

Article

Safety Analysis of Secondary Lining of Yulinzi Tunnel Based on Field Monitoring

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Abstract: In order to assess the safety of the secondary lining in water-rich Loess Tunnel, this study relies on the Yulinzi Tunnel project to continuously monitor the stress of the steel reinforcement in the secondary lining and analyze the temporal variation of the steel strain. Based on this data, the temporal variation and distribution characteristics of the internal forces of the structure were obtained through section force calculation. The safety factor of the secondary lining structure was evaluated by calculating the safety factor of the structure. It was found that the variation of the safety factor with time conforms to the exponential function, and the fitting results of different measurement points were good. This could be used to predict the future safety factor of the tunnel and evaluate its long-term safety, which has practical guidance significance for actual engineering projects. In addition, the long-term stress and structural deformation characteristics of the tunnel were obtained through numerical simulation.

Keywords: loess tunnel; field monitoring; secondary lining; regression analysis



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1. Introduction

With the improvement of China's infrastructure, the transportation network in the northwest region has also developed rapidly. However, due to the widespread distribution of loess in the northwest region, the number of loess tunnels is increasing. Due to its unique structural characteristics such as particle morphology and arrangement, high porosity, soluble salt content, and water sensitivity, loess undergoes significant changes in strength and deformation as its moisture content increases [1–4]. Due to the softening of surrounding rock in loess tunnels, some water-rich loess tunnels may experience excessive deformation, water leakage, cracking, and collapse in practical engineering [5–9]. The secondary lining is always in an unstable state [10], which will have a negative impact on the long-term stability of the structure and seriously affect the safe operation of the tunnel. In order to ensure the safe operation of loess tunnels in water-rich areas, it is particularly important to understand the internal force law of loess tunnels in water-rich areas and to study the safety evaluation of secondary lining structures in water-rich areas.

Most studies on the deformation law of tunnels have adopted field monitoring methods [11,12]. The same approach has also been used to study the surrounding rock pressure and deformation of loess tunnels. Many scholars have proposed revised formulas for calculating the surrounding rock pressure of loess tunnels, based on extensive monitoring data and statistical patterns of stratum damage [13–15], in order to guide the structural design of loess tunnels. However, some secondary lining structures of loess tunnels still crack over time, indicating that the surrounding rock pressure of loess tunnels has certain time-dependent characteristics. In response to this problem, scholars have studied the deformation law of the surrounding rock of loess tunnels and summarized the deformation law of the tunnel surrounding rock based on a large number of monitoring and measurement data. It is generally believed that after the surrounding rock is excavated,

the deformation of the surrounding rock changes with time. In general, there are three stages: the rapid deformation stage, continuous deformation stage, and slow deformation stage (creep deformation stage) [16–20]. It can be seen that the slow deformation of the surrounding rock of the loess tunnel over time is its most important feature, indicating that the loess has obvious creep characteristics, which is the main reason for the time-varying characteristics of the pressure of the surrounding rock of the tunnel. The influence of the creep of loess surrounding rock on the tunnel structure must be considered in the safety evaluation of the secondary lining structure of loess tunnels.

On this basis, scholars have carried out significant research on the secondary lining deformation of loess tunnel structures. Through the field investigation of an existing loess tunnel structure, Song and Lai [21] found that the tunnel vault cracks were more serious than the side wall cracks. Due to the infiltration of farmland irrigation and rainwater, the surrounding rock pressure gradually increased, and the cracks were constantly in development. Xie [22] systematically monitored the construction and operation period of the loess tunnel in a water-rich area and pointed out that the disturbance of tunnel construction, irrigation of farmland, and heavy rainfall significantly affected the mechanical properties of the support structure through concrete. Through data analysis of the surface strain gauge, the authors drew the conclusion that the stress of the secondary lining structure was continuously increasing. He [23,24] used the model test method test to study the gradual deterioration process of the secondary lining under the action of creep and obtained the four stages of deterioration: the elastic stage, initial failure stage, ultimate failure stage, and unstable stage. In addition, Zhao [25] measured the changes in the surrounding rock pressure and concrete strain of the secondary lining of a loess tunnel with time and obtained that the surrounding rock pressure first increased and then decreased, then tended to be stable. Regarding the secondary lining arch, the top was in tension, and the rest of the body was in compression.

Based on the above research results, it can be seen that due to the creep of the loess surrounding rock, the stress on the secondary lining structure of the tunnel increases gradually, which poses a huge hazard to tunnel safety. As a concealed engineering project, the complexity of engineering geology makes it difficult to determine the load of the surrounding rock on the secondary lining structure. The reliability of judging the safety status of the tunnel structure through the load-structure method is not high. This paper was based on the field investigation of the Yulinzi Tunnel project and numerical simulation results of long-term stress on the tunnel. The strain monitoring results of the inner and outer steel bars of the lining, the law of strain variation, long-term stress, and the safety of the loess tunnel lining are discussed and analyzed.

2. Project Overview

This research relies on the Tianyong Expressway project, which is located in Qingyang City, eastern Gansu Province. The lengths of the left and right lines are 1900 m and 1985 m, respectively, and the maximum buried depth is 112.35 m. The net distance between the left and right holes is 21~38 m.

The lithology of the strata in the tunnel site area is divided into upper and lower layers. The upper part is mainly the Quaternary Upper Pleistocene (Q3eol) Malan loess and the lower part is mainly the Middle Pleistocene (Q2eol) Lishi loess sandwiched with multiple layers of ancient soil. Where there is groundwater, the surrounding rock conditions are poor, belonging to V-level surrounding rocks.

The excavation of the tunnel adopted the excavation method of three steps and seven steps of flowing water. The tunnel adopted composite lining, and the initial support adopted C20 shotcrete with a thickness of 26 cm and an I2web I-shaped steel arch frame, without system anchors. The secondary lining was made of C35 molded concrete, and the thickness of the inverted arch and side wall was 50 cm. Geotextile and waterproof boards were laid between the primary support and secondary lining.

At the same time, Yulinzi Tunnel encountered many difficulties during the construction process. These difficulties mainly include the following three aspects:

- (1) The tunnel was surrounded by rock with low strength, classified as Class V rock. Due to the large amount of seepage, the self-bearing capacity of the surrounding rock was reduced. Tests showed that the soil moisture content was between 21% and 28%, with an average of 23.8%;
- (2) The deformation of the surrounding rock of the tunnel was large. The average settlement of the left line of the tunnel was 361.04 mm, and the maximum settlement could reach 904.6 mm; the average settlement of the right line was 235.15 mm, and the maximum settlement could reach 473.1 mm. The reserved deformation value of the tunnel design was exceeded, which caused serious damage to the primary support structure of the tunnel, distortion of the steel frame, cracking of the shotcrete, and severe violation of the primary support;
- (3) Through on-the-spot investigation, cracks in different degrees were observed in the secondary lining structure of the Yulinzi Tunnel within 1–2 years, as shown in Figure 1. Field statistics found that there were many circumferential cracks at the side walls and vaults of the tunnel. The width of the cracks at the tunnel arch and arch foot was usually less than 0.5 mm, and a few cracks exceed 1 mm in width. The pictured crack was a longitudinal crack; the width of the crack at the inverted arch could reach up to 3 mm, and the crack length can reach up to 48 m.



Figure 1. Field survey photos.

3. Data Monitoring

3.1. Monitoring Program

To evaluate the safety of the secondary lining of the tunnel, we further calculated the axial force and bending moment of the secondary lining by monitoring the reinforcement stress of the secondary lining structure and finally obtained the safety factor of the secondary lining. To make the monitoring structure representative, the secondary lining structure of the section with poor surrounding rock conditions was selected for the layout. The monitoring sections selected were the positions of ZK280+325 and ZK280+328 at the entrance of the left hole, as shown in Figure 2.

To truly reflect the internal force of different areas on the same section of the secondary lining structure, steel bar stress sensors were, respectively, arranged at the vault, arch shoulder, arch waist, and arch foot of the tunnel. When arranging the steel bar stress meter, it is necessary to arrange sensors at the corresponding positions of the steel bars on both sides of the section. There were 26 measuring points in the two monitoring sections, 6 in the vault area, 8 in the arch shoulder area, and 12 in the arch foot side wall area, as Figure 3 shows.

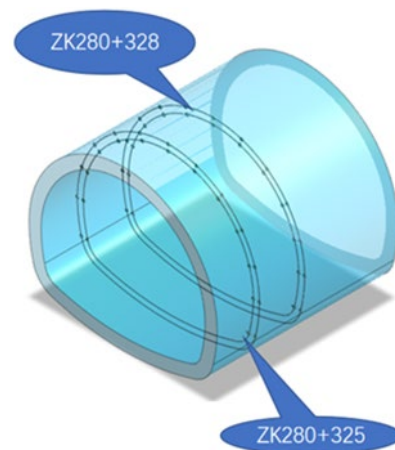


Figure 2. Longitudinal arrangement of sensors.

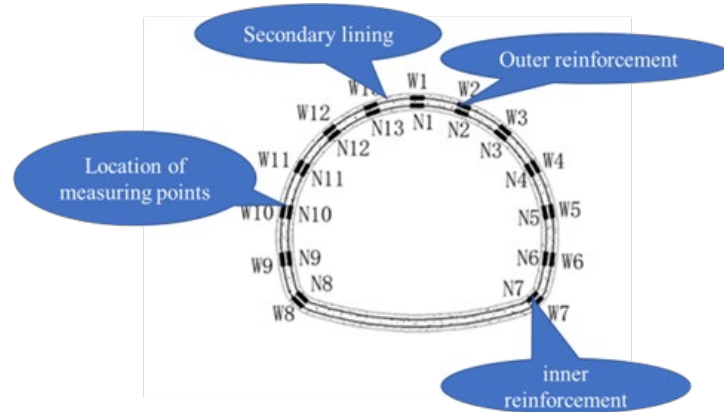


Figure 3. Lateral arrangement of sensors.

The sensor used in this measurement was the intelligent string-type steel bar stress gauge JMZX-416AT, which is mainly used in the steel bar stress measurement inside the reinforced concrete structure. For data collection, a comprehensive collection module with data receiving and storage functions and a DTU wireless transmission module were adopted. The data were exported through the comprehensive collection module and then sent to the user's computer through the wireless transmission module, which could realize the remote monitoring of data.

3.2. Monitoring Equipment

3.2.1. Smart String-Type Steel Stress Meter

The sensor used in this measurement was the smart string-type steel stress meter JMZX-416AT, which is mainly used for measuring the steel stress inside reinforced concrete structures. The measuring range of this stress meter was ± 200 MPa, with an accuracy of 1% FS.

(1) Accuracy Calibration Test of Steel Bar Stress Meter

As a method for monitoring the stress of steel bars in tunnel lining, the steel bar meter needs to use standard calibration instruments to calibrate its measurement accuracy to ensure the accuracy of the stress monitoring results of secondary lining steel bars.

The accuracy calibration test device mainly consisted of two parts, which were the steel bar meter loading device and the data acquisition device. The loading device of the steel bar meter had passed the accuracy testing standard. In this experiment, the tensile test of the steel bar stress meter was carried out by the loading device of the steel bar meter.

The data acquisition device adopted JMZX-3001L comprehensive tester, and the overall installation effect diagram is shown in Figure 4.



Figure 4. Layout of test equipment.

The steel bar stress meter number adopted the number of the measuring point arrangement, and the steel bar stress meter to be tested was fixed on the loading device through the bolts at both ends. First, the stress meter needed to be preloaded before the test starts, and the preload load was 50 MPa. After preloading, it was necessary to test the frequency when the load is 0 of the strain meter, and then load it in five levels within the range of 200 MPa, namely, 0 MPa, 50 MPa, 100 MPa, 150 MPa, and 200 MPa. Two loading tests for each steel bar stress meter were performed, increasing the load step by step, and the data was read after each level of load was applied for 15 s. The comprehensive tester could obtain the frequency value of the steel string under different loads of the steel strain meter, and the stress value could be converted through the frequency. The relationship between the frequency of the strain meter and the stress is shown in Equation (1):

$$P = KK_0(f_i^2 - f_0^2) \quad (1)$$

where P is the pressure; K is the calibration factor; K_0 takes the value 0.00071186; f_i is the measurement frequency; f_0 is the frequency when the load is 0.

The stress value data obtained by converting the actual frequency conversion of the inner and outer steel stress meters of the seven representative measuring points are summarized in Table 1. From Table 1, it can be seen that within the measuring range, the maximum error of the steel stress meter does not exceed 0.7 MPa, which meets the requirement of 1% FS Accuracy requirements, therefore the instrument is reliable in measuring steel stress.

Table 1. Sensor measured data and error statistics.

Sensor Number	Test Number	0 (MPa)	50 (MPa)	100 (MPa)	150 (MPa)	200 (MPa)	K
N1	1	0.0000	49.9234	100.0677	149.9911	200.1354	0.2209
	2	0.0000	49.7025	100.2886	150.2120	199.9145	
W1	1	0.0000	50.0346	100.0692	150.1038	200.1384	0.2166
	2	0.0000	50.0346	100.2858	150.1038	199.9218	
N4	1	0.0000	49.9968	99.9936	149.9904	199.9872	0.2232
	2	0.0000	49.9968	100.3964	149.9904	199.7640	
W4	1	0.0000	50.0000	100.0000	150.0000	200.0000	0.2000
	2	0.0000	50.0000	100.2000	150.0000	199.8000	

Table 1. Cont.

Sensor Number	Test Number	0 (MPa)	50 (MPa)	100 (MPa)	150 (MPa)	200 (MPa)	K
N5	1	0.0000	50.1052	99.9916	150.0968	199.9832	0.2188
	2	0.0000	50.1052	100.2104	150.0968	199.7644	
W5	1	0.0000	50.0792	99.9462	150.0254	200.1046	0.2122
	2	0.0000	49.8670	100.1584	150.2376	199.8924	
N7	1	0.0000	50.3040	99.9792	149.8640	199.9584	0.2096
	2	0.0000	50.0944	99.9792	150.0736	199.9584	
W7	1	0.0000	50.0460	100.0920	149.9185	199.9645	0.2195
	2	0.0000	50.0460	100.0920	149.9185	199.7450	
N8	1	0.0000	50.3040	99.9792	149.8640	199.9584	0.2096
	2	0.0000	50.0944	99.9792	150.0736	199.9584	
W8	1	0.0000	50.2272	99.9936	149.7600	199.9872	0.2304
	2	0.0000	49.9968	99.9936	149.9904	199.9872	
N10	1	0.0000	50.0400	100.0800	150.1200	199.9376	0.2224
	2	0.0000	49.8176	100.3024	150.3424	199.7152	
W10	1	0.0000	50.1860	99.9356	149.6852	200.0894	0.2182
	2	0.0000	50.0594	99.9356	149.6852	200.0894	
N11	1	0.0000	50.4212	99.9768	149.7488	199.7372	0.2164
	2	0.0000	50.6376	99.9768	149.3160	199.9536	
W11	1	0.0000	50.0400	100.0800	149.8976	199.9376	0.2224
	2	0.0000	50.0400	100.3024	149.8976	199.7152	
Maximum error		0.000	0.6376	0.3964	0.3424	0.1384	

(2) Installation of measuring equipment

As shown in Figure 5, the installation for this measurement involved welding the two ends of the steel stress gauge onto the main reinforcement steel bar being measured, and data transmission was performed through the tail transmission line. The measurement obtained by this sensor was the stress value. The steel type used in the construction site was HRB235, with an elastic modulus of 200 GPa. To obtain the strain data of the steel, the measured stress value was divided by the elastic modulus.



Figure 5. Smart string-type steel stress meter.

3.2.2. The Data Acquisition and Transmission Module

The data acquisition and transmission module consisted of a comprehensive acquisition module and a DTU wireless transmission module. As shown in Figure 6, the comprehensive acquisition module had 32 measurement channels, and the extension cable of the sensor was connected to the corresponding channel to export the data. The DTU wireless transmission module mainly included a CPU control module, a wireless communication module, and a power module. As shown in Figure 7, by inserting a SIM card and

connecting to the GPRS network, users can connect to it and send the measurement data to their computer, enabling remote monitoring of the data. The DTU needs to be connected to the comprehensive acquisition module to achieve remote measurement of the data.



Figure 6. Comprehensive acquisition module.



Figure 7. DTU wireless transmission module.

3.3. Sensor Deployment

In order to prevent damage to the signal transmission cables during the pouring process, it is necessary to secure the transmission cables to the reinforcement. In addition, in order to ensure the accuracy of subsequent internal force calculations, measurements of the secondary lining thickness at different locations are required. The field installation process is shown in Figure 8.

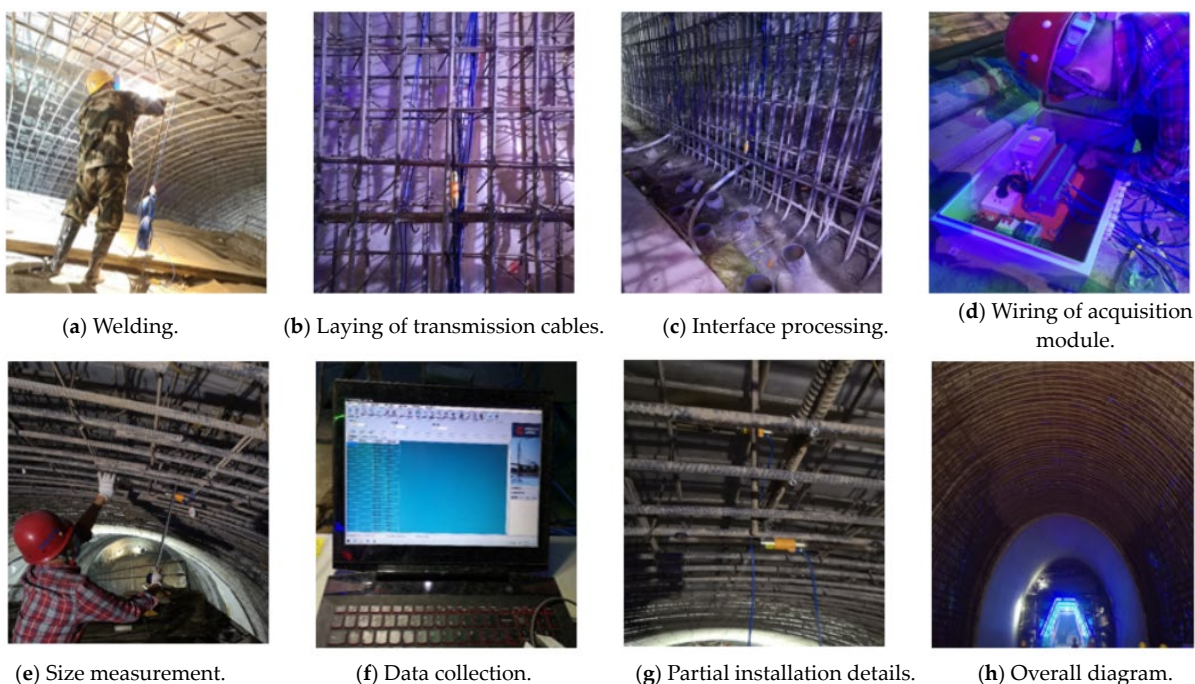


Figure 8. Field installation process.

3.4. Data Collection

The monitoring time started from pouring on 5 September 2020 and was monitored once a day until the pavement was paved on 1 May 2021, which interrupted the monitoring and lasted for nine months. Finally, the ZK280+325 cross-sectional data were used for analysis.

4. Analysis of Test Results

4.1. Change the Law of Secondary Lining Strain

After the monitoring was completed, the monitoring results showed that the strain growth curves at different positions of the secondary lining structure were quite different. To highlight the internal force variation characteristics of the arch, arch shoulder, arch foot, and inverted arch of the tunnel, the monitoring data were sorted out and analyzed in three areas, and seven representative cross-sections were selected from the three areas after data sorting for analysis. The selected seven measuring points were 1, 4, 11, 5, 10, 7, and 8, which, respectively, represent the vault, arch shoulder, arch wall, and arch foot. Figures 4 and 5, respectively, show the strain changes with time at each measuring point on the outside and inside of the section. In Figures 9 and 10, the positive values of the ordinate represent tension, and the negative value represents compression. Table 2 summarizes the tension and compression conditions at different monitoring points at the beginning and end of the monitoring period, as well as the final monitoring value at each point. The analysis results are as follows.

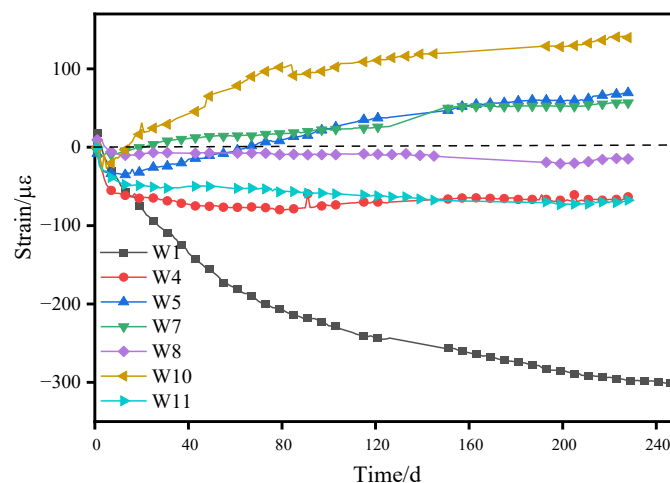


Figure 9. Strain of reinforcement outside section ZK280+325.

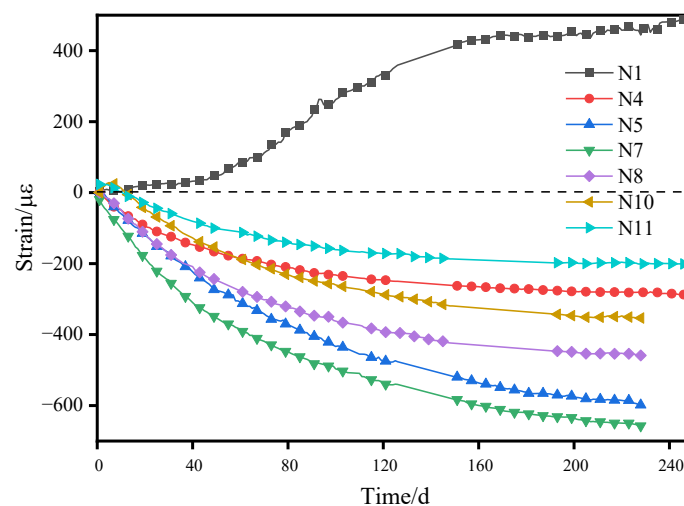


Figure 10. The strain of reinforcement inside section ZK280+325.

Table 2. Measuring point strain situation and final monitoring value.

Position	Measure Point	Tension and Compression Situation (Initial Results)	Tension and Compression Situation (Final Results)	Final Monitoring Value ($\mu\epsilon$)
Vault	W1	Compression	Compression	301
	N1	Tension	Tension	488
Arch Shoulder	W4	Compression	Compression	63
	W11	Compression	Compression	68
	N4	Compression	Compression	288
	N11	Compression	Compression	201
Arch Waist	W5	Compression	Tension	70
	W10	Compression	Tension	140
	N5	Tension	Compression	599
	N10	Tension	Compression	353
Arch Foot	W7	Compression	Tension	57
	W8	Tension	Compression	15
	N7	Compression	Compression	657
	N8	Compression	Compression	459

(1) In terms of tension and compression, the comprehensive analysis of Figures 9 and 10, and Table 2 shows that the outer side of the vault area was under compression and the inner side was under tension, and N1 had a maximum tensile strain of 488 $\mu\epsilon$ during the monitoring period. The outer side of the arch shoulder area was always in a state of compression, and the inner side was in a state of tension at the initial stage of monitoring, and quickly changed to a state of compression as time passed. The outer side of the arched waist area was under compression at the initial stage of monitoring, and then gradually transformed into tension over time, while the inner side was always under compression.

The tension and compression conditions of the left and right measuring points at the arch foot were different. Measuring point 7 at the arch foot was under compression both inside and outside at the initial stage of monitoring, and in the later stage of monitoring, measuring point 7 was under tension on the outside and compression on the inside. During the monitoring period, the maximum compressive strain of 657 $\mu\epsilon$ was observed at N7, and point 8 was compressed both medially and laterally during the monitoring period. According to the data, the stress of the steel bar at the W8 position remained unchanged, and it was judged that the sensor was damaged. The reason for this may have been the stress concentration at the arch foot position. Its pressure value was too large; therefore, the sensor here was damaged.

The cross-section of the arch shoulder area was under full-section compression, which can give full play to the compressive capacity of the concrete. Compared with other areas, this area is safer and played a role in load transmission in the overall secondary lining structure.

During the monitoring of the position of the arch waist and arch toe of the tunnel, the final stress is in a state of compression on the inside and tension on the outside, and the value of the compressive strain on the inside was very large. The results showed that the lower structure of the tunnel was subjected to a large vertical load transmitted by the upper part, but because the lateral pressure of the surrounding rock on the tunnel structure was small, the tunnel structure was in a state of extrusion deformation at the side wall. Thus, the outer reinforcement shows tensile stress.

(2) In terms of strain changes, the strain of the outer side of the vault W1 was always in a state of development with a gradually decreasing growth rate. In the first three months of monitoring, the strain value reached 84% of the final monitoring value. However, within the monitoring time range, the strain curve had not stabilized, indicating that the surrounding rock pressure at the vault position was still developing. The strain on the inner side of the vault, N1, exhibited three developmental stages over time: the early load-bearing stage

(1 d–50 d), the rapid development stage (50 d–90 d), and the slow development stage (90 d–). In the early load-bearing stage, the strain value of N1 is very small, and the growth rate changes little over time. As time progresses, the strain curve undergoes a sudden change, entering the rapid development stage. As can be seen in Figure 5, the strain value at the point of sudden change when N1 enters the rapid growth stage is approximately $50 \mu\epsilon$. In the later stages of monitoring, the strain curve enters the slow development stage, with the strain value gradually increasing over time, but the rate of increase is very small. According to the obvious three-stage development of the steel bar strain curve in the tension area of the vault position, it can be known that the concrete and steel bars in the secondary lining tension area in the early bearing stage are under tension together, and the tensile strain of the steel bar is approximately equal to the tensile strain of the concrete at this time, the strain growth at this stage is relatively slow; as the external load gradually increases, the tensile strain also gradually increases. When the tensile strain value of the steel bar exceeds the ultimate tensile strain of the concrete, the concrete will crack, and the concrete in the tension zone will no longer be under tensile stress, the sudden change point of the curve is the ultimate tensile strain of concrete. After the strain enters the rapid development stage, the stress in the tension area is borne by the steel bars, and the height of concrete cracks becomes larger and larger; after entering the slow growth stage, the surrounding rock pressure on the vault position will increase slowly, and the strain of the steel bars in the tension area will increase. It also gradually decreases, and the cracking of concrete will not develop further.

The change in the strain value of the outer side of the arch shoulder W4 and W11 was characterized by rapid growth in the early load-bearing stage. The rapid growth stage lasted for about a month, and then it tended to stabilize. The final strain value of the outer rebar remained around $65 \mu\epsilon$, which was very small compared to the stress values of N4 and N11. The rate of strain increases on the inner side of the arch shoulder and gradually decreased with the passage of time, and the strain could reach 75% of the maximum value in the first 90 days of monitoring. The strain value tended to stabilize in the later stage of monitoring. According to the strain values of the inner and outer steel bars, it can be seen that the stress state of the arch shoulder area section was in full section compression, which could fully utilize the compressive capacity of concrete.

The increase in the strain of the inner rebar at the waist and foot of the arch gradually decreased over time, but the strain curve did not stabilize within the monitoring time range, similar to the change pattern at the top of the arch, indicating that the surrounding rock pressure is still in the development stage. With the change of time, the state of the rebar strain at different positions was relatively complex, and the state was mainly affected by the self-weight of concrete, surrounding rock pressure, load of formwork trolley, and shrinkage creep of concrete. In the later stage, with the increase of surrounding rock pressure, the surrounding rock pressure became the dominant load, and the change rules of rebar strain at each section gradually tended to be unified.

4.2. Calculation of Internal Force and Safety Factor

According to the strain monitoring curve of the steel bars, it can be seen that the sections of the secondary lining structure were always under eccentric compression during the stress process. Based on the idea of component positive section bearing capacity calculation in “Principles of Structural Design” [26], the calculation Equation of the secondary lining section internal force was derived, and the bending moment and axial force of secondary lining were obtained through Equations (5) and (6). On this basis, the axial force and bending moment obtained through calculation according to the code could be further obtained through the Equation that obtains the safety factor of each measuring point on the section. The calculation idea is as detailed below:

4.2.1. Calculation Idea of Section Internal Force

The idea of calculating the internal force of the section is first based on the four assumptions of the calculation of the bearing capacity of the normal section of the eccentrically compressed member of the rectangular section, which are as follows:

- (1) Deformation coordination introduces the assumption of plane section. The assumption of the plane section refers to the assumption that the average strain distribution of concrete and longitudinal reinforcing bars within the height range of a concrete structural member along the positive section is linear after the member is subjected to forces. This assumption is applicable to eccentrically compressed members where each section satisfies this assumption during the load-bearing process;
- (2) Assume synchronous deformation of steel bar and surrounding concrete when deformation is coordinated. Ribbed steel bars are used in reinforced concrete, and there is significant friction between the steel bar surface and the concrete. During the structural loading process, the relative displacement between the steel bars and the concrete is very small. Therefore, it can be approximately assumed that the compressive strain of the steel bars is equal to the strain value of the corresponding position in the concrete;
- (3) Neglect the tensile strength of the concrete in the tension zone when the internal force is balanced. The tensile stress in concrete is very small, and once the concrete cracks, the tensile zone of the concrete no longer bears the load. Only the concrete near the neutral axis still bears partial tensile stress, but due to its small lever arm, the internal moment it bears can be neglected in the calculation process. Assuming the stress–strain relationship of the concrete in the compression zone as a rectangle.
- (4) Introduce the ideal elastic-plastic assumption of steel bars when the internal force is balanced. The reinforcement stress obeys Hooke's law before reaching the yield stress; After reaching the yield stress, the stress remains unchanged.

Based on the assumption of the normal section in the above assumptions, the strain distribution of the concrete section can be obtained. The strain distribution diagram of the section is shown in Figure 11.

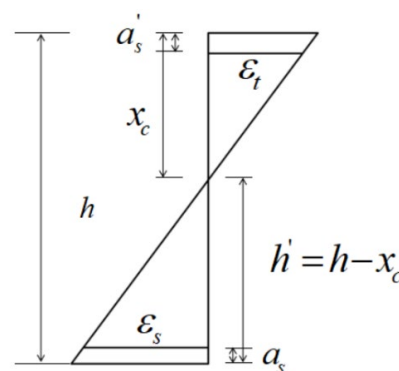


Figure 11. Strain distribution of the concrete section.

Based on the figure, the expression for the height of the compressed area of the section can be derived, as shown in Equation (2):

$$x_c = \frac{\varepsilon_t a_s + h \varepsilon_c - a_s' \varepsilon_s}{\varepsilon_t + \varepsilon_s} \quad (2)$$

The calculation Equation for the bearing capacity of the positive section of the eccentrically compressed member with a rectangular section is shown in Figure 12. When calculating the bending moment and axial force of the section, the bearing capacity can be calculated through the balance of force and torque.

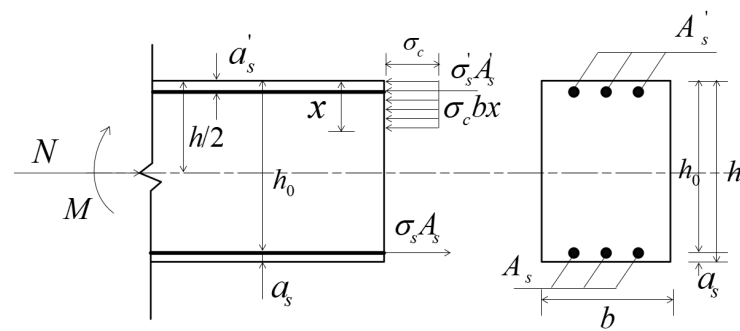


Figure 12. Internal Force Calculation Diagram.

Eccentrically loaded members can be divided into two types of stress states: large eccentrically loaded members or small eccentrically loaded members. The main difference lies in the range of the concrete compression zone and the stress state of the reinforcement in the tension zone. The stress state of the large eccentric compression section is characterized by tension on one side of the reinforcement in the tension zone, and the small eccentric compression section state is that the reinforcement on the tension side may be tensioned or compressed. Therefore, when calculating the internal force, as long as the direction is changed according to the compression or tension state, a unified calculation equation for large and small eccentric compression can be established. Taking the sum of the internal and external forces in the longitudinal direction of the member as zero, Equation (3) can be obtained.

$$N = \sigma'_s A'_s + \sigma_c b x - \sigma_s A_s \quad (3)$$

By summing up all the moments of forces on the section about the center of the section, we can obtain Equation (4).

$$M = \sigma_c b x (h - x) / 2 + \sigma'_s A'_s \left(\frac{h}{2} - a'_s \right) + \sigma_s A_s \left(\frac{h}{2} - a_s \right) \quad (4)$$

By substituting the expression of the compressed zone height (1) into (2) and (3), the final expressions of the bending moment and axial force can be obtained, as shown in Equations (5) and (6):

$$N = \varepsilon_t E_s A'_s + \varepsilon_t E_c b \alpha \frac{\varepsilon_t a_s + h \varepsilon_c - a'_s}{\varepsilon_t + \varepsilon_c} - \varepsilon_s E_s A_s \quad (5)$$

$$M = \varepsilon_t E_c b \alpha \frac{\varepsilon_t a_s + h \varepsilon_c - a'_s}{\varepsilon_t + \varepsilon_c} \left(h - \alpha \frac{\varepsilon_t a_s + h \varepsilon_c - a'_s}{\varepsilon_t + \varepsilon_c} \right) / 2 + \varepsilon_t E_s A'_s \left(\frac{h}{2} - a'_s \right) + \varepsilon_s E_s A_s \left(\frac{h}{2} - a_s \right) \quad (6)$$

where x_c is the height of the compression zone; h is the total height of the section; a_s is the thickness of the concrete cover in the tension zone; a'_s is the thickness of the concrete cover in the compression zone; ε_t is the compressive strain of the concrete at the location of the compressive reinforcement; ε_s is the tensile strain of the tensile reinforcement; M is the bending moment of the cross-section; N is the axial force of the cross-section; x is the height of the compression zone of the section, $x = \alpha x_c$, α takes 0.8; σ_c is the compressive stress of the concrete, $\sigma_c = \varepsilon_t E_c$; σ_s is the tensile stress of the reinforcement in the tension zone, $\sigma_s = \varepsilon_s E_s$; σ'_s is the compressive stress of the reinforcement in the compression zone, $\sigma'_s = \varepsilon_t E_s$; A_s is the cross-sectional area of the reinforcement in the tension zone; A'_s is the cross-sectional area of the reinforcement in the compression zone; b is the section width; h is the total height of the section; E_c is the modulus of elasticity of the concrete; E_s is the modulus of elasticity of the reinforcement.

4.2.2. Calculation Method of the Safety Factor

The strength safety factor of each section of the secondary lining needs to be calculated in two cases large eccentric compression and small eccentric compression.

According to the “Code for Design of Highway Tunnels” (JTG D70-2004) [27], the calculation equation of the section safety factor of the large eccentric compression member with reinforced concrete rectangular section is as follows:

$$K_1 = \frac{R_w bx + R_g (A'_s - A_s)}{N} \quad (7)$$

$$K_2 = \frac{R_w bx(h_0 - x/2) + R_g A'_s(h_0 - a'_s)}{Ne} \quad (8)$$

According to the “Code for Design of Highway Tunnels” (JTG D70-2004) [27], the equation for calculating the section safety factor of small eccentric compression members with reinforced concrete rectangular sections is as follows:

$$K_3 = \frac{0.5R_a bh_0^2 + R_g A'_s(h_0 - a'_s)}{Ne} \quad (9)$$

When the axial force N acts between the center of gravity of the reinforcement A_s and the center of gravity of the reinforcement A'_s , the following requirements shall be met:

$$K_4 = \frac{0.5R_a bh_0^2 + R_g A_s(h_0 - a)}{Ne'} \quad (10)$$

where K_1, K_2 is the safety factor of the large eccentric compression member; K_3 is the safety factor of the small eccentric compression member; K_4 is the safety factor of the member when the axial force acts on the centroid of the reinforcement and the centroid of the reinforcement; M is the bending moment of the cross-section; N is the axial force of the cross-section; R_w is the standard value of the concrete bending compressive ultimate strength, and the R_w of C35 concrete value is 28.1 MPa; R_g is the standard value of the tensile or compressive strength of the steel bar, and the longitudinal steel adopted HRB335, the standard value of its tensile and compressive calculation strength $R_g = R'_g = 335$ MPa; R_a is the compressive ultimate strength of concrete or masonry, and the value of C35 concrete is 22.5 MPa; A_s is the cross-sectional area of the steel bar in the tension zone; A'_s is the cross-sectional area of the steel bar in the compression zone; h is the total height of the section; h_0 is the effective height of the section, $h_0 = h - a_s$; x is the height of the compression zone of the section, $x = \alpha x_c$, α takes 0.8; b is the section width; e, e' is the distance from the center of gravity of reinforcement A_s and A'_s to the point of application of the axial force.

4.3. Variation Law of Section Internal Force

4.3.1. Required Measured Parameters

The measured parameters required for internal force calculation include the height of the secondary lining section at different measurement points and the thickness of the reinforcement protective layer on both sides. Table 3 lists the measured parameters at different measurement points and the stress state of the section.

Table 3. Required measured parameters and the stress state.

Position	Height (cm)	Thickness of the Reinforcement Protective Layer (cm)	Stress State
Vault	70	5	large eccentric compression
Arch shoulder	80	5	large eccentric compression
Arch Waist	100	10	small eccentric compression
Arch Foot	100	10	small eccentric compression

4.3.2. Time-Varying Law of Axial Force of the Secondary Lining

We calculated the axial force and bending moment values of the secondary lining structure according to Equations (5) and (6) based on the field-measured inner and outer reinforcement strain values of the selected seven measuring points and drew relevant charts. Figure 6 is the time-varying diagram of the axial force at the section of the seven representative measuring points selected above, and Figure 13 is the end of the monitoring of all measuring points. The following results are obtained from the comprehensive analysis of Figures 13 and 14.

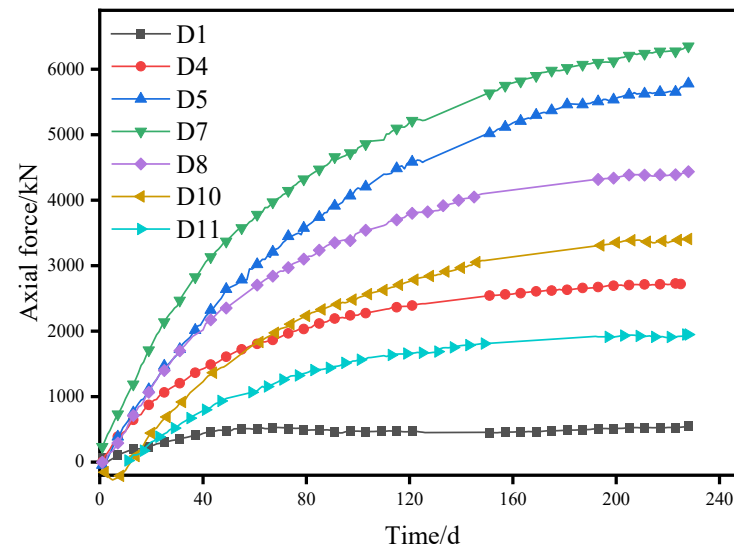


Figure 13. Time-varying axial force at section ZK280+325.

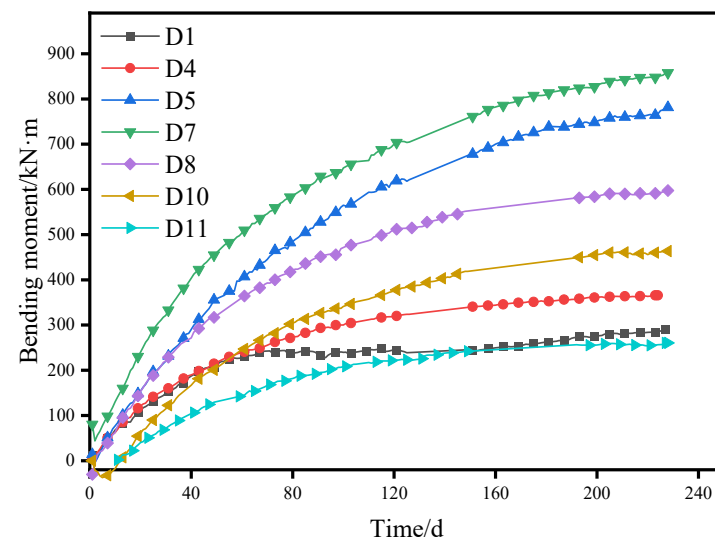


Figure 14. Time-varying bending moment of section ZK280+325.

From the slope of the curves in Figures 13 and 14, it can be observed that the axial force and bending moment at the right arch foot (D7) in the secondary lining increased at the fastest rate, reaching 625 kN·m and 4635 kN, respectively, on the 90th day. The axial force at the vault (D1) increased at the slowest rate, only reaching 484 kN by the 90th day of monitoring. The bending moment increase rate at the left arch shoulder (D11) was the slowest, only increasing to 194 kN·m on the 90th day of monitoring.

It is worth mentioning that, compared with the other two areas, the internal force of the arch shoulder area could maintain a nearly stable state in the later stage, because it

mainly bore the role of load transmission in the structure. Therefore, under the surrounding rock pressure, the growth rate of the positional internal force was not large.

Through the comprehensive analysis and comparison of the above figures, it can be seen that the axial force and bending moment of the secondary lining of the tunnel show an overall growth trend over time during the monitoring time, and the internal force growth trend of each monitoring section is the same. In the early stage of monitoring, the internal force growth of each section showed a rapid increase. In the first three months of monitoring, the axial force and bending moment of all the measuring point sections could reach at least 65% of the final value at the end of the monitoring. After that, as time went by, the growth rate of the internal force of each section gradually slowed down, showing the characteristics of “fast first and then slow” as a whole.

4.3.3. Time-Varying Law of Bending Moment of the Secondary Lining

We extract the bending moment and axial force values of all sections at the end of the monitoring and drew the bending moment and axial force diagrams, as shown in Figures 15 and 16, where the numerical unit in the bending moment diagram is $\text{kN}\cdot\text{m}$, and the numerical unit in the axial force diagram is kN .

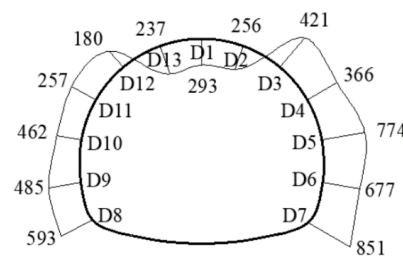


Figure 15. Bending moment of secondary lining.

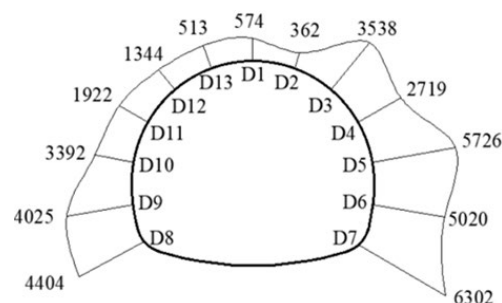


Figure 16. Axial force of secondary lining.

Figure 15 shows the bending moment diagram of the secondary lining monitoring section. In the figure, the direction of the bending moment at the vault position is opposite to that at other positions. It shows that the longitudinal cracks in the tunnel vault could be attributed to the inner side of the vault area, which was subjected to a large tensile stress. This caused the secondary lining concrete to crack and fail. Comparing the internal force values at different positions, the bending moment at the arch foot was the largest, and the bending moment values on the left and right sides were $593 \text{ kN}\cdot\text{m}$ and $851 \text{ kN}\cdot\text{m}$, respectively, mainly because the arch foot was in contact with the inverted arch and was subjected to a large force stress concentration phenomenon. The bending moment on the right side of the tunnel was generally greater than the bending moment on the left side, indicating that the surrounding rock load on the structure was greater on the right side than on the left side. The main reason was that the right side of the monitored section was the interval between the left and right holes of the tunnel. The distance (about 20 m) between the two holes can be judged according to the drawings. During the simultaneous excavation of the two tunnels, due to the simultaneous disturbance of the left and right holes to the surrounding rock, the surrounding rock on the right side of the section had

a large loosening range. During the long-term stress process, the surrounding rock on the right side of the tunnel, the pressure was greater than the surrounding rock pressure acting on the left side of the tunnel. The bending moment value of the structure increased gradually from top to bottom.

Figure 16 shows the axial force diagram of the secondary lining monitoring section. From the axial force value, it can be seen that all sections of the tunnel structure were under compression. From the perspective of the overall distribution of the axial force, the axial force of each section of the secondary lining increased continuously from top to bottom, and the axial force of the structure on the right was greater than that of the structure on the left, which is consistent with the distribution of the bending moment of the structure. In addition, it can be seen that the axial force of the substructure of the secondary lining was much greater than that of the superstructure. Comparing the numerical values of the arch foot and the vault, the axial force values of D1 and D7 were approximately 11 times different, which shows that the upper part of the tunnel is affected. The vertical downward load was relatively large and was transmitted from the upper part of the structure to the lower part of the structure; then, the vertical downward load was transmitted to the tunnel base by the arch foot. The high compressive stress at the arch waist and side walls may cause the concrete to split and fail under greater pressure, thus explaining the oblique and longitudinal cracks at the side walls of the tunnel.

On the whole, the internal force distribution of the secondary lining structure of the tunnel presented the form of “big on the right and small on the left” and “big on the bottom and small on the top”. The load was mainly transmitted from the upper part of the structure to the lower part of the structure so that the axial force value of the arch foot was the largest.

According to the comprehensive analysis of bending moment and axial force values, the secondary lining structure was subjected to a large vertical load and a small lateral load, and the structure was in a state of extrusion deformation in the side wall area. This shows that the lateral pressure coefficient of the surrounding rock is very small, and its stress mode is different from that specified in the “Code for Design of Highway Tunnels” [27] where the lateral pressure coefficient of V surrounding rock is 0.5.

4.4. Safety Evaluation of the Secondary Lining

Based on the bending moment axial force data obtained above, we can calculate the safety factor of the secondary lining structure of the tunnel according to the safety factor calculation method introduced above. Through the safety factor, we can determine the dangerous area on the secondary lining section. At the same time, a reasonable evaluation of the long-term safety of the secondary lining can be further carried out according to the development of the safety factor, as well as a more accurate evaluation of whether the secondary lining is in a safe state.

It was found through a calculation that the structure in the vault area was in a state of large eccentric compression. Therefore, the safe state of the section was mainly controlled by the yield limit tensile stress of the steel bars in the tension area. Equation (7) was used to calculate the value of the safety factor. The sections in the arch shoulder area and the arch foot area were in a small eccentric compression state, and the safe state of the section was mainly controlled by the ultimate compressive strength value of the concrete in the compression area. Therefore, Equation (9) is used to calculate the safety of the section in the area coefficient. The safety factor at different time points of the seven measuring points selected above during the monitoring period was calculated, and the change curve concerning time was drawn, as shown in Figure 17.

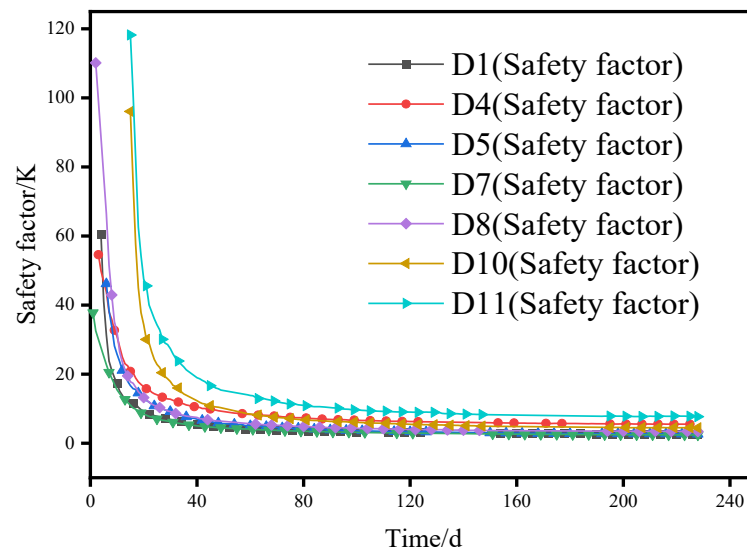


Figure 17. Time-varying safety factor of the secondary lining.

As shown in Figure 17, the safety factor of seven representative measuring points in the entire section of the secondary lining structure changed with time. The safety factor value of the curve in the figure decreased rapidly from the maximum value at the beginning of monitoring in a short period. The vault area was in a state of large eccentric compression, with a safety factor limit value of 2.4, and the rest of the sections are in a small eccentric compression state, with a safety factor limit value of 2.0.

According to the calculated value of the safety factor, it can be seen that the safety factor value of the measuring point D11 at the arch shoulder was the largest, and the safety factor was still 7.68 at the end of the monitoring, which is still far higher than the limit value of the safety factor of 2.0. The safety factor of D1 and D7 were small, and the safety factors at the end were 2.49 and 2.37, which are closer to the critical value of the safety factor. Therefore, it is necessary to focus on the secondary lining safety inspection at the end of the monitoring period. During the monitoring period, the safety factor value of each section is always decreasing, and the values all approach the limit value of the safety factor required by the code. However, within the time frame of the structural monitoring for all cross-sections, the safety degree of the structure is within the range required by the code, indicating that the structure was still in a safe state during the monitoring stage.

4.5. Regression Analysis

By combining the development trend of the internal force curve in the tunnel and the change trend of the safety factor, it can be seen that the internal force of the tunnel structure was still increasing, and the safety factor of the structure was still changing. Therefore, in the later stage of tunnel structure operation, it was impossible to determine whether the safety factor of the secondary lining structure was still within the limit range specified by the regulations. Further research and analysis on the long-term safety of tunnel structures are needed. The reliability of the regression analysis of loess tunnel surrounding rock deformation prediction in reference [17] was combined with the development trend of the safety factor scatter plot. The fitting function selected for safety factor K is:

$$K = A + Be^{Ct} + De^{Et} \quad (11)$$

where: K is the safety factor; t is the time; A, B, C, D, E are coefficients.

This article used Origin software for the regression analysis and prediction. Regression analysis was performed on the safety factor data of the seven selected measurement points. The calculation results of the safety factor fitting function of each measurement point are shown in Table 4.

Table 4. Safety factor regression analysis results.

Position	Measure Point	Safety Factor Final Calculated Value	Safety Factor Final Predicted Value	Fitting Function	R ²
Vault	1	2.48857	2.83421	$K = 2.83421 + 330.74635e^{\frac{-t}{1.93951}} + 18.10648e^{\frac{-t}{20.7685}}$	0.99761
Arch shoulder	4	5.51259	5.46581	$K = 5.46581 + 59.10097e^{\frac{-t}{7.42187}} + 11.37243e^{\frac{-t}{44.36668}}$	0.99881
	11	7.68418	8.13949	$K = 8.13949 + 10723.92089e^{\frac{-t}{3.0706}} + 56.27516e^{\frac{-t}{25.7703}}$	0.99710
Arch Waist	5	2.60743	2.81752	$K = 2.81752 + 116.28137e^{\frac{-t}{4.28193}} + 16.92097e^{\frac{-t}{29.90729}}$	0.99850
	10	4.4429	4.91474	$K = 4.91474 + 23886.1352e^{\frac{-t}{2.55562}} + 45.61348e^{\frac{-t}{23.54496}}$	0.99804
Arch Foot	7	2.37227	2.36814	$K = 2.36814 + 33.3213e^{\frac{-t}{7.39392}} + 5.8439e^{\frac{-t}{48.35646}}$	0.99951
	8	3.38962	3.35028	$K = 3.35028 + 145.33393e^{\frac{-t}{5.30275}} + 9.5858e^{\frac{-t}{43.7114}}$	0.99542

From the regression analysis results, it can be seen that the exponential function fit well with the results of different measurement points, and the correlation coefficients were all above 0.99, which proves that the change law of the safety factor conforms to the exponential function law. Since the exponential function is a bounded function, it is possible to make a reasonable prediction of the final value of the safety factor based on this function, and then evaluate the future safety of the tunnel based on whether the final predicted value is less than the critical safety factor value.

The two dangerous areas mentioned above were the vault measurement point 1 and the arch foot measurement point 7, and the final predicted values of their corresponding regression equations were 2.83421 and 2.36814, respectively. Their final predicted values were all greater than the critical safety factor value 2 of the corresponding area, which meets the tunnel safety standards. Therefore, we can reasonably predict that the Yulinzi Tunnel will still be in a safe state in the future.

5. Numerical Simulation Analysis of Long-Term Stress on Secondary Lining

The long-term stress and deformation of the secondary lining of the Yulinzi Tunnel were simulated using FLAC3D finite difference software. The simulation considered the structural response under the condition of a 25% water content in the yellow soil. The dimensions of the geological model were set to 170 m × 130 m, with a tunnel depth of 50 m. The longitudinal length of the tunnel was much greater than its transverse dimension. The model was solved using a plane strain problem, with a longitudinal dimension of 1 m. The bottom and surrounding boundaries of the model were constrained, and the self-weight stress field was applied based on the tunnel depth. The construction design parameters are shown in Table 5, and the numerical model is shown in Figure 18. During the simulation, the creep effect of the surrounding rock during the construction phase was not considered, and the analysis focused only on the stress within 100 years after the secondary lining was constructed.

Table 5. Primary support and secondary lining parameters.

	Density (kg/m ³)	Elastic Modulus (MPa)	Poisson's Ratio	Thickness (m)
Primary support	2360	2.95×10^{10}	0.2	0.2
Secondary lining	2400	3.25×10^{10}	0.2	0.7

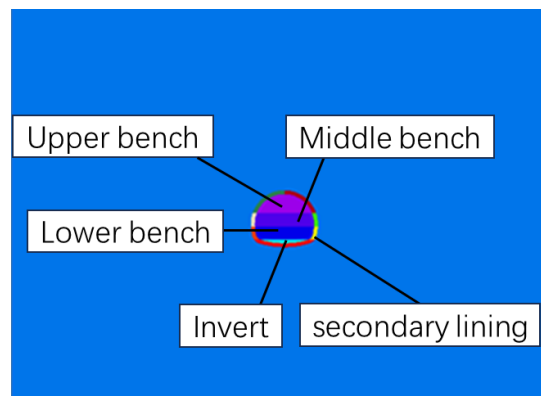


Figure 18. Numerical simulation model.

5.1. Surrounding Rock Constitutive Model and Parameters

The constitutive model used for the surrounding rock was cvisc, which is an integrated model in the FLAC3D software. This model combines the creep constitutive model, Burgers model, with the plastic constitutive model, Mohr-Coulomb model. It can reflect both creep behavior and plastic behavior. The components of the cvisc model are shown in Figure 19.

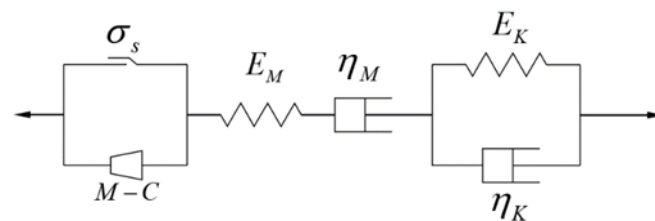


Figure 19. Cvisc model components.

Based on the physical and mechanical experimental data of laboratory soil and the calculation method of creep parameters in reference [28], the mechanical parameters of the surrounding rock were obtained, as shown in Table 6.

Table 6. Mechanical parameters of loess.

Mechanical Parameters	Formation Type	Parameter Value
Density (kg/m^3)	Loess Strata	2135
Elastic Modulus (MPa)	Loess Strata	3.7.65
Poisson's ratio	Loess Strata	0.33
Cohesion (kPa)	Loess Strata	27.04
Angle of internal friction	Loess Strata	6.34
G_M (kPa)	Loess Strata	27.44
H_M (kPa·h)	Loess Strata	343.75
G_K (kPa)	Loess Strata	10.95
H_K (kPa·h)	Loess Strata	20.69

The process of tunnel excavation can be divided into five stages: upper bench excavation, middle bench excavation, lower bench excavation, invert excavation, and secondary lining construction.

5.2. Analysis of Numerical Simulation Results

5.2.1. Deformation Analysis of Secondary Lining

In order to further analyze the displacement and deformation of the secondary lining structure, the vertical displacement curves of the vault and invert positions, as well as the horizontal displacement curve of the right side of the arch waist, are plotted separately, as shown in Figures 20 and 21.

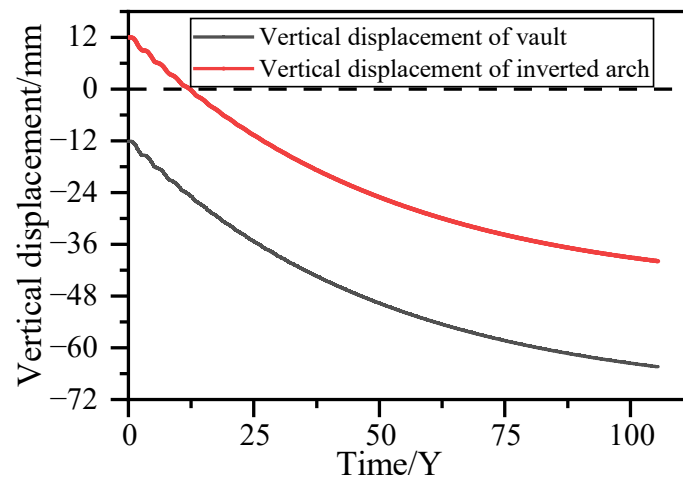


Figure 20. Vertical displacement curve of secondary lining.

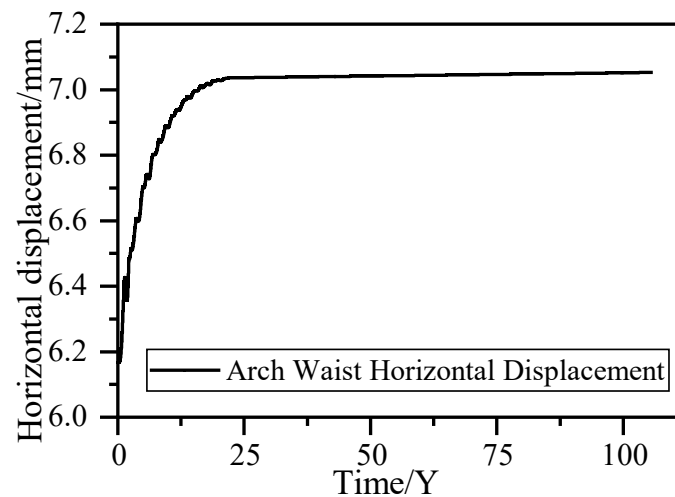


Figure 21. Horizontal displacement curve of secondary lining.

According to the analysis of Figure 20, the values of the vault and invert positions at time zero were 11.9 mm and −12.1 mm, respectively. This indicates that before the creep deformation of the surrounding rock, the secondary lining structure of the tunnel would experience arch crown settlement and invert uplift under the instantaneous deformation pressure of the surrounding rock. The relative displacement of the arch crown and invert positions remained basically unchanged in the following 100 years, indicating an overall settlement of the structure. The overall settlement of the secondary lining could reach 51 mm within 100 years.

According to the analysis of Figure 21, the lateral displacement of the right arch foot at time zero was 6.2 mm. The tunnel structure exhibited outward extrusion deformation before the creep deformation of the surrounding rock. With the passage of time, the lateral extrusion deformation of the structure gradually increased. After 20 years of creep, the lateral deformation gradually stabilized, and the lateral displacement reached 7.2 mm. Compared to the vertical settlement displacement, the lateral displacement of the secondary lining structure was relatively small. Therefore, it is necessary to control the overall settlement displacement of the secondary lining structure in the later operation of the loess tunnel.

5.2.2. Internal Force Analysis of Secondary Lining

In order to analyze the time-varying patterns of the maximum and minimum principal stresses, the maximum principal stress values at the key nodes of the intrados of the arch

and the extrados of the spandrel are derived, and the time-varying curve of the maximum principal stress is plotted as shown in Figure 21. The development trend of the minimum principal stress values on the inner side of the arch foot, the upper side of the arch section, and the lower side of the extrados section is plotted as shown in Figure 22.

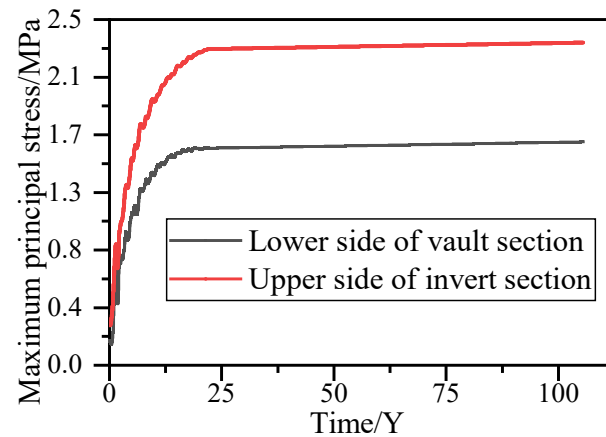


Figure 22. Maximum Principal Stress Curve.

According to the analysis of Figure 22, it can be observed that the maximum principal stress at two locations of the secondary lining structure of the tunnel showed a rapid increase in the first 20 years, followed by a stable trend. The maximum principal stress at the upper side of the invert section stabilized at 1.55 MPa, exceeding the ultimate tensile stress of C35 concrete. It could be inferred that there would be significant longitudinal cracks on the upper side of the invert section. Similarly, the maximum principal stress at the lower side of the crown section stabilizes at 2.33 MPa, which will also cause cracks due to excessive tensile stress, consistent with the actual cracking situation of the secondary lining of the tunnel.

According to the analysis of Figure 23, after being subjected to creep, the minimum principal stress at three locations of the tunnel structure showed a similar trend as the minimum principal stress. After reaching a stable state, the minimum principal stress values at the three characteristic points shown in the figure are -5.21 MPa, -14.82 MPa, and -24.5 MPa, respectively. The minimum principal stress at the inner side of the springing was the largest, exceeding the compressive strength limit of C35 concrete. This indicates that the inner side of the arch foot will experience compressive failure due to excessive compressive stress during long-term loading. The minimum principal stress values at other locations are not significant, and the structure will not experience obvious compressive failure.

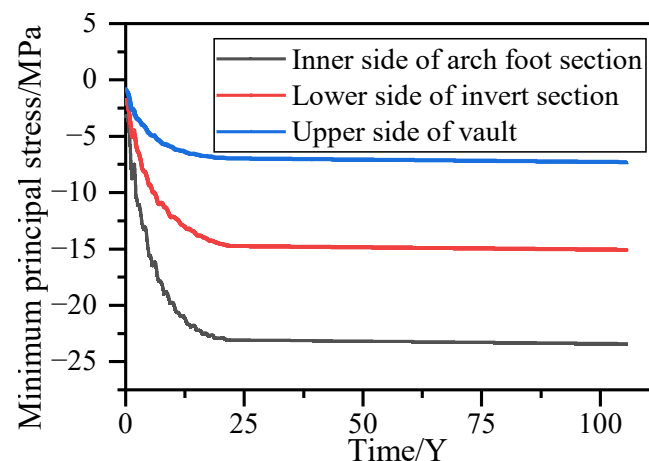


Figure 23. Minimum Principal Stress Curve.

6. Conclusions

Based on the field investigation of the Yulinzi Tunnel Project and the monitoring results of the secondary lining deformation and internal force and numerical simulation of the tunnel, this paper discusses and analyzes the strain change law, long-term stress situation, and safety of the loess tunnel lining. The main conclusions are as follows:

- (1) Based on field monitoring, strain change data of tunnel lining reinforcement over 9 months were obtained. The strain of the secondary lining reinforcement generally showed a development process of rapid increase in the early stage and slow growth in the later stage. However, the strain change curve in the arch waist and arch foot areas was more complex due to the influence of factors such as the bearing of the early structural self-weight, concrete shrinkage creep, formwork trolley load, and stress concentration;
- (2) Based on the basic assumptions in “Structural Design Principles”, a unified calculation Equation for the axial force and bending moment of concrete under different eccentric compression states was established;
- (3) The axial force and bending moment of the lining section were asymmetrically distributed in space, and the axial force and bending moment of the lower structure of the secondary lining were much larger than those of the upper structure. The main reason is that the disturbance caused by the excavation of the left and right holes caused the inconsistency of the loose range of the surrounding rock on both sides of the section and the smaller lateral pressure coefficient of the surrounding rock leads to a large difference between the vertical load and horizontal load of the surrounding rock. The development of axial force and bending moment showed a trend of increasing with time, which was basically consistent with the change rule of lining reinforcement strain, and overall showed a “fast first, slow later” characteristic;
- (4) The final safety factors of the vault D1 and arch foot D7 were 2.49 and 2.37, respectively, close to the critical value of the safety factor, which were the dangerous areas in the secondary lining section. The development of the safety factor over time is in good agreement with the change rule of the exponential function. The correlation coefficients of the regression analysis results of the seven measuring points were all above 0.99, verifying the reliability of using the exponential function to predict the future safety factor of the tunnel. The predicted safety factor values of the two dangerous areas at the vault and arch foot were both greater than the critical value of the safety factor, which can reasonably predict that the Yulinzi Tunnel will still be in a safe state in the future.
- (5) The deformation and internal force changes of the secondary lining of the tunnel in the next 100 years were obtained through numerical simulation. In the initial stage of force, the secondary lining experienced significant sinking of the arch crown, rising of the arch springing, and extrusion deformation in the horizontal direction. Over time, the vertical direction of the structure showed an overall sinking trend, with a settlement of 51 mm over 100 years. The time-dependent characteristics of the stress of the structure and the time-dependent characteristics of the calculated safety factor measured in the field are “first fast then slow, and finally tend to be stable”, which conforms to the change law of the Exponential function.

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