Failure mechanism and treatment measures of supporting structures at the portal for a shallow buried and asymmetrically loaded tunnel with small clear-distance

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Abstract

The construction of a tunnel portal faces the challenges of complex geological conditions, such as shallow burial and asymmetrical loading. In addition, the adverse effects of factors such as the layout form of the tunnel must also be considered. Thus, the construction of portals has always been the focus of tunnel engineering. Under the coupling of multiple adverse factors, such as complex geological conditions and special layout form, the tunnel portal is prone to excessive deformation, supporting structural cracking, and even collapse during excavation. In this study, a shallow buried and asymmetrically loaded tunnel with a small clear-distance in northwest China was considered as an engineering case. To address the distresses of slope instability, peeling off and block falling of primary support concrete, and cracking of secondary lining concrete in the tunnel portal construction, combined with field investigations, statistical analysis, numerical simulation, and deformation monitoring, the failure mechanism of supporting structures was deeply studied and corresponding treatment measures were proposed. The research results indicated that loose and broken gravel soil in the shallow buried section, asymmetrical loading, surface water infiltration, and short construction spacing between the two tunnels were the main triggers of supporting structures failure. Affected by topographic bias, the loose load generated by the surrounding rock on the deeply buried side squeezed the entire tunnel to the shallow buried side after portal excavation. This deformation trend became more significant after the gravel soil deteriorated by water immersion. The retaining wall produced a clockwise rotation deformation around the wall corner in the process of limiting the tunnel deviation, and the local wall body cracked owing to excessive tensile stress. The primary support concrete and secondary lining concrete produced excessive asymmetrical deformation because of significant asymmetrical loading. Concrete with excessive deformation was cracked by obvious tensile or shear stresses. The subsequent tunnel excavation had a significant negative impact on the stability of the prior tunnel. Combining the failure mechanism of the supporting structures and the characteristics of the continuous development of cracks, the treatment measures of ‘stabilize the stratum first and then treat the cracks’ were proposed, including the backfilling and tamping the shallow buried side at tunnel portal, reinforcing the interlaid rock by ground surface grouting, setting intercepting ditch at the slope top, and staggering a certain safe excavation distance between the following tunnel and the prior tunnel. Field monitoring and patrol inspection results indicated that the proposed
treatment measures achieved the expected results. The research results can provide corresponding construction experience and suggestions for similar projects in future.

Keywords Tunnel engineering · Shallow buried and asymmetrically loaded · Small clear-distance · Failure mechanism · Treatment measures

1 Introduction

In recent years, with the gradual expansion of transportation infrastructure in China, a large number of tunnels and underground projects have been constructed. Despite the remarkable developments in tunnel construction technologies, the difficulty in excavating tunnel portals is still widespread in tunnel engineering construction. Acting as the throat of a tunnel project, the portal is an integral yet challenging part of construction. The tunnel portal is often located in complex terrain with severe weathering of the surrounding rock, which negatively impacts the safety during tunnel portal constructions. If the excavation methods, supports, and auxiliary measures were unreasonable during tunnel portal construction, it would be extremely easy to cause distress, such as excessive deformation of the surrounding rock, cracking and failure of supporting structures, and even collapse (Sun et al. 2019; Qiu et al. 2020; Hu et al. 2021). These distresses not only cause construction delays and property losses but also seriously threaten the personal safety of construction personnel.

Shallow buried and asymmetrical terrains are the most common adverse geological conditions encountered in tunnel portal excavations. Shallow and asymmetrically loaded tunnels often exhibit significant asymmetric settlement and local stress concentration during the excavation process, and they are more prone to failure of the supporting structures (Qiu et al. 2015; Yang et al. 2020). Current research on shallow and asymmetrically loaded tunnels mainly focuses on the following aspects: calculation of the tunnel overlying the surrounding rock pressure (Zuo et al. 2011; Bai and Wu 2012; Yang et al. 2013b; Qiu et al. 2015), the study of deformation and stress characteristics of tunnel structures (Yang and Wang 2018; Pan et al. 2011; Lei et al. 2015; Liu et al. 2017), optimization of excavation methods and supporting measures (Yang et al. 2013a; Xiao et al. 2016), and failure mechanism of supporting structures (Xiao et al. 2014; Zhang et al. 2017; Yang et al. 2020). Combined with laboratory model tests and limit analysis methods, Qiu et al. (2015) constructed a failure model of a tunnel with shallow buried and asymmetrical loading and deduced the upper bound solution of the tunnel overlying the surrounding rock pressure based on the principle of virtual power. Zuo et al. (2011) and Bai and Wu (2012) identified limitations in the overlying surrounding rock pressure calculation of shallow and asymmetrically loaded tunnels recommended in China’s Code for Design of Road Tunnel (JTG 3370.1-2018). Therefore, the calculation formula was modified by constructing a new calculation model or programming method, and the modified formula was verified using the simulated results and measured data in actual engineering. Based on similarity theory and elastic mechanics equations, Lei et al. (2015) analyzed the stress distribution laws and failure mechanism of the surrounding rock and lining concrete under different degrees of asymmetrical loading by designing various groups of model tests. Using centrifugal test and finite element simulation methods (FEM), Liu et al. (2017) compared and analyzed the failure modes and stress distribution laws of the surrounding rock at shallow and asymmetrically loaded tunnel portals in bedding and homogeneous strata. Relying on a shallow and asymmetrically
loaded tunnel crossing a large loose sediment layer, Xiao et al. (2016) compared the stability of tunnel structures excavated by the upper and lower bench, three-bench, and single-side drift methods. The single-side drift method was recommended because of its significant advantages in controlling the surrounding rock settlement, ameliorating lining stress, and reducing the disturbance zone area. Yang et al. (2013a) simulated the excavation process of an asymmetrically loaded tunnel under multi-factor coupling and proposed that pre-reinforcement was required if the overlying surrounding rock was less than 15 m. Yang et al. (2020) analyzed the mechanism of the distresses during a shallow and asymmetrically loaded tunnel portal construction and found that asymmetrical loading, precipitation, and inadequate construction management were the main reasons for the failure event. Using FEM, Zhang et al. (2017) investigated the cracking causes of secondary lining concrete at the vault of a multi-arch tunnel. The results indicated that the shallow burial and bias on topography and the use of plain concrete led to concrete cracking owing to excessive tensile stress and negative bending moment.

In addition to objective factors such as topography and geological conditions, subjective factors such as the tunnel layout form, cross section type, and cross-sectional area are also important considerations for the construction methods and support measures adopted in tunnel portals. It is stipulated in China’s Code for Design of Road tunnel (JTG 3370.1-2018) that a sufficient safety distance should be ensured between two parallel tunnels constructed by drilling and blasting method (when the surrounding rock is classified as grade IV, V and VI, the corresponding minimum safety distance is 2.5B, 3.5B, and 4.0B, respectively, where B is the tunnel excavation span). If a safe distance cannot be ensured for various reasons, a small clear-distance tunnel should be designed. Compared with the separated tunnel, the small clear-distance tunnel has the advantages of less connection difficulty and small floor area; compared with the multi-arch tunnel, it has the advantages of giving full play to the self-supporting ability of interlaid rock, short construction period, and easy control of construction quality. Therefore, the small clear-distance tunnels have been widely used for specific terrains and geological conditions. Domestic and foreign scholars have conducted in-depth research on the construction mechanics characteristics of the small clear-distance tunnels. In Japan, Kawada et al. (1996) conducted a systematic study on the design, construction, and blasting control technology of the small clear-distance tunnels. Using model tests and field monitoring methods, Kim (2004), Wen et al. (2004), and Lee and Jacobsz (2006) pointed out that the mutual influence of the construction of the prior and following tunnels would become obvious when the clear distance of the small clear-distance tunnel was less than one time the tunnel width. Combined with the limit analysis method and reliability theory, Zhang et al. (2018) introduced a nonlinear failure criterion and obtained the safety factors and corresponding critical clearance of twin shallow tunnels under different safety levels. Using field monitoring and numerical simulation methods, Xia et al. (2007), Jiang et al. (2012), Cui et al. (2019), and Jiang et al. (2019) studied the deformation mechanism, construction mechanical behavior, and structural stability of large cross section and small clear-distance tunnels. It was found that the following tunnel excavation significantly affected the prior tunnel stability, and special excavation methods (such as CD, CRD and double-side drift method) and auxiliary measures (such as pre-reinforcement and interlaid rock reinforcement) must be carried out to alleviate the negative impact. A reinforcement measure for isolation piles was proposed by Lv et al. (2020) in Guangzhou Metro Line 8 to mitigate the mutual influence of two shield tunnel constructions with small clear distances. Jiao et al. (2019) used FEM to study the deformation characteristics of double-line shield tunnel construction under different
clear distances; if the distance between two tunnels was less than 0.8 times the tunnel diameter, corresponding reinforcement measures should be taken to ensure the surrounding rock stability.

At present, research on shallow buried and asymmetrically loaded tunnels and small clear-distance tunnels has been more in depth, and rich design and construction experience has been accumulated. However, research on tunnel engineering design and construction under the coupling of multiple adverse factors, such as loose and broken surrounding rock, shallow buried and asymmetrical terrain, and layout form of small clear-distance, is still relatively rare. China’s Code for Design of Road Tunnel (JTG 3370.1-2018) stipulates that small clear-distance tunnels with asymmetrical loading, the support parameters, construction methods, and excavation sequence must be specially designed. However, in actual projects, the design and construction of shallow and asymmetrically loaded tunnels with small clear-distance mostly rely on engineering experience. The topographical and geological conditions encountered in tunnel portals are variable and different. During the construction of shallow buried, asymmetrically loaded, and small clear-distance tunnel portal, engineering accidents continue to occur, and the failure mechanism has not yet been explored in detail. Therefore, it is necessary to conduct in-depth research on the causes of distress in the construction of shallow and asymmetrically loaded tunnel portals with small clear-distance, which can timely propose targeted treatment and prevention measures and reduce the possibility of various distresses. The research results can also provide a basis and experience for similar tunnel engineering designs and constructions in future.

In this study, a shallow buried and asymmetrically loaded tunnel with a small clear-distance in northwest China was used for an engineering case. Field investigations and statistics on the failure of supporting structures during tunnel portal construction were conducted first. Combined with a three-dimensional numerical simulation and a field monitoring method, an in-depth study of the failure mechanism of the supporting structures was conducted. Then, corresponding treatment measures were proposed based on the research results, which were successfully applied to actual projects.

## 2 Project overview

### 2.1 Topographic and geomorphic characterization

The studied tunnel is located in northwest China and is a separate bidirectional four-lane highway short tunnel. The starting and ending chainage of the left tunnel is ZK39+843–ZK40+177, with a length of 334 m; the right tunnel is YK39+866–YK40+166, with a length of 300 m. The net height and width of the tunnel were 8.98 m and 10.86 m, respectively, and the maximum buried depth was approximately 60 m. The minimum distance between the left and right tunnel axes is only 13.2 m at the portal section, which belongs to a typical small clear-distance tunnel, as shown in Fig. 1.

The overlying stratum of the tunnel is mainly a light-gray Quaternary Deluvial gravelly soil ($Q_4^{dl}$), which is composed of gravel, block stone, and humus silt, with dense surface plant roots and developed wormholes, as shown in Fig. 2. The overall structure of this stratum is loose and its stability is poor. The average layer thickness is approximately 17.4 m. The underlying stratum is mainly a yellow–brown Lower Triassic sandy slate intercalated with sandstone ($T_1$), with dense and hard rock, and good stability.
The tunnel portal is located in a gravel soil layer. The overburden thickness of the right tunnel portal is less than 3.0 m and the slope gradient exceeds 30°, which is a typical shallow buried and asymmetrically loaded tunnel portal, as shown in Fig. 3. According to the results of the field drilling exploration and laboratory rock mechanics tests, the surrounding rock grade from the portal section to YK/ZK39 + 930 is classified as grade V.

Groundwater in the tunnel site area is relatively scarce. It is mainly composed of bedrock fissure water and Quaternary pore phreatic water, which is supplied by atmospheric precipitation and stored in the gravel soil layer. The surface water is abundant. Multiple streams develop on the surface, which eventually flows into a large river nearby. Moreover, in the rainy season, precipitation is abundant and heavy rainstorms often occur.
2.2 Construction method and supporting measures at the portal

The tunnel was constructed using the new Austrian method, and the upper and lower bench method was adopted for excavation. Owing to the serious shallow and asymmetrical loading in the portal, the right tunnel near the shallow buried side was excavated first, followed by the left tunnel near the deeply buried side. In the original construction scheme, the excavation staggering distance between the two tunnels was 20 m. The composite lining was used for tunnel support. The primary support measures included the 3.5-m R25 hollow grouting anchor bolts with a longitudinal spacing of 75 cm and a circumferential spacing of 100 cm, φ6 double-layer steel mesh with a spacing of 20 × 20 cm, I20a I-steel with a spacing of 75 cm, and C25 shotcrete with a thickness of 26 cm. The secondary lining adopted C30 reinforced concrete with a thickness of 50 cm. To ensure safe excavation of the portal, the 40-m φ108 × 6 mm large pipe shed was constructed before the excavation, with a circumferential spacing of 40 cm. For severe topographic bias, a 10-m C25 concrete retaining wall was designed on the right side of the tunnel portal. In the excavation process, auxiliary measures such as pre-grouting pipe reinforcement and grouting reinforcement with lengthened anchor in the interlaid rock were also adopted. The 3-m φ42 × 4 mm hot-rolled seamless steel pipes were used in the pre-grouting pipe with an extrapolation angle of 15° and a circumferential spacing of 40 cm. The 5-m R25 hollow grouting anchor bolts were arranged at distances of 75 cm in the longitudinal direction and 100 cm in the circumferential direction to reinforce the interlaid rock. The support measures for the right tunnel portal are shown in Fig. 4.

3 Failure of supporting structures at the portal

3.1 Failure of the slope and retaining wall

When the right tunnel was excavated to 8 m, the construction of the retaining wall was partially completed, and the left tunnel portal began to excavate. When the left tunnel
was excavated to 2 m, two obvious transverse cracks appeared on the slope above the right tunnel portal. The two cracks continued to develop and then extended to the ground surface during the construction, as shown in Fig. 5. A large number of messy microcracks also appeared at the slope above the left tunnel portal. Similarly, the wall body of the retaining wall cracked at approximately 1.6 m and 5.3 m away from the ground (Fig. 5), and the crack width expanded gradually with the progress of construction. However, cracking of the slope and retaining wall did not cause the construction unit to pay sufficient attention. After rebrushing the slope and repairing the retaining wall cracks, the construction of the tunnel portal continued.

3.2 Cracking and block falling of the primary support concrete

Primary support concrete cracks first appeared at the right tunnel portal and were mainly concentrated on the left arch shoulder and waist. When the right tunnel was excavated to YK39 + 902–YK39 + 910, more serious cracks appeared in the primary support concrete, which were mainly distributed at the vault, arch shoulder, and arch waist. Even a large area of concrete fell off on the right side of the vault at YK39 + 902, as shown in Fig. 6. Meanwhile, a large number of small cracks with a width of 0.3–5 mm appeared in the primary support concrete of ZK39 + 870–ZK39 + 890 in the left tunnel, the majority of which were circumferential cracks.
3.3 Cracking of the invert

Soon after the primary support concrete cracked, the construction workers found cracks in the invert filling layer at YK39 + 866–YK39 + 886.5 and ZK39 + 849–ZK39 + 857 in the right and left tunnels, respectively, while cleaning the sediment on the ground. The crack in the right tunnel invert was approximately 20.5 m long and extended along the longitudinal...
direction of the tunnel. The location of the crack was approximately 1.3 m near the right side wall, with a width of approximately 3–10 mm. Many microcracks appeared on the right side of the tunnel axis, and the number gradually increased with excavation (Fig. 7a). The crack in the left tunnel invert was approximately 8.0 m long and extended along the longitudinal direction of the tunnel, with a width of approximately 1–6 mm (Fig. 7b).

### 3.4 Cracking of the secondary lining concrete

Cracking of the secondary lining concrete at the tunnel portal was also significant. The cracking locations were mainly concentrated at YK39 + 866–YK39 + 910 and ZK39 + 848–ZK39 + 877 in the right and left tunnels, respectively. The cracking of the lining concrete in the right tunnel was significantly more severe than that in the left tunnel. The lining concrete at the right tunnel portal cracked after the completion of pouring, and the scale of the cracks continued to expand with the continuous excavation of the right tunnel and the beginning of the left tunnel excavation. To study the causes and distribution characteristics of cracks, field investigations and statistics of secondary lining concrete cracks were carried out. Figure 8 presents the statistics of the secondary lining concrete cracks at YK39 + 866–YK39 + 910 in the right tunnel.

There were two main types of cracks in the secondary lining concrete: longitudinal and circumferential cracks; longitudinal cracks accounted for more than three-quarters of the total cracks. The cracks were mainly concentrated at the vault, arch shoulder, and arch waist, accounting for 86.21% of the total crack number, and the cumulative length accounted for 83.5% of the total crack length. Lots of cracks appeared at the vault, including longitudinal and circumferential cracks. The cracks appearing at the arch shoulders and waists were longitudinal cracks. Although the number of cracks on the two sides was slightly different, the right concrete cracking was more severe than that on the left. Especially in YK39 + 882–YK39 + 900, a longitudinal crack with a length of 18.2 m and a maximum width of 4.2 mm appeared at the right arch waist (Fig. 8). The maximum crack depth was 245 mm, which was approximately half the thickness of the lining. This crack tends to extend and develop in the excavation direction during tunnel construction. The number and scale of the cracks at the left arch shoulder and waist were relatively small, and no cracks appeared after YK39 + 896. Compared to the right tunnel, there were only two obvious longitudinal cracks in the lining concrete at the left tunnel portal. One was located at the vault, with a length of approximately 9.0 m and maximum width of 0.45 mm; the other was

![Fig. 7 Cracking of the invert at the tunnel portal](image)
located at the right arch waist, with a length of approximately 6.9 m and maximum width of 0.38 mm.

4 Analysis of failure mechanism of supporting structures

During the construction process of the tunnel portal, distresses such as slope failure, blocks falling off the primary support concrete, and cracking of the invert and secondary lining concrete occurred frequently. These cracks posed a serious danger to tunnel structure safety and buried potential safety hazards for subsequent tunnel construction and operation (Luo et al. 2020; Zhang et al. 2020; Zhao et al. 2021, 2022; Song et al. 2022). Previous research results (Lai et al. 2017; Li et al. 2020, 2021; Liu et al. 2020; Jing et al. 2021; Shi et al. 2021; Wang et al. 2021) have indicated that the failure mechanisms of the surrounding rock and tunnel supporting structures are complex and diverse and are affected by various factors. In this study, the failure mechanism of supporting structures at the tunnel portal
was analyzed by field investigation, deformation monitoring, and numerical simulation to provide a theoretical basis for subsequent treatment measures.

### 4.1 Field investigation and monitoring

After a large number of cracks appeared in the secondary lining concrete, the construction unit immediately stopped the excavation and organized multiple experts to conduct a detailed investigation on the failure causes of the tunnel portal supporting structures. Combined with the laboratory tests, field deformation monitoring, and the statistics of lining concrete cracks, it was found that the broken and loose gravel soil in the shallow buried area and severe bias on the terrain are the main reasons for the failure of supporting structures. The field monitoring results showed a significant uneven deformation at the right tunnel portal during excavation. Figure 5 presents the deformation of typical positions at the tunnel portal only after the right tunnel excavation (white arrow). The monitoring point at the vault was moved up by 6 mm, the left arch waist sunk by 16 mm, and the right arch waist shifted 6 mm to the right. After the tunnel portal was excavated, the gravel soil at the deeply buried side produced a relatively large loose load on the left side of tunnel, causing the entire tunnel structure to shift from the upper left to the lower right, which led to obvious settlement at the left arch waist while upward movement occurred at the vault. As the right side of tunnel was shallowly buried, the load generated by the gravel soil was small, and the back pressure generated by the retaining wall was insufficient, which caused the right translation at the right arch waist, and the retaining wall cracked when limiting the tunnel structure deformation. According to the statistical results (Fig. 8), concrete cracking of the left arch was mainly concentrated from the portal section to YK39 + 896, and cracking was less likely to occur beyond the YK39 + 896. This indicated that as the excavation progressed, the buried depth above the left arch gradually increased, and the bearing arch gradually formed in the surrounding rock. In contrary, the right arch concrete was less cracked at the tunnel portal, but cracks gradually appeared more frequently after YK39 + 875. The surrounding rock at the shallow buried side was not deep enough to form a bearing arch, and the loose load acted directly on the supporting concrete. The supporting structures of the tunnel in the same section underwent a severe bias load, resulting in an obvious asymmetrical deformation of the structures, which could be prone to cracking and spalling of the concrete.

The arrival of the rainy season and infiltration of surface water were another main triggers for support failure at the tunnel portal. Soon after the tunnel portal excavation, a rainy season occurred, and several heavy rainstorms were experienced during tunnel construction. The water level of the river rose and overflowed, and surface water collected in the gravel soil layer. The surface and slope cracks caused by tunnel portal excavation provided channels for the infiltration of surface water. During the excavation process of the right tunnel portal, some construction personnel successively reported that water leakage and dripping occurred at many positions on the supporting concrete. Owing to the untimely treatment of cracks at the slope and surface, atmospheric precipitation and surface water infiltrated through the cracks, resulting in the gradual saturation of gravel soil. The physical and mechanical properties of the gravel soil deteriorated rapidly after saturation, causing a significant increase in the loose load of the surrounding rock acting on the tunnel structure. Consequently, the supporting structures at the portal were subjected to a more serious bias load, and the asymmetrical deformation increased, which ultimately led to the failure
and destruction of the slope and supporting structures (Yang et al. 2020; Chen et al. 2020; Zhang et al. 2022; Xie et al. 2021; Duan et al. 2021; Huang et al. 2022).

Another trigger for the support failure was that the excavation faces of the two tunnels were staggered too close. The minimum distance between the left and right tunnel axes at the portal section was only 13.2 m (i.e., 1.2 times the tunnel width), which is a typical small clear-distance tunnel. In the actual construction process at the tunnel portal, the influence of the small clear distance was not fully considered. Construction of the left tunnel portal began when the right tunnel was excavated to 8 m. The only supporting measure for the interlaid rock was the 5-m long hollow grouting anchor bolts (Fig. 4). Soon after the left tunnel portal was excavated, cracks appeared on the slope, retaining wall, and primary support concrete in the right tunnel. This indicated that the subsequent tunnel excavation had a significant negative impact on the structural stability of the prior tunnel and intensified the failure of the supporting structures in the prior tunnel.

4.2 Numerical simulation

To further reveal the failure mechanism of the supporting structures at the tunnel portal, the finite difference software FLAC3D 5.0 was used to simulate and analyze the dynamic tunnel portal excavation process.

4.2.1 Model establishment and boundary conditions

A three-dimensional numerical model was established according to the geological exploration data and slope topography conditions of the tunnel portal, as shown in Fig. 9. To eliminate the boundary effect, the distance between the model boundaries and tunnel boundaries was thrice the tunnel width, and the length of the entire model was 100 m. The distance between the lower model boundary and tunnel arch bottom was thrice the tunnel height, and the upper model boundary was taken to the ground surface. According to the field investigation, the failure of the supporting structures mainly occurred in YK39 + 866–YK39 + 910; thus, the longitudinal length of the model was 60 m. In the numerical simulation, the normal displacement of the four vertical
boundaries was constrained, the displacement in all directions of the bottom boundary was constrained, and the top was a free boundary. The established numerical model included 296,005 elements and 167,004 nodes in total.

In the numerical simulation, the construction process and support parameters at the tunnel portal were consistent with those in the actual project. Specifically, the pipe shed was installed before tunnel excavation, and the retaining wall was constructed on the right side of the right tunnel (Fig. 9). Upper and lower bench method was adopted to excavate the right tunnel first, and the construction step length of each cycle was 1.0 m. The primary support was applied immediately after tunnel portal excavation, while the secondary lining was applied 12 m behind the tunnel face. The excavation of the left tunnel was 8 m behind the right tunnel face.

4.2.2 Material parameters and simulation cases

In this simulation, the failure of the surrounding rock followed the Mohr–Coulomb yield criterion. The elastic element was used for the retaining walls and concrete. The simulation parameters are listed in Table 1. The cable element was used to simulate the pre-grouting pipes and anchor bolts. The grouting reinforcement effect was achieved by increasing the strength of the surrounding rock in the reinforcement area. The beam element was used to simulate the pipe shed. The specific simulation parameters for the pipes, anchor bolts, and pipe shed are listed in Tables 1 and 2.

According to the field investigation results, the rainy season occurred soon after tunnel portal excavation. Atmospheric precipitation and surface water infiltrated the surrounding rock through the cracks. Therefore, the influence of the water infiltration was also considered in the simulation. Based on the model shown in Fig. 9, three excavation cases were simulated: ① only the right tunnel was excavated; ② the right tunnel was excavated first, and the left tunnel was excavated 8 m behind the right tunnel face; and ③ based on Case II, considering the influence of surface water infiltration, the saturated gravel soil parameters were used to replace the original surrounding rock parameters. The parameters of the saturated gravel soil were obtained from laboratory triaxial tests, as listed in Table 1.

| Table 1 | Parameters of surrounding rock, retaining wall and supporting structures |
|---------|---------------------------------------------------------------|
| Material types | Unit weight (kN/m³) | Elastic modulus (MPa) | Poisson’s ratio | Cohesion (kPa) | Internal friction angle (°) |
| Gravel soil | 18.5 | 50 | 0.42 | 20 | 18.6 |
| Saturated gravel soil | 21.6 | 32.5 | 0.45 | 12.6 | 13.2 |
| Slate with sandstone | 20.4 | 1560 | 0.24 | 110 | 39.2 |
| Retaining wall | 25.0 | 28,000 | 0.2 | | |
| Primary support | 25.0 | 28,000 | 0.2 | | |
| Secondary lining | 25.0 | 31,000 | 0.2 | | |
| Advanced small pipe | 78.5 | 210,000 | 0.3 | | |
| Anchor bolt | 78.5 | 210,000 | 0.3 | | |
4.2.3 Analysis of simulation results

According to field investigation and monitoring, the failure of the supporting structures in the right tunnel was far more serious than that of the left tunnel. The lining cracks in the right tunnel developed continuously with the tunnel excavation, whereas the lining cracks in the left tunnel had no development trend. Therefore, the simulation results of the surrounding rock and supporting structures in the right tunnel were analyzed emphatically. Two typical sections of YK39 + 871 and YK39 + 902 in the right tunnel were selected to analyze the deformation and stress characteristics of the surrounding rock and supporting structures.

Figure 10 presents the deformation of the surrounding rock and supporting structures at YK39 + 871 for the three cases. Table 3 shows the field monitoring and numerical simulation results of the surrounding rock deformation at various positions in Case I, in which the vertical deformation was positive upward and the horizontal deformation was positive to the right. Combined with Fig. 10a and Table 3, after the right tunnel portal excavation, the deformation of the surrounding rock and supporting structures presented an obvious asymmetry owing to the influence of topographic bias. Maximum deformation occurred at the left arch shoulder, reaching 25.181 mm. As the left side of the tunnel was deeply buried, the tunnel structure shifted to the lower right under the significant loose load generated by the gravel soil on the left side. These deformation characteristics were consistent with the field monitoring results. The deformation of the retaining wall showed the characteristics of rotating clockwise with the wall corner as the center, i.e., the deformation value gradually increased from the wall corner to the wall top. From Table 3, it can be observed that the simulation results of the surrounding rock deformation at various positions in the YK39 + 871 section were in good agreement with the monitoring results. This indicated that the established numerical model and simulation parameters conformed to the actual project, and the calculation results were reasonable and feasible.

As shown in Fig. 10a, b, the left tunnel excavation had an obvious negative influence on the stability of the right tunnel. After the left tunnel portal excavation, the surrounding rock deformation in the right tunnel increased significantly, and the asymmetrical deformation and the tendency of right tunnel to shift to the right were more pronounced. The deformation at the left arch shoulder increased from 25.181 to 36.663 mm. This shows that under a small clear-distance tunnel arrangement, the following tunnel excavation could aggravate the prior tunnel deformation, which was not conducive to prior tunnel stability. Comparing Fig. 10a–c, after the gravel soil was immersed and deteriorated, the asymmetrical loose load on the tunnel portal increased sharply. The deformation at the left arch shoulder surged to 90.878 mm, and the deviation trend of the tunnel structure from the deeply buried side to the shallow buried side was further intensified.

Figure 11 presents the deformation of surrounding rock and supporting structures at YK39 + 902 under three cases. Comparing Figs. 10 and 11, the surrounding rock load
borne by tunnel structures gradually increased with the thickness of overlying surrounding rock gradually increasing, resulting in the deformation of the surrounding rock and supporting structures also gradually increasing. The deformation was still asymmetric, and the maximum deformation position of the surrounding rock was transferred from the left arch shoulder to the right side of vault (Figs. 11a, b). Comparing Fig. 11a, b, the following tunnel excavation aggravated the surrounding rock deformation of the prior tunnel, and the maximum deformation at the vault increased from 58.525 to 76.385 mm. After the gravel soil layer was immersed and deteriorated, the surrounding rock deformation at the vault surged to 99.296 mm, and the maximum deformation shifted from the vault to the slope surface, which indicated that the slope instability was prone to occur under Case III. Thus, auxiliary measures for slope waterproofing and drainage and stratum stabilization before excavation were recommended.

Figure 12 presents the tensile stress of the retaining wall at YK39 + 871, and Fig. 12a shows the tensile stress nephogram of the retaining wall under Case III. After the tunnel portal was excavated, the right tunnel tended to shift to the right under the loose rock load on the deeply buried side, and the retaining wall built on the shallow buried side could effectively limit the tunnel deviation. However, the significant deformation of the tunnel could cause obvious tensile stress on the retaining wall body, especially at points A and B in Fig. 12a. Figure 12b shows the tensile stress at points A and B under various cases. When only the right tunnel was excavated, the maximum tensile stress on the
retaining wall appeared at point B, reaching 0.358 MPa. After the left tunnel was excavated, the maximum tensile stress was transferred to point A, reaching 1.071 MPa. After the gravel soil was immersed and deteriorated, the tensile stress at point A increased sharply to 2.031 MPa, which had exceeded the ultimate tensile strength of C25 concrete (JTG 3370.1-2018). Concurrently, the tensile stress at point B also increased to 1.847 MPa. Comparing Fig. 12a with Fig. 5, the tensile stress concentration position of the retaining wall obtained by numerical simulation was consistent with the cracking position of the retaining wall in the actual project. This indicated that the retaining wall body cracked due to excessive tensile stress in the process of limiting the tunnel deviation.

Figures 13 and 14 presents the tensile and shear stresses of primary support concrete in two typical sections, respectively. From Figs. 13a and 14a, affected by the loose rock load on the deeply buried side, the maximum tensile stress and shear stress at YK39 + 871 appeared between the left arch shoulder and waist under Case III, i.e., the position with the maximum deformation of tunnel. After the left tunnel was excavated, the maximum tensile stress increased from 1.703 to 1.952 MPa (Fig. 13c), which was close to the ultimate tensile strength of C25 concrete (JTG 3370.1-2018). The maximum shear stress also increased from 1.501 to 2.328 MPa (Fig. 14c). The concrete at the arch shoulder and waist was prone to cracking due to obvious tensile and shear stresses. As the excavation progressed, the thickness of the overlying soil increased, and the position subjected to the maximum tensile stress and shear stress on the primary support concrete shifted to the right shoulder (Figs. 13b, 14b), i.e., the position with the maximum deformation of the tunnel at YK39 + 902. After the left tunnel excavation, the maximum tensile and shear stresses increased from 1.387 and 1.042 MPa to 1.826 and 1.728 MPa, respectively (Figs. 13c, 14c). The maximum tensile and shear stresses increased rapidly to 2.555 and 2.881 MPa, respectively, when the surrounding rock was immersed and deteriorated. Under such significant tensile and shear stresses, the primary support concrete was easily cracked, peeled, and even collapsed. The stress concentration position in numerical simulation was also consistent with the position of cracking and falling blocks of the primary support concrete in the actual project (Fig. 6).

Figures 15 and 16 present the tensile and shear stresses of secondary lining concrete at two typical sections, respectively. According to Figs. 15a and 16a, the maximum tensile stress and shear stress of secondary lining concrete at YK39 + 871 appeared between the left arch

| Table 3 | Comparison between field monitoring and numerical simulation of deformation at YK39 + 871 section (unit: mm) |
|---------|------------------------------------------------------------------------------------------------------|
| Positions | Field monitoring value | Numerical simulation value |
| **Vault** | | |
| Vertical deformation | 6.0 | 4.78 |
| Horizontal deformation | 9.0 | 9.75 |
| **Left arch waist** | | |
| Vertical deformation | −16.0 | −15.36 |
| Horizontal deformation | 18.5 | 19.23 |
| **Right arch waist** | | |
| Vertical deformation | 1.5 | 1.27 |
| Horizontal deformation | 6.0 | 6.42 |
shoulder and waist in Case III, which was consistent with the stress concentration position of primary support concrete. From Figs. 15c and 16c, after the excavation of the left tunnel and the water immersion deterioration of the surrounding rock, the tensile and shear stresses increased rapidly, and the tensile stress reached 2.107 MPa in Case III, which was significantly close to the ultimate tensile strength of C30 concrete (JTG 3370.1-2018). Therefore, the secondary lining concrete at the left arch shoulder and waist was likely to crack due to excessive tensile stress (Figs. 15a, 16a). It was also consistent with the crack distribution characteristics in the actual project, i.e., there was a significantly higher number of secondary lining cracks at the left arch shoulder and waist at the tunnel portal than that in other positions in Fig. 8a. As the excavation progressed, the maximum tensile stress was transferred to the right arch shoulder (Fig. 15b). The maximum tensile stress increased to 2.292 MPa in Case III (Fig. 15b, c), which had exceeded the ultimate tensile strength of C30 concrete. The secondary lining concrete at YK39+902 had shear stress concentration at the right arch shoulder and waist (Fig. 16b). The maximum shear stress could reach 2.48 MPa in Case III (Figs. 16b, c), and the concrete at these positions was prone to cracking due to excessive shear stress. From Figs. 15b and 16b, the tensile and shear stresses concentration positions coincided with the concentrated distribution positions of the secondary lining cracks at a similar section in Fig. 8a.
5 Treatment measures

From the results of the field investigation, deformation monitoring and numerical simulation, it was found that shallow buried and asymmetrical loading, surface water infiltration, and improper construction were the main triggers for the failure of supporting structures at the tunnel portal. Combining the failure mechanism and considering the continuous development of cracks in the tunnel, a series of treatment measures that first stabilize the stratum and then treat the cracks were proposed.

5.1 Specific treatment measures

5.1.1 Stratum stabilization measures

First, during the tunnel portal excavation, the shallow buried side of the right tunnel was backfilled and compacted in time (Fig. 17). The backfilling height was 3.0 m higher than that of the tunnel vault, the backfilling slope rate was 1:1.5, and the compactness was greater than 90%. Slope rubble was then adopted for slope protection.

Second, according to the characteristics of the small clear-distance tunnel, the stability of the interlaid rock was considered the construction focus during excavation. Ground surface grouting was performed for the interlaid rock in the shallow buried section of the tunnel (YK39 + 856–YK39 + 914.5). The grouting pipes were made of φ89 mm × 6 mm hot-rolled seamless steel pipes with a spacing of 75 cm × 75 cm and a quincunx arrangement. Each pipe penetrated 5.0 m into the bedrock layer, as shown in Fig. 18. A cement slurry with a water-cement ratio of 1:1 was adopted as the grouting slurry, the grouting pressure was 0.5–1.5 MPa, and the diffusion radius of the single-hole slurry was greater than 100 cm. Because of the high cost of ground surface grouting in non-shallow buried sections, grouting in the tunnel was adopted to strengthen...
The 5-m long R25 hollow grouting bolts were replaced by 6-m long φ42 mm × 4 mm hot-rolled seamless steel pipes arranged at 75 cm in the longitudinal direction and 100 cm in the circumferential direction. The grouting parameters were consistent with the ground surface grouting.

Third, to reduce the impact of precipitation and surface water infiltration, intercepting ditches were constructed on the top slope of the tunnel portal. To reduce the negative impact of the following tunnel excavation on the structural stability of the prior tunnel, the excavation was conducted in strict accordance with the principle that the following tunnel face must be at least 20 m behind the prior tunnel lining section. During portal excavation, the principle of short excavation and short advance should be followed, and the invert must be closed into a ring in time.

5.1.2 Cracks treatment measures

To address the distresses causes by slope and surface cracks, trenches with a depth of 1.0 m and a width of 0.6 m were dug along the direction of cracks; the trenches were backfilled.
and compacted and finally covered with waterproof geotextile to prevent surface water infiltration. Treatment measures for the cracking of the retaining wall include repairing the retaining wall cracks and demolishing and rebuilding the severely cracked sections. The humus and soft soils at the base of the retaining wall were removed, and rubble was used to replace and compacted. Then, four 3.5-m long foot locking bolts were added at each steel arch in the tunnel portal and installed obliquely downward at the right arch foot at 30°–45°.

For the large area falling of the primary support concrete at YK39 + 902–YK39 + 910, the construction spacing of the steel arches was shortened from 75 to 50 cm. The length of small grouting pipes was increased from 3.0 to 5.0 m. For the invert cracks at the portal, the concrete within 75 cm on both sides of the cracks was removed to the depth of the invert bottom and then grouted with cement slurry. Subsequently, C25 concrete was used to fill the top surface of the backfill layer.

According to the cracking degree of the secondary lining, different treatment measures were adopted to repair the cracks. ① For cracks with a depth of less than 5 cm, the low-pressure rapid grouting method was used to fill the cracks. A U-shaped groove with a width of 2 cm and a depth of 1 cm was cut out at the crack, the groove surface was

![Shear stress nephogram of the primary support concrete at YK39+871 in Case III (unit: MPa).](image1)

![Shear stress nephogram of the primary support concrete at YK39+902 in Case III (unit: MPa).](image2)

![Maximum shear stress at the primary support concrete.](image3)

**Fig. 14** Shear stress of the primary support concrete
wiped with acetone, the groove was filled with epoxy resin mortar, and the crack surface was finally painted with epoxy resin slurry. For cracks with a depth of more than 5 cm but less than 15 cm, grooves with a width of 50 cm were cut on both sides of the cracks at the same depth as the crack depth, and then, C30 concrete was used for backfilling and pouring. For cracks with a depth greater than 15 cm, the secondary lining at the cracked section was demolished and reconstructed (Fig. 19), and the φ22 circumferential steel bars were adjusted to φ28 in the lining.

Finally, crack observation was taken as the focus of monitoring and provided timely feedback on crack development. Combined with other monitoring data, the treatment effect was fully understood to dynamically adjust the treatment measures.

5.2 Evaluation of treatment measures

To better understand and evaluate the effect of the treatment measures, steel string concrete strain gauges were arranged in the secondary lining concrete at YK39+890 (lining demolition and reconstruction section) and YK39+930 (subsequent excavation section). The lining concrete stresses at the vault, arch shoulder, arch waist, and arch foot were

![Tensile stress nephogram of the secondary lining concrete at YK39+871 in Case III (unit: MPa).](image)

![Tensile stress nephogram of the secondary lining concrete at YK39+902 in Case III (unit: MPa).](image)

![Maximum tensile stress at the secondary lining concrete.](image)

Fig. 15  Tensile stress of the secondary lining concrete
monitored and analyzed. The buried positions and monitoring results are shown in Fig. 20. After the implementation of the treatment measures, the stress distribution of the secondary lining concrete was still asymmetric, and the concrete stress at the shallow buried side was more unfavorable than that at the deeply buried side (Figs. 20b, c). The maximum tensile stress still occurred at right the arch shoulder, reaching 1.089 MPa and 0.482 MPa, but the value had not yet reached the ultimate tensile strength of the C30 concrete, which had a certain safety reserve. This indicated that the secondary lining stress was significantly ameliorated. Moreover, the uneven stress of the lining concrete was alleviated to a certain extent with the construction of auxiliary measures and the increase in buried depth, and the concrete at the vault and left shoulder no longer underwent tensile stress, which meant that the surrounding rock quality was strengthened effectively, and the bearing arch had gradually formed.

After the stratum stabilization measures were completed, the field monitoring results of the surrounding rock tended to be stable, and the maximum settlement value did not exceed 30 mm. After the cracks were repaired, routine inspection of the lining cracks was performed every day. The inspection indicated that there was no obvious cracking in the

![Image](image_url)
Fig. 17 The shallow buried side was backfilled and compacted

concrete during the tunnel construction and no sign of water seepage in the rainy season. Overall, the proposed and implemented treatment measures achieved the expected results.

6 Conclusions

In this study, a shallow buried and asymmetrically loaded tunnel with a small clear-distance in northwest China was considered for the case study. In response to the distresses that occurred at the tunnel portal such as instability of the slope, spalling of the primary support, and cracking of the secondary lining, in-depth research on the failure
mechanism of support structures was carried out combining with field investigation, statistical analysis, monitoring measurement, and numerical simulation. Subsequently, a series of treatment measures were proposed, and their effects were evaluated. The following conclusions were drawn from this study:

(1) The topography at the tunnel portal was shallow buried and asymmetrically loaded, and the gravel soil layer was broken, loose, and unstable. Construction coincided with the rainy season, and surface water seeped into the surrounding rock along the slope and surface cracks, further deteriorating the physical and mechanical properties of the surrounding rock. After tunnel portal excavation, the supporting structures underwent significant asymmetrical loose loads, resulting in cracking and failure owing to excessive local deformation. In addition, the staggered distance between the left and right tunnels during the excavation process was too short, and the interlaid rock was not effectively reinforced. Consequently, the excavation of the following tunnel had an obvious negative impact on the stability of the prior tunnel and aggravated the failure of the supporting structure in the prior tunnel.

(2) Field monitoring and simulation results indicated that after tunnel portal excavation, the loose load generated by the surrounding rock at the deeply buried side could cause the entire tunnel to deviate to the shallow buried side. In the process of limiting the tunnel deformation, the retaining wall produced a clockwise rotation deformation around the wall corner, and the partial wall cracked owing to the obvious tensile stress. After the surrounding rock was immersed and deteriorated, serious asymmetrical deformation and stress concentration occurred at the primary support and secondary lining owing to the obvious asymmetrical loose load, resulting in the cracking and failure of the concrete.

(3) Combined with the failure mechanism of supporting structures and the characteristics of the continuous development of cracks, a series of treatment measures that stabilize the stratum first and then treat the cracks. The specific measures included backfilling and compaction of the shallow buried side of the tunnel portal, reinforcing the interlaid rock by ground surface grouting, constructing intercepting ditches at the slope top, and excavating the following tunnel 20 m behind the lining section of the prior tunnel. The results of the field inspection and stress monitoring indicated that after the treatment measures were implemented, the supporting concrete stress was effectively ameliorated.
No obvious cracking or water leakage occurred during the subsequent tunnel construction, indicating that the proposed measures achieved a good treatment effect.

**Fig. 20** Field monitoring of secondary lining concrete stress (unit: MPa, positive values indicate compressive stress, while negative values indicate tensile stress)

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Declarations

Conflict of interest The authors declare that there is no conflict of interest in the manuscript.

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