Research on the Residual Bearing Capacity of a Rib Beam Bridge Carriageway Slab Based on Fatigue Cumulative Damage

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Abstract: In order to study the residual bearing capacity of the carriageway slab of a reinforced concrete rib beam bridge after fatigue damage, a double T-beam carriageway slab model with a high degree of symmetry was designed and fatigue tests were carried out. Based on the fatigue cumulative damage theory, the fatigue residual strength equation of concrete was improved and a constitutive fatigue model of a steel bar and concrete with arbitrary loading times was established. Because the carriageway slabs are highly symmetrical, we used a ratio of length to width and a ratio of width to height to express the dimensional changes in the carriageway slabs. Abaqus general analysis program was used to create a symmetrical T-beams model and analyze the influence of the fatigue load level, the ratio of width to height of the roadway slab, and the ratio of length to width on the remaining bearing capacity of the roadway slab. Finally, the 1stOpt fitting software was employed to fit the roadway slab’s residual bearing capacity evaluation model. The fatigue test results show that the fatigue failure form of the roadway slab is punching failure, and the development law of fatigue damage can be divided into three stages, accounting for 6.25%, 56.25%, and 37.5% of the total fatigue life. The finite element analysis results showed that the length–width ratio of the roadway slab is positively correlated with its relative residual bearing capacity, and the residual bearing capacity of the two-way slab is higher than that of the one-way slab. The fatigue load level and the aspect ratio of width to height are negatively correlated with their relative residual bearing capacity.

Keywords: reinforced concrete rib beam bridge; roadway slab; fatigue test; cumulative damage; residual bearing capacity

1. Introduction

The reinforced concrete girder bridge is typical in existing bridge structures. As an important part of the roadway slab, it undertakes the task of directly bearing the wheel load and transferring the load to the main beam. In-service bridge travel lane slabs often experience plate surface cracking, plate surface concrete breakage, travel lane slab local brittle fracture, and other diseases. The main reason for this type of damage is the fatigue damage of the roadway slab due to the development of internal damage points in the reinforcement and concrete materials under repeated wheel loads. However, many countries still only consider the design of carriageway slabs using a static load, without taking the fatigue caused by heavy traffic into account [1]. Thus, the fatigue study of reinforced concrete ribbed girder bridge travel lane slabs remains critical for fatigue-resistance design.

In recent years, scholars from various countries have conducted in-depth research on the development pattern of fatigue injury and resistance to fatigue injury, and good results have been achieved.

Research has been carried out on the fatigue damage mechanism of concrete materials [2–9], the final fatigue damage form of travelway slabs [10–12], and the fatigue...
damage mechanism of travelway slabs [13,14]. Ferdous et al. studied the residual capacity of railroad ties after fatigue loading [2] and predicted the fatigue life of GFRP composites [3]. Hümmel J et al. investigated further questions concerning strain and stiffness development and the combination of fatigue loading and marine environmental conditions with new test methods [4]. Lanwer J P et al. investigated the bonded tensile properties of high-strength fiber concrete under fatigue loading using tensile fatigue tests [7]. Deng P et al. studied the fatigue life prediction method of RC plates under stamping shear failure mode based on fracture mechanics [11]. Yu T et al. investigated the effect of long-to-short edge ratio on the fatigue performance of ribbed girder bridge carriageway slabs [14]. However, there is little in-depth research analyzing the residual bearing capacity of reinforced concrete travelway slabs after experiencing different fatigue load cycles, and how to evaluate and effectively predict their residual bearing capacity.

In addition, previous studies have mainly considered the effects of external factors on the fatigue performance of reinforced concrete carriageway slabs [14]. These include fatigue loading stress level, the number of cycles of fatigue loading [15], low-to-high stress ratio, the location of fatigue loading [16], type of fatigue loading [4], and environmental conditions [17–19]. There are fewer studies on the effects arising from internal factors such as the width-to-height [19] and length-to-width ratios [14] of the travel lane slab. This has led to a lack of theoretical support for designers in the fatigue-resistant design of travel lanes.

In view of this, in this study, we carried out fatigue tests on a carriageway slab. The effect of fatigue load level and carriageway slab length to width and width to height ratios on the residual bearing capacity of the carriageway slab was studied based on the accumulation injury theory. A fatigue residual load-carrying capacity evaluation model was then established for travel lane slabs to provide a scientific basis for load-carrying capacity evaluation of in-service bridges and fatigue resistant design of a reinforced concrete ribbed girder bridge.

Compared with other studies, this thesis more comprehensively considers the influence of external factors and internal factors on the fatigue residual bearing capacity of the traveled way slab and establishes a prediction model of the residual bearing capacity of the traveled way slab with the influence of multiple factors, instead of the influence of a single factor. This makes the prediction model more realistic and reasonable. The results of the study can provide important theoretical support for the evaluation of residual load capacity and fatigue resistance design of bridges.

2. Experiment Overview

2.1. Overview of the Test Beam

In consideration of the limitations and costs of indoor testing, this test is based on a similar principle of designing a scaled-down travel lane slab with a model that focuses on the mechanical properties of concrete and tensile reinforcement.

Ziming Liu [20] suggested that the mechanical properties of the prototype beam and the test beam can be studied by satisfying the following conditions: (1) the strain and reality remain equal; (2) the model stress is equal to reality; and (3) the strain or stress maintains a certain proportional relationship. According to these three conditions, similar ratios of bending moments and so on can be found using Table 1. For parameters such as concrete strength grade, reinforcement location, reinforcement rate, and reinforcement strength grade model and prototype beams are kept the same, which simplifies the analysis, and it is seen that the geometric dimension ratio of the two is the ratio of the modulus of elasticity. Based on the similarity of the material modulus of elasticity and geometry combined with Table 1, the similarity coefficients of each physical quantity of the model and prototype beams in Table 2 can be derived. Therefore, this experimental model can replace the prototype beam and reflect the real bridge rib beam.
Table 1. Table of load conversion relationship between model beam and prototype beam.

| Load Type                              | Strain Relations | Stress Relations | Displacement Relationships |
|----------------------------------------|------------------|------------------|-----------------------------|
| Concentrated load $F$                  | $\varepsilon_p = \frac{F_p}{F_m} \left( \frac{L_m}{L_p} \right)^2 \varepsilon_m$ | $\sigma_p = \frac{F_p}{F_m} \left( \frac{L_m}{L_p} \right)^2 \sigma_m$ | $\Delta_p = \frac{F_p}{F_m} \left( \frac{L_m}{L_p} \right)^2 \varepsilon_m \Delta_m$ |
| Bending moment $M$                     | $\varepsilon_p = \frac{M_p}{M_m} \left( \frac{L_m}{L_p} \right)^3 \varepsilon_m$ | $\sigma_p = \frac{M_p}{M_m} \left( \frac{L_m}{L_p} \right)^3 \sigma_m$ | $\Delta_p = \frac{M_p}{M_m} \left( \frac{L_m}{L_p} \right)^3 \varepsilon_m \Delta_m$ |
| Unit length of Uniform load $q'$      | $\varepsilon_p = \frac{q_p}{q_m} \frac{L_m}{L_p} \varepsilon_m$ | $\sigma_p = \frac{q_p}{q_m} \frac{L_m}{L_p} \sigma_m$ | $\Delta_p = \frac{q_p}{q_m} \frac{L_m}{L_p} \varepsilon_m \Delta_m$ |
| Unit area on Uniform load $q$          | $\varepsilon_p = \frac{q_p}{q_m} \frac{E_p}{E_m} \varepsilon_m$ | $\sigma_p = \frac{q_p}{q_m} \frac{E_p}{E_m} \sigma_m$ | $\Delta_p = \frac{q_p}{q_m} \frac{E_p}{E_m} \varepsilon_m \Delta_m$ |
| Uniform load per unit volume (self-weight) $g$ | $\varepsilon_p = \frac{g_p}{g_m} \frac{L_p}{L_m} \varepsilon_m$ | $\sigma_p = \frac{g_p}{g_m} \frac{L_p}{L_m} \sigma_m$ | $\Delta_p = \frac{g_p}{g_m} \frac{L_p}{L_m} \varepsilon_m \Delta_m$ |

Table 2. The summary table of the similarity coefficients of various physical quantities of fatigue phenomenon between the model beam and the prototype beam.

| Project           | Geometric Dimensions | Area | Elastic Modulus | Poisson Ratio | Reinforcement Ratio | Stress | Strain | Deflection | Bending Moment | Concentrated loads |
|-------------------|----------------------|------|-----------------|---------------|---------------------|--------|--------|------------|-----------------|-------------------|
| Model Beam        | $L_m$                | $A_m$| $E_m$           | $\mu_m$       | $\rho_m$            | $\varepsilon_m$ | $\sigma_m$ | $l_m$ | $M_m$ | $F_m$ |
| Prototype Beam    | $L_p$                | $A_p$| $E_p$           | $\mu_p$       | $\rho_p$            | $\varepsilon_p$ | $\sigma_p$ | $l_p$ | $M_p$ | $F_p$ |
| similarity coefficient | 1/4                 | 1/16 | 1               | 1             | 1                   | 1      | 1      | 1           | 1/64            | 1/16              |

A symmetrical double T-beam model using two identical T-beams was designed with reference to the literature [21]. The test model of the carriageway slab was designed with a symmetrical double T-beam. If the three-beam and two-plate are situated at the double-point plate loading, they will cancel each other out. As single-point loading has the most unfavorable working conditions, in favor of safety considerations, this paper calculates the single-point lane plate loading.

The one-way carriageway slab was 3850 mm long by 512.5 mm wide by 40 mm thick, with a long-to-short side ratio of 7:50. The model was made of C40 concrete, and the carriageway slab was configured with HRB400 grade reinforcement. Six bars of 6 mm in diameter and six bars of 3 mm in diameter were used for the horizontal reinforcement, and 94 bars of 6 mm in diameter were used for the vertical reinforcement, with a section reinforcement ratio of 0.0057. Two travel lane plates, ST and FT, were prefabricated for the test. ST was the reference carriageway slab for determining the load capacity test of the carriageway slab; FT was used for fatigue testing. The structural dimensions and reinforcement of the model are shown in Figure 1.

Figure 1. Test model construction dimensions and reinforcement diagram (unit: mm).
During the fabrication of the test beam for the travel lane slab, test on the mechanical properties were conducted for the steel and concrete materials. According to the specifications [22], three standard pieces were made for tensile, and three standard cubic compressive specimens and three standard axial compressive specimens were reserved for the concrete of each beam and tested in accordance with the specification [23]. The validity of the test data was verified by using statistical theory. The test results are shown in Tables 3 and 4.

### Table 3. Yield strength and ultimate tensile strength of the main reinforcement under stress.

| Number | Measured Diameter | Measured Strength (MPa) | Average Value (MPa) | Standard Deviation | Coefficient of Variation | Strength Standard Value (MPa) |
|--------|-------------------|-------------------------|---------------------|-------------------|--------------------------|-------------------------------|
| HPB 6-1 | 5.84              | 377                    | 369.00              | 6.16              | 0.02                     | 358.86                        |
| HPB 6-2 | 5.72              | 362                    |                     |                   |                          |                               |
| HPB 6-3 | 5.98              | 368                    |                     |                   |                          |                               |
| HPB 6-1 | 5.84              | 512                    |                     |                   |                          |                               |
| HPB 6-2 | 5.72              | 503                    | 507.67              | 3.68              | 0.01                     | 501.61                        |
| HPB 6-3 | 5.98              | 508                    |                     |                   |                          |                               |
| HRB 6-1 | 5.84              | 424                    |                     |                   |                          |                               |
| HRB 6-2 | 5.72              | 432                    | 426.67              | 3.77              | 0.01                     | 420.46                        |
| HRB 6-3 | 5.98              | 424                    |                     |                   |                          |                               |
| HRB 6-1 | 5.84              | 644                    |                     |                   |                          |                               |
| HRB 6-2 | 5.72              | 616                    | 628.67              | 11.59             | 0.02                     | 609.61                        |
| HRB 6-3 | 5.98              | 626                    |                     |                   |                          |                               |
| HRB 10-1 | 9.80             | 414                    |                     |                   |                          |                               |
| HRB 10-2 | 9.92             | 433                    | 422.33              | 7.93              | 0.02                     | 409.29                        |
| HRB 10-3 | 9.96             | 420                    |                     |                   |                          |                               |
| HRB 10-1 | 9.80             | 624                    |                     |                   |                          |                               |
| HRB 10-2 | 9.92             | 637                    | 628.33              | 6.13              | 0.01                     | 618.25                        |
| HRB 10-3 | 9.96             | 624                    |                     |                   |                          |                               |

### Table 4. Measured value of concrete compressive strength.

| Number | Measured Strength (MPa) | Size Conversion | Average Value (MPa) | Standard Deviation | Coefficient of Variation | Cube Compressive Strength (MPa) | Axial Compressive Strength (MPa) |
|--------|-------------------------|-----------------|---------------------|-------------------|--------------------------|---------------------------------|---------------------------------|
| 1      | 42.4                    | 40.28           | 40.69               | 0.40              | 0.01                     | 40.04                          | 26.82                           |
| 2      | 43.4                    | 41.23           |                     |                   |                          |                                 |                                 |
| 3      | 42.7                    | 40.57           |                     |                   |                          |                                 |                                 |
| 4      | 34.1                    | 32.40           |                     |                   |                          |                                 |                                 |
| 5      | 38.5                    | 36.58           | 36.07               | 2.82              | 0.08                     | 31.44                          | 21.06                           |
| 6      | 41.3                    | 39.24           |                     |                   |                          |                                 |                                 |
| 7      | 36.1                    | 34.30           |                     |                   |                          |                                 |                                 |
| 8      | 39.2                    | 37.24           | 37.46               | 2.68              | 0.07                     | 33.05                          | 22.14                           |
| 9      | 43.0                    | 40.85           |                     |                   |                          |                                 |                                 |
| 10     | 44.7                    | 42.47           |                     |                   |                          |                                 |                                 |
| 11     | 28.1                    | 26.70           | 40.90               | 1.57              | 0.04                     | 38.32                          | 25.67                           |
| 12     | 41.4                    | 39.33           |                     |                   |                          |                                 |                                 |

### 2.2. Measurement Point Layout

(1) Strain measurement points.

Two concrete strain gauges of type 120-100AA, noted as S1–S4, were fixed in horizontal and vertical directions at the bottom of the interrupted panel across the carriageway slab. One strain gage of type 120-3AA was attached to each of the two transverse reinforcement bars in the carriageway slab, noted as S5 and S6, respectively.

(2) Deflection measurement points.

A total of six deflection measurement points were employed and placed at the span at the bottom edge of each rib in the pivot section. One deflection measurement point was also placed at the bottom span edge of the carriageway slab. These measurement points are noted as D1–D7, respectively. Each deflection measurement point used a displacement meter with a range of 20 cm and an accuracy of 0.01 mm.

The strain and deflection measurement points are arranged as shown in Figures 2 and 3.
To ensure efficient data acquisition, a TDS-530 static data acquisition instrument was used for strain and deflection data acquisition, and a ZBL-F101 crack observer was used to observe the cracks. Photographs of the instruments in action are shown in Figure 5.

A total of six deflection measurement points were employed and placed at the span ends and mid-spans of the test beam. These measurement points are noted as D1–D7, respectively. Each deflection measurement point used a displacement transducer with a range of 20 cm and an accuracy of 0.01 mm.

Figure 3. Top view of strain and deflection measurement points (unit: mm).

2.3. Test Apparatus and Devices

(1). Test loading.

A PLS-500 electro-hydraulic servo dynamic and static test system was used to load the test beam at the mid-span position. The effect of the paving layer on the 45° diffusion of the wheel load was also considered [24]. The area of the loading head was calculated as a rectangle of 200 mm in the transverse bridge direction and 100 mm in the longitudinal bridge direction to simulate the effect of the wheel load.

The loading device is shown in Figure 4.

Figure 4. Diagram of the loading device.
(2). Collection instruments.

To ensure efficient data acquisition, a TDS-530 static data acquisition instrument was used for strain and deflection data acquisition, and a ZBL-F101 crack observer was used to observe the cracks. Photographs of the instruments in action are shown in Figure 5.

![Data collector](image)

**Figure 5.** Data collector.

### 2.4. Fatigue Testing

**(1) Reference beam static load test.**

The static load test was loaded by force-controlled loading, which was then changed to displacement-controlled loading after the steel reinforcements yielded. Loading was graded at 10 KN per stage. When the carriageway slab was about to crack and was close to damage, the grading was appropriately increased in order to determine its cracking load and ultimate load-carrying capacity. To ensure the accuracy of the test loading, the fatigue testing machine was calibrated before the formal test with a tensile pressure sensor to determine the dynamic head. The linear regression equation after calibration was obtained as:

\[
y = 1.004x + 0.215
\]

where \(x\) is the actual loading value, and \(y\) is the computer input value.

**(2) Fatigue load test.**

Results obtained from fatigue tests on reinforced concrete carried out by domestic and foreign scholars show that when a fatigue load level of 0.65 is selected, the carriageway slab can guarantee fatigue damage within 2 million cycles [25–28]. Thus, the fatigue load level in our experiment was set at 0.65.

The fatigue test was performed using force-controlled loading, using a sine wave that was closer to the moving vehicle load with a loading frequency of 3.8 Hz. The fatigue load had an upper limit of 75 KN, a lower limit of 15 KN, and an amplitude of 30 KN.

### 3. Fatigue Test Analysis of the Carriageway Slab

#### 3.1. Reference Beam Static Load Test

The measured deflection at the bottom of the ST reference beam, the transverse tensile strain in the concrete, and the tensile strain in the transverse reinforcement with load are shown in Figure 6. The crack development diagram and damage pattern are shown in Figures 7 and 8, respectively.

As can be seen from Figure 6, the damage process of the carriageway slab under static loading is roughly divided into three stages. When \(P < 40\) KN, the structure is basically in an elastic state, linear increase in carriageway slab deflection, reinforcement and concrete strain. When \(P = 40\) KN, the concrete at the bottom edge of the slab is cracked...
in tension, and longitudinal cracks appear. The development of deflection, reinforcement, and concrete strain at the bottom of the slab is accelerated at this point. When P = 80 KN, the stress redistribution phenomenon occurs in the carriageway slab, and diagonal cracks appear. Thereafter, the reinforcement yields, and the crack at the bottom of the slab increases suddenly, eventually reaching a crack width of 1.4 mm. This is followed by a loud noise, and the roadway slab is damaged in the form of punching damage. The final crack morphology is a family of scattered cracks from the center of the plate to the long-span direction, these cracks are largely symmetrical in distribution, as shown in Figures 7 and 8. The final load capacity of the carriageway slab is approximately 115 KN.

![Figure 6. Deflection and strain curve.](image)

![Figure 7. Damage pattern at the bottom of the carriageway slab.](image)
3.2. Analysis of Fatigue Test Results

The fatigue loading was suspended when the number of fatigue cycles reached 0, 100,000, 500,000, 1,000,000, and 1,500,000. A static load of 0–75 KN was then, respectively, applied. The carriageway slab crack, slab bottom deflection, and reinforcement strain data are shown in Figures 9–12.

![Figure 9](image_url)

**Figure 9.** Strain versus number of cycles of fatigue loading for FT tensile reinforcement under different loads.
Figure 10. Relationship between FT mid-span deflection and number of fatigue load cycles under different loads.

Figure 11. Fatigue damage pattern.

Figure 12. Fatigue crack development in the beam rib at 1.62 million cycles: (a) left beam rib (b) left beam rib.
It can be observed in Figures 9 and 10 that the deflection of the carriageway slab and the strain of the reinforcement under different loads shows a three-stage trend with an increase in fatigue number. When the number of fatigue cycles is within 100,000, the reinforcement strain and the deflection at the bottom of the slab increase linearly, and no significant macroscopic cracks are seen. When the number of fatigue cycles exceeds 100,000, a longitudinal crack of 0.2 mm in width appears at the bottom of the plate, followed by a scattered family of cracks centered on this crack. There is a roughly steady linear increase in reinforcement strain and slab bottom deflection at this stage, but the increase is smaller. When the number of fatigue cycles exceeds one million, the strain on the steel increases rapidly, the deflection at the bottom of the plate increases rapidly and the steel yields into the plastic phase. Yielding of the reinforcement proves the start of damage to the travel lane slab. The final carriageway slab experiences punching shear damage after 1.6 million fatigue cycles, at which point the maximum crack width at the bottom of the slab reaches 1.2 mm.

4. Carriageway Slab Fatigue Damage Model

4.1. Reinforcement Fatigue Damage Model

An ideal elastic-plastic model, i.e., a bilinear model, is employed in this work. This model is used for steel bars with longer flow widths and is more suitable for our tests. Before constructing the constitutive fatigue model of steel bars under arbitrary fatigue loading times, the following assumptions are made:

(1) During any fatigue loading, the reinforcing steel constitutive model is an ideal elasto-plastic model, and the yield stress in its constitutive fatigue model is the residual fatigue strength of the reinforcing steel.

(2) The elastic modulus of the steel bar does not degrade during the fatigue loading process [29];

(3) The stress (σ-min) corresponding to the lower limit of the equal amplitude fatigue load is a certain value during fatigue loading, and the strength damage of the reinforcement during fatigue loading is in accordance with the linear damage criterion.

4.1.1. Calculation of the Fatigue Life of Steel Bars

It is known from rebar fatigue life tests that there is a double log-linear relationship between $N_f$ and $\Delta \sigma$ in the life curve of rebar [29]. The object of study in this paper is deformed rebar, and the equation of the S-N curve relationship for its fatigue life is adopted from Equations (1), as proposed by Zeng and Li [30].

$$\lg N_f = 15.1348 - 4.3817 \lg (\Delta \sigma)$$

where $N_f$ represents the fatigue life, and $\Delta \sigma$ represents the stress amplitude.

4.1.2. Calculation of Fatigue Residual Strength of Steel Reinforcement

Based on the residual strength model proposed by Zhu Xiancun [31] and combined with the type of testing in this paper, logarithmic as well as parametric corrections are used to obtain Equation (2) to determine the residual strength of the reinforcement at any number of times.

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

where $f_y$ represents the yield strength of the reinforcement; $N_t$ represents the number of fatigue cycles; $N_f$ is the fatigue life; $\sigma_{\text{max}}$ is the stress in the reinforcement.
4.1.3. Constitutive Relation of Reinforcement in Tension under Fatigue Cumulative Damage State

It is also necessary to calculate the fatigue residual strain of the reinforcement to determine the constitutive relation of reinforcement in tension under the fatigue cumulative damage state. As the modulus of elasticity of the reinforcement does not degrade during fatigue loading, the fatigue residual strain of the reinforcement \( \Delta \varepsilon_r \) can be calculated, which is calculated as shown in Equation (3).

\[
\Delta \varepsilon_r (N_t - 1) = \left[ \sigma_r(N_t) - \sigma_r(N_t-1) \right] / E_s
\]  

(3)

The yield strain \( \varepsilon_y(N_t) \) corresponding to when the reinforcement yields can be obtained from the elastic-plastic model’s constitutive relation to the reinforcement and the residual strength formula of the reinforcement. Its calculation method is shown in Equation (4).

\[
\varepsilon_y(N_t) = \Delta \varepsilon_r(N_t - 1) + \frac{\sigma_r(N_t)}{E_s}
\]  

(4)

From the above assumptions, as well as the formula, based on the bilinear ideal elastoplastic model, as well as the fatigue cumulative damage theory, the constitutive fatigue model of steel reinforcement under any number of fatigue loading is obtained as Equation (5), and the model curve is shown in Figure 13:

\[
\sigma(N_t) = \begin{cases} 
E_s \varepsilon(N_t) & (\Delta \varepsilon_r(N_t - 1) < \varepsilon(N_t) \leq \varepsilon_y(N_t)) \\
\sigma_{max} \left[ 1 - \frac{\varepsilon(N_t)}{\varepsilon_y(N_t)} \left( 1 - \frac{\sigma_{max}}{\sigma_y} \right) \right] & (\varepsilon(N_t) > \varepsilon_y(N_t))
\end{cases}
\]  

(5)

Figure 13. Constitutive model of reinforcement under fatigue load damage.

4.2. Fatigue Damage Model for Concrete

The fatigue cumulative damage theory and the constitutive model for concrete in the literature [32] clearly indicate that the calculation of concrete fatigue life, modulus of elasticity degradation, and concrete residual strength must be obtained to construct a constitutive model for concrete. The uniaxial stress-strain relationship curve of concrete is shown in Figure 14. The following assumptions are made before constructing the constitutive model for concrete: Under fatigue loading, the modulus of elasticity of concrete is only related to the number of fatigue cycles;
4.2. Fatigue Damage Model for Concrete

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- Under fatigue loading, the modulus of elasticity of concrete is only related to the number of fatigue cycles;
- The fatigue residual strength of concrete at fatigue failure is equal to the stress corresponding to the peak strain of the concrete constitutive relation curve;
- Under fatigue loading, the tensile and compressive constitutive consists of two parts before the peak strain, a straight line and a curve, which are represented by a straight line. After the peak strain, the concrete constitutive fatigue is the same as the concrete uniaxial constitutive softening section.

4.2.1. Calculation of Fatigue Life of Concrete

In this paper, the fatigue life of concrete is calculated using the S-N curve equation fitted by the Fatigue Thematic Group [33], as shown in Equation (6):

$$\log N_f = 14.7 - 13.5 \frac{\sigma_{\text{max}} - \sigma_{\text{min}}}{f_{\text{ck}} - \sigma_{\text{min}}}$$  (6)

In the equation, $N_f$ represents the fatigue life of concrete; $f_{\text{ck}}$ represents the standard value of concrete compressive strength; $\sigma_{\text{max}}$ represents the maximum concrete stress; $\sigma_{\text{min}}$ represents the minimum value of concrete stress. $N_f$ represents the fatigue life of concrete.

4.2.2. Degradation of the Modulus of Elasticity of Concrete

The degradation of the modulus of elasticity of concrete is the most important factor contributing to the degradation of concrete stiffness. Holmen [34] found that the elastic modulus decreased by 33% when concrete was fatigued. This paper adopts Holmen’s formula, shown in Equation (7):

$$E_r = \left(1 - 0.33 \frac{N_t}{N_f}\right)E_c$$  (7)

where $E_r$ represents the fatigue residual modulus of elasticity of concrete; $E_c$ represents the modulus of elasticity of undamaged concrete; $N_t$ represents the number of cycles of fatigue loading.

4.2.3. Calculation of the Fatigue Residual Strength of Concrete

When the fatigue residual strength is lower than the upper limit of the fatigue load, the concrete is damaged, and the concrete fatigue damage criterion is as Equation (8):

$$\sigma_r(N_f) \leq \sigma_{\text{max}}$$  (8)
A fatigue residual strength model for concrete should not be overly complex. Comparing the research results of domestic and foreign scholars, we improved the residual strength formula established by J Schaff, and the relevant parameters were determined according to the fatigue tests of Xianhong Meng [35]. The model conforms to the basic laws of fatigue residual strength for concrete, as shown in Equation (9):

$$\sigma_r(N_t) = f_{cu,k} - \left(f_{cu,k} - \sigma_{\max}\right)^\nu$$

where $\sigma_r(N_t)$ represents the fatigue residual strength of concrete for $N_t$ cycles of fatigue loading; $f_{cu,k}$ represents the standard value of the compressive strength of the concrete; $\nu$ represents the test parameter, which is determined by Meng Xianhong’s test as 2.055 in compression and 2.514 in tension.

4.2.4. Compressive and Tensile Constitutive Relationships of Concrete in a State of Cumulative Fatigue Damage

Employing Equations (6)–(9) and the assumptions used to construct the constitutive fatigue model of concrete, the modulus of elasticity $E_r$ and the fatigue residual strength $\sigma_r(N_t)$ can be calculated for concrete of any strength class under an equal amplitude fatigue load for an arbitrary number of fatigue load cycles. The above parameters can then be used to continue deriving the fatigue damage constitutive structure relationship for concrete.

The peak compressive strain of concrete for the fatigue damage constitutive structure relationship is calculated as follows.

Equation (10) is first used to calculate the peak compressive strain of concrete for the fatigue damage constitutive relation:

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

where $\varepsilon_{c,r}(N_t)$ represents the peak compressive strain of concrete for $N_t$ fatigue load cycles; $\sigma_{c,r}(N_t)$ represents the fatigue residual strength of concrete for $N_t$ cycles of fatigue load.

To calculate the damage evolution parameter, it is necessary to consider the descent section parameter $\alpha_c$.

$$\alpha_c = 0.157\sigma_{c,r}(N_t)^{0.785} - 0.905$$

Using the peak point as a demarcation, the damage parameters are calculated for each of the two segments, bringing in the ratio $x$ of $\varepsilon$ to $\varepsilon_{c,r}(N_t)$ and the parameters $\rho_c$, $n$:

$$x = \varepsilon / \varepsilon_{c,r}(N_t)$$

$$\rho_c = \frac{\sigma_{c,r}(N_t)}{E_r\varepsilon_{c,r}}$$

$$n = \frac{1}{1 - \rho_c}$$

The pressure damage evolution parameter $d_c$ is denoted as:

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

where $x \leq 1$ is the ascending section and $x > 1$ is the descending section.
The concrete fatigue principal structure can be established at this point. Equation (16) shows that the calculated stress is actually nominal stress, which needs to be converted to real stress in the principal concrete plastic damage structure model.

$$\sigma_{\text{nom}} = (1 - d_c)E_r\varepsilon$$  \hspace{1cm} (16)

The constitutive relationship of concrete in tension under arbitrary damage is similar to the process of establishing the constitutive model of concrete in compression damage, which will not be repeated here, and is denoted as Equation (17).

$$\sigma = (1 - d_c)E_r\varepsilon$$  \hspace{1cm} (17)

Based on the above concrete uniaxial constitutive model, a fatigue cumulative damage model is established. Due to space limitations, only the fatigue damage calculation results of the NT-1 carriageway slab at 1 million fatigue cycles are given here in Tables 5 and 6.

**Table 5.** Plastic constitutive damage of concrete under 1 million fatigue loads.

| Real Stress (MPa) | Inelastic Strain | Damage Parameter | Inelastic Strain |
|-------------------|------------------|------------------|------------------|
| 9.4626            | 0.000000         | 0.000000         | 0.000000         |
| 13.5180           | 0.000710         | 0.000010         | 0.000710         |
| 11.7388           | 0.002121         | 0.000551         | 0.002121         |
| 9.6247            | 0.003546         | 0.000668         | 0.003546         |
| 8.0358            | 0.004945         | 0.000738         | 0.004945         |
| 6.8643            | 0.006323         | 0.000783         | 0.006323         |
| 5.9794            | 0.007685         | 0.000815         | 0.007685         |
| 4.7450            | 0.010381         | 0.000858         | 0.010381         |
| 3.9308            | 0.013051         | 0.000884         | 0.013051         |

**Table 6.** Tensile damage plastic constitutive of concrete under 1 million fatigue loads.

| Real Stress (MPa) | Inelastic Strain | Damage Parameter | Inelastic Strain |
|-------------------|------------------|------------------|------------------|
| 1.6464            | 0.000000         | 0.000000         | 0.000000         |
| 1.6301            | 0.000010         | 0.000010         | 0.000010         |
| 1.1526            | 0.000116         | 0.000441         | 0.000116         |
| 0.8591            | 0.000214         | 0.000606         | 0.000214         |
| 0.6964            | 0.000306         | 0.000693         | 0.000306         |
| 0.5930            | 0.000396         | 0.000746         | 0.000396         |
| 0.5208            | 0.000484         | 0.000783         | 0.000484         |
| 0.4256            | 0.000657         | 0.000830         | 0.000657         |
| 1.6464            | 0.000000         | 0.000000         | 0.000000         |

5. Study of the Residual Load Carrying Capacity of the Carriageway Slab

Based on the fatigue accumulation damage theory, we used the Abaqus general analysis program to establish different ratio of width to height, different ratio of length to width, different fatigue load level, and different fatigue load cycle of 32 models in total. Because the carriageway slabs are highly symmetrical, we used the ratio of length to width and the ratio of width to height to express the dimensional changes in the carriageway slabs. The lane plate residual bearing capacity finite element analysis model is shown in Table 7. T3D2 truss units were used for the model reinforcement, and C3D8R solid units were used for the concrete. An embedded region defined the contact between the reinforcement cage and the concrete. The restraint of a simply supported beam and the application of concentrated forces by means of a reference point restraining the force zone were simulated. The model was solved implicitly (Abaqus/Standard), using the direct method. Full Newton iterations (full Newton method) were used, with severely discontinuous iterations of the transformation, employing the inheritance from the previous analysis step and defaulting
to slope loads throughout the analysis step. The concrete finite element mesh is shown in Figure 15a, and the steel finite element mesh is shown in Figure 15b.

In order to verify the validity of the finite element model, the fatigue test analysis was performed on the fatigue test model, and the comparison between the results of the simulation analysis and the test results are shown in Figures 16 and 17. The deflection stress cloud is shown in Figure 18.

As can be seen from the figure, the measured test curve matches well with the simulated analysis curve. The simulated analysis curve is also divided into three stages; the number of fatigue loads is 0–100,000 times, and the deflection and strain are linearly related to the number of fatigue times; the number of fatigue times is 100–1,000,000 times, and the growth rate of deflection and strain is slow; the number of fatigue times is 1–1,500,000 times, and the growth of deflection and strain is accelerated.

**Table 7.** Finite element model statistics for carriageway slabs.

| Beam   | Number of Horizontal Dividers | Length to Width La/Lb | Width to Height Lb/h | Fatigue Load Levels | Remarks                  |
|--------|--------------------------------|-----------------------|----------------------|---------------------|--------------------------|
| NT1    | No centre crossbeam           | 7.50                  | 12.8                 | 0.55                |                          |
| NT2    | No centre crossbeam           | 7.50                  | 12.8                 | 0.55                |                          |
| NT3    | One centre crossbeam          | 3.80                  | 12.8                 | 0.65                |                          |
| NT4    | Two centre crossbeams         | 1.60                  | 12.8                 | 0.75                |                          |
| NT5    | Three centre crossbeams       | 1.88                  | 12.8                 | 0.65                |                          |
| NT6    | Three centre crossbeams       | 1.88                  | 10.3                 | 0.65                | Fatigue loading times of 500,000, 1,000,000, 1,500,000, and 2,000,000 cycles |

**Figure 15.** Concrete and Rebar finite element mesh: (a) Concrete (b) Rebar.

**Figure 16.** The relationship between the deflection of the bottom of the plate and the number of cycles of fatigue load.
The maximum error between the simulated deflection value and the measured value is 4% and the maximum error between the strain value and the measured value is 4.8% for the same number of fatigue loadings. When the fatigue damage occurred in the lane slab at 1.62 million times, the measured steel strain value was 2045 με, and this value was used as the yield strain of the steel, but in the simulation analysis, the strain value of the steel reached 2130 με at 1.62 million times, and the strain value of the steel was 2009 με at 1.5 million times, so it was considered that the steel yielded between 1.5 and 1.62 million times, and the results were close to the test. It proves that the model can simulate the fatigue performance of the travel lane slab well.

Figure 16. The relationship between the deflection of the bottom of the plate and the number of cycles of fatigue load.

Figure 17. The relationship between the strain of the reinforcement and the number of cycles of fatigue load.

Figure 18. The deflection stress cloud.

The maximum error between the simulated deflection value and the measured value is 4% and the maximum error between the strain value and the measured value is 4.8% for the same number of fatigue loadings. When the fatigue damage occurred in the lane slab at 1.62 million times, the measured steel strain value was 2045 με, and this value was used as the yield strain of the steel, but in the simulation analysis, the strain value of the steel reached 2130 με at 1.62 million times, and the strain value of the steel was 2009 με at 1.5 million times, so it was considered that the steel yielded between 1.5 and 1.62 million times, and the results were close to the test. It proves that the model can simulate the fatigue performance of the travel lane slab well.
5.1. Effect of Fatigue Load Level on the Residual Load Capacity of the Carriageway Slab

Under the conditions of $\frac{L_a}{L_b} = 7.5$ and $\frac{L_b}{h} = 12.8$, the residual load-carrying capacity of the carriageway slabs at different fatigue load cycles for three fatigue load levels $S_1 = 0.55$, $S_2 = 0.65$, and $S_3 = 0.75$ are shown in Table 8. Part of the concrete and steel damage cloud diagrams are shown in Figures 19–21. Figure 22 shows the relationship between the residual bearing capacity of the horizontal carriageway slab with different fatigue loads and the number of cycles of fatigue load.

Table 8. Residual bearing capacity of damaged road slab (KN).

| Fatigue Load Level | 500,000 Times | 1000,000 Times | 1500,000 Times | 2000,000 Times |
|-------------------|--------------|--------------|--------------|--------------|
| $S_1$             | 94.71        | 90.55        | 87.59        | 85.29        |
| $S_2$             | 79.20        | 75.55        | 72.30        | 49.44        |
| $S_3$             | 77.41        | 72.44        | 32.60        | —            |

Figure 19. Fatigue load level $S_1$, fatigue is 1.5 million times the residual load-bearing capacity concrete damage cloud. (a) Top view cloud map; (b) Elevation map cloud map.
As can be seen from Figure 22, the greater the fatigue load level, the lower the residual load capacity of the carriageway slab at the same number of cyclic loads. When the load level is 0.55, the residual bearing capacity of the carriageway plate with the increase in the number of fatigue loading does not appear to significantly reduce the trend. After cyclic loading 2 million times, its residual bearing capacity is 85.29 KN, and the plate bearing capacity is 74%. When the fatigue load level is 0.65, the residual load capacity-fatigue loading curve takes a clear turn at 1.5 million cycles, and the residual load capacity is only 43% of the plate load capacity after 2 million cycles of fatigue loading. At a fatigue load level of 0.75, the curve takes a significant turn at 1 million cycles, and the residual load capacity is only 28% of the plate load capacity at 1.5 million cycles of loading. The fatigue load level is also increased from 0.65 to 0.75, and the fatigue curve turning point is lowered from 1.5 to 1 million cyclic loadings, an advance of 500,000 cycles.

Figure 20. Fatigue load level S1, fatigue is 2 million times the residual load-bearing capacity concrete damage cloud. (a) Top view cloud map; (b) Elevation map cloud map.
Figure 21. Fatigue load level S1, fatigue is 2 million times residual load-bearing capacity reinforcement stress cloud. (a) Main View Cloud Map; (b) Top view cloud map.
Figure 22. Residual load-carrying capacity versus number of cycles of fatigue loading.

5.2. Influence of Length to Width Ratio on the Residual Load-Carrying Capacity of the Carriageway Slab

Fatigue analysis models with La/Lb ratios of 7.5, 3.8, 1.9, and 1.6 were obtained by varying the number and spacing of the crossbeam at a fatigue load level of 0.65 and Lb/h = 12.8. The results are shown in Table 9. Due to the different initial bearing capacities of different La/Lb carriageway slabs, the residual bearing capacity of each model’s carriageway slab was converted to a relative residual bearing capacity using Equation (18), and the calculation results are shown in Figure 20.

\[ P_c = \frac{P_r - P_u S}{P_u - P_u S} \]  

where \( P_c \) represents the relative residual bearing capacity of the carriageway slab, \( P_r \) represents the residual bearing capacity of the carriageway slab, \( P_u \) represents the undamaged carriageway slab bearing capacity, and \( S \) represents fatigue load level.

Table 9. Residual load capacity of damaged carriageway slabs (KN).

| La/Lb | 500,000 Times | 1,000,000 Times | 1,500,000 Times | 2,000,000 Times |
|-------|---------------|----------------|----------------|-----------------|
| 7.5   | 79.20         | 76.36          | 72.37          | 49.46           |
| 3.8   | 85.44         | 83.00          | 77.49          | 62.19           |
| 1.9   | 98.21         | 92.26          | 87.38          | 79.03           |

Figure 23 illustrates that the smaller the La/Lb of the carriageway slab, the greater its relative residual load capacity under the same number of fatigue loading. The relative residual load capacity of two-way board slabs with La/Lb below 2 is significantly higher than that of the one-way board. At a fatigue level of 0.65 and cyclic loading of 1 million cycles, the relative residual load capacity of a carriageway slab with La/Lb of 7.5 is 0.12, while the residual load capacity of a carriageway slab with La/Lb of 1.6 is 0.34 of the pre-damage load capacity, which is 2.8 times higher than La/Lb = 7.5. Therefore, from the point of view of the residual bearing capacity of the carriageway slab, it is recommended to use a two-way board with La/Lb less than 2 when designing the carriageway slab for fatigue resistance.
5.3. Effect of Width to Height Ratio on the Residual Load-Carrying Capacity of the Carriageway Slab

Under the condition that the length to width ratio of the carriageway slab is 1.9 and the fatigue load level is 0.65 (both are certain), the fatigue analysis model with different Lb/h is used to calculate the relative residual bearing capacity for 500,000, 1,000,000, and 1,500,000 cycles of loading. The modeling results are shown in Table 10.

Table 10. Residual load capacity of damaged carriageway slabs (KN).

| Lb/h   | 500,000 Times | 1,000,000 Times | 1,500,000 Times | 2,000,000 Times |
|--------|---------------|-----------------|-----------------|-----------------|
| 12.8   | 105.03        | 101.28          | 98.19           | 79.30           |
| 11.4   | 100.18        | 97.49           | 93.44           | 76.22           |
| 10.3   | 98.26         | 92.00           | 87.46           | 79.10           |

As can be seen from Figure 24, under the condition that La/Lb and the fatigue load level are certain, the smaller the Lb/h of the carriageway slab, the greater the relative residual bearing capacity of the carriageway slab. The relative residual load capacity of the carriageway slab with Lb/h = 12.8 at a fatigue level of 0.65 and a cyclic load of 1.5 million cycles is 0.16, while the relative residual load capacity of the carriageway slab with Lb/h = 10.3 is 0.38, which is nearly twice as high as Lb/h = 12.8.
5.4. Residual Load-Carrying Capacity Evaluation Model for Carriageway Slabs

In order to evaluate the residual bearing capacity of different types of carriageway slabs, this paper takes the length to width ratio \( \frac{L_a}{L_b} \), the width to height ratio \( \frac{L_b}{h} \), the fatigue load level, and the number of fatigue load cycles as independent variables and the relative residual bearing capacities of the carriageway slabs as dependent variables. The general global optimization algorithm is used to fit the mathematical evaluation model of the residual bearing capacity of the carriageway slab using 1stOpt fitting software. The residual bearing capacity of each model is converted into the relative residual bearing capacity according to Equation (18), and the mathematical expression of the evaluation model is denoted in Equation (19).

\[
y = \frac{p_1(p_6 + p_8 x_1 + p_{10} x_2 + p_9 x_3 + p_7 x_4)}{(1 + x_3)(1 + p_3 x_1 + p_5 x_2 + p_4 x_3 + p_2 x_4)} + \frac{p_{11} x_4}{p_{12} x_2}
\]  

where \( p_1, p_2, p_3 \ldots p_{12} \) are fitting parameters, which are equal to \(-0.0081, 1.3957, -0.4017, -0.0861, 2.6440, 79.7945, 1161.3893, 349.2907, 94.6238, -4221.9091, -5.3805, \) and \(-5.1671 \), respectively.

\( x_1 \) — Roadway slab length to width ratio \( \frac{L_a}{L_b} \);
\( x_2 \) — Fatigue load level;
\( x_3 \) — Roadway slab width to height ratio \( \frac{L_b}{h} \);
\( x_4 = \frac{N_t}{N_g} \);
\( N_t \) — Fatigue load cycles;
\( N_g \) — Target number of fatigue load cycles, the default is 2 million.

The correlation coefficient \( R = 0.97 \) for Equation (19) is calculated to be greater than 0.95, indicating a good fit. Additionally, the residual sum of squares \( \text{SSE} = 0.41 \) is within the range of \([-2, 2]\), proving that the stability of the parameters of the fitted equation is effective. Therefore, the mathematical model between the residual load capacity of the carriageway slab and the length-to-width ratio, width-to-height ratio, fatigue load level, and fatigue loading times is reasonable.

6. Conclusions

The fatigue damage morphology and fatigue damage mechanism of reinforced concrete ribbed girder bridge lane slabs were obtained through fatigue testing of lane slabs in this work. A fatigue analysis model of the carriageway slab was established based on the fatigue cumulative damage theory. The influence of the fatigue load level, carriageway slab length to width ratio, and width to height ratio on the residual load capacity of the carriageway slab was studied, and the residual load-bearing capacity evaluation model of the carriageway slab was obtained by polynomial fitting. The main conclusions were obtained as follows.

(1) Fatigue damage to the carriageway slab took the form of punching damage. The fatigue damage development pattern could be divided into three stages, accounting for 6.25%, 56.25%, and 37.5% of the total fatigue life. The first stage was the initial fatigue phase, where the number of fatigue load cycles was between 0 and 100,000. At this stage, the reinforcement strain and the deflection at the bottom of the slab increased linearly, and no significant macroscopic cracks were observed. When the number of fatigue cycles reached 100,000 times, the damage entered the second stage, i.e., the stage of stable development of fatigue damage. At this point, a longitudinal crack with a width of 0.2 mm appeared at the bottom of the plate, and a family of scattered fissures centered on this fissure emerged. After that, the reinforcement strain and the bottom deflection of the slab increased linearly and steadily, but the increase rate was low. When the number of fatigue loading exceeded 1 million times, the carriageway slab entered the accelerated fatigue development and damage stage. The concrete strain, the deflection of the bottom of the slab, and the concrete crack width all accelerated, and the reinforcement yielded and entered the plastic stage. Finally, the driveway slab underwent punching shear damage at the fatigue number.
of 1.6 million times, at which time the maximum crack width of the bottom of the slab reached 1.2 mm.

(2) Under the condition that the length to width ratio and width to height ratio of the carriageway slab was certain, the greater the fatigue load level was, the lower the residual load capacity of the carriageway slab. When the fatigue load level was 0.55, the residual bearing capacity of the carriageway slab did not show a significant reduction trend with the increase of fatigue loading times. After 2 million times of cyclic loading, its residual bearing capacity was 85.29 KN, which was still 74% of the plate bearing capacity. This result indicates that carriageway slabs with fatigue load levels of 0.55 and below are essentially unfatigued after 2 million loadings. When the fatigue load level was 0.65, the curve of residual bearing capacity-fatigue cyclic loading times had an obvious turning point at 1.5 million times. After 2 million times of fatigue loading, the residual bearing capacity was only 43% of the plate bearing capacity. When the fatigue load level was 0.75, the curve turned obviously after 1 million times; when the load was 1.5 million times, the residual bearing capacity was only 28% of the plate bearing capacity. The fatigue load level rose from 0.65 to 0.75, and the turning point of the fatigue curve was reduced from 1.5 million cycles to 1 million cycles, 500,000 cycles earlier.

(3) Under the condition that the width to height ratio of the roadway slab was constant and the same fatigue load level, the smaller the length to width ratio of the roadway slab, the greater the residual bearing capacity. Thus, the residual bearing capacity of the two-way slab is higher than that of the one-way slab. When the fatigue level was 0.65, and the number of cyclic loadings reached 1 million, the relative residual bearing capacity of the roadway slab with a length to width ratio of 7.5 was 0.12, and the residual bearing capacity of the roadway slab with a length to width ratio of 1.6 accounted for the pre-damage bearing capacity 0.34, which was 2.8 times the length to width ratio of 7.5. Therefore, from the perspective of the residual bearing capacity of the roadway slab, it is recommended to use a two-way slab with a length to width ratio of less than 2 when carrying out the anti-fatigue design of roadway slabs.

(4) Under the condition that the length to width ratio of the carriageway slab was constant and the fatigue load level was the same, the smaller the width to height ratio of the carriageway slab, the greater its residual bearing capacity. When the fatigue level was 0.65, and the number of cyclic loads was 1.5 million times, the relative residual bearing capacity of the roadway slab with a width to height ratio of 12.8 was 0.16, while the relative residual bearing capacity of the roadway slab with a width to height ratio of 10.3 was 0.38, which is twice the difference.

(5) Using the length to width ratio, width to height ratio, fatigue load level, and number of fatigue load cycles of the carriageway slab as independent variables, and the relative residual load capacities of the carriageway slab as dependent variables, the residual load capacity evaluation model of the carriageway slab was obtained using 1stOpt fitting software.

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References

1. Deng, P.; Matsumoto, T. Determination of dominant degradation mechanisms of RC bridge deck slabs under cyclic moving loads. *Int. J. Fatigue* **2018**, *112*, 328–340. [CrossRef]

2. Ferdous, W.; Manalo, A.; Salih, C.; Yu, P.; Abousnina, R.; Heyer, T.; Schubel, P. Behaviour of polymer filled composites for novel polymer railway sleepers. *Polymers* **2021**, *13*, 1324. [CrossRef]

3. Ferdous, W.; Manalo, A.; Peauril, J.; Salih, C.; Reddy, K.R.; Yu, P.; Schubel, P.; Heyer, T. Testing and modelling the fatigue behaviour of GFRP composites—Effect of stress level, stress concentration and frequency. *Eng. Sci. Technol. Int. J.* **2020**, *23*, 1223–1232. [CrossRef]

4. Hümme, J.; von der Haar, C.; Lohaus, L.; Marx, S. Fatigue behaviour of a normal-strength concrete—number of cycles to failure and strain development. *Struct. Concrr. 2016*, *17*, 637–645. [CrossRef]

5. von der Haar, C.; Hümme, J.; Marx, S.; Lohaus, L. Untersuchungen zum Ermüdungsverhalten eines höherfesten Normalbetons. *Beton-Und Stahlbetonbau* **2015**, *110*, 699–709. [CrossRef]

6. Heek, P.; Mark, P. Ermüdung von Beton bei mehrachsiger und mehrstufiger Beanspruchung. *Beton-Und Stahlbetonbau* **2019**, *114*, 537–547. [CrossRef]

7. Lanwer, J.P.; Höper, S.; Gietz, L.; Kowalsky, U.; Empelmann, M.; Dinkler, D. Fundamental Investigations of Bond Behaviour of High-Strength Micro Steel Fibres in Ultra-High Performance Concrete under Cyclic Tensile Loading. *Materials* **2021**, *15*, 120. [CrossRef]

8. Gebühr, G.; Pise, M.; Anders, S.; Brands, D.; Schröder, J. Damage Evolution of Steel Fibre-Reinforced High-Performance Concrete in Low-Cycle Flexural Fatigue: Numerical Modeling and Experimental Validation. *Materials* **2022**, *15*, 1179. [CrossRef]

9. Cui, C.; Song, L.; Liu, J.; Yu, Z. Corrosion-Fatigue Life Prediction Modeling for RC Structures under Coupled Carbonation and Repeated Loading. *Mathematics* **2021**, *9*, 3296. [CrossRef]

10. Zhuang, C.; Zhang, J.; Jiang, R. Fatigue flexural performance of short-span reinforced concrete t-beams considering overloading effect. *Balt. J. Road Bridge Eng. (RTU Publ. House)* **2020**, *15*, 89–110. [CrossRef]

11. Deng, P.; Matsumoto, T. Fracture Mechanics–Based Fatigue Life Prediction Method for RC Slabs in Punching Shear Failure Mode. *J. Struct. Eng. 2020*, *146*, 04019186. [CrossRef]

12. Zeng, Z.; Hu, J.; Huang, X.; Wang, W.; Huang, Z.; Shuaibu, A.A.; Yuan, Y.; Xie, Z.; He, X. Research on Fatigue Damage Evolution of the Base Plate Structure of China Railway Track System III Type Slab Ballastless Track under Heavy Haul Train Load. *Appl. Sci. 2022*, *12*, 1694. [CrossRef]

13. Fujiyama, C.; Maekawa, K. A computational simulation for the damage mechanism of steel-concrete composite slabs under high cycle fatigue loads. *J. Adv. Concrr. Technol. 2011*, *9*, 193–204. [CrossRef]

14. Yu, T.; Zhang, L.; Liu, Z. Study on the effect of length–short side ratio on the fatigue performance of the rib girder bridge’s carriageway slab. *Int. J. Struct. Integr. 2020*, *12*, 241–260. [CrossRef]

15. Li, H.S. Research on Fatigue Performance of Roadway Slabs and Diaphragms of Reinforced Concrete T-Beam Bridges; Northeast Forestry University: Harbin, China, 2019.

16. Wu, Z.W. Research on Fatigue Life Analysis Method of RC Bridge Deck under Random Traffic Loads; Harbin Institute of Technology: Harbin, China, 2016.

17. Fang, J.; Ishida, T.; Fatallah, E.; Tsuchiya, S. Full-scale fatigue simulation of the deterioration mechanism of reinforced concrete road bridge slabs under dry and wet conditions. *Eng. Struct. 2021*, *245*, 112988. [CrossRef]

18. Wang, Y.H. *Study on Mechanical Properties of Concrete under Axial Tension-Compression Fatigue Loading*; Dalian University of Technology: Dalian, China, 2010.

19. Liu, Z.Z.; Yu, T.L.; Zhang, L.L. Study on the effect of load level and carriageway slab thickness on fatigue performance of concrete T-girder bridges. *Int. J. Struct. Integr. 2020*, *12*, 284–305. [CrossRef]

20. Liu, Z.M. Model test research of bridge structure. *Bridge Constr. Bridge Constr.* **1999**, *4*, 1–12.

21. CCCC Second Highway Planning and Design Institute Co., Ltd. *General Diagram of Bridge and Culvert*; CCCC Second Highway Survey and Design Institute: Beijing, China, 2015.

22. GB/T 228.1-2010; Metallic Materials—Tensile Testing at Ambient Temperature. Standards Press of China: Beijing, China, 2010.

23. GB/T50081-2016; Standard for Test Method of Mechanical Properties on Ordinary Concrete. China Architecture & Building Press: Beijing, China, 2015.

24. JTG3362-2018; Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts. People’s Communications Press: Beijing, China, 2018. (In Chinese)

25. Cheng, J. *Study on the Fatigue Behavior of Reinforce Concrete Continuous Beams Prestressed with External CFRP Tendons*; Southeast University: Nanjing, China, 2017.

26. Sonoda, K.; Horikawa, T. Fatigue Strength of Reinforced Concrete Slabs under Moving Loads. In *Proceedings of the IABSE Colloquium, Fatigue of Steel and Concrete Structures*, Lausanne, Switzerland, 24–26 March 1982; pp. 445–462.

27. Suthiwarapirak, P.; Matsumoto, T. Fatigue analysis of RC slabs and repaired RC slabs based on crack bridging degradation concept. *J. Struct. Eng. 2006*, *132*, 939–948. [CrossRef]

28. Oh, B.H.; Kim, S.H. Realistic models for local bond stress-slip of reinforced concrete under repeated loading. *J. Struct. Eng. 2007*, *133*, 216–224. [CrossRef]
29. Song, Y.P.; Zhao, S.B.; Wang, R.M. Nonlinearity of Fatigue Behavior of Reinforced Concrete Finite Element Method Analysis. *Chin. J. Oceanogr. (Chin. Version)* 1993, 3, 116–125.

30. Zeng, Z.B.; Li, Z.R. Fatigue S-N Curve of Reinforcing Bars for Ordinary Concrete Beams. *China Civ. Eng. J.* 1999, 32, 10–14.

31. Zhu, X.C. *Numerical Simulation of Fatigue Failure Process for In-Service Reinforced Concrete Bridge*; Tianjin University: Tianjin, China, 2010.

32. GB 50010-2010; Code for Design of Concrete Structures. China Construction Industry Press: Beijing, China, 2010.

33. Task Force on Concrete Fatigue. *Research on the Method of Checking Fatigue Reliability of Concrete Flexural Members*; China Construction Industry Press: Beijing, China, 1994.

34. Holmen, J.O. Fatigue of concrete by constant and variable amplitude loading. *Spec. Publ.* 1982, 75, 71–110.

35. Meng, X.H. *Experimental and Theoretical Research on Residual Strength of Concrete under Fatigue Loading*; Dalian University of Technology: Dalian, China, 2006.