rock, the sidewall drift method is commonly adopted in engineering practice to control excavation-induced disturbance and surrounding rock deformation. Yang et al. [11] conducted a comparative study, based on numerical simulations, of the composite/conventional double sidewall drift method, the three-bench method, and the three-bench method with a reserved core in a shallow-buried super-large-span tunnel. The results showed that the composite/conventional double sidewall drift method provides better surrounding rock stability after excavation, and that the overall safety of the initial support is superior to that of the three-bench method. Wen et al. [12] compared the CRD method, the three-bench seven-step method, and the double sidewall drift method, and concluded that the double sidewall drift method performs best in controlling deformation in large-section tunnels; however, the support system needs to be closed into a ring as soon as possible to reduce the risk of stress concentration. Xu et al. [13] carried out a comparative study of the double sidewall drift method and the three-bench four-step excavation method for an ultra-large-section tunnel. The results showed that the double sidewall drift method induces less disturbance to the surrounding rock and offers advantages in deformation control and construction safety; they further recommended adopting a zoned selection strategy in accordance with the geological conditions. Qiao et al. [14] aiming to address the low construction efficiency and high cost of the double sidewall drift method in super-large-section loess tunnels, proposed a “five-step single sidewall drift method” and suggested placing the middle diaphragm at approximately 45% of the tunnel width to reduce surrounding rock deformation while lowering the load borne by the steel arches. Huang et al. [15] investigated soft-rock tunnels under high in situ stress by comparing multiple excavation methods and validating the results with field measurements and pointed out that the advance pilot tunnel enlargement method can effectively reduce surrounding rock deformation and stress levels by releasing in situ stress in advance. Chen et al. [16] focusing on a shallow-buried ultra-large-section tunnel in tuff, pointed out that the SD method is suitable for grade V surrounding rock, whereas the HB method is more feasible and economical for grade III–IV surrounding rock. Ma et al. [17] focusing on shallow-buried ultra-large-span tunnels in weak tuff, proposed a construction method selection chart applicable to weak surrounding rock tunnels with spans of 17–34 m. Both studies provide a reference basis for construction method optimization and parameter design for tunnels in tuff strata.

To improve construction efficiency and deformation control in large-section tunnels through weak surrounding rock, numerous comparative and optimization studies have been conducted on the benching method. Pan et al. [18] performed a comparative evaluation and optimization of excavation methods for large-section tunnels in sandstone–mudstone strata based on numerical simulations combined with field monitoring. They concluded that the double-bench method is more suitable under grade IV surrounding rock conditions, and that optimizing the excavation sequence reduced crown

settlement, floor uplift, and horizontal convergence by 50.4%, 39.6%, and 57.8%, respectively; they also emphasized that the arch waist is a stress-concentration zone and a key area requiring enhanced support. Yang [19], Qiao [20], Chen [21], and others combined numerical analyses with field monitoring to investigate the deformation mechanisms and optimization strategies for three-bench-type methods in large-section tunnels through weak surrounding rock. They revealed that repeated excavation disturbances can cause unloading and expansion of the loosened zone, inducing asymmetric large deformation, and proposed pre-reinforcement measures consisting of “advance curtain grouting + pipe roofing + three-bench temporary invert.” In addition, comparisons among different three-bench methods showed that the scheme of “three-bench temporary invert + advance pipe roofing,” which enables earlier closure of the support into a ring, is generally superior to the three-bench seven-step method in deformation control and suppression of the plastic zone. Wang et al. [22] focusing on large-section tunnels in water-rich soil–sand interface strata, compared multiple excavation methods and concluded that the “three-bench method with a reserved core” offers advantages in controlling surrounding rock convergence and improving construction efficiency. Zhu et al. [23] optimized the bench parameters for tunnels in water-rich fault fracture zones, suggesting that the bench heights for the HB method should be 0.45 H/0.35 H/0.20 H, and that the upper and lower benches on both the left and right sides for the CD method should each be set to 0.50 H, so as to balance surrounding rock stability and construction safety.

Previous studies on weak surrounding rock conditions such as sandstone–mudstone strata have mainly focused on comparative analyses of traditional excavation methods. However, the applicability of different bench excavation methods to sandstone–mudstone strata of different surrounding rock grades has not yet been systematically clarified. Existing studies have mainly used displacement response as the primary validation index, while comparatively less attention has been paid to the evolution of surrounding rock pressure and support loads. Meanwhile, the inconsistency between the excavation stages in numerical simulation and the actual start time of field monitoring has rarely been explicitly incorporated into the analytical framework. This study takes a large-section highway tunnel in sandstone–mudstone strata as the research object and investigates the applicability of the two-bench method and the three-bench method under Grade IV and Grade V surrounding rock conditions, respectively. By integrating numerical simulation with field monitoring data, this study comprehensively analyzes the responses of deformation, surrounding rock pressure, and rock bolt axial force. It further incorporates the effect of monitoring time lag, establishes a graded applicability analysis framework for bench excavation methods, and identifies the haunch as the key control zone for construction monitoring and support optimization.

2. Engineering Background

The tunnels are located in Guangyuan City, Sichuan Province, China, in the mountainous area along the northern margin of the Sichuan Basin and are typical trans-mountain tunnels. At the entrance of Tunnel 1, the ground surface slope is approximately 10–20°, and the overburden is mainly gravelly clay; at the exit, the surface slope is about 20–30°, with sporadic outcrops of bedrock. At the entrance of Tunnel 2, the ground surface slope is approximately 25–40°; at the exit, the slope is about 25–35°, and bedrock is extensively exposed at both the entrance and exit sections. Both tunnels cross mountain ridges, and the terrain along the alignment is strongly undulating, with alternating steep and gentle slopes (Figure 1). The maximum burial depths of Tunnel 1 and Tunnel 2 are 328 m and 320 m, respectively. The surrounding rock is mainly composed of mudstone and sandstone, with an overall soft lithology that is highly susceptible to excavation disturbances. The rock

mass structure is dominated by thin- to medium- and thick-bedded strata, with overall good integrity; however, the interbedded cementation is loose and local structural fractures are well developed, which makes the rock mass prone to block fall, roof collapse, and localized caving during construction.

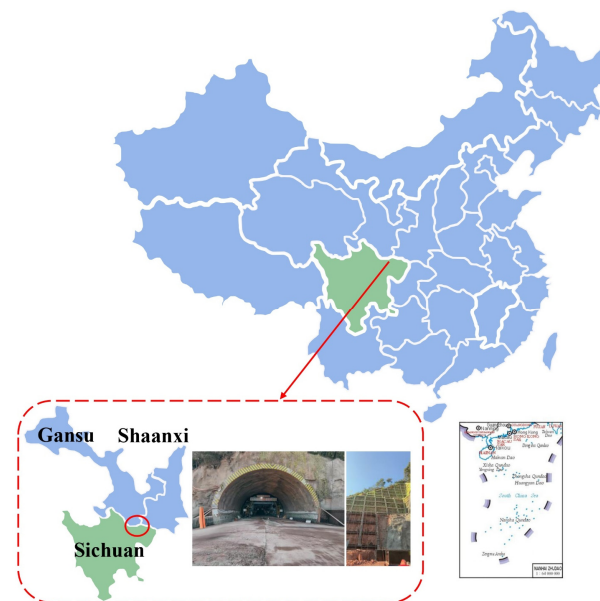


Figure 1. Geographical location of the tunnel.

To address the problems of high disturbance sensitivity and poor stability during tunnel construction in weakly layered surrounding rocks such as sandstone–mudstone strata, this paper, taking Tunnel 1 and Tunnel 2 as engineering cases, conducts a study on bench method construction by combining numerical simulation with field monitoring. The feasibility of implementing the bench method under such geological conditions is elucidated, and potential mechanically weak zones are identified, thereby providing a reliable basis for construction method optimization and rational selection of support parameters.

3. Adaptability Analysis of Bench Method in Sandy Mudstone Strata

3.1. Numerical Model Construction

FLAC3D6.0 was used to construct the tunnel model, and the two-bench and three-bench excavation methods were simulated separately. The numerical model is shown in Figure 2. To eliminate the influence of tunnel excavation on the model boundaries, the model boundaries were set at a distance of 3 to 5 times the tunnel diameter from the tunnel [18], resulting in overall model dimensions of 100 m (length) $\times$ 120 m (width) $\times$ 100 m (height). The y-axis represents the tunnel excavation direction, while the x-axis and z-axis denote the horizontal and vertical directions, respectively. A monitoring section is arranged at $y = 60$ m.

In the initial stress field, the vertical stress was determined according to the tunnel burial depth and the unit weight of the overlying strata, while the horizontal stress was represented by the lateral pressure coefficient K_0 . Considering that the tunnel burial depth is approximately 320–328 m and the surrounding rock density is about 2400 kg/m³, an equivalent overburden load stress of 8 MPa was adopted in the numerical model. Although the sand–mudstone strata in this project exhibit a certain degree of bedding and heterogeneity, the sandstone interlayers exposed in the upper bench are only about 0.3–0.5 m thick. Overall, the surrounding rock mass is still dominated by mudstone. These thin sandstone layers mainly occur as local interbeds rather than as thick, hard rock strata that control the overall mechanical response of the surrounding rock. Due to the fact that this study mainly

focuses on the differences in mechanical response among different excavation methods and their engineering applicability, it is reasonable to adopt the assumptions of an equivalent continuum and an equivalent initial stress field. Variations in the lateral pressure coefficient K_0 may affect the local stress concentration and deformation magnitude at locations such as the haunch and the invert. On this basis, $K_0 = 1$ was taken as the reference value in this study, and a subsequent parametric sensitivity analysis was conducted to further examine the influence of K_0 variation on the calculated results. The results show that changes in K_0 mainly affect the absolute values of surrounding rock response, but do not alter the relative variation characteristics among different excavation methods or the main conclusions of this study.

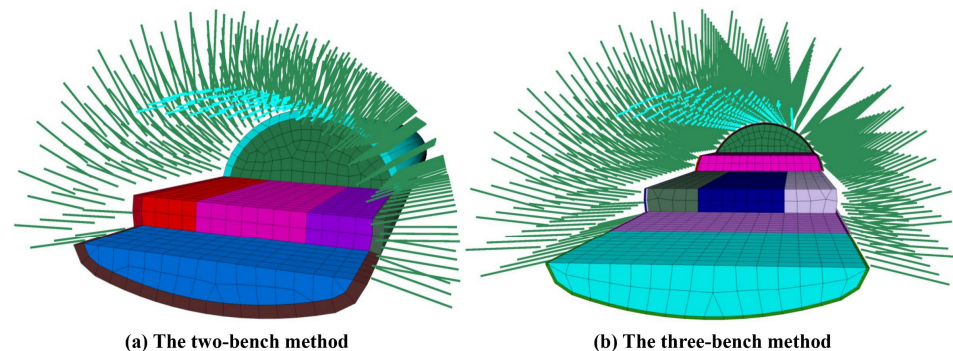


Figure 2. Schematic diagram of tunnel construction methods.

The boundary conditions are defined such that a vertical constraint is applied to the bottom of the model, and normal constraints are applied to the four lateral sides. The surrounding rock is simulated using solid elements with a Mohr–Coulomb constitutive model. Advance small pipes are modeled with beam elements, rock bolts with cable elements, and shotcrete and steel arches with shell elements. An equivalent elastic modulus is determined by converting the moduli of shotcrete and steel arch frame, and an elastic constitutive model is adopted. The secondary lining is simulated using solid elements.

The surrounding rock parameters were mainly determined based on the results of uniaxial compression, Brazilian splitting, and shear tests conducted on field-collected samples. Among these parameters, the elastic modulus and cohesion primarily reflect the differences in the degree of weakness of the surrounding rock under different rock mass grades. Considering that the surrounding rock in this project is overall dominated by mudstone and that the sandstone interlayers are relatively thin, the differences in Poisson's ratio and internal friction angle between grade IV and grade V surrounding rock are relatively limited. To highlight the dominant influence of changes in surrounding rock grade on the response of the excavation methods, the same values of Poisson's ratio and internal friction angle were adopted for both rock grades in the baseline cases, while the differences in mechanical properties were mainly represented by the elastic modulus and cohesion. The specific parameters are listed in Table 1. The material parameters of the advance small pipes, anchor cables, steel arch frame, and secondary lining are determined with reference to relevant engineering codes and literature. The equivalent elastic modulus and equivalent unit weight of the initial support are obtained through formula-based calculations. The specific parameters are listed in Table 2.

Table 1. Stratum simulation parameters.

Grade	Density/ $\text{kN}\cdot\text{m}^3$	Elastic/GPa	Cohesion/MPa	Friction/ $^\circ$	Poisson
IV	24	4.0	3.0	32	0.35
V	24	2.0	0.5	32	0.35

Table 2. Simulation parameters of tunnel support structure.

Structure	Type	Density/kN·m ³	Elastic/GPa	Poisson
Steel arch frame	/	78.5	200	0.3
Cable	Cable	78.5	200	0.3
Advance small pipe	Beam	78.5	200	0.3
Invert arch	Solid	26	30	0.2
Backfill concrete	Solid	26	30	0.2
Initial support	Shell	22	23	0.2
Secondary lining	Solid	26	30	0.2

According to Equation (1), the elastic moduli of the steel arch frame and shotcrete are converted to obtain the equivalent elastic modulus of the initial support, while the equivalent unit weight is calculated based on Equation (2).

$$E_e = E_c + \frac{E_g \times S_g}{S_c} \quad (1)$$

$$\gamma_e = \gamma_c + \frac{nS_g}{b}(\gamma_g - \gamma_c) \quad (2)$$

where E_e , E_c , and E_g are the equivalent elastic modulus of the initial support, the elastic modulus of the shotcrete, and the elastic modulus of the steel arch frame, respectively, in GPa; S_g and S_c are the cross-sectional areas of the steel arch frame and the shotcrete, respectively, in m²; γ_e , γ_c , and γ_g are the equivalent unit weight of the initial support, the unit weight of the shotcrete, and the unit weight of the steel arch frame, respectively, in kN·m^{−3}; n is the number of steel arch frames per unit length; and b is the shotcrete thickness, in m.

To assess the influence of key parameters on the numerical results, a sensitivity analysis was performed on the elastic modulus E , cohesion c , and lateral pressure coefficient K_0 . The maximum crown settlement, floor uplift, and horizontal convergence at the haunch were adopted as the evaluation indices. As shown in Tables 3 and 4.

For the two-bench excavation method under Grade IV surrounding rock conditions, when the elastic modulus fluctuates by $\pm 20\%$ relative to the reference value, the variations in the maximum crown settlement, floor uplift, and horizontal convergence at the haunch are 8.8–12.2%, 8.1–11.0%, and 7.8–9.8%, respectively. When the cohesion varies by $\pm 20\%$, the variation ranges of the three indices are 4.7–6.1%, 5.9–8.1%, and 4.9–6.9%, respectively. When the lateral pressure coefficient K_0 changes from 0.8 to 1.2, the variation ranges of the three indices are 4.7–6.1%, 3.7–5.1%, and 6.9–7.8%, respectively. It can be seen that, under Grade IV surrounding rock conditions, the elastic modulus has the most significant influence on the deformation magnitude, followed by cohesion, whereas K_0 is more sensitive to the horizontal convergence at the haunch.

For the three-bench excavation method under Grade V surrounding rock conditions, the deformation responses induced by parameter perturbations are generally greater than those under Grade IV surrounding rock conditions. When the elastic modulus varies by $\pm 20\%$, the variation ranges of the maximum crown settlement, invert heave, and horizontal convergence at the haunch reach 11.9–15.1%, 10.1–13.1%, and 9.8–12.1%, respectively. When the cohesion varies by $\pm 20\%$, the variation ranges of the three indices are 7.9–10.1%, 8.9–11.0%, and 6.8–9.1%, respectively. When K_0 changes from 0.8 to 1.2, the variation ranges of the three indices are 6.9–9.0%, 5.9–8.0%, and 9.8%, respectively. This indicates that, under weaker surrounding rock conditions, surrounding rock deformation is more sensitive to changes in stiffness parameters, strength parameters, and initial stress conditions. Among

them, the horizontal convergence at the haunch remains the most sensitive indicator for reflecting changes in K_0 .

Overall, within the parameter variation ranges considered in this study, changes in E , c , and K_0 all lead to adjustments in the absolute values of surrounding rock deformation, but do not alter the main findings of this study.

Table 3. Sensitivity analysis results of key parameters for Grade IV surrounding rock under the two-bench method.

Case	E/GPa	c/MPa	K_0	Crown Settlement/mm	Floor Uplift/mm	Horizontal Convergence/mm
1	4.0	3.0	1.0	14.8	13.6	10.2
2	3.2	3.0	1.0	16.6	15.1	11.2
3	4.8	3.0	1.0	13.5	12.5	9.4
4	4.0	2.4	1.0	15.7	14.7	10.9
5	4.0	3.6	1.0	14.1	12.8	9.7
6	4.0	3.0	0.8	14.1	13.1	9.5
7	4.0	3.0	1.2	15.7	14.3	11.0

Table 4. Sensitivity analysis results of key parameters for Grade V surrounding rock under the three-bench method.

Case	E/GPa	c/MPa	K_0	Crown Settlement/mm	Floor Uplift/mm	Horizontal Convergence/mm
1	2.0	0.5	1.0	37.8	23.7	26.4
2	1.6	0.5	1.0	43.5	26.8	29.6
3	2.4	0.5	1.0	33.3	21.3	23.8
4	2.0	0.4	1.0	41.6	26.3	28.8
5	2.0	0.6	1.0	34.8	21.6	24.6
6	2.0	0.5	0.8	35.2	22.3	23.8
7	2.0	0.5	1.2	41.2	25.6	29.0

3.2. Surface Settlement

The distribution of surface settlement on both sides of the tunnel centerline is shown in Figure 3. Under the two-bench method, the maximum surface settlement is 29.1 mm, whereas under the three-bench method, the maximum value increases to 47.2 mm. In both cases, the settlement trough is approximately symmetric about the tunnel centerline, with the maximum settlement occurring near the tunnel axis and gradually decreasing toward both sides. Compared with the two-bench method, the three-bench method produces a significantly larger settlement magnitude and a steeper settlement gradient near the tunnel centerline, indicating a stronger vertical disturbance to the ground surface.

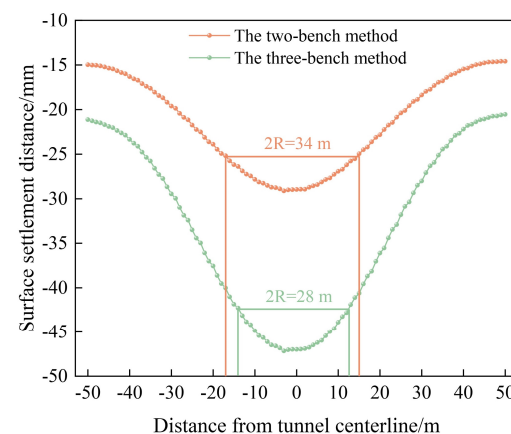


Figure 3. Surface settlement.

To further quantify the characteristics of the settlement trough, the single-side influence radius R is defined herein as the horizontal distance corresponding to the point where the absolute settlement attenuates to 10% of the maximum settlement, and the influence width is taken as $2R$. According to Figure 3, the single-side influence radii of the two-bench and three-bench methods are approximately 17 m and 14 m, corresponding to influence widths of 34 m and 28 m, respectively. This indicates that, under the present criterion, the lateral influence range of the two-bench method is slightly larger than that of the three-bench method.

In addition, the settlement troughs were characterized using the Gaussian expression:

$$S(x) = S_{\max} e^{\frac{-x^2}{2i^2}} \quad (3)$$

where $S_{\max}$ is the maximum surface settlement, i is the settlement trough width parameter, and x is the horizontal distance from the tunnel centerline.

The width coefficient is expressed as follows:

$$K = \frac{i}{z_0} \quad (4)$$

where z_0 is the tunnel burial depth, which is taken as 324 m in this study.

Based on the above calculation, the width coefficients K for the two-bench method and the three-bench method are 0.095 and 0.094, respectively. This indicates that the difference in settlement trough width between the two excavation methods is relatively small, and that their main distinction lies in the magnitude of the settlement response rather than in the overall extent of the settlement trough. This reflects that the three-bench method tends to induce a deeper but more concentrated settlement trough, whereas the two-bench method produces a relatively gentler but slightly wider surface response. The ground settlements induced by both construction methods remain within the safe limits for construction, indicating that, in grade IV and V surrounding rocks composed of sandstone–mudstone strata, the settlement caused by the benching method does not pose a construction risk.

3.3. Analysis of Tunnel Surrounding Rock Deformation

As shown in the figure, the complete curve represents the continuous influence of the model, from the first excavation to the final excavation, on the crown settlement, floor uplift, and horizontal convergence at the monitoring section. However, this does not correspond to actual monitoring conditions, because field monitoring can only commence after excavation reaches the monitoring section and the instruments are installed (here, “one time” is equivalent to “one day” in practice). When excavation reaches the 20th time, the upper benches of both construction methods have arrived at the monitoring section. Therefore, the true crown settlement, floor uplift, and horizontal convergence after the 20th excavation time should be equal to the numerical simulation values minus those at the 20th time.

The crown settlements induced by the two-bench and three-bench methods tend to stabilize 15 and 21 days, respectively, after the upper bench excavation passes the monitoring section, with stable values of 14.8 mm and 37.8 mm (Figure 4). The floor uplift induced by the two-bench and three-bench methods tend to stabilize at 17 and 25 days, respectively, after the upper bench excavation passes the monitoring section, with stable values of approximately 13.6 mm and 23.7 mm (Figure 5). Because grade IV surrounding rock has a higher strength than grade V rock, the support ring is formed earlier under the two-bench method; consequently, the crown settlement and floor uplift stabilize earlier and exhibit smaller stable values. In contrast, grade V surrounding rock is weaker and more

plastic, leading to a longer deformation process; therefore, under the three-bench method, the stable values of crown settlement and floor uplift are larger, and the time required to reach the stable stage is longer.

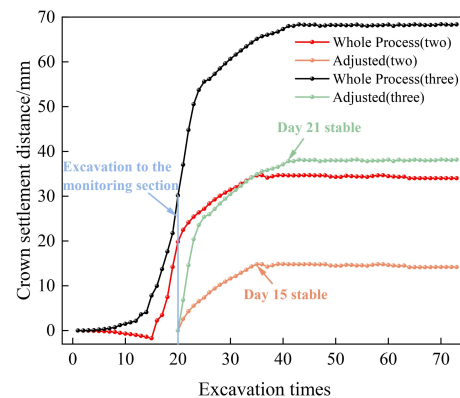


Figure 4. Crown settlement distance.

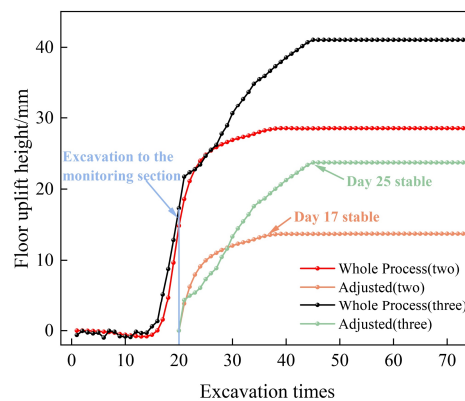


Figure 5. Floor uplift height.

Under the two-bench method, horizontal convergence increases relatively slowly during the first 5 days after upper-bench excavation, and then rises rapidly after lower-bench excavation. This is because excavation of the lower bench weakens the confinement provided by the surrounding rock, leading to a pronounced increase in deformation. Ultimately, the horizontal convergence under the two-bench method tends to stabilize on day 18, with a stable value of approximately 10.2 mm. Under the three-bench method, horizontal convergence increases slowly after upper-bench excavation, but the growth rate accelerates following middle-bench excavation. After lower-bench excavation, the convergence increases rapidly again and eventually stabilizes on day 24, with a stable value of approximately 26.4 mm (Figure 6).

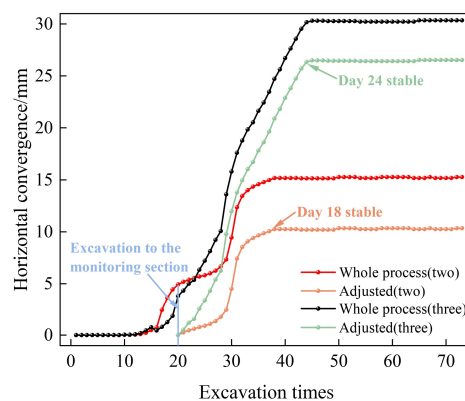


Figure 6. Horizontal convergence.

When the bench excavation method is applied in sandstone–mudstone strata, the two excavation methods exhibit somewhat different deformation characteristics. For the two-bench method, crown settlement is the largest, floor uplift is the second largest, and horizontal convergence at the haunch is the smallest. For the three-bench method, crown settlement remains the largest, horizontal convergence at the haunch is the second largest, and floor uplift is the smallest. Overall, neither excavation method caused significant deformation of the surrounding rock, and the overall construction risk remained within a controllable range.

3.4. Tunnel Face Extrusion Displacement

As shown in Figure 7, under the two-bench method, the maximum extrusion displacement occurs at the center of the upper bench, reaching approximately 18.7 mm, whereas the maximum extrusion displacement of the lower bench is 7.4 mm. The deformation of the tunnel face is symmetric about the tunnel centerline and expands outward in a ring-like pattern, exhibiting a pronounced symmetric deformation characteristic. Under the three-bench method, the maximum extrusion displacement occurs at the center of the middle bench, with a value of 68.4 mm, whereas the maximum extrusion displacements of the upper and lower benches are 65.7 mm and 65.2 mm, respectively. The deformation pattern of the three-bench method is relatively smooth, indicating a more uniform deformation distribution.

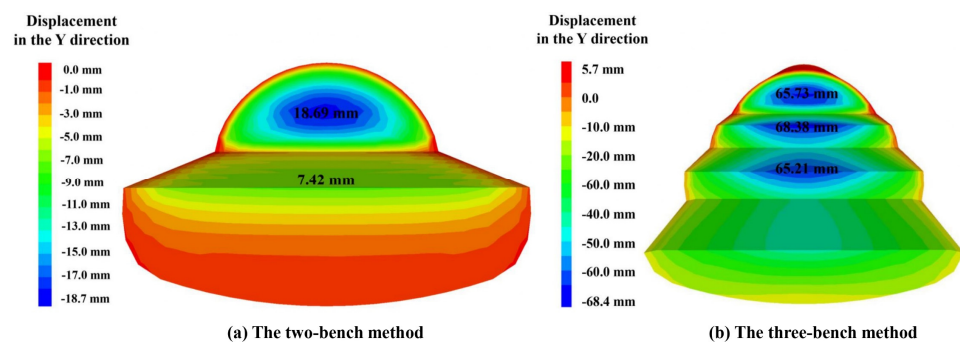


Figure 7. Extrusion displacement contour maps of the three construction methods.

3.5. Mechanical Interpretation of the Self-Supporting Capacity of Surrounding Rock Under the Bench Method

It should be noted that the self-supporting capacity of surrounding rock does not simply refer to the material strength of the rock mass itself, but rather to the ability of the surrounding rock to form a relatively stable load-bearing structure after excavation through stress redistribution, transverse arching, and the spatial confinement effect of the tunnel face. Therefore, the advantage of bench excavation does not merely come from reducing the excavation area of a single step, but more essentially from changing the internal mechanical response and load transfer path of the surrounding rock during excavation. By dividing the excavation process into stages, abrupt stress release and rapid expansion of the plastic zone can be alleviated, while earlier installation of the primary support becomes possible, which promotes coordinated rock–support interaction under a lower disturbance level.

From a more comprehensive perspective on surrounding rock control mechanisms, the mobilization of the self-supporting capacity of surrounding rock depends not only on a rational excavation sequence, but also on the improvement of structural integrity and load transfer capacity through measures such as anchorage and grouting. Recent studies have shown that reamed inverted wedge-shaped anchorage can significantly increase the initial anchoring force while maintaining a relatively high residual bearing capacity [24]. Meanwhile, grouting pressure and fracture conditions directly affect the slurry diffusion

radius and the reinforcement range [25]. In essence, both approaches optimize the load transfer path and the overall bearing capacity of the surrounding rock–support system.

4. Comparative Analysis of Field Monitoring Data

4.1. The Construction Process and On-Site Monitoring Plan of the Two-Bench and Three-Bench Methods

To ensure structural safety and surrounding rock stability during tunnel construction in sandstone–mudstone strata using the two-bench and three-bench methods, two monitoring sections were arranged for each tunnel. Surrounding rock deformation, surrounding rock pressure, and bolt axial force were monitored using displacement sensors, pressure cells, and rock bolt dynamometers, respectively. The measurement accuracy was $\pm 0.1\%$ F.S. for the displacement sensors, $\pm 0.1\%$ F.S. for the pressure cells, and $\pm 0.5\%$ F.S. for the rock bolt dynamometers. To improve the accuracy and reliability of the monitoring results, regular instrument inspections and zero-drift checks were carried out during the monitoring process.

The monitoring instruments at the crown and spandrel were installed promptly after the completion of upper-bench excavation and the installation of the initial support. The installation timing of the monitoring instruments at the haunch varied depending on the excavation method: Under the two-bench method, the instruments were installed after the completion of lower-bench excavation, that is, approximately 10 days after upper-bench excavation. Under the three-bench method, the instruments were installed after the completion of middle-bench excavation, that is, approximately 3 days after upper-bench excavation. After the instruments had been installed and their readings had stabilized, the initial monitoring values were recorded promptly, and subsequent data were collected continuously at the prescribed monitoring frequency.

Field monitoring data inevitably involve a certain degree of uncertainty during acquisition. The main influencing factors include deviations in sensor installation position, the time lag between excavation and monitoring setup, construction-induced disturbance, and insufficient contact between the sensors and the surrounding medium. These factors may cause local scatter in the monitoring results and lead to some discrepancies between the monitored and simulated results. However, through instrument calibration, standardized installation, and full-process data verification, the monitoring results still show a high overall level of reliability and can effectively characterize the evolution of surrounding rock deformation and stress. The layout of the monitoring points is shown in Figure 8.

In this study, the monitored rock bolts were 6 m long, whereas those installed at unmonitored locations were 3 m long. All rock bolts were arranged radially along the tunnel excavation profile, with a circumferential spacing of 2.4 m. All rock bolts were installed using a full-length grouting method. Cement mortar was used as the grouting material, and the water–cement ratio was controlled within the range of 0.4–0.5. For the monitored rock bolts, axial force testing was conducted using a two-point measurement scheme, with axial force sensors installed at 1 m and 5 m from one end of the bolt to characterize the mechanical responses of the shallow and deep anchorage sections, respectively.

4.2. Analysis of Tunnel Crown Settlement and Tunnel Floor Uplift Under the Benching Method

The time-dependent evolution of crown settlement and floor uplift under the two-bench and three-bench methods is shown in Figure 9, and both generally exhibit a deformation pattern characterized by “rapid increase in the early stage–slower growth in the middle stage–gradual stabilization in the late stage”. Under the three-bench method, crown settlement increases continuously from day 1 to day 24 and tends to stabilize on day 24, with a stable value of approximately 26.3 mm. The floor uplift also develops continuously

from day 1 to day 22 and stabilizes on day 22, with a stable value of about 20.3 mm. After entering the stable stage, the curves exhibit fluctuations within a certain range, indicating that the surrounding rock–support system has achieved overall convergence during the process of stress redistribution and structural adjustment. In contrast, the surrounding rock deformation under the two-bench method converges more readily and stabilizes earlier. Specifically, crown settlement increases continuously from day 1 to day 17 and tends to stabilize on day 17, with a stable value of approximately 14.0 mm; floor uplift develops from day 1 to day 15 and stabilizes on day 15, with a stable value of about 10.3 mm. In the stable stage, the curves are smoother with smaller oscillation amplitudes, indicating that, in grade IV surrounding rock within sandstone–mudstone strata, the two-bench method performs well in controlling unloading effects of the surrounding rock and promoting a stable stress state in the surrounding rock–support system.

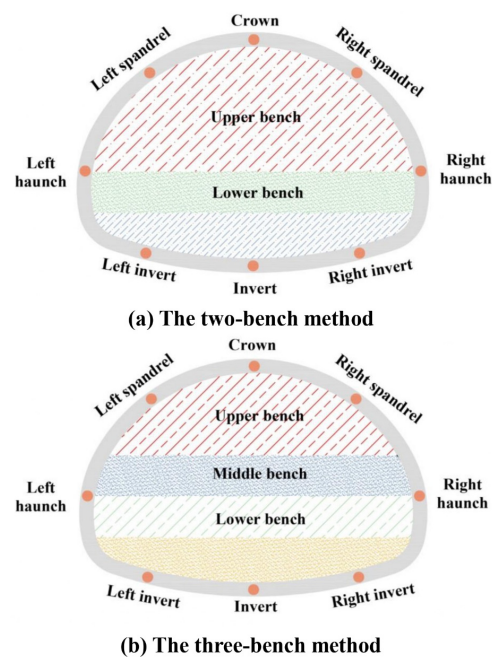


Figure 8. Schematic diagram of monitoring point layout.

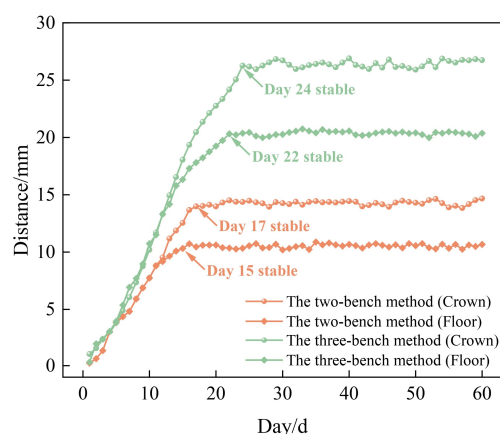


Figure 9. Analysis of tunnel crown settlement and tunnel floor uplift changes under the benching method.

In sandstone–mudstone strata, grade IV and grade V surrounding rocks are constructed using the two-bench method and the three-bench method, respectively. The results indicate that the crown settlement and floor uplift under the three-bench method are slightly greater than those under the two-bench method, and that the stabilization time lags by

about 7 days; although certain fluctuations occur during the stable stage, all deformations remain within the safe range for construction. Compared with the numerical simulation results, the benching method delays the time for crown settlement and floor uplift to reach stability by 2–3 days, but the stable values are essentially the same. This indicates that, under weak surrounding rock conditions such as sandstone–mudstone, the use of the benching method can effectively reduce the deformation of the surrounding rock at the tunnel crown and floor, thereby enhancing surrounding rock stability during construction.

4.3. Analysis of Horizontal Convergence of Surrounding Rock at the Tunnel Haunch Under the Benching Method

As shown in Figure 10, under the two-bench method, horizontal convergence at the haunch increases rapidly from day 1 to day 17, reaches a maximum value of 23.1 mm on day 22, then gradually decreases, and tends to stabilize on day 25, with a stable value of approximately 13.2 mm. In contrast, under the three-bench method, horizontal convergence increases slowly from day 1 to day 5, then rapidly from day 6 to day 19, reaches a maximum value of 14.1 mm on day 25, and subsequently decreases gradually, tending to stabilize on day 32 with a stable value of approximately 20.8 mm. Under grade V surrounding rock conditions, the stabilized horizontal convergence of the three-bench method is slightly greater than that of the two-bench method (grade IV), with a difference in only 7.6 mm, and the maximum convergence of the three-bench method is also only slightly larger than that of the two-bench method. The stabilization time of horizontal convergence with the benching method is mainly concentrated between 20 and 32 days.

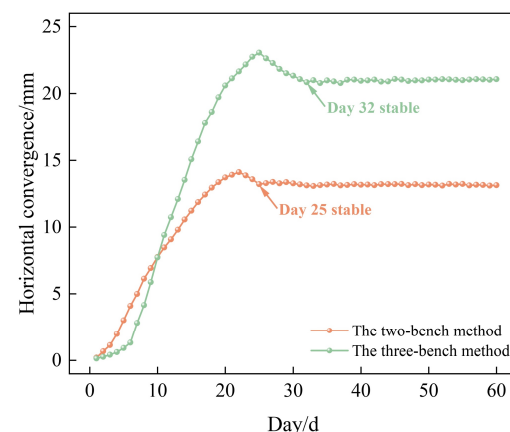


Figure 10. Analysis of horizontal convergence changes at the tunnel haunch under the bench excavation method.

Compared with the numerical simulation results for horizontal convergence, the benching method delays the time for horizontal convergence to reach stability by 7 days, but the stable values are essentially the same. This indicates that the benching method provides effective control of the surrounding rock at the haunch and can significantly reduce deformation in this region of the tunnel. In the early stage of the three-bench method, the curve exhibits a concave shape because excavation of the middle bench has been completed while the lower bench has not yet been excavated, so the surrounding rock at the haunch remains partially constrained, resulting in a relatively slow increase in horizontal convergence. As excavation of the lower bench proceeds and the confinement of the surrounding rock is released, horizontal convergence begins to increase rapidly.

Based on the stabilized monitoring values of crown settlement, floor uplift, and horizontal convergence at the haunch, it can be seen that under both excavation methods, crown settlement is generally the largest, horizontal convergence at the haunch is the second

largest, and floor uplift is the smallest. However, the differences among the deformation indices vary somewhat between the two construction methods. For the two-bench method, the values of crown settlement and horizontal convergence at the haunch are relatively close. For the three-bench method, however, the difference between horizontal convergence at the haunch and floor uplift is relatively small.

4.4. Analysis of the Evolution of Surrounding Rock Pressure at Different Tunnel Locations Under the Benching Method

Figure 11 (left) and Figure 11 (right) show the time-dependent evolution of surrounding rock pressure at each monitoring point under the two-bench and three-bench methods, respectively, and the surrounding rock pressure at all monitoring points exhibits a pronounced attenuation trend. In Figure 11 (left), day 0 is taken as the onset of upper-bench excavation, and day 10 corresponds to the excavation of the lower bench. Under the two-bench method, the initial surrounding rock pressures at the crown, left spandrel, right spandrel, left haunch, and right haunch are 21.6 MPa, 19.8 MPa, 20.5 MPa, 65.7 MPa, and 75.3 MPa, respectively. The surrounding rock pressures at the crown and the left and right spandrel decay rapidly over 1–9 days, then enter a phase of gradual decline from days 9–20, and tend to stabilize on day 20, with stable values of approximately 1–2 MPa. In contrast, the surrounding rock pressures at the left and right haunch exhibit a “rapid-to-gradual” decreasing trend over days 11–42 (i.e., within 32 days after lower-bench excavation) and reach stability on day 42, with stable values of approximately 2–5 MPa. In Figure 11 (right), day 0 is taken as the onset of upper-bench excavation, day 3 corresponds to middle-bench excavation, and day 6 corresponds to lower-bench excavation. Under the three-bench method, the initial surrounding rock pressures at the crown, left spandrel, right spandrel, left haunch, and right haunch are 31.6 MPa, 30.8 MPa, 30.5 MPa, 85.3 MPa, and 87.7 MPa, respectively. The surrounding rock pressures at the crown and the left and right spandrel decay rapidly over 1–14 days, then enter a phase of gradual attenuation from days 14–26, and tend to stabilize on day 26, with stable values of approximately 3–4 MPa. In contrast, the surrounding rock pressures at the left and right haunch exhibit a pronounced stage-wise evolution: they decay slowly over days 4–6, decrease rapidly over days 7–12, and then attenuate slowly again from days 12–40, tending to stabilize on day 40 with stable values of approximately 8–9 MPa. This is because monitoring begins immediately after instrument installation following middle-bench excavation, and the confinement of the rock mass remains strong before the lower bench is excavated, thereby limiting stress release in the surrounding rock. Once the lower bench is excavated on day 6, the confinement is weakened and the surrounding rock pressure at the haunch continues to be released, resulting in an attenuation pattern of “slow–fast–slow,” with a relatively long stress-release duration.

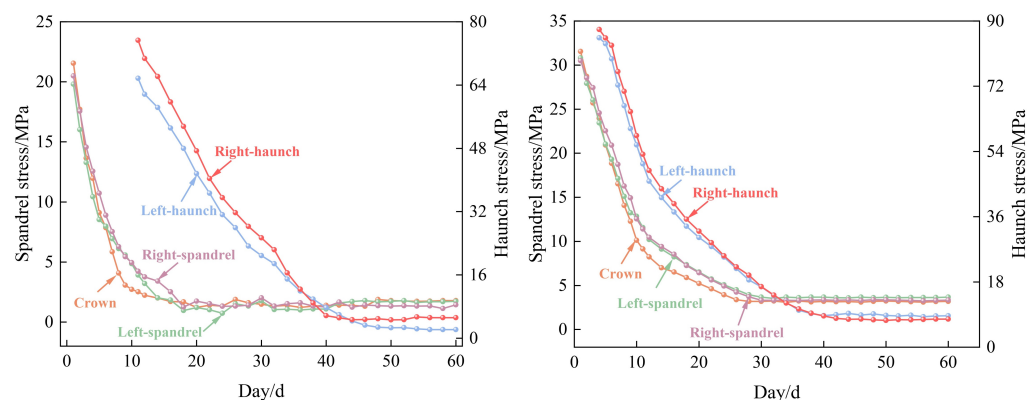


Figure 11. Time-dependent evolution of surrounding rock pressure at different tunnel locations: **(left)** the two-bench method; **(right)** the three-bench method.

The initial surrounding rock stresses at the crown and spandrel are generally at relatively low levels, and the residual stresses after entering the stable stage are basically maintained at 1–4 MPa, indicating that the initial support system has good load-bearing and force-transmission capacities and that the stress–deformation response at the tunnel crown is well coordinated. The left and right haunch exhibit high initial stress levels, a long stress-release duration, and a large attenuation magnitude, indicating that this region is readily affected by stress redistribution under excavation disturbance and is prone to in situ stress concentration. It should therefore be treated as a key zone for protection and monitoring control during both tunnel construction and operation.

4.5. Analysis of the Evolution of Bolt Axial Force at Different Tunnel Locations Under Different Construction Methods

Figure 12 (left) and Figure 12 (right) present the monitored axial forces of anchor bolts at the spandrel and haunch under the two-bench and three-bench methods, respectively. Under the two-bench method, the axial force of anchor bolts at the haunch exhibits a “rapid-to-gradual” evolution trend and tends to stabilize on day 42. Because monitoring at the haunch begins only after lower-bench excavation, a 10-day lag is observed in the figure; taking the onset of haunch monitoring as the reference, 32 days are required to reach stability. After entering the stable stage, the axial forces of the deep anchor bolts at the haunch stabilize at 96.3 kN and 90.3 kN, respectively, while those of the shallow anchor bolts stabilize at 69.2 kN and 62.3 kN. In contrast, the axial forces of the spandrel anchor bolts increase gradually and stabilize on day 30, with stable axial forces of 53.0 kN and 55.1 kN for the deep anchor bolts and 32.0 kN and 32.0 kN for the shallow anchor bolts, respectively. Under the three-bench method, the axial force of anchor bolts at the haunch exhibits a stage-wise evolution characterized by “slow–fast–slow” and tends to stabilize on day 46; taking the onset of haunch monitoring as the reference, 42 days are required to reach stability. After entering the stable stage, the axial forces of the deep anchor bolts at the haunch stabilize at 119.1 kN and 109.1 kN, respectively, while those of the shallow anchor bolts stabilize at 93.3 kN and 89.1 kN. In contrast, the axial forces of the spandrel anchor bolts increase gradually and stabilize on day 40, with stable deep-bolt axial forces of 76.1 kN and 62.5 kN and stable shallow-bolt axial forces of 47.1 kN and 43.1 kN, respectively.

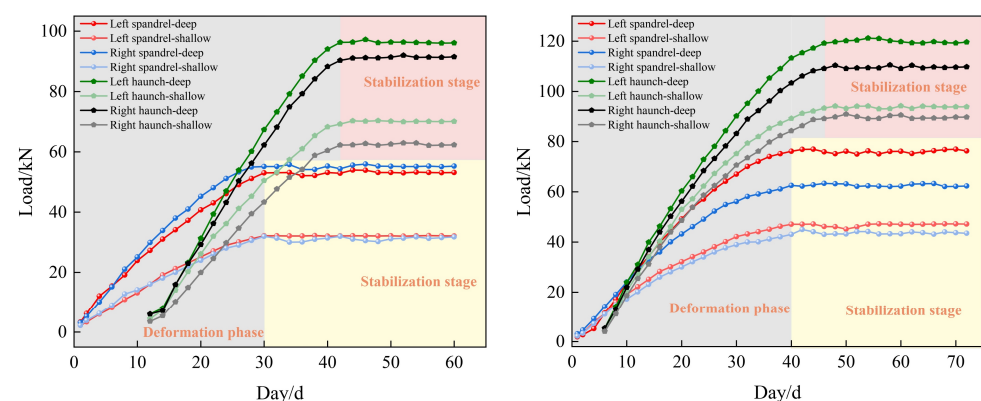


Figure 12. Time-dependent evolution of anchor-bolt axial force at the tunnel spandrel and haunch: (left) the two-bench method; (right) the three-bench method.

A comparison of the two construction methods indicates that the axial forces of anchor bolts at the haunch are generally significantly higher than those at the spandrel, suggesting that the support structure at the haunch carries greater loads and experiences more concentrated stresses. Further analysis of the time-dependent evolution of haunch anchor

bolt axial force reveals clear differences between the two methods, mainly governed by the installation time of the load cells and the differing confinement conditions of the lower rock mass. In the two-bench method, the load cells are installed after lower-bench excavation, when the confinement of the lower rock mass is markedly weakened. Consequently, deformation and stress release of the surrounding rock at the haunch become more pronounced, leading to a rapid increase in anchor bolt axial force. As the surrounding rock–support system gradually approaches equilibrium, the growth rate of axial force decreases and then tends to stabilize. Under the three-bench method, monitoring begins immediately after middle-bench excavation; however, the lower bench has not yet been excavated, and the lower rock mass still provides strong confinement, causing the axial force to increase slowly at first. After completion of lower-bench excavation, confinement is further weakened and stress redistribution intensifies, leading to a rapid increase in axial force; subsequently, as the surrounding rock–support system approaches equilibrium, the growth slows again and reaches a stable state.

5. Discussion

5.1. Novelty and Distinction from Previous Studies

- (1) This study establishes the correspondence between bench excavation methods and surrounding rock grades in sandstone–mudstone strata, namely that the two-bench method is suitable for Grade IV surrounding rock, whereas the three-bench method is more appropriate for Grade V surrounding rock. This conclusion advances previous research from general comparisons of construction methods to a systematic understanding of the matching relationship between surrounding rock grade and excavation method under weak layered rock conditions, thereby further strengthening the theoretical basis for selecting bench excavation methods.
- (2) The field validation in this study was not limited to displacement response analysis. In addition to examining crown settlement, floor uplift, and horizontal convergence, the study further incorporated the time-dependent evolution of surrounding rock pressure and rock bolt axial force. This makes it possible to more comprehensively reveal the interaction mechanism between the surrounding rock and the support structure during staged excavation. It can be used not only to determine the applicability of different bench excavation methods, but also to clarify the internal load transfer path and its evolution during construction.
- (3) This study explicitly considered the inconsistency between the excavation stages in the numerical simulation and the actual start time of field monitoring, which was caused by instrument installation conditions. By calibrating the numerical results against the actual monitoring start point, the comparison between the simulation results and the field monitoring data was made more realistic and methodologically rigorous. Meanwhile, the haunch was identified as the key control zone during construction, characterized by relatively delayed stress release, relatively high residual surrounding rock pressure, and relatively large rock bolt axial force. These characteristics have not been adequately emphasized in previous studies on bench excavation methods involving numerical simulation and field validation. Therefore, the contribution of this study lies not only in validating the applicability of bench excavation methods, but also in revealing the key mechanisms of the surrounding rock–support interaction and providing a basis for the focused arrangement of engineering monitoring and support optimization.

5.2. Engineering Guidance

For large-section highway tunnels in sandstone–mudstone formations similar to those in this project, the selection of the construction method should be based on a comprehensive evaluation of factors such as surrounding rock class, deformation stabilization time, face extrusion response, characteristics of surrounding rock pressure release, and the efficiency of mechanized construction, rather than on a single deformation index alone. Overall, under Grade IV surrounding rock conditions, the two-bench method offers the advantages of a simpler construction sequence and higher mechanized construction efficiency, provided that surrounding rock stability can be maintained and the support system can be closed in a timely manner. Under Grade V surrounding rock conditions, the three-bench method is more suitable for controlling the excavation exposure range and mitigating the effects of surrounding rock unloading; however, it requires simultaneous strengthening of face restraint, haunch monitoring, and staged support regulation.

Based on the monitoring and numerical analysis results, a hierarchical monitoring system covering “ground surface–in-tunnel deformation–critical load-bearing locations” should be established for construction monitoring in similar projects. The ground settlement monitoring range should extend at least 14–17 m on both sides of the tunnel axis. Within the tunnel, monitoring should focus on the crown, invert, and haunches, with the haunches designated as priority zones for intensified monitoring. Under the two-bench construction method, the main stage controlling surrounding rock deformation generally occurs approximately 20–32 days after excavation; under the three-bench method, this stage is prolonged to about 26–37 days. Considering that the stabilization of surrounding rock pressure and rock-bolt axial force at the haunch shows a certain time lag, the corresponding monitoring should be continued until at least about 43–45 days after excavation so as to ensure effective identification of delayed stress adjustment.

When the monitoring results indicate that the surrounding rock pressure at the haunch remains consistently higher than that at the crown and the arch shoulder, and that the rock-bolt axial force stays at a relatively high level over a prolonged period, the haunch should be identified as a priority zone for support optimization. Priority should be given to measures such as locally increasing the rock-bolt density, enhance the composite load-bearing action of shotcrete and steel ribs, strictly controlling the bench spacing, and accelerating the closure of the initial support system. For tunnels excavated by the three-bench method in Grade V surrounding rock, face restraint should be strengthened particularly during the transition from the middle bench to the lower bench. When necessary, auxiliary measures such as advanced small pipes, local grouting, and a temporary invert should also be adopted to effectively control the further development of haunch convergence and face extrusion deformation.

5.3. Limitations and Applicability

The engineering case considered in this study is a large-section tunnel excavated in an interbedded sandstone–mudstone formation dominated by mudstone and containing thin sandstone layers. The tunnel has a burial depth of approximately 320–328 m, the surrounding rock is classified as Grade IV–V, and the support parameters were determined based on site conditions. The numerical analysis was conducted based on the equivalent continuum assumption and under a simplified initial stress condition ($K_0 = 1$). Complex geological factors, such as bedding-plane slip, fault structures, groundwater seepage, and strongly anisotropic stress fields, were not explicitly considered. Therefore, the conclusions obtained in this study are primarily applicable to large-section tunnels excavated in weakly cemented, layered soft rock formations dominated by mudstone or sandstone–mudstone interbeds, particularly in engineering scenarios where both deformation control of the

surrounding rock and the efficiency of mechanized construction must be taken into account. In complex geological settings, such as thick hard sandstone strata, fault fracture zones, strong groundwater influence, pronounced tectonic stress anisotropy, or squeezing strata conditions, the deformation mechanisms of the surrounding rock and the characteristics of rock–support interaction may differ significantly from those reported in this study. Under such conditions, the applicability of the two-bench method and the three-bench method should still be further evaluated in conjunction with specific engineering projects through targeted numerical simulations, field monitoring, and optimization of support parameters, so as to verify the applicability limits and extension conditions of the conclusions presented in this study.

6. Conclusions

- (1) Under grade IV and grade V surrounding rock conditions, crown settlement stabilizes at 14.0 mm on day 17 and at 26.3 mm on day 24, respectively; floor uplift stabilizes at 10.3 mm on day 15 and at 20.3 mm on day 22, respectively; and horizontal convergence stabilizes at 13.2 mm on day 25 and at 20.8 mm on day 32, respectively. These results agree well with the numerical simulations, indicating that the proposed numerical model can reasonably reproduce the deformation evolution of the surrounding rock under the benching method. All monitored deformation values remain within the allowable range for safe construction, demonstrating that the benching method can effectively control tunnel deformation in sandstone–mudstone strata under matched surrounding rock conditions.
- (2) Under the two-bench and three-bench methods, the surrounding rock pressures at the tunnel crown and spandrel require 20 and 26 days, respectively, to decrease to 1–4 MPa, whereas those at the haunch require 32 and 37 days, respectively, to decrease to 2–5 MPa and 8–9 MPa. The surrounding rock stress along the cross-section exhibits a temporal sequence in which the crown and spandrel stabilize first, followed by the haunch; moreover, the stabilized stress level at the haunch remains relatively high, indicating that this location is strongly influenced by construction disturbance and stress redistribution and should therefore be treated as a key zone for monitoring and enhanced support.
- (3) Under the two-bench method, the axial forces of anchor bolts at the tunnel spandrel and haunch tend to stabilize on day 30 and day 32, with stable values of 43–55 kN and 90–97 kN, respectively. Under the three-bench method, the anchor bolt axial forces at the spandrel and haunch stabilize on day 40 and day 42, reaching 89–94 kN and 109–120 kN, respectively. The anchor bolt axial force at the haunch is generally higher than that at the spandrel, indicating that, when the benching method is used in sandstone–mudstone strata, the load-bearing role of the haunch is pronounced and should be treated as a key control zone for support and monitoring.

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