

Article

A Field Study to Measure the Surrounding Stress of Rock and Supporting Structure of a Steep Tunnel with a Combination of Hard and Soft Rock Layers under Plate Compression

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Abstract: Tunnels excavated in a combination of hard and soft rock strata with high ground stress are prone to large deformations, collapse, and other disasters. The Yongfeng Tunnel, a reconstruction and expansion of the G544 line, suffered severe high ground stress from plate compression. This paper studied the surrounding rock pressure and supporting structure stress characteristics of tunnels with a combination of hard and soft rock strata with high ground stress by using earth pressure cells, surface strain gauges, and embedded strain gauges to test all stress related to the surrounding rock, primary support, and secondary lining. It was found that the contact pressure (P_1) between the initial support and the surrounding rock and the contact pressure (P_2) between the initial support of the leading tunnel were distributed in the direction of vertical stratification, while the contact pressures (P_1 and P_2) of the lagging tunnel were different due to the excavation unloading of the leading tunnel. The maximum stress positions of the initial support of the leading tunnel and the lagging tunnel were located in the left arch waist and the vault, respectively. However, the maximum stress position of the secondary lining was generally located on the side wall. The research results presented herein can guide future tunnel construction projects.

Keywords: tunnel engineering; combination of hard and soft rock strata; high ground stress; field monitoring; surrounding rock pressure; lining stress characteristics; spatial stress distribution; earth pressure cells; surface strain gauges



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1. Introduction

With the rapid development of China's transportation facilities, more tunnels are being constructed or planned [1]. However, topographic cutting and plate tectonic compression have induced high ground stress at the construction sites in Southwest China [2,3]. The effects of high ground stress must therefore be considered and overcome when building tunnels in the southwest. In addition, the layered surrounding rocks are characterized by laminations and deformation anisotropy, which render the tunnels traversing this stratigraphic environment highly susceptible to large deformation hazards. The main disasters include primary support cracking [4,5], arch distortion [6,7], and collapse [8,9], all of which significantly threaten the safety of construction personnel and mechanical equipment. Therefore, it is necessary to better understand the stress characteristics of tunnel support in areas comprising layered surrounding rock with high ground stress.

Numerous researchers have studied tunnels' deformation and mechanical properties in layered surrounding rock with high ground stress, mainly using model tests [10–16] and numerical simulations [17–21]. These methods are effective for investigating tunnels in layered surrounding rock because they have high repeatability, are low cost, and are less time-consuming [22]. However, they are also affected by various factors, such as parameter

selection, scale, and test conditions [23,24]. Thus, matching between the research results and the construction site is difficult to guarantee.

In contrast, field tests represent an effective strategy for obtaining reliable data [24]. Guo et al. [25] reported that the deformation of the surrounding rock in a carbonaceous slate tunnel was asymmetric, and the stress of the steel frame was more prominent in the upper part than in the lower part through field measurements. Wu et al. [26] used a field test approach to investigate how the mechanical characteristics of supporting structures and the surrounding rock deformed in response to different influencing factors. Chen et al. [27] demonstrated that the supporting structure of the layered surrounding rock tunnel experienced significant axial force and bending moment, respectively, in the direction parallel to bedding and perpendicular to bedding. Guo et al. [28] confirmed that a two-step method incorporating an inverted arch can effectively mitigate the deformation of steeply inclined layered surrounding rock tunnels based on field test methods. Wu et al. [5] found that the distribution of surrounding rock pressure in phyllite surrounding rock tunnels was uneven, and the primary support and secondary lining were mainly subjected to compressive stress.

Although previous studies have elucidated important details about layered surrounding rock tunnels, these studies mainly focused on single-layered soft rock or hard rock [29]. Compared with a rock stratum comprising a single type of rock, the soft and hard interbedded surrounding rock not only has the characteristics of transverse isotropy of layered rock mass in deformation but also has the characteristics of anisotropy in strength [30]. Therefore, the mechanical response of soft and hard interbedded tunnels is more complex in high ground stress environments. Existing research is generally insufficient to guide engineering practice.

Field tests have mainly focused on layered surrounding rock tunnels in a natural in situ stress field, while the influence of the additional secondary stress field on the experimental results was often ignored. In fact, the construction of two separate or small-spacing tunnels is often inconsistent. The tunnel associated with faster construction progress is typically equivalent to excavation in a natural in situ stress field. However, the tunnel with relatively slow construction progress is equivalent to excavation in the secondary stress field generated under the influence of the tunnel with fast construction progress. To date, the stress characteristics of tunnel lining in the secondary stress field have often been overlooked, particularly for a separated tunnel.

To fill the current knowledge gaps in the field, this study took the Yongfeng Tunnel of the G544 line in the Sichuan Province of China as the engineering context. Field measuring equipment including earth pressure cells, surface strain gauges, embedded strain gauges, and comprehensive testers were used to systematically investigate (i) the contact pressure between the initial support and the surrounding rock, (ii) the stress of the steel frame, (iii) the contact pressure between the secondary lining and the initial support, and (iv) the secondary lining concrete stress of the soft–hard interbedded tunnel with high ground stress. Based on the findings, this paper proposed several recommendations to guide engineering practice.

2. Engineering Background

2.1. Project Overview

The Yongfeng Tunnel of the G544 reconstruction and expansion project is located on the northwest edge of the Sichuan Basin and the eastern edge of the Qinghai–Tibet Plateau, as illustrated in Figure 1. The tunnel is a separated two-hole two-lane highway tunnel with a clear distance of 24–38 m. The starting and ending mileage piles of the left and right tunnels are ZK6 + 545 ~ ZK9 + 465 and K6 + 507 ~ K9 + 495, respectively. The lengths of the right and left tunnels are 2988 and 2920 m, respectively. The tunnel's maximum buried depth is 513 m. The tunnel's surrounding rock is primarily interbedded slate and phyllite, with trace amounts of argillaceous shale and dolomite. The thickness of soft rock (phyllite and argillaceous shale) and hard rock (sandy slate and dolomite) is between 1 and 10 cm

and 2 and 20 cm, respectively. The dip angle of these strata is between 55° and 60° . It has the characteristics of bedding development and strong anisotropy, as shown in Figure 2. The average saturated compressive strength of soft rock (phyllite) is 5.06 MPa, indicating its poor strength.

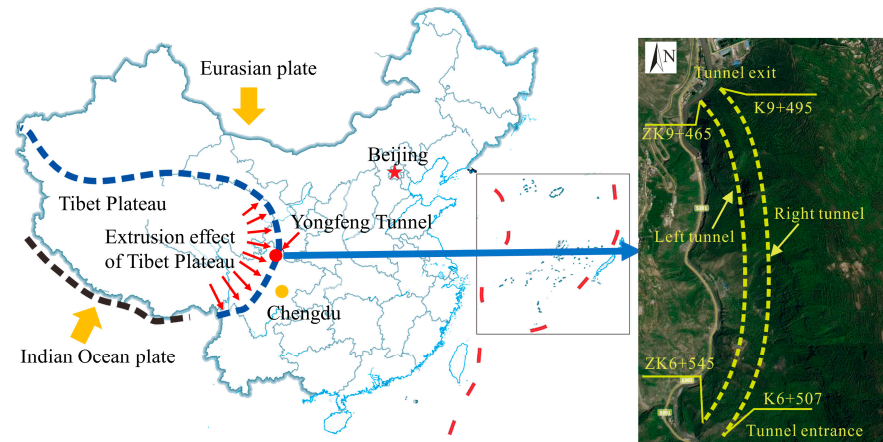


Figure 1. Location of the Yongfeng Tunnel.

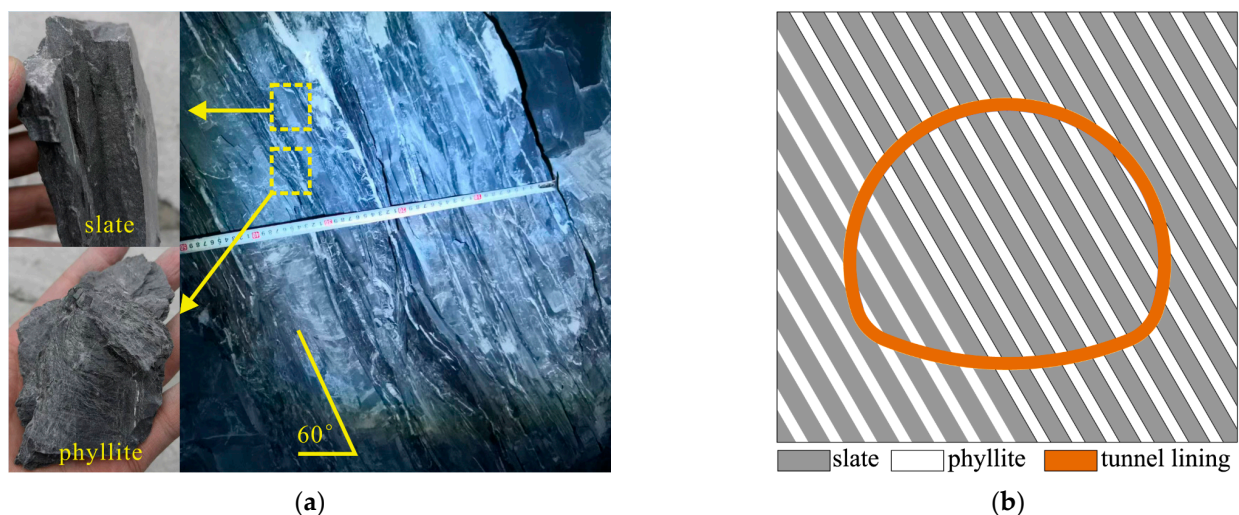


Figure 2. Yongfeng Tunnel face: (a) tunnel face physical map; (b) schematic diagram of tunnel face.

Affected by the collision between the Indian Ocean plate and the Eurasian plate and the extrusion effect of the Tibetan Plateau on the eastern region, as shown in Figure 1, the tunnel area has strong high ground stress. According to the hydraulic fracturing method, the maximum principal stress of the in situ stress of the tunnel at a depth of 360 m is 13.05 MPa, the minimum principal stress is 8.72 MPa, and the vertical principal stress is 8.98 MPa. These metrics show a tight control over horizontal tectonic stress. The strength stress ratio ($R_c/\sigma_{\max}$) is 0.38, indicating that the tunnel is in an extremely high ground stress field environment. These factors cause many large deformations during tunnel construction, including initial support cracking, shotcrete falling block, arch distortion, and secondary lining cracking, as shown in Figure 3.

2.2. Supporting Parameters

Composite lining was employed for the Yongfeng Tunnel's support, in accordance with the design concept of the new Austrian method. The test segment of the tunnel has the same parameters and composition of support. The initial support was composed of shotcrete, anchor rods, I-beams, and steel mesh. The shotcrete model was C25, the

I-beam steel model was 18 I-steel, and other specific parameters are shown in Table 1. The secondary lining was made up of 50 cm of C30 concrete, $\Phi 20/20$ steel bars, and $\Phi 8$ stirrups.



Figure 3. Typical hazard photos of the Yongfeng Tunnel: (a) twisted side wall arches; (b) arch waist sprayed concrete falling blocks.

Table 1. Main support parameters of the two test sections.

Support Type	Item	Numerical Value
Fore poling	Advanced small catheter model	$\Phi 42 \times 4$
	Length	4.5 m
	Spacing	40 cm
	Layout area	120°
Initial support	Sprayed concrete model	C25
	Thickness of shotcrete	24 cm
	Anchor model	$\Phi 25 \times 5$ hollow grouting bolt
	Anchor length	3 m
	Anchor pitch	120×120 cm
	Steel arch model	I18
	Spacing of the steel arch	60 cm
	Wire mesh model	$\Phi 8$
	Wire mesh spacing	20×20 cm
Secondary lining	Secondary lining concrete model	C30
	Thickness	50 cm
	Main steel bar model	$\Phi 20$
	Main steel bar spacing	20 cm

2.3. Excavation Method

The tunnel's building process called for the right tunnel to be dug out before the left, i.e., the right tunnel is the leading tunnel, and the left tunnel is the lagging tunnel. The tunnel was excavated from the exit to the entrance. The test parts of the tunnel were constructed using the upper and lower step approach, as illustrated in Figure 4. The upper and lower steps had excavation heights of 7.0 and 1.75 m, respectively, and the inverted arch's excavation height was 1.5 m. The construction of the tunnel was shut down for approximately two months due to a large deformation of the tunnel. The start times of each construction process of the test sections are shown in Figure 5.

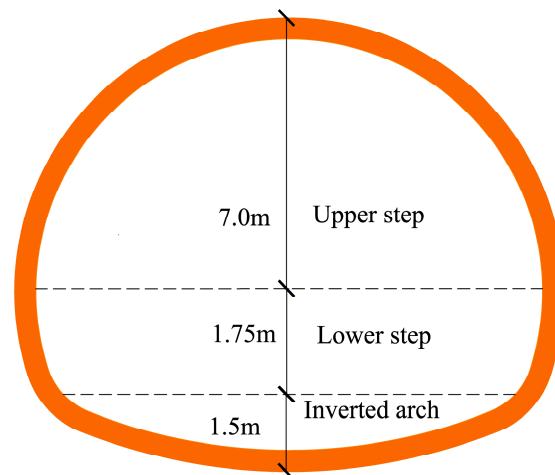


Figure 4. Yongfeng Tunnel excavation schematic diagram.

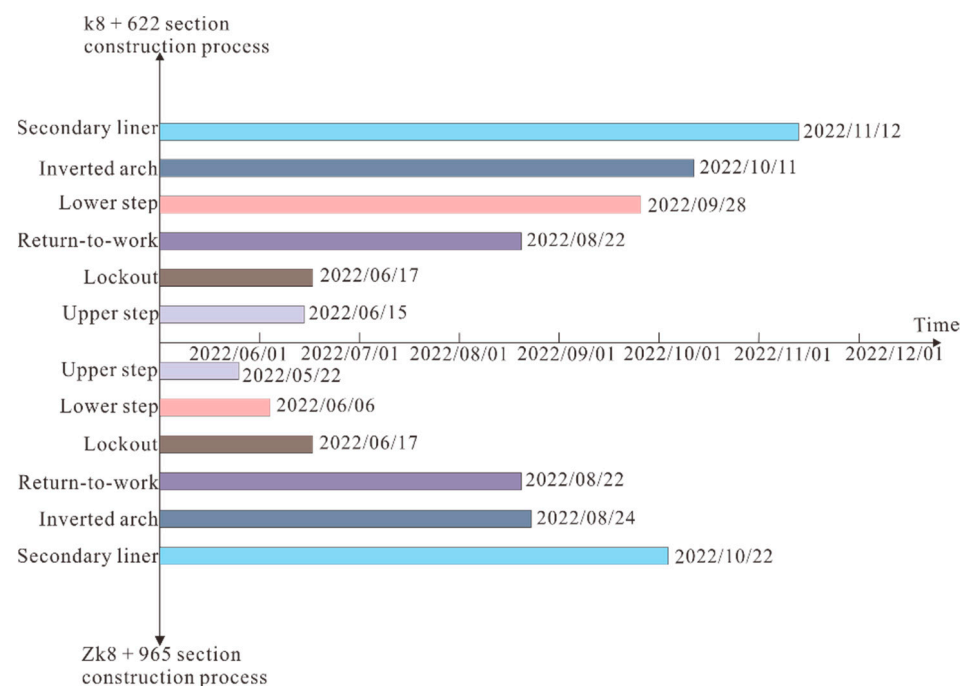


Figure 5. Test section construction process start times.

3. Test Scheme

The ZK8 + 965 portion of the left tunnel (the lagging tunnel) and the K8 + 622 part of the right tunnel (the leading tunnel) were chosen for mechanical testing to clarify the mechanical response of the tunnel support. The geology of the two test sections is the same, and the surrounding rock grade is V grade. The distances of the tunnels in the two test sections are 35 and 29 m, respectively, as depicted in Figure 6. The tunnel's subterranean depths at the K8 + 622 and ZK8 + 965 sections are 513 and 330 m, respectively.

The mechanical test parameters of the tunnel included the contact pressure P_1 between the surrounding rock and the primary support, the contact pressure P_2 between the primary support and the secondary lining, the stress of the primary support steel frame, and the stress of the secondary lining concrete. The test positions included vaults, left and right haunches, left and right side walls, left and right arch feet, and inverted arches. The specific monitoring layout is depicted in Figure 7a. The contact pressure was tested by a string earth pressure box. The maximum pressure cell measurement is 2 MPa, and the accuracy is 0.5% FS. Before setting the pressure box, the earth pressure box was fixed on the steel

plate welded with U-shaped steel bars. In addition, the surrounding rock was initially sprayed with concrete and hammered flat, and then the earth pressure box was installed, as displayed in Figure 7b.

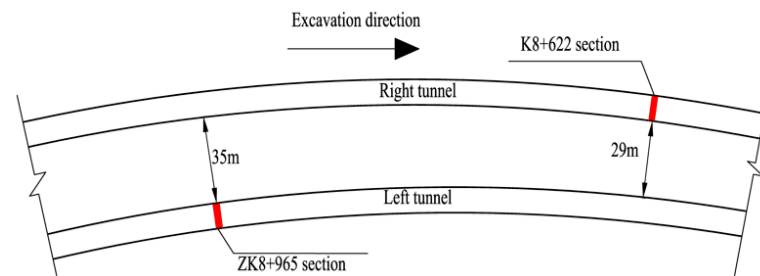


Figure 6. Layout of the monitoring section.

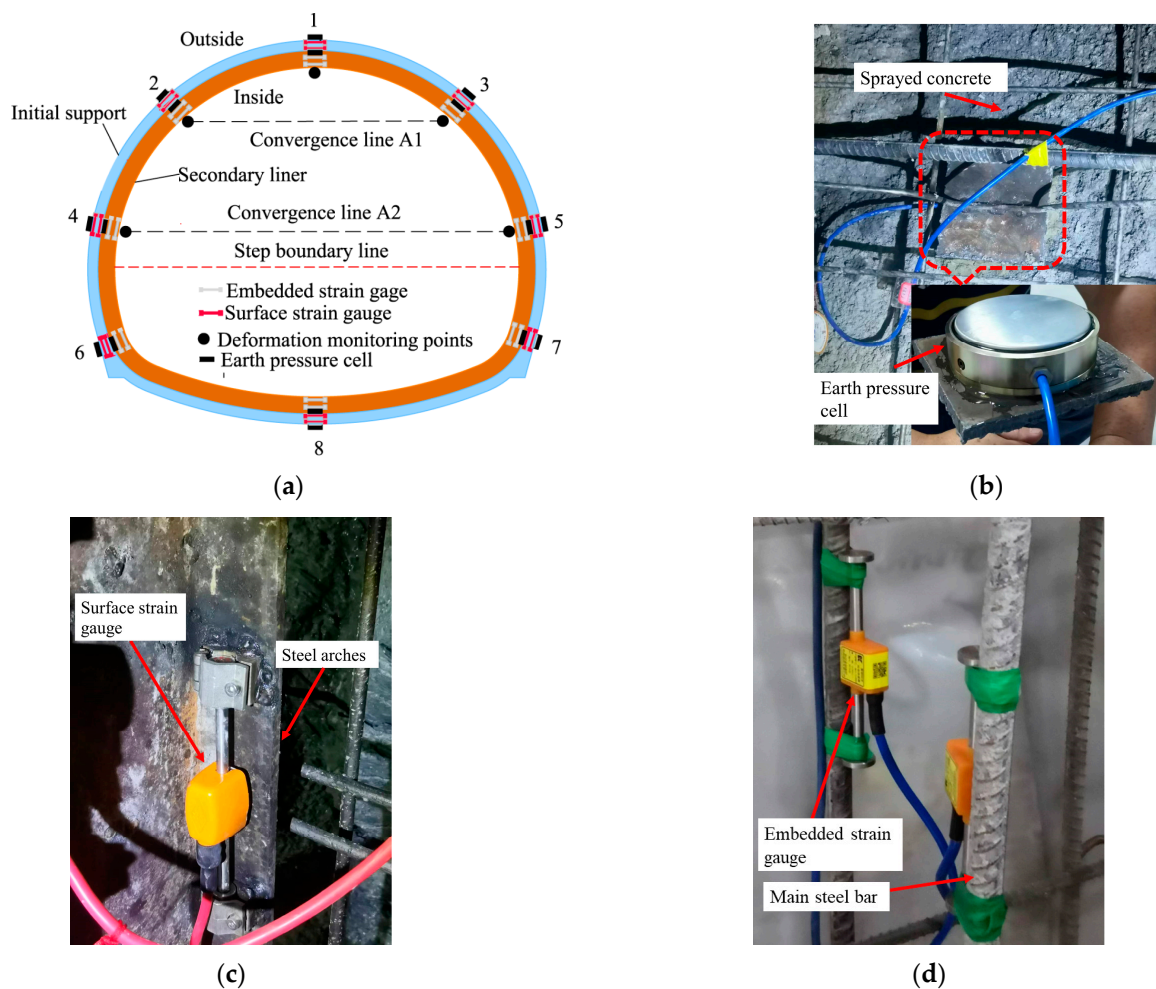


Figure 7. Layout and installation of on-site monitoring points: (a) schematic diagram of monitoring point layout; (b) installation of soil pressure box; (c) installation of surface strain gauge; (d) installation of embedded transformer.

The stress of the steel arch was tested by a surface strain gauge, and the strain measurement range and accuracy are $\pm 2500 \mu\epsilon$ and 0.5% FS, respectively. The steel arch's top and lower flanges are where the surface strain gauges were installed. The order of installation was to first weld the fixed support on the upper and lower flanges and then install the surface strain gauge on the support, as shown in Figure 7c. The embedded strain gauge was used to test the secondary liner concrete's stress, and the strain measurement range and

accuracy are $\pm 1500 \mu\epsilon$ and 0.5% FS, respectively. The embedded strain gauge was fixed on the circumferential main steel bar on both sides of the secondary lining through tape or wire. At the same time, it should be noted that the embedded strain gauge should be parallel to the circumferential main reinforcement of the secondary lining and perpendicular to the tunnel direction, as displayed in Figure 7d. The mechanical test data of the tunnel were collected using a comprehensive tester. The test frequency was 1 time/day up to 60 days after the equipment installation, which can be adjusted appropriately in the later period. The monitoring time was from May 2022 to September 2023.

4. Results and Analysis

4.1. Characteristics of Contact Pressure P_1 between Primary Support and Surrounding Rock

(1) Time variation characteristics

The temporal variations in contact pressure P_1 are depicted in Figure 8. Figure 8a illustrates that the contact pressure P_1 of the left tunnel (the lagging tunnel) fluctuated significantly with time due to the influence of construction. After the upper bench was excavated, P_1 increased sharply due to the rapid release of ground stress. In particular, the contact pressure P_1 at the left arch waist (#3), left wall (#5), and right wall (#4) exhibited a significant increase. Following the lowest step's construction, P_1 first decreased rapidly and then increased slowly. This is similar to the results of Li et al. [31]. This was because the steel arch was momentarily suspended while the lower step was being built. During the shutdown period, P_1 increased slowly with time due to the reduction in construction disturbance. After the construction of the inverted arch, P_1 first decreased briefly and then increased, and the reason was similar to that of the construction of the lower step. After approximately 70 days of secondary lining construction, the contact pressure P_1 of each part gradually stabilized. This occurs as a result of the concrete strength of the secondary lining developing and the surrounding rock stress being modified, which redistributes the load on the secondary lining as well as the primary support.

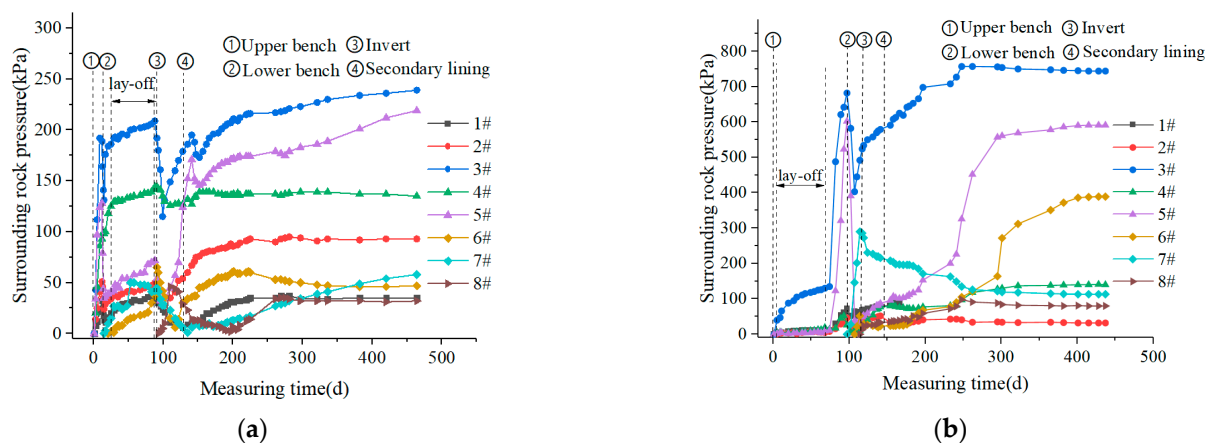


Figure 8. Time-dependent variation in contact pressure P_1 of the test section: (a) ZK8 + 965 section of the left tunnel; (b) K8 + 622 section of the right tunnel.

In contrast to the left tunnel, the contact pressure P_1 of the right tunnel (the leading tunnel) underwent a slow increase rather than a sharp increase after the upper step construction. This was because the tunnel entered the shutdown stage 2 days after the construction of the upper step, the stress released by the surrounding rock was limited, and there was no construction disturbance in the later stage. However, the contact pressure P_1 at the left hance (#3) increased significantly during this stage because it was easy to generate extrusion pressure between soft and hard rock strata due to the uncoordinated deformation of soft and hard rock. The extrusion force and the excavation unloading effect of the tunnel will cause the thin-layer rock strata parallel to the tangential direction of the tunnel contour to undergo bending instability failure. After construction resumed, the

contact pressure P_1 increased sharply, particularly in the left wall (#5) and the left arch waist (#3). This suggested that the left wall and left arch waist were more affected by the construction disruption. The contact pressure P_1 tended to stabilize after approximately 140 days of secondary lining construction.

Comparing the time variation curves corresponding to the contact pressures P_1 of the left and right tunnels revealed that the two have the following common points: (1) The surrounding rock pressure P_1 required a long time to reach stability; some measuring points need 400 days to gradually reach stability. This parameter was affected by the shutdown, and the rheology of the surrounding rock under high ground stress was significant. (2) The contact pressures P_1 of the left side wall (#5) and the left hance (#3) changed dramatically. This was mainly due to the large horizontal tectonic stress and the bending/buckling of the thin layer of rock parallel to the tangential direction of the tunnel contour.

(2) Spatial distribution characteristics

The spatial distribution of contact pressure P_1 is illustrated in Figure 9. The contact pressure P_1 of the right tunnel (the leading tunnel) showed a vertical stratification distribution pattern on the cross-section. That is, the contact pressure P_1 was larger in the vertical stratification direction. The maximum contact pressure P_1 was located at the left haunch (#3), which was 744 kPa, followed by the left wall (#5) and the right arch foot (#6), which were 591 and 389 kPa, respectively. These findings suggested that the bedding of the rock stratum regulates the tunnel's contact pressure P_1 .

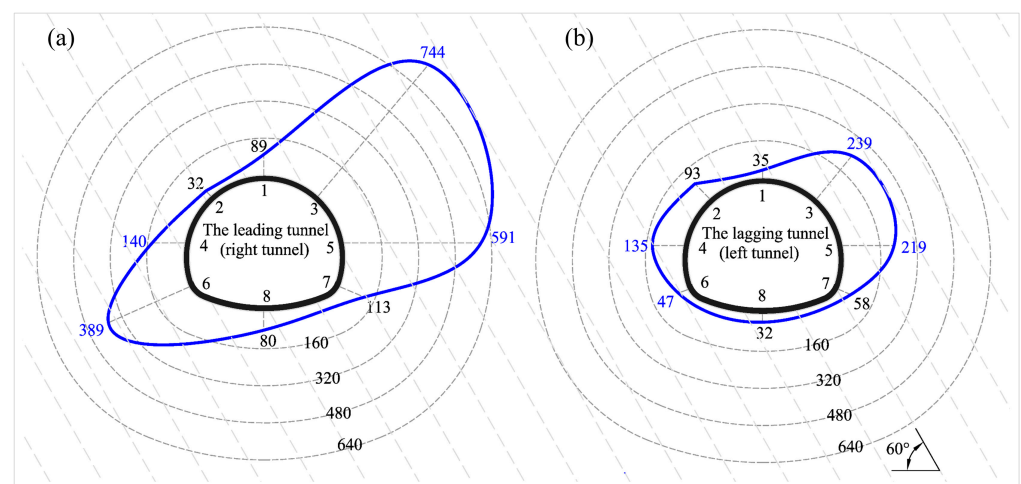


Figure 9. Spatial distribution of contact pressure P_1 of the test section (Units: kPa); (a) K8 + 622 section of the right tunnel; (b) ZK8 + 965 section of the left tunnel.

In contrast to the right tunnel, the contact pressure P_1 of the left tunnel (the lagging tunnel) showed an uneven distribution pattern (“large in the middle, small up and down” and “large in the left and small in the right”) on the cross-section. This was mostly due to the lagging tunnel’s construction being affected by the leading tunnel’s unloading impact. Specifically, it has a major impact on the surrounding rock close to the leading tunnel (the right tunnel), such as at the right arch foot (#6) position. Therefore, the spatial distribution patterns of the contact pressures P_1 of the leading and lagging tunnels were inconsistent. This was also one of the reasons why the deformation and failure of the lagging tunnel (the left tunnel) were mainly concentrated on the left side of the tunnel, as seen in Figure 10. However, the maximum contact pressure P_1 of the left tunnel was situated in the same position as in the right tunnel, and it was also at the left hance (#3), which was 239 kPa. Second, the contact pressures P_1 at the left wall (#5) and the right wall (#4) were also larger, 219 and 135 kPa, respectively, owing to the similar geological conditions of the two sections.

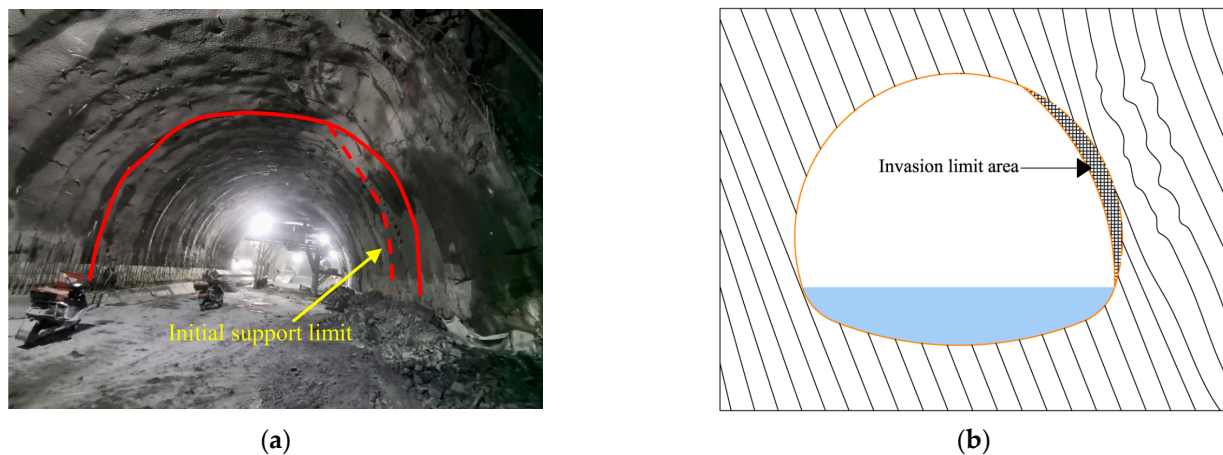


Figure 10. Typical asymmetric deformation of the Yongfeng Tunnel: (a) physical image of asymmetric deformation in tunnels; (b) schematic diagram of asymmetric deformation in tunnels.

In addition, to explore the adaptability of each theory to the surrounding rock pressure of a steeply inclined surrounding rock tunnel, the surrounding rock pressure of the test section was calculated by using the Specifications for Design of Highway Tunnels [32], the Specification for Design of Hydraulic Tunnels [33], and the Protodyakonov theory [34]. Table 2 shows that the tunnel design specification's calculation of the vertical surrounding rock pressure was higher than the observed value. However, the calculation result of the Protodyakonov theory was close to the measured value. It showed that the existing tunnel design code cannot be used to calculate the surrounding rock pressure of steeply inclined soft and hard interbedded tunnels, and the Protodyakonov theory should be used to calculate it. For the horizontal surrounding rock pressure, all calculation results were far less than the measured maximum horizontal pressure. It showed that neither the tunnel design specification nor the Protodyakonov theory can effectively evaluate the horizontal surrounding rock pressure of steep tunnels with surrounding rock.

Table 2. The surrounding rock pressure value of each calculation method (kPa).

Pressure of Surrounding Rock	Specifications for Design of Highway Tunnels	Specification for Design of Hydraulic Tunnels	Protodyakonov's Theory
Vertical pressure	256	76.8	82
Horizontal pressure	128	20.5	69.9

4.2. The Strain of the Primary Support Steel Arch

(1) Time variation characteristics

Figure 11 shows the strain of the steel arches of the left and right tunnels over time. The test results indicated that the steel arch has yielded in some positions. Therefore, strain was more effective than stress to reflect the stress of the arch [1]. It is important to highlight that compressive strain is denoted by a positive value, whereas tensile strain is represented by a negative value.

Figure 11a,b show that the inner and outer strains of the steel of the left tunnel (the lagging tunnel) are similar over time. The steel arch's strain increased significantly once the upper steps were finished, especially at the vault (#1). After 12 days, the strain of the arch at the vault (#1) reached its maximum. The internal and external strains were 2494 and 1861 $\mu\epsilon$, respectively, both exceeding the yield strain of the arch frame of 1780 $\mu\epsilon$. This indicated that the structure of the vault was dangerous, and this was mainly due to the horizontal extrusion of tectonic stress. After the inverted arch construction, the arch strain of each part increased rapidly because the entire supporting system formed a completely closed structure when the inverted arch was closed. Thus, it can bear greater amounts of

stress. The arch strain at the left wall (#5) became more complex after the excavation of the lower step, showing a shift from compression to tension. This was due to the multiple collapses of the poor surrounding rock at this location. The strain of the arch tended to be stable at all measuring points after the secondary lining was completed. On the one hand, the stress adjustment of the surrounding rock has been completed. On the other hand, the load acting on the primary support has been transferred to the secondary lining structure.

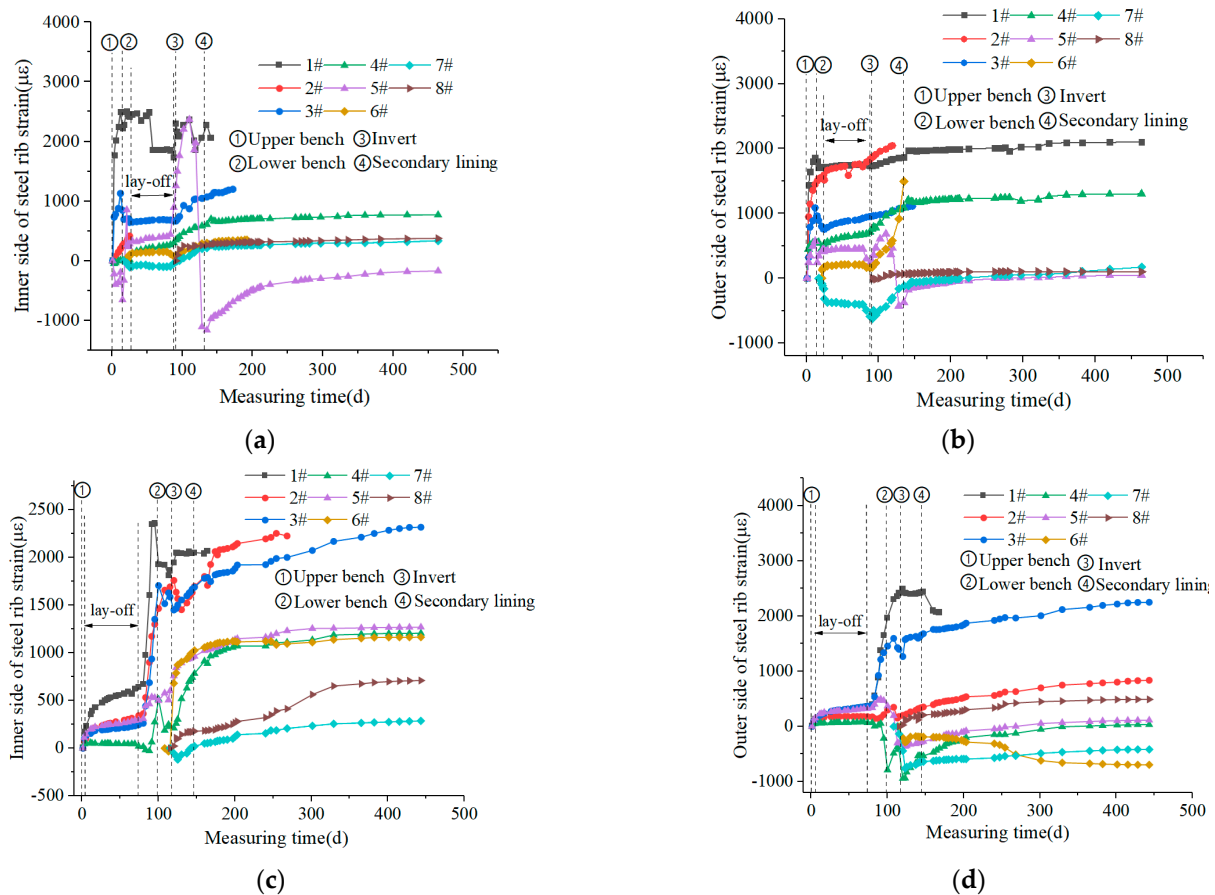


Figure 11. Strain variation in the initial support arch of a typical cross-section of a tunnel over time: (a) inside the arch frame ZK8 + 965 section in the left tunnel; (b) the outer side of the arch frame ZK8 + 965 section in the left tunnel; (c) inside the arch frame K8 + 622 section in the right tunnel; (d) outer side of the arch frame K8 + 622 section of the right tunnel.

The variation in the arch strain of the right tunnel (the leading tunnel) over time is illustrated in Figure 11c,d. The strain variations of the inner and outer sides of the steel arch of the right tunnel (the leading tunnel) are similar over time. When the upper step was constructed, the strain in the right tunnel's arch frame increased gradually, in contrast to the strain characteristics of the left tunnel's arch frame. This was because the closer the test section was to the tunnel face, the greater the virtual support force could be provided by the tunnel face. The test section was only approximately 2 m away from the tunnel face during the excavation of the upper steps until the tunnel construction was stopped. After resuming tunnel construction, the strain of the right arch increased sharply. This was due to the fact that when the distance between the test section and the tunnel face grew, the virtual support force provided by the tunnel face at the test section decreased. To resist deformation, the stress of the steel arch will increase sharply. Furthermore, the tunnel's excavation was continuously destroying the surrounding rock, which increased the stress on the supporting structure.

(2) Spatial distribution characteristics

Figure 12 displays the spatial distribution of the strain of the steel arches. The steel frames of the left and right tunnels were mainly compressed, although a few parts were tensioned. There was a distribution pattern of “large in the upper and small in the lower” in the strain of the steel frames of the left and right tunnels. Figure 12a,b show that the maximum compressive strains of the inner and outer sides of the steel frame of the left tunnel were located at the vault (#1), which were 2057 and 2100 $\mu\epsilon$, respectively. These exceeded the yield strain of the arch. The tensile strain appeared on the inner side of the left wall (#5), which was 170 $\mu\epsilon$. Figure 12c,d show that the maximum compressive strains of the inner and outer sides of the steel frame of the right tunnel were located at the left haunch (#3), which were 2316 and 2252 $\mu\epsilon$, respectively. All of these exceeded the yield strain of the structure (1780 $\mu\epsilon$), indicating that the structures at the vault and the left haunch were dangerous. Significant concrete spalling was also observed in the right arch waist, as shown in Figure 13.

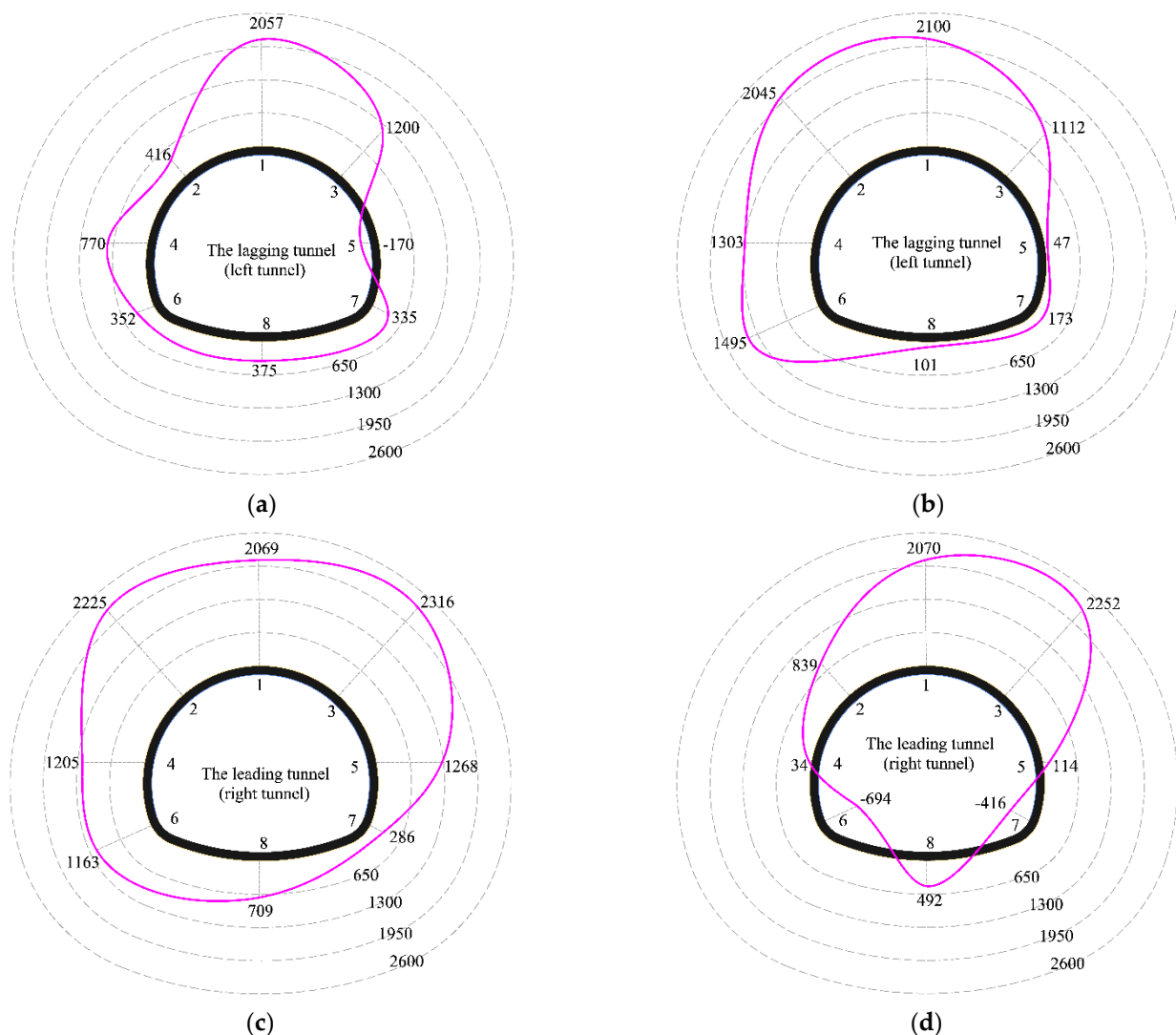


Figure 12. Spatial distribution of strain in a typical cross-section arch frame of the tunnel (Units: $\mu\epsilon$): (a) inside the arch frame ZK8 + 965 section in the left tunnel; (b) the outer side of the arch frame ZK8 + 965 section in the left tunnel; (c) inside the arch frame K8 + 622 section in the right tunnel; (d) outer side of the arch frame K8 + 622 section of the right tunnel.

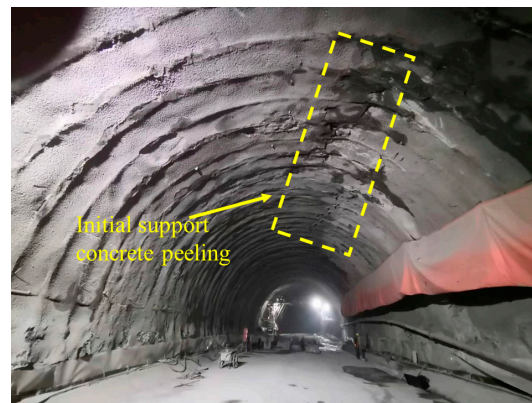


Figure 13. Initial support concrete peeling of the tunnel.

4.3. Characteristics of Contact Pressure P_2 between Primary Support and Secondary Lining

(1) Time variation characteristics

Figure 14 illustrates how the contact pressure P_2 varies over time. Over time, the contact pressures P_2 in both the left and right tunnels increased due to the growing strength of the secondary lining concrete and the continuous adjustment of its bearing ratio. In particular, the contact pressure P_2 of the left wall (#5) changed most sharply with time due to the poor surrounding rock and the large horizontal tectonic stress. Additionally, the contact pressure P_2 took a long time to stabilize, and some measuring points were not stable for more than 350 days. These results showed that the soft and hard interbedded surrounding rock has a long failure duration and strong rheological properties in a high ground stress environment.

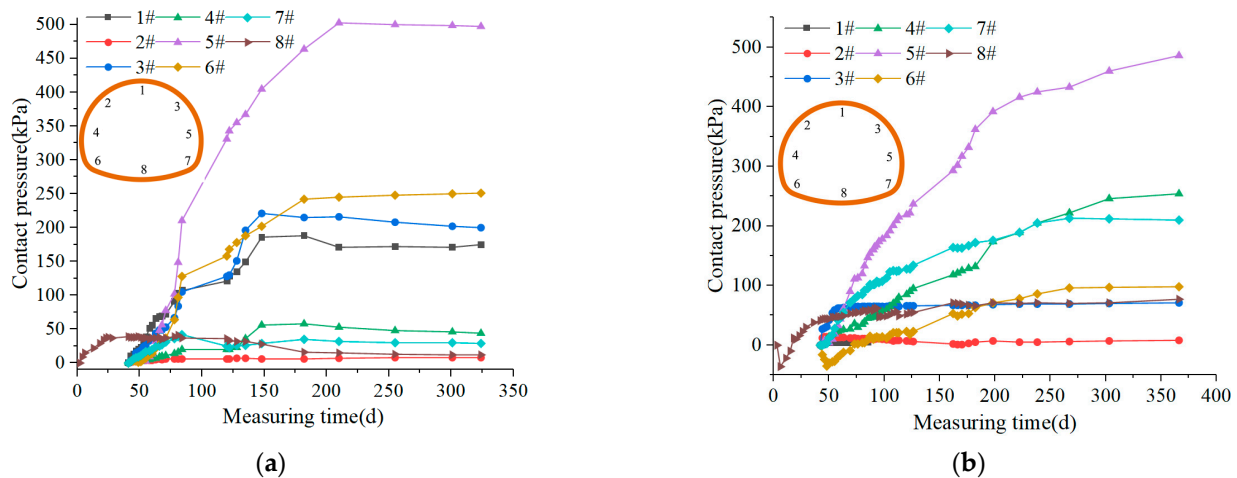


Figure 14. Time-dependent variation in contact pressure P_2 of the test section: (a) K8 + 622 section of the right tunnel; (b) ZK8 + 965 section of the left tunnel.

(2) Spatial distribution characteristics

The spatial distribution map of the contact pressure P_2 is displayed in Figure 15. The spatial distributions of P_2 and P_1 in the two test sections were similar. The contact pressure P_2 in the right tunnel (the leading tunnel) was distributed in a vertical laminar pattern. The maximum contact pressure P_2 was located on the left wall (#5), which was 497 kPa, followed by the right arch foot (#6) and the left arch waist (#3), which were 251 and 200 kPa, respectively. Thus, it was evident that the surrounding rock bedding and ground stress controlled the contact pressure P_2 of the right tunnel (the leading tunnel). The contact pressure P_2 of the left tunnel (the lagging tunnel) showed uneven distribution patterns of “large in the middle, small up and down” and “large on the left and small on the right”. The maximum contact pressure P_2 was located on the left wall (#5), which was 486 kPa,

followed by the right wall (#4), which was 254 kPa. The reason for the spatial distribution differences in terms of contact pressure P_2 between the two test sections was analogous to the reason governing the contact pressure P_1 .

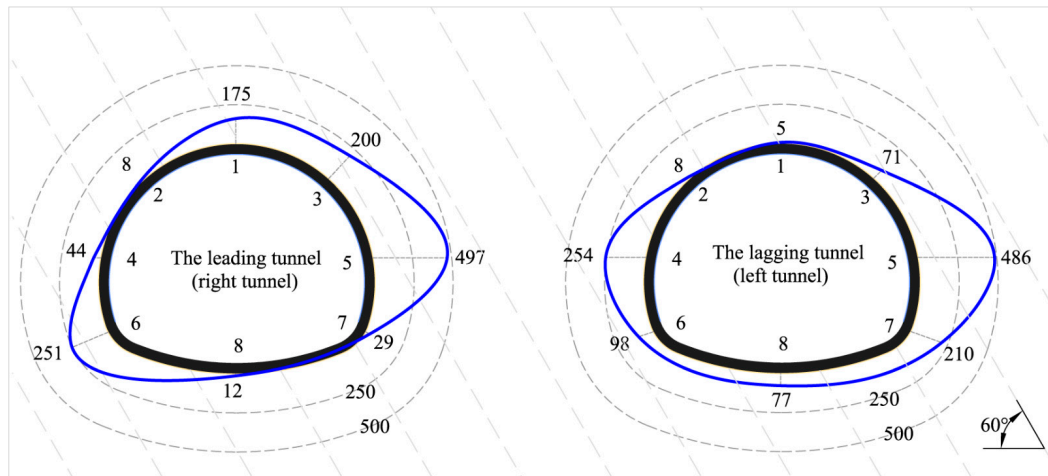


Figure 15. Spatial distribution of contact pressure P_2 of the test section (Units: kPa).

However, it should be noted that the maximum values of the contact pressures P_1 and P_2 were located at different positions in the left and right tunnels. The maximum contact pressures P_1 of the left and right tunnels were located at the left arch waist (#3), while the maximum contact pressures P_2 were located at the left wall (#5). These results suggested that the secondary lining can provide strong stiffness or even adjust the position of the stress concentration.

(3) Load sharing ratio of primary support and secondary lining

In order to study the load-sharing ratio of the primary support and the secondary lining of the tunnel, the load-sharing ratio of the primary support structure was defined as $P_1/(P_1 + P_2)$, and the load-sharing ratio of the secondary lining structure was defined as $P_2/(P_1 + P_2)$. The load sharing of the tunnel's primary support and secondary lining is shown in Figure 16. The secondary lining not only serves as a safety reserve but also bears a significant load, sharing the surrounding rock load with the initial support structure. The average sharing ratios of the secondary lining structure of the right tunnel (the leading tunnel) and the left tunnel (the lagging tunnel) were 31.2% and 49.0%, respectively. During excavation in the natural ground stress field of the leading tunnel (the right tunnel), the ground stress was released quickly and violently, causing the initial support to bear a larger amount of the load and the second lining to bear a smaller amount of the load. However, owing to the unloading effect of the leading tunnel (right tunnel), the stress release of the surrounding rock of the lagging tunnel (left tunnel) is not as severe as that of the leading tunnel. Thus, the primary support bore less load, while the secondary lining bore a larger load.

4.4. Stress Characteristics of Secondary Lining Concrete

(1) Time variation characteristics

Figure 17 illustrates how the secondary liner concrete's stress changed over time. Positive and negative values correspond to compressive and tensile stress, respectively. The stresses in the secondary lining concrete of the left and right tunnels have been observed to increase over time, and as of the 360th day of monitoring, these stresses have yet to reach a stable state. These results implied the presence of strong rheological features in the surrounding rock around the tunnel. The stress on the secondary lining concrete in some parts of the left tunnel increased first (tensile stress) and then decreased (compressive stress), e.g., the arch waist (#3, #2) and vault (#1) on the inner side of the secondary lining; the side wall (#4), arch foot (#7), arch waist (#2), and vault (#1) on the outer side of the secondary

lining. The potential cause of this phenomenon could be attributed to the thermal stress within the secondary lining and the concrete's hardening shrinkage.

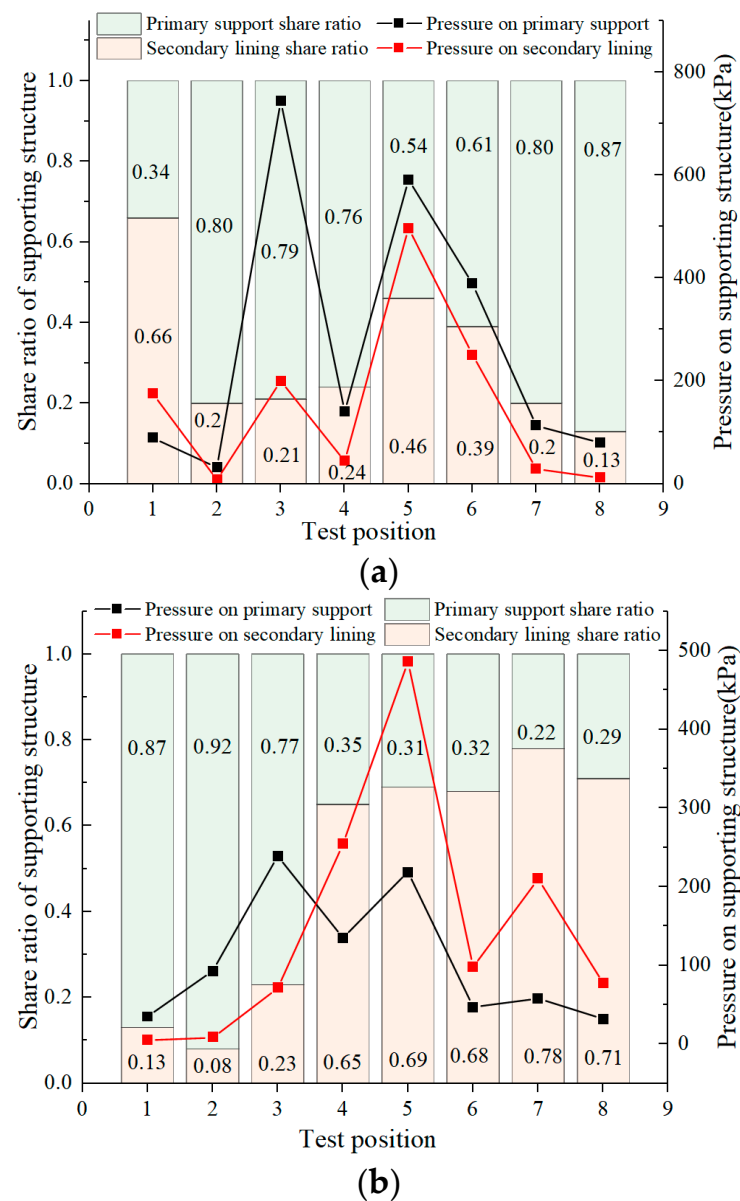


Figure 16. Comparison of the loads shared by the initial support and secondary lining of the test sections: (a) load-sharing ratio of K8 + 622 section in the right tunnel; (b) load-sharing ratio of ZK8 + 965 section in the left tunnel.

(2) Spatial distribution characteristics

Figure 18 shows the spatial distribution map of the stress on the secondary lining concrete of the tunnel, where positive and negative values represent compressive stress and tensile stress, respectively. The inner and outer sides of the secondary lining concrete of the left tunnel were mainly compressed, although a few parts were tensioned. The overall stress distribution was extremely uneven, and there was clear bias pressure. The stress distribution patterns of the inner and outer sides of the secondary lining of the left tunnel corresponded to “right big, left small”. The left side wall (#5) has the highest compressive stress on the inner side of the secondary lining of the left tunnel, measuring 6.9 MPa. The right arch foot (#6) has the second-highest compressive stress, measuring 4.5 MPa. The maximum tensile stress on the inner side of the secondary lining was 5.9 MPa at the

inverted arch (#8). This was likely caused by the poor surrounding rock at the inverted arch and the uplift after excavation.

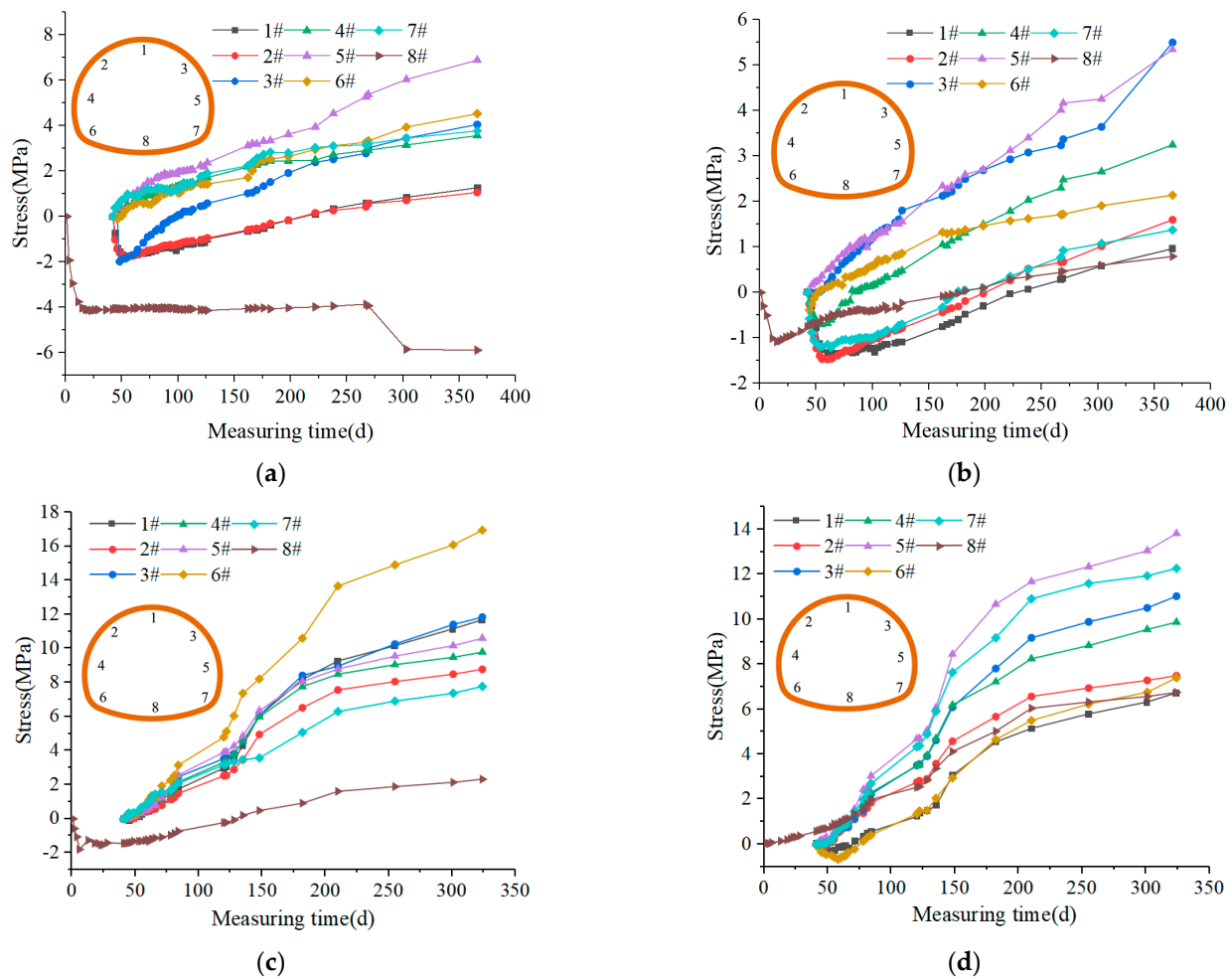


Figure 17. Strain variations in the secondary lining concrete in the test sections over time: (a) inside the secondary lining of section ZK8 + 965 in the left tunnel; (b) outer side of the secondary lining of section ZK8 + 965 in the left tunnel; (c) inside the secondary lining of section K8 + 622 in the right tunnel; (d) outer side of the secondary lining of section K8 + 622 in the right tunnel.

In contrast, the secondary lining of the right tunnel exhibits compression on both its inner and outer sides. However, there were noticeable variations between the inner and outer sides of the right tunnel's secondary lining's spatial distribution patterns. The stress on the outer side of the secondary lining of the right tunnel was distributed in a "left big, right small" mode. The maximum stress occurred at the left arch waist (#5) with a value of 13.8 MPa, followed by the left arch foot (#7) with a value of 12.3 MPa. However, the stress on the inside of the secondary lining indicated a clear vertical bedding distribution pattern, i.e., the stress on the secondary lining was larger in the vertical bedding direction. The maximum stress in the secondary lining occurred at the right arch foot (#6) with a value of 16.9 MPa, and then at the left arch waist (#3) with a value of 11.8 MPa. This indicates that the right tunnel is not only affected by the horizontal tectonic stress but also by the surrounding rock bedding surface. Additionally, the left and right tunnels' secondary lining concrete's maximum stress was lower than the concrete's compressive design value (30 MPa). Thus, the secondary lining structures are safe.

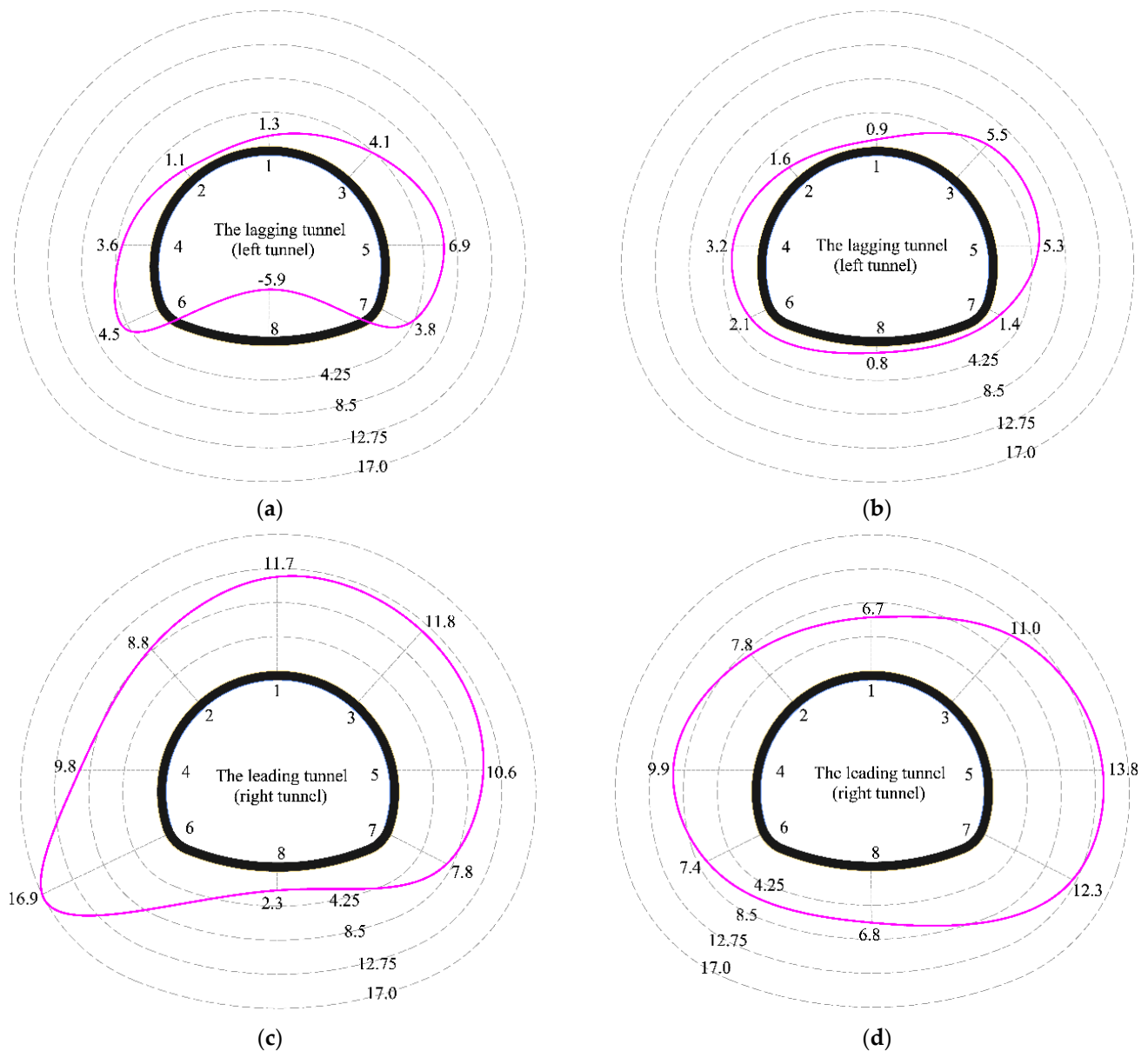


Figure 18. Spatial distributions of stress in the secondary lining concrete of the tunnel test sections: (a) inside the secondary lining of section ZK8 + 965 in the left tunnel; (b) outer side of the secondary lining of section ZK8 + 965 in the left tunnel; (c) inside the secondary lining of section K8 + 622 in the right tunnel; (d) outer side of the secondary lining of section K8 + 622 in the right tunnel.

5. Design and Construction Optimization Measures

The monitoring data indicated that the steel arch at the local position of the tunnel has yielded and is in an unsafe state. Consequently, the tunnel's construction and support must be optimized in light of these monitoring findings. The specific optimization suggestions are as follows.

Owing to the uncoordinated deformation of soft and hard interbedded layers and the influence of bedding planes, the tunnel was affected by severe bias, i.e., the bias loads at the left arch waist and the left side wall were more prominent. As a result, the surrounding rock at the tunnel's left side wall and left arch waist need to be strengthened. First, grouting reinforcement should be applied to the surrounding rock to lessen the uncoordinated deformation between soft and hard rock strata and enhance the surrounding rock's self-bearing capacity. Second, the distance between the anchor bolts at the left arch waist and the left side wall is shortened, increasing the rock's support point and lowering the risk of the

surrounding rock bending failure. The long and short anchor method is also effective for optimizing the layered surrounding rock tunnel [35]. Specifically, it can cause the thin layer of rock to form a composite beam while increasing the thickness of the rock layer to reduce the damage to the surrounding rock. Moreover, it can enhance the surrounding rock's strength properties. In addition, the stiffness of the steel arch should be improved, and steel fiber shotcrete should be used to increase the support strength to resist the deformation of the surrounding rock and prevent the shotcrete from cracking.

To properly release the ground stress and reduce the bias load of the support, the design reserved deformation of the tunnel should be increased, and the asymmetric reserved deformation design should be adopted. In other words, the tunnel's left side should have more reserved deformation than its right. The leading tunnel has an unloading effect on the lagging tunnel; therefore, the construction sequence of the left and right tunnels can also be adjusted. These suggested actions provide a guide for designing and building tunnels in locations with comparable strata.

6. Discussion

In this paper, the surrounding rock pressure and supporting structure stress of soft and hard interbedded tunnels with high ground stress were systematically monitored by field tests. The results showed that the contact pressure P_1 between the initial support and surrounding rock of the tunnel was asymmetrically distributed in a vertical bedding direction. This is different from the distribution characteristics of surrounding rock pressure in homogeneous surrounding rock tunnels. Luo et al. [36] confirmed that the surrounding rock pressure distribution of homogeneous surrounding rock tunnels has obvious symmetry. This may be due to the development of soft and hard interbedded surrounding rock tunnel bedding, the anisotropy of surrounding rock strength is significant, and the surrounding rock in the vertical bedding direction is prone to bending instability during tunnel excavation. At the same time, this result is also different from the layered surrounding rock of a single type of rock. Wu et al.'s [26] and Wang et al.'s [37] studies have shown that the surrounding rock pressure of a steeply inclined layered surrounding rock tunnel with a single type of rock is distributed along the bedding direction. This is mainly due to the difference in mechanical parameters between the layers. When the friction coefficient and cohesion between the beddings are weak, it is easy to produce slip failure along the bedding direction.

Moreover, compared with the previous studies of layered surrounding rock tunnels [5,25], this paper found that the leading tunnel had an obvious unloading effect on the lagging tunnel, especially for the contact pressure P_1 in the vertical bedding direction of the adjacent tunnel side. This can provide guidance and suggestions for the excavation sequence of layered surrounding rock tunnels. In addition, the monitoring results also showed that the maximum strain of the initial support structure of the leading tunnel was the same as the position of the maximum value of the contact pressure P_1 , which was located at the left arch waist. It shows that bedding is one of the main factors affecting the stress of supporting structures. However, the maximum strain of the primary support structure of the lagging tunnel was located at the vault, indicating that the ground stress is also an influencing factor affecting the stress of the support structure.

7. Conclusions

In the context of the Yongfeng Tunnel, this report analyzed the surrounding rock pressure and mechanical characteristics of the supporting structures of soft and hard interbedded tunnels in high ground stress environments using field tests. The following are the key findings:

- The contact pressures (P_1 and P_2) took a long time to reach stability (a few measuring points over 400 d). In space, the contact pressures (P_1 and P_2) of the leading tunnel (the right tunnel) were distributed in the direction of vertical stratification, while the

contact pressures (P_1 and P_2) of the lagging tunnel were differed due to the excavation unloading of the leading tunnel.

- The maximum values and positions of the contact pressures P_1 and P_2 were different. The maximum contact pressures P_1 of the left and right tunnels were located at the left arch waist, which were 239 and 744 kPa, respectively. The maximum contact pressures P_2 of the left and right tunnels were located at the left side wall, which were 486 and 497 kPa, respectively, indicating that the secondary lining can adjust the position of the stress concentration.
- The inner and outer sides of the steel frames of the left and right tunnels were mainly compressed, although a few parts were tensioned. The spatial distribution was characterized as “large in the upper part, and small in the lower part”. The maximum strains (inside and outside) of the left and right arches were 2057, 2100, 2316, and 2252 $\mu\epsilon$, respectively, which were located at the vault and the left arch waist, respectively. These numbers were higher than the arch’s yield strain, indicating that the structure was hazardous.
- It took a long time (>350 days) for the stress of the secondary lining concrete to stabilize. The secondary lining concrete stress (inside and outside) of the left tunnel and the outside of the right tunnel were asymmetrically distributed. However, the inside of the secondary lining of the right tunnel showed a clear vertical bedding distribution. The maximum stress of the secondary linings of the left and right tunnels was located at the left side wall and the right arch foot.
- Based on the pressure distribution of the tunnel surrounding rock and the stress characteristics of the supporting structure, some engineering suggestions were proposed. Specifically, it was recommended to grout the surrounding rock at the bias position, shorten the bolt spacing, combine the long and short bolts, increase the stiffness, increase the reserved deformation, and adopt an asymmetric reserved deformation design.

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