Research Article

Study on Dynamic Response Characteristics and Damage Mechanism of Tunnel Lining at Entrance of Shallow Bias Tunnel

Lin Li,¹ Xiaodan Guo,¹ Zuyin Zou ,¹ Zhanyuan Zhu,¹ Zihong Guo,¹ Weimin Xiao,¹ and Deping Guo²

¹School of Civil Engineering, Sichuan Agricultural University, Dujiangyan 611830, China

²Sichuan Railway Investment Group Co., Ltd., Chengdu 610094, China

Correspondence should be addressed to Zuyin Zou; 382358357@qq.com

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The structural damage of the lining structure at the entrance of a tunnel is the most common instability problem. The instability problem may cause dynamic effects such as earthquakes and blasting. Based on the seismic damage data collected from previous major earthquakes at the entrance of shallow-buried tunnel, the shaking table test and numerical simulation are used to analyze dynamic response characteristics and damage evolution characteristics of the tunnel in the shallow-buried hole at 30°. The study revealed the stress characteristics of tunnel lining and the mechanism of structural damage under earthquake excitation. The research results show that the biased tunnel (30°) is susceptible to damage on the unsymmetrical loading side, the biased ground surface leads to acceleration, and high speed also significantly increases the effect. The biased side leg of the tunnel lining cross section is a location with a large internal force distribution. The biased tunnel has a relatively unfavorable internal force value distribution and a larger peak, and the peak at the larger bias side has the largest peak value. The skewback and spandrel portion of the biased tunnel lining load are more likely to be damaged.

1. Introduction

The tunnel opening section is an area that is often subject to changing conditions during the transition from the ground to the underground, it is inevitable that the tunnel encounters a bias phenomenon due to insufficient precautions or poor dynamic effects. The so-called unsymmetrical loading tunnel refers to the tunnel with its support being subject to bias load, where its surrounding rock pressure exhibits apparent unevenness caused by topographical factors, geological factors, and engineering factors [1].

The seismic damage at the entrance of an unsymmetrical loading tunnel mainly includes the collapse of the slope at the entrance and cracks in the lining structure [2]. There are many reports about the damage of tunnel openings caused by earthquakes. For example, there was an opening of the tunnel that was damaged during the 1999 Chi-Chi earthquake [3–5] (Figure 1). In 2008, there were a large number of earthquakes in the Wenchuan earthquake in China. Serious

damage occurred at the opening section of the unsymmetrical loading tunnel [6–8] (Table 1 and Figure 2). The characteristics of the epicenter at the opening of the unsymmetrical loading tunnel section became the focus of research and serves as the decisive place for seismic fortification.

The seismic response of underground structures has been studied by many researchers using various methods, including theoretical analysis, numerical simulation, and physical model tests [9–13]. On-site investigation of the Kumamoto earthquake by Zhang et al. [14] showed that the main seismic damage was cracks seen at the entrance of tunnel lining. Genis and Jai et al. [8, 15–17] performed dynamic response analysis on the stability of the tunnel opening. Wang et al. [18] proposed a method for identifying damage indicators of tunnel lining structures based on wavelet residual force vector. Varma et al. [19] established a universal discrete element coding model, proving that shallow-buried tunnel linings are more vulnerable to

FIGURE 1: Lining crack of tunnel in chi-chi earthquake [4].

TABLE 1: Investigation results of seismic damage for tunnels during Wenchuan earthquake.

Tunnel name	Length (m)	Distance to epicenter (km)	Fault amount	Destruction feature
Longxi	3691	0.5–2	1	Rock falls; pavement uplift; lining crack; lining dislocation; cable trench; water leakage
Taoguan	625	15	0	Portal failure; rock falls; headwall damage; lining crack
Longdongzi	1071	2	4	Rock falls; pavement uplift; lining crack; lining dislocation; cable trench; water leakage
Shaohuoping	451	1	1	Portal failure; rock falls; headwall damage; lining crack; lining dislocation; water leakage
Zaojiaowan	1926	5	0	Portal failure; headwall damage; lining crack; lining dislocation; cable trench; water leakage
Caopo	759	20	0	Portal failure; rock falls; lining crack; cable trench; water leakage
Dankanliangzi	1567	24	1	portal failure; rock falls; headwall damage
Zipingpu	4090	5	10	Rock falls; pavement uplift; lining crack; lining dislocation; cable trench

(a)

(b)

FIGURE 2: Damage patterns of tunnel linings during the Wenchuan earthquake 2008, China. (a) Damage at the entrance of Longxi tunnel [8]. (b) Cracking at the portal of Taoguan tunnel [7].

damage under seismic loads. Wang et al. [20] performed a series of shaking table tests on a scaled-up tunnel model under earthquake excitation to identify damage to the tunnel lining. Wang et al. [21] carried out a large-scale shaking table model test of a small-clearance shallow-buried biased tunnel, finding that the acceleration amplification coefficient and change trend of the left-hole lining are quite different compared with the right-hole lining. Most of the previous research studies are designed to verify the analysis method and provide experimental data regarding the ultimate

stability of the tunnel. However, the study has not been extensively performed to the damage mechanism of the earthquake on the tunnel lining of the shallow-buried unsymmetrical loading tunnel.

In this paper, the shaking table test and numerical simulation are combined to study the lining damage mechanism of the shallow-buried tunnel opening section under the condition of 30° bias angle and 60° elevation slope. First, the shaking table model test is used to study the damage characteristics of the tunnel lining of the shallow-

buried bias tunnel. Then, the dynamic finite element analysis method is used to analyze the characteristics, deformation, and internal force response laws of the full-time dynamic response of the tunnel lining of the shallow-buried bias tunnel. Finally, the simulation results are compared with the model test results to prove the rationality of the shaking table test and the reliability of the numerical simulation.

2. Test Plan

2.1. Model Test Device. The tunnel model is built in a rigid, sturdy box with a length, width, and height of 1.3 m, 1.0 m, and 1.0 m, respectively (Figure 3(b)), which is anchored on a shaking table (Figure 3(c)). The main parameters of the shaking table are shown in Table 2. With the direction perpendicular to the excitation direction, the wall of the box is lined with a molded polystyrene foam board with a thickness of 70 mm. Meanwhile, a smooth PVC film is pasted on the box walls at both ends of the model to reduce friction resistance on the surface where box comes in contact with soil (Figure 3(a)). A layer of crushed stone is laid on the bottom of the model box to increase the frictional resistance on the contact surface, so as to avoid relative sliding of the bottom plate of the model body when excited (Figure 3(b)).

2.2. Model Material Parameters. For the model test of rock mass materials, the geometric dimensions, boundary conditions and acting loads of the model, the bulk density, strength and deformation characteristics of the model rock mass, etc. must meet the similarity requirements expressed as follows [22]:

$$\begin{aligned} C_\sigma &= C_l C_\gamma, \\ C_\sigma &= C_E C_\epsilon, \\ C_\mu &= 1, \\ C_\delta &= C_l C_\epsilon, \end{aligned} \quad (1)$$

where C_σ is stress similarity ratio, C_l is geometric similarity ratio, C_E is elastic modulus similarity ratio, C_μ is Poisson's similarity ratio, C_γ is bulk density similarity ratio, C_ϵ is strain similarity ratio, and C_δ is deformation similarity ratio. C is the similarity ratio of physical parameter between the model and the prototype, respectively. Other ratios were calculated according to their relations with the basic ratios, as shown in Table 3.

The subscripts m and p represent the model and prototype, respectively, and C_b , C_ρ , and C_a represent the similarity ratios of geometry, density, and acceleration, respectively. This study takes C_l and C_ρ as 1/50 and 1, respectively.

According to the orthogonal test theory, it is determined that the main materials of the surrounding rock are aggregate and cement. The aggregate is composed of fine sand and the cement is composed of gypsum and lime. The fine sand: cement = 4:1 and gypsum: lime = 7:3. The water content accounts for about 13%. The similar materials for

lining use gypsum with similar mechanical properties as concrete. According to existing research results, water is used. Based on the obtained similarity ratio, water: gypsum = 1:1.5. Material mechanical parameters of surrounding rock and lining model: the surrounding rock material has bulk density 17 kN/m³, cohesion 2 kPa, internal friction angle 25°, elastic modulus 0.035 GPa, and Poisson's ratio 0.37 and lining material has bulk density 24 kN/m³, elastic modulus 0.56 GPa, and Poisson's ratio 0.2. The test process is shown in Figure 4.

2.3. Test Introduction. In the test, the El-Centro wave is used as the input wave of the shaking table. The input seismic wave is selected, as shown in Figure 5. Before excitation, the white noise is scanned with a peak value of 0.07 g to make the model compact. In each subsequent set of experiments, a white noise scan was also entered to observe the changes in the dynamic characteristics of the system. Load along the cross section of the tunnel increases the input acceleration peak value (0.1 g) step by step. Before each load, a small amplitude (0.07 g) white noise excitation was input, as shown in Table 4.

The sensors used in shake table tests include accelerometers and strain gauges. The accelerometers and strain gauges were used to measure acceleration and strain on and around the tunnel lining, respectively. The sensor arrangement is also different at each test due to different test objectives. Figure 6 shows the arrangement of the instruments during the test phase. The accelerometer A01 is mounted directly on the shake table and keeps a record of the history of the input-based acceleration of the excitation. The accelerometers are arranged at the left- and right-arch shoulders above the tunnel lining; strain gauges are arranged at the left and right spandrel and skewback of the tunnel lining.

3. Analysis of Test Results

3.1. Acceleration Response Characteristics of Lining Structure. By comparing and analyzing the Fourier spectra of accelerations of the left and right spandrel of the tunnel lining of Figure 7 and 8, it can be found that the Fourier spectra of the tunnels with the same peak acceleration of the left and right spandrel are similar. Compared with the same peak acceleration of the left and right abutments, the right abutment on the biased surface facing the empty slope aggravates the dynamic stress response of the tunnel lining, and the peak value of the Fu-type spectrum is greater than the left abutment value. By analyzing the Fourier spectrum curves of different peak accelerations on the same side, it can be found that the dynamic response of the tunnel lining is positively correlated with the acceleration peak. The larger the acceleration peak, the more intense the Fourier spectrum attenuation, which also increased the peak value.

3.2. Failure Morphology of Tunnel Lining. During the test, the high-speed camera was used to record the damage process on the front and side of the tunnel section. After each

FIGURE 3: Model test device. (a) Polystyrene foam board. (b) Model box. (c) Shaking table. (d) Gravel base.

TABLE 2: Main parameters of the shaking table.

Parameters	Specification
Platform size	1.6 m × 1.2 m
Degree of freedom of motion	Single degree of freedom
Load capacity	1.5 t
Maximum displacement and acceleration	±1.5 m, ±1.0 g
Working frequency	0.01–10 Hz

TABLE 3: The similar constants.

Physical quantity	Similar relation	Similar constants
L	C_L	1/50
ρ	C_ρ	1
E	C_E	1/50
ν	C_ν	1
t	$C_t = C_L C_\rho^{0.5} C_E^{-0.5}$	0.141
a	$C_a = C_L^{-1} C_\rho^{-1} C_E$	1
u	$C_u = C_L C_E$	1/50
σ	$C_\sigma = C_E$	1/50
ε	$C_\varepsilon = C_E^{-1} C_L C_\nu$	1

excitation is completed, each damage status of the tunnel lining under earthquake effect must be checked, and the distribution of cracks must be carefully recorded. Figure 9 shows the crack expansion of a tunnel lining under an

earthquake with increased intensity. Cracks appeared in the tunnel lining at a level of 0.4 g. Subsequently, existing cracks were observed at the shoulder of the tunnel lining and new cracks appeared at a strength level of 0.5 g. The existing cracks were further expanded, and inclined cracks appeared at the strength level of 0.5 g. Figure 8 displays the distribution of final crack and damage details. Cracks appeared at skewback, side wall, and spandrel of tunnel lining under earthquake effect.

3.3. Structural Damage of Tunnel Lining. It can be seen from Figure 10 that the peak value of the axial force almost appears at the right skewback. The internal force of the peak right is greater than that of left skewback, and a larger axial force value appears when the pressure is greater. Meanwhile, the axial force values are all pressure effects. The internal force of the bending moment demonstrates positive and negative alternatives, indicating that the tunnel lining is subjected to repeated tensile and compressive cyclic loads under ground motion action. The load is reflected by alternating tensile and compressive stresses. As the fatigue load and the compressive performance of the concrete is far greater than the tensile performance, thus the lining structure is often prone to tensile damage. In summary, it can be seen that the right side of the tunnel lining is the internal force of control section, and the skewback is most likely to be damaged and the damage response of the spandrel is also large under earthquake effect.

FIGURE 4: Model test setup. (a) Lining model. (b) Smoothing of lining. (c) Pouring surrounding rock. (d) Test operating platform.

FIGURE 5: EL wave: acceleration-time histories and its corresponding Fourier spectrum.

TABLE 4: Loading sequence for the shaking table tests.

Test ID	Input waveform	PGA (g)
1	EL wave	0.1
2	EL wave	0.2
3	EL wave	0.3
4	EL wave	0.4
5	EL wave	0.5

FIGURE 6: Surrounding rock and layout of transducers (B/C), strain gauges; (A), accelerometer).

FIGURE 7: Acceleration spectrum curve of different measuring points. (a) 0.3 g A1 acceleration-time history. (b) 0.3 g A3 acceleration-time history. (c) 0.3 g A7 acceleration-time history. (d) 0.3 g A5 acceleration-time history.

FIGURE 8: Peak contrast.

4. Numerical Simulation Analysis

4.1. Boundary Conditions. In order to establish an artificial boundary, the infinite continuous medium is cut off, and a continuous spring-damper-centralized mass system is applied at the cutoff, that is, the artificial boundary, as shown in Figure 11.

In order to overcome the inconvenience caused by the actual processing and calculation, we ignore the mass M and fix one end of the damper connected to the mass M to form the artificial boundary of the viscous damper + spring. The specific implementation method is shown in Figure 12. As shown in the figure, the coordinates X and Y in the figure are tangential to the artificial boundary, and Z is the normal direction. The parameters of the physical elements on the viscoelastic artificial boundary node in the figure are

$$\begin{aligned} K_1 &= K_2 = \frac{2G}{R} \sum_{i=1}^I A_i, \\ C_1 &= C_2 = \rho c_s \sum_{i=1}^I A_i, \\ K_3 &= \frac{4G}{R} \sum_{i=1}^I A_i, \\ C_3 &= \rho c_p \sum_{i=1}^I A_i. \end{aligned} \quad (2)$$

Among them, $\sum A_i$ is the area represented by the nodes on the artificial boundary, and for the case shown in Figure 11, $I=4$.

4.2. Damage Model. It is difficult to analyze the concrete defect form and damage mechanism and determine the effective force area from the microscopic level, so indirect

methods are needed to determine the material damage. The concrete damage constitutive model is used by Lubliner et al. [23]. The strain equivalence principle is proposed by Lubliner et al. [23]. It is assumed that the strain caused by stress acting on the damaged material is equivalent to the strain caused by effective stress acting on the nondestructive material. According to this principle, the actual constitutive relationship of the damaged material can be obtained from the nominal stress in the nondestructive material:

$$\varepsilon = \frac{\sigma}{E} = \frac{\tilde{\sigma}}{E} = \frac{\sigma}{(1-D)E}, \quad \sigma = (1-D)E\varepsilon. \quad (3)$$

Among them, D is the elastic modulus damage parameter, E is the elastic modulus of the material, and E is the elastic modulus of the damaged material. From (3),

$$D = 1 - \frac{\tilde{E}}{E}. \quad (4)$$

Derivation of (3) gives

$$\frac{d\sigma}{d\varepsilon} = \frac{dE}{d\varepsilon} (1-D)\varepsilon + E(1-D) - E\varepsilon \frac{dD}{d\varepsilon}. \quad (5)$$

When the load is unloaded to a certain value, because the damage is irreversible, that is, the damage value is not reduced during the unloading process, that is, $dD/d\varepsilon = 0$, where E is the constant of the elastic modulus of the material without damage, so (5) can be changed to

$$D = 1 - \frac{1}{E} \frac{dD}{d\varepsilon}. \quad (6)$$

The elastic modulus of the damaged material is the slope of the unloading curve, so it can also be called the unloading modulus so that the elastic modulus of the damaged material can be obtained by unloading.

4.3. Model Establishment. The finite element program is used for numerical simulation. The stress and deformation of the biased tunnel are very complicated during the earthquake. It is a three-dimensional stress and deformation problem. In order to improve the accuracy of the simulation results, the three-dimensional problem is dealt with during modeling. The numerical simulation model is shown in Figure 13. This time the biased tunnel is compared with the unbiased tunnel. The elastoplastic constitutive model is adopted for the surrounding rock of the tunnel and the damage constitutive model is adopted for the tunnel lining. Seismic waves are input in the axial direction of the vertical tunnel, and the loading method is the same as the shaking table test.

4.4. Damage Evolution and Cracking Mechanism of Tunnel Lining Opening

4.4.1. Damage Evolution of Tunnel Lining Structure. According to the damage status of the tunnel lining under the earthquake, Figure 14 shows the damage cloud diagram of the tunnel lining when the damage expands at various

FIGURE 9: Lining crack. (a) Cracks in left skewback of bias section. (b) Cracks in right skewback of bias section. (c) Longitudinal cracks at right spandrel of portal section. (d) Ring crack of right spandrel in bias section. (e) Longitudinal crack of right spandrel in bias section.

FIGURE 10: Time-history diagram of internal force. (a) Maximum bending moment. (b) Maximum axial force.

FIGURE 11: Normal boundary.

FIGURE 12: Artificial boundary.

FIGURE 13: 3D bias model.

FIGURE 14: Damage map of tunnel lining.

time nodes. The direction of earthquake incidence is the same as the test direction, where damagec is the state variable for compression damage expansion and 1 represents complete damage and cracking of the unit while 0 represents

no damage and cracking of the unit. 0.4 g was adopted as the peak acceleration of seismic waves in this section.

When $t = 1.970s$, it can be seen that the outer edge of the right arch foot at the tunnel opening section first appeared a

FIGURE 15: Maximum stress.

zone of compression damage under the action of axial ground motion along the tunnel through judging (Figure 14), and the damage was along the outer edge of the tunnel lining which is extend to the inner edge. When $t = 1.994$ s, the damage area at the right spandrel continues to expand, and there is also a compression damage area at the left spandrel. When $t = 2.310$ s, the left and right spandrel of the tunnel lining appear throughout the entire tunnel lining in the form of compression damage. When $t = 2.911$ s, the damage area of the left spandrel of the tunnel lining section appeared, and the damage of the tunnel lining was further expanded until $t = 5.256$ s, and at this point, the right spandrel of the tunnel lining appeared in the form of the damage. It can be seen at $t = 5.386$ s that when the left and right spandrel and left and right skewback of the tunnel lining show damage areas with the increase of earthquake time, it shows continuous expansion trend, and the damage area continues to expand until $t = 15$ s. The damage areas of the left and right spandrel and the left and right skewback of the tunnel lining became gradually stabilized and no longer expanded. When $t = 39.95$ s, it is the cloud image of the final compressed damage area presented by the tunnel lining at the end of the earthquake.

In the damage distribution diagram in Figure 14, it is not difficult to find that the left spandrel of the tunnel lining of the tunnel section, the right spandrel, the left skewback, and the right skewback are the areas which are most prone to compressive damage from the beginning to the last compression of damage area under earthquake when tunnel is under a shallow-buried conduction, among which the right arch leg of the tunnel lining has the largest compression

damage. At the same time, it was found the damage of tunnel lining rapidly expanded within 2s under the action of earthquake between $t = 1.970$ s and $t = 2.911$ s, which has obvious impact on tunnel lining.

4.4.2. Analysis of the Maximum Principal Stress Seismic Dynamic Response of the Tunnel Lining Structure. In order to further study the damage and cracking behavior of the tunnel lining structure, the skewback and spandrel points on the tunnel lining of the opening section are selected as key points (Figure 13). Figure 15 shows the cloud diagrams of maximum principal stress of the tunnel lining structure at the earthquakes of 1.970 s, 2.310 s, and 39.95 s.

When the tunnel lining structure is subjected to damage, namely, $t = 1.970$ s, the cloud diagram of principal stress cloud diagram of the tunnel lining structure is shown in the figure. At the point, the maximum principle stress appears at skewback along negative direction of X-axis, which was 1.496 MPa. When $t = 2.310$ s, that is, the maximum principal stress at this moment is 2.657 MPa when the maximum peak intensity of the earthquake action is reached. At this time, the stress value of the tunnel lining structure along the X-axis skewback side basically reaches 2.244 MPa, indicating the maximum principal stress. The damage crack at the maximum position is caused by the concrete tensile strength that reaches the limit value. When $t = 39.95$ s, the maximum value of the tunnel maximum principal stress is located at the positive abutment position of the X-axis which is 1.513 MPa. At this time, the damage area of the tunnel lining abutment and arch foot tends to be stable, and the maximum principal stress of the tunnel lining structure is fully released.

FIGURE 16: Time history of maximum principal stress response.

TABLE 5: Key points A1–A8 maximum principal stress value and corresponding time C25 =1.78.

Key point	A1	A2	A3	A4	A5	A6	A7	A8
Stress (MPa)	1.993	1.838	1.788	1.848	1.875	1.993	1.533	1.951
Time (s)	5.418	9.565	5.289	5.386	2.939	5.321	2.825	9.257

And, the maximum value decreases and a corresponding position shift occurs.

Figure 16 shows the time-history curve of maximum principal stress at key points of the tunnel lining structure. According to Figure 16 and Table 5, it can be seen that the maximum principal stress of 1.57 MPa first appears at point A7 when $t = 2.825$ s, and when $t = 2.939$ s, that is, the maximum principal stress of 1.875 appears at the right skewback of the tunnel lining structure. At the moment, the crack is observed at A5, and the maximum principal stress of A3, A6, A4, and A1 reached 1.78 MPa successively in the following 7 seconds. The maximum principal stress of A2 and A8 finally reached 1.78 MPa, and later, crack was witnessed. After the peak value of the seismic wave, the

maximum principal stress of the corresponding key points at the various moments of the damage and crack propagation of the tunnel lining structure in the later period reached the peak value. However, all the principle stresses were less than 1.78 MPa, which indicates that the concrete cracking in the later period can realize the development of crack under less stress, which will cause damage to the concrete. When $t = 39.95$ s, the maximum principal stresses at A1–A8 key points are all closer to 1.0 MPa when ground motion stops.

4.4.3. Analysis of the Minimum Principal Stress Seismic Dynamic Response of the Tunnel Lining Structure. Figure 17 shows the cloud diagrams of minimum principal stress of the tunnel lining structure at the earthquakes times of

FIGURE 17: Minimum stress.

FIGURE 18: Continued.

FIGURE 18: Time history of minimum principal stress response.

TABLE 6: Minimum principal stress values at key points A1–A8 and corresponding times C25 = 1.78

Key point	A1	A2	A3	A4	A5	A6	A7	A8
Stress (MPa)	−1.847	−2.791	−2.029	−1.512	−2.244	−3.501	−2.248	−2.551
Time (s)	5.218	5.450	5.418	5.386	5.328	5.468	5.428	4.388

1.970 s, 2.310 s, and 39.95 s. When $t = 1.970$ s, a large stress was generated at the right skewback of the tunnel lining structure, and the minimum principal stress reached -2.114 MPa. At this time, the cracking occurred at the skewback of the tunnel lining structure. When $t = 2.310$ s, the minimum principal stress is -1.327 MPa, and the minimum principal stress witnessed an increment. Since the skewback of the tunnel is under high pressure for a long time, the cracks at the skewback is quickly generated within 1s. When $t = 39.95$ s, the minimum principal stress was transferred to the arch shoulder, and the minimum principal stress was 1.990 MPa.

Figure 18 shows time-history curve of the minimum stress at the key points of the tunnel lining structure A1–A8. By combining Table 6 and Figure 18, it can be seen that the minimum principal stress from A1 to A8 occurs at approximately 5.0 s, and the minimum principal stress of each key point stress is -2.791 MPa. The cure of minimum principal stress at each key point fluctuated slightly but all tended to be stable. It can be seen from the time chart of the minimum principal stress that the stress curve at the right skewback (A5, A6) fluctuates relatively sharply. Meanwhile, it can be found that the tunnel lining structure has a significant bias effect comparing left and right skewback and left and right spandrel at same time.

4.4.4. Verify Numerical Model. Combining the results of seismic tunnel damage investigations with model test failure locations is used to verify the maximum and minimum principal stresses and damage locations of the numerical simulation.

5. Conclusion

In this paper, a series of shaking table tests and numerical simulations were performed to the shallow-buried biased tunnels under seismic action. And, a detailed description of the design and numerical simulation of the shaking table test are also made. The damage of the tunnel lining is studied by sweep frequency of white noise and combining seismic excitation. The results of the analysis are discussed in terms of lining damage, lining acceleration, and axial force of bending moment, and this research gets these conclusions:

- (1) The lining at entrance of the shallow-buried biased tunnel exhibits a significant biasing effect.
- (2) The biasing effect promotes the unfavorable stress distribution of the tunnel structure.
- (3) Meanwhile, the serious stress concentration occurs at the lining spandrel and skewback. The unfavorable stress distribution is more likely to cause damage to the tunnel. The tunnel lining earthquake damage easily occurs at the spandrel and skewback and gradually expands with the continuous damage of the earthquake.
- (4) Under the action of the earthquake, the axial force of the tunnel structure shows a compressive effect and bending moment demonstrates a cyclic stress of tension and compression, where the bending moment at the right skewback displays the largest internal force, and it is also a serious damage area.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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