Effect of Temperature on the Fatigue Life Assessment of Suspension Bridge Steel Deck Welds under Dynamic Vehicle Loading

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The present study proposes a novel fatigue life prediction considering the temperature load, which may be neglected in the traditional assessment of suspension bridge steel deck welds under dynamic vehicle load. Vehicle fatigue, pavement temperature, and temperature gradient models are developed based on the test data from the weight-in-motion system, U-rib welds, pavement temperature, and environment temperature. The U-rib-to-deck and U-rib-to-U-rib welds fatigue stresses are obtained considering both vehicle and temperature loads with transient analysis method in ANSYS package. Then, the temperature gradient fatigue stress spectra are calculated. After that, the fatigue life of two weld types is predicted considering the coupled vehicle-temperature loads. The results indicate that the fatigue stress varies linearly with the temperature of the asphalt concrete. The effect of the temperature on the weld’s fatigue life decreases as the distance increases between the welds and the pavement. The dynamic vehicle load results in a higher fatigue stress than the temperature gradient, indicating that the vehicle load contributes mainly to the bridge’s fatigue damage. Finally, it is calculated that the fatigue damage of two weld types is magnified 5.06 and 1.50 times when the temperature effect is considered after 100-year service of Nanxi Yangtze River Bridge.

1. Introduction

In recent decades, orthotropic steel bridge decks (OSDs) have been widely adopted as the preferred configuration of steel bridges due to their light weight, high material utilization, wide application range, large load capacity, and convenient construction [1]. However, fatigue cracking often initiates in welded joints under the coupling effects of dynamic vehicle load, temperature action, and erosion environment [2–4], in which dynamic vehicle load is representative [5–8]. Some researchers have studied the fatigue durability of OSD by considering the stresses induced by the vehicle loads. Maljaars [9] compared the current fatigue load models and measured vehicle load of today’s European traffic. Pipinato et al. [10, 11] proposed a fatigue evaluation method for existing steel bridges, which is relatively simple and can be easily adopted by end users. Guo et al. [12] collected the OSD weld strain data of the Runyang bridge and found that the joint weld strain in the summer is significantly greater than that in the winter. However, the vehicle load varies slightly between the winter and summer. This indicates that the seasonal temperature variations affect the stress of the steel deck joints. In [13], it was found that Young’s modulus of the asphalt concrete pavement shows an insignificant effect on the fatigue stress of welded joints in the ambient temperature field. Wang et al. [14] discussed the fact that the asphalt concrete pavements may transfer axle loads at different temperatures. A linear regression equation was obtained between the pavement temperature and the fatigue stress of typical welds. Liu et al. [15] proposed a functional expression of fatigue reliability of typical OSD welds under vehicle and temperature loads. The influence of ambient temperature on weld durability of steel bridge decks was also discussed in a probabilistic manner. One may believe that the influence of temperature on the fatigue damage of steel bridge decks is induced by weakening the
load-transfer capacity of the pavement. However, the secondary internal forces induced by temperature are neglected, which may also contribute to the fatigue damage. For example, the transverse and vertical temperature gradients of flat steel box girders under sunshine radiation are significant [16, 17]. Therefore, the effects of temperature must be considered in fatigue life prediction of OSD welds [18, 19].

The present study develops a standard fatigue model induced by a dynamic vehicle load. Simultaneously, a pavement temperature probability model is also accomplished based on the test data of the weight-in-motion (WIM) and temperature monitoring systems of the Nanxi Yangtze River Bridge. An OSD temperature gradient history curve is simulated. Then, the fatigue stress of weld points is analyzed when the temperature gradient of steel box girders is considered. Finally, the effects of pavement temperature and structure temperature on weld fatigue of steel bridges are discussed.

As shown in Figure 1, the Nanxi Yangtze River suspension bridge is on Yibin-Luzhou Expressway in Sichuan Province, China. The WIM system was installed in the direction of Luzhou, and the weighting platforms were deployed under the vehicle lanes, respectively. One may use resistive sensors on the weighting platform to collect vehicle parameters such as type, gross weight, axle weight, speed, and the time interval to the next vehicle. Figure 2 shows the strain sensors, environment, and pavement temperature sensors with a sampling frequency of 5 Hz in the mid-span of the bridge. Vibrating string strain sensors with a sampling frequency of 50 Hz are installed on the U-rib-roof and U-rib-U-rib welding points corresponding to the loading positions of the two wheels in the main girder driveway.

2. Vehicle Load Modeling

2.1. Vehicle Load Probability Model. The vehicle loads contain multiple types due to different vehicle load rates. The vehicle load probability model may be described using a Gaussian mixture distribution [20], yielding

\[ G_{ci}(x|\theta) = \sum_{i=1}^{M} w_i N(x|\mu_i, \sigma_i^2) = \sum_{i=1}^{M} w_i \frac{1}{\sqrt{2\pi}\sigma_i} \exp \left[ -\frac{(x - \mu_i)^2}{2\sigma_i^2} \right] \]  

(1)

where \( \mu_i \) is the mean value of the vehicle weight; \( \sigma_i^2 \) is variance; \( w_i \) is the weight; and \( \theta \) is a vector and defined as a matrix parameter, respectively.

The vehicles may be summarized as six typical models: C2, C3(1), C3(2), C4, C5, and C6 [3]. The parameters of the multipeak probability distribution model are shown in Table 1 [3].

2.2. Vehicle Load Modeling. The equivalent vehicle load may be obtained based on Miner’s equivalent linear damage theorem [21]:
$E = 11031 \times 10^{-0.0169T}$, \hspace{1cm} (4)

where $E$ is Young’s modulus of asphalt concrete, and $T$ is the temperature of asphalt concrete.

3.2. Stress of OSD. The moving vehicles generate time-history stresses of OSD, which are calculated by transient analysis method. The steps are as follows: first, the starting time is the moment when the front wheel of the first vehicle contacts the bridge. The time of arrival at the function node is then determined depending on the vehicle’s speed and distance. Finally, the loading time of the nodes is calculated based on the vehicle speed, wheelbase, and the partition width of finite element mesh. Deleting the previous loading step and cycling the above three steps successively, the traffic flow is simulated across the bridge.

C4 is taken as a representative vehicle to discuss the effect of different asphalt concrete material on weld point stress. The stress-time curves under different temperatures of asphalt material are analyzed. As seen in Figure 5(a), the maximum tensile stress of Weld 1 is 15.3 MPa and 60 MPa.
when the pavement temperature is 0°C and 60°C, respectively, and the ratio (60 MPa/15.3 MPa) reaches 3.92. Similarly, in Figure 5(b), this ratio is 1.48. The difference of the two ratios is so significant because the distances are different between the weld points and the asphalt pavement. The closer the weld to the pavement, the greater the influence of the extended load capacity of the pavement.

In order to validate the transient analysis of the FEM, a comparison is taken between the test data and the numerical results. There are two steps for the process of validation. The first step is to confirm the vehicle model (the truck type, the axle space, the axle load, and the truck’s transversal position) and Young’s modulus of the asphalt pavement based on the WIM data and AC temperature sensor. Then, the numerical stress is obtained. The second step is to compare the test data with the numerical results. The stress of a detail has been analyzed using the FEM. The second is to select the stain-time data as shown in Figure 6 and compare them with the...
4. Fatigue Stress Spectrum

4.1. Temperature Extremum Model of Steel Box Girder.

There are insufficient statistical data available to cover the whole life cycle of OSD bridges. Therefore, data from a measured period are extrapolated to formulate an extreme temperature model for a reference period. Temperature varies daily and seasonally, and the maximum daily temperature is a nonstationary time series. To obtain a stable extremum time series, a generalized extremum distribution model is used to describe the seasonal measured extremum distribution, yielding [27]

\[
F_1(T) = \exp \left\{ -\left[ 1 + \frac{k_1(T - \alpha_1)}{\beta_1} \right] \frac{1}{k_1} \right\} I(T). \tag{5}
\]

The function \( I(T) \) is written as
\[ I(T) = \begin{cases} 1 & \left[ 1 + k_1 (T - \alpha_1) \right] > 0, \\ 0 & \left[ 1 + k_1 (T - \alpha_1) \right] \leq 0, \end{cases} \]

where \( k_1, \alpha_1, \) and \( \beta_1 \) are the shape, position, and proportion parameters of the first measured quarter's extremum model, respectively. \( F_1(T) \) is an extreme value type I distribution when \( k_1 < 0 \), while \( F_1(T) \) is an extreme value type III distribution when \( k_1 > 0 \), respectively. The temperature probability density function (PDF) for design life is expressed as

\[ F_N(T) = F_1^N(T), \]

where \( N \) is the number of years for the service life. Taking the first derivative of (7) to obtain the distribution of the extreme probability density in the fundamental period yields

\[ f_N(T) = NF_1^N(T)f_1(T). \]

The mean and variance expressions are expressed as

\[ \mu_N = \int_{-\infty}^{\infty} x f_N(T)dT = \mu_1 + \frac{\ln N}{\pi} \sqrt{6} \sigma_1, \]

\[ \sigma_N = \int_{-\infty}^{\infty} (x - \mu_N)^2 (T)dT = \sigma_1. \]

Taking the temperature measurement point T1(Figure 2) as an example, the maximum and minimum daily temperatures of four seasons (spring (SP), summer (SM), autumn (AU), and winter (WI)) are extracted, and the PDF is obtained using extreme value extrapolation methods. The maximum likelihood estimation method was used to evaluate the parameters of the extreme value model, as shown in Figure 8. Table 3 shows the parameters of the extreme temperature model.

4.2. Latin Hypercube Sampling. Latin Hypercube Sampling (LHS) can be used to stratify and sample probabilistic models uniformly. With this sampling method, all probability intervals are covered by all sample points [28].
steps for LHS are as follows: firstly, the n-dimensional vector space is divided into m intervals, and each interval has the same probability. Secondly, a point is randomly selected from each interval in each dimension. Finally, take these sample points to get a vector. As shown in Figure 9, the temperature extremum models are sampled 360 to obtain the temperature times series data for one whole year at point T1. The curve of the sample data conforms to the characteristics of seasonal temperature variation, which indicates that the LHS method is effective for temperature extremum sampling.

4.3. Daily Temperature Curve Simulation. In the previous section, the maximum and minimum daily temperatures at point T1 of the steel box girder were extracted using the LHS method. The daily temperature function is formulated based on the daily temperature extrema, which may be described by a sine function [29]:

\[ T = A \sin(\omega t) + B, \]  

where \( t \) is the time; \( \omega, A \) and \( B \) are the shape parameters and expressed as

\[ \omega = \frac{2\pi \times t_c}{24 \times 60}, \]

\[ A = \frac{T_{\text{max}} - T_{\text{min}}}{2}, \quad B = \frac{T_{\text{max}} + T_{\text{min}}}{2}, \]

where \( t_c \) is the temperature measurement period and set to 10 min. Figure 10 shows the simulated data of daily T1 temperature time series for one year.

4.4. Time Variation of Temperature Difference. Ignoring the secondary internal forces on the structure, the expansion and deformation of the steel box girder result in energy being consumed for the overall heating or lowering of the steel box girder of the suspension bridge [30]. In this section, the fatigue stress under a temperature gradient is discussed. The temperature difference are expressed as

\[ T_{i,j} = \sum_{i=1}^{n} (T_i - T_j). \]  

There are no shelters on top of point \( T_2 \), so it is directly exposed to solar radiation. \( T_1 \) and \( T_3 \) are covered by the collision barrier, so that the solar radiation effect reduces. The temperature differences of the roof are divided into two groups: \( T_{2,1} \) and \( T_{2,3} \). The annual temperature time-history curve of each measurement point is then obtained based on the measured statistical data and the simulation method.

Figure 11 shows the temperature differences of \( T_{2,1} \) and \( T_{2,3} \) for one year. The simulated data are segmented using a yellow dividing line. As seen in Figure 11(a), the summer sustains the highest transverse temperature difference of the steel box girder. For \( T_{2,1} \), the maximum values of the lateral positive temperature difference in the four seasons were 12.3°C, 14.0°C, 13.6°C, and 6.5°C, respectively. The negative temperature difference varies little in the four seasons, with the extreme values being −3.5°C, −3.3°C, −2.8°C, and −4.0°C, respectively. Similarly, in Figure 11(b), the extreme lateral positive temperature difference of \( T_{2,3} \) in the four seasons is 10.1°C, 13.5°C, 13.8°C, and 8.2°C, respectively, while the minimum negative temperature difference is −3.5°C.

4.5. Temperature Gradient Stress Simulation. The temperature load is simulated as the adult load at 8 nodes of the shell 63 unit. In order to verify the accuracy of the numerical model, the same temperature gradient in [31] is input for comparative calculations. The temperature gradient is simulated in the finite element model as shown in Figure 12. Under the effect of the transverse temperature difference, the maximum tensile stress of the structure is 28 MPa (cf. 24 MPa in [15]), and the stress distributions are similar (see Figure 12(b)). This verifies the accuracy of the present numerical model.

Based on the statistical data of the temperature difference and the finite element calculation, the rain flow counting method is used to obtain the fatigue stress spectrum of the temperature gradient. As seen in Figure 13,
the average fatigue stress amplitude of the annual temperature difference stress spectrum of the focal point under the transverse temperature difference is 7.8 MPa, while the maximum stress amplitude is 19.5 MPa, respectively. Under the vertical temperature difference of the diaphragm, the average fatigue stress amplitude of the focus is 12.1 MPa, while the maximum is 28.3 MPa, respectively. The vertical temperature difference has greater effects on the diaphragm than the roof. It is worth noting that the temperature difference stresses have two types: the temperature difference effect on the horizontal roof and the vertical diaphragm.

5. Fatigue Damage Analysis

The European Union’s code (Eurocode) provides the fatigue strength reference curve for most OSD welds [32]. According to Eurocode, the U-rib-roof weld is a type 50 weld, and the U-rib butt weld is a type 71 weld, respectively. The detailed fatigue strength curve is expressed as

\[ N \cdot S^3 = K_C \quad (0 < N \leq 5 \times 10^6), \]
\[ N \cdot S^5 = K_D \quad (5 \times 10^6 < N \leq 10^8), \]

where \( S \) is the fatigue stress amplitude, \( N \) is the fatigue life corresponding to \( S \), and \( K_C \) and \( K_D \) are the fatigue strength coefficients, respectively; the expressions for \( K_C \) and \( K_D \) are

\[ K_C = S_C^3 \cdot 2 \times 10^6 \quad (0 < N \leq 5 \times 10^6), \]
\[ K_D = S_D^5 \cdot 5 \times 10^6 \quad (5 \times 10^6 < N \leq 10^8). \]

For U-rib-roof welds, \( K_C \) and \( K_D \) are 7.16 \times 10^{11} \) and 1.90 \times 10^{15}, respectively. For U-rib-U-rib, \( K_C \) and \( K_D \) are 2.50 \times 10^{11} \) and 3.17 \times 10^{15}, respectively.

It is assumed that the structural weld points experience the cyclic stress \( S \) with \( N \) times, where the number of stress cycles is \( n_i \) when \( S > S_D \), and the number of stress cycles is \( n_j \) when \( S \leq S_D \), respectively. According to the Miner linear damage criterion, the structural weld fatigue damage can be calculated as

\[ D = \sum_{S > S_D} \frac{n_i S_i^3}{K_C} + \sum_{S \leq S_D} \frac{n_j S_j^5}{K_D}. \]
Figure 10: Temperature distribution curve of T1.

Figure 11: Measured transverse temperature gradient curve of OSD. (a) T21. (b) T23.

Figure 12: Temperature difference load model and stress effect on steel box girder roof. (a) Temperature gradient load model (Unit: C). (b) Temperature gradient weld stress nephogram.
Assuming that the fatigue damage is equal to (17), the following expression can be obtained using the equivalent damage criterion:

\[
\frac{n S_{eq}^2}{K_D} = \sum_{S > S_D} n S_i^3 + \sum_{S < S_D} n_j S_j^5 \quad (18)
\]

When the constant amplitude stress \(S_{eq}\) is less than \(S_D\), \(S_{eq}\) yields

\[
S_{eq} = \left( \frac{\sum_{S > S_D} n S_i^3 / K_C + \sum_{S < S_D} n_j S_j^5 / K_C}{n/K_D} \right)^{1/5} \quad (19)
\]

When the constant amplitude stress \(S_{eq}\) is greater than \(S_D\), \(S_{eq}\) yields

\[
S_{eq} = \left( \frac{\sum_{S > S_D} n S_i^3 / K_C + \sum_{S < S_D} n_j S_j^5 / K_C}{n/K_C} \right)^{1/5} \quad (20)
\]

\[
n = n_i + n_j. \quad (21)
\]

The equivalent stress amplitude of the pavement temperature and weld is expressed by the linear regression equation [6]:

\[
S_{eq} = c_j T + d_j \quad (22)
\]

where \(c_j\) and \(d_j\) are the coefficients of the linear regression. The parameters of the fatigue stress amplitude under the 6 typical vehicle types are shown in Table 4.

Based on the WIM system monitoring system, asphalt pavement temperature, and main girder temperature monitoring data, the fatigue load data are converted into weld stress data using finite element analysis. Then, the number of fatigue stress range were counted using a simple rainflow counting algorithms [33]. The fatigue effect time series of two types of key nodes are obtained for one year. As seen in Figure 14, the equivalent stress of Weld 1 varies greatly and reaches its maxima in August. The ratio is 1.92 between the average stress amplitude in August and December. This is because Young’s modulus reduces in summer, and it is difficult to transfer and disperse the local loads. On the other hand, the stress of Weld 1 is higher than that of Weld 2 since Weld 1 is closed to the asphalt concrete surface, while Weld 2 is far from the pavement layer.

As seen in Figure 15, the cycles of the equivalent stress amplitude fluctuate slightly with different seasons, and the average \(N_d\) in the summer is a little higher than the other seasons. According to the traffic statistics of the WIM system, the daily traffic volume is relatively stable, and the traffic volume is almost the same in the four seasons, indicating that the seasonal temperature variation is independent of \(N_d\). The temperature of the asphalt surface is the main contribution to \(N_d\). The increase of the pavement temperature increases the fatigue stress on Weld 1, and in a large number of cases, stress amplitudes that are normally below the fatigue limit stress threshold of the Eurocode code cross “the boundary” into the “threshold” region, thus increasing the cycle number of fatigue stress amplitudes. Figure 16 shows a daily fatigue damage comparison of the two weld types. For Weld 1, the maximum daily fatigue damage value differs from the corresponding minimum value by nearly two orders of magnitude. The increase of stress amplitude will exponentially increase the fatigue damage of materials [34], and the difference of the equivalent stress amplitude is the main reason for the great difference of daily fatigue damage.

6. Fatigue Life Prediction

6.1. Influence of Temperature on Fatigue Life. Assuming that the traffic flow of the bridge remains constant throughout the years, the fatigue damage value \((D)\) of the bridge is expressed as

\[
D = \sum_{d=1}^{365} N_d \cdot m S_{eq}^2 \quad (23)
\]
where $m$ is the service time and calculated by year. The weld suffers fatigue failure when $D=1$. Figure 15 shows the fatigue damage development curves of the two welds. The damage progression rate of Weld 1 is significantly higher than that of Weld 2. There is no fatigue failure of the two welds since $D=0.157$ for Weld 1 and $D=0.028$ for Weld 2, respectively, when the bridge serves for 100 years. The predicted fatigue life is 636 and 3521 years for Weld 1 and Weld 2, respectively. The linear accumulation of fatigue damage at the welds is induced by the combinations of vehicle load, asphalt temperature, and the temperature gradient of the steel box girder as shown in Figure 17.

It is now assumed that the asphalt concrete pavement temperature of the steel box girder remains constant ($T=20^\circ\text{C}$), indicating that Young’s modulus of the asphalt concrete pavement is constant. The fatigue damage development curve of the OSD welds is calculated with/without taking into account the effect of temperature gradient. In Figure 17(a), the fatigue damage of Weld 1 with coupled vehicle-temperature loads is 5.06 times higher than that only considering the dynamic vehicle load for 100 years of service, indicating that the fatigue damage of Weld 1 is more sensitive to temperature. In Figure 17(b), the fatigue damage of Weld 2 with coupled vehicle-temperature loads is 1.50 times higher than that only considering the dynamic vehicle load. The vehicle load contributes more than the coupled loads, indicating that the fatigue damage in Weld 2 is less sensitive to temperature.

| Types of vehicle | $c_j$ | $d_j$ | $c_j$ | $d_j$ |
|------------------|-------|-------|-------|-------|
| C2               | 0.72  | 6.10  | 0.09  | 15.00 |
| C3(1)            | 0.79  | 9.30  | 0.12  | 16.60 |
| C3(2)            | 0.82  | 7.451 | 0.09  | 15.70 |
| C4               | 0.85  | 9.14  | 0.14  | 17.50 |
| C5               | 0.80  | 8.67  | 0.15  | 20.12 |
| C6               | 0.88  | 12.50 | 0.20  | 21.48 |

Figure 14: Daily equivalent stress range curve for the two welds.

Figure 15: Daily stress range number curve for the two welds.
Figure 16: Daily fatigue damage curve for the two welds. (a) Daily fatigue damage curve for Weld 1. (b) Daily fatigue damage curve for Weld 2.

Figure 17: Fatigue damage contribution rates of the two welds. (a) Fatigue damage contribution rate of Weld 1. (b) Fatigue damage contribution rate of Weld 2.

Figure 18: Fatigue damage curves for the two welds for different traffic growth rates. (a) Weld 1. (b) Weld 2.
6.2. Influence of Traffic Growth Rate on Fatigue Life.
Assuming a traffic growth rate coefficient \( \alpha \), the detailed fatigue damage expression of the orthotropic plate is modified as

\[
D = \sum_{d=1}^{365} N_d \cdot \frac{S_{eq}}{K_d} \cdot \left( 1 + \frac{(m-1)\alpha}{2} \right)
\]  

(24)

Figure 18 shows the fatigue damage developments of the two welds under different traffic growth rates. Both welds do not suffer from fatigue failure within the designed service life when the traffic growth rate is less than 5%.

7. Conclusion

The present study develops a set of fatigue life assessment methods for OSD weld details considering the coupled action of vehicle-temperature loads based on the monitoring data of the WIM system and the temperature sensors in the health monitoring system of the Nanxi Yangtze River Bridge. The following conclusions are drawn:

1. Under constant vehicle loads, there is a linear relationship between the asphalt pavement temperature and the equivalent stress amplitude. For welds close to the pavement layer (Weld 1), the coupling of vehicle-temperature loads has a significant amplifying influence on the stress effects. For the welds far from the pavement layer (Weld 2), the influence is small.
2. The secondary temperature gradient stress cannot be ignored in fatigue damage accounting. However, due to the temperature gradient limits, the stress amplitude is relatively small and contributes only marginally to the fatigue damage accumulation.
3. The main season contributing to fatigue damage of the main girder weld details bridge is summer. The ambient temperature effect should be taken into account in fatigue design of OSD bridges. The number of fatigue load spectrum cycles of the temperature gradient is obviously smaller than that of vehicle loads, and the contribution of vehicle loads to fatigue damage plays a major role in the coupling action of the two.

Data Availability

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

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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