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Research Article

Research on Fatigue Performance of the Rib Beam Bridge Carriageway Slab Based on Cumulative Damage Theory

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The purpose of this paper is to study the development law of fatigue damage of the carriageway slab of reinforced concrete ribbed girder bridge and to provide theoretical support for carriageway slab design. The fatigue test of reinforced concrete rib beam bridge carriageway slabs was conducted. Based on the material fatigue damage theory, the fatigue damage model was established by ABAQUS software. The fatigue performance, failure mechanism, and the effect of fatigue level on the fatigue performance of the carriageway slab were studied. The experimental research results show that when the slab of the rib beam bridge was fatigued, and the radial crack appeared at the bottom of the slab, which belonged to punching shear failure. The fatigue failure process was divided into three stages as follows: in the initial stage of fatigue, the deflection and reinforcement strain increased linearly with fatigue times, and in the stable development stage, the bottom deflection and reinforcement strain increased steady progression. With the increase in fatigue cycles, the fatigue accumulation damage was gradually accumulated, and the stiffness of the carriageway slab gradually decreases. In the rapid development stage, the deflection, reinforcement strain, and crack growth speed were significantly increased. According to the fatigue damage model, the simulation analysis results show that the accumulated damage to the bridge carriageway was more serious with the increase in fatigue load level. The fatigue damage degree was lower when the design fatigue load level is not greater than 0.55.

1. Introduction

In small and medium-span reinforced concrete rib beam bridges, the carriageway slab is an indispensable component, which directly bears the wheel load and transmits the force to the beam rib. During the normal use of the bridge, the internal damage to the carriageway slab accumulates over a long time repeated, and the carriageway slab will lose its bearing capacity due to that cumulative damage. However, the fatigue failure design of the carriageway slab has not been paid much attention in the current design of reinforced concrete rib beam bridges.

In 1971, Batchclor et al. [1] proposed that the fatigue residual strength of the carriageway slab was about 0.5 after 2 million fatigue cycles load under the fatigue test of the scaled reinforced concrete carriageway slab.

In 1982 and 1988, Sonoda et al. [2, 3] *t* observed that the crack shape was radial when the bottom of the slab was damaged through the fatigue test of the bridge slabs.

In 2009, A. Pipinato et al. [4] conducted material properties and high-cycle fatigue shear tests on a decommissioned railway bridge. The test results showed that the riveted shear joint was replaced by preloaded bolted joints that comply with Eurocode fatigue design requirements.

Also, Pipinato [5] et al. performed static fatigue tests of the connection state of bridge rivets on the same railway bridge in 2011 and compared the fatigue design curve with the test results. The results showed that the fatigue performance of rivets was equivalent to that of nonpreloaded bolts.

In 2019, Li [6] conducted the influence of the concrete strength grade, reinforcement ratio, different load levels, length-to-short side ratio, and thickness on the fatigue performance of the bridge slabs and announced the fatigue development law and fatigue failure form of the carriageway slab.

Although scholars have carried out some research on reinforced concrete roadway slabs, the research on the fatigue performance of carriageway slabs based on fatigue tests and fatigue cumulative damage theory has not been in-depth. In this paper, the fatigue test was carried out in the carriageway slab, and a fatigue model was established based on the cumulative damage theory. The fatigue performance of the carriageway slab was studied systematically.

2. Fatigue Test Overview

2.1. Fatigue Model Design. In this test, the design of the rib beam bridge slab model was referred to as the standard drawing of a bridge with a span of 16 m [7]. The carriageway slab test model adopts the double T beam model, the concrete strength grade is C40, the test model is 4 m long, the calculated span is 3.85 m, the beam height is 325 mm, and the midspan beam rib width is 45 mm. Only the end transverse beams are set. The transverse and longitudinal reinforcement is equipped with steel bars with a diameter of 6 mm. The length-to-short side ratio of the road slab is 6.51. The construction size and reinforcement of the carriageway slab model is shown in Figure 1.

2.2. Fatigue Test Equipment and Main Test Equipment. In this paper, the PLS-500 test system was adopted in the fatigue test load device, and the YHD-200 and TDS-530 were adopted to measure the displacement and collect the data of the slab. The PLS-500 test system and TDS-530 are shown in Figures 2 and 3. The tension and pressure sensors were used to calibrate before the test. The moving head to obtain the regression equation was as follows:

$$y = 1.004x + 0.215, \tag{1}$$

where x is the actual loading value and y is the computer input value.

2.3. Fatigue Test Loading and Control. Yang et al. [8] summarized the loading methods of prestressed concrete fatigue tests by domestic and foreign scholars. Seventy percent of them used constant-amplitude fatigue loads, and the rest used variable-amplitude fatigue loads. Because the waveform of variable-amplitude fatigue is complex, the requirements for the testing machine are relatively high.

According to Yang et al. [8], seventy percent of scholars used constant-amplitude fatigue loads in the test. In this paper, the fatigue load used in this test is equal-amplitude fatigue load, and the frequency is set to 3.8 Hz. The bearing capacity of the carriageway slab is determined to be 115 kN through the static load test. According to the experience of reinforced concrete fatigue tests conducted by domestic and foreign scholars [2, 4], the fatigue load level was set to be 0.65, and the upper and lower limits of the fatigue load are 75 kN and 30 kN, and the estimated fatigue cycles are 20,000 cycles, 50,000 cycles, 100,000 cycles, 500,000 cycles, 1 million cycles, 1.5 million cycles, and 2 million cycles. In order to avoid concentrated stress on the carriageway slab, a steel plate is set under the moving head, whose area is 200×100 mm and thickness was 2 cm, and a rubber plate was set under the steel plate whose thickness was 2 cm.

2.4. Measuring Point Layout. Four concrete strain gauges and three displacement gauges were set at the bottom of the slab span of the carriageway slab, the transverse strain gauges are S1 and S2, the longitudinal strain gauges are S3 and S4, and the displacement gauges are D1 to D3. Two reinforced strain gauges S5 and S6 are arranged on the tendons. The layout of measuring points across the interrupted surface is shown in Fatigue Figure 4.

3. Carriageway Slab Test Analysis

The relationship between the deflection of the slab bottom, steel bar, and the number of fatigue load cycle times is shown in Figures 5 and 6. The cracks in the bottom of the carriageway slab after the damage is shown in fatigue Figure 7.

Based on the changes in deflection and cracks under the fatigue load cycles, the fatigue failure process of the carriageway slab can be divided into three stages as follows:

When the fatigue cycles were 0 to 100,000 cycles, the deflection, steel bar strain, and the number of fatigue cycles were linearly related. When the number of fatigue cycles reaches 50,000, a longitudinal crack with a width of 0.2 mm appears on the bottom of the slab. After that, the cracks developed slowly, and the fatigue damage to the road slab was slight, which could be regarded as the primary stage of fatigue damage.

When the fatigue cycles were 100,000 to 1,000,000, the fatigue damage of the bridge developed stably. The bottom deflection and steel bar strain increased rapidly with the fatigue cycles. The slope of the deflection curve was increased to a certain extent compared with the initial fatigue period. The fatigue accumulation damage accumulated, and the carriageway slab stiffness gradually decreased. When the fatigue cycles reached 500,000 cycles, scattering cracks developed to the beam rib with a width of 0.3 mm, and then the cracks developed steadily. When the number of fatigue cycles reaches 1 million, the longitudinal crack width reached 0.8 mm, and the maximum width of the scattering crack was 0.6 mm.

When the fatigue cycles exceeded 1 million, it was the stage of accelerated fatigue development. As the number of fatigue cycles increased, the slope of the deflection fatigue curve suddenly becomes steeper, and the growth rate of deflection, steel bar strain, and crack increase was significantly accelerated. The fatigue cycles reached 1.62 million cycles, the steel bars yielded, and the cracks at the bottom of the slab were sudden and violent. The width of the longitudinal cracks was 1.8 mm, and the maximum width of the scattering cracks was 1.6 mm, resulting in punching shear failure of the carriageway slab.

4. Based on the Theoretical Analysis of Concrete Cumulative Damage

4.1. Mathematical Model of Material Constitutive

4.1.1. Concrete Constitutive Model. The fatigue load causes the degradation of the elastic modulus and the strength of the concrete. According to the concrete constitutive

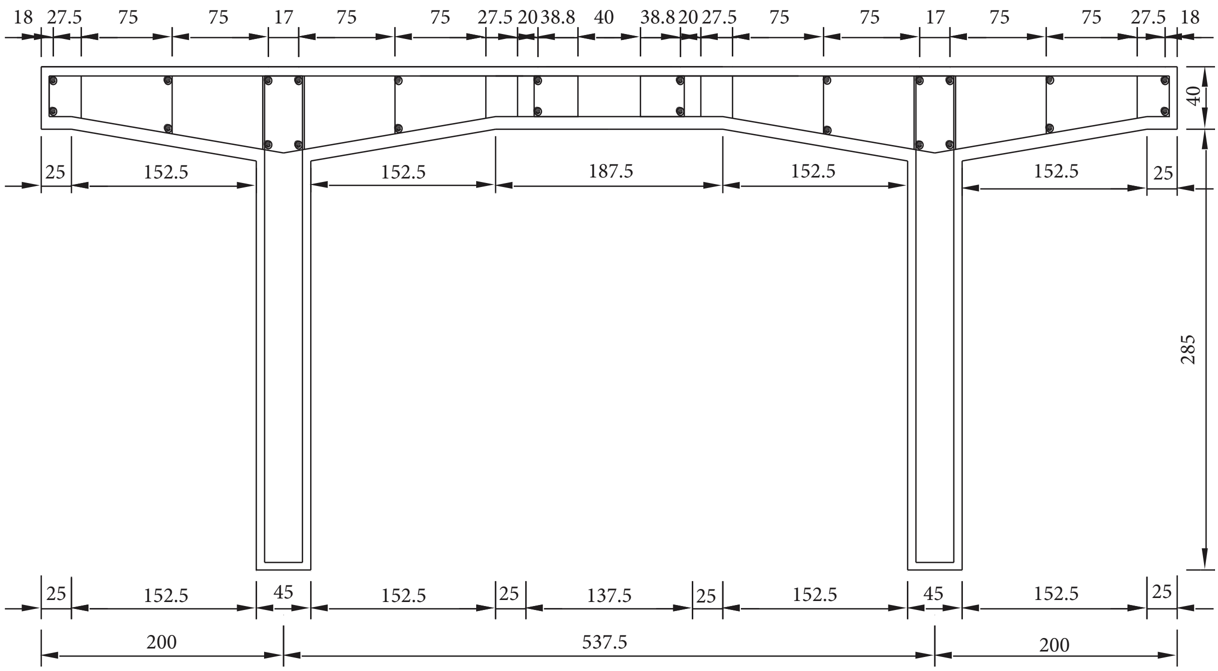

Figure 1: Structural dimension and reinforcement drawing the of carriageway slab (unit: mm).

Figure 2: PLS-500 test system.

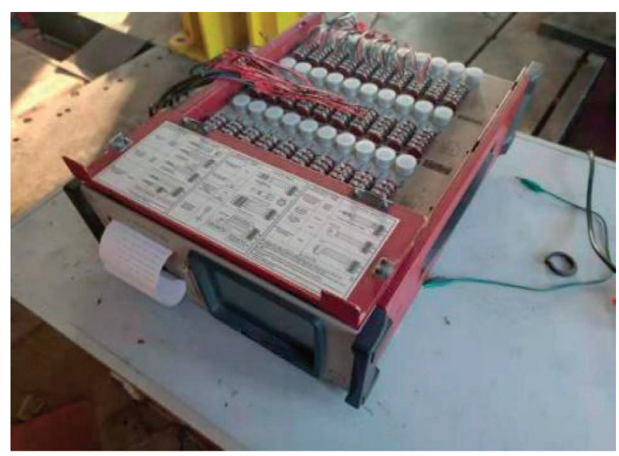

Figure 3: Japan TDS-530 data acquisition instrument.

relationship in the fatigue damage theory, it is necessary to calculate the fatigue life of the concrete, the degradation of the elastic modulus, and the residual strength of the concrete to construct the assumption of the fatigue constitutive model. Figure 8.

(1) Concrete fatigue life calculation. According to the S–N curve equation fitted by the structural fatigue task group [9], the fatigue life of concrete is calculated as follows:

$$lgN_f = 14.7 - 13.5\frac{\sigma_{\max} - \sigma_{\min}}{f_{ck} - \sigma_{\min}}, \tag{2}$$

where N_f is the fatigue life of concrete, f_{ck} is the standard value of concrete compressive strength, $\sigma_{\max}$ is the maximum stress of concrete, and $\sigma_{\min}$ is the minimum stress of concrete.

(2) The elastic modulus of concrete degrades. To calculate the degradation of the elastic modulus of concrete, Holmen [10] found that the elastic modulus of concrete was reduced by 33% when fatigue failure occurred and proposed a formula for calculating the degradation of the elastic modulus of concrete, which is as follows:

$$E_r = \left(1 - 0.33\frac{N_t}{N_f}\right)E_c, \tag{3}$$

where E_r is the fatigue residual elastic modulus of concrete, E_c is the elastic modulus of concrete which is not damaged, and N_t is the number of fatigue load cycles.

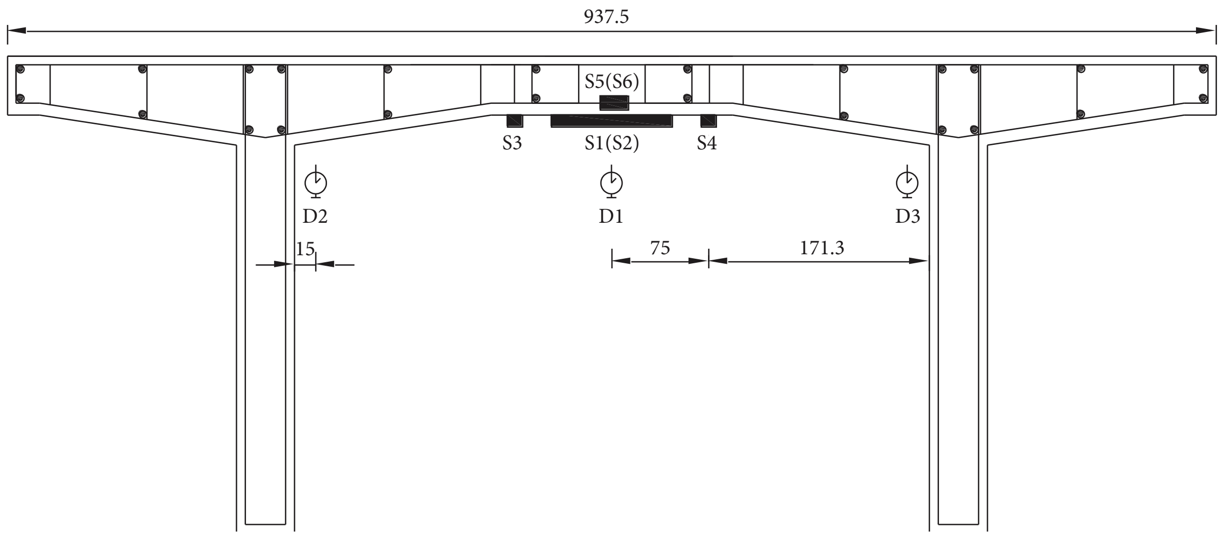

Figure 4: Layout drawing of measuring points across the interrupted surface.

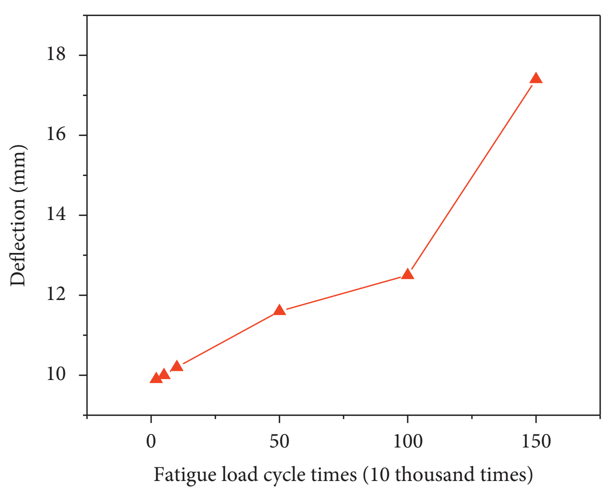

Figure 5: The relationship between the deflection of the slab bottom and the number of fatigue load cycles.

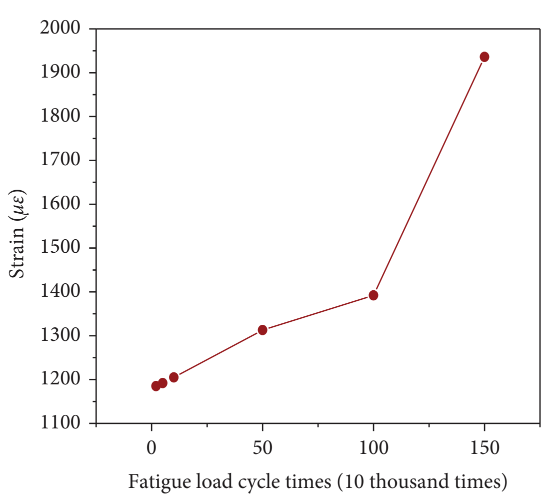

Figure 6: The relationship between the steel strain and the number of fatigue load cycles.

(3) Calculation of fatigue residual strength of concrete. When the residual fatigue strength is lower than the fatigue load limit, the concrete will be damaged, and thus the concrete fatigue failure criterion is

$$\sigma_r\left(N_f\right) \le \sigma_{\max}, \tag{4}$$

The fatigue residual strength model of concrete should not be too complicated. By comprehensively comparing the research results of domestic and foreign scholars, the residual strength formula established by J Schaff was improved, and relevant parameters were determined according to the fatigue test of Meng [11]. This model conforms to the basic rules of the residual fatigue strength model of concrete.

$$\sigma_r\left(N_t\right) = f_{cu,k} - \left[f_{cu,k} - \sigma_{\max}\right]\left(\frac{lg\left(N_t\right)}{lg\left(N_f\right)}\right)^{\nu}, \tag{5}$$

where $\sigma_r(N_t)$ is fatigue load cycling, N_t is the subfatigue residual strength of concrete, $f_{cu,k}$ is the concrete compression standard value, and υ is the test parameters determined by Meng [11] test compression 2.055, tension 2.514.

(4) Compressive and tensile constitutive relation of concrete under the damage state. Based on the fatigue constitutive model assumption of the components in Equations (2) to (5), the elastic modulus E_r and residual fatigue strength $\sigma_r(N_t)$ of concrete of any strength grade under the action of constant-amplitude fatigue load and any number of fatigue load cycles can be calculated. Finite element analysis software was used to simulate the fatigue performance of lane slabs. It is necessary to calculate the specific stress-strain relationship under different damage states and then use the above parameters to continue to deduce the fatigue damage constitutive relationship of concrete.

First, the peak compressive strain of concrete with the constitutive relation of the fatigue damage is calculated as follows:

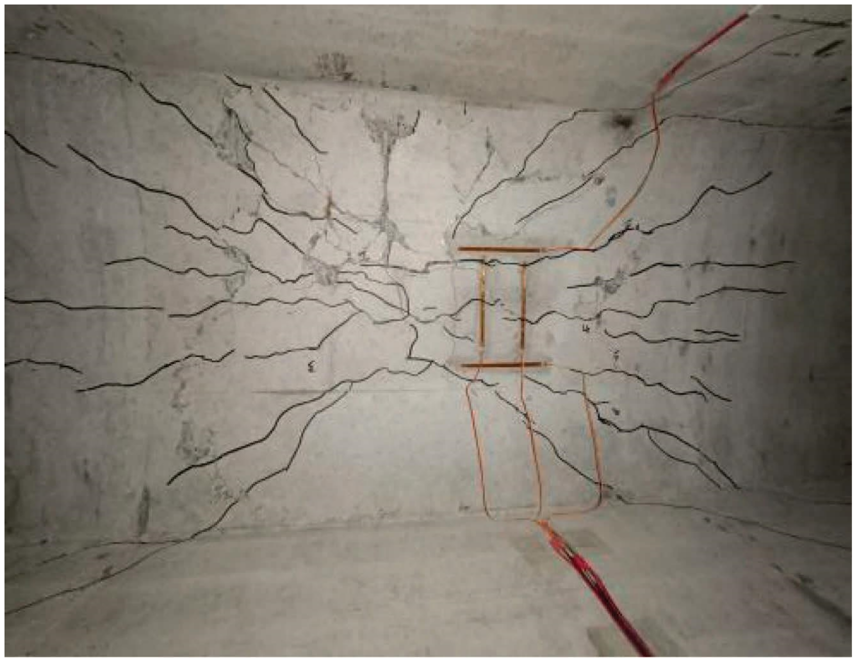

Figure 7: The cracks in the bottom of the carriageway slab after damage.

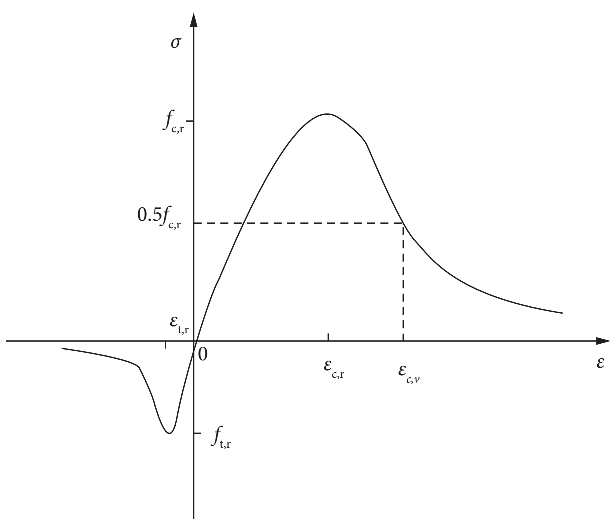

Figure 8: Uniaxial stress-strain curve of concrete.

$$\epsilon_{c,r}\left(N_t\right) = \left(700 + 172\sqrt{\sigma_{c,r}\left(N_t\right)}\right) \times 10^{-6}, \tag{6}$$

where $\varepsilon_{c,r}(N_t)$ is the peak compressive strain of concrete with N_t cycles of fatigue load, $\sigma_{c,r}(N_t)$ is the fatigue load cycle, and N_t is the time residual fatigue strength of concrete.

To calculate the damage evolution parameter d_c, the descending section parameter α_c should be considered.

$$\alpha_c = 0.157\sigma_{c,r}\left(N_t\right)^{0.785} - 0.905. \tag{7}$$

Taking the peak point as the boundary, the damage parameters of the two sections were calculated by substituting the ratio of ε to $\epsilon_{c,r}(N_t)$ x and parameter ρ_c, n:

$$x = \frac{\epsilon}{\epsilon_{c,r}\left(N_t\right)}, \tag{8}$$

$$\rho_c = \frac{\sigma_{c,r}\left(N_t\right)}{E_r \epsilon_{c,r}}, \tag{9}$$

$$n = \frac{1}{\left(1 - \rho_c\right)}. \tag{10}$$

Compression damage evolution parameters d_c are as follows:

$$d_c = \begin{cases} 1 - \dfrac{\rho_c n}{n - 1 + x^n}, & (x \leq 1) \\ \\ 1 - \dfrac{\rho_c}{\alpha_c (x-1)^2 + x}, & (x > 1) \end{cases}, \tag{11}$$

where $x \leq 1$ is the ascending segment; $x > 1$ is the descending segment.

At this point, a concrete fatigue constitutive model was established. As shown in formula (12), the calculated stress is actually nominal stress, which needs to be transformed into real stress in the concrete plastic damage constitutive model.

$$\sigma_{\text{nom}} = \left(1 - d_c\right) E_r \epsilon. \tag{12}$$

The calculation of the tensile constitutive relation of concrete under any damage is the same, which is not described too much. See Formula (13).

$$\sigma = \left(1 - d_t\right) E_r \epsilon. \tag{13}$$

Owing to space limitations, only the calculation results of concrete fatigue damage when the fatigue load level is 0.65 and the fatigue cycles are 500,000 are given, as shown in Tables 1 and 2, and the elastic parameters of concrete are listed in Table 3.

Table 1: Plastic constitutive model of concrete under compressive damage under 1.5 million fatigue loads.

Measured stress (MPa)	Inelastic strain	Damage factor	Inelastic strain
12.6250	0.000000	0.000000	0.000000
18.0358	0.000601	0.000010	0.000601
13.8205	0.002221	0.000530	0.002221
9.9474	0.003824	0.000674	0.003824
7.6077	0.005354	0.000754	0.005354
6.1199	0.006843	0.000802	0.006843
5.1065	0.008309	0.000835	0.008309
3.8277	0.011199	0.000877	0.011199
3.0593	0.014058	0.000902	0.014058

Table 2: Plastic constitutive model of concrete under tensile damage under 500,000 cycles of fatigue load.

Measured stress (MPa)	Inelastic strain	Damage factor	Inelastic strain
2.2020	0.000000	0.000000	0.000000
1.2521	0.000140	0.000461	0.000140
0.8368	0.000259	0.000640	0.000259
0.6415	0.000367	0.000727	0.000367
0.5281	0.000471	0.000779	0.000471
0.4533	0.000573	0.000813	0.000573
0.3597	0.000775	0.000856	0.000775
0.2635	0.001175	0.000899	0.001175

Table 3: Concrete material parameters.

Density (Tonne/mm^3)	Young's modulus (MPa)	Poisson's ratio	Expansion Angle (°)	Eccentricity	Double uniaxial compressive strength ratio	Yield surface coefficient	Viscous parameters
2.8×10^{-12}	32500	0.2	30	0.1	1.16	0.667	0.0005

4.1.2. Constitutive Model of Rebar. In this paper, the ideal elastoplastic model, namely the double linear model, was adopted, as shown in Figure 9. This model is more suitable for this test because it is applied to steel bars with a longer flow amplitude. It is assumed that in any fatigue loading, the yield stress in the fatigue constitutive model is the residual fatigue strength of steel bars, the elastic modulus of steel bars does not degrade in the fatigue loading process [12], the stress $\sigma_{\min}$ corresponding to the limit value under a constant fatigue load is a certain value, and the strength damage of steel bars in the fatigue loading process conforms to the linear damage criterion.

(1) Calculate the fatigue life of reinforcement. In the life curves of N_f and $\Delta\sigma$ obtained from fatigue tests of deformed steel bars in China, the relationship between A and B is log-log-linear, and the general expression is shown as 3–13:

$$lgN_f = b + klg(\Delta\sigma), \tag{14}$$

where N_f is the fatigue life of reinforcement, b, k is the test constant obtained by experiment, and $\Delta\sigma$ is the reinforcement stress amplitude.

The research object of this study is the $S-N$ curve relation equation of the fatigue life of deformed steel bars.

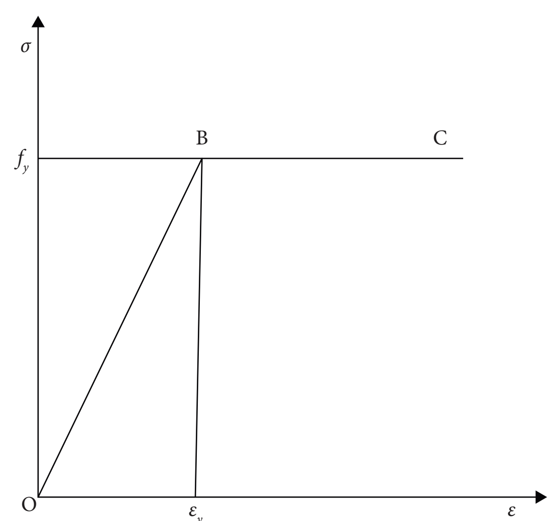

Figure 9: Ideal elastoplastic model.

The formula proposed by the China Academy of Railway Sciences [13] was as follows:

$$lgN_f = 15.1348 - 4.3817lg(\Delta\sigma). \tag{15}$$

(2) Calculate residual fatigue strength of reinforcement. Based on the residual strength model proposed by Zhu [14] and combined with the test type in this study, logarithm and parameter corrections were carried out to obtain the formula of residual strength of steel bar at any number of times:

$$\sigma_r(N_t) = f_y\left[1 - \frac{lg(N_t)}{lg(N_f)}\left(1 - \frac{\sigma_{\max}}{f_y}\right)\right], \quad (16)$$

where f_y is the yield strength of reinforcement, N_t is the fatigue cycle number, N_f is the fatigue life, and σ max is the reinforcement stress.

(3) Tensile constitutive relation of reinforcement under fatigue cumulative damage. Finally, the tensile constitutive relation of steel bars under the fatigue cumulative damage state was determined, and the residual fatigue strain of the steel bars was calculated. Because the elastic modulus of steel bars did not degenerate during the fatigue loading process, the residual fatigue strain of steel bars $\Delta\varepsilon_r(N-1)$ can be obtained, and its calculation formula is as follows:

$$\Delta\epsilon_r(N_t - 1) = \frac{[\sigma_r(N_t) - \sigma_r(N_{t-1})]}{E_s}. \quad (17)$$

Then, according to the constitutive relation of the elastic-plastic model of reinforcement and the formula of the residual strength of reinforcement, the yield strain $\epsilon_y(N_t)$ corresponding to the yield of reinforcement can be obtained, and its calculation formula is as follows:

$$\epsilon_y(N_t) = \Delta\epsilon_r(N_t - 1) + \frac{\sigma_r(N_t)}{E_s}. \quad (18)$$

Based on the above assumptions and formulas, the calculation formula (19) of the fatigue constitutive model under any number of fatigue loads is obtained based on the double-line ideal elastic-plastic model and the fatigue cumulative damage theory. The model curve is shown in Figure 10.

$$\sigma(N_t) = \begin{cases} E_s\epsilon(N_t), & \left(\Delta\epsilon_r(N_t - 1) < \epsilon(N_t) \le \epsilon_y(N_t)\right) \\ \sigma_{\max}\left[1 - \dfrac{lg(N_t)}{lg(N_f)}\left(1 - \dfrac{\sigma_{\max}}{f_y}\right)\right], & \left(\epsilon(N_t) > \epsilon_y(N_t)\right) \end{cases}. \quad (19)$$

4.2. Fatigue Model. The fatigue model was established by ABAQUS software. The steel bar is selected as a T3D2 truss element, and the concrete is selected as a C3D8R solid element. Material parameters are entered and assigned to the part according to Section 3. To improve the calculation accuracy, the element mesh at the midspan section of the carriageway slab is refined. The reinforcement cage and concrete are embedded in the area, and the model adopts the restraint method of simply supported beams. The full Newton iteration method (full Newton method) is used, the conversion is severely discontinuous, and the method inherited from the previous analysis step is adopted, and the slope load is used in the entire analysis step by default. The carriageway slab model is shown in Figure 11.

4.3. Fatigue Analysis Effectiveness. To verify the validity of the finite element model, fatigue test analysis is carried out on the fatigue test model. The comparison between the results of the simulation analysis and the test results are shown in Figures 12 and 13.

It can be announced that the measured test curve was in good agreement with the simulation analysis curve. The simulation analysis curve was also divided into three stages. The number of fatigue loads was 0–100,000 cycles, and the deflection, strain, and fatigue times are linearly related. The number of fatigue cycles was between 100,000 and 1 million cycles, and the growth rate of deflection and strain was faster than that in the initial stage of fatigue. When the number of fatigue was between 1 million and 1.5 million cycles, the deflection and strain increased rapidly.

Under the same number of fatigue cycles, the maximum error between the simulation analysis deflection value and the measured value was 4%, and the maximum error between the strain value and the measured value was 4.8%. The steel strain was measured at 1.62 million cycles, and the simulated analysis was 1.58 million cycles. The error of the fatigue life was 2.53%, which proved that the model could simulate the fatigue performance of the carriageway slab well.

5. The Effect of the Fatigue Load Level on Fatigue Performance

To study the influence of the fatigue load level on the fatigue performance of the carriageway slab, the fatigue damage model established based on the fatigue damage theory is used to carry out the fatigue test analysis with the fatigue load levels of 0.55, 0.65, and 0.75, respectively. The relationship curve between the deflection of the carriageway slab and the fatigue load level is shown in Figures 14 and 15.

The simulation analysis results show that under the condition of a certain number of fatigue loads, the higher the fatigue load level, the greater the deflection of the carriageway slab and the more the accumulated damage. When the fatigue load level was 0.55, with the increase of the fatigue

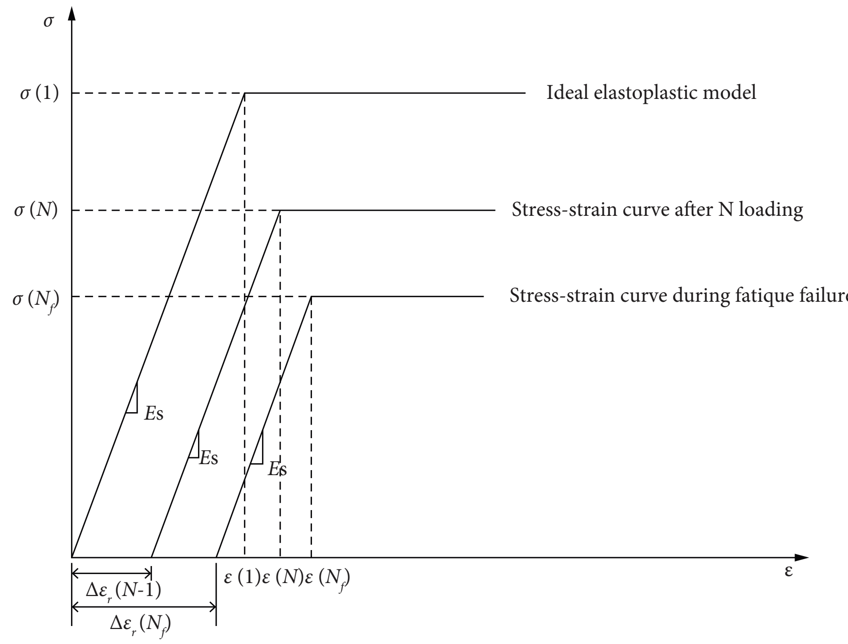

Figure 10: Constitutive model of reinforcement under fatigue load damage.

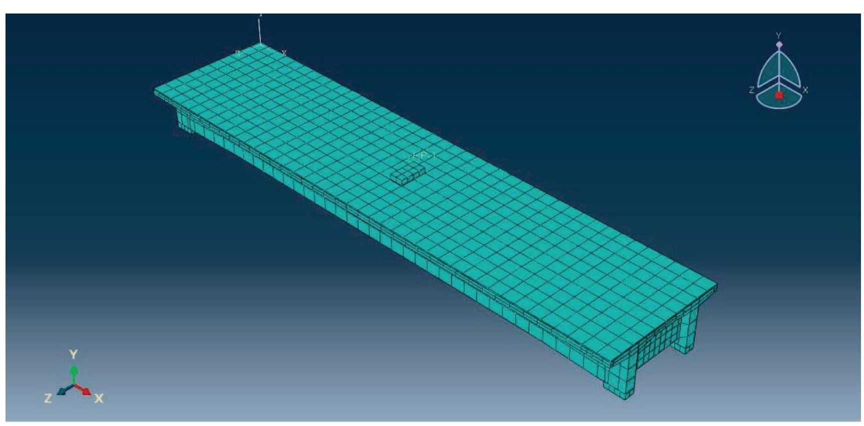

Figure 11: Carriageway slab model.

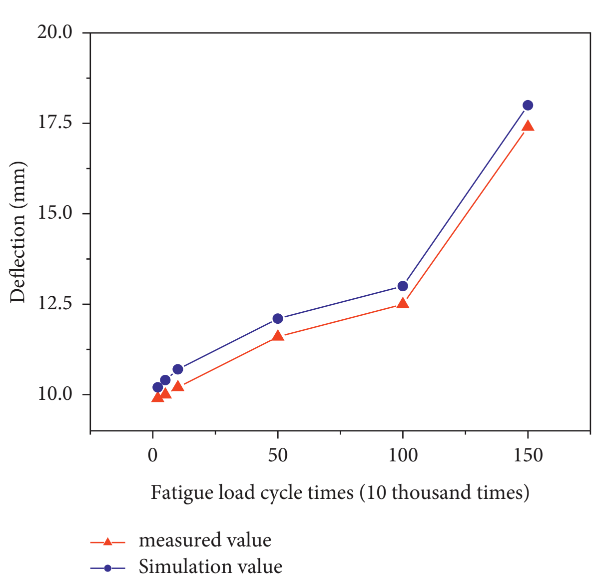

Figure 12: The relationship between the deflection of the slab bottom and the number of fatigue load cycles.

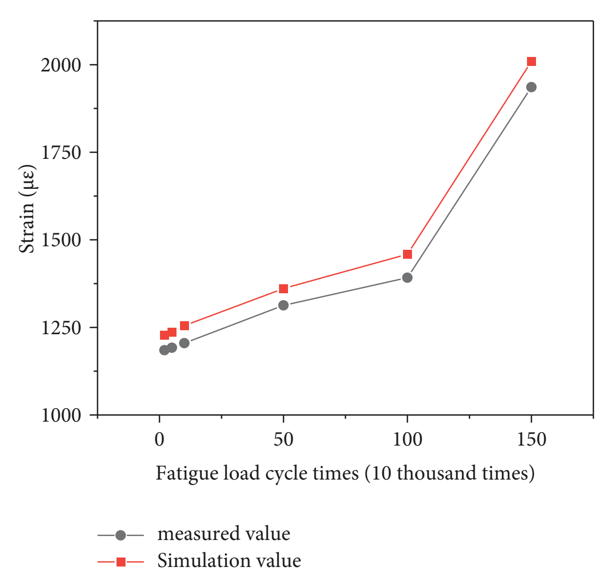

Figure 13: The relationship between the steel strain and the number of fatigue load cycles.

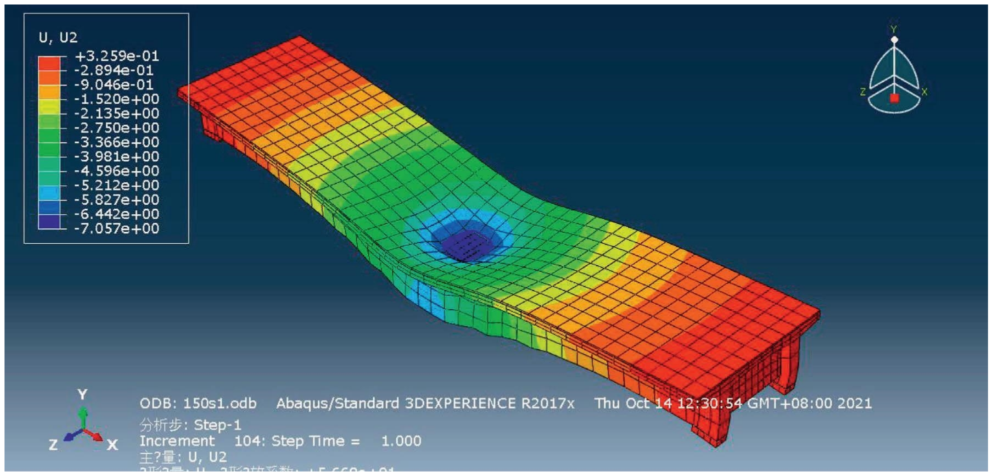

Figure 14: Fatigue load level 0.55, 1.5 million times deflection graphic.

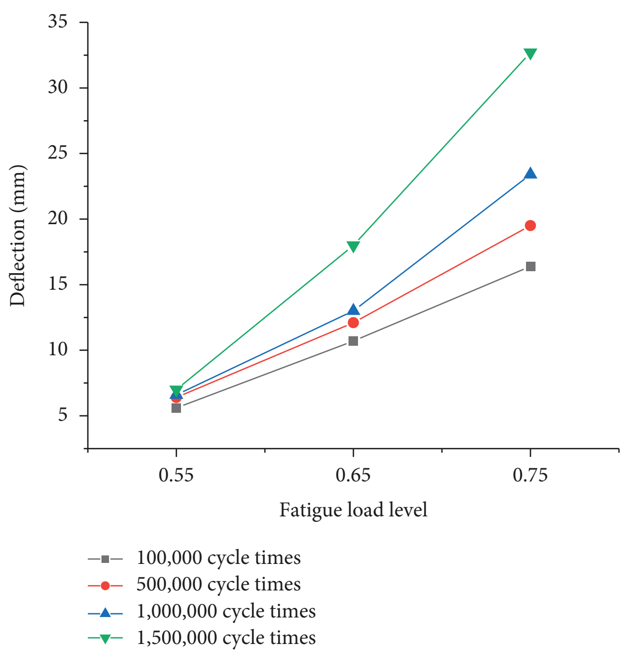

Figure 15: Horizontal relation curve between deflection and the fatigue load level.

load level, the deflection did not increase significantly. When the fatigue load level was 0.65, the deflection increased by 7.3 mm, which was 5.6 times the fatigue load level of 0.55. When the level was 0.75, the deflection increased by 16.3 mm, which was 12.5 times the fatigue load level of 0.55 and 2.2 times that of 0.65.

Liu [15] conducted a fatigue test of a reinforced concrete bridge carriageway slab. When the fatigue load level was 0.629, the fatigue damage degree of the bridge deck was more than 1.5 million times, which was similar to the test results. When the load levels were 0.270 and 0.386, no fatigue damage occurs. Through this test, it is considered that this conclusion is too conservative, and the design fatigue load level of the carriageway slab is recommended not to exceed 0.55.

6. Conclusion

(1) The mechanism of fatigue failure of the rib beam bridge carriageway slab is revealed. The cracks at the bottom of the slab are radial and belong to punching shear failure.

(2) The fatigue failure process is divided into three stages:

  (1) When the fatigue cycles were 0–100,000 cycles, the deflection, steel bar strain, and the number of fatigue were linearly related.

  (2) When the fatigue cycles were 100,000–1,000,000, the fatigue damage of the bridge developed stably. The bottom deflection and steel bar strain increased rapidly with the fatigue cycles. The slope of the deflection curve was increased to a certain extent compared with the initial fatigue period. The fatigue accumulation damage gradually accumulated, and the carriageway slab stiffness gradually decreased.

  (3) When the fatigue cycles exceeded 1 million, it was the stage of accelerated fatigue development. As the fatigue increased, the slope of the deflection fatigue time-history curve suddenly becomes steeper, and the growth rate of deflection, steel bar strain, and crack growth was significantly accelerated.

(3) The fatigue model was established based on the cumulative damage theory, and the measured test curve is in good agreement with the simulation analysis curve, which verifies the validity of the model. The simulation analysis curve is also divided into three stages. Under the same number of fatigue loads, the maximum error between the simulation analysis deflection value and the measured value was 4%, and the maximum error between the strain value and the measured value was 4.8%. The steel strain was measured at 1.62 million cycles, and the simulated analysis was 1.58 million cycles. The fatigue

life of both of them was about 1.6 million times, which proved that the model could simulate the fatigue performance of the carriageway slab well.

(4) Using the fatigue model established based on the fatigue damage theory, the fatigue test analysis was carried out with the fatigue load levels of 0.55, 0.65, and 0.75, respectively. The simulation analysis results show that under the condition of a certain number of fatigue loads, the higher the fatigue load level, the greater the acceleration of the deflection of the roadway slab and the more the accumulated damage. Based on the conclusions of other scholars, it is considered that the design fatigue load level of the carriageway slab should not be greater than 0.55.

Data Availability

The opju data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

The study was approved by Northeast Forestry University Civil Engineering.

References

[1] B. V. Batchclor, B. E. Hcwitt, and P. Csagoly, "An investigation of the fatigue strength of deck slabs of composite steel-concrete bridge[J]," *Transportation Research Record*, vol. 664, pp. 153–161, 1978.

[2] K. Sonoda and T. Horikawa, "Fatigue strength of reinforced concrete slabs under moving loads[C]," in *Proceedings of the IABSE Colloquium, Fatigue of Steel and Concrete Structures*, pp. 445–462, IABSE, Lausanne, 1982.

[3] K. Sonoda, T. Horikawa, and H. Kobayashi, "Fatigue failure mechanism of bridge deck RC slabs[R]," in *Proceedings of the First East Asian Conference on Structual Engineering and Construct*, Bangkok, September 1986.

[4] A. Pipinato, C. Pellegrino, O. S. Bursi, and C. Modena, "High-cycle fatigue behavior of riveted connections for railway metal bridges," *Journal of Constructional Steel Research*, vol. 65, no. 12, pp. 2167–2175, 2009.

[5] A. Pipinato, M. Molinari, C. Pellegrino, O. S. Bursi, and C. Modena, "Fatigue tests on riveted steel elements taken from a railway bridge," *Structure and Infrastructure Engineering*, vol. 7, no. 12, pp. 907–920, 2011.

[6] H. Li, *Research on Fatigue Performance of Reinforced Concrete Carriageway Slab and Diaphragms of T-Beam Bridge[D]*, Northeast Forestry University, 2019.

[7] CCCC Second Highway Survey and Design Institute, *General Diagram of Bridges and culverts[Z]*, CCCC Second Highway Survey and Design Institute, Beijing, 2015.

[8] Ou Yang, X. Zhang, J. Huo, and J. Chen, "Research Status of fatigue performance of prestressed concrete beam[J]," *Journal of Architecture and Civil Engineering*, vol. 34, no. 04, pp. 85–95, 2017.

[9] Concrete fatigue group, *Study on Checking Calculation Method of Fatigue Reliability of concrete Flexural Members[J]*, China Architecture and Architecture Press, Beijing, 1994.

[10] J. O. Holmen, "Fatigue of concrete by constant and variable amplitude loading," *ACI Special Publications*, vol. 75, pp. 71–110, 1982.

[11] X. Meng, *Experimental and Theoretical Research on Residual Strength of Concrete under Fatigue Loading[D]*, Dalian University of Technology, 2006.

[12] Y. Song, S. Zhao, R. Wang, and S. Li, "Nonlinear finite element analysis of fatigue behavior of reinforced concrete[J]," *Hai Yang Xue Bao*, no. 03, pp. 116–125, 1993.

[13] M. Yao, *Study on Fatigue Behavior of concrete under Constant Amplitude and Variable Amplitude Repeated stress[R]*, Report of Scientific Research Institute of Railway Ministry, 1990.

[14] X. Zhu, *Numerical Simulation of Fatigue Failure Process for In-Service Reinforced Concrete Bridge[D]*, Tianjin university, 2010.

[15] Zi-zheng Liu, Yu Tian-lai, and L.-lin Zhang, "Study on the effect of load level and carriageway slab thickness on fatigue performance of concrete T-girder bridges[J]," *International Journal of Structural Integrity*, vol. 2, 2020.